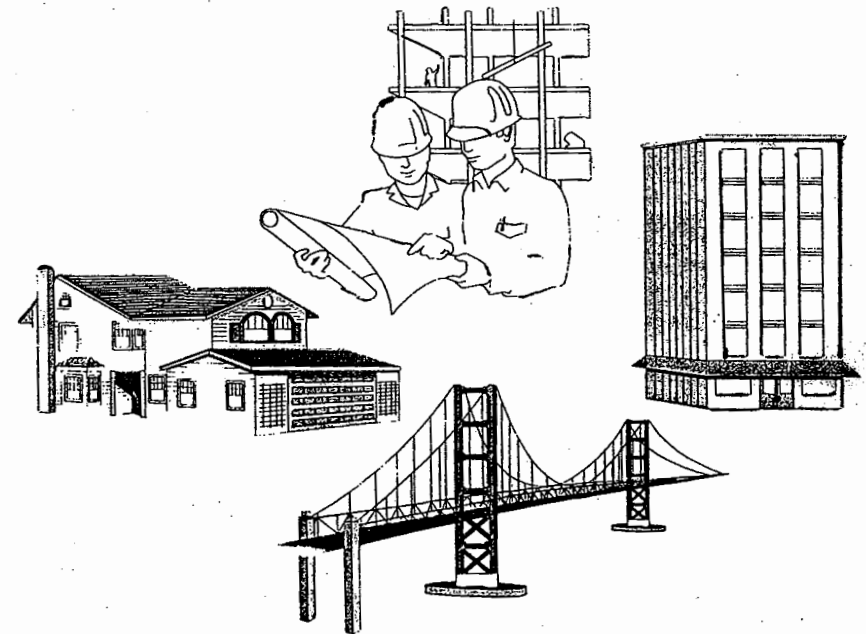


BEHAVIOR, ANALYSIS AND DESIGN OF STEELWORK CONNECTIONS

VOLUME (3)

BY

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Sixth Edition

**Based on the Egyptian Code of Practice
of Steel Constructions and Bridges (2001)
Allowable Stress Design (ASD)**

2008

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حقوق الطبع محفوظة ، لا يجوز نقل أي جزء من الكتاب
بأية وسيلة ما إلا بأذن كتابي من المؤلف شخصياً





PREFACE

This modest work is my third volume which provides the behavior and analysis of the common structural connections. The load path through the steel components is analyzed and then followed by stress checks.

Chapters 1 to 3 provide the required background necessary to design the non-pretensioned and the high strength bolted connections as well as the welded connections. Chapters 4 to 6 give detailed descriptions of the flexible, rigid and semi-rigid connections. Chapters 7, 8 and 9 provide in details the splices, the fatigue requirements and the base connections.

The material of this volume is a national and up-to-date tool for the design of steelwork connections. The material here will serve undergraduate and Postgraduate Students as well as Engineers.

This fourth edition applies the modifications included in the new edition of the Steel Egyptian Code of Practice for Constructions and Bridges (2001) using the Allowable Stress Design (ASD) concept.

I acknowledge this volume to my beloved Egypt.

Prof. Dr. El-sayed Bahaa Machaly

01/01/2008



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- (2) Dr. *Sherif Safar*, who solved the majority of the examples in the previous editions knowing that only some modifications regarding the allowable stresses for different fasteners have
- (3) been applied.

Eng. *Saad Ghaprial Yakoub*, who drew the illustration, the graphics and the tables using **Autocad**, **Artist** and **Excell** programs respectively. Eng. *Saad* Printed also this book using both **Winword** and **Chewriter** programs.

Prof. Dr. El-sayed Bahaa Machaly

01/01/2008



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CHAPTER 1



CHAPTER 1

RIVETED AND NON PRETENSIONED (ORDINARY)

BOLTED CONNECTIONS

1.1 Introduction:

A practical steel structure would be an assemblage of individual structural elements joined together by connections.

By their nature, connections are jumble of local effects. Loads can be applied through welds, pins, bolts, or rivets and may be distributed unevenly to small portions of the joint. Most connections are highly indeterminate with the distribution of stress depending upon the deformation of the fasteners. Local restraints may prevent the deformation necessary for a desirable stress distribution. For these reasons, a purely theoretical approach to connection problem is always difficult and often nearly impossible.

Currently, the major connecting devices in structures are rivets, bolts, and welds. Pins and miscellaneous mechanical connections used to a lesser degree. Rivets, bolts, and welds are used extensively and frequently. The economic advantage of one over the other is so small as to be uncertain. At one time riveting prevailed, but it has been superseded in importance by welding and high strength bolting.

This chapter deals with connections using mechanical fasteners. It presents a discussion of the types, properties, and installations of rivets and bolts followed by an analysis of each type of connections. Design procedures are based on the provisions of the Egyptian Code of Practice for Steel Structures (2001).

1.2 Different Types of Rivets and Method of Installation:

The use of rivets is one of the oldest methods of joining metal elements. Rivets are, now, seldom used as a main fitting in structural steel fabrications. Fig. 1.1.a shows the general layout of a rivet which consists of a rivet head and a shank. The rivet material is usually of the same strength as the parent structure.

The method of installation is essentially that of heating the rivet to a light cherry-red color, inserting it into a hole and then applying pressure to the preformed head while, at the same time, squeezing the plain end of the rivet to form a rounded head as indicated in Fig.1.1.b During this process the rivet shank completely or nearly fills the hole into which it has been inserted in. Upon cooling, the rivet shrinks, thereby providing a clamping force action. This force varies from rivet to rivet and, therefore, is difficult to count on in design calculations.

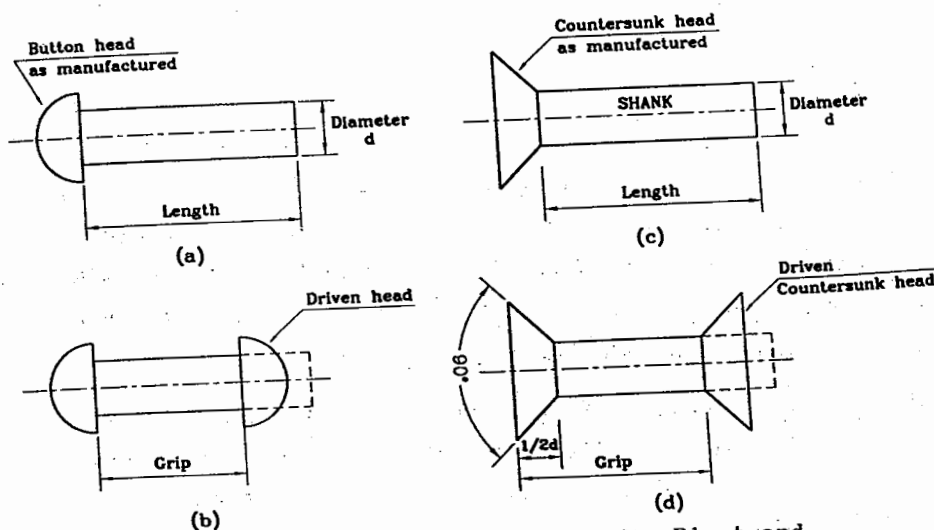


Fig. 1.1 General Layout of a Rivet and Method of Installation.

Figs 1.1.c-d show a countersunk rivet as manufactured with driven countersunk head. This type of rivets is used only for special cases of connections.

Rivet holes are made either by punching a small thickness (material is damaged around the hole) or by drilling (perfect hole). The diameter of the hole is made larger than that of the rivet; $\Delta d < 1.5$ mms, where Δd is the difference between shank's diameter before closing and hole diameter.

1.3 Material properties, types and method of installation of the non-pretensioned bolts (ordinary bolts) :-

1.3.1 Material Properties :

For non-pretensioned bolts, where the forces acting transverse to the shank of the bolt are transmitted either by shear or bearing as will be given in section 1.6, the nominal values of the yield strength F_{YB} and the ultimate tensile strength F_{UB} are given in table 1.1 :-

Table 1.1

Bolt grade	4.6	4.8	5.6	5.8	6.8	8.8	10.9
F_{YB} (t/cm ²)	2.4	3.2	3.0	4.0	4.8	6.4	9.0
F_{UB} (t/cm ²)	4.0	4.0	5.0	5.0	6.0	8.0	10.0

The bolt grades are used in conjunction with structural components in steel up to 52.

Bolts of grades lower than 4.6 or higher than 10.9 shall not be utilized.

Bolts of grades 4.6 up to 6.8 are made from low or mild carbon steel, and are the least expensive type of bolt for light structures.

Grade 8.8 is of heat-treated high strength steel and grade 10.9 is also of heat-treated, but is also of alloy steel.

1.3.2 Different types:

Non-pretensioned (ordinary bolts) can be of a hexagonal nut and head, square nut and head, or of a countersunk head (see Figs. 1.2.a, b, c).

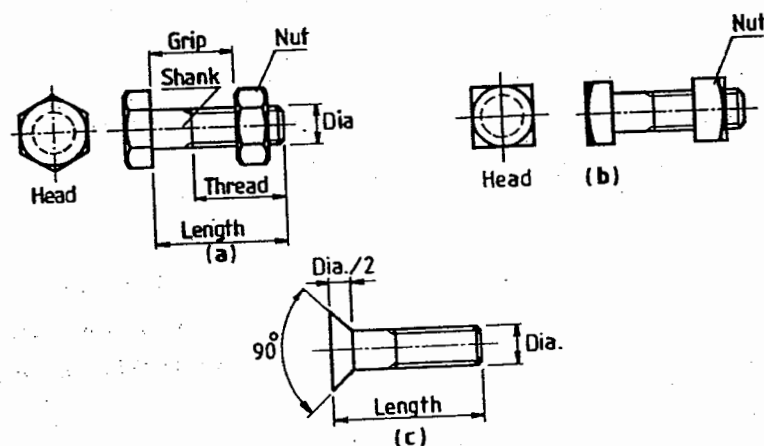


Fig. 1.2 Types of Ordinary Bolts.

The two main types are as follows :-

(a) Fitted Bolts or Turned Bolts:

These are bolts whose shank fills the finished hole as much as possible. The hole clearance is less than 0.3 mms. ($\Delta d < 0.3$ mms). This type of bolts was primarily used in connections which required close fitting in drilled holes, such as riveted construction where it was not possible to drive satisfactory rivets. They are now rarely if never used in ordinary structural connections, since high strength bolts are better and cheaper. They can be used to interact with rivet.

(b) Unfinished Bolts (Bolts Without Fit):

These bolts must not be used to interact with rivets and fitted bolts in the same connection. Hole clearance ranges from 1 mm. to 3 mms.

These bolts are also called common, machine or rough bolts and may have either hexagonal or square heads and nuts.

1.3.3. Holes, clearances, washers and nuts requirements :-

1.3.3.1 Holes :

1 - Holes for bolts may be drilled or punched unless specified.

2 - Where drilled holes are required, they may be sub-punched and reamed.

3 - Slotted holes shall either be punched in one operation, or else formed by punching or drilling. Two round holes are completed by high quality flame cutting and dressing to ensure that the bolt can freely travel.

1.3.3.2 Clearances in holes for fasteners :

1 - Except for fitted bolts or where low clearance or oversize holes specified, the nominal clearance in standard holes shall be as follows :-

1mm for M12 and M14 bolts.

2mm for M16 up to M24 bolts.

3mm for M27 and larger.

2 - Holes with 2mm nominal clearance may also be specified for M12 and M14 bolts provided that the design meets the requirements specified in sections 1.6.1.1 and 1.6.1.2.

3 - Unless special clearance are specified the clearance of fitted bolts shall not exceed 0.3 mms.

1.3.3.3 Nuts constructional precautions :

- 1 - For structures subject to vibration, precautions shall be taken to avoid any loosening of nuts.
- 2 - If non-pretensioned bolts are used in structures subject to vibrations, the nuts should be secured by locking devices or other mechanical means.
- 3 - The nuts of pretensioned bolts may be assumed to be sufficiently secured by the normal tightening procedure.

1.3.3.4 Washers utilities :

- 1 - Washers are not required for non-pretensioned bolts except as follows :-
 - A taper washer shall be used where the surface is inclined at more than 3° to a plane perpendicular to the bolts axis.
 - Washers shall be used where this is necessary due to a requirement to use longer bolt in order to keep the bolt threads out of a shear plane or out of a fitted hole.
- 2 - Hardened washers shall be used for pretensioned bolts
 - A hardened washer shall be used under the bolt head as well as under the nut, whichever is to be rotated.

1.3.3.5 Tightening of bolts :

- 1 - Non-pretensioned bolts shall be tightened sufficiently to ensure that sufficient contact is achieved between the connected parts.
- 2 - It is not necessary to tighten non-pretensioned bolts to predetermined value. However as an indication, the tightening required should be :-
 - That which can be achieved by one man using a normal prodger spanner.

- Or up to the point where an impact wrench first starts to impact.

1.3.4. Installation:

As it has been mentioned before in Section 1.3.1 there exists a clearance between the hole and the diameter of the bolt. This clearance allows the installation of the bolt in the hole of the connected plates. The bolt being installed, we turn the nut till complete tightening. To achieve this, two keys are used, one to turn the nut and the other to prevent rotation of the head. The most common types of keys used are shown in Fig. 1.3.

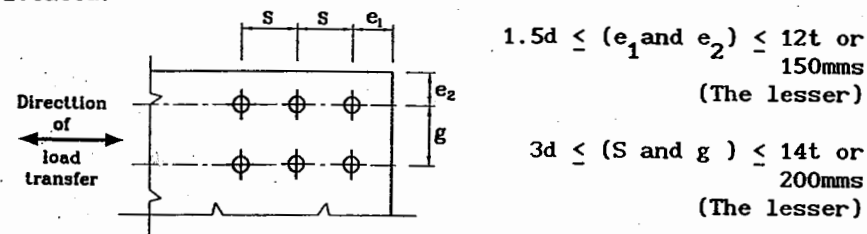


Fig. 1.3 Types of Keys used for Installing Non Pretensioned (Ordinary Bolts).

1.4 Positioning of holes for bolts and rivets :-

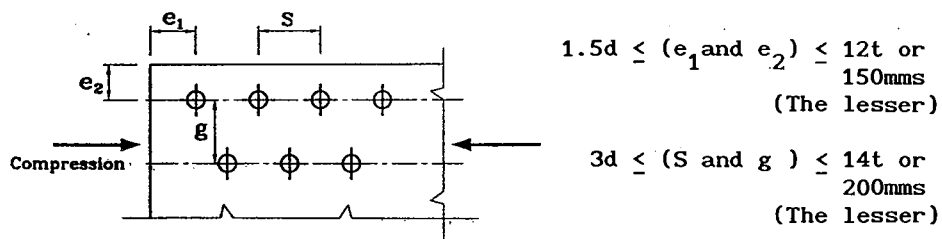
1.4.1 Definitions and Notations:

- Gauge line: It is the line on which rivets or bolts are located.



t = The smallest connected thickness.

Fig. 1.4.a Spacing in Tension or Compression Members.



t = The smallest connected thickness.

Fig. 1.4.b Symmetrically Staggered Spacing in Compression Members.

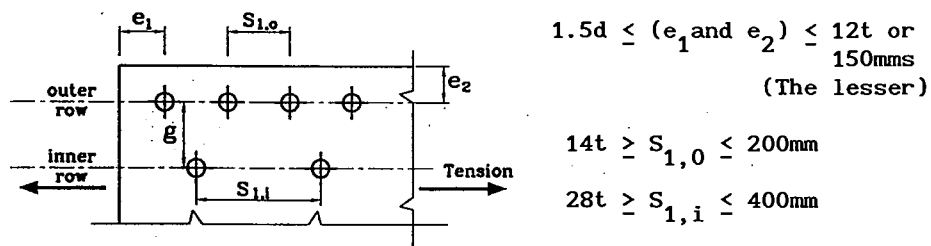
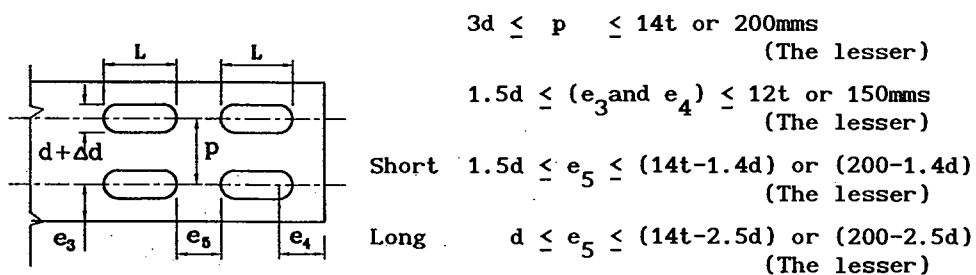


Fig. 1.4.c Staggered Spacing in Tension Members.



	d (mm)	12	16	20	22	24	≥ 27
Short	L (mm)	12+5	16+7	20+7	22+7	24+8	d + 10
Long	L (mm)	30	40	50	55	60	2.5 d

Fig. 1.4.d Slotted Holes.

If one uses one gauge line, as shown in Fig.1.4.a it must be located at equal distances from the inside of the angle.

- Spacing "S": Distance between centers of rivets or bolts in direction of load transfer (Pitch)
- End distance " e_1 ": Distance between the center of any rivet or bolt to the edge in the direction parallel to the axis of the steel structural element.
- Spacing "g": Distance between two rows of rivets or bolts perpendicular to the axis of the direction of load transfer (gauge distance)
- e_2 : Edge distance from the center of a fastener to the adjacent edge of any steel element measured at right angles to the direction of load transfer.
- d : Diameter of rivet or bolt.
- ϕ : Diameter of hole = $d + \Delta d$

1.4.2 Limitations :

1.4.2.1 Basis :

1 - The positioning of the holes for bolts and rivets shall be done such as to prevent corrosion and local buckling, and to facilitate the installation of the bolts and rivets.

2 - The positioning of the holes shall be also in conformity with the limits of validity of the rules used to determine the design bearing strength of the bolts and the rivets as given in section 1.6.1.2

1.4.2.2 Minimum end distance :

1 - The end distance e_1 from the center of a fastener to the adjacent end of any steel element, measured in the direction of load transfer (see figure 1.4.a) should not be less than $1.5d$, where d is the nominal bolt diameter.

2 - The end distance e_1 should be increased if necessary to provide adequate bearing resistance (see section 1.6.1.2)

**1.4.2.3 Minimum edge distance :**

The edge distance e_2 from the center of a fastener to the adjacent end of any steel element, measured at right angles to the direction of load (Fig. 1.4.a) should not be less than $1.5d$.

1.4.2.4 Maximum end and edge distance :

The maximum end or edge distance should not exceed the lesser of 12 times the thickness (t) of the smallest connected part or 150mm.

1.4.2.5 Minimum spacing :

1 - The spacing (S) between centers of fasteners in the direction of load transfer (see figure 1.4.a) should not be less than $3d$.

2 - The spacing (g) between rows of fasteners, measured perpendicular to the direction of load transfer (see figure 1.4.a) should not normally be less than $3d$.

1.4.2.6 Maximum spacing in compression members :

The spacing (S) of the fasteners in each row and the spacing (g) between rows of fasteners should not exceed the lesser of $14t$ or 200mm. Adjacent rows of fasteners may be symmetrically staggered (figure 1.4.b).

1.4.2.7 Maximum spacing in tension members :

In tension members the center to center spacing $S_{1,i}$ of fasteners in inner rows may be twice that given in Section 1.4.2.6 for compression members, provided that the spacing $S_{1,0}$ in the outer row along each edge does not exceed that given in Section 1.4.2.6 (see figure 1.4.c).

1.4.2.8 Slotted holes :

1 - The minimum distance (e_3) from the axis of the slotted hole to the adjacent end or edge of any steel element should not



be less than $1.5d$ (see figure 1.4.d).

2 - The minimum distance (e_4) from the center of the end radius of a slotted hole to the adjacent end or edge of any steel element should not be less than $1.5d$ (see figure 1.4.d).

1.4.3 Considerations Concerning the thickness of the Connected Plates:

(a) Considering " t " as the minimum thickness of the connected plates, the following conditions must be satisfied:

$$\left. \begin{array}{l} d \geq t + 2 \text{ mm} \quad \text{if } t \leq 20 \text{ mm.} \\ d \geq 22 \text{ mm} \quad \text{if } t \geq 20 \text{ mm.} \end{array} \right\} \begin{array}{l} \text{(to avoid the bearing} \\ \text{failure of bolts).} \end{array}$$

(b) The total thickness of the connected plates $\sum_{i=1}^n t_i \leq 4d$ (to avoid bending failure of bolts).

Where n is the total number of the connected plates and t_i is the thickness of the i th plate.

1.4.4 Diameters and Nominal Cross Sectional Areas of Non-Pretensioned (Ordinary Bolts) :-

Table 1.2 gives the stress cross sectional area A_s and the shank's area of non-pretensioned (ordinary bolts) for different standard values of the bolt's diameter.

Table 1.2. Section Properties of Ordinary Bolts

Diameter "d" in mm	12	16	20	22	24	27	30	33	36
Area of shank $A = \frac{\pi d^2}{4} \text{ (cm}^2\text{)}$	1.13	2.01	3.14	3.8	4.52	5.73	7.06	8.56	10.18
Stress cross sectional area $A_s \text{ (cm}^2\text{)}$	0.84	1.54	2.45	3.03	3.53	4.59	5.61	6.94	8.17

1.5 Different Types of Connections:

There are many types of connections to join the different structural steel members. In this chapter, they will be classified according to the manner of stressing the fasteners.

(a) Concentric Shear Connections:

Figs. 1.5.1 and 1.5.2 shows some typical connections of the concentric shear type. The common types are as given below:

- Truss member connection.
- Splice of truss members.
- Splice of girder's flange.
- Splice of beam-column flange.
- Continuity of beams.

In all such types, the fasteners are subjected to a concentric axial shearing force that will induce shear and bearing stresses.

(b) Eccentric-Shear Connections:

Some typical configurations of this type as depicted in Fig. 1.6, are:

- 1- Crane bracket connection.
- 2- Flexible beam-to-column connection.
- 3- Flexible secondary-to-main beam connection.
- 4- Splice of beam's web.
- 5- Splice of beam-column's web.

In all such types of connections, shear and bearing stresses will result from the applied shearing force and torsional moment.

Type	Configuration	Shearing Force "Q"
1-Truss Connection.		$Q = F_i$
		$Q = F_i$
2-Splice of Truss Member.		$Q = \text{Strength of Member}$ $Q = F_t \cdot A_{\text{net}}$
		$Q = \text{Strength of Member}$ $Q = F_t \cdot A_{\text{net}}$

Fig. 1.5.1 Concentric Shear Connections.



Type	Configuration	Shearing Force "Q"
3-Splice of Girder's Flange.		$F = M/d_b$ $Q = F$
4-Splice of Flange of Beam-Column		$M_f = \frac{M \cdot I_f}{I}$ $N_f = \frac{N \cdot A_f}{A}$ $F = M_f/d_b + N_f/2$ $Q = F$
5-Continuity of Beams.		$F = M/d_b$ $Q = F$

Fig. 1.5.2 Concentric Shear Connections "Cont..".



Type	Configuration	Shearing Force	Torsional Moment
1-Bracket Connection.		$Q = P$	$M_t = P \cdot e$
2-Flexible Beam to Column Connection.		$Q = R$	$M_t = R \cdot e$
3-Flexible Secondary to Main Beam Connection.		$Q = R_1$	$M_t = R_1 \cdot e$
4-Splice of Beam's Web.		$Q = Q$	$M_t = M_w + Q \cdot e$ $M_w = \frac{M \cdot I_w}{I}$
5-Splice of Beam Column's Web.		$\bar{Q} = Q$ $ Q = \frac{N \cdot A_w}{A}$	$M_t = M_w + Q \cdot e$ $M_w = \frac{M \cdot I_w}{I}$

Fig. 1.6 Eccentric Shear Connections.

Pure Tension Connections:

Two illustrative examples are shown in Fig. 1.7.

Hanger connection.

Truss-to-column connection.

Such connections are subjected to a pure tensile force using tensile stresses in the connecting bolts.

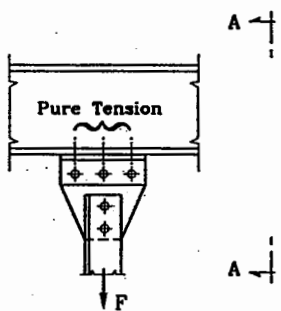
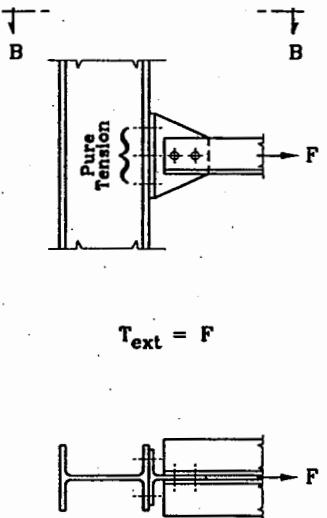
(a) Hanger Connection.	(b) Truss to Column Connection.
 Pure Tension $T_{ext} = F$ Sec. A-A	 Pure Tension $T_{ext} = F$ Sec. B-B

Fig. 1.7 Pure Tension Connections.

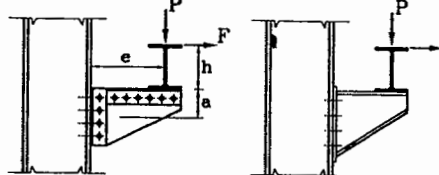
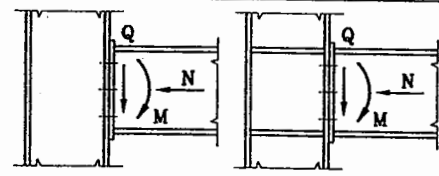
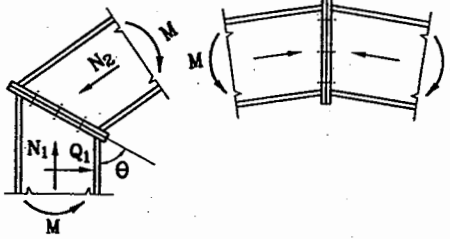
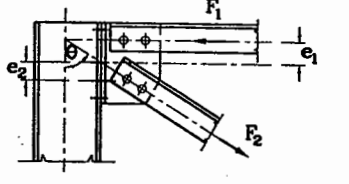
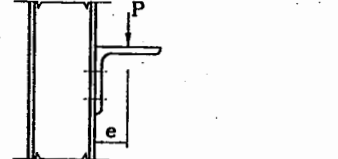
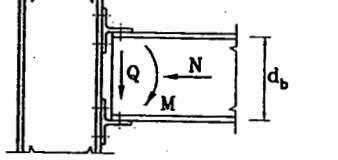
Type	Configuration	Shearing Force	Moment+Ext. Tension
1-Bracket.		$Q=P$	$T_{ext}=F$ $M=P.e+F(h+a)$
2-Beam-Column.		$Q=Q$	$T_{ext}=N$ $M=M$
3-End Plate Corner Connection.		$Q=N_1.\cos\theta$ $-Q_1.\sin\theta$	$T_{ext}=N_1.\sin\theta$ $+Q_1.\cos\theta$ $M=M$
4-Column-Truss Connection.		$Q=F_2.\cos\theta$	$T_{ext}=F_2.\sin\theta$ $-F_1$ $M=F_1.e_1 - F_2.e_2.\sin\theta$
5-Seat Angle Connection.		$Q=P$	$M=P.e$
6-T-Connection		$Q=Q$	$T_{ext}=\frac{M}{d_b} - \frac{N}{2}$

Fig. 1.8 Moment Connections.

Moment Connection:

This type of connection is subjected to a moment inducing side stresses in the fasteners. In addition, the connection be subjected to normal and shearing forces. If all straining ions are present, they will produce shear and bearing stresses in addition to the tensile stresses.

Some typical moment connections are illustrated in Fig. 1.8. ch are as follows:

Bracket crane track-girder connection.

Beam- column rigid connection.

Corner end plate rigid connection.

Column-truss connection.

Seat angle connection.

Beam-to-column T connection.

6. Non-Pretension Bolts Subjected to Concentric Shear:

6.1. Modes of Failure and Nominal Stresses:

There are several possible modes of failure that can occur in riveted and bolted connections. The most common modes of failure will be considered in the following:

6.1.1 Shear Failure:

1 - The shearing failure may be either in the shank of the bolt as indicated in Fig.1.9.a. or in the plate as in Fig.1.9.b. The shearing failure through bolt shank may be avoided by providing a sufficient number of bolts to reduce the nominal shear stress to the value prescribed by E.C.P. (2001) as follow :-

$$R_{\text{shear}} = q_b \cdot A_s \cdot m \quad 1.1$$

Where R_{shear} = resistance of rivet or bolt in shear ..

q_b = bolt allowable shear stress.

A_s = the tensile stress area of bolt.

m = number of planes subjected to shear.

2 - The allowable shear stress q_b for bolt grades 4.6, 5.6 and 8.8 shall be taken as follows :-

$$q_b = 0.25 F_{Ub} \quad \dots\dots\dots 1.2$$

3 - For bolt grades 4.8, 5.8, 6.8 and 10.9 the allowable shear stress q_b is reduced to the following :-

$$q_b = 0.2 F_{Ub} \quad \dots\dots\dots 1.3$$

4 - For bolts where the threads are excluded from the shear plane, the gross crosssectional area of bolt (A) is to be utilised.

5 - The values for design of shear strength given in equations 1.2 and 1.3 are to be applied only where the bolts in holes are of nominal clearances not exceeding those for standard holes as specified in 1.3.3.2 .

6 - M12 and M14 bolts may be used in 2mm clearance holes provided that for bolts of strength grade 4.8, 5.8, 6.8 and 10.9 the design shear stress is to be reduced by 15% .

7 - The shear failure of plate (Fig. 1.9.b)can be avoided by providing sufficient edge distance as given in figure 1.4

1.6.1.2 Bearing Failure:

1 - The bearing failure of the bolt itself may be prevented by providing a sufficient number of bolts to reduce the stresses to the values prescribed by E.C.P. The behavior is shown in Fig. 1.9.c.



$$R_b = F_b \cdot d \cdot t \quad (\text{case of 2 plates})$$

$$R_b = F_b \cdot d \cdot \min \sum t \quad (\text{case of several plates})$$

1.4

here :

 R_b = resistance of rivet or bolt in bearing.

 F_b = allowable bearing stress.

 d = diameter of bolt.

 $\min \sum t$ = smallest sum of plate thicknesses in the same direction of bearing pressure.

2 - For distance (s) center to center of bolts not less than $3d$, and for end distance in the line of force not less than $1.5d$, the allowable bearing stress F_b (t/cm^2) is as follows :-

$$F_b = \alpha \cdot F_U \quad \dots\dots\dots 1.5$$

where :

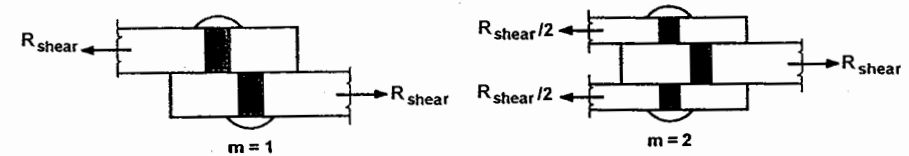
 F_U = the ultimate tensile stress of connected plates.

As the limitation of deformation is the relevant criteria the α -values of equation 1.5 are given in table 1.3

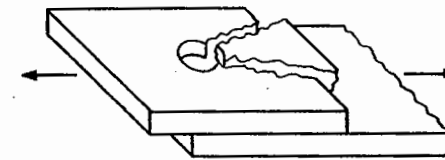
Table 1.3

	End distance in direction of force (e_1)			
	$\leq 3.0d$	$\leq 2.5d$	$\leq 2.0d$	$\leq 1.5d$
α	1.2	1.0	0.80	0.6

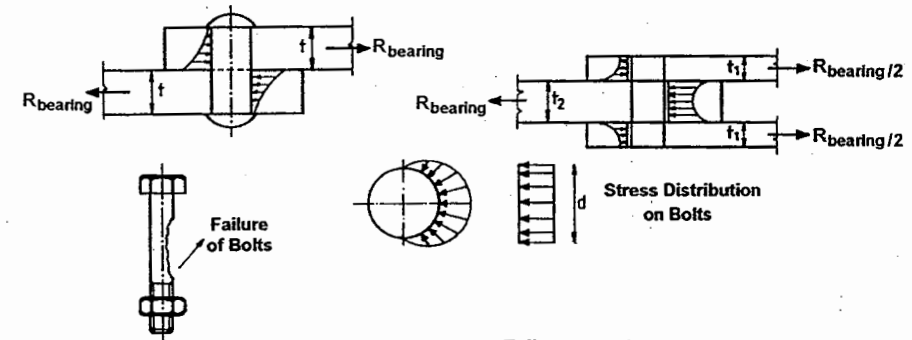
3 - The bearing failure of bolts (Fig. 1.9.c) is prevented either by using sufficient number of bolts or by using thicker plates. The bearing failure of plates (Fig. 1.9.d) is prevented by providing sufficient end distance.



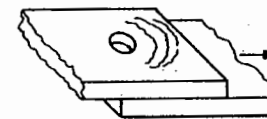
(a) Shear Failure of Bolts



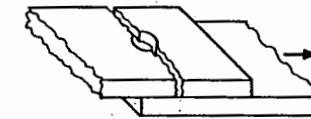
(b) Shear Failure of Plate



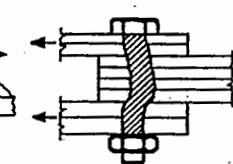
(c) Bearing Failure of Bolts



(d) Bearing Failure of Plate



(e) Tensile Failure of Plate



(f) Bending Failure of Bolts

Fig. 1.9 Shear and Bearing Failure.

6.1.3 Tensile Failure:

The tension failure of plate (Fig.1.9.e.) can be avoided by providing sufficient breadth of plates as given in Section 1.4.1.

6.1.4 Bending Failure of Bolts:

The bending failure of bolts (Fig. 1.9.f) can be avoided by using the limitations previously outlined in Section 1.4.3.

6.2. Real Distribution of Forces:

The analysis of riveted or bolted connections is a complicated structural problem. As long as the connection remains elastic and the load transmitted from one plate to another by rivets (or bolts), the distribution of forces is not uniform. Fig. 1.10, shows a six-row joint where the distribution of forces is not uniform.

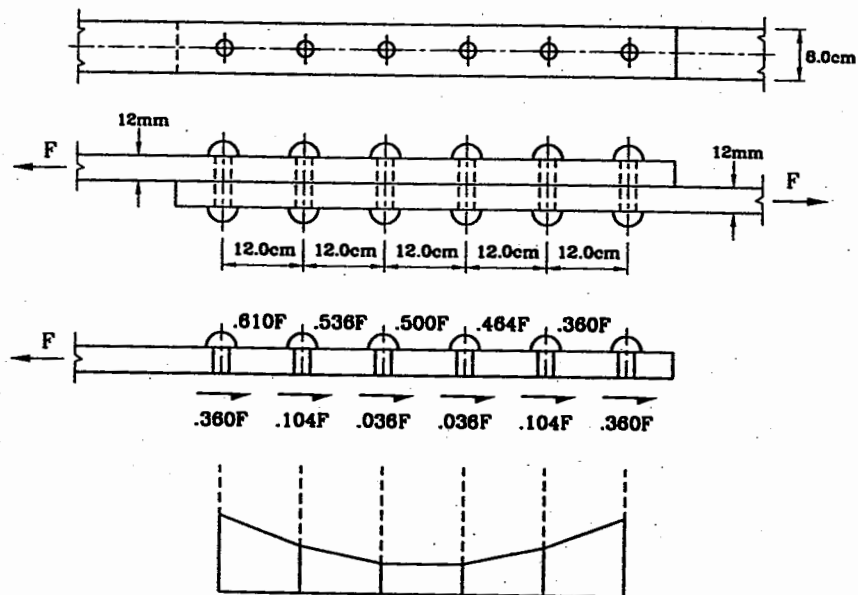


Fig. 1.10 Typical Force Distribution in a Riveted (Bolted) connection.

The exterior rivets carry more than twice their share of the total force, and the interior rivets carry very little. The distribution of forces depends on the stiffness of the connected plates, the initial friction, the alignment of fasteners and holes, the degree of hole filling, and the deformations of the different components. For long joints where the distance between the centers of end fasteners is more than $15d$ the shear and bearing stresses are reduced according to Section 1.12.3.

Refer to the following examples:

Example 1.1: Bolted connection of separate truss members.

Example 1.2: Bolted connection of an unloaded lower chord-truss connection of separate members.

Example 1.3: Unloaded chord continuous connection.

Example 1.4: Bolted connection of a loaded upper chord-truss connection of continuous separate members.

Example 1.5: Separate loaded chord connection.

Example 1.6: Splice of a truss member.

Example 1.7: Connection of a continuous beam.

Example 1.8: Flexible beam-to-beam connection.

1.6.3. Notes:

The allowable shear and bearing stresses previously given in Eqs. 1.2, 1.3 and 1.5 are valid only for case of loading (I). For case (II) values of q_b and F_b are to be increased by 20%.

1.7 Eccentric Shear Connection:

"Shearing force $Q=P$, and Torsional moment $M_t=P.e$ "

The basic assumptions considered in the analysis of eccentric connection, shown in Fig. 1.11.a, are as follows:

- (1) The center of gravity C.G. of the group of bolts is assumed to coincide with the center of rotation of the connection C.R. This assumption is only true if we consider that the



two bracket plates are rigid enough.

- (2) The concentric shearing force $Q=P$ will be assumed to be equally distributed on the whole number of bolts "n" and act in the same direction as the shearing force:

$$R_{Q, \text{ bolt}} = Q/n \downarrow \quad (a)$$

- (3) The resulting force on bolts due to the moment $M_t = P.e$ is assumed to be proportional to the distance r_i and acts perpendicularly to it " $F_i \perp r_i$ "; r_i being the distance from the center of the i th bolt to the C.G. of the group of bolts.

This assumption will lead to the following relation:

$$F_1/r_1 = F_2/r_2 = \dots = F_i/r_i = \dots = F_n/r_n = \text{Constant.}$$

Multiplying by r_i / r_i , we get:

$$F_1 \cdot r_1 / r_1^2 = F_2 \cdot r_2 / r_2^2 = \dots = F_i \cdot r_i / r_i^2 = \dots = F_n \cdot r_n / r_n^2 = \text{constant}$$

$$\text{i.e. } \frac{F_1 r_1 + F_2 r_2 + \dots + F_i r_i + \dots + F_n r_n}{r_1^2 + r_2^2 + \dots + r_i^2 + \dots + r_n^2} = \frac{F_n}{r_n} = \text{Constant}$$

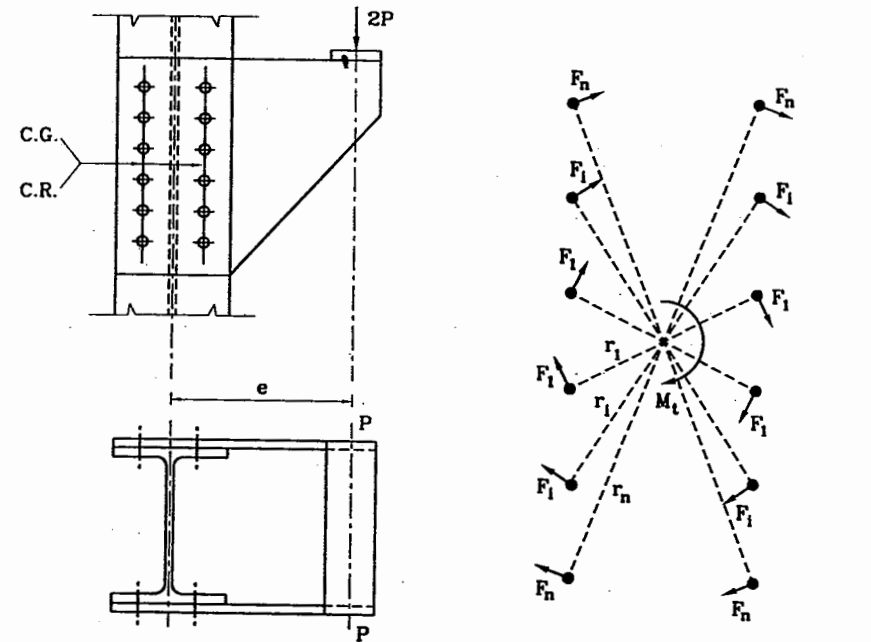
$$\text{Hence: } \frac{\sum_{i=1}^n F_i r_i}{\sum_{i=1}^n r_i^2} = F_n / r_n \quad (b)$$

- (4) From equilibrium of moments, we have:

$$F_1 r_1 + F_2 r_2 + \dots + F_i r_i + \dots + F_n r_n = M_t = P.e \quad (c)$$

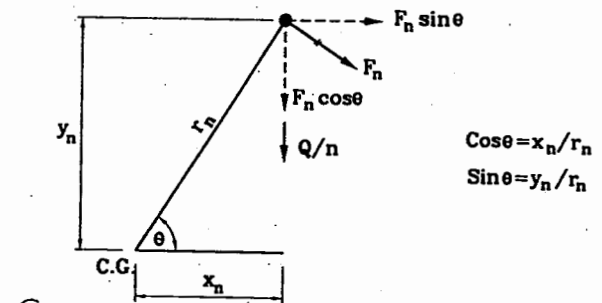
- (5) Substituting from (C) into (b), we get:

$$M_t / \sum_{i=1}^n r_i^2 = F_n / r_n$$



(a) Bracket Plates.

(b) Force Acting Upon the i -th Bolt Due to Torsional Moment.



(c) Force Acting Upon the i -th Bolt Due to Torsional Moment.

Fig. 1.11 Eccentric Shear Connections.

Therefore:

$$F_n = M_t \cdot r_n / \sum_{i=1}^n r_i^2 \quad 1.6$$

It is quite clear that the farthest bolt from the C.G. is the most stressed one as deduced from assumption (3).

- (6) The resultant force can be obtained by combining Equ. 1.6 with equation (a), and it should be checked as indicated in Fig. 1.11.c.

$$R_n = \sqrt{\left[\frac{Q}{n} + \frac{M_t \cdot X_n}{\sum_{i=1}^n r_i^2} \right]^2 + \left[\frac{M_t \cdot Y_n}{\sum_{i=1}^n r_i^2} \right]^2} \quad 1.7$$

R_n = The summation of the induced forces on the most stressed bolt or rivet due to shear and torsional moment respectively.

$$R_n \leq R_{\text{least}}$$

Where R_{least} is the least resistance as prescribed by Equations 1.1 and 1.4

Refer to the following Examples

Example 1. 9: Column-truss connection with framing - angles.

Example 1.11: Bracket connection.

1.8. Bolts of Non-pretension type Subjected to External Tensile Force:

When bolts of the non-pretension type are externally loaded in tension by a tensile force T_{ext} , the tensile strength of a

single bolt (R_t) shall be the allowable tensile bolt stress (F_{tb}) times the bolt stress area (A_s).

$$R_t = F_{t.b} \cdot A_s \quad \dots\dots\dots 1.8$$

$$\text{with } F_{tb} = 0.33 F_{Ub} \quad \dots\dots\dots 1.9$$

1.9. Bolts Subjected to External Tension in Addition to Shear:

If the bolted connection is subjected, in addition to the external tensile force " T_{ext} ", to a shearing force " Q ", the following circular interaction equation is to be satisfied :-

$$\left[\frac{R_{sh.a}}{R_{sh}} \right]^2 + \left[\frac{R_{t.a}}{R_t} \right]^2 \leq 1 \quad \dots\dots 1.10$$

where :

$R_{sh.a}$ = the actual shearing force in the fastener due to the applied shearing force.

$R_{t.a}$ = the actual tension force in the fastener due to the applied external tension force.

R_{sh} and R_t are the allowable shear and tensile strength of the fastener as previously given in equations 1.1 and 1.8 .

1.10. Moment Connection:

The tensile forces of the moment connection of Fig. 1.12.a are transmitted by the bolts while the compressive forces are pressing the end plate toward the column.

This is due to the installation of ordinary bolts which does not need to reach any specified torque, and so separation exists between the connected plates. Hence the initial pretension is small and can be neglected.

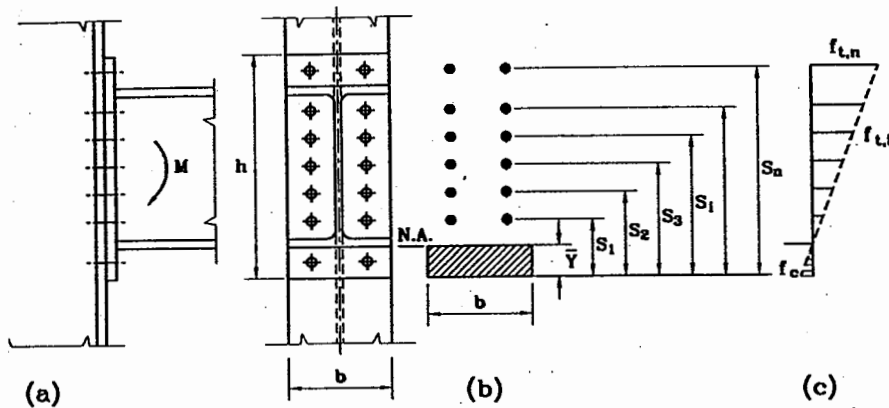


Fig. 1.12 Pure Moment Connections.

When applying a moment "M", the neutral axis will be at a distance $\bar{Y}$ from the lower edge of the compressed end plate (Fig. 1.12.b.). The neutral axis can easily be determined considering that the ordinary bolts above the neutral axis will be subjected to tensile stresses, while the portion of the end plate below the neutral axis will be subjected to a compressive stress.

By taking the first moment of area about the neutral axis we get:

$$\frac{b \bar{y}^2}{2} = m A_s [(S_1 - \bar{Y}) + (S_2 - \bar{Y}) + \dots + (S_i - \bar{Y}) + \dots + (S_n - \bar{Y})] \quad (a)$$

Where m = number of rows.

A_s = Stress cross sectional area of bolts as given by Table 1.2.

Solving equation (a) $\bar{y}$ can be computed. The most stressed bolt (n) will be the farthest bolt (Fig. 1.12.c.). and the

induced stress can be calculated as follows:

$$f_{t.b(M)} = \frac{M \cdot (S_n - \bar{Y})}{I}$$

$$I = \frac{b \bar{Y}^3}{3} + m A_s [(S_1 - \bar{Y})^2 + (S_2 - \bar{Y})^2 + \dots + (S_i - \bar{Y})^2 + \dots + (S_n - \bar{Y})^2] \quad (b)$$

The most stressed bolt must be checked in accordance to clause 6.4.3. E.C.P. 2001, and previously given by Equ 1.9

$$F_{t.b} \leq 0.33 F_{Ub}$$

Notes:

- For the computation of the position of the neutral axis we can assume that $\bar{Y}$ is equal to $h/6$ or $h/7$.
- The bolts lying inside the area in contact ($b \times \bar{y}$) have been neglected in equations (a) and (b).

1.11. Connections Subjected to Bending Moment in addition to Shearing Forces and External Tensile Force:

When the connection is subjected to shear (Q), a tension (T_{ext}) and a bending moment (M), the design strength for the most stressed bolt, must satisfy the following:

$$R_{t.a} = \frac{T_{ext}}{n} + A_s \cdot \frac{M \cdot (S_n - \bar{Y})}{I} \quad \dots \quad (a)$$

$$R_{sh.a} = \frac{Q}{n} \quad \dots \quad (b)$$

where :

$R_{t.a}$, $R_{sh.a}$ = the actual tensile and shearing force in the fastener respectively.

n = the total number of bolts in the connection.

The circular interaction equation 1.10 is to be satisfied:

$$\left[\frac{R_{sh.a}}{R_{sh}} \right]^2 + \left[\frac{R_{t.a}}{R_t} \right]^2 \leq 1$$

Refer to the following examples:

Example 1.9: Bolted column-truss connection using angles.

Example 1.10: Bolted column-truss connection using T-section.

Example 1.11: Bracket connection with braking force.

1.12 General Considerations Concerning the Use of Rivets and Non-Pretensioned bolts (Ordinary Bolts) :-

The following considerations are in accordance to E.C.P. 2001 clauses (6.6), (6.7.1), (6.7.2), (6.7.3), (6.7.4) and (6.7.5), which are as follows :

1.12.1 Allowable Shear Rupture Strength :

At beam end connections, where the top flange is coped and for similar situations where failure might occur by shear along a plane through the fasteners or by a combination of shear along a plane through the fasteners plus tension along any perpendicular plane such as the end of a beam web or as thin bolted gusset plates in double shear (see Fig. 1.13) the allowable shear stress acting on the plate net shear area A_{sh} is to be increased by 15% , i.e :-

$$q = 0.4 F_y \quad \dots\dots\dots 1.11$$

While the allowable tensile strength on the net tension area A_t is to be increased by 25% , i.e :-

$$F_t = 0.725 F_y \quad \dots\dots\dots 1.12$$

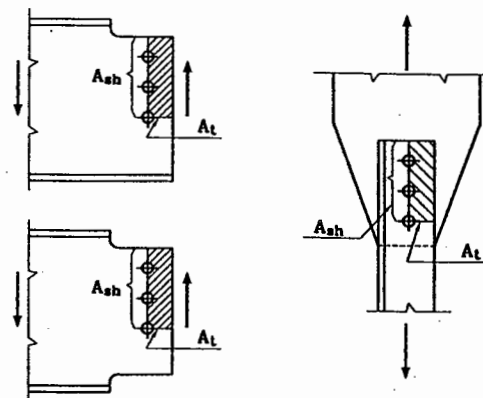


Fig. 1.13 Failure by Tearing out of shaded area.

1.12.2 Long Grip :

1 - Bolts of grade 4.6 and 4.8, which carry calculated stresses fulfilling section 1.6. with a grip exceeding 5 times the bolt diameter (d) shall have their number increased 1% for each 1mm increase in the grip

2 - This provision for other grades of bolts shall be applicable only with grip exceeding $8d$.

1.12.3 Long Joints :

1 - Where the distance L_i between the centers of end fasteners in a joint, measured in the direction of the transfer of force (see Fig. 1.14) is more than $15d$, the allowable shear and bearing stresses q_b and F_b of all the fasteners calculated as specified in Sections 1.6.1.1 and 1.6.1.2 shall be reduced by a reduction factor B_L given by the following :-

$$B = 1 - \left(\frac{L_i - 15d}{200 d} \right) \quad \dots\dots\dots 1.13$$

Where $0.75 \leq B_L \leq 1.0$

2 - This provision is not to be applied, where there is a uniform distribution of force transfer over the length of the joint such as the transfer of the web of I-section to the column flange.

1.12.4 Single Lap Joints with One Bolt :

1 - In single lap joints with only one bolt, (see Fig. 1.15) the bolt shall be provided with washer under both the head and nut to avoid pull-out failure.

2 - The bearing strength is determined in accordance with section 1.6.1.2 shall be limited to :-

$$R_b \leq 0.75 F_U \cdot d \cdot t \quad \dots\dots 1.14$$

3 - In case of high strength bolts of grades 8.8 and 10.9 hardened washers should be used for single lap joints with only one bolt.

1.12.5 Fasteners Through Packings :

1 - where bolts transmitting load in shear and bearing pass through packings of total thickness t greater than one - third of the bolt diameter d , the allowable shear stress calculated as specified in Section 1.6.1.1 shall be reduced using a reduction factor B_b as follows :-

$$B_b = \frac{9d}{8d + 3t_p} \quad \dots\dots\dots 1.15$$

Where $B_b \leq 1.0$

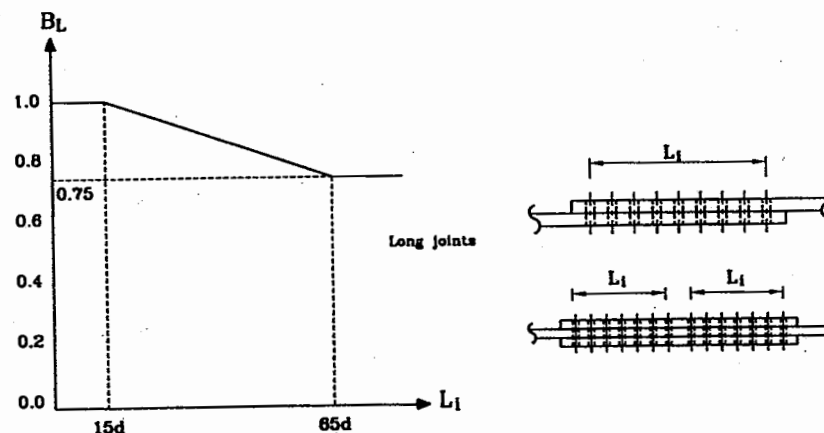


Fig. 1.14 Long Joints.

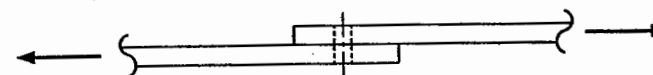


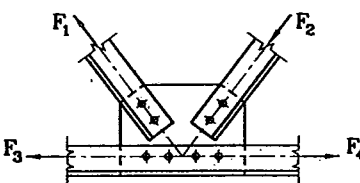
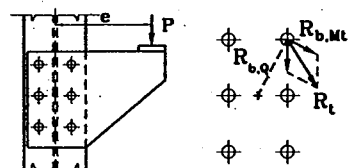
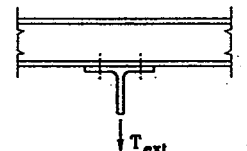
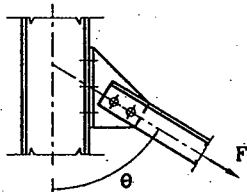
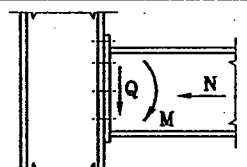
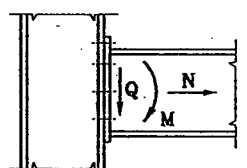
Fig. 1.15 Single Lap Joint with one bolt.

1.12.6 Anchor Bolts and Tie Rods :

1 - The allowable shear and tensile stresses through the threaded portion as prescribed in Sections 1.6.1.1 and 1.8 are restricted to bolts of different grades.

2 - For other threaded parts with cut threads as anchor bolts or threaded tie rods fabricated from round steel bars, where the threads are cut by the steelwork fabricator and not by a specialist bolt manufacturer, the allowable shear and tensile stresses given by equations 1.2, 1.3 and 1.9 are to be decreased by 15%.

Table 1.4 "Non-Pretensioned (Ordinary) Bolted Connections"

Type	Configuration	Straining Actions	Allowable Stresses (E.C.P. 2001)
1-Axial Shear Concentric Connection.		$Q = F$ $R_{sha} = F/n$	$Q_b \leq R_{least}$ R_{least} Smaller of:- $R_{sh} = q_b \cdot (A_o r A_s) \cdot m$ $R_b = F_b \cdot d \cdot \min \text{ of } \Sigma t$
2-Eccentric Connection.		$Q = P$ $M_t = P \cdot e$ $R_{b, Q} = P/n$ $R_{b, Mt} = \frac{M_t \cdot r}{\Sigma r^2}$	$R_t = \text{Vectorial Summation of } R_{b, Mt} \text{ and } R_{b, Q}$ $R_t \leq R_{least}$
3-External Pure Tension.		$T_{ext} = F$ $T_{ext, b} = \frac{T_{ext}}{n}$	$\frac{T_{ext, b}}{A_s} \leq 0.33 F_{Ub}$
4-Combined Shear & Tension.		$Q = F \cdot \cos \theta$ $T_{ext} = F \cdot \sin \theta$ $Q_b = F \cdot \cos \theta / n$ $T_{ext, b} = \frac{F \cdot \sin \theta}{n}$	$\frac{Q_b}{A_s} \leq q_b$ $\frac{T_{ext, b}}{A_s} \leq 0.33 F_{Ub}$ + Equ (1.10)
5-Combined Shear & Bending Moment.		$N = -N$ $Q = Q$ $Q_b = Q/n$ $M = T_{ext, M}$ (Equ 1.11)	$\frac{Q_b}{A_s} \leq q_b$ $\frac{T_{ext, M}}{A_s} \leq 0.33 F_{Ub}$ + Equ (1.10)
6-Combined Shear, Tension Bending Moment.		$T_{ext} = +N$ $Q = Q$ $T_{ext, b} = N/n$ $+ f_{t(b, Q)} \cdot A_s$ (Equ 1.11) $Q_b = Q/n$	$\frac{Q_b}{A_s} \leq q_b$ $\frac{T_{ext, b} + T_{ext, M}}{A_s} \leq 0.33 F_{Ub}$ + Equ (1.10)

1.13 Summary:

Table 1.4 summarize the allowable stresses in accordance with the E.C.P. 2001 for major types of Non-Pretensioned (ordinary bolted) connections.

** **

Example (1.1):

Determine the no. of bolts of the non-pretension type Grade 6 required to connect a compression member (D.F. = - 22.5 ton se (I)) with a gusset plate of St. 37 with 1.0 cm thick. The mber is 2 $\angle$ b.t.b. (90 x 90 x 9).

Using bolts of the non-pretension type : (d = 16mm - Grade 4.6)

$$q_b = 0.25 \times 4 = 1.0 \text{ t/cm}^2 \quad \dots\dots \text{Equ. 1.2}$$

$$R_{d.sh.} = 2 \times 1.0 \times \frac{\Pi (1.6)^2}{4} \quad (\text{Shear planes are excluded from threaded portion})$$

$$= 4.02 \text{ tons.}$$

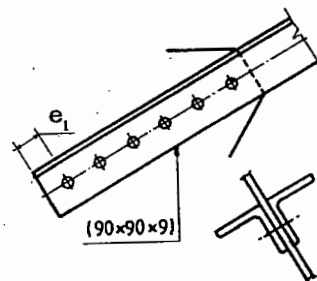
Assuming :- $e_1 = 2d$; $\alpha = 0.80$

$$F_b = 0.80 \times 3.6 = 2.88 \text{ t/cm}^2$$

(Table 1.3)

$$R_{bearing} = 2.88 \times 1.60 \times 1.0$$

$$= 4.608 \text{ tons. (Equ 1.4)}$$



No. of Non-Pretensioned bolts required:

$$\frac{22.50}{4.02} = 5.59 \approx 6.0 \text{ bolts}$$

B. Using High Strength Bolts: (10.9) (d 16 mm)

$$P_s = 3.16 \times 2 = 6.32 \text{ tons.}$$

↑
d.sh

$$\text{No. of H.S.B required: } \frac{22.50}{6.32} = 3.56 = 4.0 \text{ bolts}$$

Example (1.2):

Design the shown bolted bottom chord connection Grade (5.8) of Non-Pretensioned bolts considering the two bottom members Separate.

Gusset Plate thickness is 1.0 cm.

Member	D.F.	Case	X.Section
1	10.20 t	(II)	2 $\angle$ _s 45x45x5
2	-5.90 t	(II)	2 $\angle$ _s 55x55x6
3	16.3 t	(II)	2 $\angle$ _s 60x60x6
4	22.8 t	(II)	2 $\angle$ _s 70x70x7

A. Using Bolts of the non-pretension type: (d = 16 mm)

$$q_b = 0.2 \times 5 = 1.0 \text{ t/cm}^2 \quad (\text{Eqn. 1.3})$$

$$R_{d.sh} = 2 \times 1.0 \times \Pi \frac{(1.6)^2}{4} \times 1.20 = 4.82 \text{ t.}$$

✓ Case loading (II)

$$F_b = 0.80 \times 3.6 = 2.88 \text{ t/cm}^2 \quad (e_1 = 2d)$$

$$R_b = 2.88 \times 1.60 \times 1.0 \times 1.20 = 5.53 \text{ ton.}$$

Member	D.F.	X.Section	$n_{(req.)}$	$n_{(chosen)}$
1	10.2 t	45x45x5	2.11	3.0
2	-5.90 t	55x55x6	1.22	2.0
3	16.3 t	60x60x6	3.38	4.0
4	22.8 t	70x70x7	4.73	5.0

It has to be noted that no need to use Grade (5.8) as the failure is governed by shear. If we use Grade (4.6) we get the same number of bolts.

It is also to be noted that no need to increase the end distance e_1 in order to increase R_b as the failure is governed by shear failure.

Using High Strength Bolts: (10.9) ($d = 12$ mm) ($\mu = 0.4$) Case II

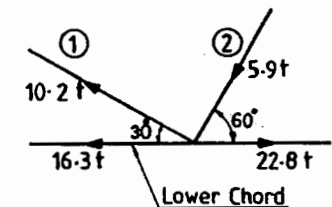
$$P_s = 2 \times 2.01 = 4.03 \text{ tons.}$$

Member	D.F.	X.Section	$n_{(req.)}$	$n_{(chosen)}$
1	10.2 t	45x45x5	2.53	3.0
2	-5.90 t	55x55x6	1.46	2.0
3	16.3 t	60x60x6	4.04	5.0
4	22.8 t	70x70x7	5.65	6.0

Example (1.3):

Design the same connection of Example (1.2) if members (3)& (4) were continuous. The section to be used for the lower chord is 2 < b.t.b. (70x70x7). (Case of loading (II) for all members). Bolts are of Grade (4.6) - Gusset Pl. is of St. 37 and its thickness is 10 mm.

Solution



A. Using Bolts of the non-pretension type: ($d = 12$ mm)

$$q_b = 0.25 \times 4 = 1.0 \text{ t/cm}^2$$

$$R_{d,sh} = 2 \times 1.0 \times \frac{\pi (1.2)^2}{4} \times 1.20$$

$$= 2.71 \text{ tons. (The shear plane is outside the threaded portion)}$$

$$F_b = 0.80 \times 3.6 = 2.88 \text{ t/cm}^2 \quad (e_1 = 2d)$$

$$R_{bearing} = 2.88 \times 1.20 \times 1.0 \times 1.20$$

$$= 4.14 \text{ tons.}$$

Member	D.F.	X.Section	$n_{(req.)}$	$n_{(chosen)}$
1	10.2 t	45x45x5	3.76	4.0
2	-5.90 t	55x55x6	2.17	3.0

In the lower chord: $n_{\text{req.}} = \frac{22.8 - 16.3}{2.71} = 2.39 = 3.0 \text{ bolts.}$

$$\begin{aligned} \text{or } n_{\text{req.}} &= n_1 \cos 30^\circ + n_2 \cos 60^\circ \\ &= 4 \cos 30^\circ + 3.0 \cos 60^\circ \\ &= 4.96 = 5.0 \text{ bolts.} \end{aligned}$$

Take which is greater

$$n_{(\text{chosen})} = \underline{5.0 \text{ bolts.}}$$

B. Using High Strength Bolts: (10.9) ($d = 12 \text{ mm}$) ($\mu = 0.4$) Case II
Members (1) & (2) remains with the same no. of H.S.B as
Ex. (1.2).

$$P_s = 2 \times 2.01 = 4.02 \text{ tons}$$

$$n (\text{Member 1}) = \frac{10.2}{4.02} = 2.53 \quad (\text{use 3 H.S.B.})$$

$$n (\text{Member 2}) = \frac{5.9}{4.02} = 1.46 \quad (\text{use 2 H.S.B.})$$

In the lower chord: $n_{\text{req.}} = \frac{22.8 - 16.3}{4.02} = 1.61 = 2.0 \text{ H.S.B.}$

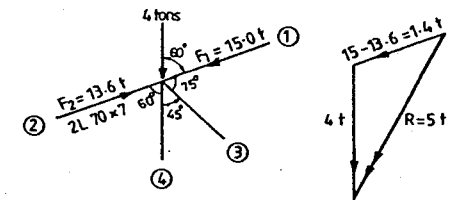
or

$$\begin{aligned} n_{\text{req.}} &= 3.0 \cos 30^\circ + 2 \cos 60^\circ = 3.59 \\ &= 4.0 \text{ H.S.B.} \end{aligned}$$

Example (1.4): (Loaded Connection)

Design the given top chord connection where (1) & (2) are a continuous member.

Thickness of gusset plate
= 1.0 cm.



Triangle of Forces
(to Get the Resultant)

To get the No. of bolts for members (3) & (4), proceed as
Example (2.1).

For the Top Chord: Resultant force $R = 5.0 \text{ tons.}$ (from diagram)

Case (I)

A. Using Bolts of the non-pretension type: ($d = 12 \text{ mm}$) Grade (4.6)

$$R_{\text{d.sh}} = 2.26 \text{ tons.} \quad R_{\text{bearing}} = 3.45 \text{ tons} \quad (\text{from Ex. 1.5})$$

$$\text{no. of bolts required} = \frac{5.0}{2.26} = 2.20 = 3.0 \text{ bolts.}$$

N.B. We must check: $n_{1-2} = n_3 \cos 75^\circ \pm n_4 \cos 60^\circ$

And then take the greater no. of bolts.

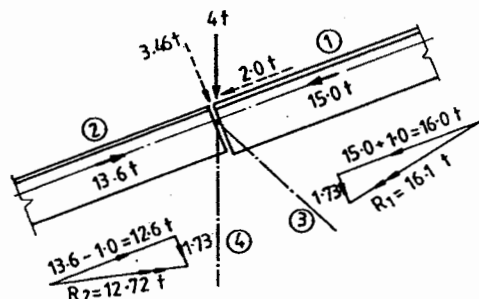
B. Using High Strength Bolts: ($\mu = 0.4$)

$$\text{H.S.B. (10.9) - } d = 12 \text{ mm} - P_s = 2 \times 1.69 = 3.38 \text{ tons.}$$

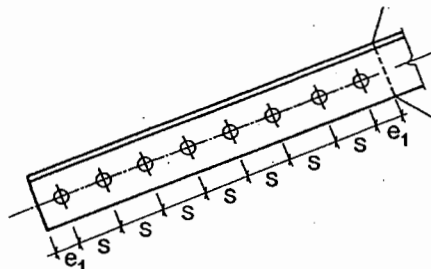
$$\text{no. of H.S.B.} = \frac{5.0}{3.38} = 1.14 = 2.0 \text{ H.S.B.}$$

Example (1.5): (Loaded Connection)

For the connection shown in the figure below find the no. of bolts in members (1) & (2) which are separate. (Case of loading I)



A. Using Bolts of the non-pretension type: ($d = 12 \text{ mm}$) (Grade 4.6)



$$q_b = 0.25 \times 4 = 1.0 \text{ t/cm}^2$$

$$R_{d.sh} = 2 \times 1.0 \times \frac{\pi (1.2)^2}{4} = 2.26 \text{ tons}$$

$$F_b = 0.80 \times 3.6 = 2.88 \text{ t/cm}^2 \quad (e_1 = 2d)$$

$$R_b = 2.88 \times 1.2 \times 1 = 3.45 \text{ tons}$$

$$n_1 = \frac{16.10}{2.26} = 7.10 = 8.0 \text{ bolts.}$$

$$n_2 = \frac{12.72}{2.26} = 5.60 = 6.0 \text{ bolts.}$$

Distance L_i between the distance of end fasteners is

$$L_i = 7s = 7 \times 3d = 21d > 15d$$

Use eqn 1.13 to get the reduction factor B_L :

$$B = 1 - \frac{L_i - 15d}{200d} = 1 - \frac{21d - 15d}{200d} = 0.970$$

$$q_b = 0.25 \times 4 \times 0.970 = 0.970 \text{ t/cm}^2$$

$$R_{sh} = 2 \times 0.970 \times \frac{\pi \times (1.2)^2}{4} = 2.19 \text{ tons}$$

$$n_1 = \frac{16.1}{2.19} = 7.30 = 8.0 \text{ bolts.}$$

B. Using High Strength Bolts: ($\mu = 0.4$)

$$(10.9) (d = 12 \text{ mm}) P_s = 3.38 \text{ tons.}$$

$$n_1 = \frac{12.72}{3.38} = 3.76 = 4.0 \text{ H.S.B.}$$

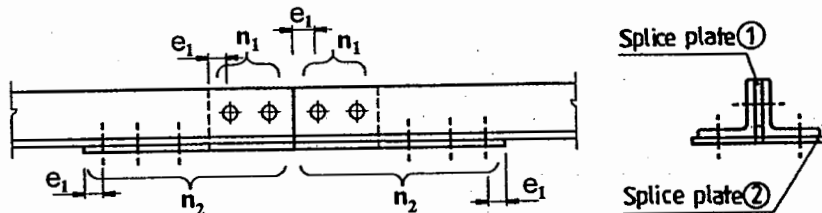
$$n_2 = \frac{16.10}{3.38} = 4.76 = 5.0 \text{ H.S.B.}$$

Example (1.6): Splice of Truss Members

It is required to design a splice for a lower tension member of a roof truss. The cross section is composed of 2 $\angle$ 75x75x8. Use Non-Pretensioned bolts of Grade 5.6 - $d = 20$ mm. The design force of the member is 24 tons (case I). Thickness of gusset plate = 8 mm of St 37 ($F_u = 3.6 \text{ t/cm}^2$). Then redesign the connection using H.S.B.

SolutionAlternative (1) [Two Splice Plates]1. Dimensions of Splice Plate (1):

Splice plate (1) [7.2x8 mm] can transmit $(7.2 - 2.2) \times 0.80 \times 1.40 = 5.6$ tons.



$$\text{Max Strength} = 2 (11.5 - 2.2 \times 0.8) \times 1.4 = 27.27 \text{ tons}$$

$$\text{The remaining force to be transmitted} = 27.27 - 5.6$$

$$(\text{By splice plate (2)}) = 21.67 \text{ tons.}$$

$$q_b = 0.25 \times 5 = 1.25 \text{ t/cm}^2 \quad (\text{Grade 5.6})$$

$$R_{d.sh} = 2 \times 1.25 \times \frac{\pi \times (2)^2}{4} = 7.85 \text{ tons} \quad (d = 20)$$

$$F_b = 1.20 \times 3.6 = 4.32 \text{ t/cm}^2 \quad (\text{take } e_1 = 3d)$$

$$R_b = 4.32 \times 2.0 \times 0.80 = 6.91 \text{ tons.}$$

$$(d = 20)$$

Field Splice

$$n_1 = \frac{5.6}{6.91} \times 1.15 = 0.93 \text{ take 1 bolt}$$

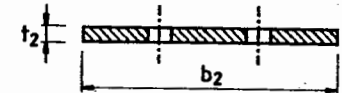
For one bolt connection reduce F_b according to Eqn 1.14

$$R_b = 0.75 \times 3.6 \times 2.0 \times 0.8 = 4.32 \text{ tons} ; n_1 = 5.6/4.32 = 2 \text{ bolts}$$

2. Dimensions of Splice Plate (2):

$$(b_2 - 2 \times 2.2) t_2 = \frac{21.67}{1.40} = 15.47 \text{ cm}^2$$

$$\text{Choose } b_2 = 18 \text{ cms.}$$



$$t_2 = \frac{15.47}{(18 - 2 \times 2.2)} = 1.13 \text{ cm take } t_2 = 12 \text{ mms.}$$

Non-pretension bolts :- Grade 5.6

$$R_{s.sh} = 0.25 \times 5 \times \frac{\pi \times (2)^2}{4} = 3.927 \text{ tons} \quad (d = 20)$$

$$R_b = 1.2 \times 3.6 \times 2 \times 0.80 = 6.91 \text{ tons.} \quad (d = 20)$$

$$n_2 = \frac{21.67}{3.927} \times 1.15 = 6.34 \text{ take } n_2 = 8 \text{ bolts}$$

(4 bolts on each row)

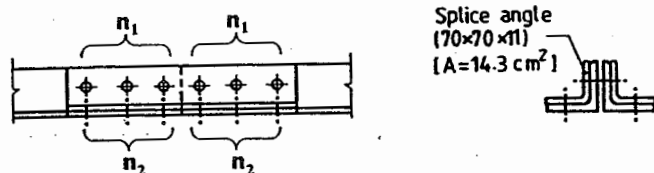
Using High Strength Bolts: (10.9) - $\mu = 0.4$ - M20

$$P_s = 4.93 \text{ tons}$$

$$n_1 = \frac{5.6}{2 \times 4.93} \times 1.15 = 0.65 = 1 \text{ H.S.B. (10.9)-(d=20 mms)}$$

$$n_2 = \frac{21.67}{4.93} \times 1.15 = 5.054 = 3 \text{ bolts on each angle (H.S.B-10.9)}$$

Alternative (2) [Splice Angles]



Net area of splice angles $\geq$ Net area of member angles.

$$\begin{aligned} \text{Net area of member angles} &= 2 (11.5 - 2 \times 2.2 \times 0.80) \\ &= 15.96 \text{ cm}^2. \end{aligned}$$

$$\begin{aligned} \text{Net area of splice angles} &= 2 (14.3 - 2 \times 2.2 \times 1.1) \\ &= 18.92 \text{ cm}^2 > 15.96 \text{ cm}^2 \text{ (o.k.)}. \end{aligned}$$

For the Determination of (n_1) and (n_2) :

$$\begin{aligned} R_{s.sh} &= 3.927 \text{ tons.} \\ (d &= 20) \end{aligned}$$

$$\begin{aligned} R_b &= 6.91 \text{ tons} \\ (d &= 20) \end{aligned}$$

$$n_1 = n_2 = \frac{27.27 \times 1.15}{4 \times 3.927} = 1.99 \text{ bolts} \quad \text{take (2 bolts)}$$

If Grade (4.6) is utilised :-

$$R_{s.sh} = 0.25 \times 4 \times \frac{\pi \times (2)^2}{4} = 3.14 \text{ tons}$$

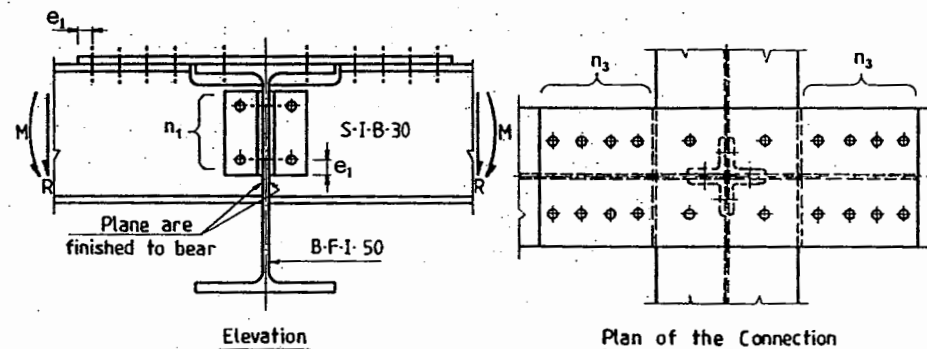
$$R_b = 0.8 \times 3.6 \times 2 \times 0.8 = 4.608 \text{ tons ; where } e_1 = 2d$$

$$n_1 = n_2 = \frac{27.27 \times 1.15}{4 \times 3.14} = 2.49 \quad \text{Take (3 bolts)}$$

Example (1.7): Continuity of Beams - Concentric Shear:

Design a bolted moment splice connection allowing for continuity $M = 7.0 \text{ mt}$, $R = 7.20 \text{ tons}$. The secondary beam is a S.I.B. No. 30 while the main beam is a B.F.I.B. No. 50. Use $d = 16 \text{ mm}$. - Grade 5.6 - All steel elements are of St 37 ($F_U = 3.6 \text{ t/cm}^2$).

Solution



1. Top Plate:

$$\text{Force in top plate} = \frac{M}{d} = \frac{7 \times 100}{(30 - 1.62)} = 24.66 \text{ tons.}$$

For the top plate use the same breadth as the S.I.B. No. 30 $b = 12.50 \text{ cms}$ while the thickness equals 2 cms.

$$f_t = \frac{24.66}{2 (12.5 - 2 \times 1.80)} = 1.38 \text{ t/cm}^2 < 1.40 \text{ Safe}$$

$$2. \text{ H.S.B. } d = 16 \text{ mms} - P_s = 3.16 \text{ tons} - \mu = 0.4$$

$$n_3 = \frac{24.66}{3.16} = 7.8 \quad \text{use 8 H.S.B. (10.9) M. 16.}$$

$$n_1 = n_2 = \frac{7.20}{2 \times 3.16} = 1.13 \text{ take 2 H.S.B. (10.9) - M 16.}$$

Using Bolts of the non-pretension type: (d = 16 mm) (Grade 5.6)

$$\text{up plate } R_{s.sh} = 0.25 \times 5 \times \frac{3.14 \times 1.6^2}{4} = 2.51 \text{ tons.}$$

$$R_b = 0.8 \times 3.6 \times 1.62 \times 1.60 = 7.46 \text{ tons. (} e_1 = 2d \text{)}$$

$$n_3 = \frac{24.66}{2.510} = 9.80 \text{ bolts take 10 bolts, 5 bolts in each row}$$

S.I.B. No. 30

$$R_{s.sh} = 2.510 \text{ ton.}$$

$$R_{d.s} = 2 \times 2.510 = 5.020 \text{ tons}$$

$$R_b = 0.8 \times 3.6 \times 1.08 \times 1.60 = 4.97 \text{ tons}$$

$$n_1 = n_2 = \frac{7.20}{4.97} = 1.43 \text{ bolts } \therefore \text{ take "2" bolts}$$

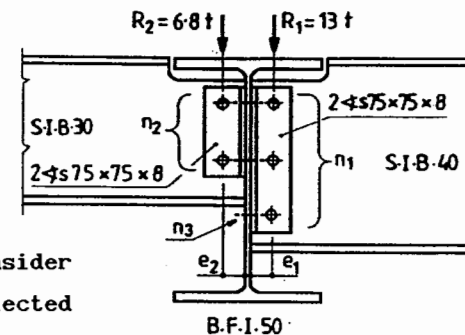
N.B. The bottom flange of the S.I.B. No. 30 must be finished to bear in order to transmit the compression force by contact.

Example (1.8): Connection of Secondary Beams to Main Beams

Design the flexible connection

shown in Fig. using:

Ordinary bolts d = 20 mms.
Grade (4.6)



For group of bolts (3) consider that the eccentricity is neglected and compare the results with the case where eccentricity is taken into consideration.

Use for the framing angles two angles 75 x 75 x 8.

Solution

1. Connection of Web Secondary to the Framing Angles:

$$\text{S.I.B. No. 40 ; } t_w = 1.44 \text{ cm.}$$

(a) Using Ordinary Bolts:

$$R_{d.sh} = 2 \times 0.25 \times 4.0 \times \frac{\pi \times 2^2}{4} = 6.27 \text{ tons.}$$

$$R_b = 0.8 \times 3.6 \times 2 \times 1.44 = 8.29 \text{ tons. (} e_1 = 2d \text{)}$$

$$n_1 = \frac{13}{6.27} = 2.07 = 3.0 \text{ bolts}$$

$$\text{S.I.B. No. 30 ; } t_w = 1.08 \text{ cm.}$$

$$R_{d.sh} = 6.27 \text{ tons.}$$

$$R_b = 0.8 \times 3.6 \times 2 \times 1.08 = 6.22 \text{ tons.}$$

$$n_2 = \frac{6.80}{6.22} = 1.09 = 2.0 \text{ bolts.}$$

b) Using H.S.B.: $P_s = 2 \times 3.16 = 6.32 \text{ tons.} - M 16 \text{ mms.}$

$$n_1 = \frac{13.0}{6.32} = 2.05 \text{ take } n_1 = 2 ; n_2 = \frac{6.80}{6.32} = 1.075$$

$$\text{take } n_2 = 2.0$$

1. Connection of Framing Angles with Main Beam : (Eccentricity Neglected)

$$\text{B.F.I } N \approx 50 ; t_w = 1.6$$

(a) Using Non-Pretensioned bolts M20 (Grade 4.6) :-

$$R_{s.sh} = 3.135 \text{ tons.} ; R_b = 0.80 \times 3.6 \times 2 \times 1.60$$

$$R_{d.sh} = 6.27 \text{ tons.} = 9.21 \text{ tons.}$$

Having from previous paragraph $n_1 = 3.0 \text{ bolts.} ; n_2 = 2.0 \text{ bolts.}$

Then we'll use on the outstanding leg 6.0 bolts and 4.0 bolts respectively for the S.I.B. No. 40 and No. 30. (Front+Back)

$$\text{Load per bolt from S.I.B. No. 30} = \frac{6.80}{4} = 1.70 \text{ ton} < R_{s.sh}$$

$$\text{Load per bolt from S.I.B. No. 40} = \frac{13.0}{6} = 2.16 \text{ tons} < R_{ss} \text{ (o.k.)}$$

$$\text{Load on common bolt} = 1.70 + 2.16 = 3.86 \text{ tons} < R_{d.s} \text{ (o.k.)}$$

(b) Using High Strength Bolts: M16 - $P_s = 3.16 \text{ tons} :-$

$$\text{Load per H.S.B. S.I.B. No. 30} = \frac{6.8}{4} = 1.7 \text{ tons.}$$

$$\text{Load per H.S.B. S.I.B. No. 40} = \frac{13}{6} = 2.16 \text{ tons.}$$

$$\text{Load on common H.S.B.} = 1.7 + 2.16 = 3.86 \text{ tons} < 2 P_s$$

Connection of Connecting Angles with Main Beam : " Eccentricity Considered"

Connection of Main Beam with

S.I.B. No. (40)

$$e = \frac{7.5 - 0.8}{2} + 0.8 = 4.15 \text{ cms}$$

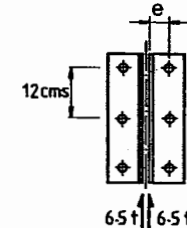
$$M_t = 6.50 \times 4.15 = 26.975 \text{ t.cm}$$

$$Q = 6.50 \text{ tons}$$

$$R_{(Q)} = \frac{6.50}{3.0} = 2.16 \text{ tons}$$

$$R_{(M_t)} = \frac{26.975 \times 12}{2 \times 12^2} = 1.123 \text{ ton}$$

$$R_{\text{total}} = \sqrt{2.16^2 + 1.123^2} = 2.43 \text{ tons} < R_{ss} \text{ (o.k.)}$$



Connection of S.I.B. No. (30)

with the Main Beam

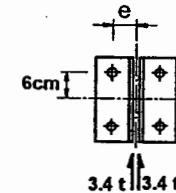
$$M_t = 3.40 \times 4.15 = 14.11 \text{ t.cm}$$

$$Q = 3.40 \text{ tons}$$

$$R_{(Q)} = \frac{3.4}{2} = 1.70 \text{ tons}$$

$$R_{(M_t)} = \frac{14.11 \times 6}{2 \times 6^2} = 1.175 \text{ ton}$$

$$R_{\text{total}} = \sqrt{1.7^2 + 1.175^2} = 2.06 \text{ tons} < R_{ss} \text{ (o.k.)}$$



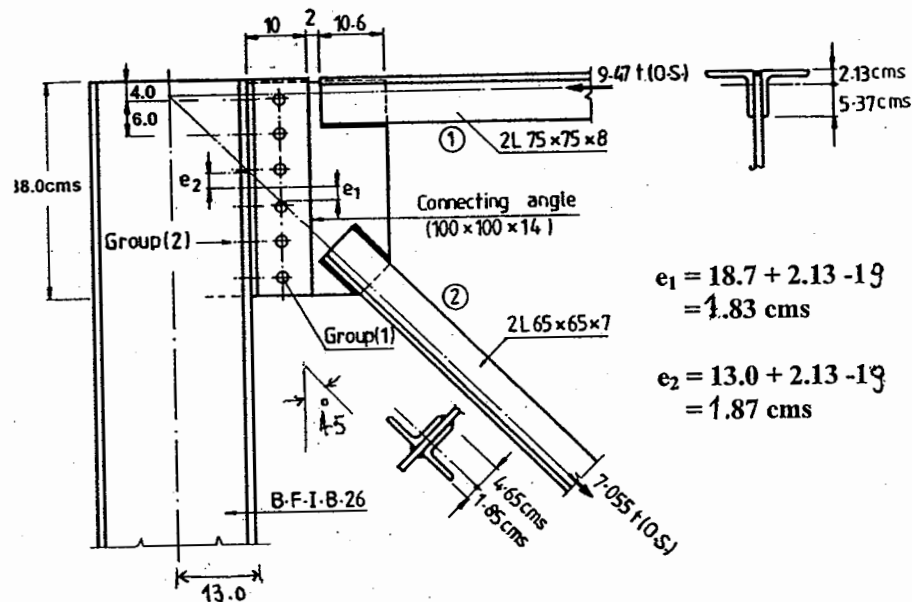
N.B. For H.S.B proceed as for O.B. and compare R_t with P_s .

$$R_{\text{total}} = 2.43 \text{ tons} < P_s \text{ (3.16 tons) O.K.}$$

$$\text{also } R_{\text{total}} = 2.06 \text{ tons} < P_s \text{ (3.16 tons) O.K.}$$

**Example (1.9): Design of Column - Truss Corner Connection**

Bolts of the non-pretensioned type Grade 4.6 and the connecting angle are of St 37.



$$e_1 = 18.7 + 2.13 - 19 = 1.83 \text{ cms}$$

$$e_2 = 13.0 + 2.13 - 19 = 1.87 \text{ cms}$$

1. Using Ordinary Bolts (d = 20 mm):**1.1. Design of Welds:**

Member (1): 2 $\angle$ 75 x 75 x 8

D.F. = - 9.47 tons (I) thickness of gusset plate = 1.0 cm

$$\therefore F_1 = \frac{9.47 \times 2.13}{7.50} = 2.68 \text{ tons}; F_2 = \frac{9.47 \times 5.37}{7.50} = 6.78 \text{ tons}$$

Take $S_1 = 0.80$ cm as the thickness of angle.

$$\therefore L_1 = \frac{2.68}{2 \times 0.80 \times 0.72} + 2 \times 0.80 = 2.32 + 1.60 = 6.60 \text{ cms.}$$

$$\therefore L_2 = \frac{6.78}{2 \times 0.80 \times 0.72} + 2 \times 0.80 = 5.88 + 1.60 = 7.48 \text{ cms.}$$



Member (2): 2 $\angle$ 65 x 65 x 7

D.F. = + 7.055 tons (o.s)

$$\therefore F_1 = 2.0 \text{ tons}; F_2 = 5.05 \text{ tons}$$

Take $S = 0.70$ cm

$$\therefore L_1 = \frac{2}{2 \times 0.70 \times 0.72} + 2 \times 0.70 = 1.98 + 1.40 = 5.0 + 1.40 = 6.40 \text{ cms.}$$

$$\therefore L_2 = \frac{5.05}{2 \times 0.70 \times 0.72} + 2 \times 0.70 = 5.0 + 1.40 = 6.4 \text{ cms.}$$

1.2. Design of Group of Bolts (1): (M20) :-

Assume to use connecting angle (100 x 100 x 14)

Take (6) bolts $\therefore$ pitch = $\frac{36}{6} = 6$ cms.

End distance = $2d = 4$ cms.

Straining Actions:

$$Q = 7.055 \cos 45^\circ = 5.0 \text{ tons } \downarrow$$

$$N = 9.47 - 7.055 \cos 45^\circ = 4.48 \text{ tons } \leftarrow$$

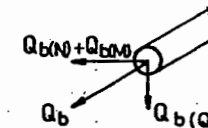
$$M_t = 9.47 \left(\frac{38}{2} - 2.13 \right) + 7.055 \cos 45^\circ \cdot e_1 \text{ where } e_1 = 1.83 \text{ cms from Fig.}$$

$$\therefore M = 168.9 \text{ t.cm.}$$

$$\therefore Q_{b(Q)} \downarrow = \frac{5.0}{6.0} = 0.83 \text{ ton}$$

$$\therefore Q_{b(N)} \leftarrow = \frac{4.48}{6} = 0.746 \text{ ton}$$

$$\therefore Q_{b(M_t)} \leftarrow = \frac{168.9 \times 15}{2 (15^2 + 9^2 + 3^2)} = 4.02 \text{ tons}$$



$$Q_{total} = \sqrt{(4.02 + 0.746)^2 + (0.83)^2} = 4.83 \text{ tons}$$

For bolts of Grade (4.6). M20 :-

$$R_{d.sh} = 2 \times 0.25 \times 4 \times \frac{\pi \times 2^2}{4.0} = 6.27 \text{ tons.}$$

$$R_{bearing} = 0.8 \times 3.6 \times 1.0 \times 2.0 = 5.76 \text{ tons. (end distance 2d)}$$

$$Q_{total} = 4.83 < R_{bearing} \therefore \text{Safe}$$

3. Design of Group of Bolts (2):

In order to guess the position of the N.A. we can assume that it lies at $h/7$ from the upper edge.

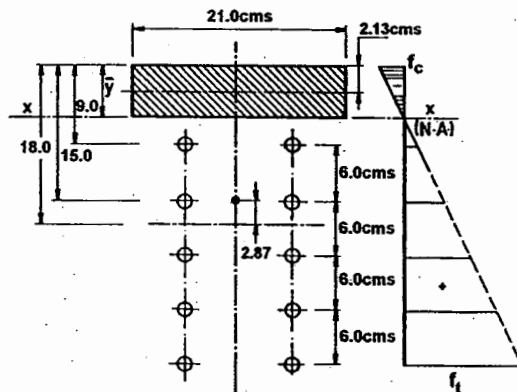
Then $h/7 = \frac{38}{7} = 5.42 \text{ cms}$ which means that we have 5 rows of bolts in tension while the 6th row is inside the compression zone.

The first moment of area about (x-x) will give the following equation:

$$\frac{21.0 \bar{y}^2}{2} = \frac{2\pi \times 2^2 \times 0.75}{4.0} \left[(10 - \bar{y}) + (16 - \bar{y}) + (22 - \bar{y}) + (28 - \bar{y}) + (34 - \bar{y}) \right]$$

$$\therefore \bar{y} = 6.0 \text{ cms.}$$

$$I_x = \frac{(6.0)^3 \times 21}{3} + \frac{2\pi \times 2^2 \times 0.75}{4} \left[4.0^2 + 10.0^2 + 16.0^2 + 22.0^2 + 28.0^2 \right]$$



$$I_x = 9707.4 \text{ cm}^4$$

Straining Actions:

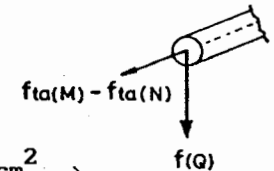
$$Q = 7.055 \sin 45^\circ = 5.0 \text{ tons. } \downarrow$$

$$N = 9.47 - 7.055 \cos 45^\circ = 4.47 \text{ tons } \leftarrow$$

$$M = 9.47 (5.83 - 2.13) + 7.055 \cos 45^\circ (19 - 5.83 - 1.87)$$

$$\text{Where } e_2 = 2.87 \text{ cms}$$

$$M = 81.4 \text{ ton.cm}$$



$$f_{ta(M)} = \frac{81.4 \times (34 - 6.0)}{9707.4} = 0.235 \text{ t/cm}^2 \rightarrow$$

$$f_{ta(N)} = \frac{4.48}{12 \times 2.45} = 0.152 \text{ t/cm}^2 \leftarrow$$

$$f_{ta}(\text{total}) = 0.235 - 0.152 = 0.083 \text{ t/cm}^2$$

$$R_{s.sh} = 0.25 \times 5.0 \times 2.45 = 3.063 \text{ tons}$$

$$R_t = 0.33 \times 3.6 \times 2.45 = 2.910 \text{ tons}$$

$$R_{ta} = 0.083 \times A_s = 0.108 \times 2.45 = 0.2 \text{ tons} < R_t$$

$$R_{sha} = \frac{5}{12} = 0.416 \text{ tons} < R_{s.sh}$$

Applying the interaction circular (equ. 1.10)

$$\left[\frac{0.416}{3.063} \right]^2 + \left[\frac{0.2}{2.910} \right]^2 = 0.022 < 1 \therefore \text{Safe}$$

$$T_{\text{ext}}(\text{total}) = f_{\text{ta}}(\text{total}) \times \text{Area of bolt } (A_s = 2.45 \text{ cm}^2)$$

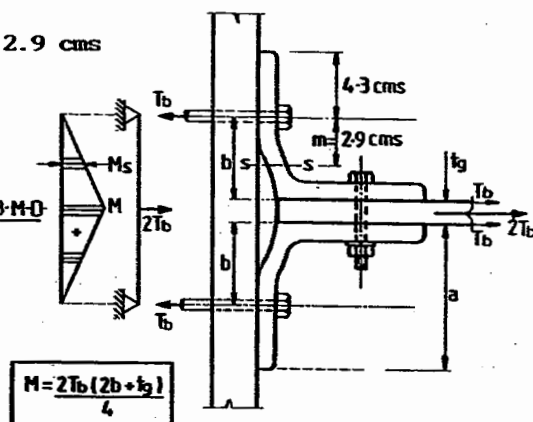
$$= 0.083 \times 2.45 = 0.2 \quad \text{tons} = R_{ta} = T_b$$

$$\therefore M_s = T_{\text{ext}} \times m$$

$$m = \left(\frac{10 - 1.4}{2} \right) - 1.4 = 2.9 \text{ cms}$$

$$\therefore M_s = 0.59 \text{ t.cms.}$$

$$\therefore t_{\text{req.}} = \sqrt{\frac{6 \times 0.59}{6 \times 1.40}}$$
$$= 0.65 \text{ cms.}$$



If at (s-s) we consider the full solid section use $F_b = 0.72 F_y$
 $= 1.728 \text{ t/cm}^2$ instead of 1.4 t/cm^2 - $t = 0.58 \text{ cms.}$

Hence choose for the framing angles $100 \times 100 \times 10$.

Assume using 6 bolts (10.9)

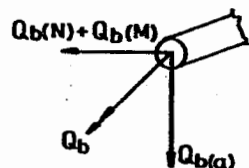
Straining Actions:

N = 4.48 tons ←

Q = 5.0 tons ↓

$$M = 173.85 \text{ ton.cm.}$$

$$\therefore Q_{bN} = 0.746 \text{ ton} \leftarrow \quad \therefore Q_{bM} = 4.02 \text{ tons} \leftarrow$$



$$\therefore Q_{b0} = 0.83 \text{ ton } \downarrow$$

$$\therefore Q_b = \sqrt{(Q_{bN} + Q_{bM})^2 + (Q_{bQ})^2} = 4.83 \text{ tons.}$$

For steel (37) and $\mu = 0.4$ case of loading (I).

$$\therefore P_s = 3.16 \times 2 = 6.32 \text{ tons}$$

(double shear)

$\therefore Q_b < P_s \quad \therefore \text{Safe}$

Straining Actions:

$Q = 5.0 \text{ tons } \downarrow$

N = 4.48 tons ←

$$M = 9.47 (19 - 2.13) - 7.055 \cos 45^\circ \times 2.87 = 146.9 \text{ t.cm}$$

$$I_x = \frac{38^3 \times 21}{12} = 96026 \text{ cm}^4$$

$$f_1 = \frac{146.9 \times 19}{96026} = 0.029 \text{ t/cm}^2$$

$$f_2 = \frac{146.9 \times 13}{96026} = 0.02 \text{ t/cm}^2$$

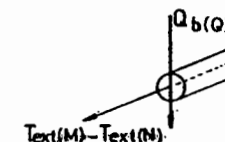
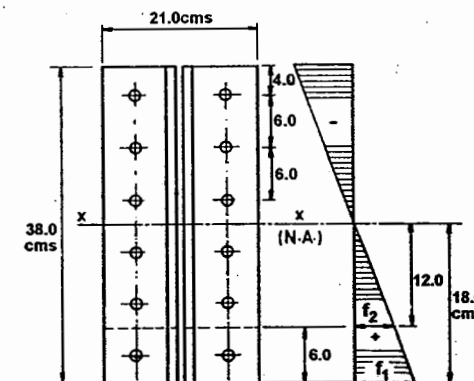
$$T_{\text{ext. (M)}} = \frac{0.029 + 0.02}{2} \times 7 \times \frac{21}{2}$$

$$\rightarrow$$

$$= 1.8 \text{ ton}$$

$$T_{\text{ext. (N)}} = \frac{4.48}{12} = 0.373 \text{ ton.} \quad \leftarrow$$

$$Q_{b(Q)} = \frac{5}{6} = 0.83 \text{ ton } \downarrow \text{ (6 bolts in compression)}$$



From E.C.P. (Table 2.5), using H.S.B.

10.9) (M16). $\rightarrow (T = 9.89 \text{ tons } P_s = 3.16 \text{ tons})$

$$P_s = 3.16 \text{ tons} > Q_{b(Q)} \text{ Safe}$$

As no pure tension exists

$$\frac{1.8}{0.8 \times 9.89} = 0.22 < 1.0 \text{ Safe}$$

3. Design of Connecting Angle:

$$T_{\text{ext}}(\text{total}) = 1.8 - 0.373 \\ = 1.427 \text{ ton.}$$

$$M = \frac{2 \times 1.427 \times (2 \times 5.7 + 1.0)}{8} \\ = 4.42 \text{ ton.cm.}$$

$$\therefore t_c = \sqrt{\frac{6 \times 4.42}{7 \times 1.40}} = 1.64 \text{ cm} > \text{req.}$$

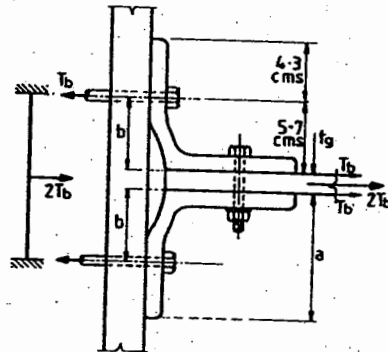
$$(t_c = 1.40 \text{ cm}) \text{ Unsafe}$$

actual

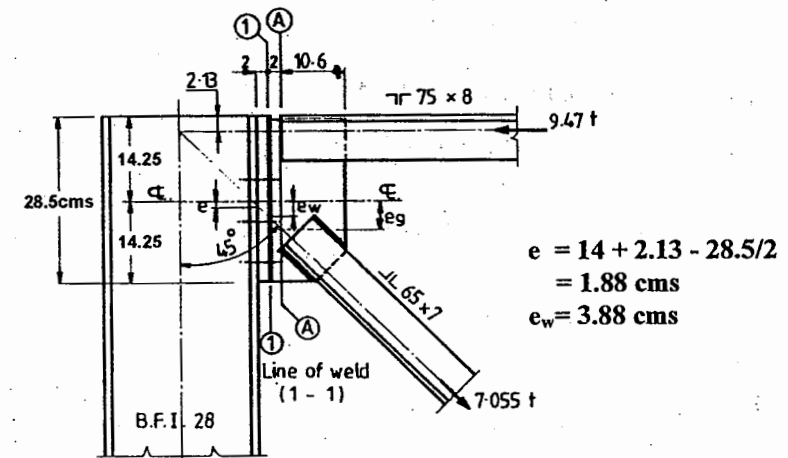
If plastification is allowed (Refer to chapter 4):

$$\text{ence: } t_c = \sqrt{\frac{6 \times 3.72}{6 \times 2.4}} = 1.24 \text{ cm} < 1.40 \text{ cms (o.k.)}$$

req.



Example (1.10): Column Truss Connection with a Tee-Stub:



1. Design of H.S.B. Connecting End Plate to Column Flange:

Straining Actions:

$$N = 9.47 - 7.055 \cos 45^\circ = 4.48 \text{ tons} \leftarrow$$

$$Q = 7.055 \sin 45^\circ = 5.00 \text{ tons} \downarrow$$

$$M = 9.47 \left(\frac{28.5}{2} - 2.13 \right) + 7.055 \cos 45^\circ \cdot e; e = 1.88 \text{ cm from Fig.}$$

$$\therefore M = 124 \text{ ton.cm.}$$

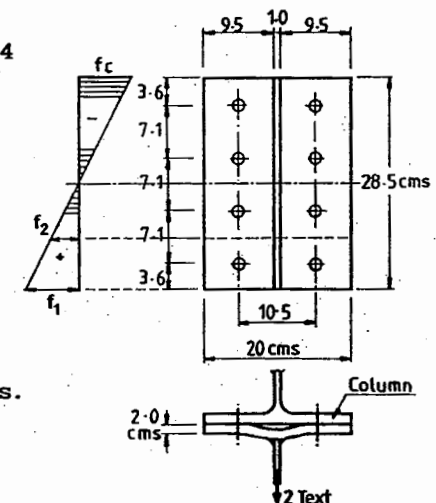
$$I_x = \frac{(28.5)^3 \times 20}{12} = 38581.88 \text{ cm}^4$$

$$f_1 = \frac{124 \times 14.25}{38581.88} = 0.046 \text{ t/cm}^2$$

$$f_2 = \frac{124 \times 7.1}{38581.88} = 0.023 \text{ t/cm}^2$$

$$T_{\text{ext}}(M) = \frac{0.046 + 0.023}{2} \times 10 \\ \times 7.15 = 2.46 \text{ tons.}$$

$$T_{\text{ext}}(N) = \frac{4.48}{8.0} = 0.56 \text{ ton}$$



$$Q_{b(Q)} = \frac{5}{4} = 1.25 \text{ ton} \downarrow \text{ (4 bolts in compression)}$$

Assume to use 4 bolts (8.8) (d = 12 mm).

$$\text{From E.C.P. } (T = 5.29 \times 0.70 = 3.71 \text{ tons, } Q = 1.69 \times 0.70 = 1.183 \text{ ton})$$

$$\therefore P_s = 1.183 > Q_{b(Q)} \text{ Safe}$$

$$\text{As no pure tension exists } \therefore \frac{T_{\text{ext}(M)}}{0.8 T} \leq 1.0$$

$$\frac{2.46}{0.80 \times 5.29} = 0.58 < 1.0 \text{ Safe}$$

2. Design of End Plate:

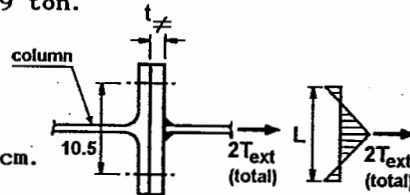
$$T_{\text{ext}(\text{total})} = 2.46 - 0.56 = 1.9 \text{ ton.}$$

$$M = 2 T_{\text{ext}(\text{total})} \times L / 8$$

$$M = \frac{2 \times 1.9 \times 10.50}{8} = 5.0 \text{ t.cm.}$$

$$\therefore t_{\text{req.}} = \sqrt{\frac{6 \times 5.0}{7.15 \times 1.40}} = 1.73 \text{ cm} = 1.8 \text{ cm}$$

$$\text{Take } t_{\text{req.}} = 2.0 \text{ cms}$$



3. Design of Weld:

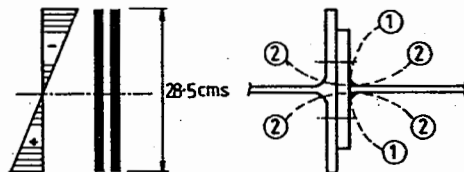
Section (1-1): (no contact)

$$Q = 5.00 \text{ tons} \downarrow$$

$$N = 4.48 \text{ tons} \leftarrow$$

$$M = 9.47 \times \left(\frac{28.5}{2} - 2.13 \right) +$$

$$7.055 \cos 45 \times e_w$$



$$\text{N.B. } M_{11} \cong M_{22}$$

Where $e_w = 3.88 \text{ cms}$ (from Fig.)

$$M = 134 \text{ ton.cm.}$$

Assume size of weld $S = 1.0 \text{ cm}$

$$I_x = \frac{(28.5)^3 \times 1.0 \times 2}{12} = 3858.19 \text{ cm}^4$$

$$f_{wMt_{\perp}}^{\rightarrow} = \frac{134 \times 28.5 / 2}{3858.19} = 0.495 \text{ t/cm}^2$$

$$f_{wN_{\perp}}^{\leftarrow} = \frac{4.48}{2 \times 1.0 \times 28.5} = 0.078 \text{ t/cm}^2$$

$$f_{t_{\perp}} = 0.495 + 0.078 = 0.573 \text{ t/cm}^2$$

$$q_{[Q/\downarrow]} = \frac{4.48}{2 \times 1.0 \times 28.5} = 0.078 \text{ t/cm}^2 \downarrow$$

$$f_{\text{eq.}} = \sqrt{(0.573)^2 + 3(0.078)^2} = 0.588 \text{ t/cm}^2 < 0.72 \times 1.1$$

Section (2-2):

$$q_{[Q/\downarrow]} = 0.078 \text{ t/cm}^2$$

$$q_{[N]_{\perp}}^{\leftarrow} = 0.078 \text{ t/cm}^2$$

$$q_{(M_t)_{\perp}}^{\leftarrow} = 0.495 \text{ t/cm}^2$$

$$\therefore q_{\text{total}} = \sqrt{(0.078)^2 + (0.078 + 0.495)^2} = 0.578 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2$$

∴ We can reduce size of weld according to equivalent stresses

$$\therefore S = \frac{1.0 \times 0.578}{1.1 \times 0.72} = 0.73 \text{ cm} = 8 \text{ mms.}$$

4. Check on Gusset Plate Thickness: (Critical Sec. A-A)

$$N = 4.48 \text{ tons} \leftarrow$$

$$Q = 4.48 \text{ tons} \downarrow$$

$$M = 9.47 \left(\frac{28.5}{2} - 2.13 \right) + 7.055 \times \cos 45^\circ \times e_g$$

$$\text{Where } e_g = e_w = 3.88 \text{ cms (from Fig.)}$$

$$\therefore M = 134 \text{ ton.cm.}$$

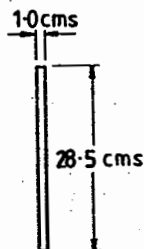
$$f_b = \frac{N}{A} + \frac{M}{Z}$$

$$\therefore f_b = \frac{4.48}{28.5 \times 1.0} + \frac{134 \times (28.5 / 2)}{\frac{1.0 \times (28.5)^3}{12}} = 1.147 \text{ t/cm}^2$$

For solid rectangular section

$$F_b = 0.72 F_y = 1.728 \text{ t/cm}^2 > 1.147 \text{ t/cm}^2 \quad (\text{O.K.})$$

$$\therefore q = \frac{3 \times 4.48}{2 \times 1.0 \times 28.50} = 0.235 \text{ t/cm}^2 < 0.35 \times 2.4 \quad (\text{o.k.})$$



Example (1.11):

Design the bracket shown in Fig. to resist a vertical load $V = 21.0$ tons and $H = 1.60$ ton. The braking force = 2.857 tons. Design the bolts considering that the applied loads are static loads. Bolts Grade 4.6. The bracket plate is of St. 37

Solution

Preliminary Dimensioning:

The dimensions of the bracket can be assumed as shown. Where the thickness of the bracket is taken equal to the column flange.

The vertical applied force $Q_y = 21.0$ tons is transmitted to the column flanges via the two lower triangular gusset plates. While the horizontal shock 1.6 tons is transmitted directly via the horizontal upper plate.

For a case of loading (I) only the vertical $Q_y = 21.0$ tons is included in the design. Neglect the effect of lateral shock (1.6 tons) its effect is transmitted directly to the column.

1. Section (S_1-S_1):

1.1. Straining Actions:

$$Q_y = 21.0 \text{ tons.}$$

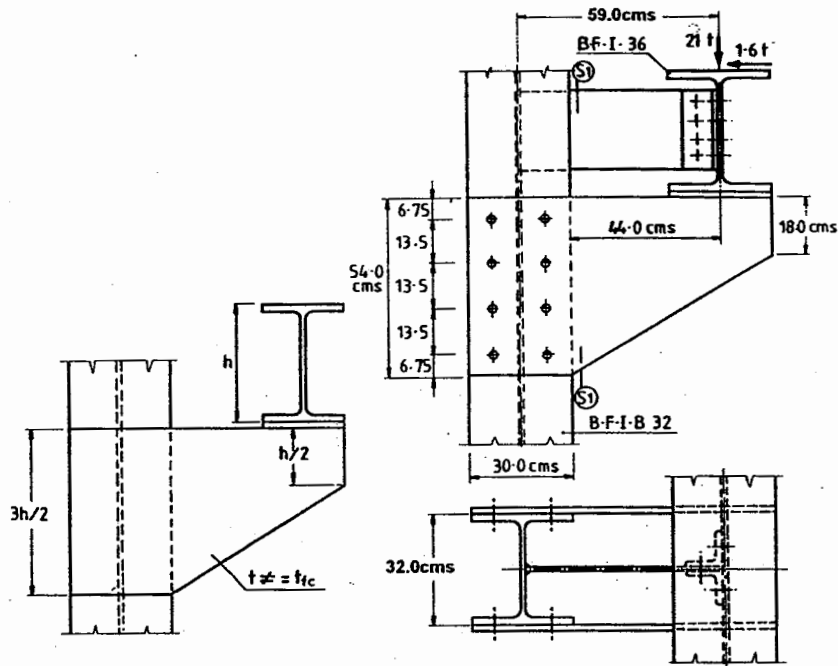
$$M_x = 21 \times 44 = 924 \text{ t.cm.}$$

1.2. Properties of Area:

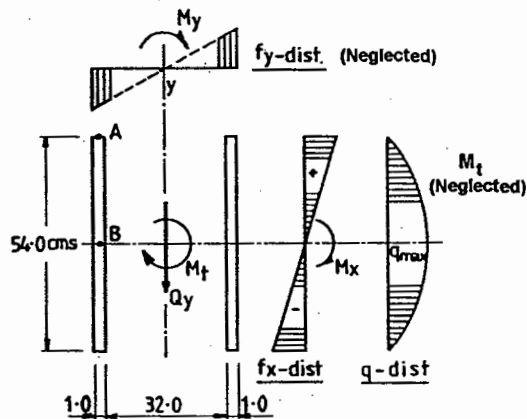
$$A = 2 \times 54.0 \times 1.0 = 108 \text{ cm}^2.$$

where the thickness of the triangular gusset plates = 1.0 cm.

$$I_x = 2 \times \frac{54^3 \times 1.0}{12} = 26244 \text{ cm}^4$$



(A) Case of Loading (I) without Bracking Force



1.3. Stresses:

$$f_{(A)} = \frac{M_x}{Z_x} = \frac{924 \times 27}{26244} = 0.95 \text{ t/cm}^2 < 0.72 F_y \text{ t/cm}^2 \text{ (o.k.)}$$

(solid section)

$$q_{(B)} = \frac{3 \times 21}{2 \times 108} = 0.29 \text{ t/cm}^2 < 0.35 \times 2.4 \text{ (o.k.)}$$

2. Ordinary Bolts Connecting Bracket to Column:

Use two rows of 4 bolts each.

Vertical pitch = 135 mms ; Horizontal pitch = 150 mms.

2.1. Straining Actions:

$$Q_y = 21.0 \text{ tons.}$$

$$M_t = 21 \times (44 + 15) = 1239 \text{ t.cm.}$$

2.2. Properties of Area:

Assume diameter of bolts $d = 20 \text{ mms}$

Threads are excluded from the shear plane

$$A = \frac{\pi \times (2)^2}{4.0} = 3.14 \text{ cm}^2$$

$$r_1 = \sqrt{(7.5)^2 + (6.75)^2} = 10.09 \text{ cms;}$$

$$r_2 = \sqrt{(7.5)^2 + (20.25)^2} = 21.594 \text{ cms;}$$

$$I_p = 2 \times 4 \times 3.14 \times (10.09^2 + 21.594^2) = 14.27 \times 10^3 \text{ cm}^4$$

2.3. Stresses:

$$R_{Q \downarrow} = \frac{21}{2 \times 8} = 1.31 \text{ tons}$$

$$R_{M_t \downarrow} = \frac{1239 \times 7.5}{4544.8} = 2.45 \text{ tons}$$

$$R_{M_t \rightarrow} = \frac{1239 \times 20.25}{4544.8} = 5.52 \text{ tons}$$

$$R_{\text{total}} = \sqrt{(1.31 + 2.054)^2 + (2.52)^2} = 6.46 \text{ tons.}$$

$$R_{\text{ss}} = 0.25 \times 4 \times \frac{\pi d^2}{4} = 3.14 \text{ tons (Grade 4.6)}$$

$$R_b = 1.2 \times 3.6 \times 1.0 \times 2 = 8.64 \text{ tons (} e_1 = 6.75 \text{ cm} > 3d \text{)}$$

$$R_{\text{total}} > R_{\text{ss}} \quad \text{Unsafe}$$

Hence:

use bolts of Grade (10.9) M22 ($A = 3.8 \text{ cm}^2$)
with 2 rows and 4 bolts in each.

Where:

$$R_{\text{ss}} = 0.2 \times 9 \times 3.8 = 6.84 \text{ tons} > R_{\text{total}}$$

(B) Case of Loading (II) with Braking Force

$$\text{B.F.} = \text{Braking force} = \frac{1}{7} \times 2 \times 10 = 2.857 \text{ tons}$$

The braking force is assumed to be resisted by one bracket.

1. Section (S_1-S_1):

1.1. Straining Actions:

$$Q_y = 21.0 \text{ tons.}$$

$$M_x = 21 \times 44 = 924 \text{ t.cm.}$$

$$M_y = 2.857 \times 44 = 125.7 \text{ t.cm}$$

$$M_t = 2.857 \times (36 + 1 + \frac{54}{2}) = 182.8 \text{ t.cm}$$

1.2. Properties of Area of bracket plate :

$$A = 2 \times 54.0 \times 1.0 = 108 \text{ cm}^2.$$

$$I_x = 2 \times \frac{54^3 \times 1.0}{12} = 26244 \text{ cm}^4$$

$$I_y = 2 \times \left[\frac{54 \times 1^3}{12} + 54 \times 1.0 \times 16.5^2 \right] = 29412 \text{ cm}^4.$$

$$I_p = 26244 + 29412 = 55656 \text{ cm}^4.$$

1.3. Stresses: (For case II stress are increased by 20%)

Point (A):

$$f_A = \frac{924 \times 27}{26244} + \frac{125.7 \times 17}{29412} = 1.0 \text{ t/cm}^2$$

$$q_{Mt} = \frac{182.8 \times 31.90}{55656} = 0.1 \text{ t/cm}^2$$

$$f_{\text{equ.}} = \sqrt{(1.0)^2 + 3(0.1)^2} = 1.03 \text{ t/cm}^2 < 1.1 \times 1.4 \times 1.2 \text{ (o.k.)}$$

Point (B):

$$q_Q^\downarrow = \frac{3 \times 21}{2 \times 108} = 0.29 \text{ t/cm}^2$$

$$q_{Mt}^\downarrow = \frac{182.8 \times 17}{55656} = 0.056 \text{ t/cm}^2$$

$$q_{\text{total}} = 0.29 + 0.057 = 0.34 \text{ t/cm}^2 < 1.2 \times 0.35 \times 2.4 \text{ (o.k.)}$$

2. Non-Pretensioned Bolts Connecting Bracket to Column Flange:

Straining Actions (10.9) - M22 :-

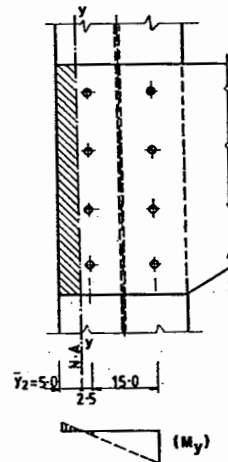
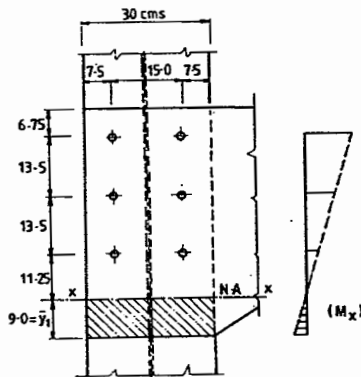
$$M_x = 2.857 \times (36 + 1 + \frac{54}{2}) = 182.84 \text{ t.cm.}$$

$$M_y = 2.857 \times (44 + 15) = 168.56 \text{ t.cm.}$$

$$M_t = 21 \times (44 + 15) = 1239 \text{ t.cm.}$$

$$Q_y = 21.0 \text{ tons. ; } N = 2.857 \text{ tons}$$

2.1. Properties of Area:



$$M(2-2) - A_s = 3.03 \text{ cm}^2 \quad A = 3.8 \text{ cm}^2$$

$$\text{Assume } \bar{y}_1 = \frac{h}{6} = \frac{54}{6} = 9 \text{ cms}$$

$$I_{xx} = 2 \left[\frac{30 \times 9^3}{3} + 2 \times 3.03 \times (11.25^2 + 24.75^2 + 38.25^2) \right]$$

where

$$A_s = 3.03 \text{ cm}^2 \quad (d = 22 \text{ mms})$$

$$I_{xx} = 41269.86 \text{ cm}^4$$

$$\text{Assume } \bar{y}_2 = \frac{30}{6} = 5.0 \text{ cms}$$

$$T_1(M_x) = \frac{182.84 \times 38.25 \times 2.45}{41236.687} = 0.513 \text{ tons}$$

$$T_2(M_y) = \frac{168.56 \times 17.5 \times 2.45}{12075} = 0.74 \text{ tons}$$

$$T_3(N) = \frac{2.857}{18} = 0.357 \text{ tons}$$

(The braking force will compress one gusset and will move outward the other gusset - Hence use 8 bolts).

$$T(\text{total}) = 0.513 + 0.74 + 0.357 = 1.61 \text{ tons}$$

$$R_t = 0.33 F_{Ub} \cdot A_b = 0.33 \times 10 \times 3.03 = 9.99 \text{ tons} > 1.61 \text{ tons} \quad \text{O.K.}$$

$$I_p = 2 \times 4 \times (10.09^2 + 21.594^2) = 4544.8 \text{ cm}^4$$

$$Q_Q^\downarrow = 21/16 = 1.313 \text{ tons ; } Q_{Mt}^\downarrow = \frac{1239 \times 7.5}{4544.8} = 2.04 \text{ tons}$$

$$Q_{Mt}^\rightarrow = \frac{1239 \times 20.25}{4544.8} = 5.52 \text{ tons}$$

$$R_{s.sh} = 0.2 \times 9 \times 3.8 = 6.84 \text{ tons} \quad (\text{Grade 10.9}) (M22)$$

$$Q_{(\text{total})} = \sqrt{(1.313 + 2.04)^2 + (5.52)^2} = 6.46 \text{ tons} < 6.84 \text{ tons} \quad \text{O.K.}$$

Applying the circular interaction equation we get :-

$$\left[\frac{6.46}{6.84 \times 1.2} \right]^2 + \left[\frac{1.61}{9.99 \times 1.2} \right]^2 = 0.63 < 1 \quad \text{Safe}$$

CHAPTER 2

CHAPTER 2

HIGH STRENGTH BOLTED CONNECTIONS

2.1 Introduction:

As has been previously outlined, when Non-Pretensioned bolts are used the nuts are tightened sufficiently to prevent relative movement between the connected members. This will induce an initial tension in the bolts. No attempt is made to control this initial tension and so it was neglected.

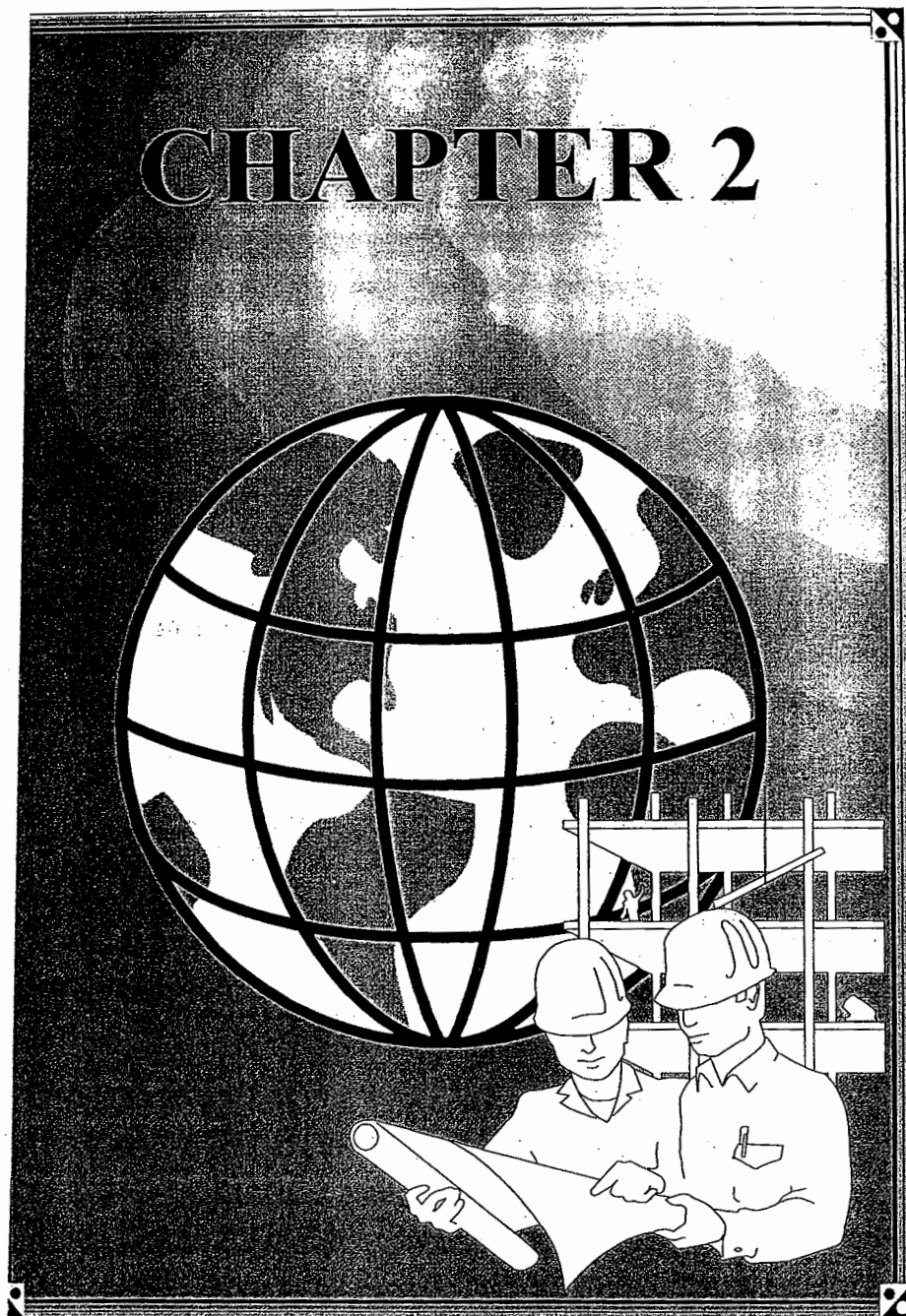
For many years the same uncontrolled methods of tightening were used with higher strength bolts in structural connections. Such bolts were employed in lieu of rivets where driving was difficult.

Aside from the high permissible bolt stresses, design procedures were the same as for ordinary mild steel bolts. In particular initial tension was neglected since it was indefinite.

Within the last thirty years high strength bolts have become the prime field fastener of structural steel. High strength bolts are tightened to a desired initial tension and the desirable features of the large clamping forces obtainable in high strength bolts have been recognized.

The bolts used are either the type (8.8) or (10.9). Simplified methods of controlling tension under field conditions are available.

It is assumed that, whenever high-strength bolted joints are considered, they are to be pretensioned according to one of the established methods.





2.2 Different Types of High Strength Bolts:

At the present time the two basic types of high strength bolts as specified by the E.C.P. 2001 are the (8.8) and the (10.9) types. Both types have hexagonal heads and have shorter thread lengths which makes it easier to exclude the threads from the shearing plane.

Fig. 2.1 and Table 2.1 show the different dimensions characterizing the two types.

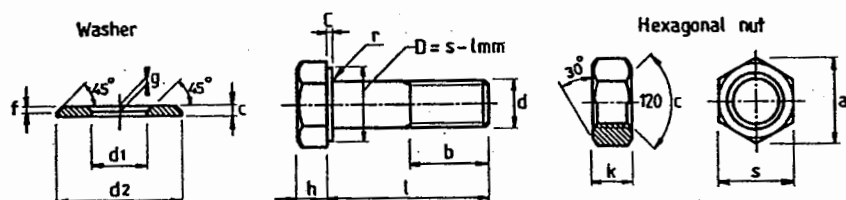


Fig. 2.1 General Layout of Bolt, Nut and Washer.

For high strength bolts (10.9) a hardened washer under each bolt head and nut must be used in order to develop a good distribution of stresses throughout the plates as shown in Fig. 2.2.

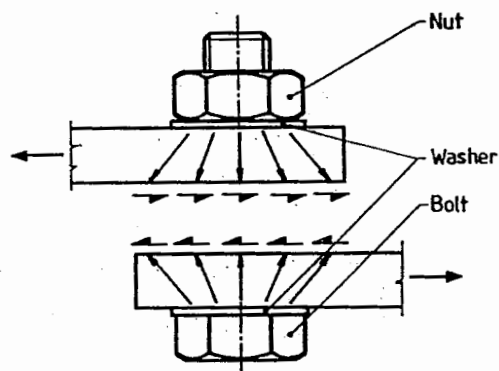


Fig. 2.2. Stress Distribution in Plates of a Bolted Connection



Table 2.2 shows the mechanical characteristics of the two types of bolts (8.8) and (10.9).

Table 2.1. Dimensions of High Strength Bolts

Nominal diameter d mms.	18	20	22	24	27
a	31.2	34.6	36.9	41.6	47.3
h	12	13	14	15	17
b 1(1)	28	29	31	32	35
1(2)	33	34	36	37	40
c	0.4	0.4	0.4	0.5	0.5
s	27	30	32	36	41
k	15	16	18	19	22
d ₁	19	21	23	25	28
d ₂	34	36	40	44	50
c	4	4	4	4	4
f	1.5	1.5	1.5	2	2
g	1.7	1.7	2.2	2.2	2.2

Dimensions are in mms.

(1) For $l \leq 80$ mms.

(2) For $80 \text{ mm} < l < 300 \text{ mm}$.

Table 2.2

Type	F_{yb} (t/cm ²)	F_{Ub} (t/cm ²)	F_{yb} / F_{Ub}
8.8	6.4	8.0	0.8
10.9	9.0	10.0	0.9

Connections made with high strength bolts may be of the friction type (material joined being clamped together by the tension induced in the bolts by tightening) or of the bearing type (material joined being prevented from moving by the bolt shank). In bearing type, bolt threads may be included in or excluded from the shear plane as has been given in Chapter (1).

2.3. Tension-elongation Curve for High Strength Bolts:

The load-elongation characteristics of a high strength bolt are more significant than the stress-strain diagram of the parent metal because performance is controlled by the threads. Also the stress varies along the bolt as a result of the gradual introduction of force from the nut and the change in section from the threaded to the unthreaded portion.

Curve 1 of Fig. 2.3 is a typical load-elongation curve of a (8.8) high strength bolt which has been placed in holding device, mounted in a testing machine, and tested to failure in simple tension. In practice, pretension is not applied in simple tension.

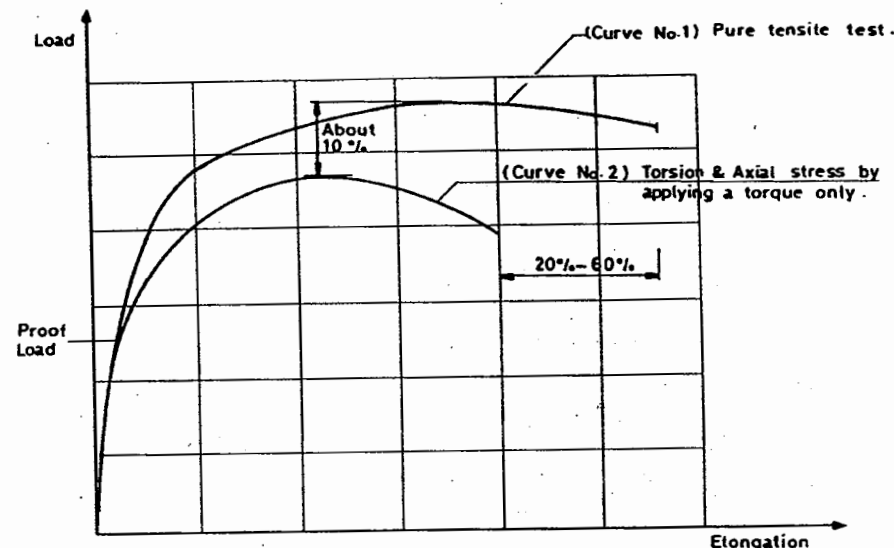


Fig. 2.3. Load-elongation Curve of High Strength Bolts

Curve 2 of Fig. 2.3. is obtained by tightening a nut against the compressive resistance of the connected material. As torque is applied to the nut, the portion not resisted by friction between the nut and the gripped material is transmitted to the bolt. While the torque is active, it causes torsional shearing stresses which combine with the tensile stress to result in a condition different from that in a pure tension test. If tightening is continued beyond the normal limit, it is possible, in this way, to reach fracture of the bolt. Ultimate axial strength of bolts (8.8) twisted to failure are approximately 10 percent less than those of bolts pulled to failure. The total elongation at rupture is reduced by a percentage ranging from 20 to 60.



In practice, bolts are often tightened into the inelastic range. The question then arises as what will happen if we apply tensile force following a tightening torque. The prior plastic torsional deformation appears to have a negligible effect on tensile resistance. Tests have shown that even when torque reaches the ultimate strength in a torque-tension test before continuing loading in simple tension, there is no significant decrease in ultimate tensile strength as shown in Fig. 2.4.

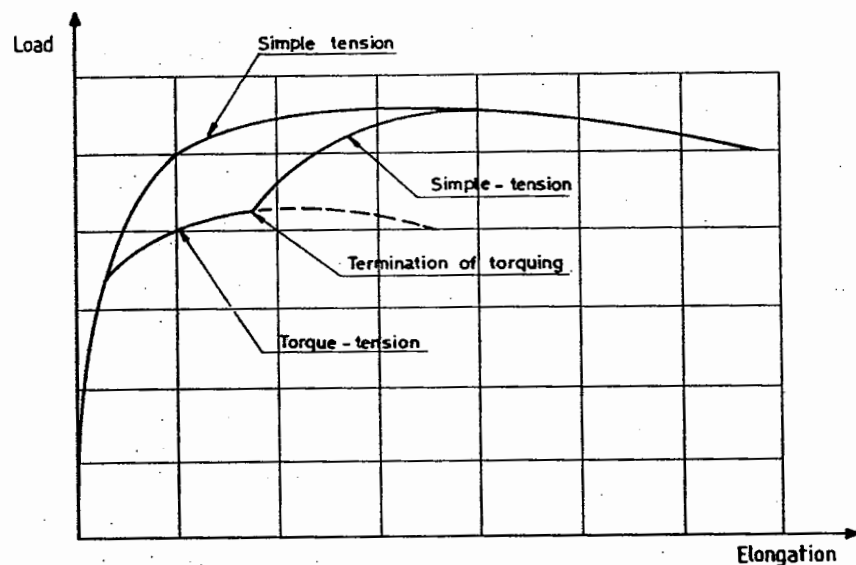


Fig.2.4

2.4 Installation Procedures:

Current installation procedures aim to pretension high strength bolts to their specified proof load (pretension load). One of the earlier working methods was based on the observation that, while the bolt remains elastic, the torque (M_a) required to produce a given pretension is approximately proportional to



the product of the pretension and the bolt diameter,

$$M_a = K.T.d. \quad (2.1)$$

where d = nominal bolt diameter.

T = bolt pretension.

K = dimensionless coefficient of proportionality

M_a = the torque to be applied.

Although K may be expected to vary with the material and the surface condition of the plates, nuts, and washers, a fair approximation of the required torque is obtained by taking $K=0.2$ when a hardened washer is used under whichever element is turned, the nut or the bolt head. When this method is used, installation is checked by a hand torque wrench or controlled by a pneumatic impact wrench set to stall or otherwise indicate the attainment of the desired torque.

Many structures were erected successfully using this procedure but improved methods were sought because:

- Equation 2.1. is approximate.
- When one tries to obtain the proof load through torque control, there is a relatively small margin of safety against the ultimate strength of the bolt as indicated in Fig. 2.3.

Current specifications permit two methods of tightening:

(a) Calibrated wrench method:

The E.C.P. (2001) " clause 6.5.3." permits the use of the calibrated wrench tightening method. In this method, calibrated wrenches are used in which either the amount of torque or angle of rotation of the nut is indicated.

Torque wrenches which have a cut-out device to limit the amount of torque must be employed. Wrenches may be of the manual, pneumatic, or electric type. Equation (2.1) is used to compute the required torque.

The turn of nut method:

In the turn of nut method the pretension force in the bolt developed by applying a specific rotation to the nut from an initially snug position which causes a specific amount of strain in the bolt. A nut is considered to be snug when impacting begins as it is being tightened by an impact wrench. This snugness or initial tightness position will induce a clamping force and is corresponding to half a turn of the nut. This induced clamping force occurs sufficiently far along the flat portion (Fig. 2.5) but exceeds the proof load.

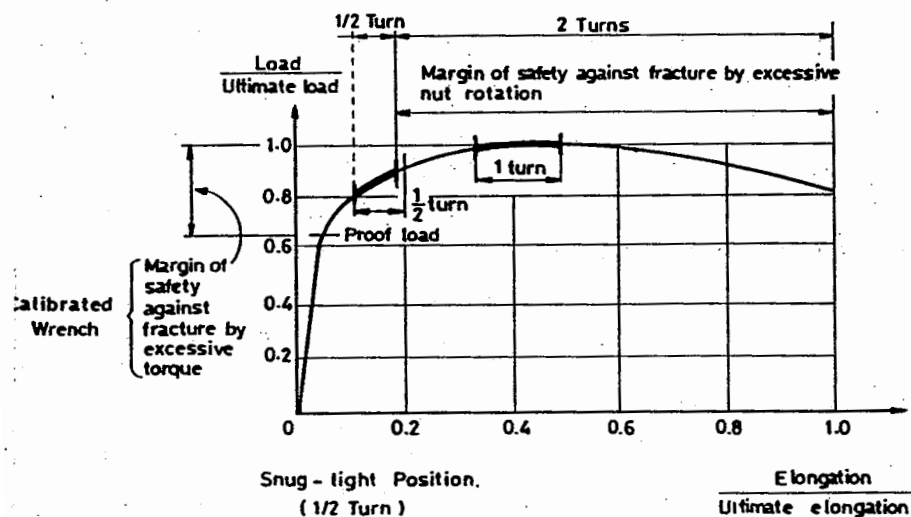


Fig. 2.5 Comparison of Tension Control Methods.

One half turn of the nut from the snug-tight position strains the bolt one half thread-pitch, but the corresponding elongation fall within a narrow range as indicated by Fig. 2.5. It is important to notice that it generally takes 2.5 turns from the snug position until fracture of the bolt. This means that there is a margin of safety against fracture during tightening of about 5 to 1. Even one full turn leaves a margin of about 2.5 to 1.

It has to be noticed that when using the calibrated wrench method the criterion of failure is the strength. Fig. 2.5. shows the corresponding margin of safety against fracture. Thus there is a greater margin of safety against fracture through inadvertent over tightening than when torque control is used.

Table 2.3 shows the required nut rotation from snug tight position for the different cases of high strength bolts.

Table 2.3. Required Nut Rotation for High Strength Bolts

When both outer faces of bolted parts are normal to the bolt axis, or when one face is normal to the axis, and the other face is sloped not more than 1:20. (Beveled washer not used).		When both outer faces of bolted parts are sloped not more than 1:20. (Beveled washers not used).
Bolt length up to 8 diameters or 20 cms.	Bolt length exceeding 8 diameters or 20 cms.	For all bolt lengths.
$\frac{1}{2}$ turn	$\frac{2}{3}$ turn	$\frac{3}{4}$ turn

N.B. Nut rotation is the rotation relative to bolt regardless of which element (nut or bolt) is turned.

Bolt length is measured from the underside of the head to the end of the shank.

In clause 6.5.3 the E.C.P. 2001 states following two different methods for the achievement of the turn of nut method :-

a) The parts to be joined are first brought into contact by making the bolts snug tight by a few impacts of an impact wrench. Following the initial step each nut is tightened one half turn.

b) The bolt is first tightened using a wrench until the several plies of the joint achieves a " snug fit " after which the nut is further turned by the amount :-

$$\alpha = 90^\circ + t + d \quad (2.2)$$

Where :

α = Rotation in degrees.

t = Total thickness of connected parts in mms.

d = Bolt diameter in mms.

2.5. The Bolt Pretension:

From the early 1950's the minimum required pretension was taken equal to the proof load. But it has been observed that when the turn of the nut is used, the proof load may not be achieved due to the relaxation factor and other parameters. We need 70 percent of the maximum tensile strength in order to create the pretension which can be computed as follows:

$$T = (0.7) \cdot F_{yb} \cdot A_s \quad (2.3)$$

where

T = pretension force.

F_{yb} = Yield stress of the bolt material (Table 2.2).

A_s = Mean of outer and inner areas of the threaded section = stress area as given in Table (2.5).

2.6. Friction-Type and Bearing-Type High Strength Bolted Connections:

E.C.P. (2001) allows for two categories of high strength bolts, the friction type and the bearing type. Bolts in either types are installed by the same process so that the same pretension is provided. The difference between the two types lies entirely in the factor of safety provided against slip.

The friction type is so-called because it has a higher factor of safety against slip and, therefore, it is most suitable when stress reversal or cyclical loading may occur. The higher factor of safety provides good fatigue resistance.

The bearing type allows slip condition to occur and then brings the bolt shank into contact with the side of the hole. The clearance Δd varies from 1 mm to 3 mm according to the corresponding bolt diameter as previously outlined in section 1.3.3.2.

2.7. Friction Type High Strength Bolted Connection:

2.7.1. Single High Strength Bolt Subjected to Concentric Shear:

Fig. 2.6. shows the forces induced in a typical high strength bolted connection. The pretension acting on the head and nut of the bolt produces a clamping force T which brings the two plates into contact. The clamping forces, in turn, produce a resistance to shear P_s which is a product of the clamping force T and the coefficient of friction μ between the two steel surfaces as given by E.C.P. (clause 6.5.2.2).

$$P_s = \mu T / V \quad (2.4)$$

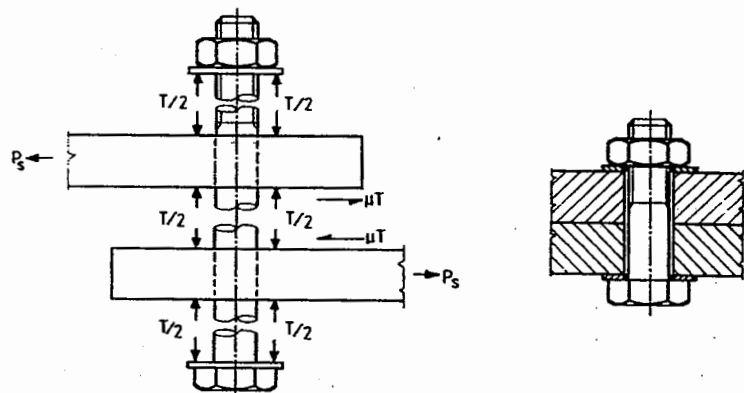


Fig. 2.6. Forces in a Typical Bolted Connection.

where :

P_s = The safe frictional load carried by one H.S. bolt with one plane of shear.

T = pretension in the bolt.

μ = friction coefficient.

V = safety factor with regard to slip.

The coefficient of friction μ depends on many factors such as the condition of the plates, including the presence of oil, paint, or mill scale. The slip coefficient is determined using the load at which the friction bond is definitely broken and the two surfaces slip with respect to each other by a relatively large amount. It is recommended to prepare the contact surfaces by an appropriate method such as:

- Sand blasting.
- Steel girt blasting.
- Flame Cleaning.

The E.C.P. 2001 in clause 6.5.2.2. sets the friction of coefficient μ which depends on the condition of preparation according to the following :-

$\mu = 0.5$ for class "A" surfaces.

$\mu = 0.4$ for class "B" surfaces.

$\mu = 0.3$ for class "C" surfaces.

The different classes depend on the following treatments :-

In class "A" :-

- Surfaces are blasted with shot or girt with any loose rusts removed , no painting.
- Surfaces are blasted with shot or girt and spray metallized with Aluminium.
- Surfaces are blasted with shot or girt and spray metallized with a Zinc based coating.

In class "B" :-

- Surfaces are blasted with shot or girt and painted with an alkali - zinc sillicate painting to produce a coating thickness of 50 - 80 μm .

In class "C" :-

- Surfaces are cleaned by wire brushing or flame cleaning, with loose rust removed.

Tables 2.4 illustrates the appropriate values of the safety factor V which is in accordance to the E.C.P. (2001) clause 6.5.2.3.

Table 2.4. Slip Factor of Safety

Case of loading	Values of V	
	Ordinary steel	Parts of bridges, Cranes and Crane girders with dynamic loads
Case I	1.25	1.6
Case II	1.10	1.4

Table 2.5 gives, according to the E.C.P. 2001, the values of T , A , A_s , and P_s for different diameters of high strength bolts.

Refer to the following examples:

Example 1.1: Connection of a truss member.

Example 1.2: Unloaded truss connection (separate lower chord members).

Example 1.3: Unloaded truss connection (continuous lower chord members).

Example 1.4: Loaded truss connection (separate upper chord members).

Example 1.5: Loaded truss connection (continuous upper chord members).

Example 1.6: Splice of truss member.

Example 1.7: Continuity of Beams.

2.7.2. High Strength Bolted Connections Subjected to Eccentric Shear:

This type of connection will be subjected to a shearing force Q and a torsional moment M_t . The bracket connection is a

Table 2.5
High Strength Bolts (Quality 10.9)*

Bolt Diameter (d) mm	Bolt Area (A) cm ²	Stress Area (A _s) cm ²	Pre-stressing (T) tons	Required Torque (M _a) Kg.m	Resistance of One Bolt per One Friction Surface (P _s) t					
					Ordinary Steel Work		Bridge & Cranes			
					St. 37 & 42-44 (u=0.4)		St. 50-55 (u=0.5)		St. 50-52 (u=0.5)	
					Case of Loading		Case of Loading		Case of Loading	
					I	II	I	II	I	II
M 12	1.13	0.84	5.29	12	1.69	2.01	2.11	2.52	1.32	1.65
M 16	2.01	1.57	9.89	31	3.16	3.37	3.95	4.71	2.47	3.09
M 20	3.14	2.45	15.43	62	4.93	5.9	6.17	7.36	3.85	4.82
M 22	3.80	3.03	19.08	84	6.10	7.27	7.63	9.1	4.77	5.96
M 24	4.52	3.53	22.23	107	7.11	8.45	8.89	10.6	5.55	6.94
M 27	5.73	4.59	28.91	157	9.25	11.03	11.56	13.78	7.22	8.55
M 30	7.06	5.61	35.34	213	11.30	13.48	14.13	16.86	8.83	10.46
M 36	10.18	8.17	51.47	372	16.47	19.64	20.58	24.55	12.86	15.24

$$P_s = \mu \cdot T / v$$

$$M_a = 0.2 d \cdot T$$

$$T = 0.7 \cdot F_{yb} \cdot A_s$$

* For H. S. B. of Quality 8.8, the above Values shall be reduced by 30%.

typical configuration of this type and the analysis will be similar to that given in Section 1.7 of the ordinary bolted connections.

The only difference will be in the limiting upper value of R_t .

Effect of shear Q :-

$$Q_b = Q/n$$

Effect of torsional moment M_t :-

$$F_n = M_t \cdot r_n / \sum r_i^2$$

Resultant Force :-

$$R_t = \sqrt{(Q/n + M_t X_n / \sum r_i^2)^2 + (M_t Y_n / \sum r_i^2)^2}$$

(2.5)

where :

n = the total number of H.S.B.

The following must be satisfied :-

$R_t \leq P_s$; P_s follows Equation 2.4 or Table 2.5.

Refer to the following examples:

1.9. Column-truss connections with framing angles.

1.10. Column-truss connections with a Tee-stub.

2.1. Bracket connection.

2.7.3. Single High Strength Bolt Subjected to Pure Tensile Force:

Consider two plates connected together by a single high strength bolt (Fig. 2.7.a.). As the nut is tightened by a specified torque in order to obtain the pretension force, the bolt will elongate and compress the plates as shown in Fig. 2.7.b. Assuming that the bolt and plates remain elastic

over the range considered, the force in each will be proportional to the change in length, ΔL , i.e.,

$$\left. \begin{aligned} T &= K_b \cdot \Delta L \\ C &= K_p \cdot \Delta L \end{aligned} \right\} \quad (a)$$

where T = pretension force in the bolt.

C = contraction force in the plates (Clamping force).

K_b, K_p = spring constants of bolt and plates respectively.

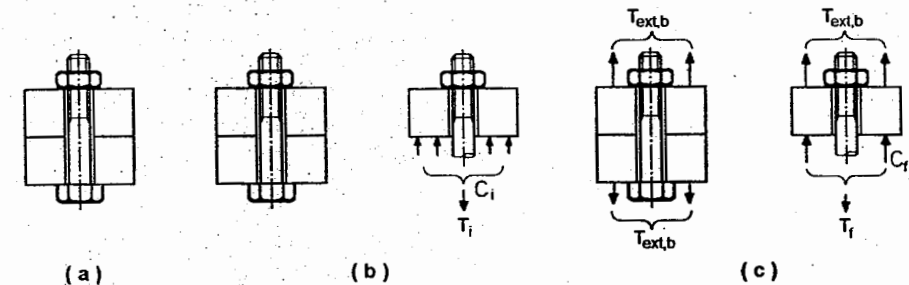


Fig.2.7.

For simplicity we can approximate K_b as being equal to $A_b E/L$ and K_p as equal to $A_p E/L$, where A_b and A_p are the effective acting areas of bolt and plates respectively. Relation (a) can be written in the following form:

$$\left. \begin{aligned} T &= (A_b E/L) \Delta L \\ C &= (-A_p E/L) \Delta L \end{aligned} \right\} \quad (b)$$

It has to be noted that the force T is concentrated in the bolt, while C is resisted by a larger effective area of the plate. For practical conditions where d is the diameter of bolts we can consider an effective plate area $(2.5 d \times 2.5 d)$ which gives:

$$A_p/A_b = (2.5 d \times 2.5 d) / (\frac{\pi d^2}{4} \times 0.8) \cong 10$$

here: $\frac{\pi d^2}{4} \times 0.8 \cong$ the stress bolt area.

From equilibrium of forces for the position (b) we can write:

$$T_i = C_i \quad (c)$$

Next applying an external pure tensile force $T_{ext,b}$ (Fig. 7.c.) both the bolt and the plates will expand. If the expansion is not greater than the initial contraction of the plates, some compression will remain and we can write the following:

$$T_f = C_f + T_{ext,b} \quad (d)$$

Up to the point at which the plate regain its original unstressed thickness, the expansion of the bolt and the plates must be the same. Calling this common elongation ΔL , and considering that a part of the external force $T_{ext,b}$ is taken by the bolt (i.e. ΔT) and the other part by the plates (i.e. ΔC), thus:

$$\left. \begin{aligned} \Delta T &= (A_b E/L) \Delta L \\ \Delta C &= (A_p E/L) \Delta L \end{aligned} \right\} \quad (e)$$

and $\Delta T/\Delta C = A_b/A_p$

Using the previous value of $A_p/A_b = 10$ in (e) we deduce that $\Delta T = 0.1 \Delta C$, or $\Delta C = 10 \Delta T$.

Prior separation, $T_f = T_i + \Delta T$ and $C_f = C_i - \Delta C$.

Since $C_i = T_i$ and $\Delta C = 10 \Delta T$ we can write $C_f = T_i - 10 \Delta T$.

When separation between plates occurs $C_f = 0$ or from the previous equation, $\Delta T = T_i/10$. The external force and bolt force at this point are then equal to $1.1 T_i$.

At higher loads the bolt force increases directly with the applied load to the point of failure.

The Egyptian code of practice 2001 (Clause 6.5.2.4.) allows a maximum permissible external tension in the direction of the axis not exceeding 0.8 of the pretension of the bolt. This gives the following relation:

$$T_{ext,b} \leq 0.8 T \quad (2.6)$$

In addition to the applied force $T_{ext,b}$, the bolts shall be proportioned to resist the additional induced prying force (P) as shown in figure 2.8.

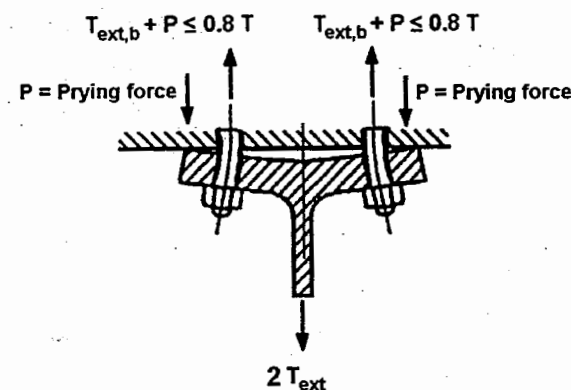


Fig. 2.8 Prying Force.

The prying force (P) depends on the relative stiffness and the geometrical configuration of the steel element composing the connection. The prying force (P) should be determined according to section 2.8 and hence the following check is to be satisfied :-

$$T_{\text{ext},b} + P \leq 0.8 T \quad (2.7)$$

2.7.4. High Strength Bolts Subjected to External Tension Combined with Shear:

Bolts in a connection may be subjected to external tension in addition to shear. The external applied tension in the direction of the bolt axis reduces the clamping action of the bolt (pretension in bolt).

The safety of high strength bolts must be checked against the following: (E.C.P. 2001 clause 6.5.2.5.).

$$Q_b \leq \mu (T - T_{\text{ext}}) / \gamma \quad (2.8)$$

$$T_{\text{ext},b} + P \leq 0.8 T \quad (\text{as } 2.6)$$

Refer to Example (2.2): Combined tension and shear.

2.7.5. High Strength Bolts Subjected to Tensile Stresses Resulting from Moment:

Method "A":

When the fasteners used are high strength bolts (10.9) or (8.8), an initial tension will result from the installation procedure. Assuming that the proof load in the bolts is not exceeded, the plates or sections making up the connection will be in complete contact with the column. Considering the pure moment connection in Fig. 2.9, the position of the neutral axis

will be at the mid-height of the connection as indicated.

The stress distributions depicted in Fig.2.9 are described in the following:

- (a) Initial compressive stress on contact area (F_{ci}).
- (b) stress on contact area due to applied moment ($F_{t,M} = F_{c,M}$).
- (c) Final stress on contact area is the superposition of cases (a) and (b).

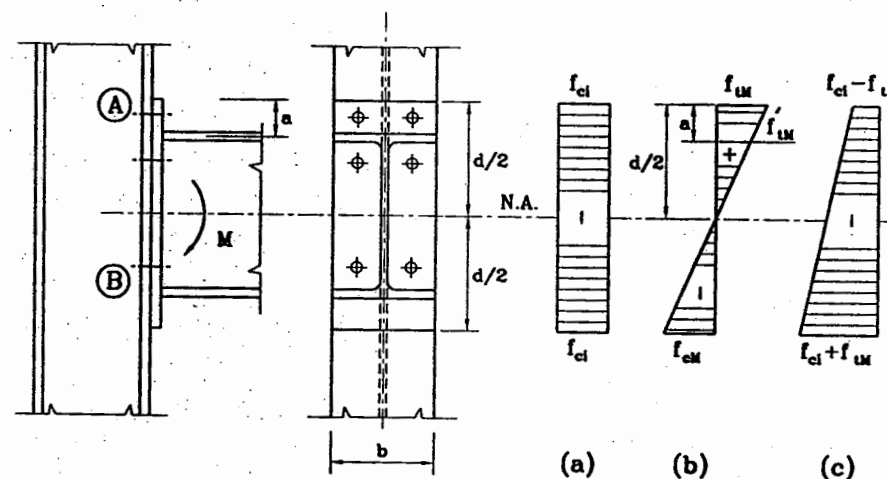


Fig. 2.9 Pure Moment Connections.

The initial compressive contact stress is assumed to be uniform over the area (bxd) and is equal to:

$$f_{ci} = \sum T_i / b.d. \quad (a)$$

Where $\sum T_i$ = The pretension load times the number of bolts.

The tensile stress $f_{t,M}$ at the top due to the applied moment is given by :-

$$f_{t,M} = \frac{M \cdot d/2}{I} = 6 M/bd^2 \quad (b)$$

For simplicity, the compressive stress on contact areas (f_{ci}) can be neglected. Naturally this is a conservative approach. The induced net tension load on the bolt will be equal to the product of the corresponding area of each fasteners time $f_{t,M}$.

The most stressed bolt is the farthest bolt from the neutral axis which can be checked as follows:

Calculate

$$f_{t,M} = 6M/bd^2$$

$$f'_{t,M} = f_{t,M} \cdot \left(\frac{d/2-a}{d/2}\right)$$

$$T_{ext,b,M} = \left(\frac{f_{t,M} + f'_{t,M}}{2}\right) a \cdot b \quad (2.9)$$

In addition to the resulting ($T_{ext,b,M}$) of Equ. 2.9 the bolts shall be proportioned to resist the additional induced prying force (P). This prying force (P) should be determined according to Section 2.8.

The induced maximum tensile Force ($T_{ext,b,M}$) due to the applied moment (M) in addition to prying force (P) that may occur, must not exceed the condition given by the E.C.P. 2001 in Clause 6.5.2.6 as follows :

$$T_{ext,b,M} + P \leq 0.8 T \quad (2.10)$$

Equation 2.10 is in accordance with clause 6.5.2.6. of the E.C.P. 2001.

Method "B":

It is to be well observed that the behavior of the end-plate moment connection is a function of the detailed steel components (i.e. thickness of end plate, diameter of bolts, spacing between bolts, ..(etc). Thus it is no longer possible to present definite methods of analysis.

In the following paragraphs another approximate method is presented. This method is based on the following assumptions:

- The clamping force between plates is acting only at the neighborhood of the washers and is of indeterminate distribution. Hence its effect can be neglected on the overall behavior of the connection.
- The compression force is transmitted via the lower beam flange to the end plate and then to the column flange. Hence the action line of this compressive force may be assumed to lie at mid-thickness of the lower compression flange.
- The resulting tension forces in the bolts are assumed to be proportional to their corresponding lever arms (y_i) as shown in Fig. 2.10.

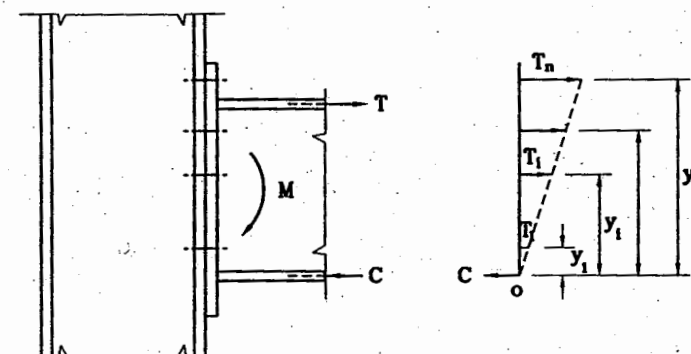


Fig. 2.10



Fig. 2.10 illustrates the configuration of an end-plate moment connection, which is in accordance with the previous assumptions.

From equilibrium of moments about point (o), we can compute the following:

$$M = \sum_{i=1}^n T_i Y_i \quad (a)$$

Where:

$$\frac{T_1}{Y_1} = \frac{T_2}{Y_2} = \dots = \frac{T_i}{Y_i} = \dots = \frac{T_n}{Y_n} \quad (b)$$

The distance $Y_1, Y_2, \dots, Y_i, \dots, Y_n$ are known from the geometry of the connection. If equation (b) is combined with (a) in terms of T_n , we get the following relation:

$$M = (T_n / Y_n) (\bar{Y}_1^2 + \bar{Y}_2^2 + \dots + \bar{Y}_i^2 + \dots + \bar{Y}_n^2)$$

i.e.

$$M = (T_n / Y_n) \cdot \sum_{i=1}^n Y_i^2 \quad (c)$$

From relation (c) the value of $T_n = T_{\text{ext},b,M}$ can be determined and then checked in accordance with the E.C.P. clause 6.5.2.6. i.e.

$$T_{\text{ext},b,M} + P \leq 0.8 T \quad (\text{as 2.10})$$

N.B. In chapter 6 the author proposes a more accurate analysis which lies between methods (A) and (B). The proposed method is based on a parametric study which induces the bolt pretension, the clamping forces, the geometrical configuration etc.



2.7.6. High Strength Bolts Subjected to Tensile Stresses resulting from Moment in addition to Shear:

The E.C.P. (2001) clause (6.5.2.6) requires the satisfaction of the following conditions:

$$T_{\text{ext},b,M} + P \leq 0.8 T \quad (\text{as 2.10})$$

$$Q_{\text{bolt}} \leq \mu T / \gamma \quad (\text{as 2.11})$$

In moment connections of the type shown in Fig 2.9 the loss of clamping forces in region "A" is always coupled with a corresponding increase in contact pressure in region "B". The clamping force inducing the frictional resistance the shear is to be checked using Equ. 2.11 .

2.7.7. High Strength Bolts Subjected to Tensile Stress resulting from Moment and External Tension Force combined with Shear:

When the connection is subjected to a shearing force Q , an external tension force T_{ext} , and a bending moment M , the induced forces in the most stressed bolt must satisfy the following (E.C.P. 2001 clause 6.5.2.7) .

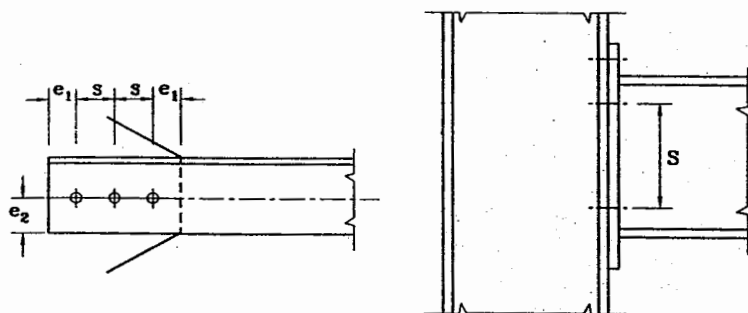
$$Q_b \leq \mu (T - T_{\text{ext},b}) / \gamma \quad (2.12)$$

Concerning the induced tensile force, equations 2.7 and 2.10 will be combined according to the following equation:

$$T_{\text{ext},b} + T_{\text{ext},b,M} + P \leq 0.8 T \quad (2.13)$$



2.7.8. Constructional Considerations:

Fig. 2.11 Pitch "S" and Edge distance " e_1 ", " e_2 "

The pitch "S" should preferably be taken $3d$ in the direction of the force as indicated in Fig. 2.11. On the other hand, the edge distance " e_1 " should not be less than $1.5d$, while the edge distance " e_2 " not less than $1.5d$. Fig. 2.11. depicts the case of an end moment connection. The pitch "S" between the tension and compression zones may have higher limits.

Refer to the following examples:

Example 1.9: Truss corner connection using connecting angle.

Example 1.10: Truss corner connection using a T section.

Example 2.3: Corner rigid frame connection (constant depth).

Example 2.4: Corner rigid frame connection (haunched).

Example 2.5: Apex rigid frame connection.

Example 2.6: Eave rigid connection (inclined end plates).

2.8. Bolted Beam-to-Column Connections and the Prying Force Effect:

The analyses of ordinary as well as high strength bolted connections carrying moments have been previously given in Sections 1.10 and 2.7.5 respectively. In those sections, the end plate moment connection was the only analyzed configuration,



while the other available configurations are given in the following sections.

In the following section the phenomenon of what we call PRYING FORCE is analyzed. Its effect on the behaviour of the end plate moment connections may be predominant.

PRYING FORCE EFFECT:

In order to clarify the PRYING FORCE phenomenon, consider a Tee-stub which is assumed to be attached to a rigid support by high strength bolts as shown in Fig. 2.12.a. If the bolts are tightened by the corresponding pretension (T), a balancing compression (C_1) equal to (T) is considered to be localized around the bolt (i.e. under the washer) as illustrated in Fig. 2.12.b.

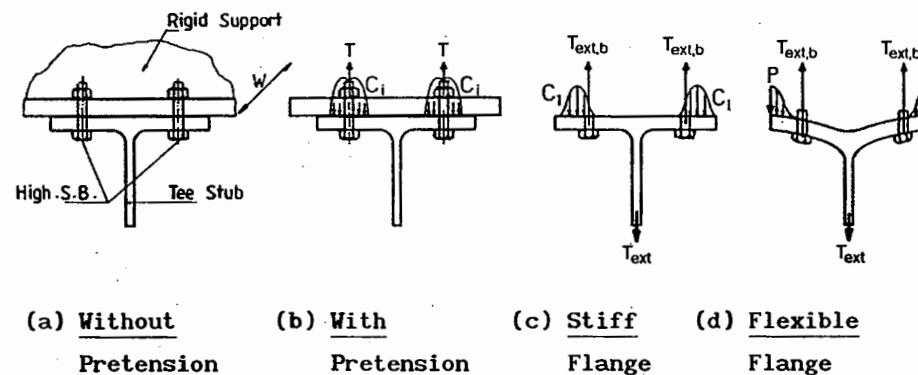


Fig. 2.12

Figs. 2.12.c and 2.12.d illustrate configurations of a stiff flange and a flexible one respectively. If an external force T_{ext} is applied to the Tee-stem, the flange will flex. Depending on the flange thickness, the two outer overhangings may press on the rigid support causing a force P which we call PRYING FORCE. This will be the situation of the

flexible flange as shown in Fig. 2.12.d. The prying action for a stiff flange is of less importance compared to a flexible one.

MATHEMATICAL MODEL:

In order to construct a mathematical model representing the prying force (P) as well as the applied force (T_{ext}), the following assumptions are considered:

(a) The prying force is represented by a concentrated load (P) acting at the outer tips of the flange.

(b) The contact pressure between the flange and the rigid support will remain around the bolts shaft after the application of the external force T_{ext} . (i.e. stage of no separation).

(c) The assumptions of the beam theory will be considered valid.

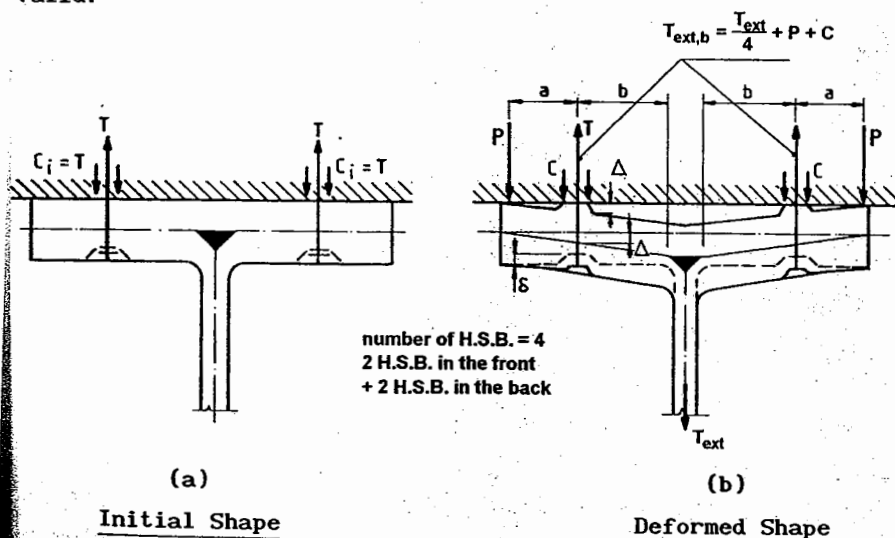


Fig. 2.13. Simplified Model

Fig. 2.13, illustrates the mathematical model as well as the induced forces, where Fig. 2.13.a shows a Tee-stub attached to a rigid support, with 4 bolts tightened to their pretension (T). After tightening, a compressive force (C_1) is induced between the surfaces in contact with ($T = C_1$).

Fig. 2.13.b, illustrates the deformed shape of the Tee-stub where the external force T_{ext} is applied. Denoting by Δ the deflection of the flange mid-thickness at the bolt centerline, one can deduce that the flange contraction will be subjected to a relief. An elongation Δ of the bolt shaft will correspond consequently to this stage of loading.

EQUILIBRIUM EQUATION:

Denote by P the created prying force per bolt at the tips of the flange at a distance "a" from the center of the bolt shaft and C the residual contact compression force (something less than T). From statics the induced total force in each bolt ($T_{ext,b}$) is :

$$T_{ext,b} = T_{ext} / 4 + P + C \quad (a)$$

(which will be consequently greater than (T)).

Knowing the geometric dimensions a and b as well as the pretension (T), three unknowns will remain in the model, namely Δ , P and C .

Deformation Equations:

The three unknowns Δ , P and C can be computed, if three deformation equations are established as follows:

- The first corresponds to the bolt elongation.

- The second corresponds to the flange expansion (relief of contraction) in the region surrounding the bolt shaft.

- The third corresponds to the deflection of the mid-thickness line of the Tee-stub flange at the center of the bolt shaft.

From bolt elongation we deduce:

$$\Delta = \frac{T_{\text{ext}}/4 - T}{K_b} \quad (b)$$

From flange expansion we get:

$$\Delta = \frac{T - C}{K_p} \quad (c)$$

where K_b , K_p are the bolt and flange spring constants or stiffnesses.

Using relation (a) in (c) we get:

$$\Delta = \frac{(T_{\text{ext}}/4 + P) - (T - C)}{K_b} \quad (d)$$

Considering a uniform distribution of forces along the breadth $2W$ of the Tee-stub flange, thus the flange can be assimilated to an elastic simple beam as shown in Fig. 2.14.

Applying one of the known methods of theory of structures, the deflection Δ of the elastic beam can be computed as shown in Fig., 2.14.c.

$$\text{i.e. } \Delta = \frac{a \cdot b^2}{E(wt^3/12)} \left\{ \frac{T_{\text{ext}}/4}{2} - \frac{a}{b} \left[\frac{1}{3} \left(\frac{a}{b} \right) + 1 \right] \cdot P \right\} \quad (e)$$

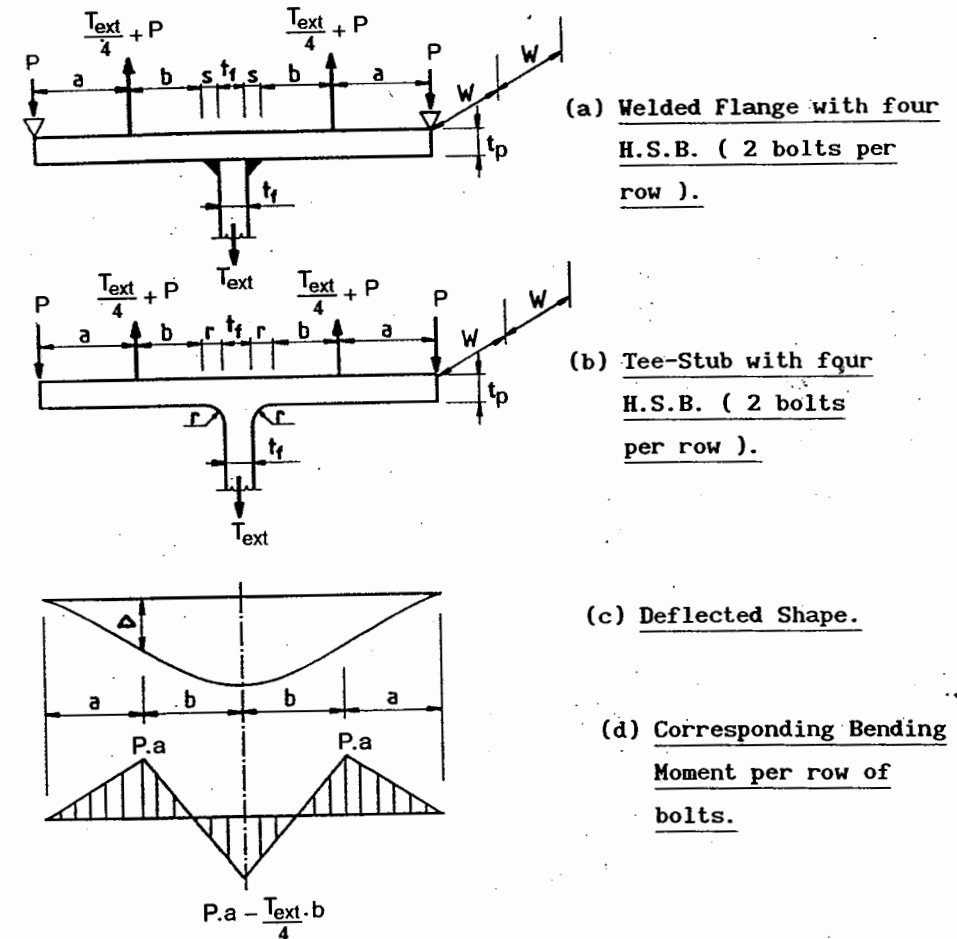


Fig. 2.14.

PRYING FORCE EQUATION:

Eliminating Δ and $(T - C)$ from equations (c), (d) and (e) and solving for P we get:

$$P = \left[\frac{\frac{1}{2} - \frac{E.w.t^3/12.a.b^2}{K_p + K_b}}{\frac{a}{b} \left(\frac{a}{3b} + 1 \right) + \frac{E.w.t^3/12.a.b^2}{K_p + K_b}} \right] \cdot T_{ext}/4 \quad (2.14)$$

Assuming (A_p) the area in contact between plates surrounding the bolt shaft 1.5 times the bolt shaft stress area A_s (i.e. $A_p = 1.5 A_s$) and replacing in Equ. 2.14 the values of K_b and K_p by the following:

$$K_b = \frac{A_s \cdot E_b}{t}; \quad K_p = \frac{1.5 A_s \cdot E_b}{t} \quad \text{we obtain:}$$

$$P = \left[\frac{\frac{1}{2} - \frac{w.t_p^4}{30 a.b^2.A_s}}{\frac{a}{b} \left(\frac{a}{3b} + 1 \right) + \frac{w.t_p^4}{30 a.b^2.A_s}} \right] \cdot T_{ext}/4 \quad (2.15)$$

Test results have shown some difference when compared to those computed by Equ. 2.15

In order to obtain correlation, an empirical modification is suggested by putting the value $(3/4 a)$ instead of (a) in the first term of the denominator. This modification is based on the fact that the prying force is rather distributed on an area rather than concentrated at the flange tip. This means that the force (P) will be somewhat displaced towards the vertical center line. Applying this modification, Equ. 2.15 becomes:

$$P = \left[\frac{\frac{1}{2} - \frac{w.t_p^4}{30 a.b^2.A_s}}{\left(\frac{3a}{4b} \right) \left(\frac{a}{4b} + 1 \right) + \frac{w.t_p^4}{30 a.b^2.A_s}} \right] \cdot T_{ext}/4 \quad (2.16)$$

If the second term in the numerator is greater than $1/2$ the result of the calculation is negative, this means that the prying force does not affect the bolt force. On the other hand, at higher loads, bolts will have elongated sufficiently to permit separation of plates at bolt line. This will cause complete relief of the local compression force C . In order to deduce the corresponding equation for a separation stage one can replace $C = 0$ in Equations (c) and (d), thus Equ. 2.16 becomes :

$$P = \left[\frac{\frac{1}{2} - \frac{w.t_p^4}{12 a.b^2.A_s} \left(1 - \frac{T}{T_{ext}/4} \right)}{\left(\frac{3a}{4b} \right) \left(\frac{a}{3b} + 1 \right) + \frac{w.t_p^4}{12 a.b^2.A_s}} \right] \cdot T_{ext}/4 \quad (2.17)$$

where the compressive stiffness of the flange K_p has no longer any influence on the prying force.

Refer to Example 2.5 and 2.6..

2.9 Tee-Stub Bolted Connections:

The Tee-stub connection is a common configuration to transmit the beam moment to the supporting column. Its behaviour and analysis is a straight forward application to the prying

force phenomenon of Section 2.8.

Generally shop bolts are used to connect the Tee-stub stem to the beam flange and field bolts to attach the Tee-stub flange to the supporting column (see Fig. 2.15.a). The Tee-stub stem may be shop-welded to the beam flange and the Tee-stub flange field bolted to the column flange. For this last configuration, refer to Fig. 2.15.a where the Tee-stub stem will behave as a connecting plate.

The following analysis is based on the equations of Section 2.8.1 for high strength bolts. The use of ordinary bolts or rivets applying the behaviour outlined in Section 2.8.1 is uncertain due to the absence of representative test data.

(a) Group of Bolts (1):

The beam moment M is transmitted to the upper and lower Tee-stub stems by two equal and opposite forces ($T_b = C_b = M/d_b$) via the group of bolts (1) carrying each a shearing force ($Q_b = T_b/n_1$), where n_1 is the total number of bolts of group (1). One must check that the induced single shear (Q_b) does not exceed the frictional resistance (P_s).

(b) Tee-Stub Stem:

Generally the Tee-stub section is obtained by cutting the required length from a B.F.I section. Consequently the Tee stem and the Tee flange thicknesses are respectively those of the web and the flange of the HEA OR HEB. The Tee stem will be subjected to tensile stresses, calculations are based on the gross area if high strength bolts are utilized.

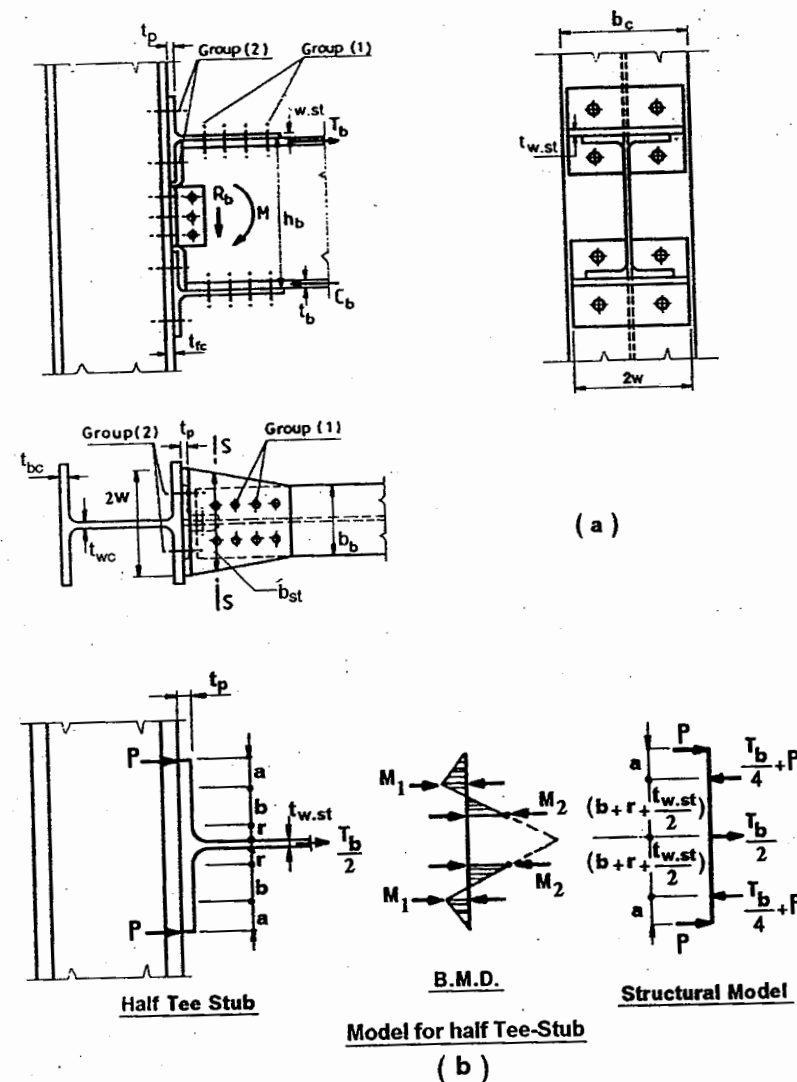


Fig. 2.15

$$f_t = \frac{T}{b_{st} \cdot t_{w.st}}$$

(f)

Where b_{st} = the breadth of the Tee stem at a Section (s-s) passing through the nearest bolt row to the column.

$t_{w.st}$ = the Tee stem thickness.

(c) Group of Bolts (2):

The force T_b is then transmitted to the group of bolts (2) via the Tee-stub flange. In addition to the induced external tensile force (T_b/n_2), a prying force P calculated as in Section 2.8 is to be added. (see Fig. 2.15.b).

$$\text{Force on bolts of group (2)} = \frac{T_b}{n_2} + P \quad (g)$$

Where n_2 = the total number of bolts of group (2).

It is to be noticed that the effect of the prying force (P) can increase considerably the external tensile force induced in bolts(2). Since the main goal for this type of connections is to prevent any separation between the Tee-stub flange and the supporting column flange, the E.C.P. 2001 checks the separation according to the following:

$$\frac{T_b}{n_2} + P \leq 0.8 T \quad (h)$$

Where T = the pretension on bolt.

(d) The Tee-Stub flange:

The bending moments (M_1) in the Tee-stub flange at the bolt line and (M_2) at the outer edge of the fillet are to be calculated. The prying force (P) is assumed to act at a distance (a). The induced bending stresses are computed as follows:

$$f_t = \frac{6(\text{greater of } M_1, M_2)}{w.t_p^2} \quad 2.18$$

$$\left. \begin{aligned} \text{With } M_1 &= P.a \\ M_2 &= P.a - T_{ext,b} \cdot b \end{aligned} \right\} \quad 2.19$$

where $T_{ext,b} = T_b/4$

(e) The Beam Compression Flange:

The compression beam flange will induce bearing stresses on the column flange which will tend to decrease the initial pretension. Thus no new design problems are revealed. It will be of good practice to consider the same dimensioning for the compression region as for the tension one.

(f) Crippling of Column Web: (Refer to Chapter 3 Section 3.9)

The concentration of stresses on the column flange are to be checked against the phenomena of the web-column crippling according to the relation of Section 3.9.5. Thus referring to Fig. (2.15.c, d) one can deduce:

$$t_{w.c} \geq \frac{b_b \cdot t_b}{t_{w.st} + 2.t_p + 5K} \quad (i)$$

Where : $K = 2 t_{fc}$

(g) Bending of Column Flange: (Refer to Chapter 3 Section 3.9)

At the beam flange tension side, the column flange may bent if its thickness is not sufficient. Due to the lack of accurate information in the existing literature regarding the analysis, the following relation may be utilized:

$$t_{fc} \geq 0.4 \sqrt{b_b \cdot t_b} \quad (j)$$

(h) Vertical Shear Resistance:

The beam reaction R_b can be transmitted to the column flange using one of the configurations of Chapter (4).

Refer to Example 2.6.

2.10. End Plate Moment Connections:

Because of the large number of bolts required, together with the necessity to get more rigid connections, the Tee-stub configuration has fallen into disuse.

Most of the common rigid bolted connections make use of end plates namely: the end plate moment connections. Fig. 2.16, illustrates the wide variety of their applications:

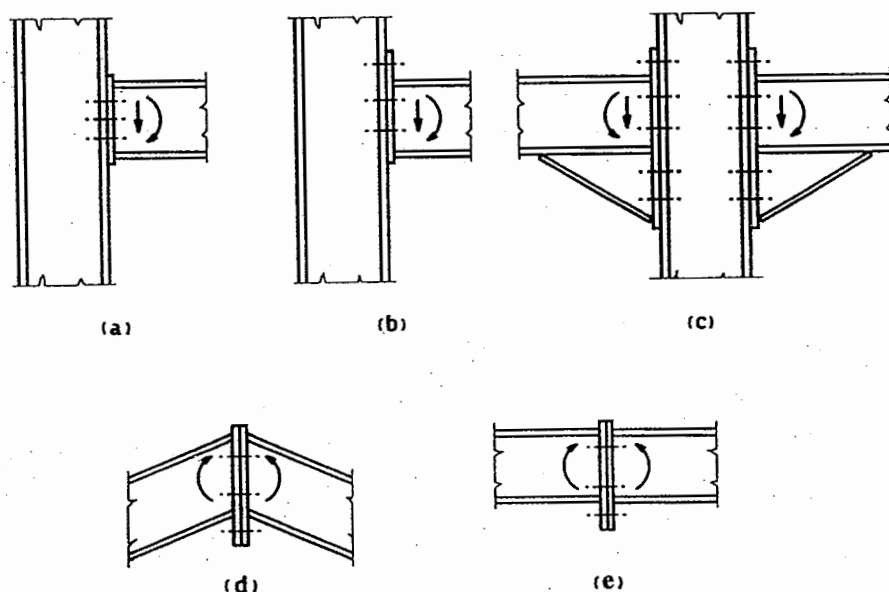


Fig. 2.16. End Plate Moment Connections

- (a) the flushed end plate.
- (b) the extended end plate.
- (c) the haunched end plate.
- (d) the apex end plate.
- (e) the splice end plate.

Types (a),(b),(c) and (d) are analyzed in the following paragraphs while type (e) will be presented in Chapter 7.

It is to be well observed that the behaviour of the end plate moment connection is a function of the detailed steel components (i.e. thickness of end plate, diameter of bolts, spacing between bolts, ... etc). Thus it is no longer possible to present definite methods of analysis. Different laboratories all over the world are still studying this type of connections. The complexity is due to the different steel plate elements comprising the connection which are subjected to concentration of stresses of all types.

In the following paragraphs the analysis is based on semi-empirical methods which allow a safe design for the bolts and the end plate thickness.

2.10.1. The Flushed End Plate Moment Connection:

For a connection having all the bolts located between the two flanges of the beam we name it flushed end plate or short end plate connection.

In order to compute the bolt external tensile force induced by the beam moment one can proceed using one of the two methods previously outlined in Section 2.7.5.

The induced prying force can be neglected. Its effect for the flushed type is of minor importance.

It is to be observed that this type of configuration is not very efficient structurally, the cancellation of the row of bolts outside the tension flange will increase considerably the bolt forces, the lever arm being reduced. Thus it is not recommended to use this type of connection configuration unless the end moment is significantly less than the beam mid-span moment.

For the determination of the end plate thickness, it will be assumed that the critical section for bending is located at the nearest bolt row w.r.t to the tension flange (see Fig. 2.17). The moment acting on this section is taken as the beam tension force $T_b = \frac{M}{d_b}$ time the distance from the center of the beam tension flange to bolt center line i.e. $(e + \frac{1}{2} t_b)$. The induced bending stresses will follow the relation:

$$f_b = \frac{T_b \cdot (e + 1/2 t_b)}{2 \cdot w \cdot t^2 / 6} \quad (1)$$

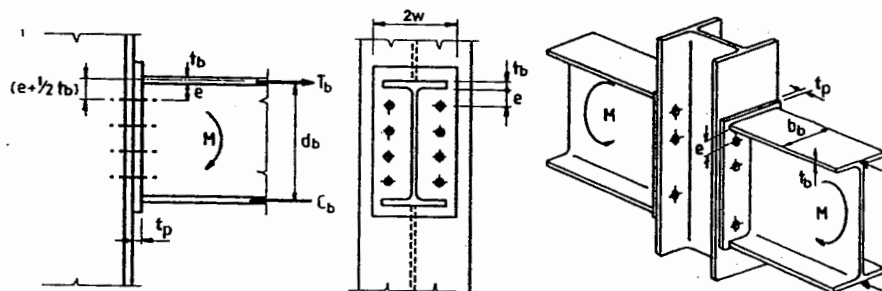


Fig. 2.17

Hence, using the allowable bending stress the corresponding end plate thickness is computed.

Refer to example 2.7.

2.10.2. The Extended End Plate Connection:

End plate connection where some of the bolts are situated outside the tension beam flange is named the extended end plate moment connection. This type has the advantage that the bending lever arm of the bolts is increased and thus decreasing the external tensile forces carried on the bolts. Also the bending in the end plate is reduced. It is to be noticed that the ideal configuration is to locate the bolts at the immediate neighborhood of the beam tension flange.

The behavior at the neighborhood of the beam tension flange may be idealized as the Tee-stub connection analyzed in Section 2.8.2 as shown in Fig. 2.18.a by the dotted rectangle abcd. Similarly as in the Tee-stub connection Equ. 2.16 is utilized to take the prying force into account (see Fig. 2.18.d). Applying one of the methods previously outlined in Section 2.7.5, one can determine if only two rows of bolts at the neighborhood of the beam tension flange are sufficient to overcome the beam moment (see Fig. 2.18.b). Otherwise an additional row or rows are to be provided. It is essential to add a row of bolts near the beam compression flange in order to prevent this part from springing. It is of common practice to locate the bolts at minimum spacing. Safety against crippling of the column web is to be ensured, otherwise stiffeners are to be utilized as will be given in Chapter 3 (Section 3.9).

In order to facilitate the analysis one can proceed according to the following steps: (see Fig. 2.18)

(a) Calculate the beam tension force ($T_b = M/d_b$)

(b) As a first estimate assume two rows of bolts located at the neighborhood of the tension flange. Use two bolts for each row.

(c) Compute the external tension force per bolt assuming equal share (i.e. $T_{\text{ext},b,M} = T_b/4$).

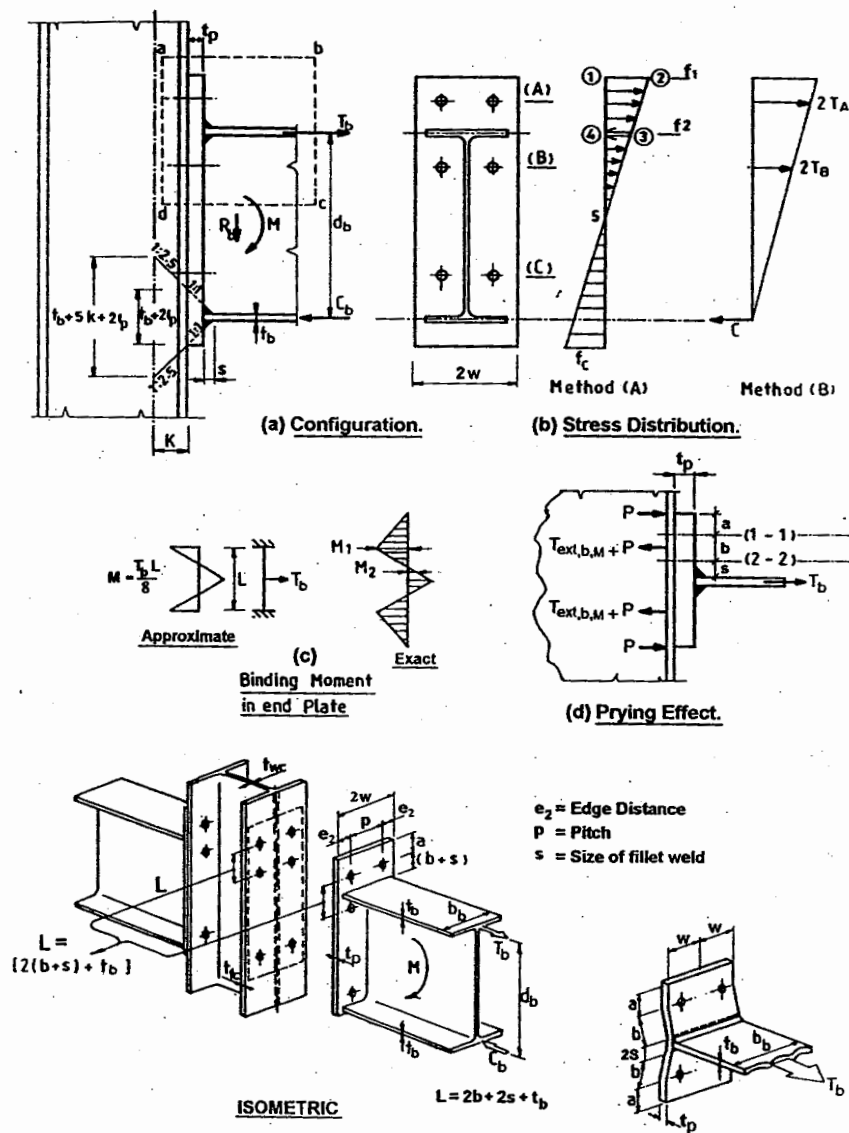


Fig. 2.18

(d) Calculate the required safe pretension ($T = \frac{T_b}{0.8 \times 4}$).
 Make use of the E.C.P. 2001 condition:

$$T_{\text{ext},b,M} \leq 0.8 T$$

(e) Use Table 2.5. to choose the appropriate bolt diameter. For building structures use diameters till 22 mms, otherwise use an additional row in the tension zone.

(f) Compute an approximate end plate thickness using the model shown in Fig. 2.18.c where the effect of the prying force is neglected. The end plate is assumed to be a beam with fixed supports at the bolts center lines. Hence the induced bending moment is $M = T_b L/8$ where L is the distance between the two upper bolts rows ($L = 2b + 2s + t_b$). Thus $t_p = \sqrt{\frac{6.M}{2.w.F_b}}$ ($F_b = 0.58F_y$ = the allowable bending stress. The following equation can be utilised:

$$t_p = 0.6123 \sqrt{\frac{M.(2b + 2s + t_b)}{d_b.w.F_b}} \quad 2.20$$

Where :-

M = Beam Moment.

b = Internal distance of bolts with respect to the Tee-Stub web or the beam flange.

w = Breadth of end plate or Tee-Stub flange regarding one column of high strength bolts.

= Half breadth of end plate or half breadth of Tee-Stub flange for the case of two columns of H.S.B.

s = Fillet weld size.

t_b = Flange beam thickness ; $d_b = h - t_b$.

h = Height of beam cross section.

t_f & d_b = Flange beam thickness and depth.

F_b = Allowable bending stress of end plate material.

(g) Compute the induced prying force using Equation 2.16 where the end plate thickness corresponds to step (f).

$$(T_{ext} / 4 = T_b / 4)$$

(h) Proceed an exact analysis using one of the methods (A) or (B) given in Section 2.7.5, to determine the exact stress distribution. Then calculate the induced external bolt tensile force ($T_{ext,b,M}$). Fig. 2.18.b shows the corresponding distribution of forces and stresses.

(i) Check if there is any separation between the column flange and the end plate using the following relation:

$$T_{ext,b,M} + P \leq 0.8 T_i \quad (m)$$

where $T_{ext,b,M}$ is the exact induced external tension force on the most stressed bolt using either Method (A) or (B)

P = The corresponding prying force.

T = The bolt pretension.

(j) Compute the exact induced bending moment in the end plate choose the greater of M_1 and M_2 of Fig. 2.18.c.

$$\text{Where : } \left. \begin{array}{l} M_1 = P \cdot a \\ M_2 = P \cdot a - T_{ext,b,M} \cdot b \end{array} \right\} \quad (\text{as 2.19})$$

Hence calculate the induced bending stress choosing $M =$ Greater of M_1 and M_2 and hence :-

$$t_p = \sqrt{\frac{6 \cdot (M_1 \text{ or } M_2)}{w \cdot F_b}} \quad (\text{as 2.18})$$

Where :

w = Half breadth of end plate = half breadth of Tee-Stub flange. (case of two columns of bolts)

If the bolts can withstand the beam moment without separation and the end plate thickness is sufficient, stop the design. Otherwise repeat the process increasing the end plate thickness. Consequently the prying force P will be affected, thus the bolts are to be rechecked.

Refer to Examples 2.8, 2.9, 2.10 and 2.11.

2.10.3. The Haunched End Plate Connection:

If the beam end moment becomes of great importance, consequently the required number of bolts increases considerably and the maximum allowed fillet weld thickness will be insufficient.

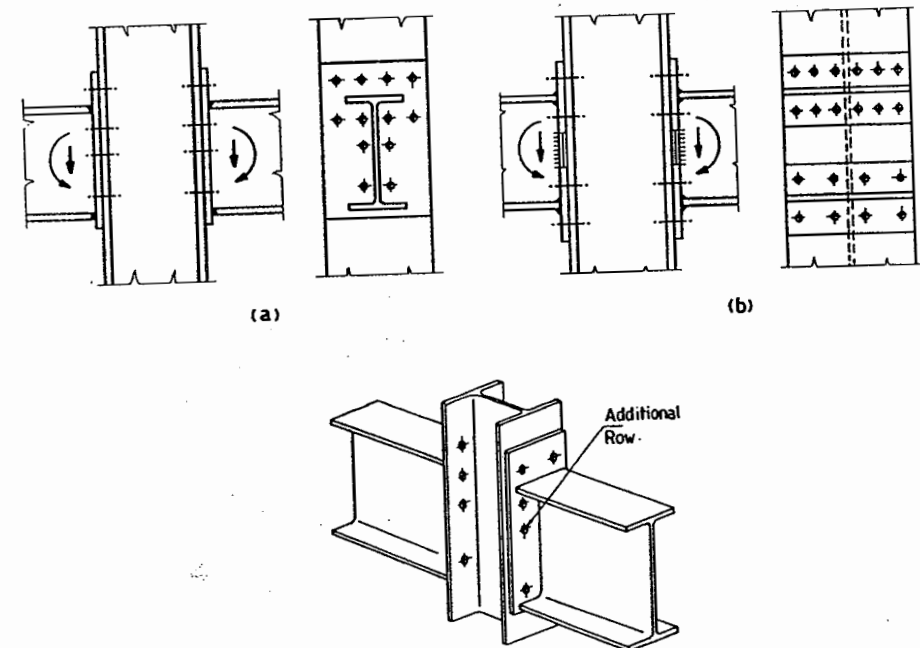


Fig. 2.19

Hence one can think first to retain the extended end plate of Section 2.8.2 using a greater number of bolts and groove (butt) welds as shown in Fig. 2.19.a. A second alternative configuration is illustrated in Fig. 2.19.b where the Tee-stub is designed to transmit the end beam moment to the column flange. The number of bolts is consequently increased. A bearing end plate is welded to the beam web to transmit the beam vertical reaction to the column flange via the bolts located in the compression zone.

The third alternative is to provide the connection with a haunch as illustrated in Fig. 2.20. The question now is what is the size of the haunch to be provided?. Choose if possible the haunch depth as the beam depth; the fabrication cost is minimized as the haunch section will be that of the beam. This will allow to cut the haunch from the same rolling section as the beam. Fillet welds will attach the haunch to the column flange. It is of good practice to provide a Slope 1:2 to 1:5 or to choose the haunch length one tenth the span.

The notations regarding Fig. 2.20 are as follows:

- (1) Fillet welds connecting haunch web to beam flange.
- (2) , (3), and (4) Fillet welds connecting beam and haunch flanges to the end plate.
- (5) Fillet welds connecting the haunch web to the end plate.
- (6) Fillet welds connecting the haunch flange to the beam flange.
- (7) Tension bolts.
- (8) End plate.
- (9) stiffener.

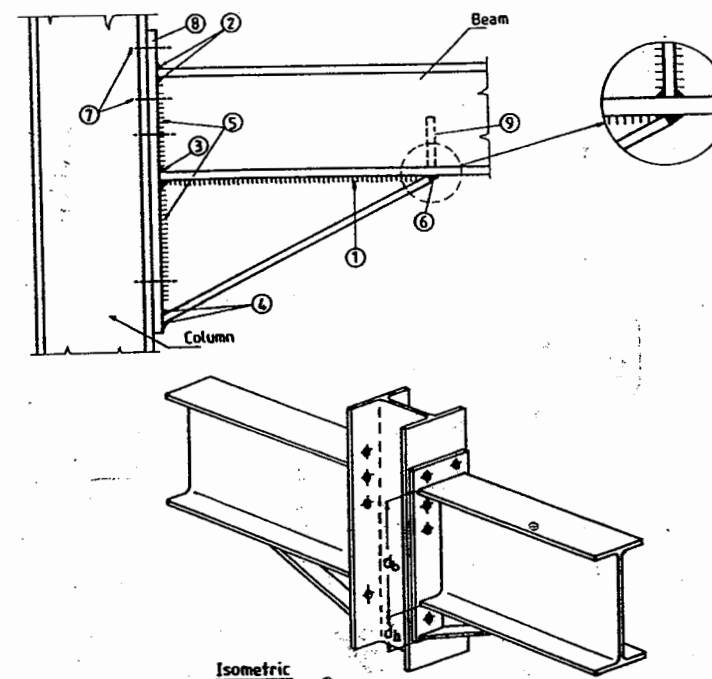
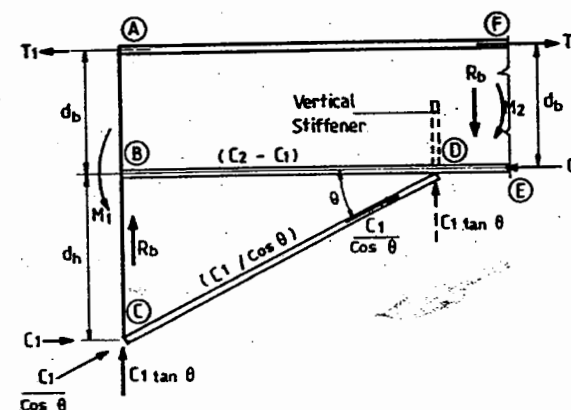


Fig. 2.20. Haunched End Plate Connection

For the analysis and the design consider the haunch ABCDEF of Fig. 2.21 subjected to the following forces:



Equilibrium
of Forces

Fig. 2.21

- At joints (A) and (C) the two horizontal forces T_1 and C_1 are equal and opposite in direction, where:

$$T_1 = C_1 = \frac{M_1}{d_b + d_h}$$

- At joints (F) and (E) the two horizontal forces T_2 and C_2 are equal and opposite in direction, where:

$$T_2 = C_2 = \frac{M_2}{d_b}$$

- At (C) the horizontal force C_1 can be replaced by a vertical component $C_1 \tan \theta$ and an inclined one $C_1 / \cos \theta$.

- From equilibrium conditions at joint (D) at the abrupt change of cross section, the beam flange portion BD will be subjected to a compression force equal to $(C_2 - C_1)$. While the vertical component $C_1 \tan \theta$ will tend to compress the joint D, thus requiring a vertical stiffener (shown dotted in Fig. 2.21).

Once the diagram of forces of Fig. 2.21 has been determined, the design of the different structural steel components can be accomplished.

In order to avoid repetition Refer to Example 2.7.

N.B. :-

Table 2.6 summarizes the different types of high strength bolted connections with the limiting stresses as specified by E.C.P. 2001.

Table 2.6 High Strength Bolted Connections (Friction Type).

Type	Configuration	Straining Actions	Allowable Stresses (E.C.P. 2001)
1-Concentric Shear Connection.		$Q = F$ $Q_b = F/n$	$Q_b \leq P_s$ $Q_b \leq \sqrt{A} T / 8$
2-Eccentric Shear Connection.		$Q = P$ $M_t = P.e$ $R_Q = P/n$ $R_{Mt} = \frac{M_t \cdot \sum r_i^2}{\sum r_i^2}$	$R_t = \text{Vectorial Summation of } R_{b, Mt} \text{ and } R_{b, Q}$ $R_t \leq P_s$
3-External Pure Tension.		$T_{ext} = F$ $T_{ext, b} = F/n$	$T_{ext, b} \leq 0.8T + P$
4-Combined Shear & Tension.		$Q = F \cdot \cos \theta$ $T_{ext} = F \cdot \sin \theta$ $Q_b = F \cdot \cos \theta / n$ $T_{ext, b} = \frac{F \cdot \sin \theta}{n}$	$Q_b \leq P_s \sqrt{(T - T_{ext, b}) / 8}$ $T_{ext, b} \leq 0.8T + P$
5-Combined Shear & Bending Moment.		$M = M$ $Q = Q$ $Q_b = Q/n$ $M - T_{ext, b, M}$	$Q_b \leq P_s \leq \sqrt{A} T / 8$ $T_{ext, b, M} \leq 0.8T + P$
6-Combined Shear, Tension & Bending Moment.		$N = N$ $Q = Q$ $T_{ext} = N$ $Q_b = Q/n$ $T_{ext, b} = N/n$ $M - T_{ext, b, M}$	$Q_b \leq P_s (1 - \frac{T_{ext, b}}{T})$ $\frac{T_{ext, b}}{0.8T} + \frac{T_{ext, b, M}}{0.8T} \leq 1$

Example (2.1):

Design the bracket connection previously outlined in chapter (1) (example 1.11). Using high strength bolts. (Applied loads are static).

Solution(A) Case of Loading (I)Without Braking Force

The design of the triangular gusset plates transmitting the applied loads from the crane track girder to the column flange is as previously given in example (1.11) of chapter (1).

1. High Strength Bolts Connecting Bracket to Column:

Use H.S.B (10.9); two rows of 4 bolts each vertical pitch = 135 mms; Horizontal pitch = 150 mms.

1.1. Straining Actions:

$$Q_y = 21.0 \text{ tons}$$

The lateral shock $H = 1.6$ tons is directly transmitted to the column.

$$M_t = 21 \times (44 + 15) = 1239 \text{ t.cms}$$

1.2. Properties of Area and Check of Stresses:

The friction between the triangular gusset plate and the column flange is neglected and hence the analysis will follow exactly that when using ordinary bolts.

Choose the same geometrical configuration as previously given in example (1.11) and from previous results we have :-

$$Q_{b\downarrow} = 1.313t; Q_{b(M_t)\downarrow} = 2.045t; Q_{(M_t)\rightarrow} = 5.520t$$

$$Q_{t\downarrow} = \sqrt{(1.313+2.045)^2 + (5.52)^2} = 6.46 \text{ tons}$$



Using table 2.5 choose 8 H.S.B (10.9) $d = 22 \text{ ms}$.

Where: $P_s = 6.1 \text{ tons} < 6.46 \text{ tons}$. (Unsafe)

The difference being less than 5% the design can be allowed where the friction between plates has been neglected which will increase the resistance.

(B) Case of Loading (II) with Braking Force2. High Strength Bolts Connecting Bracket to Column:

Use H.S.B.'s: Try two rows of 4 bolts at 135 mms vertical pitch and 150 mms horizontal pitch as shown in figure.

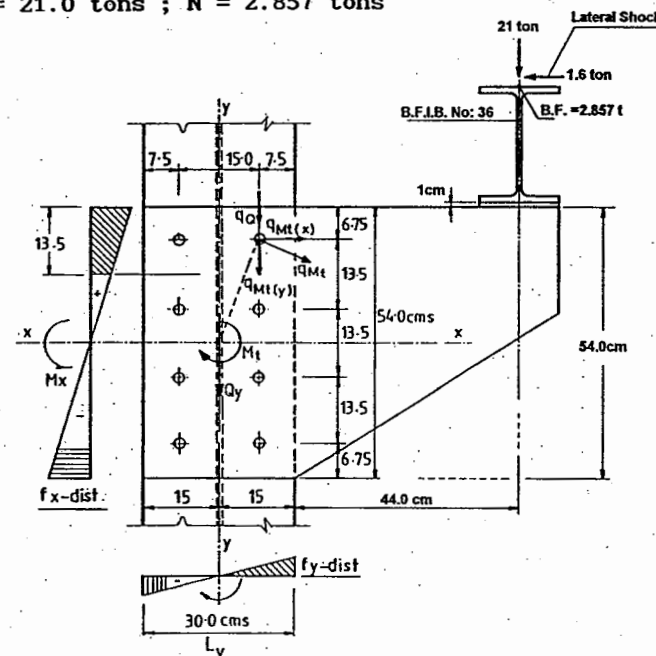
2.1. Straining Actions:

$$M_x = 2.857 \times (36 + 1 + 27) = 182.84 \text{ t.cm.}$$

$$M_y = 2.857 \times (44 + 15) = 168.56 \text{ t.cm.}$$

$$M_t = 21 \times (44 + 15) = 1239 \text{ t.cm}$$

$$Q_y = 21.0 \text{ tons}; N = 2.857 \text{ tons}$$



2.2. Properties of Area:

$$\left. \begin{aligned} I_x &= 2 \times 54^3 \times 30 / 12 = 787320 \text{ cm}^4 \\ I_y &= 2 \times 30^3 \times 54 / 12 = 243000 \text{ cm}^4 \\ I_p &= I_x + I_y = 1030320 \text{ cm}^4 \end{aligned} \right\} \begin{array}{l} \text{(clamping between} \\ \text{bracket and the} \\ \text{column flange)} \end{array}$$

2.3. Stresses:

$$T_1 \text{ (due to } M_x) = \frac{182.84 \times 20.25}{787320} \times 15 \times 13.50 = 0.95 \text{ ton}$$

$$T_2 \text{ (due to } M_y) = \frac{168.56 \times 7.50}{243000} \times 15 \times 13.50 = 1.05 \text{ ton.}$$

$$T_3 \text{ (due to } N) = 2.857 / 8 = 0.357 \text{ ton}$$

$$T_{\text{total}} = 0.95 + 1.05 + 0.357 = 2.357 \text{ tons}$$

$$T_{\text{ext.}N} = 0.357 \text{ ton}$$

$$Q_Q^\downarrow = 21/16 = 1.313 \text{ ton}$$

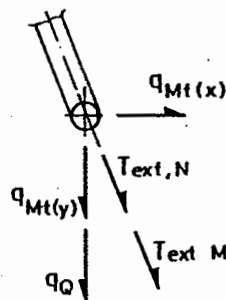
$$Q_{Mt}^\downarrow = \frac{1239 \times 7.50}{2 \times 4 \times (10.09^2 + 21.594^2)} = 2.045 \text{ tons}$$

$$Q_{Mt}^{\rightarrow} = \frac{1239 \times 20.25}{2 \times 4 \times (10.09^2 + 21.594^2)} = 5.52 \text{ tons}$$

$$Q_{\text{total}} = \sqrt{(5.52)^2 + (2.045 + 1.313)^2} = 6.46 \text{ tons}$$

$$Q_b = 6.46 \text{ tons.} \quad \therefore Q_{b, \text{all}} > P_s \left(1 - \frac{T_{\text{ext.}N}}{T}\right)$$

$$Q_{b, \text{all}} = \left[\frac{\mu T}{\gamma} \left(1 - \frac{T_{\text{ext.}N}}{T}\right) \right] \times 1.20$$



$$\therefore Q_{b, \text{all}} = \left[\frac{0.4 \times T}{1.25} \left(1 - \frac{0.357}{T}\right) \right] \times 1.20$$

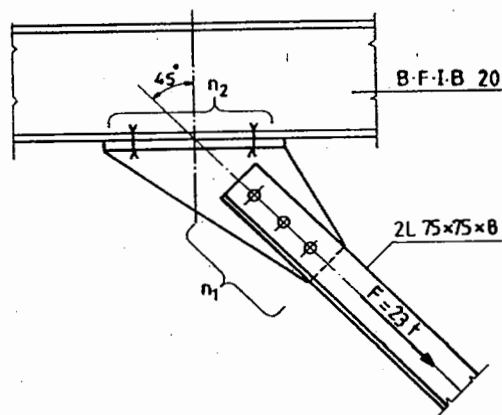
Use 16 bolts M (22) H.S.B. Type (10.9) 8 bolts on each side of the column. (T = 19.08 tons)

$$Q_{b, \text{all}} = \left[6.1 \times \left(1 - \frac{0.357}{19.08}\right) \right] \times 1.20$$

$$= 7.18 \text{ tons} > Q_b \quad \therefore \text{safe}$$

Example (2.2):

Design the shown connection using high-strength bolts of friction type ($d = 20$ mm) type (10.9); $P_s = 4.93$ t; $T = 15.43$ t.



Solution

For* the Group of Bolts (n_1):

$$n_1 = \frac{23}{2 \times 4.93} = 2.30 \text{ bolts} , \quad \text{Take } n_1 = 3.0 \text{ bolts}$$

For the Group of Bolts (n_2):

$$Q = 23 \cos 45^\circ = 16.26 \text{ tons}; \quad T = 23 \sin 45^\circ = 16.26 \text{ tons}$$

$$T_{\text{ext,b}} = \frac{16.26}{4} = 4.065 \text{ tons} < 0.80 \times 15.43 \quad \text{Safe}$$

$$Q_{\text{bolt}} = \frac{16.26}{4} = 4.065 \text{ tons.}$$

$$Q_{b,all} = \frac{\mu (T - T_{ext})}{\gamma} = \frac{0.4 (15.43 - 4.065)}{1.25} = 3.60 \text{ tons.}$$

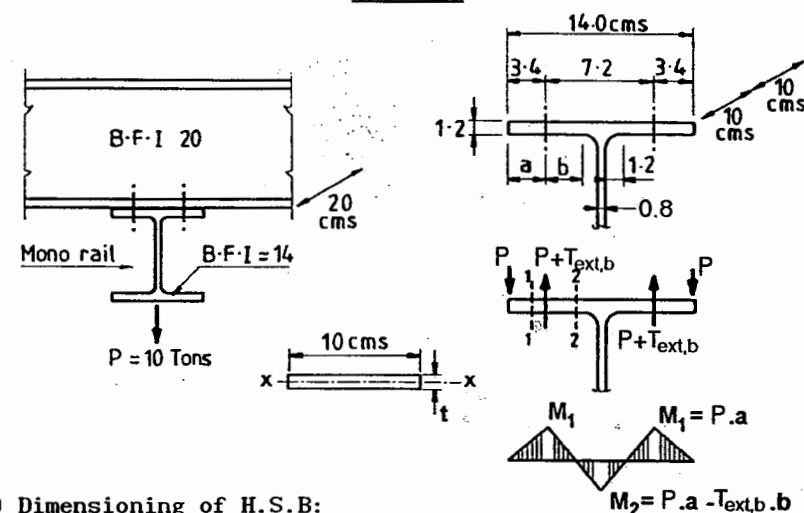
< 4.065 tons

Hence use 6 bolts M 20 it will be safe.

Example (2.3):

Design the connection shown in the Fig. below. Compute the prying force according to Section 2.8.1.

Solution



(1) Dimensioning of H.S.B:

The tension per bolt

$$T_{\text{ext},b} = 10/4 = 2.5 \text{ tons. (External tension)}$$

Having the condition: $T_{\text{ext. b}} + P \leq 0.8 T$

Assume as a first approximation 0.7 instead of 0.8

$$\text{Hence: } T_i = \frac{T_{\text{ext, b}}}{0.7} = \frac{2.5}{0.7} = 4.16 \text{ tons.}$$

From table 2.5 choose 4 H.S.B (10.9) M_{12} ($T = 5.29$ t).

For the computation of the prying force use Equation 2.16

$$a = 3.4 \text{ cms} \quad ; \quad 2b = 14 - 0.8 - 2 \times 3.4 - 2 \times 1.2 = 4 \text{ cms.}$$

web thickness
a
fillet dimension

$b = 2 \text{ cms}$; $w = 20/2 = 10 \text{ cms}$ (as we have 2bolts per row)

$t_p = 1.2 \text{ cms}$; $A_s = 0.84 \text{ cm}^2$.

$$P = \left[\frac{\frac{1}{2} - \frac{wtp^4}{30ab^2A_s}}{\left(\frac{3a}{4b}\right)\left(\frac{a}{4b} + 1\right) + \frac{wtp^4}{30ab^2A_s}} \right] T_{\text{ext}, b}$$

$$P = 0.245 \times 2.5 = 0.61 \text{ tons.}$$

Hence the total force carried by each bolt is:

$$2.5 + 0.61 = 3.11 \text{ tons} < 0.8 \times 5.29 < 4.23 \text{ tons. o.k.}$$

(2) Check of Thickness of Monorail Flange :

$$M_1 = 0.61 \times 3.4 = 2.07 \text{ t.cms.}$$

$$M_2 = 0.61 \times 3.4 - 2.5 \times 2.0 = 2.93 \text{ t.cms.}$$

$$Z_{xx} = (10 \times t^2) / 6 ; F_b = 1.4 \text{ t/cm}^2$$

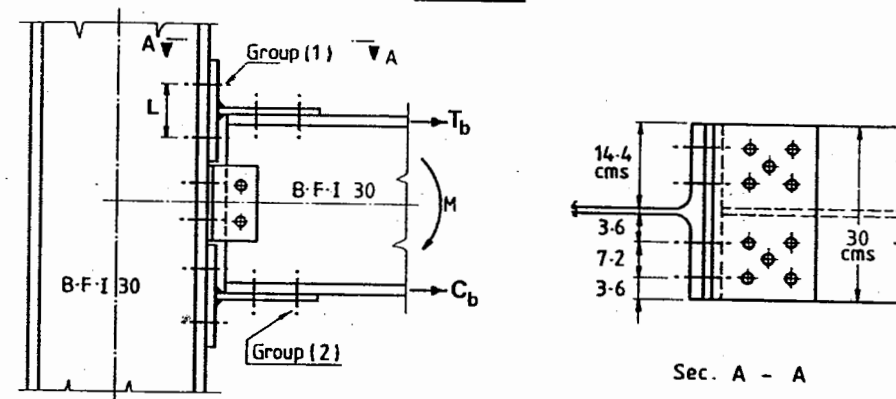
$$t_p = \sqrt{\frac{6 \times 2.93}{1.4 \times 10}} = 1.12 \text{ cms.} < 1.2 \text{ cms. o.k.}$$

Note that if we consider that the section regarding M_2 being a solid rectangular section ($F_b = 0.72 F_y$), but as the behaviour of the end plate is complicated we prefer to be coservative ($F_b = 0.58 F_y$).

Example (2.4) Moment Split Beam Tee-Stub-Connection:

Design a split-beam T-connection of a B.F.I# (30). The connection is subjected to a moment $M = 8.00 \text{ mt.}$ The European T-Sections can be utilized in the connection.

Solution



(1) High Strength Bolts:

- Bolts Group (1):

$$T_b = \frac{M}{d_b} = \frac{8 \times 100}{(30 - 2)} = 28.57 \text{ ton.}$$

Assume no. of bolts = 8.0 bolts.

$$\therefore T_{\text{ext}, b} = \frac{28.57}{8} = 3.57 \text{ ton. } T_{\text{ext}, b} + P \leq 0.8 T$$

Use as a first approximation 0.7 instead of 0.8

$$\therefore T_{i \text{ req.}} = \frac{T_{\text{ext}, b}}{0.80} = \frac{3.57}{0.80} = 4.46 \text{ tons}$$

$$\therefore \text{Use } 8 \text{ H.S.B'S } \underline{M(16) \text{ type } (10.9)} [T_i = 9.89 \text{ ton}].$$

- Bolts Group (2): Use H.S.B's M_{16} type (10.9) $P_s = 3.16 \text{ tons.}$

$$\text{No. of bolts req.} = n = \frac{800}{28 \times 3.16} = 9.04 \approx 10.0 \text{ bolts}$$

Assume to use 5 bolts per row.

Hence number of rows = 2

(2) Thickness of T-Stub Web:

$$\text{Area required} = \frac{800}{28 \times 1.40} = 20.40 \text{ cm}^2$$

$$\therefore b \times t = 20.40 \text{ cm}^2 \quad \text{Assume } b_{Tee} = b_b = 30 \text{ cm}$$

$$\therefore t_{wst} = \frac{20.40}{30} = 0.68 \text{ cm (use } t_{wst} = 1.0 \text{ cm)}$$

req.

(3) Thickness of T-Stub flange:

$$a = 1.5d = 2.4 \text{ cm}; b = 1.5d = 2.4 \text{ cm}; s = 1.0 \text{ cm}; t_{wst} = 1.0 \text{ cm}$$

$$L = 2b + 2s + t_{wst} = 2 \times 2.4 + 2 \times 1.0 + 1.0 = 7.8 \text{ cms.}$$

$$T = \frac{800}{28} = 28.57 \text{ tons.}$$

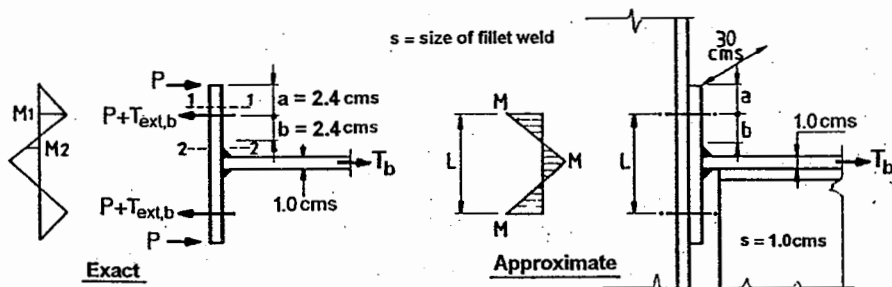
$$M = \frac{T L}{8} = \frac{28.57 \times 7.8}{8} = 27.85 \text{ t.cms. (Approximate)}$$

$$t_{p,req} = \sqrt{\frac{6 \times 27.85}{30 \times 1.4}} = 1.99 \text{ cms take } t = 2.0 \text{ cms.}$$

Equation 2.20 will give the same value of $t_{p,req}$:-

$$t_p = 0.6123 \sqrt{\frac{M \cdot (2b + 2s + t_b)}{d_b \cdot w \cdot F_b}}$$

$$t_p = 0.6123 \sqrt{\frac{800(2 \times 2.4 + 2 \times 1 + 1)}{28 \times (30/2) \times 1.4}} = 1.99 \text{ cms.}$$



(4) Prying Force Effect: (Eqn. 2.16)

$$a = 1.5 d = 2.4 \text{ cms}, b = 1.5 d = 2.4 \text{ cms},$$

Assume to use two rows of bolts of 2 bolts each :-

$$T_{ext,b} = \frac{28.57}{4} = 7.1425 \text{ tons}$$

$$\text{As } T_{ext,b} + P \leq 0.8 T \text{ (use the factor 0.7)}$$

$$T_{ext,b} = \frac{7.1425}{0.7} = 10.2 \text{ tons (M20)}$$

$$w = \frac{30}{2} = 15.0 \text{ cms}; t_p = 2.0 \text{ cms}; A_s = 2.45 \text{ cm}^2; T = 15.43 \text{ tons}$$

two bolts per row

Applying in Equation (2.16) we get:

$$P = \left[\frac{\frac{1}{2} - \frac{w t_p^4}{30 a b^2 A_s}}{\left(\frac{3a}{4b}\right) \left(\frac{a}{4b} + 1\right) + \frac{w t_p^4}{30 a b^2 A_s}} \right] T_{ext,b}$$

$$P = 0.224 T_{ext,b} = 0.224 \times 7.1425 = 1.6 \text{ tons.}$$

$$\begin{aligned} \text{Total force carried by each bolt of group "1"} &= 7.1425 + 1.6 \\ &= 8.74 < 0.8 \times 15.43 \\ &\text{O.K.} \end{aligned}$$

$$M_1 = 1.6 \times 2.4 = 3.84 \text{ t.cms.}$$

$$M_2 = 1.6 \times 2.4 - 7.1425 \times 2.4 = 13.3 \text{ t.cm.}$$

} Equ. 2.19

$$t_{req} = \sqrt{\frac{6 \times 13.3}{1.4 \times 15}} = 1.95 \text{ cms} < 2.0 \text{ cms. o.k (Equ. 2.18)}$$

Example (2.5):

Design an extended end-plate moment beam to column connection subjected to a moment $M = 24$ mt and shear $Q = 16.0$ tons. The beam is a B.F.I. 30 and the column is a B.F.I. 30.

Solution

(1) Welds:

Use k-welds around the flanges as the fillet weld will not withstand the full moment capacity of the beam.

Around the web, the fillet welds are used to carry the shearing force $Q = 6.0$ ton.

$$\therefore L_w = h - 4 \times t_{fb} \\ = 30 - 4 \times 2 = 22 \text{ cm}$$

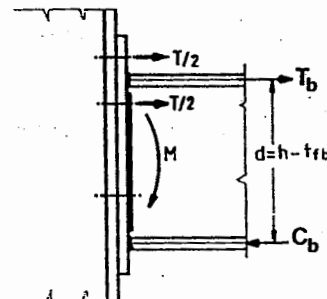
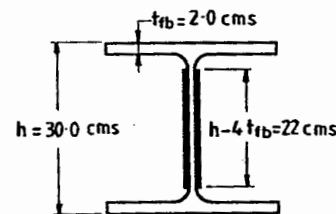
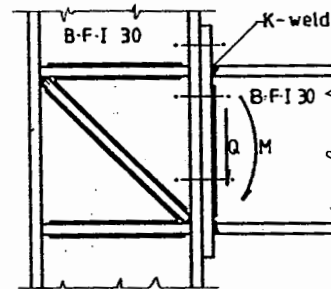
$$\therefore s = \frac{Q}{2 L_w \times f_{all}} \\ = \frac{16}{2 \times 22 \times 0.72} = 0.505 \text{ cm}$$

$\therefore$ Take $s = 6 \text{ mm}$.

(2) Approximate Analysis:

(A) High Strength Bolts:

Assume that the flange tension force will be resisted equally by the two upper rows of bolts.



$$\therefore T_b = \frac{M}{h - t_{fb}} = \frac{M}{d_b} = \frac{24 \times 100}{(30 - 2)} = 85.70 \text{ tons.}$$

If we use four bolts per row:

Hence:

$$T_{\text{bolt}} = \frac{85.70}{2 \times 4.0} = 10.70 \text{ ton.}$$

As an approximation where the prying force is neglected use the factor 0.7 instead of 0.8 get the following :-

$$T_{b, \text{req}} = \frac{10.70}{0.70} = 15.28 \text{ ton.}$$

Use high strength bolts M_{22} (10.9) $[T = 19.08 \text{ ton}]$.

(B) End - Plate Design:

Consider the end-plate to be a fixed beam at the locations of the upper two rows of bolts, having a concentrated load " T_b " at the mid-span.

$$\text{Where: } T_b = \frac{M}{d_b} = 85.70 \text{ ton.}$$

$$b = 1.5d = 3.3 \text{ cm}$$

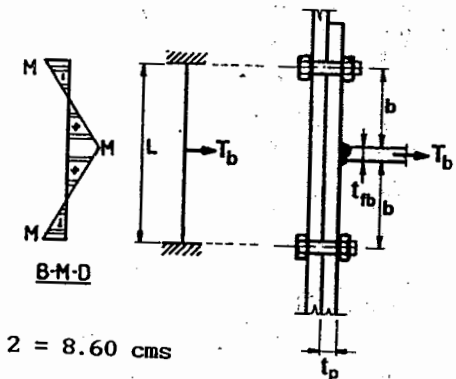
$$\therefore L = 2b + t_{fb} = 2 \times 3.3 + 2 = 8.60 \text{ cms}$$

$$\therefore M = \frac{T_b \cdot L}{8} = \frac{85.7 \times 8.60}{8} = 92.13 \text{ t.cm.}$$

$$\therefore t_p = \sqrt{\frac{6 \times 92.13}{30 \times 1.40}} = 3.63 \text{ cm.}$$

Equation 2.20 can be utilised :-

$$t_p = 0.6123 \sqrt{\frac{M \cdot (2b + t_b)}{d_b \cdot w \cdot F_b}}$$



$$t_p = 0.6123 \sqrt{\frac{800(2 \times 3.3 + 2)}{28 \times (30/2) \times 1.4}} = 3.627 \text{ cms.}$$

$$\therefore \text{take } t_{ep} = 3.80 \text{ cm.}$$

(3) Exact Analysis:

(A) High Strength Bolts:

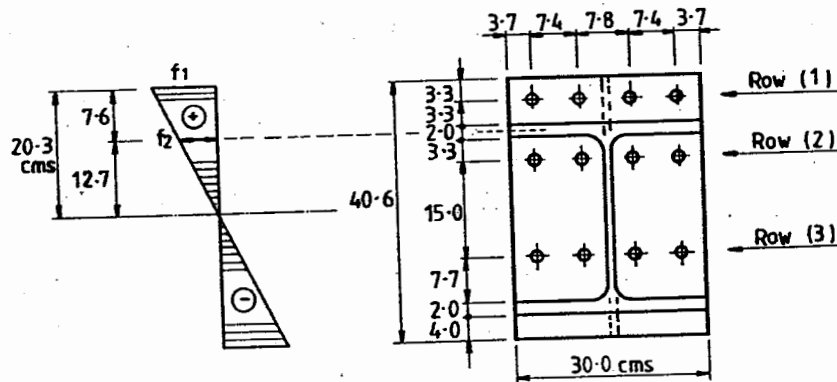
$$f_1 = \frac{M \times y_1}{I} = \frac{2400 \times 20.3}{30 \times 40.6^3 / 12} = 0.29 \text{ t/cm}^2.$$

$$f_2 = \frac{M \times y_2}{I} = \frac{0.29 \times 12.7}{20.3} = 0.18 \text{ t/cm}^2.$$

Row "1":

$$T_{\text{ext},b,M} = \frac{0.29 + 0.18}{2} \times 7.60 \times \frac{30}{4} = 13.4 \text{ ton.}$$

$$0.80 T_i = 0.80 \times 19.08 = 15.26 \text{ ton.} > T_{\text{bolt}} \quad (\text{o.k.})$$



Stress distribution

Row "2":

$$T_{\text{ext},b,M} = \frac{0.18}{2} \times 12.7 \times \frac{30}{4} = 8.57 \text{ tons.}$$

$$0.80 T_i = 15.26 > T_{\text{bolt}} \quad (\text{o.k.})$$

Check of Shear Stresses on bolts:

The safe shear load value of a M_{22} bolt = $P_s = 6.91$ ton.

This corresponds to the row of H.S.B. in the compression zone.

$$Q_{\text{bolt}} = \frac{16}{4} = 4.0 \text{ ton} < P_s \therefore \text{safe.}$$

(B) Check the Prying Forces in Bolts:

For working stress design, the prying force is;

$$P = \left[\frac{\frac{1}{2} - \frac{wtp}{30ab^2A_s}}{\left(\frac{3a}{4b}\right)\left(\frac{a}{4b} + 1\right) + \frac{wtp}{30ab^2A_s}} \right] T_{\text{ext},b,M} = 0.015 T_{\text{ext},b,M}$$

Where:

$$T_{\text{ext},b,M} = \text{force/bolt} = 13.4 \text{ ton.}$$

$$w = 30/4 = 7.5 \text{ cms. (4 columns of bolts)}$$

$$t_p = 3.80 \text{ cms.}$$

$$A_s = 3.03 \text{ cm}^2.$$

$$a = b = 3.30 \text{ cm.}$$

$$\therefore P = 0.015 \times 13.4$$

$$= 0.2 \text{ tons}$$

$$T_{\text{ext},b,M} + P = 13.4 + 0.2$$

$$= 13.6 \text{ ton.}$$

$$< 0.8 \times 19.08$$

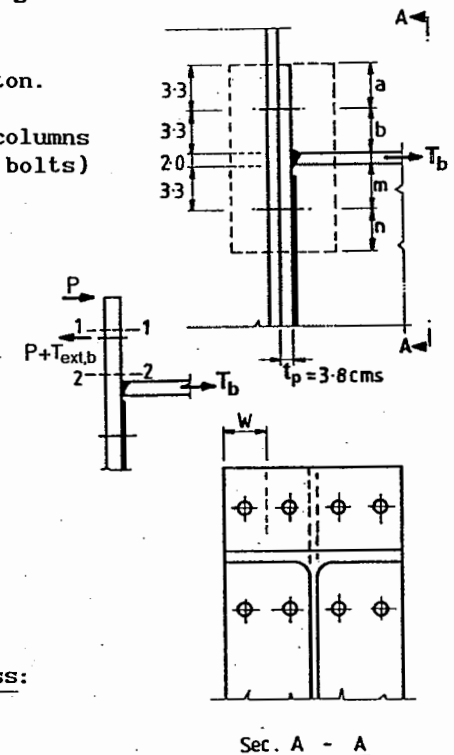
$$< 15.26 \text{ tons}$$

$$\text{O.K.}$$

(C) Check of End-Plate Thickness:

$$M_1 = P \cdot a = 0.2 \times 3.3$$

$$= 0.66 \text{ t.cms.}$$





$$M_2 = P \cdot a - T_{\text{ext},b,M} \cdot b = 13.6 \times 3.3 - 0.66 \\ = 44.22 \text{ t.cms.}$$

$$t_{\text{ep}} = \sqrt{\frac{6 \times 44.22}{1.4 \times (30/4)}} \cong 5.0 \text{ cms.}$$

It is to be noticed that the consideration of the prying force has increased the end-plate thickness to 5 cms. instead of 3.8 cms.

(4) Crippling of Column Web, Tension of Column Flange and Stability of Corner Web: Refer to Chapter (3) - Section (3.9)

- Crippling:

$$K = 2 \times 2.0 = 4.0 \text{ cms,}$$

$$t_{\text{fb}} = 2.0 \text{ cms, } t_p = 5.0 \text{ cms}$$

$$L = 2.0 + 2 \times 5.0 + 2 \times 2.5 \times 4 = 32.2 \text{ cms.}$$

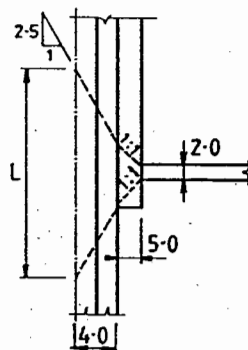
$$\text{Resisting area} = 32.2 \times 1.2 \\ = 38.64 \text{ cms}$$

$$\text{Applied area} = 30 \times 2 \\ = 60 \text{ cms.} \\ \text{Unsafe.}$$

Hence use 2 stiffeners

$$A_{\text{st}} = \frac{60 - 38.64}{2} = 10.68 \text{ cm}^2.$$

$$b_{\text{st}} = \frac{30 - 1.2}{2} = 14.4 \text{ cms take } b_{\text{st}} = 14.0 \text{ cms.}$$



$$t_{\text{st}} = \frac{10.68}{14.0} = 0.76 \text{ cms. (Area) } [b_{\text{st}} / t_{\text{st}} \leq (25 / \sqrt{F_y} = 16)]$$

$$t_{\text{st}} = \frac{14}{16} = 0.875 \text{ cms (local buckling)}$$

$$\text{take } t_{\text{st}} = 0.9 \text{ cms, } b_{\text{st}} = 14.0 \text{ cms.}$$

Tension of Column Flange: Refer to Chapter (3) - Section (3)

Minimum thickness required to column flange:

$$t_{\text{fc}} \geq 0.40 \sqrt{b_b \cdot t_b}$$

$$\therefore t_{\text{fc}} \geq 0.40 \sqrt{30 \times 2} = 3.10 \text{ cm.}$$

$\therefore$ Use stiffeners : 140 x 10 mms as before.

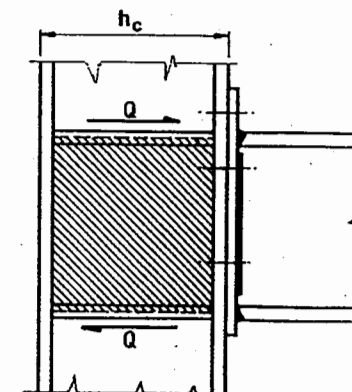
Stability of the Corner: Refer to Chapter (3) - Section (3)

$$Q = T = \frac{M}{d_b} = \frac{2400}{30 - 2} \quad (d_b = h_b - t_b) \\ = 85.70 \text{ ton.}$$

Resistance of Column Web in Shear:

$$Q_{\text{all.}} = h_c \times t_{\text{wc}} \times q_{\text{all.}} \\ = 30 \times 1.2 \times 0.84 \\ = 30.24 \text{ tons.}$$

$$\therefore Q_{\text{all.}} = 30.24 \text{ tons} < 85.70 \text{ tons.} \\ \therefore \text{Unsafe.}$$



Use Diagonal Stiffener:

$$\begin{aligned}\text{Remaining Shear} &= Q - Q_{\text{all.}} = F \\ &= 85.70 - 30.24 \\ &= 55.46 \text{ ton.}\end{aligned}$$

$$\therefore \text{Resultant force } R = \frac{55.46}{\cos \beta}$$

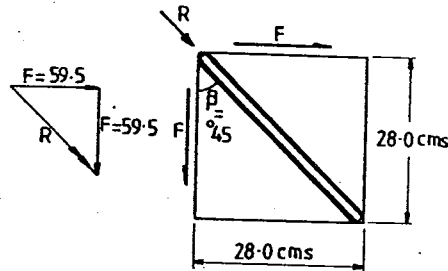
$$\text{Where: } \beta = \frac{90^\circ}{2} = 45^\circ$$

$$\therefore R = 55.46 \sqrt{2} = 78.44 \text{ ton.}$$

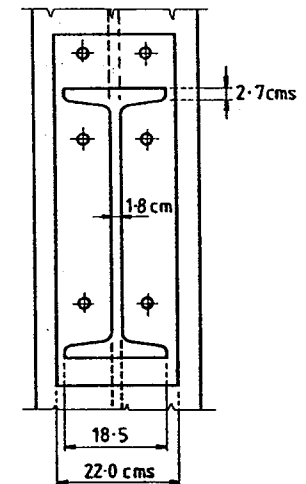
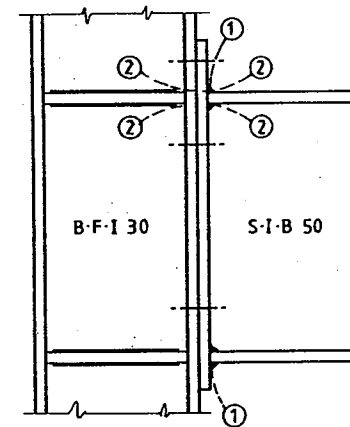
$$\therefore \text{Area of one stiffener} = \frac{87.44}{2 \times 1.4} = 28.01 \text{ cm}^2.$$

$$\therefore \text{Take } b_{\text{st}} = 14.0 \text{ cm}$$

$$\therefore t_{\text{st}} = \frac{28.01}{14.0} \cong 2.0 \text{ cm}$$

Example (2.6):

Design a rigid frame bolted end-plate connection subjected to a moment $M = 28 \text{ mt}$ and shear $Q = 16.8 \text{ tons}$. Axial tension in the beam = 1.50 ton. The beam is a S.I.B. # 50 and the column is a B.F.I. # 30.

Solution

(1) Welds of Beam to end-plate: (Refer to Chapter (3) - welded connections :-

To Estimate the Weld Size Around the Flanges:

Assume that the moment "M" is transmitted to the end-plate through the flange welds only. (Section 2-2)

$$\therefore T_b = \frac{M}{d_b} = \frac{28 \times 100}{(50 - 2.70)} = 59.20 \text{ ton.}$$

$\therefore$ Length of weld lines around one flange:

$$\begin{aligned}L_w &= b_{fb} + (b_{fb} - t_{wb} - 2 \times 2.7) \text{ (see Sec. 1-1 next page)} \\ &= 18.50 + (18.50 - 1.80 - 5.4) = 29.8 \text{ cm.}\end{aligned}$$

$$\therefore \text{Required weld size} = s = \frac{T_b}{L_w \times F_{pw}} = \frac{59.20}{29.8 \times 0.72}$$

$$\therefore s = 2.76 \text{ cm.}$$

.. Use fillet welds of 2.70 cm thickness around the beam flanges which correspond to the flange thickness.

Weld Size Around the Beam Web:

Only the shearing force Q will be transferred by the welds around the web:

Weld length = $2 (50 - 2 \times 2.70) = 78.40 \text{ cm.}$

$$\text{Weld size} = s = \frac{16.8}{78.40 \times 0.72} = 0.297 \text{ cm.}$$

∴ Use weld size of 5 mm thickness around the web.

Check of Safety of Welds at Section (1-1): (Exact Analysis):

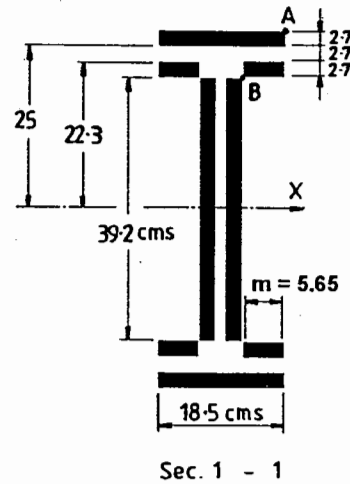
$$m = (18.5 - 1.8 - 2 \times 2.7) / 2 = 5.65 \text{ cms}$$

$$I_x = 2 \times 0.50 \times \frac{39.2^3}{12} + 4 \left[5.65 \times 2.70 \times (22.3 - 1.35)^2 \right] + 2 \left[18.50 \times 2.70 \times (25 + 1.35)^2 \right]$$

$$I_x = 107208 \text{ cm}^4.$$

$$\begin{aligned} A_w &= 78.40 \times 0.50 + 4 \times 5.65 \times \\ &\quad 2.70 + 2 \times 18.5 \times 2.70 \\ &= 200.12 \text{ cm}^2. \end{aligned}$$

$$\therefore f_{tA} = \frac{N}{A} + \frac{M \cdot y}{I}$$



$$= \frac{1.50}{200.12} + \frac{2800 \times 27.7}{107208} = 0.72 \text{ t/cm}^2 \text{ (Safe).}$$

For point (B) the stresses will be unsafe, hence increase weld size around the web to 8 mms.

$$\therefore I_x = 110220 \text{ cm}^4.$$

$$\therefore A_w = 223.64 \text{ cm}^2.$$

$$\therefore f_{tA} = \frac{1.50}{223.64} + \frac{2800 \times 27.70}{110220} = 0.71 \text{ t/cm}^2 \text{ (Safe).}$$

$$\therefore f_{tB} = \frac{1.50}{223.64} + \frac{2800 \times (39.2/2)}{110220} = 0.5 \text{ t/cm}^2.$$

$$\therefore q_B = \frac{17.0}{2 \times 0.8 \times 39.2} = 0.27 \text{ t/cm}^2.$$

$$\therefore f_{eq} = \sqrt{(0.5)^2 + 3(0.27)^2} = 0.687 \text{ t/cm}^2 < 0.72 \times 1.1 \text{ ton/cm}^2$$

. . Safe.

The size of weld $s = 2.7$ cms is excessive, hence it is recommended to use a haunch beam-to-column connection as given in Example 5.17.

(2) Approximate Analysis:

Design of High Strength Bolts:

Assume that the flange force in the beam is carried by the upper two rows of bolts, i.e. those adjacent to the beam tension flange.

$$T_b = \frac{N}{2} + \frac{M}{d_b} = \frac{1.50}{2} + \frac{2800}{(50 - 2.7)} = 59.95 \text{ ton.}$$

$$\therefore T_{\text{ext, b, M}} = \frac{59.95}{4} = 14.99 \text{ ton. as } T_{\text{ext, b, M}} + P \leq 0.8 T$$

Use 0.7 instead of 0.8 to overcome (P).

$$\therefore T_{\text{req.}} = \frac{14.99}{0.70} = 21.23 \text{ ton.}$$



$$P = \left[\frac{\frac{1}{2} - \frac{wtp^4}{30ab^2A_s}}{\left(\frac{3a}{4b}\right)\left(\frac{a}{4b} + 1\right) + \frac{wtp^4}{30ab^2A_s}} \right] \cdot (T_{\text{ext},b} + T_{\text{ext},b,M})$$

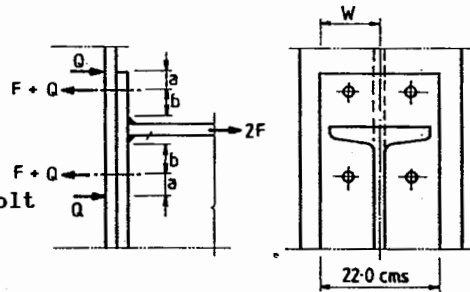
Where: Total tension/bolt = 21.61 + 0.25 = 21.86 tons

$$W = \frac{22}{2} = 11.0 \text{ cm.}$$

$$a = b = 4.0 \text{ cms.}$$

$$t_p = 5.0 \text{ cm.}$$

$$A_s = \text{nominal area of bolt} \\ = 4.59 \text{ cm}^2$$



$P = -ve$, Hence the prying force has no effect.

Having : 21.61 + 0.25 = 21.86 tons < 0.8 x 29.1 < 23.28 tons

∴ Safe

(5) Crippling of Column Web, Tension of Column Flange and the Stability of the Web Corner: Refer to Chapter (3) - Section (3)
Crippling:

$$K = 2 \times 2.0 = 4.0 \text{ cms, } t_b = 2.7 \text{ cms, } t_p = 5.0 \text{ cms}$$

$$L = 2.7 + 2 \times 5.0 + 2 \times 2.5 \times 4 = 32.7 \text{ cms.}$$

$$\text{Resisting area} = 32.7 \times 1.2 = 39.24 \text{ cms}^2$$

$$\text{Applied area} = 18.5 \times 2.7 = 49.95 \text{ cm}^2$$

Use 2 stiffeners

$$A_{st} = \frac{49.95 - 39.24}{2} = 10.71 \text{ cm}^2$$

Take the stiffeners width equal to the beam flange width.



$$b_{st} = \frac{18.50 - 1.20}{2} = 8.65 \text{ cm.}$$

$$\text{take } t_{st} \geq \frac{b_{st}}{16} \geq 0.54 \text{ cm}$$

$$\text{take } t_{st} = 0.7 \text{ cm. } \therefore A_{st} = 8.6 \times 0.7 = 6.02 \text{ cm}^2 \\ \therefore \text{Safe.}$$

Tensioning of Column Flange: Refer to Chapter (3) - Section (3.9)

Minimum thickness required for column flange.

$$t_{fc} \geq 0.40 \sqrt{A_{fb}}$$

$$\therefore t_{fc} \geq 0.40 \sqrt{2.70 \times 18.50} = 2.83 \text{ cm.} > 2.0 \text{ cms unsafe}$$

∴ Use stiffeners 86 x 7 mms as crippling.

Stability of the Corner: Refer to Chapter (3) - Section (3.9)

$$Q = T = \frac{M}{d_b} + \frac{N}{2} = 59.95 \text{ ton.}$$

Resistance of Column Web in Shear =

$$h_c \times t_{wc} \times q_{all}$$

$$= 30 \times 1.2 \times 0.84$$

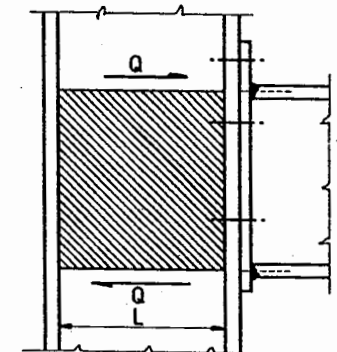
$$= 30.24 \text{ tons.}$$

$$\text{Remaining force} = 59.95 - 30.24 \\ = 29.71 \text{ tons}$$

∴ Use 2 additional cover plates with thickness (t_c) where,
(30 - 4 x 2) x 2 t_c x 0.84 = 29.71 tons.

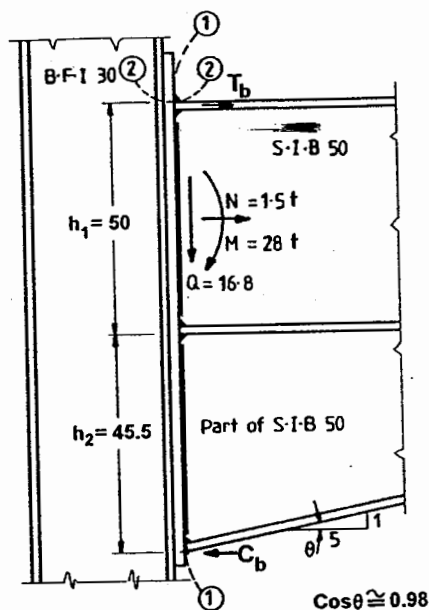
$$\therefore t_c = 0.804 \text{ cm.}$$

i.e. Use two additional cover plates each of 9 mms thickness.
(hatched area)



**Example (2.7):**

Design the connection of Example 2.9 using a haunch cut from the beam section.

Solution

$$h_1 = 50 \text{ cms}$$

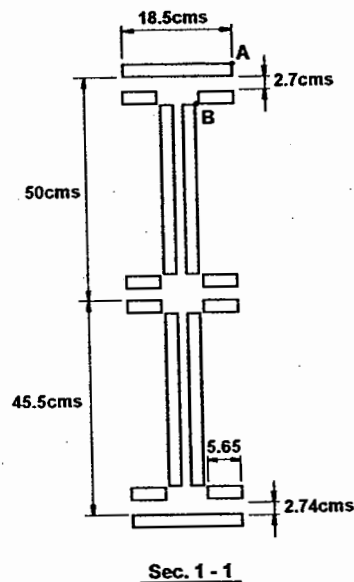
$$h_2 = (50 - 2 \times 2.7) / \cos \theta = 45.5 \text{ cms}$$

(1) Design of Welds:**Section (2-2)**

$$C_b = T_b = \frac{28 \times 100}{(95.5 - 1.35 - 1.375)} = 30.2 \text{ tons.}$$

$$q_w = \frac{30.2 / \cos \theta}{(18.5 + 2 \times 5.65) s} ; \text{ Let } q_w = 0.72 \text{ t/cm}^2.$$

Get $s = 1.43$, Take $s = 15 \text{ mms.}$

**Section (2-2)**

$$q_w = \frac{30.2}{(18.5 + 2 \times 5.65) 1.5} = 0.67 < 0.72 \text{ t/cm}^2 \text{ O.K.}$$

Section (1-1)

Size of weld around flanges and web 15mms and 4mms respectively

Height of welds around web = $95.5 - 4 \times 2.7 - 2 \times 2.75 = 79.2 \text{ cms.}$

$$I_{xx} = 2 \times 0.4 \times \frac{76.6^3}{12} + 4 \times 5.65 \times 1.5 \times 43.68^2 + 2 \times 18.5 \times 1.5 \times 49.1^2 = 228.441 \times 10^3 \text{ cm}^4$$

$$f_{tA} = \frac{28 \times 100 \times 50.45}{228441} = 0.618 \text{ t/cm}^2 < 0.72 \text{ (o.k.)}$$

$$f_{tB} = 0.618 \times \frac{42.3}{50.45} = 0.518 \text{ t/cm}^2 ; q_B = \frac{16.8}{2 \times 76.6 \times 0.4} = 0.274 \text{ t/cm}^2$$

$$F_{\text{equ}(b)} = \sqrt{(0.518)^2 + 3(0.274)^2} = 0.7 \text{ t/cm}^2 < 1.1 \times 0.72$$

(2) Design of H.S.B.:**Approximate Design:**

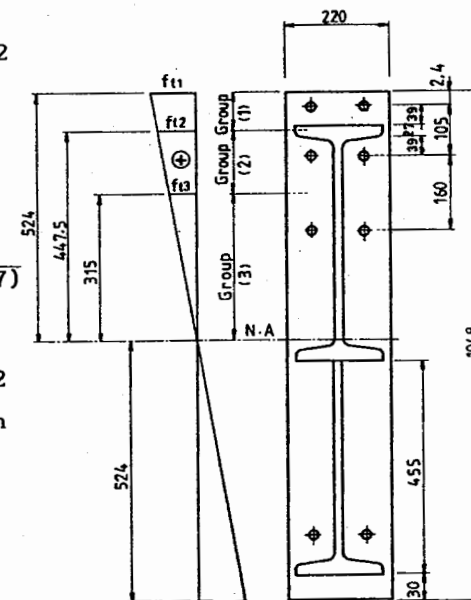
$$T_b = C_b = \frac{28 \times 100}{(95.5 - 1.35 - 1.377)} = 30.2 \text{ tons.}$$

Assume to use 2 rows of 2 bolts each around the tension flange.

$$T_{\text{req}} = \frac{30.2}{4 \times 0.7} = 10.78 \text{ tons.}$$

Choose $M_{16} (10.9) T = 9.89 \text{ tons}$

Something less than 10.78 tons because we have a third row of H.S.B. which will participate to resist the moment.



End Plate:

$$M = \frac{T_b \cdot L}{8}, \quad T_b = 30.2 \text{ tons}$$

$$L = 3.9 + 2.7 + 3.9 = 10.5 \text{ cms}$$

$$M = \frac{30.2 \times 10.5}{8} = 39.6 \text{ t.cm}$$

$$t_p = \sqrt{\frac{6 \times 39.6}{22 \times 1.4}} = 2.77 \text{ cms},$$

Take $t = 3.0 \text{ cms}$.

Exact Analysis:

$$f_{t1} = \frac{28 \times 100 \times 52.4}{I_{xx}}; \quad I_{xx} = \frac{22 \times 104.8^3}{12} = 2110207.9 \text{ cm}^4$$

$$f_{t1} = 0.0695 \text{ t/cm}^2$$

$$f_{t2} = \frac{0.0695 \times 44.75}{52.4} = 0.0593 \text{ t/cm}^2$$

$$f_{t3} = \frac{0.0695 \times 31.5}{52.4} = 0.0418 \text{ t/cm}^2$$

Group (1):

$$T_{\text{ext},b,M} = \frac{1}{2} \left(\frac{0.0695 + 0.0593}{2} \right) \times 22 \times 7.65 = 5.4 \text{ ton}$$

Group (2):

$$T_{\text{ext},b,M} = \frac{1}{2} \left(\frac{0.0593 + 0.0418}{2} \right) \times 22 \times 13.25 = 7.36 \text{ tons}$$

Group (3):

$$T_{\text{ext},b,M} = \frac{1}{2} \times \frac{1}{2} \times 0.0418 \times 22 \times 31.5 = 7.2 \text{ tons}$$

$$\text{The effect of tension force} = \frac{1.5}{8} = 0.187 \text{ tons.}$$

$$\text{Total Tension force per bolt} = 7.36 + 0.187 = 7.547 \text{ tons}$$

Effect of Prying Force:

$$T_{\text{ext},b} + T_{\text{ext},b,M} = 7.54 \text{ tons.}$$

$$a = 2.4 \text{ cm}; \quad b = 2.4 \text{ cm};$$

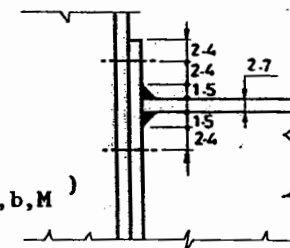
$$W = 11.0 \text{ cm}; \quad t_p = 3.0 \text{ cm.}$$

$$P = \left[\frac{0.5 - 1.37}{0.93 + 1.37} \right] \cdot (T_{\text{ext},b} + T_{\text{ext},b,M})$$

$$= -ve$$

The prying force has not effect on the bolts.

Then having $7.547 < 0.8 \times 9.89 < 7.92 \text{ tons}$ ∴ Safe



(3) Crippling of Column Web Refer to Chapter (3) - Section (3.9):-

Area of applied force =

$$18.5 \times 2.75 \times \cos\theta = 49.9 \text{ cm}^2$$

Resisting area =

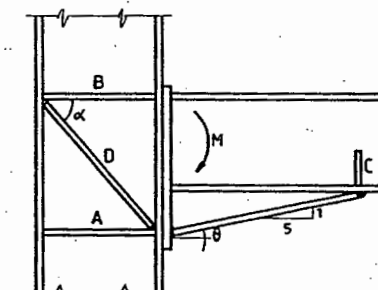
$$1.2 (2.75 + 2 \times 3 + 5 \times 2 \times 2)$$

$$= 34.50 \text{ cm}^2$$

$$A_{st} = \frac{49.90 - 34.50}{2} = 7.7 \text{ cm}^2.$$

(A)

Choose $b_{st} = 14.0 \text{ cms}$;

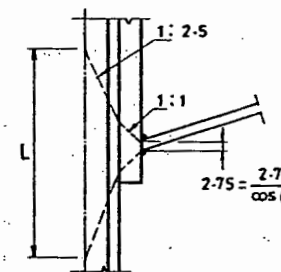


$t_{st} = 0.9 \text{ cms}$ to prevent local buckling

(4) Bending of Column Flange:

$$t_{fc} \geq 0.4 \sqrt{18.5 \times 2.7} \geq 2.82 \text{ cms}$$

Unsafe, hence use 2 stiffeners as for crippling.



2 stiffeners "B" (14.0 x 0.9) cms

(5) Shear Resistance of Corner Web:

$$\text{Applied Shear} = \frac{M}{d_b} = 30.2 \text{ tons.}$$

$$\text{Resistance} = 30 \times 1.2 \times 0.84 = 30.24 \text{ tons.}$$

$$\text{Remaining Shear} = 30.2 - 30.24 = \text{Zero}$$

The corner is safe regarding the shear resistance.

(6) Design of Stiffener "C":

$$F_c = T \times \frac{\sin \theta}{\cos \theta} = 30.2 \times 0.2 = 6.04 \text{ tons.}$$

$$A_{st} = \frac{6.04}{2 \times 1.4} = 2.15 \text{ cm}^2$$

(c)

$$\text{Choose } b_{st} = \frac{18.5 - 1.8}{2} = 8.35$$

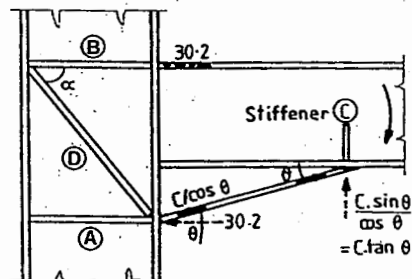
$$t_{st} = 0.6 \text{ cms. (local buckling prevented)}$$

N.B. It is to be noticed that the crippling and the bending of column flange have been analyzed on ultimate design basis. This naturally is leading to a conservative dimensioning. While if one would like to apply an allowable design approach; the following is to be considered.

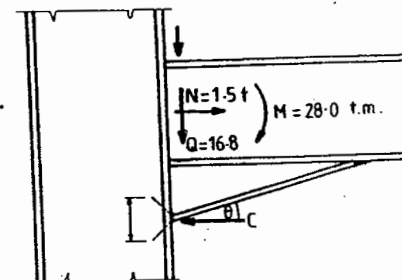
(7) Crippling:

$$C = T = \frac{28 \times 100}{95.5 - 1.35 - 1.37} = 30.2 \text{ tons.}$$

$$C = \text{Applied horizontal force}$$



$$\begin{aligned} \text{Resistance} &= 1.2 \left(\frac{2.7}{\cos \theta} + 2 \times \right. \\ &\quad \left. t_{wc} \left[3 + 5 \times 2 \times 2 \right] \times \right. \\ &\quad \left. 1.8 = 62.14 \text{ tons.} \right. \\ &\quad \left. f_{crp} \right. \end{aligned}$$



Safe and no need to use stiffeners opposite to the compression flange.

(8) Bending of Column Flange (Exact Analysis):

$$\text{Applied force} = 30.2 \text{ tons.}$$

$$\begin{aligned} \text{Resistance} &= 0.8 (7 \times 2^2 \times 1.4 + 0.15 \times 18.5 \times 2.7) \\ &= 37.35 \text{ tons.} \end{aligned}$$

Safe and no need to use stiffeners opposite to the tension flange.

N.B.

1. It is to be noticed that regarding steps (7) and (8) where the crippling and the bending of column flange show that using an exact analysis there are no need to use stiffeners.

We have to note that the E.C.P. 2001 does not allow to use these analysis.

2. Regarding the shearing force $Q = 16.8$ tons the assumption previously utilized were the bolts in the compression zone are resisting this shear is unsafe where :-

$$Q = \frac{16.8}{2} = 8.4 \text{ tons} > (P_s = 3.16 \text{ tons})$$

So we can use 4 H.S.B. in the compression zone (10.9) M 20 :-

$$Q = \frac{16.8}{4} = 4.2 \text{ tons} > (P_s = 4.93 \text{ tons})$$

Example (2.8):

$$M = 43.13 \text{ mt.}$$

$$N = -6.30 \text{ tons.}$$

$$Q = 7.0 \text{ tons.}$$

1. Welds:

Fillet welds result an unsafe connection, hence use k - welds to connect the beam flanges to the end plate.

Vertical fillet welds (1) will carry the shearing force Q.

Hence:

$$Q = 7 = 2 \times L \times s \times 0.72$$

$$L = 42.5 - 4 \times 2.60 = 32.1 \text{ cms.}$$

$$S = \frac{7}{2 \times 32.1 \times 0.72} = 0.155 \text{ cm}$$

Take $S = 4 \text{ mms.}$

2. H.S.B.'s:

Approximate solution: (first estimate of the number of bolts)

Assume a tension force $T_b = \frac{M}{d_b} - \frac{N}{2}$ will be carried by the two

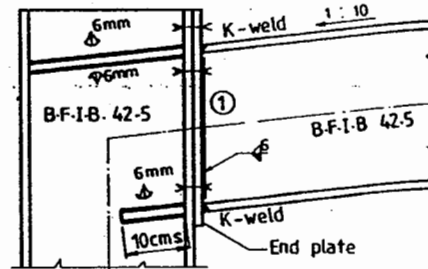
rows of bolts around the tension flange.

$$T_b = \frac{43.13}{42.5 - 2.6} - \frac{6.3}{2} = 104.90 \text{ tons.}$$

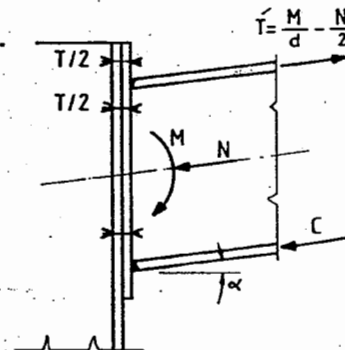
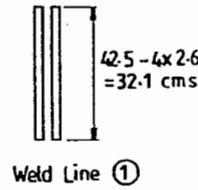
$$T_b = 104.9 \cos \alpha = 104.37 \text{ tons}$$

no. of bolts/row = 4 (2 rows)

$$T_{\text{ext},b,M} = \frac{104.37}{4 \times 2} = 13.1 \text{ tons.}$$



$$\cos \alpha = 0.995$$



As we have only 2 rows in the tension zone T_b is resisted totally by these 8 bolts.

Having the condition $T_{\text{ext},M} + P \leq 0.8$, use for first approximation without the prying force (P) the factor 0.7 instead of 0.8.

$$T_{\text{required}} = \frac{13.1}{0.70} = 18.7 \text{ tons.}$$

Refer to table (2.5) and choose H.S.B. (10.9), $d = 22 \text{ mms}$ where $T = 19.08 \text{ tons}$. This means that we have:

* 8 bolts M22 (10.9) in the upper two rows.

3. Thickness of End Plate:

$$T'_b = \frac{M}{d_b} - \frac{N}{2} = 104.90 \text{ tons}$$

$$T_b = T' \cos \alpha = 104.37 \text{ tons}$$

$$M = \frac{T_b \cdot L}{8} = \frac{104.37 \times 10.1}{8} = 130.5 \text{ cm.t}$$

Where as shown in Fig. of step (4) (next page)

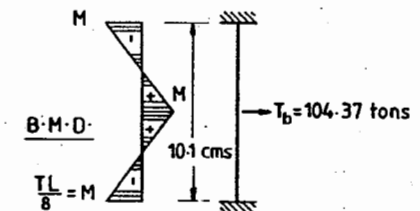
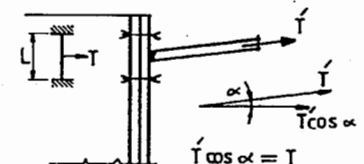
$$L = 3.7 + 2.61 + 3.7 = 10.1 \text{ cms}$$

$$F_t = \frac{M \times t/2}{b \cdot t^3/12} = \frac{6 M}{b \cdot t^2}$$

Where : $b = \text{breadth of end plate} = 30 \text{ cms}$

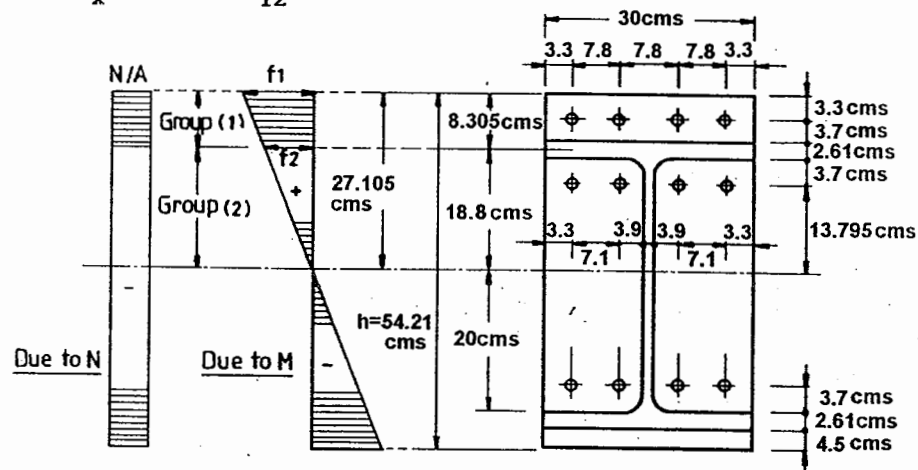
$$t = \sqrt{\frac{6 M}{b \times F_b}} = \sqrt{\frac{6 \times 130.5}{30 \times 1.4}} = 4.32 \text{ cms}$$

Take $t = 45 \text{ mms.}$



4. Exact Analysis:

$$I_x = 30 \times \frac{(54.21)^3}{12} = 398270 \text{ cm}^4$$



$$f_1 = \frac{4313 \times 27.105}{398270} = 0.293 \text{ t/cm}^2$$

$$f_2 = 0.293 \times \frac{18.8}{27.105} = 0.203 \text{ t/cm}^2$$

$$f_N = \frac{N}{A} = \frac{6.30 \cos \alpha}{54 \times 30} = 0.004 \text{ t/cm}^2$$

Effect of $\cos \alpha = 0.995$
on vertical dimensions

Where $\frac{t_b}{\cos \alpha} = \frac{2.6}{\cos \alpha} = 2.61$

$$h = \frac{42.5}{\cos \alpha} + 3.7 + 3.3 + 4.5 = 54.21 \text{ cms.}$$

For Group (1):

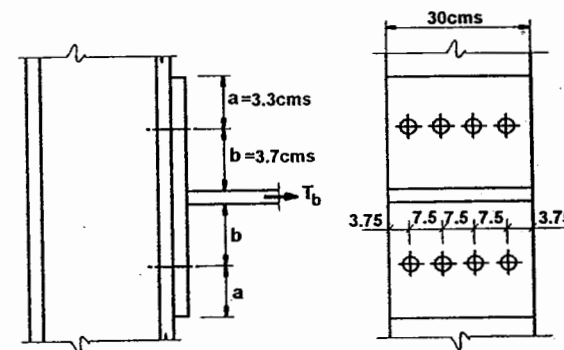
$$\begin{aligned} \text{Tension/bolt (due to M \& N)} &= T_{\text{ext (total)}} \\ &= \left[\frac{(0.293 + 0.203)}{2} - 0.004 \right] \times 30 \\ &\quad \times 8.305 \times \frac{1}{4} = 15.19 \text{ tons.} \end{aligned}$$

For Group (2):

$$T_{\text{ext (total)}} = \left[\frac{0.203}{2} - 0.004 \right] \times 18.8 \times 30 \times \frac{1}{4} = 13.74 \text{ tons}$$

For the determination of the prying force P get the following dimensions from figure below :-

$$a = 3.3 \text{ cms} ; b = 3.7 \text{ cms} ; t_b = 2.6 \text{ cms}$$



Applying in equation 2.16 where :-

$$P = \left[\frac{\frac{1}{2} - \frac{w \cdot t_p^4}{30 a \cdot b^2 \cdot A_s}}{\left(\frac{3a}{4b} \right) \left(\frac{a}{4b} + 1 \right) + \frac{w \cdot t_p^4}{30 a \cdot b^2 \cdot A_s}} \right] \cdot T_{\text{ext, b, M}}$$

w = breadth of end plate regarding one column = $30/4 = 7.5 \text{ cms}$

$A_s (M_{22}) = 3.03 \text{ cm}^2 ; t_p = 4.5 \text{ cms (from step 3)}$

$$P = \frac{0.5 - 0.949}{0.818 + 0.949} = -ve$$

Having a negative sign the prying has no effect on the design neither for bolts nor for the end plate.

Then :-

$$T_{\text{total}} = 15.19 \text{ tons} < 0.8 \times 19.08 < 15.26 \text{ tons}$$

∴ The bolts of group (1) are safe.

Consequently group of bolts (2) are safe.

Check on Shear:

For M(22) (10.9) H.S.B. : $P_s = 6.1$ tons

Assume that the 4 bolts in the compression are transmitting the shear.

$$Q_b = \frac{7.0}{4} = 1.75 \text{ ton} < 6.1 \text{ tons} \therefore \text{Safe}$$

5. Design of the Corner Connection:

Refer to Chapter 3 section (3.9)

5.1. Crippling of Column Web:

$$K = 2 t_c = 2 \times 2.60 = 5.20 \text{ cms.}$$

$$\text{Applied area} = 30 \times 2.6 \approx 78 \text{ cm}^2$$

$$\text{Resisting area} = \left[\frac{t_b}{\cos \alpha} + 2t_p + 5k \right] t_{wc}$$

$$= (2.61 + 2 \times 4.5 + 5 \times 5.2) \cdot 1.4 = 52.65 \text{ cm}^2$$

$$\text{Area of one stiffener} = \frac{78 - 52.65}{2}$$

$$\approx 12.7 \text{ cm}^2$$

$$\text{Let } b_{\text{stiff.}} = 14.3 \text{ cms ; } \frac{b}{t} \leq 16 \text{ (local buckling condition).}$$

$$\text{Take } t_{\text{stiff.}} = 0.9 \text{ cm to prevent the local buckling.}$$

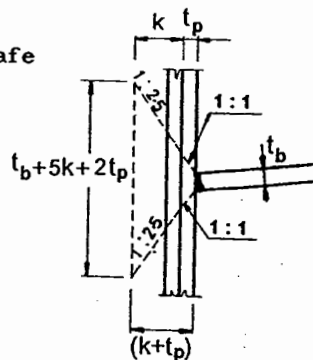
$$\therefore A_{\text{stiff.}} = 14.3 \times 0.9 = 12.87 \text{ cm}^2 \quad (\text{O.K.})$$

Welds Connecting Stiffener:

Based on allowable stresses ($F_c = 1.4 \text{ t/cm}^2$; $F_{crp} = 0.75 f_y$) we get:

$$78.0 \times 1.4 - 52.64 \times 0.75 \times 2.4 = 4 \times (42.5 - 2 \times 5.2) \times 0.72 \times s$$

$$s = 0.16 \text{ cm} \quad \text{Take } s = 4 \text{ mms.}$$

5.2. Safety Against Tension Flange: Refer to Chapter 3 section (3.9)

$$t_{wc \min} = 0.40 \sqrt{A_{fb} \cos \alpha} \approx 3.53 \text{ cms} > 2.6 \text{ cms} \quad (\text{Unsafe})$$

Use 2 stiffeners (140 x 9) as those opposite to the compression flange.

Weld side S = 4 mms.

5.3. Stability of Corner:

$$Q = \frac{M}{d_b} = \frac{43.13 \times 100}{(42.5 - 2.6)} = 108.1 \text{ tons.}$$

Here the applied compression force is neglected.

$$\text{Resistance} = \frac{42.5}{\cos \alpha} \times 1.4 \times 0.84 = 50.23 \text{ tons} < Q \quad \text{Unsafe}$$

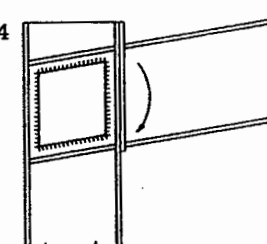
Hence use 2 cover plates of thickness t_c each.

$$\text{Remaining area} = 108.1 - 50.23 = 57.86$$

$$57.86 = 2 t_c \times \frac{(42.5 - 4 \times 2.6)}{\cos \alpha} \times 0.35 \times 2.4$$

$$t_c = 0.933 \text{ cms.}$$

$$\text{Take } t_c = 1.0 \text{ cm.}$$

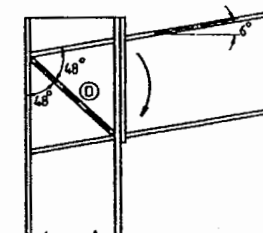


The other alternative is to use 2 diagonal stiffeners where the induced force is:

$$F_D = \frac{(108.1 - 50.23) \cos 6^\circ}{2 \sin 48^\circ} = 38.79 \text{ tons}$$

$$A_{\text{st.D}} = \frac{38.79}{1.4} = 27.7 \text{ cm}^2$$

$$\text{Choose } b_{\text{st}} = 14 \text{ cms} , t_{\text{st}} = 2.0 \text{ cms}$$



$$f_2 = \frac{4313 \times 31.84}{1246221.5} = 0.11 \text{ t/cm}^2$$

$$f_3 = \frac{4313 \times 18.435}{1246221.5} = 0.064 \text{ t/cm}^2$$

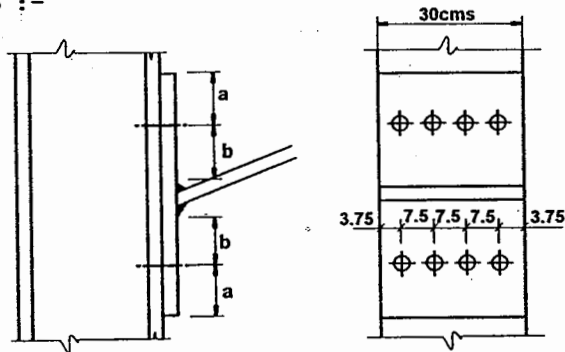
For Group (1):

$$T_{\text{ext},b,M} = \frac{(0.1372 + 0.11)}{2} \times \frac{7.805 \times 30}{4} = 7.23 \text{ tons.}$$

For Group (2):

$$T_{\text{ext},b,M} = \frac{(0.11 + 0.064)}{2} \times \frac{13.405}{4} \times 30 = 8.746 \text{ tons.}$$

For the determination of the prying force (P) apply in Equation 2.16 :-



$$a = 1.5d = 2.4\text{cms} ; b = 1.5d = 2.4\text{cms} ; t_p = 3.6\text{cms (step 2)}$$

$$w \text{ per row of bolts} = 30 / 4 = 7.5 \text{ cms} ; A_s (\text{M16}) = 1.57 \text{ cm}^2$$

$$P = \left[\frac{\frac{1}{2} - \frac{w \cdot t_p^4}{30 a \cdot b^2 \cdot A_s}}{\left(\frac{3a}{4b} \right) \left(\frac{a}{4b} + 1 \right) + \frac{w \cdot t_p^4}{30 a \cdot b^2 \cdot A_s}} \right] \cdot T_{\text{ext},b,M}$$

$$P = \frac{0.5 - 1.934}{0.9375 + 1.934} = -ve \text{ No effect of the prying force.}$$

Then for group (1) of 'H.S.B. :-

$$T_{\text{ext},b,M} = 8.746 \text{ tons} > 0.8T \text{ where :-}$$

$$0.8T = 0.8 \times 9.89 = 7.9 \text{ tons (Unsafe)}$$

Hence use M 20 instead of M 16 it will be safe.

Check of Shear: As all the bolts have the same diameter Shearing force will be divided equally by the lower row.

$$Q_{(\text{bolt})} = \frac{7}{4} = 1.75 \text{ ton.}$$

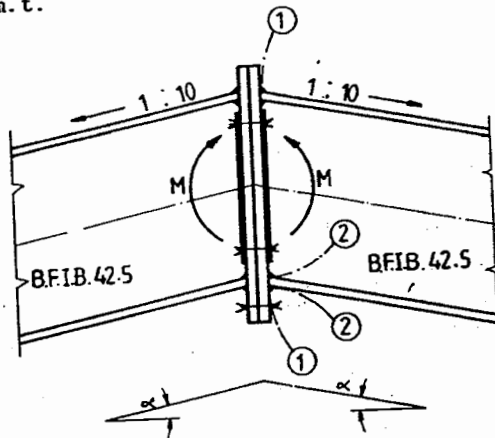
$$d = 20 \text{ mms. , } P_s = 4.93 \text{ tons} > Q_{(\text{bolt})} \therefore \text{ Safe}$$

4. For the design of the fillet weld lines as well as the crippling and the stability of the corner connection refer to the previous example.

Example (2.10):

Design the apex connection shown in the figure where:

$$M = 25 \text{ m.t.}$$



1. Welds:

Use fillet welds:

$$S \text{ (around web)} = 1.40 \text{ cm.} \quad S \text{ (around flange)} = 1.80 \text{ cm.}$$

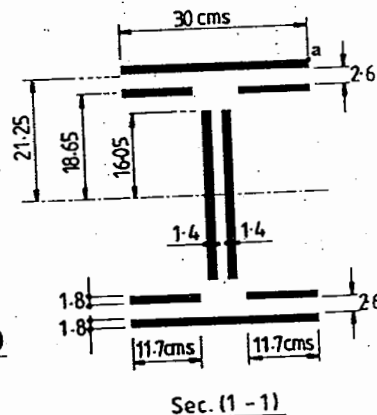
Neglect the small effect of the inclination.

Section (1-1):

$$I_x = 2 \times 30 \times 1.80 (21.25 + 0.9)^2 + 4 \times 11.70 \times 1.80 \times (18.65 - 0.90)^2 + 2 \times 1.40 \times \frac{(32.1)^3}{12} = 87246 \text{ cm}^4.$$

$$F_t(\text{point a}) = \frac{M_x Y}{I} = \frac{2500 \times (21.25 + 1.80)}{87246} = 0.66 \text{ t/cm}^2 < 0.72$$

Safe



Or we can use K - weld.

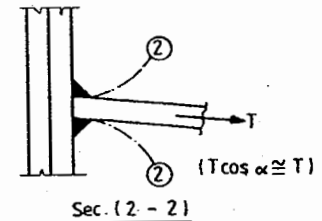
Section (2-2):

$$L = 30 + 2 \times 11.70 = 53.40 \text{ cms.}$$

$$S = 1.80 \text{ cm.}$$

$$T = \frac{2500}{42.5 - 2.6} = 62.7 \text{ tons.}$$

$$\vec{q}_2 = \frac{T}{A_{\text{weld}}} = \frac{62.70}{53.40 \times 1.80} = 0.65 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2. \quad (\text{Safe})$$



2. Design of H.S.B.:

Approximate Solutions:

$$T_b = \frac{M}{d_b} = 62.70 \text{ tons.}$$

This tension force will be carried by the two rows of bolts adjacent to the tension flange.

Assume to use 2 rows of H.S.B. (10.9) around the tension flange.

$$\text{* If we use H.S.B. d 22 mms (10.9) } T_{\text{bolt}} = 19.08 \text{ tons.}$$

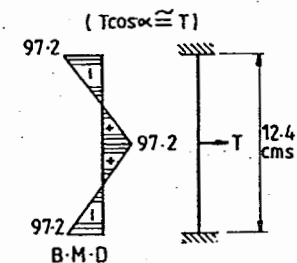
$$\text{number} = \frac{62.70 \times \cos \alpha}{0.80 \times 19.08} = 4.08 \rightarrow 5 \text{ bolts}$$

Use 5 bolts d 22 mms. in the lower two rows.

3. Thickness of End Plate:

$$T = \frac{M}{d} = 62.70 \text{ tons.}$$

$$M_1 = \frac{Tl}{8} = \frac{62.70 \times \cos \alpha \times 12.40}{8}$$



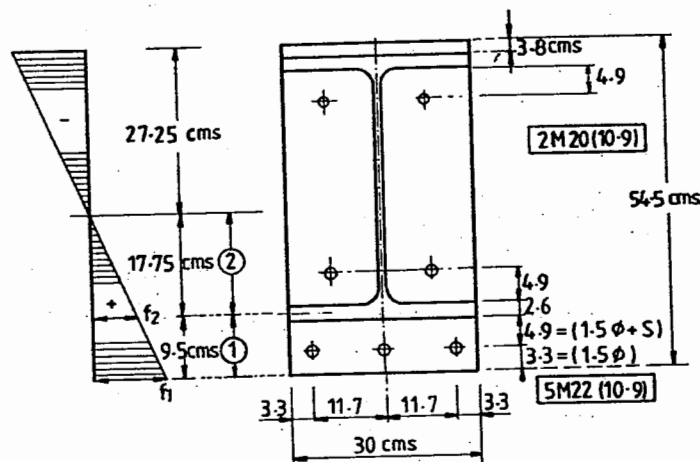


$$= 96.63 \text{ cm.t}$$

$$\therefore t = \sqrt{\frac{6M}{bf_t}} = \sqrt{\frac{6 \times 96.63}{30 \times 1.40}} = 3.72 \text{ cms.}$$

$\therefore$ Take $t = 3.80 \text{ cms.}$

4. Exact Analysis:



$$I_x = 30 \times \frac{(54.5)^3}{12} = 404696.55 \text{ cm}^4$$

$$f_1 = \frac{2500 \times 27.25}{404696.55} = 0.168 \text{ t/cm}^2$$

$$f_2 = 0.168 \times \frac{17.75}{27.25} = 0.11 \text{ t/cm}^2$$

Group (1):

$$T_{\text{ext}(M)}/\text{bolt} = \frac{(0.168 + 0.11)}{2} \times 9.50 \times 30 \times \frac{1}{3} = 13.2 \text{ tons.}$$



$$0.8 T_{\text{bolt}} = 0.80 \times 19.08 = 15.26 \text{ tons} > T_{\text{ext}(M)} \text{ (Safe)}$$

Group (2):

$$T_{\text{ext}(M)}/\text{bolt} = \frac{0.11}{2} \times 17.75 \times 30 \times \frac{1}{2} = 14.6 \text{ tons} < 0.80 T_{\text{bolt}} \text{ (Safe)}$$

N.B.

The prying force is not affecting the design as Eqn. 2.16 is negative.

Example (2.11): End Moment High Strength Bolted Connections

End plates - Type (3)-

Splice Flange Plates - Type (1)-

Design the following moment connection using high strength bolts (10.9). Choose two different concepts as follows:

(a) End plate moment connection:

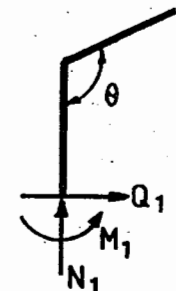
(b) Splice plate flange connection.

$$\theta = 107^\circ$$

$$M_1 = 30 \text{ mt}$$

$$Q_1 = 3.0 \text{ tons}$$

$$N_1 = 10 \text{ tons}$$



(a) End Plate Moment

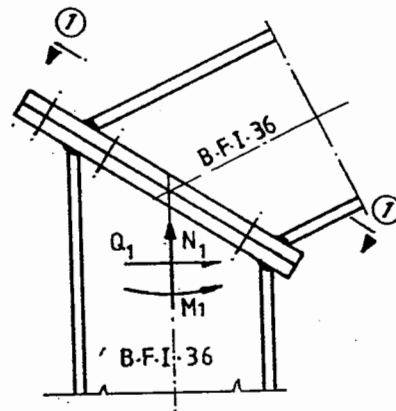
Connection

1. Straining Actions:

$$N = 10 \sin 53.5^\circ + 3 \cos 53.5^\circ = 9.81 \text{ tons.}$$

$$Q = 10 \cos 53.5^\circ - 3 \sin 53.5^\circ = 3.52 \text{ tons.}$$

$$M = 30.0 \text{ mt.}$$



2. Approximate Solution:

$$d = \frac{36 - 2.40}{\sin 53.5^\circ} = 41.8 \text{ cms.}$$

$$T = \frac{M}{d} = \frac{3000}{41.8} = 71.77 \text{ tons}$$

Assume to use 2 rows each

(4) bolts.

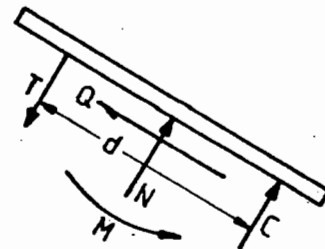
$$T_{\text{bolt required}} = \frac{7.17}{8 \times 0.80} = 11.20 \text{ tons}$$

(from Table (2.6) use
d = 20 mms)

3. The End Plate:

$$M_{\text{end plate}} = \frac{71.77 \times 8.98}{8} = 80.56 \text{ t.cm.}$$

$$t_{\text{end plate}} = \sqrt{\frac{80.56 \times 6}{30 \times 1.40}} = 3.39 \text{ cms} \quad \text{Take } t = 4.0 \text{ cms.}$$



4. Exact Solution:

$$I = \frac{30 \times (54.78)^3}{12} = 410966 \text{ cm}^4$$

$$f_1 = \frac{3000 \times 27.39}{410966} = 0.2 \text{ t/cm}^2$$

$$f_2 = 0.2 \times \frac{19.9}{27.39} = 0.145 \text{ t/cm}^2$$

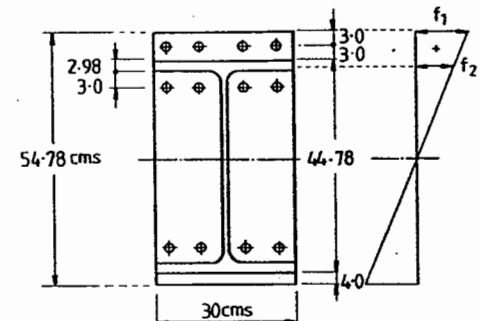
$$T_{\text{ext}} = \frac{1}{4} \left[\frac{(0.2 + 0.145)}{2} \times 7.49 \times 30 \right] \text{ (M)}$$

$$= 9.7 \text{ tons} < 0.80 \times 15.43 \text{ tons.}$$

$$Q_{\text{bolt}} = \frac{3.52}{4} = 0.88 \text{ ton} < P_s = 4.93 \text{ tons.}$$

The normal force "N" -compression- has been neglected.

N.B. Butt welds are used to connect flanges to the end plate.
Fillet welds are used to connect web to the end plate.



(b) Splice Plate Flange Connection

Assume moment is resisted by bolts in friction.

$$\text{Force per bolt's group} = \frac{M}{d} = \frac{3000}{(36-2.4)} = 89.28 \text{ tons}$$

$$n = \frac{89.28}{4.93} = 18.1 \text{ Take } n = 20 \text{ H.S.B. M20 } (P_s = 4.93 \text{ tons})$$

$$\text{Stress in splice plate} = \frac{89.28}{30 \times t}$$

$$\text{Allowable stress} = 1.40 \text{ t/cm}^2 ; t = 2.2 \text{ cms}$$

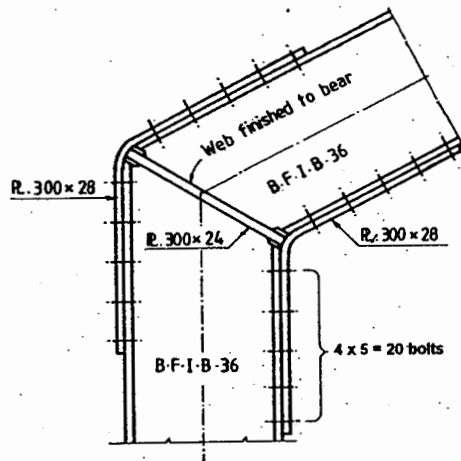
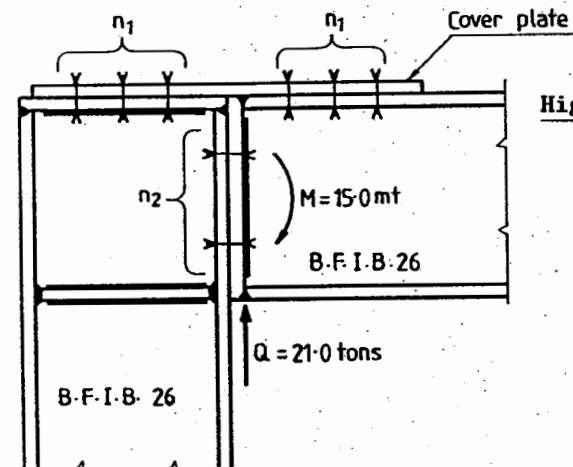
$$\text{Force in diagonal plate} = 2 \times 89.29 \cos 53.5^\circ = 106.22 \text{ tons.}$$



$$\text{Stress in diagonal plate} = \frac{106.22}{30 \times 2.6} = 1.36 \text{ ton/cm}^2 < 1.40$$

Safe

↑
thickness of the diagonal plate

**Example (2.12):**

High Strength Bolts
Type No. 1



Design the connection shown in Fig. in order to resist a moment of 15 mt. and a shear of 21 tons. Assume that the top flange connection resists the moment and the web connection resists the shear. Determine the number of 20 mm diameter high-strength bolts of the friction type.

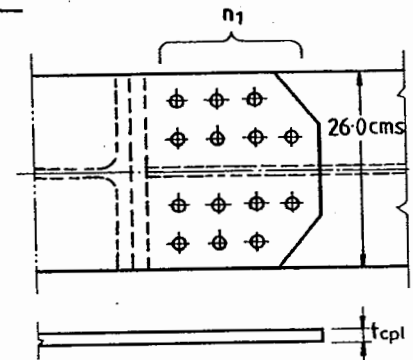
Solution

For H.S.B. $d = 20$ mms.

$$P_s = 4.93 \text{ tons}$$

$$F_{(\text{top cover plate})} = \frac{1500}{26 - 1.8} = 61.98 \text{ tons}$$

$$n_1 = \frac{61.98}{4.93} = 12.50 \text{ bolts}$$



Take $n_1 = 14$ bolts (see figure)

$$t_{(\text{cover plate})} = \frac{61.98}{26 \times 1.4} = 1.703 \text{ cms}$$

Take $t = 1.8$ cms

Number of bolts required for the web:

$$n_2 = \frac{21.0}{4.93} = 4.78 \text{ bolts, Take } n_2 = 6 \text{ bolts}$$

CHAPTER 3



CHAPTER 3 WELDED CONNECTIONS

3.1. Introduction:

The analysis and behavior of welded connections presented in this chapter will be in many ways similar to the bolted connections previously illustrated in chapters 1 and 2. Generally welded connections are simpler because drilling of holes for bolts is avoided and gusset plates can be eliminated. Using welding, smaller connections are available and are usually lighter in weight.

3.2. Process of Welding:

Welding is the process of joining two or more pieces of material. There are many different welding processes, but those used to weld steel all have in common the idea of heating the pieces to be joined until they become soft enough to fuse. Heat is used to melt the base metal and filler material in order that flow of material will occur i.e., the fusion will occur.

The most common welding processes, particularly for welding structural steel, use electrical energy as the heat source; the most often used method is the electric arc welding.

There are many varieties of arc welding, but those of interest here have in common the employment of a steel rod as one electrode and the pieces to be joined as the other. After being connected to a generator, the tip of the rod is brought close to the area to be welded. An arc is struck, and a current passes through the system, melting the end of the rod and local regions of the parts to be connected. By directing and moving the rod in the proper fashion, molten metal from it is deposited

in the desired amount and to the desired form in the gap between the two main pieces. This filler metal fuses with the melted regions of two parts and, when cool, forms an integral bridge between them. The properties of the welding rod (usually called the electrode) must be as carefully controlled as those of the main parts. Careful control of current and voltage and a degree of manipulative skill on the part of the welder are also required to produce a weld of desired strength.

Arc welding may be done either by hand or by machine. The former is most suited to the field and irregular welds in the shop. The latter is most adaptable to regular production type of work in the shop and, occasionally, the field. Hand welding is the older process, but the use of machine welding is growing rapidly as automatic machines and production techniques are developed. Generally, welds of equivalent size and strength can be produced by the two methods.

The zone to be welded is usually blanketed in an atmosphere which protects the molten region from the encroachment of impurities. This atmosphere is supplied by a flux, which may be a fusible coating on the welding rod, a fusible powder spread over the line of weld, or a gas sprayed over the weld.

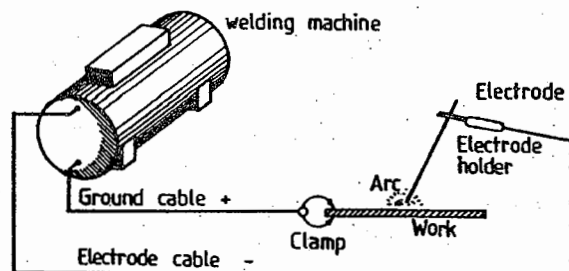


Fig. 3.1 Process of Welding

3.3. Welding Positions :-

As the most common welding processes use electrical energy as the heat source, the most often used is the electric arc. Since the depositing of weld metal in metal-arc welding is by the electromagnetic field and not by gravity, the welder is not limited to flat or horizontal welding positions. The four basic welding positions are shown in Fig. 3.2 where :-

a) In the flat position weld metal can be deposited faster because gravity is working with the welder, so large electrodes and high currents can be used.

b) In the vertical and overhead positions, electrode diameter below 4 mm (or at most 5 mm) are to be utilized otherwise weld metal runs down.

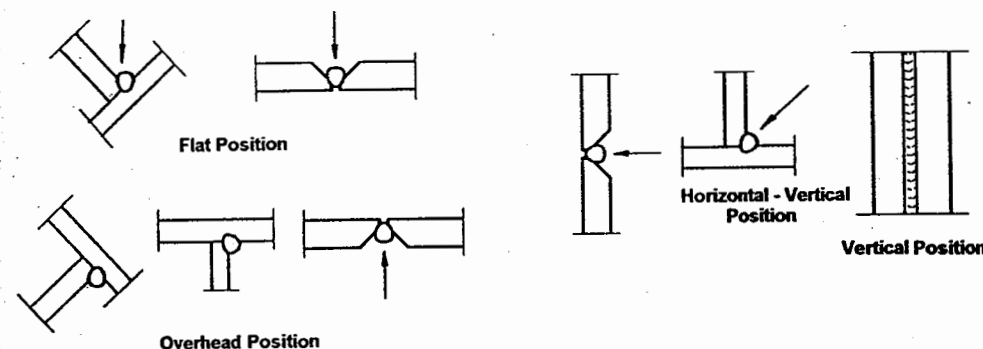


Fig. 3.2 Welding positions.

c) The designer should avoid whenever possible the over head position, since it is the most difficult one.

d) Welds in the shop are usually in the flat position, where manipulating devices can be used to rotate the work in a flat position.

e) Field welds may require any welding position depending on the orientation of the connection.



3.4. Different Welding processes :-

This section emphasizes the welding processes used in the welding of carbon and low-alloy steel for buildings and bridges.

3.4.1. Shielded Metal Arc Welding (S.M.A.W.) :-

This is the most usual type of welding where heating is accomplished by means of an electric arc between a coated electrode and the material to be joined. The welding circuit is shown in Fig. 3.3.a.

Coated electrodes are utilized where their composition differ with respect to the welding steel as follows :-

a) For mild steel, electrodes are made of low carbon composition where the extruded coatings contain aluminium, silicon and other deoxidizers which offers excellent ductility and excellent crack resistance to weld shrinkage strains.

b) For high-carbon or low alloy steels, low hydrogen electrodes are required with a carbonate of soda or "lime" coating.

c) The higher the tensile strength of steel being welded, the lower the electrode coating moisture must be to avoid weld cracking.

The coatings for electrodes E70 whose Min Tensile stress is 4.95 t/cm^2 can contain a maximum of 0.04 percent moisture while the electrodes E120 whose Min Tensile stress 8.4 t/cm^2 is limited to 0.0715 percent.

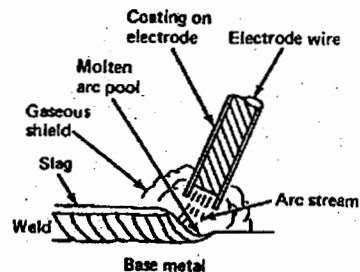


Fig. 3.3.a Shielded Metal Arc Welding.



In this process, the coated electrode is consumed as the metal is transferred from the electrode to the base material during the welding process. The coating is converted partly as follows :-

a) A part is converted into a shielding gas.

b) A part is converted into other materials to refine the grain structure of the weld metal.

c) A part is converted into a blanket of slag over the molten pool and the weld to protect it from oxygen and nitrogen in the air, and also retards cooling.

The storage of low hydrogen electrode must be in drying ovens near the points of use in the shop where unfortunately coatings will pick up moisture if stored under damp or high humidity conditions.

All welding positions are permitted (i.e flat, vertical, overhead and horizontal).

3.4.2. Submerged Arc Welding (S.A.W.) :-

The submerged arc process can be achieved using automatic or semi automatic equipment and usually used in shop. The major limitation of this process is that the work must be positioned for near flat or horizontal positions.

In this process, as shown in Fig. 3.3.b, the arc is not visible because it is covered by a blanket of granular, fusible material. The bar metal electrode is consumable in that it is deposited as filler material. The end of electrode is kept continuously shielded by the molten flux over which is deposited a layer of unfused flux in its granular state.

The flux is the characteristic feature of this process and is laid automatically along the seam ahead of the advancing electrode. It provides a cover which protects the weld pool against the atmosphere and serves to clean the weld metal.

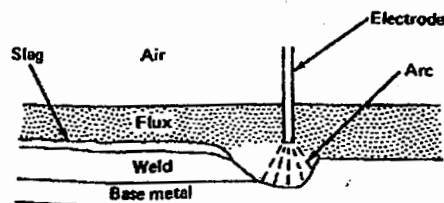


Fig. 3.3.b Submerged Arc Process.

The electrodes are of mild steel having the notation E6 or E7 indicating a minimum tensile strength of 4.25 and 4.95 t/cm² respectively. Fluxes are designated by a prefix F followed by two digit numbers indicating tensile strength and impact strength requirements for the required resulting weld. The combination of electrode and flux is usually designated together.

The submerged arc process provides uniformly high quality, good ductility, high impact strength, high density and good corrosion resistance.

Fluxes must be kept dry in storage to avoid moisture pickup and the consequent chance of hydrogen cracking in higher yield strength steels or in highly restrained joints particularly in thick members.

3.4.3. Gas Shielded Metal Arc Welding (G.M.A.W.) :-

In this process the shielding is provided entirely from an externally supplied gas or gas mixture while the electrode is a continuous wire supplied from a coil through the electrode holder.

For welding carbon steels the electrode is an uncoated mild steel deoxidized carbon manganese steel. While for welding low alloy steel a deoxidized low alloy electrode material is necessary.

Usually for welding carbon steel and low alloy steels the best overall performance is obtained using a mixture of 80% CO₂ and 20% Helium as a shielding gas. A mixture of 75% Argon and 25% CO₂ can react with the weld metal.

The use of 100% CO₂ as a shielding gas is the least expensive and has become extensively used for welding of lower carbon and low alloy steels usually used in building and bridges.

The shielding gas in addition to protecting the molten metal from the atmosphere serves to control the metal transfer characteristics, to affect the penetration and to control the undercutting.

3.4.4. Flux Cored Arc Welding (F.C.A.W.) :-

In this process, instead of a coating on the electrode, which is not feasible for a continuously fed electrode wire, the coating material, or flux, is contained in the core of the electrode.

This process may be achieved with or without external gas shielding where usually a CO₂ shielding gas is utilised.

The appropriate electrodes used for obtaining a butt weld matching a base metal of grade St 37 is designated E6xt having a minimum yield stress of 3.45 t/cm². For other steel grades and other types of welds see Table 3.3.

This process is useful for field welding in severe cold weather condition and can be utilized in any welding position.



3.5. Weldability and Steel Properties :-

The weldability is the capacity of a metal to be welded under the fabrication conditions imposed. The weldability is enhanced by low carbon, fine grain size and restricted (low) thickness.

As carbon steel is the term applied to steels containing the maximum percentages of elements other than iron :-

(a) carbon 1.7, (b) manganese 1.65, (c) silicon 0.6 and (d) copper 0.6 . The carbon steels include material from ingot iron containing essentially no carbon to cast iron which has at least 1.7 percent carbon. The structural carbon steels are of the mild carbon category with (0.15 - 0.29) percent carbon. Satisfactory economical welding without preheat, postheat or special welding electrodes can usually be accomplished when carbon content does not exceed 0.3 percent. The ideal chemical analysis of the carbon steels for good weldability is as given in Table 3.1.

Table 3.1 Carbon Steel Composition
for Good Weldability.

Element	Nominal Range Percent
Carbon	0.06 - 0.25
Manganese	0.35 - 0.8
Silicon	0.10 max.
Sulfur	0.035 max.
Phosphorus	0.03 max.

Table 3.2 abstracts the requirements covering weldability related variables where the toughness and resilience are measures of the ability of a metal to absorb mechanical energy are obtained from the tension test of uniaxial test (stress - strain diagram). The resilience, the toughness and the transition temperature from the Charpy V-notch impact test are the metal characteristics and are as follows :-



Table 3.2 Weldability Variables.

Grade of Steel	Nominal Values of Yield Stress F_y and Ultimate Strength F_u		Thickness t	Maximum Thickness of Statically Loaded Structural Elements (mm)	Resilience tcm/cm^3	Toughness tcm/cm^3	Charpy V-notch Test Temperature T_{cv}			
							T_{cv} Test (°C)	Minimum Energy J (joules) for Thickness (mm)		
	$t \leq 40$ mm	$40 \text{ mm} < t \leq 100$ mm						> 10 mm ≤ 150 mm	> 150 mm ≤ 250 mm	
	F_y (N/cm^2)	F_u (N/cm^2)					F_y (N/cm^2)	F_u (N/cm^2)		
St 37	2.40	3.60	2.15	3.40	250	1.54	840	-20°	27	23
St 44	2.80	4.40	2.55	4.10	150	2.31	945	-20°	27	23
St 52	3.60	5.20	3.35	4.90	130	3.0	1050	-20°	27	23



i) The resilience :- is the amount of elastic energy able to be absorbed by a unit volume of material loaded in tension i.e. it equals the area under the stress-strain diagram up to the yield stress.

ii) The toughness :- relates to the total energy, both elastic and inelastic, able to be absorbed by a unit volume of material before it fractures. For uniaxial tension, it is the area under the tension stress-strain curve out to the fracture point where the diagram terminates.

iii) Transition temperature of Charpy V-notch test :-

Since uniaxial tension rarely exists in real structures, a more practical index of toughness is used based on the more complex stress condition (triaxial) below the root of a notch. Notch toughness is the term used to describe the resistance of a metal to the start of propagation of a crack at the base of a standard notch. Notch toughness is most commonly measured by the Charpy V-notch test. This test uses a small rectangular simply supported beam with a V-notch at midlength. The bar is fractured by a blow from a swinging pendulum. The amount of energy is calculated from the height the pendulum raises after breaking the specimen. The Charpy V-notch test is widely used to determine the transition temperature T_{cv} from brittle to ductile behavior.

3.6. Different Types of Welds:

The welds can be classified into four main categories which are the groove or butt welds, the fillet, the slot and the plug welds as shown in Fig. 3.4. Each basic type of weld has specific advantages which determine the limits of its usage. These four types constitute all the structural welded connections existing in practice. Slot and plug welds are rarely used. They represent five percent of the whole usage of



welds. On the other hand, fillet welds are the most used types of welds and they represent about eighty percent, while groove welds are representing the remaining 15 percent.

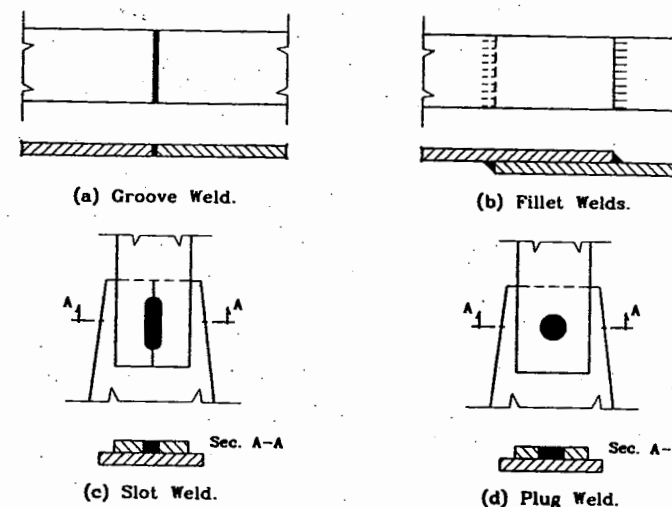


Fig. 3.4

3.7. Electrode Requirements :-

Since welds must transmit the entire load from one member to another, welds must be sized accordingly and to be formed from the correct electrode material.

The required electrode material depends on the base metal in the member as well as the process of welding and is as given in Table 3.3.

After the proper electrode is specified to match the steel in the pieces being joined the diameter of welding electrode must be selected. The particular size of the electrode selected is based on the size of the weld to be made and on the output of the welding apparatus since most welding machines have controls for reducing the current output, electrodes smaller than the maximum capacity should be used.

Table 3.3 Electrodes used for welding. (partI)

Process of Welding	Type of welding and the corresponding base metal, grade	Electrode Strength	
		Min Yield Stress t/cm ²	Min Tensile Stress t/cm ²
Shield Metal Arc Welding (S.M.A.W)	Butt weld with St. 37	3.45	4.25
	Butt weld with St. 52	4.15	4.95
	Fillet weld with St. 37, St. 44 and St. 52		
Submerged Arc Welding (S.A.W)	Butt weld with St. 37	3.45	4.25
	Butt weld with St. 52	4.15	4.95
	Fillet weld with St. 37, St. 44 and St. 52		
Gas Metal Arc Welding (G.M.A.W)	Butt weld with St. 37	3.45	4.25
	Butt weld with St. 52	4.15	4.95
	Fillet weld with St. 37, St. 44 and St. 52		
Flux Cored Arc Welding (F.C.A.W)	Butt weld with St. 37	3.45	4.25
	Butt weld with St. 52	4.15	4.95
	Fillet weld with St. 37, St. 44 and St. 52		

Table 3.3 Electrodes used for welding. (partII)

Chemical Composition	Weld Position	Remarks
<u>Electrode:</u> Low carbon <u>Coating:</u> Aluminum, Silicon, other deoxidizers	All weld Positions	Storage of electrodes in drying ovens near the points of use in the shop is a must.
<u>Electrode:</u> Medium Mn (1%) Normal carbon (0.12%) <u>Flux :</u> Finely powdered constituents glued together silitates	Flat or Horizontal weld Positions	Flux must be kept in storage, usually used in shop.
<u>Electrode:</u> Uncoated mild steel, deoxidized carbon manganize steel <u>Shielding Gas:</u> 75% Argon + 25% CO ₂ or 100% CO ₂	Flat or Horizontal weld Positions	CO ₂ is the least shielding used in buildings and bridges.
<u>Electrode:</u> Low carbon (0.05% Max) <u>Flux:</u> Filled inside the electrode core (Self Shielded)	All weld Positions	Useful for field welding in severe cold weather conditions.



The common diameter of electrodes for hand welding are 4 and 5 mm. For flat position 6 mm can be used. A 8 mm fillet weld size is the maximum size allowed in one run with 5 mm electrodes. For large sizes several runs in arc welding are to be made, while for gas processes any size can be made in one run.

Table 3.3 shows the strength properties and composition of electrode to match the process of welding, the type of welding and the corresponding base metal grade. For base metal of grades 44 and 52, only low hydrogen electrodes are to be utilized.

3.8. Design and Strength of Butt (Groove) Welded Connections :-

3.8.1. Nomenclature of the common terms :-

Fig. 3.5 shows the nomenclature terms for groove welds.

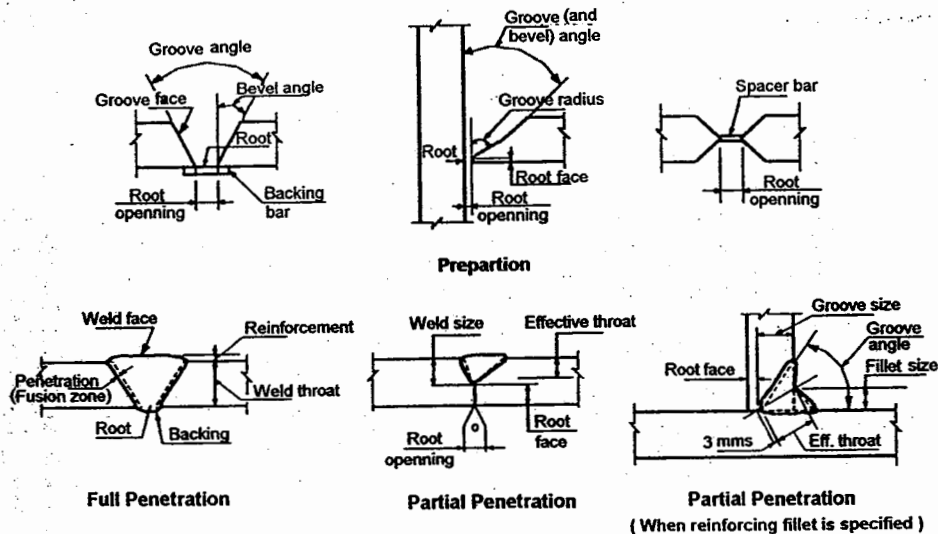
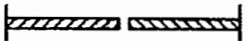


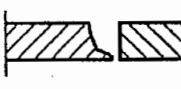
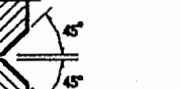
Fig. 3.5 Butt (Groove) Weld Nomenclature.

3.8.2. Different Types of Butt (Groove) Welds :-

Table 3.4 shows the different types of Butt (Groove) welds classified according to their particular shape and named according to their specified edge penetration requirement.

Table 3.4. Types of Groove Welding.

1.  Square - Butt - Welded from one side: up to 1.5 mm thick-no gap. - Welded from both sides using Normal Electrodes: up to 3 mm. thick-no gap. up to 5 mm. thick-take 1.5 mm gap. - Welded from both sides using deep penetration electrodes: up to 16 mm. -no gap.	2.  Square Butt with Backing Bar - Normal Electrodes: up to 5 mm. thick. 6mm gap. up to 13mm. thick. 8mm gap. - Deep Penetration Electrodes: up to 13mm. thick. 6mm gap.
3.  Single "V" Butt Weld - Included Angle: 60° for flat position. 70° for vertical position. 80° for over head position. - Root Thickness: 0-3 mm. - Thickness up to 25 mm. - Gap 1.5 mm - 3 mm.	4.  Single "V" Butt Weld With Backing Bar - Included angles as(3). - Gap 3 mm. - 5 mm. - Thickness up to 25 mm.
5.  Double "V" Butt Weld - Included angle gap and root thickness as(3) - Thickness 13 mm. - 50 mm.	6.  Single "U" Butt Weld - Included angle 20° - 40° - Gap 3 mm. - 6 mm. - Root thickness 3mm.-5mm. - Root radius 3mm.-10mm. - Thickness 25mm.-50mm.

7.  <u>Double "U" Butt Weld</u> • Dimensions as (6). • Thickness 38 mm. upwards.	8.  <u>Single "J" Butt Weld</u> • Included angle 20° - 30°. • Gap 3mm. - 5mm. • Root thickness 3mm.-5mm. • Root radius 5mm.-10mm. • Thickness 25mm - 50mm.
9.  <u>Double "J" Butt Weld</u> • Dimensions as (8). • Thickness 38 mm upwards.	10.  <u>Single Bevel Butt Weld</u> • Included angle 45° - 50°. • Gap 3 mm. - 6 mm. • Root thickness 1.5mm-3mm. • Thickness up to 25 mm.
11.  <u>Double Bevel Butt Weld</u> • Dimensions as (10). • Thickness 25 mm. upwards.	12.  <u>Double Bevel Butt Weld</u> • Angles as shown. • Gap 1.5 mm - 3 mm. • Root thickness 1.5mm-3mm. • Thickness up to 38 mm.

Regarding the advantages, the economy, and the effects of each type, the following remarks are to be considered :-

i) Double-bevel, double-vee, double-J and double-V groove welds are more economical than single welds of the same type because of less contained volume.

ii) Bevel or Vee grooves can be flame cut and therefore are less expensive than J and V grooves which require planning or arc-air gouging.

iii) Single V welding is achieved from one side, it is difficult to prevent distortion, This type is usually economical over 25 mms thickness.

iv) Single U welding is achieved from one side, the distortion is less than the single-Vee and is not economical under 19 mms thickness.

v) Double-Vee is a balanced welding with reduced distortion, requires reversals and is not recommended below 38 mms thickness.

vi) Double-U is a balanced welding with reduced distortion, requires reversals and is not recommended below 38 mms thickness.

vii) Groove welds joining plates of different thickness shall preferably be made with a gradual thickness change not exceeding 1:4 as shown in Fig 3.6.a. for tension members. In compression members there is no need for a gradual thickness transition. The difference in thickness may be balanced by a slop in the weld metal rather than machining the parent metal as shown in Fig. 3.6.b.

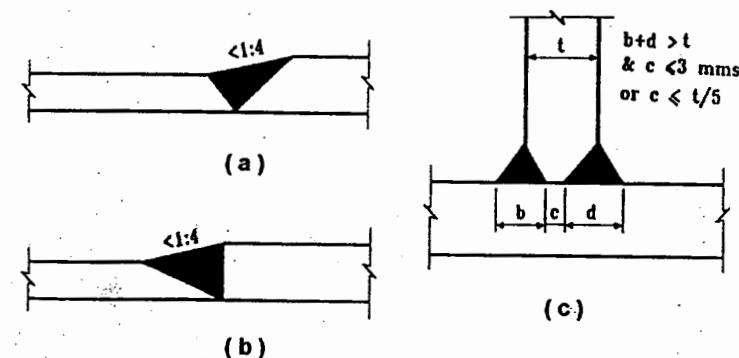


Fig. 3.6 Groove Welds for Plates of Different Thickness.



viii) Tee-Groove welds of Fig. 3.6.c are accepted even if they are not completely welded achieving a partial penetration groove weld if the total weld thickness is greater than the parent metal thickness, i.e. $(b + d) > t$ and $t/5 \geq c \leq 3$ mm. If these requirements are not fulfilled the Tee-Groove welds are to be analysed as being fillet welds according to the provisions of section 3.9.

3.8.3. The Groove Weld effective area :-

The effective area is the product of the effective thickness dimension times the length of the weld. The effective thickness dimension of a full penetration groove weld is the thickness of the thinner part joined as shown in Fig.3.7.a.

Incomplete penetration groove welds and unsealed groove welds are not recommended, but when it is impossible to avoid their use, the effective thickness of weld is taken as the sum of the actual penetrated depths as shown in Fig. 3.7.b and c.

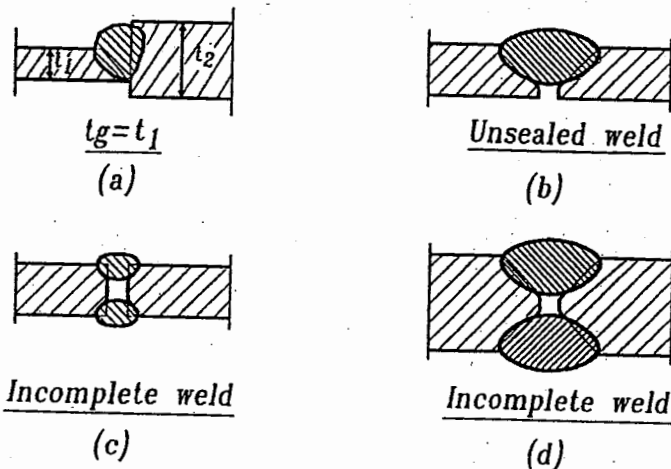


Fig. 3.7 Incomplete penetration and unsealed welds.



Partial penetration butt welds as shown in Fig. 3.8. are used in column splices and in connecting parts of built up members when the weld is not required to develop substantial stress in the connected material.

It is not recommended to use partial penetration butt welds except in column splices or in other members subjected primarily to axial compressive stress.

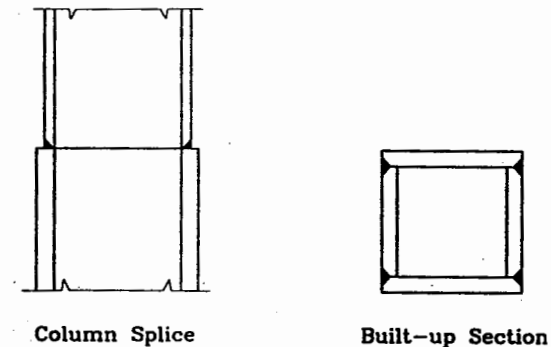


Fig.3.8.

3.8.4. Strength of Butt (Groove) Welds :-

The effective area of the complete penetration groove weld is of the same strength as the base metal. For permissible stresses two values are considered, the first for good welds fulfilling the requirements of the specifications, the second value for excellent welding where all welds are examined to guarantee the efficiency of the joint as given in Section 3.16.

i) Permissible stresses for static loading are shown in Table 3.5.

Where F_c , F_t and q are the allowable compression, tension and shear stresses of the base metal.

Table 3.5

Type of joint	Kind of Stress	Allowable Stress for	
		Good weld	Excellent weld
Butt and K-weld	Compression	$1.0 F_c$	$1.1 F_c$
	Tension	$0.7 F_t$	$1.0 F_t$
	Shear	$1.0 q$	$1.1 q$

ii) For fatigue loading refer to Chapter 7

3.8.5. Behavior and Analysis of Butt Welded Connections:

3.8.5.1. Strength and Nominal Stresses:

A full penetration Butt weld, (Fig. 3.9.a) must have a strength as high as the strength of the connected steel plates. When using one of the electrodes available on the market, we must be careful by choosing the appropriate electrode as mentioned by Table 3.3.

It is clear now, that the design analysis of a full-penetration butt weld in a statically loaded structure is thus quite elementary. We must check only that the actual stresses should not exceed the allowable stresses prescribed by the E.C.P. 2001 as mentioned by Table 3.5.

Fig. 3.9.b shows a butt weld subjected to an axial tensile force. The induced stresses are uniform and are computed as:

$$f_{ta} = \frac{P}{A_w} \leq 0.7 F_t \text{ (Good Weld)} \quad (3.1)$$

where $A_w = b.t$, and f_{ta} = the actual axial tensile stress.

If the force P is a compressive force equation (3.1) is still valid i.e.:

$$f_{ca} = \frac{P}{A_w} \leq 1.0 F_c \text{ (Good Weld).} \quad (3.2)$$

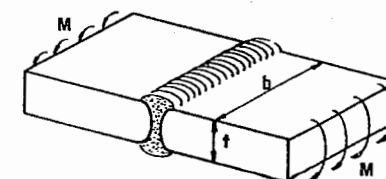
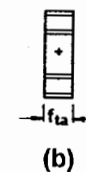
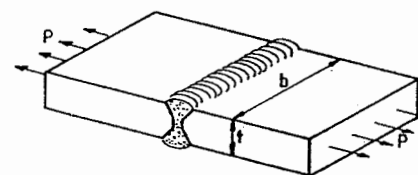
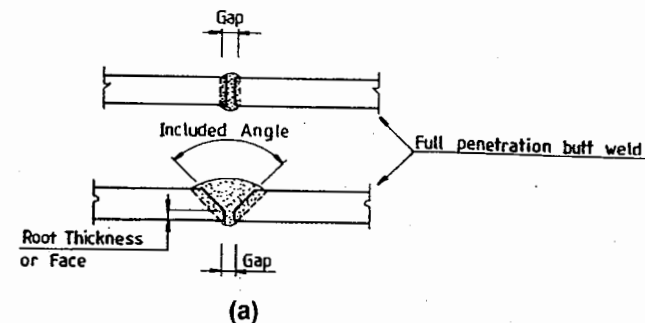


Fig. 3.9

On the other hand, Fig. 3.9.c shows a butt weld subjected to flexure (moment). The induced stress will follow the relation:

$$f_{ta} = f_{ca} = \frac{M}{Z} \leq F_{\text{allowable}} \quad (3.3)$$

where $Z = \frac{b t^2}{6}$

$F_{\text{allowable}}$ = Allowable stress as given in Table (3.5).



3.8.6. Constructional restrictions and remarks :-

1- Single V and U butt welds shall be sealed, whenever possible by depositing a sealing run of weld metal on the back of the joint. Where this is not done, the max. stress in the weld shall be (except as provided otherwise below) not more than one-half of the corresponding permissible stresses indicated in Table 3.5.

2- In the case of single and double V and U butt weld 18 mm. and over in size, in dynamically loaded structures, the back of the first run shall be cut out to a depth of at least 4 mm. prior to the application of subsequent runs. The grooves thus formed and the roots of single V and U butt welds shall be filled in and sealed.

3- When it is impossible to deposit a sealing run of weld metal on the back of the joint, then, provided backing material is in contact with the back of the joint, and provided also the steel parts are bevelled to an edge with a gap not less than 3 mm. and not more than 5 mm. To ensure fusion into the root of the V and the backing material at the back of the joint, the permissible stresses may be taken as specified in Table 3.5.

4- Possible defect that may result in discontinuities within the weld are to be avoided. Some of the more common defects are : incomplete fusion, inadequate joint penetration, porosity, undercutting inclusion of slag and cracks (refer to Section 3.15).

5.a- Butt welds shall be built up so that the thickness of the reinforcement at the center of the weld is not less than the following :

- Butt welds < 30 mm in size reinforce by 10 %
- Butt welds > 30 mm in size reinforce by 3 mm



b- Where a flush surface is required, specially in dynamic loading, the butt weld shall be built up as given in (a) and then dressed flush.

3.9. Design, Strength and Limitations of Fillet Welded Connections:

3.9.1. Nomenclature of the common terms :-

Fig 3.10 shows the nomenclature of the common terms for fillet welds.

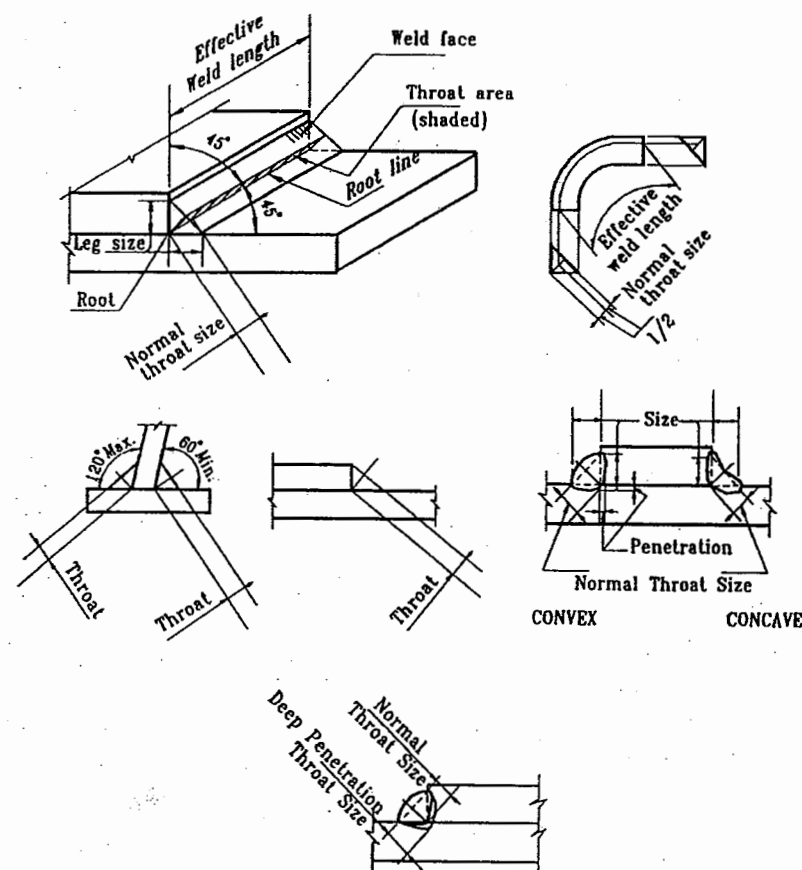


Fig. 3.10 Fillet Weld Nomenclature.



3.9.2. Different Types of Fillet Welded Connections :-

Fillet welds are made between plate surface which are usually at right angles, but the angle between the plates may vary from 60° to 120° . Tee joints, corner welds and cruciform joints are all combinations of fillet welds and are as shown in Fig. 3.11.

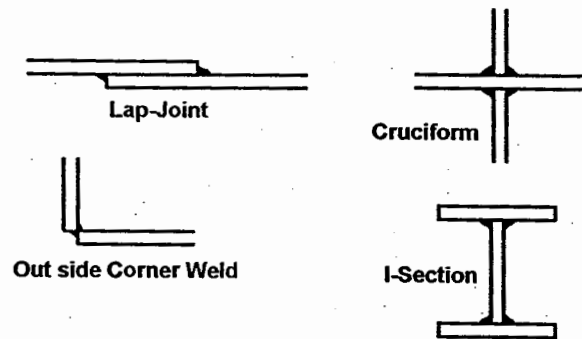


Fig. 3.11 Combinations of Fillet Welded Connections.

The ideal fillet is normally of the mitre shape which is an isosceles triangle as shown in Fig. 3.12 (h). The mitre and convex welds are stronger than a concave fillet weld of the same leg length when the weld is subject to static loadings, but the concave is stronger when subject to dynamic loadings.

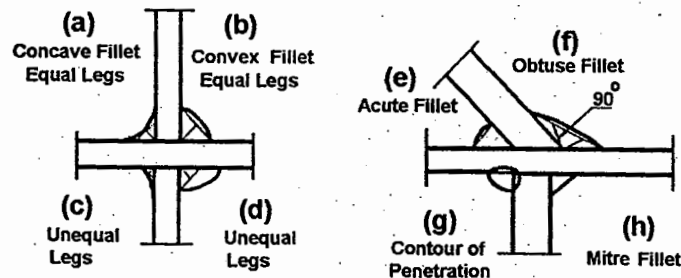


Fig. 3.12 Fillet Weld Configurations.



Vertical welds made upwards in one run, are generally convex. Usually low currents produce the convex welds.

The penetration of the weld should reach the root where the contour of penetration is usually as shown in (g) of Fig. 3.12.

3.9.3. Strength of Fillet Welds :-

3.9.3.1. Effective Area of Fillet Welds :-

The effective weld section is equal to the largest triangle which can be inscribed between the fusion surface and the weld surface, provided there is as a minimum root penetration, this penetration is not taken into account. The effective throat (t) is then the distance from the root to the surface of the isosceles triangular weld along the line bisecting the root angle as shown in Fig. 3.13.

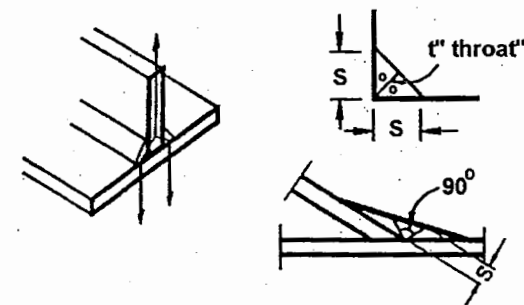


Fig. 3.13 Dimensions of Size and Throat of Fillet Weld.

Fillet welds are stressed across the throat (t) of the weld, while their size is specified by the leg length (s), where :

$$t = K.s$$

The value of "K" depends on the angle between the fusion face and it may be taken as follows :

Degree.	60°-90°	91°-100°	101°-106°	107°-113°	114°-120°
K	0.7	0.65	0.6	0.55	0.5

3.9.3.2. Strength and Permissible stresses of Fillet Weld

Connections :-

The actual stress in a fillet weld loaded in an arbitrary direction can be resolved into the following components:

$f_{\perp}$ = The normal stress perpendicular to the axis of the weld.

$q_{//}$ = The shear stress along the axis of the weld.

$q_{\perp}$ = The shear stress perpendicular to the axis of the weld.

These stresses shall be related to the size (s) of the legs of the isosceles triangle inscribed in the weld seam if the angle between the two surface to be welded is between 60° and 90°, when this angle is greater than 90° the size of the leg of the inscribed rectangular isosceles triangle shall be taken.

The permissible stresses F_{pw} for all kinds of stress for fillet welds must not exceed the following :

$$\text{All kind of stresses } F_{pw} \leq 0.2 F_U \quad 3.4$$

Where F_U is the ultimate tensile stress of the base metal.

In case where welds are simultaneously subject to normal and shear stresses, they shall be checked for the corresponding principal stresses. For this combination of stresses, an effective stress value f_{eff} may be utilized and the corresponding permissible weld stress is to be increased by 10 % as follows :

$$f_{eff} = \sqrt{(f_{\perp})^2 + 3(q_{\perp} + q_{//})^2} \leq 1.1 F_{pw} \quad 3.5$$

The effective length of a fillet weld is usually taken as the overall length of the weld minus twice the weld size (s) as deduction for end craters.

3.10. Fillet Welded Connections Subjected to Concentric Shear:

The force acting on a fillet weld may be either parallel, transverse or inclined to the axis of the weld as shown in Fig. 3.14. a, b, c. Welds of tee joints (Fig.3.14.d) are also fillet welds. The stress distribution in the weld depends on the relative orientation of load and weld axes. A few design specifications take these differences into account, but most do not.

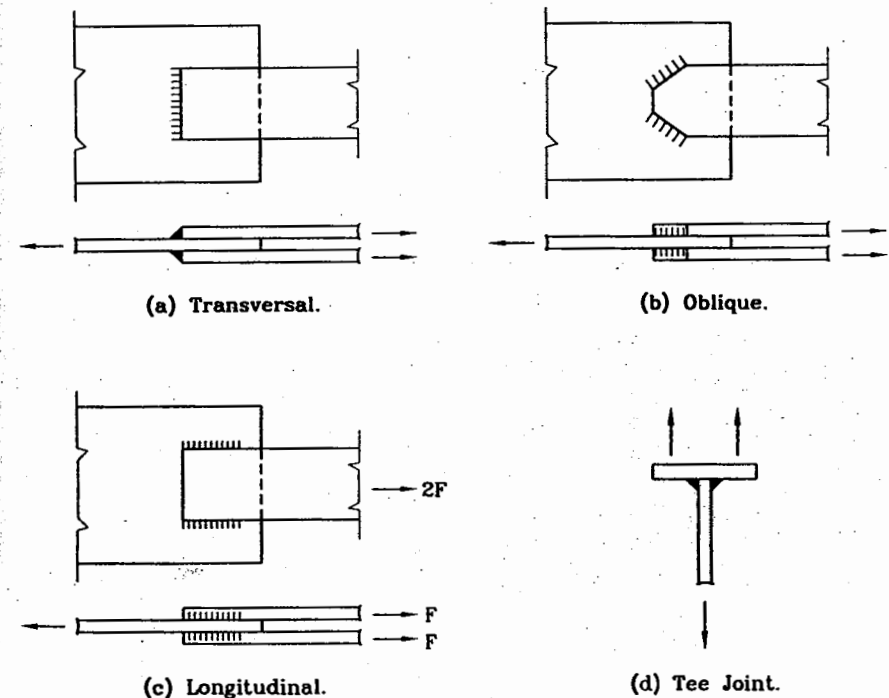


Fig 3.14. Axial Shear Fillet Welds



3.10.1. Stress for Longitudinal Fillet Welded Connections:

Fig. 3.15.a. shows a typical longitudinal fillet welded connection of weld line (L). At distances, out of the welds, the force in the upper and lower plates may be uniformly distributed (Fig. 3.15.b), but at sections near the welds it is highly concentrated.

Fig 3.15.c shows the actual shear distribution in the welded zone. The magnitude of its variation from points A to B will depend upon the length of the weld line as well as the ratio of the rigidity of the two plates being joined.

It is clear that if we consider a uniform distribution along the lines of welds (q_{average}), the failure will occur through the throat plane as indicated in Fig. 3.15.d.

Using the assumption of uniformity, the problem is becoming a very elementary one. The length of weld lines (L), the size of welds (S), and the force (F), must maintain a shear stress allowed by E.C.P. as previously given in equation 3.4 :

$$q_a = \frac{F}{2L \times S} \leq 0.2 F_U \quad (3.6)$$

Where:

q_a = the actual weld shear stress.

S = the size of the fillet weld.

L = the length of the lines of fillet welds.

It is to be noticed that according to the E.C.P. 2001 the computation of stresses are based on the size of weld while in the American and European specifications the calculation of stresses is based on the throat plane.

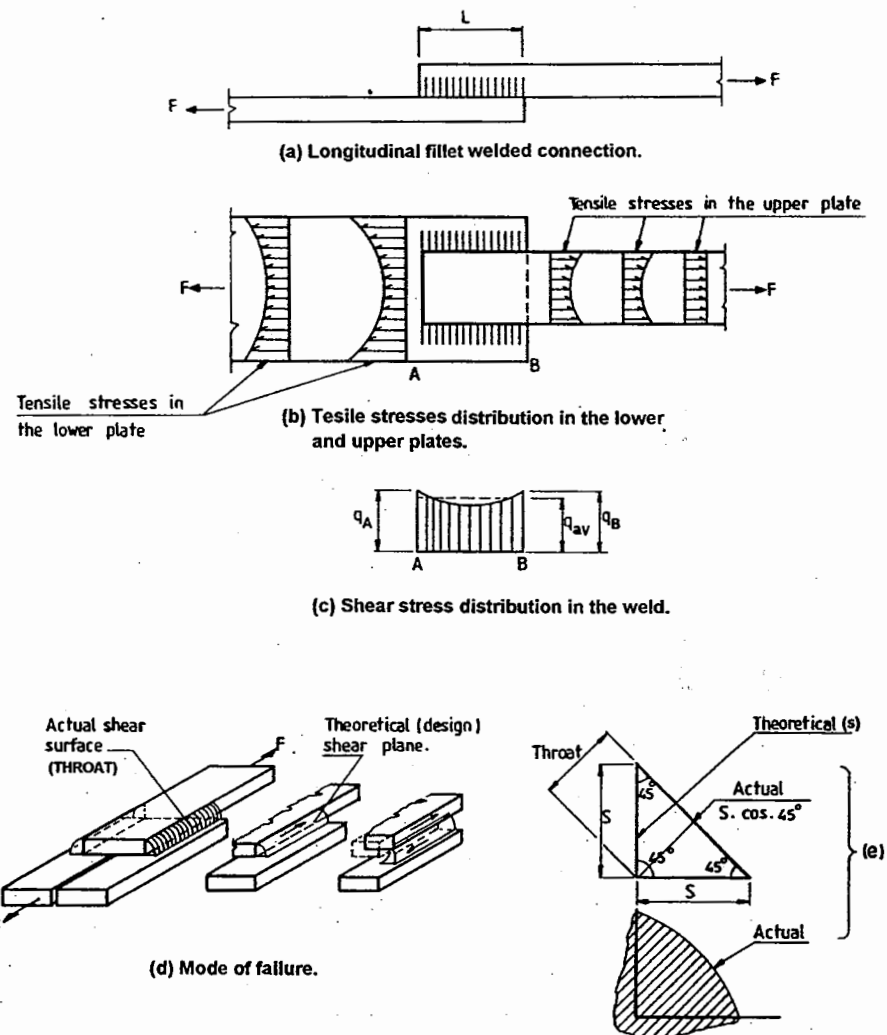


Fig. 3.15 Stress Distribution in Longitudinal Fillet Welds.

3.10.2. Transverse and Tee-joint Fillet Welds:

The typical shear stress distribution for transverse fillet welds is shown in Fig. 3.16.a. The actual distribution is unequal and will depend on the length of weld line (L) and on the relative rigidities of the connected plates.

If fillet welds are used to connect tee joints, the stress distribution is somewhat more complicated as indicated in Fig. 3.16.b.

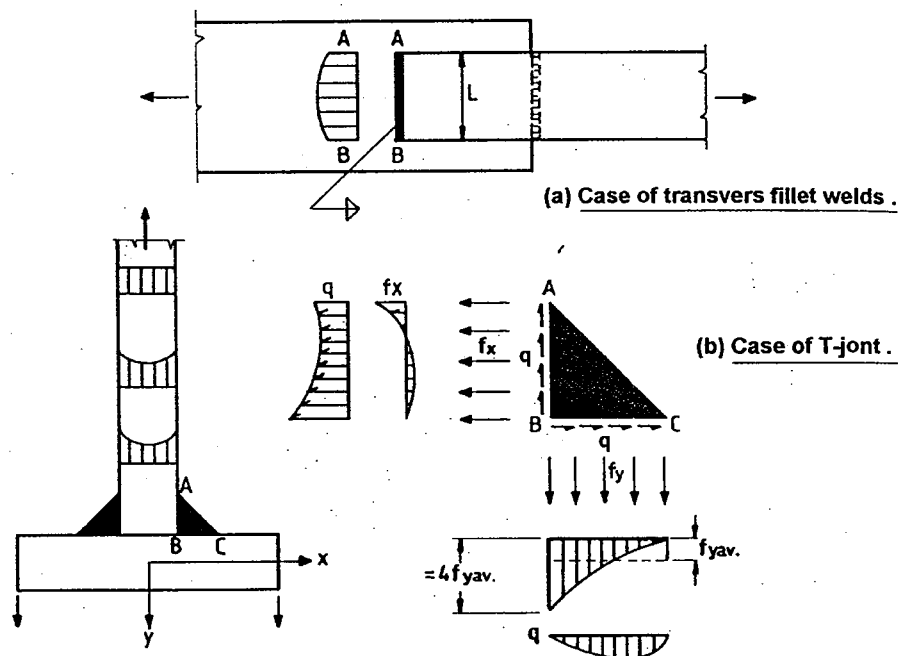


Fig. 3.16. Stress Distribution in Transverse Fillet Welds

Exact analysis of such types of connections would be tedious. In order to eliminate complications of analysis, we consider a uniform distribution of shear stresses along all lines of fillet welds. This means that equation 3.3 is valid for transverse and tee-joint fillet welds. The allowable stresses of all kinds must not exceed $0.2 F_u$.

3.6.4. Different Types of Welded Concentric Shear Connections:

The different configurations of fillet welded connections subjected to axial shear are shown in Fig. 3.17.

Type	Configuration	Straining Actions
1-Truss Connection.		F
2-Splice of Truss Chords.		$F_4/2$
3-Continuity of Beams.		$F=M/d$
4-Built-Up Section.		Shear Flow
5-Base Column.		H

Fig. 3.17 Concentric Shear Welded Connections.

Refer to The Following Examples:

Example 3.1: Fillet welded connection of a single member.

Example 3.2: Fillet welded connection of an unloaded joint.

Example 3.3: Fillet welded connection of a loaded joint.

Example 3.4: Fillet welded connection of a loaded joint (welding embedded).

3.10.4. Different Limitations Regarding Fillet Welds :-

3.10.4.1. Deposited Fillet Weld Metal :-

a) The limiting angles between fusion faces for load transmission shall not be greater than 120° and not less than :

- 60° for flat and down hand welding.
- 70° for vertical welding.
- 80° for overhead welding.

b) The minimum leg length of fillet weld as deposited shall not be less than the specified size. The throat of a fillet as deposited shall be not less than $6/10$ and $9/10$ of the minimum leg length in the case of concave and convex fillets respectively as shown in Fig. 3.18

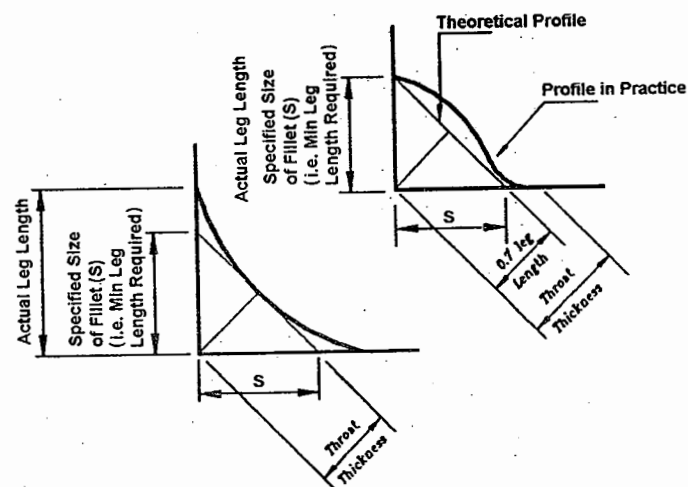


Fig. 3.18.

3.10.4.2. Size of Fillet Welds :-

a) The maximum size of fillet weld should not exceed the thickness of the thinner plate to be welded.

b) It is recommended that the following limitations in sizes of fillet welds as related to the thicker part to be joined should be observed as shown in Fig. 3.19.

- . Below 20 mms. size "S" ≥ 5 mms.
- . 20 - 30 mms. size "S" ≥ 6 mms.
- . 30 - 50 mms. size "S" ≥ 8 mms.
- . 50 - 100 mms. size "S" ≥ 10 mms.

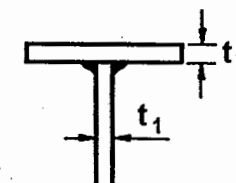


Fig. 3.19.

c) The minimum size of fillet welds for buildings is 4 mms. and for bridges is 6 mms.

3.10.4.3. Fillet Weld Length :-

a) The effective length for load transmission should not be less than 4 times the weld size (s) or 5 cms whichever is largest.

b) The maximum effective length of fillet welds should not exceed 70 times the size. Generally in lap joints longer than 70 s a reduction factor β allowing for the effects of non-uniform distribution of stress along its length is to be utilized where :

$$\beta = 1.2 - 0.2L / 70s \quad (3.7)$$

L = Overall length of the fillet weld ; $\beta < 1.0$

c) There are no limitations for the length of fillet weld for beam to column connections as well as for the flange to web weld in welded built up plate girders (see Fig. 3.20.).

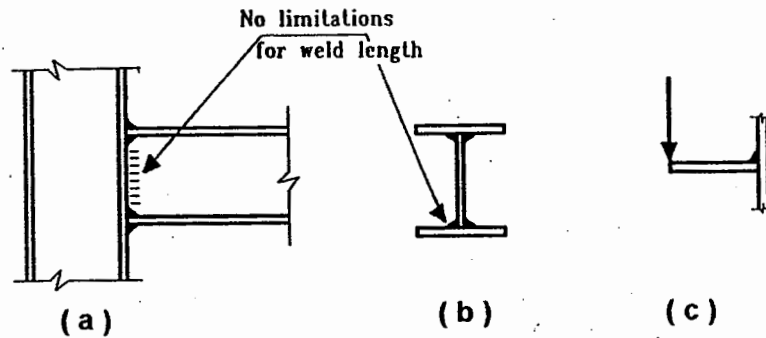


Fig. 3.20.

3.10.4.4. Single Fillet Weld :-

a) Single Fillet welds subjected to normal tensile stress perpendicular to the longitudinal direction of the weld are not to be utilized (Fig. 3.20.c).

b) For single side fillet weld between the flanges and web in I girders the weld shall be made with a penetration of at least half the web thickness.

c) For single side fillet flange to web weld, this fillet weld shall be completed on the other side of the web and made symmetrical at supports and at the position of concentrated loads where the web is not stiffened by vertical stiffeners.

d) Single side fillet welds may be utilized only for static loads.

3.10.4.5. Intermittent Fillet Welds :-

Intermittent fillet welds may be used to carry calculated stresses. The following restrictions are to be applied :-

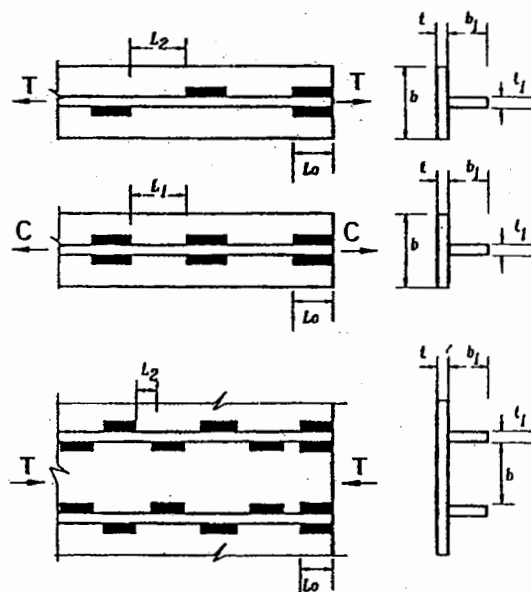
a) Intermittent welds shall not be used in parts intended to transmit stresses in dynamically loaded structures.

b) The clear distance between effective length of consecutive intermittent fillet welds, whether chained (L_1) or staggered (L_2), shall not exceed 12 times the thickness of thinner part in compression or 16 times in tension and in no case shall it exceed 20 cms (see Fig. 3.21).

c) In a line of intermittent fillet welds, the welding shall extend to the ends of the connected parts. For staggered welds this applies generally to both edges but need not apply to subsidiary fittings or components such as intermediate stiffeners.

d) For a member in which plates are connected by means of intermittent fillet welds, a continuous fillet weld shall be provided on each side of the plate for a length (L_0) at each end equal to at least three quarters of the width of the narrower plate connected (see Fig. 3.21).

e) Bridge stiffeners and girder connections are permitted to be directly welded with the compression flange, in the case of the tension flange, intermediate plates shall be inserted between the flange and the stiffeners in order to prevent weakening of the flange by transverse welds. Where intermittent welds are used, the clear distance between consecutive welds, whether chained or staggered shall not exceed 16 times the thickness of the stiffener. The effective length of such weld shall not be less than 10 times the thickness of the stiffener in the case of staggered welds and 4 times in the case of chained welds, or one quarter the distance between stiffeners whichever is smallest.



$L_0 \geq 0.75 b$ or $0.75 b_1$ - whichever is smaller.

$L_1 \text{ \& } L_2 \leq 16 t$ or $16 t_1$ or 200mm - whichever is smallest.
(tension member)

$L_1 \text{ \& } L_2 \leq 12 t$ or $12 t_1$ or 200mm - whichever is smallest.
(compression member)

Fig. 3.21. Intermittent Fillet Welds.

3.11. Eccentric Fillet Welded Connections

"Shearing Force and Twisting Moment":

3.11.1. Behavior and Analysis:

Referring to Fig. 3.22, the weld is acted upon by an eccentrically applied shearing force. The conventional method

for proportioning such connections is similar to the method used in riveted and bolted connections as previously outlined in section 1.7. The torsional moment ($M_t = P.e$) and the concentric shearing force (P) are treated separately. Then the resulting stresses are superimposed.

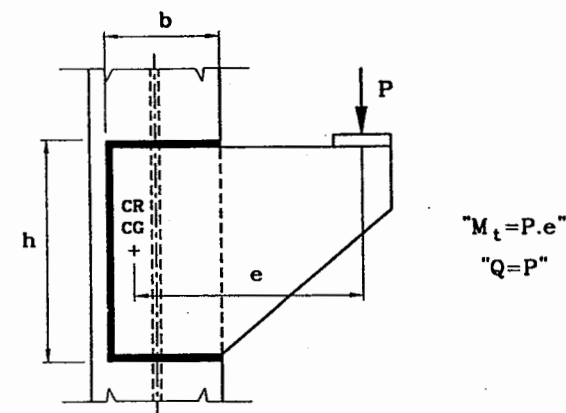


Fig.3.22. Eccentric Fillet Welded Connection

The following assumptions will be considered:

- The center of rotation of the group of welds coincides with their center of gravity (C.G).
- The shearing force is assumed to be uniformly distributed over the full length of the group of welds.
- The torsional shear stress in the weld varies linearly from the centroid of the effective weld area and is acting normal to the line joining the weld point to the center of gravity.

In brief the above mentioned assumptions simplify the procedure of analysis into the following:

a- Determine the center of gravity of the group of welds by treating the welds as linear elements having unit thickness ($S=1$). Table 3.6 gives formulae for the determination of position of the center of gravity for different configurations.

b- The stress due to the direct shear "P" will follow the relation:

$$q_Q^\downarrow = P / A_w \quad (a)$$

Where A_w = Summation of lengths of the lines of welds.
 $= s \cdot (h + 2b)$.

c- The shear torsional stress can be computed from the relation:

$$q_{Mt}^\downarrow = M_t \cdot r / I_p \quad (b)$$

Where M_t = the torsional moment = $P \cdot e$

r = Radial distance from the centroid to the point

Where the stress is to be calculated.

I_p = Polar moment of inertia of the group of welds.
 $= I_x + I_y$

The polar moment of inertia is illustrated in Table 3.6, for different weld configurations. The weld size "S" is taken equal to unity.

It has to be noticed that the most stressed weld point is the farthest weld point with respect to the center of gravity.

For simplicity it is convenient to use cartesian coordinates as indicated in Fig. 3.23.

Equation (b) will take the form :

$$q_{Mt,x}^{\rightarrow} = \frac{M_t \cdot h/2}{I_p} \quad (c)$$

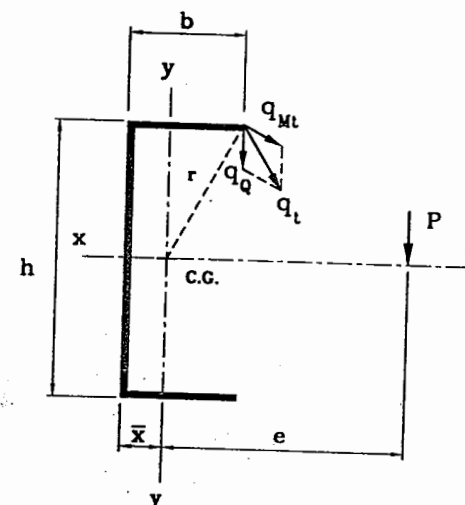
$$q_{Mt,y}^{\downarrow} = \frac{M_t \cdot (b-\bar{x})}{I_p} \quad (d)$$

Where $M_t = P \cdot e$

The resultant shear stress q_t must not exceed the allowable values prescribed by the E.C.P. 2001 :-

$$q_t \cdot s = \sqrt{\left(\frac{P}{A_w} + \frac{M_t \cdot (b-\bar{x})}{I_p}\right)^2 + \left(\frac{M_t \cdot h/2}{I_p}\right)^2} \leq 0.2 F_U \quad (3.8)$$

In the above equation, one may use " $q_t = 0.2 F_U$ " in order to determine the safe size "s" of the group of welds.



" $M_t = P \cdot e$ "

Fig. 3.23

Table 3.6 Sectional Properties of Fillet Welds (Unit Thickness)

No.	Type of Sec. b=width h=height	C.G. Location ($\bar{X}$, $\bar{Y}$)	Sec. Modulus $Z=I_x/\bar{Y}$	Polar Moment of Intertia $I_p=\text{at C.G.}$
1		$\bar{X} = \bar{Y} = 0$	$Z = \frac{h^2}{6}$	$I_p = \frac{h^3}{12}$
2		$\bar{X} = \bar{Y} = 0$	$Z = \frac{h^2}{3}$	$I_p = \frac{h(3b^2+h^2)}{6}$
3		$\bar{X} = \bar{Y} = 0$	$Z = bh$	$I_p = \frac{b(3h^2+b^2)}{6}$
4		$\bar{X} = \frac{b^2}{2(b+h)}$ $\bar{Y} = \frac{h^2}{2(b+h)}$	$Z = \frac{4bh+h^2}{6}$	$I_p = \frac{(b+h)^4 - 6b^2h^2}{12(b+h)}$
5		$\bar{X} = \frac{b^2}{2b+h}$ $\bar{Y} = 0$	$Z = bh + \frac{h^2}{6}$	$I_p = \frac{8b^3+6bh^2+h^3}{12} - \frac{h^4}{(2b+h)}$
6		$\bar{X} = 0$ $\bar{Y} = \frac{h^2}{2h+b}$	$Z = \frac{2bh+h^2}{3}$	$I_p = \frac{b^3+6bh^2+8h^3}{12} - \frac{h^4}{(2h+b)}$
7		$\bar{X} = \bar{Y} = 0$	$Z = bh + \frac{h^2}{3}$	$I_p = \frac{(b+h)^3}{6}$
8		$\bar{X} = 0$ $\bar{Y} = \frac{h^2}{2h+b}$	$Z = \frac{2bh+h^2}{3}$	$I_p = \frac{8h^3+b^3}{12} - \frac{h^4}{2h+b}$
9		$\bar{X} = \bar{Y} = 0$	$Z = bh + \frac{h^2}{3}$	$I_p = \frac{b^3+3b^2+h^3}{6}$
10		$\bar{X} = \bar{Y} = 0$	$Z = \pi r^2$	$I_p = 2\pi r^3$

Type	Configuration	Straining Action
1-Bracket Connection.		$Q=P$ $M_t=P.e$
2-Flexible Beam to Column Connection.		$Q=R$ $M_t=R.e_2$
3-Flexible Secondary to Main Beam Connection.		$Q=R$ $M_t=R.e_2$ $M=R.e_1$
4-Splice of Beam's or Column's Web.		$Q=Q$ $M_t=M_w+Q.e$ $M_w = \frac{M.I_w}{I}$

Fig. 3.24 Eccentric Shear Welded Connections.

3.11.2. Different Types of Eccentric Welded Connections:

Fig. 3.24 depicts the various types of eccentric welded connections.

Refer to the following examples:

Example 3.5: Bracket.

Example 3.6: Seat angle.

Example 3.7: Beam-column flexible welded connection.

3.12. Fillet Welded Connections Subjected to Bending Stresses:

3.12.1. Analysis and Behavior:

In this section, the weld line through section (1-1) will be subjected to bending as shown in Fig. 3.25.a. While weld line through section (2-2) will be subjected to torsion.

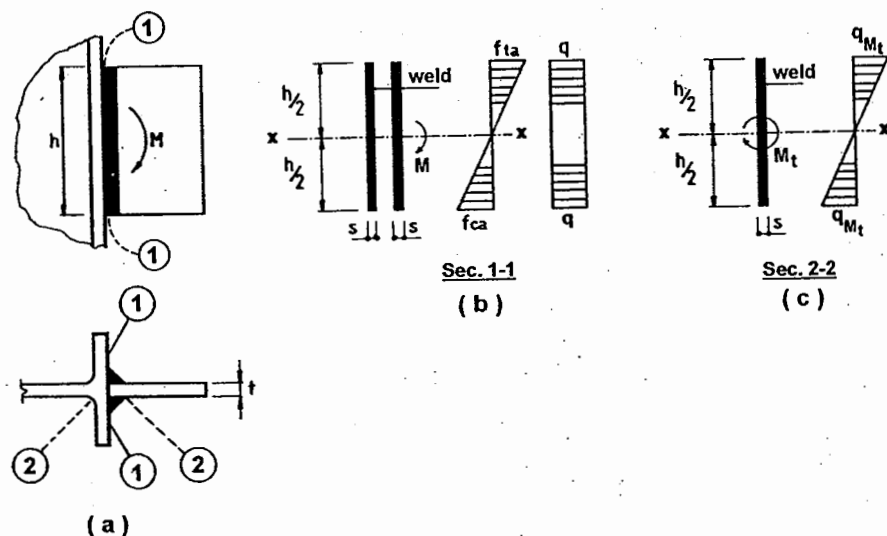


Fig. 3.25. Fillet Welds Subjected To Bending

(i) Weld lines through section (1-1) :-

The moment "M" is considered to be completely transmitted via to the lines of weld which are acting as rectangular elastic beam elements (see Fig. 3.25.b). The maximum stress is regarded as bending stress along section 1-1, therefore:

$$f_{ta_{\perp}} = f_{tc} = M / Z_x = 3M / sh^2 \leq 0.2 F_U \quad (3.9)$$

Where :

$$Z_x = I_x / (h/2) = (2sh^3/12) / (h/2) = sh^2/3.$$

(ii) Weld lines through section (2-2) :-

The fillet weld through section 2-2 will be subjected to a torsional moment $M_t = M$ and the stresses are to be calculated as given in section 3.11.1. (see Fig. 3.25.c).

$$q_{Mt_{\perp}} = \frac{M (h/2)}{2 I_p} \approx \frac{M (h/2)}{2 I_x} \approx \frac{M}{2 Z_x}$$

Where :

$$I_p \approx I_x ; I_x = s.h^3/12 ; Z_x = s.h^2/6$$

$$q_{Mt_{\perp}} = \frac{M}{2(s.h^2/6)} = \frac{3M}{s.h^2} \leq 0.2 F_U \quad (3.10)$$

If a concentric shearing force "Q=P" is applied in addition to the moment, the induced shear stresses are computed as follows:

$$q_Q^{\perp} = P/2.s.h$$

and either the principal or the equivalent stresses would be checked on section 1-1, while on section 2-2 the vector

sum of shear stresses should be computed.

Refer to the following Examples:

Example 3.6: Seat angle. ..(Refer to chapter 4 for more detailed-analysis)

Example 3.7: Beam-column flexible welded connection.

Example 3.8: Beam-Column rigid welded connection.

3.13. Plug Welds, Slot Welds, and Fillet Welds in Holes and Slots:

3.13.1. Behavior and Analysis :-

Plug and slot welds (see Fig. 3.26) are used most often to tie two plates together and, in particular, to reduce the unsupported dimensions of cover plates in compression. This is useful to prevent overlapping parts from buckling. They also may be used for shear transmission. This use is generally reserved for locations where it is impractical to use a fillet weld.

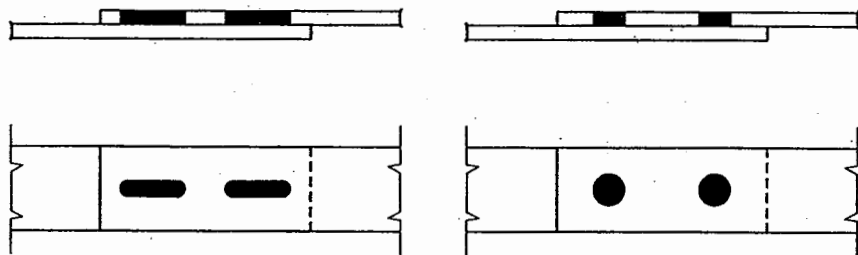


Fig. 3.26. Plug and Slot Welds

The critical section for either a plug weld or a slot weld is the faying surface between the connected parts. The unit shearing resistance on this section is essentially the same as that of a fillet weld as has been mentioned in section 3.10 Equ. 3.6.

The following observations are to be considered:

- (i) Plug and slot welds should not be used to transmit tension, that is, a force normal to the faying surface. Tensile resistance depends largely upon the degree of penetration of the weld, which is apt to be rather uncertain in either a plug or a slot weld.
- (ii) Plug and slot welds are distrusted because of the difficulty of inspection.
- (iii) Plug and slot welds appear to be excellent on the surface but may contain voids at the critical section (Fig.3.27).

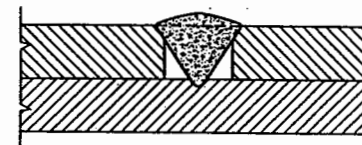


Fig. 3.27

Welds of the type shown in Fig. 3.28, are similar in appearance to slot welds but are simply fillet welds placed around the edges of slots cut in the plate to increase the length of edge available for welding.

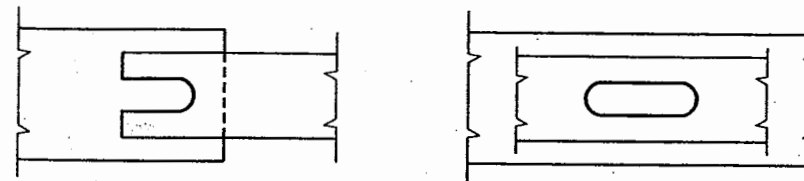


Fig. 3.28. Fillet Welds in Slots



3.13.2. Proportions and Spacing of Holes and Slots :-

The proportions and spacing of holes are illustrated in Fig. 3.29. and Table 3.7.

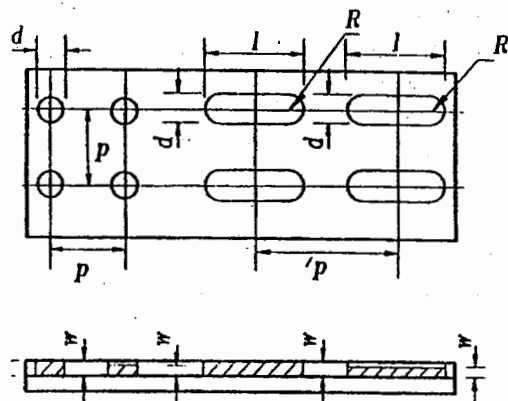


Fig. 3.29.

The minimum and maximum of the diameter (d), the length of plug weld (L), the spacing (p) and (p') between slot and plug welds as well as the depth of the filling weld metal (W) are as given in Table 3.7.

Table 3.7

Plate thickness (t) mm	Min. Hole dia. or slot ($d_{\min}$) mm	Holes and Slot proportions spacing and depth of weld	
5 & 6	14	$d \geq (t+8)$ mm preferably rounded next higher odd 2 mm; also $d \leq 2.25 t$ or $d_{\min} + 3$ mm whichever is greater	
7 & 8	16	$p \geq 4d$	Depth of filling of plug or slot weld :- - Where $t \leq 16$ mm, $W = t$ - Where $t \geq 16$ mm, $W = \frac{t}{2}$ but not less than 16 mm
9 & 10	18	$p' \geq 2L$	
11 & 12	20	$L \leq 10t$	
13 & 14	22	$R = d/2$	
15 & 16	24	$R \geq t$	



N.B. It is to be noted that there are no limitations for the edge distances where the ends of slot and plug welds may extend to the edge of the connected parts.

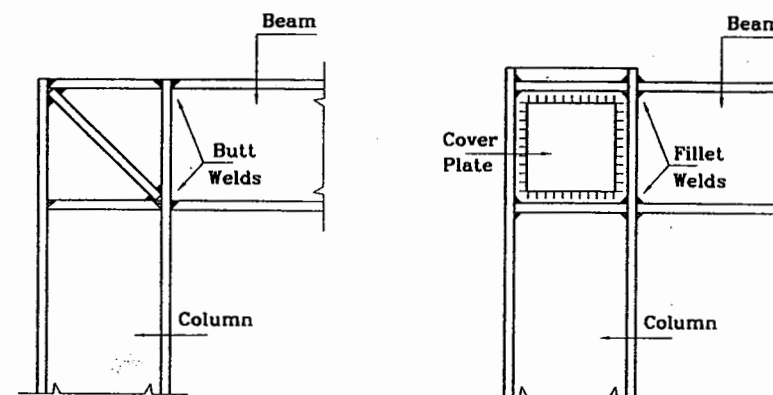
3.14. Welded Moment Corner Connections:

The most simple rigid welded moment connection is the corner beam to column connection. The beam is connected to the column by either direct butt weld (Fig. 3.30.a) or by lines of fillet welds as shown in Fig. 3.30.b.

The butt weld is the simplest rigid connection, no analysis is needed in a fully butt welded joint for, if the weld metal is as strong as the parent metal, it is simply, a continuation of the beam. Butt weld requires precision in fabrication and fitting-up that may render it less desirable than other types.

3.14.1. Behaviour and Analysis :-

Three Problems are associated with this type of connections which are as follows:



(a) Butt Weld Configuration.

(b) Fillet Weld Configuration.

Fig. 3.30

- (i) Crippling of the web of the column at the vicinity of the compression flange of the beam (Fig. 3.31.a).
- (ii) The bending of the column flange opposite to the beam's tension flange (Fig. 3.31.b).
- (iii) The instability or the shear resistance at corners of the column web (Fig. 3.31.c).

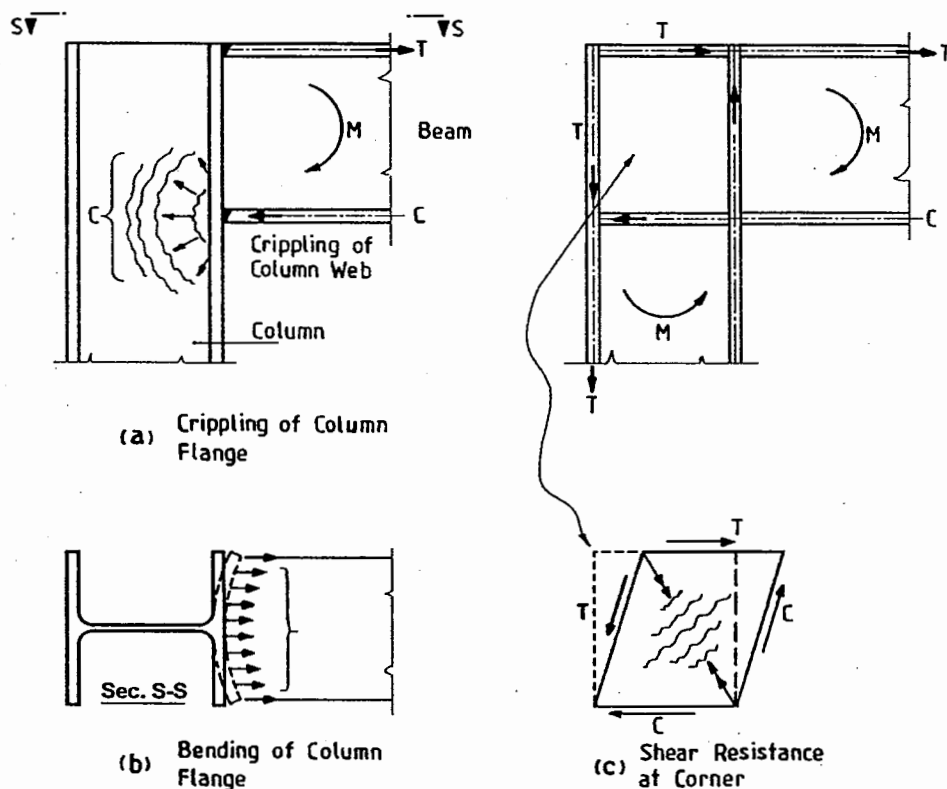


Fig. 3.31

(i) Crippling of Column Web:

As has been previously outlined, the crippling of the column web will exist in the region where the beam flange causes compression. The behavior of the beam-column connection is similar to that of the beams treated previously in Chapter (5) volume (1)

Assuming the beam and the column to be of the same material, the parameters which one would suspect to control the possibility of the column web crippling or yielding are:

- (a) The beam flange dimensions.
- (b) The column web thickness.
- (c) The column flange fillet dimension "K".

For the analysis consider a beam compression flange bearing against a column flange as shown in Fig. 3.32.

Assume that when the compression flange reaches the yield stress the corresponding resultant compressive force C is distributed along the base of the fillet (K) on a 1:2.5 slope as shown in Fig. 3.32.a. If the induced stresses along the distance $(t_b + 5k)$ of the column web do not exceed the yield stresses, one can deduce that the thickness of the column web is sufficient to withstand crippling. This behavior can be formulated as follows:

$$C_u = \text{the ultimate applied compressive force} = b_b \cdot t_b \cdot F_y$$

$$F_w = \text{the ultimate capacity of the column web} = (t_b + 5K) \cdot t_{wc} \cdot F_y$$

Equating $C_u = F_w$ we can compute the thickness of the column web t_{wc} required to prevent crippling: (see clause 6.9.4 E.C.P. 2001).

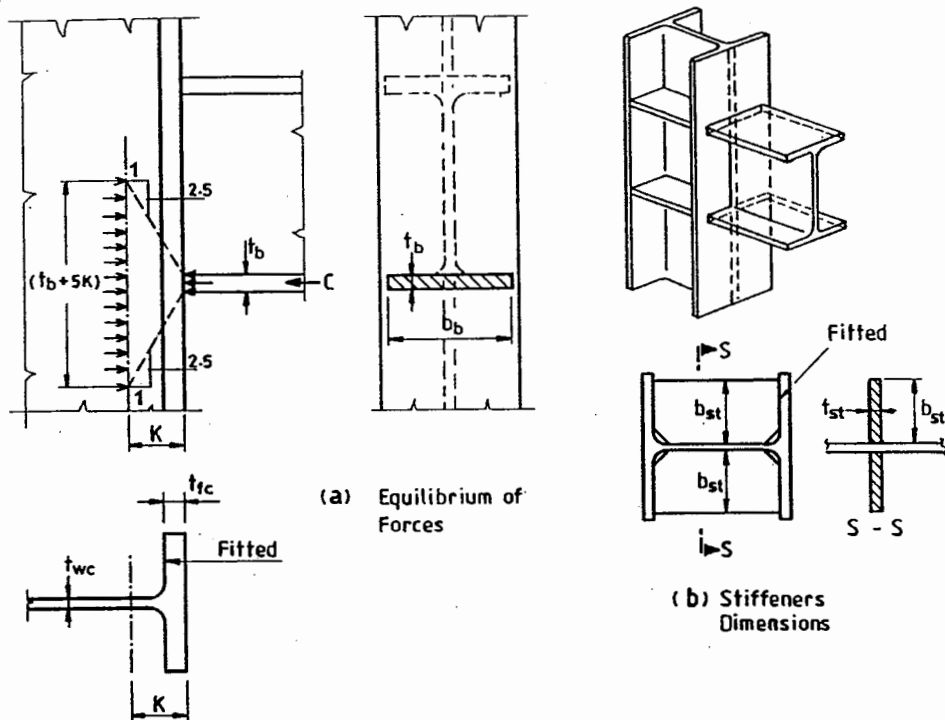


Fig. 3.32. Crippling Phenomenon

i.e;

$$t_{wc} \geq \frac{b_b t_b}{(t_b + 5K)} \quad (3.11)$$

If the condition of equation (3.11) is not satisfied, crippling will occur and a horizontal pair of stiffeners are to be provided. For the determination of the cross sectional area of these stiffeners the following relation is to be utilized: (see Fig. 3.32.b)



$$A_{st} = b_b \cdot t_b - (t_b + 5K)t_{wc} \quad (3.12.a)$$

$$A_{st} = 2 b_{st} \cdot t_{st}$$

b_{st} = width of the horizontal plate stiffener

t_{st} = thickness of the plate stiffener.

These horizontal stiffeners are to be well fitted to the column flange and in order to prevent the local buckling the ratio b_{st}/t_{st} must follow :-

$$b_{st} / t_{st} \leq 25 / \sqrt{F_y} \quad (3.12.b)$$

(ii) The Bending of the Column Flange:

At the vicinity of the beam tension flange, the column flange tend to bend outward as shown by the dotted deflected shape of Fig. 3.33.

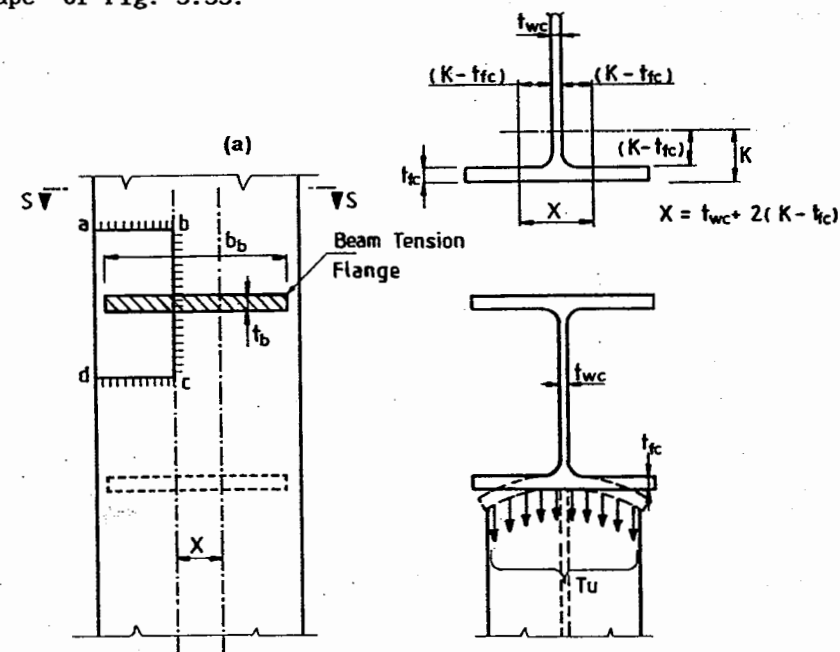


Fig. 3.33



The parameters which one would suspect to control the possibility of the column flange bending are:

- 1- The beam flange dimensions inducing the tensile force.
- 2- The column flange thickness " t_{fc} ".

A yield-line analysis have been applied on a portion of the column flange as shown by the rectangle abcd of Fig. 3.33.a.

The results of the yield-line analysis assuming the sides ab, bc and cd fixed have proved that the ultimate load or the ultimate capacity (P_u) is:

$$P_u = \alpha \cdot F_y \cdot t_{fc}^2 \quad (a)$$

Where α is a factor which varies from 3.5 to 5.0 for the common beam-to-column connection configurations.

If the lower limit of the value of α is applied in equation (a) we get:

$$P_u = 3.5 \cdot F_y \cdot t_{fc}^2 \quad (b)$$

The total breadth of the column flange can be replaced by three portions: two similar portions behaving each as previously given in equation (b) and a central portion of width "X" assumed to be stressed to the yield stress. Hence the total ultimate capacity can be computed as follows:

$$P_u = 2(3.5 F_y \cdot t_{fc}^2) + X \cdot t_b \cdot F_y$$

$$\text{i.e.} \quad P_u = 7 \cdot F_y \cdot t_{fc}^2 + X \cdot t_b \cdot F_y \quad (c)$$

The ultimate applied tension force T_u can be written in



the following form:

$$T_u = b_b \cdot t_b \cdot F_y \quad (d)$$

Equating $P_u = T_u$ we get:

$$t_{fc}^2 \geq \frac{b_b \cdot t_b \cdot X}{7} \quad (e)$$

For the majority of beam-to-column connections X/b_b varies from 0.15 to 0.2, one can choose the lower limit and apply in equation (e) to get the required thickness of the column flange to prevent bending. Hence:

$$t_{fc}^2 \geq \frac{0.85 b_b t_b}{7}$$

$$\text{i.e.} \quad t_{fc} \geq 0.36 \sqrt{b_b t_b} \quad (f)$$

One can decrease the ultimate capacity of equation (c) by 20 percent. This decrease can be considered somewhat as a factor of safety and hence equation (f) becomes:

$$t_{fc} \geq 0.4 \sqrt{b_b t_b} \quad (3.13)$$

(iii) Instability or Shear Resistance of the Corner Web:

The main problem here is the abrupt change in direction of flanges. It is to be ensured that the applied straining actions (M, N and Q) are safely transferred from one structural steel element to the other.

Fig. 3.34, illustrates a square corner connection where the moment "M" is replaced by two opposite and equal forces "T" and "C". For example the tension force "T" is transferred from the beam to the column as shear through the corner web.

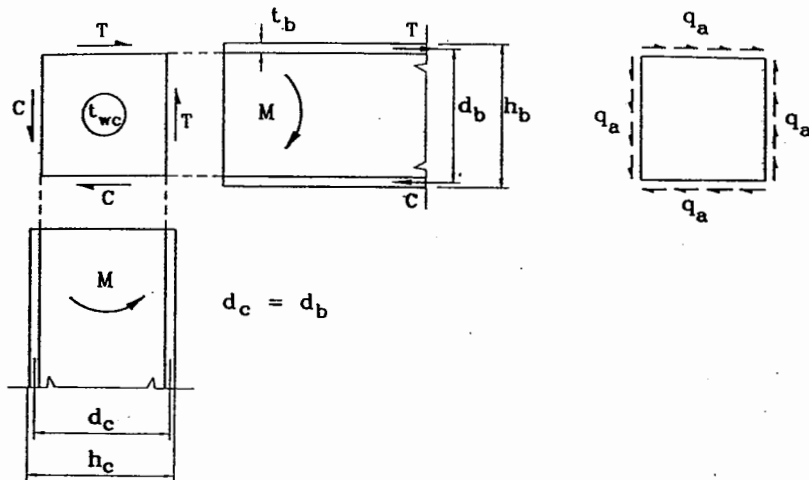


Fig. 3.34

Having replaced the moment "M" by two equal and opposite forces, we can write the following:

$$T = C = \frac{M}{d_b} \quad (a)$$

$$\text{Where } d_b = h_b - t_b$$

These forces T and C are transferred as shear into the corner web within the distance d_c as shown in Fig. 3.34. The resulting shear stress " q_a " within the corner web is:

$$q_a = \frac{T}{t_{wc} \cdot h_c} \quad (b)$$

Where h_b, h_c = height of beam and column cross sections respectively.

t_{wc} = the thickness of the corner web.

Combining equations (a) and (b) we get the following condition :-

$$t_{wc} \geq \frac{M / d_b}{(0.35 F_y) \cdot h_c} \quad (3.14)$$



If Equ. 3.14 is not satisfied this means that the actual shear stress exceeds the permissible stress (i.e. $0.35 F_y$), the actual thickness of the corner web " t_{wc} " is insufficient. Some way must be found to accommodate this situation by choosing one of the alternatives (a) or (b).

(a) The Use of Diagonal Stiffeners:

A pair of diagonal stiffeners must be added to transfer the remaining of the flange force "T" as a diagonal component.

The portion of the flange force " F_a " which may be transferred as shear within the web by stressing it to the permissible limit " $q = 0.35 F_y$ " is to be calculated (see Fig. 3.35).

$$F_a = (0.35 F_y) \cdot h_c \cdot t_{wc} \quad (c)$$

The remaining force F_e is computed using equations (a) and (c):

$$F_e = \frac{M}{d_b} - (0.35 F_y) \cdot h_c \cdot t_{wc} \quad (d)$$

Hence the diagonal remaining force F_D induced in the diagonal stiffener as shown in Fig. 3.35 is:

$$F_D = \left[\frac{M}{d_b} - (0.35 F_y) \cdot h_c \cdot t_{wc} \right] / \cos \theta \quad (3.15)$$

The required diagonal stiffening area is:

$$A_{st} = 2 b_{st} \cdot t_{st} = \frac{F_D}{F_c} \quad \text{with} \quad (b_{st} / t_{st} \leq 25 / \sqrt{F_y})$$

where

F_c = the allowable compressive stress = $0.58 F_y$.

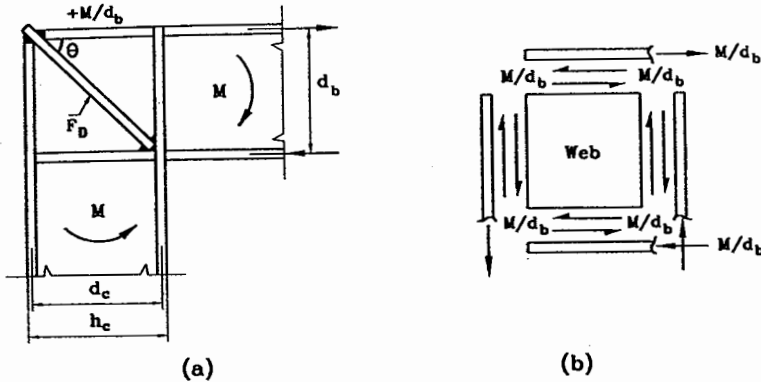


Fig: 3.35

(b) The use of Additional Cover Plates:

If the applied shear stresses of equation (3.14) exceeds the allowable limit the other possibility is to provide additional cover plates of thickness (t_c) welded to the corner web.

Replacing t_{wc} by the new developed thickness ($t_{wc} + t_c$) in equation (C) and equating with equation (a) we get:

$$\frac{M}{d_b} = (0.35 F_y) \cdot h_c \cdot (t_{wc} + t_c) \quad (3.16)$$

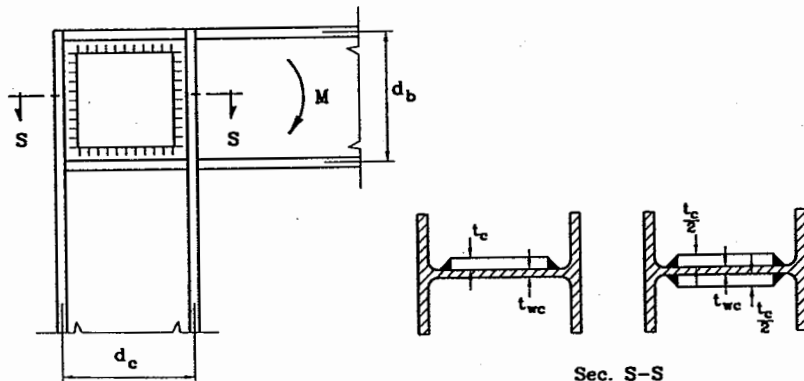


Fig. 3.36



In equation (3.16) the permissible shear stress "q" is replaced by $(0.35 F_y)$ and hence the thickness of the cover plate " t_c " can be computed. (see Fig. 3.36).

3.14.2. Different Types of Welded Moment Connections:

Various types of welded moment connections are given in Fig. 3.37.

Refer to the following Examples:

Example 3.9: Fillet welded corner connection.

Example 3.10: Butt welded corner connection.

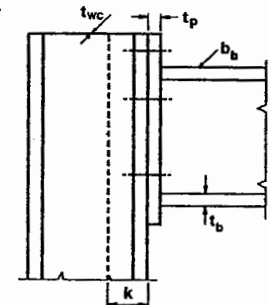
Example 3.11: Haunched fillet welded corner connection.

3.14.3 General Remarks regarding the coener connection :-**1. Beam-to-column connection (Rolled Sections) :-**

For beam-to-column connections with end plates of thickness (t_p) connected to the column flange with high strength bolts Equ. 3.11 to be modified to the following form :-

$$t_{wc} \geq \frac{b_b \cdot t_b}{t_b + 2t_p + 5k} \quad (\text{as 3.11})$$

while Eqs. 3.12, 3.13, 3.14, 3.15 and 3.16 remain unchanged

**2. Corner welded connections (column is a built up section) :-**

Eqs. 3.11, 3.12 and 3.13 remain unchanged while in Eqs. 3.14, 3.15 and 3.16 h_c (height of column cross section) is to be replaced by the height of the web only ($h_c - t_{fc}$). This is due to the fact that the column web shear resistance for rolled sections is $(h_c \cdot t_{wc} \cdot 0.35 F_y)$, while for built up sections the shear resistance equals $[(h_c - 2t_{fc}) \cdot t_{wc} \cdot 0.35 F_y]$.

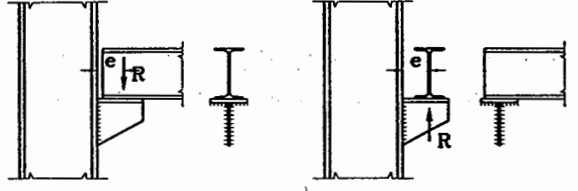
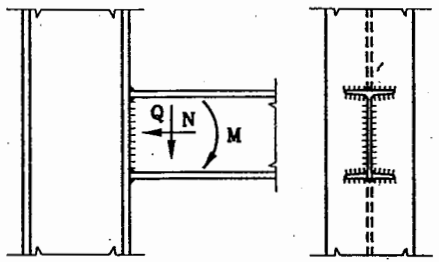
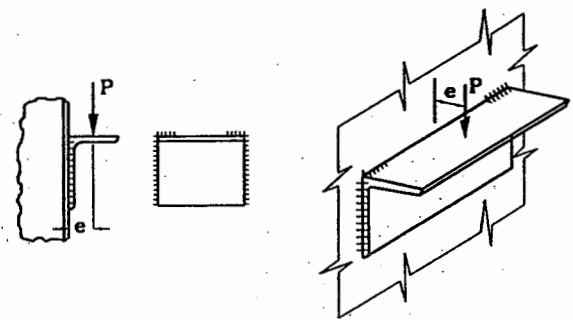
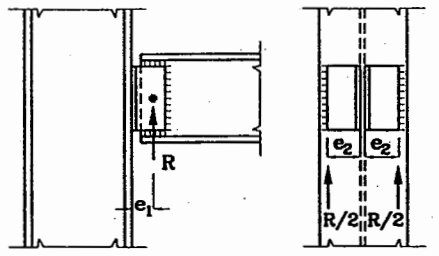
Type	Configuration	Straining Actions
1-Bracket		$Q=R$ $N=Zero$ $M=R.e$
2-Beam to Column Rigid Connections.		$Q=Q$ $N=N$ $M=M$
3-Seat Angle Connection.		$Q=R$ $N=Zero$ $M=R.e$
4-Beam to Column or Beam to Beam Flexible Connections.		$Q=R/2$ $N=Zero$ $M=R.e_1/2$

Fig. 3.37 Welded Moment Connections.

3.15. Technological Considerations:

It can be noticed that the previous sections have presented the different types of welds, the strength and the design. Concerning the factors affecting the quality of welded connections, the following restrictions are to be applied in order to avoid unfavourable weld details.

3.15.1. Lamellar Tearing :

Lamellar tearing is a separation (or crack) in the base metal, caused through thickness weld shrinkage strains.

The probability of this failure can be minimized by :-

- Using small weld size providing the shrinkage to be accommodated.
- The welding procedure should also establish a welding sequence such that the component restraint and internal restraints in the weldment are held to minimum.
- The use of welding procedure with low hydrogen weld and an effective preheating minimize lamellar tearing.

Some joints susceptible to lamellar tearing can be improved by careful detailing as shown in Fig. 3.38.

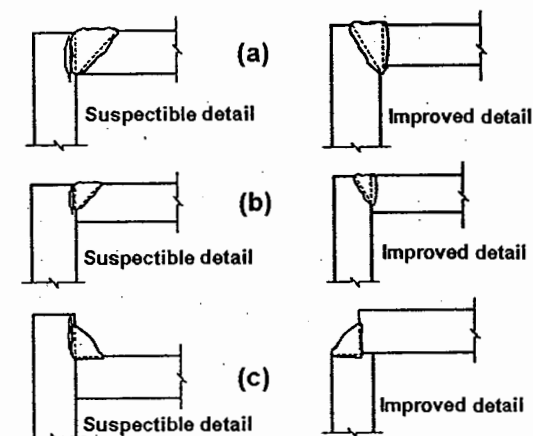


Fig.3.38. Improved Welded Connections to Reduce Lamellar Tearing



3.15.2. Notches and Brittle Fracture :

The one sided fillet welds can result in severe notches as shown in Fig.39.a. The remedy is to use two fillets one on each side. A similar condition arises with partial penetration groove welds.

Backing bars can cause a fatigue weld notch if they are welded as shown in Fig.39.b. A remedy would be to weld in the groove as in Fig.39.c, where any undercut would be filled, or at least backed up by the final weld joint. The backing bars should also be continuous through its length.

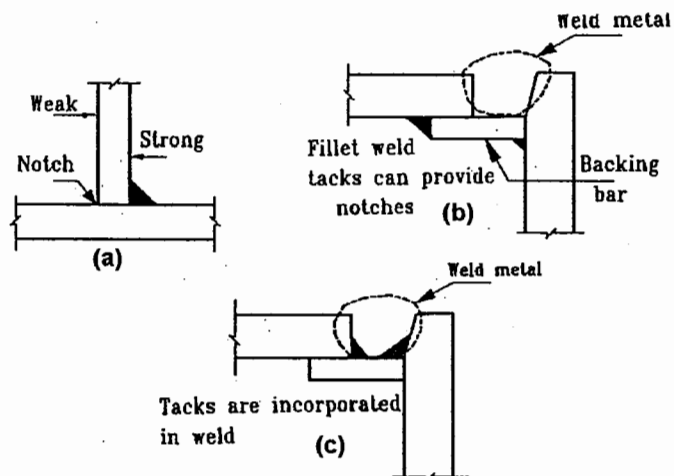


Fig.3.39. Notches and Brittle Fracture.

3.16. Weld Inspection Methods :

The designer must specify in the contract document the type of weld inspection required as well as the extent and application of each type of inspection.

Table 3.8 summarizes the characteristics and capabilities of the five most commonly used methods for welding inspection.



Table 3.8 Characteristics of Common Weld Inspection Methods.

Inspection Method	Characteristics and Applications	Limitations
Visual (VT)	Most common, most economical. Particularly good for single pass.	Detects surface imperfections only.
Dye Penetrant (DPT)	Will detect tight cracks, open to surface.	Detects surface imperfections only. Deep weld ripples and scratches may give false indications.
Magnetic Particle (MT)	Will detect surface cracks and subsurface cracks to about 2 mm depth with proper magnetization. Indications can be preserved on clear plastic tape.	Requires relatively smooth surface. Careless use of magnetization prods may leave false indications.
Radiographic (RT)	Detects porosity, slag, voids, irregularities, lack of fusion. Film negative is permanent record.	Detects must occupy more than about 1 1/5 % of thickness to register. Only cracks partial to impinging beam register. Radiation hazards. Exposure time increases with thickness.
Ultrasonic (UT)	Detects cracks in any orientation, Slag, lack of fusion, inclusions, lamellar tears, voids. Can detect a favorably oriented planar reflector smaller than 1mm. Regularly calibrate on 1 1/4 mm dia drilled hole. Can scan almost any commercial thickness.	Surface must be smooth, Equipment must be frequently calibrated. Operator must be qualified. Exceedingly coarse grains will give false indications. Certain geometric configurations give false indication of flaws.

3.17 Summary:

Table 3.9 summarizes the allowable stress according to the E.C.P. 2001 for major types of fillet welded connections.



Table 3.9 Fillet Welded Connections.

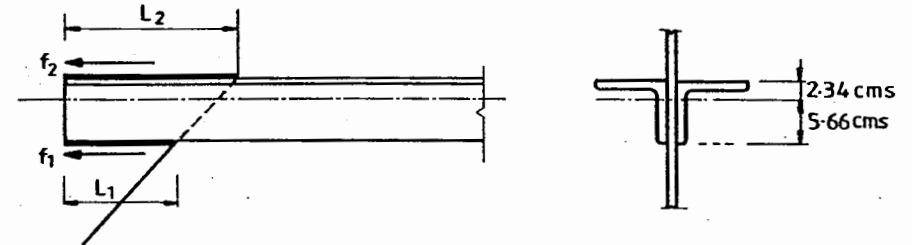
Type	Configuration	Straining Actions	Allowable Stresses (E.C.P. 2001)
1-Axial Shear Concentric Connection.		$Q = F_i$ $q_{w//} = F_i / L.S$	$q_{w//} \leq 0.2F_U$
2-Eccentric Connection.		$Q = P$ $M_t = P.e$ $q_{w//} = \frac{P}{\sum L.S}$ $q_{w.Mt} = \frac{M_t \cdot r_{max}}{I_p}$	$q_t = \text{Vertical Summation of } R_{w.Mt} \text{ and } R_{w.Q}$ $q_t \leq 0.2F_U$
3-External Pure Tension.		(1)-(1) $q_{w//} = F/2LS$ (2)-(2) $f_{w\perp} = F/2LS$	$q_{w//} \leq 0.2F_U$ $f_{w\perp} \leq 0.2F_U$
4-Combined Shear & Tension.		(1)-(1) $Q = F \cdot \cos\theta$ $T = F \cdot \sin\theta$ $q_{w//} = \frac{F \cdot \cos\theta}{2LS}$ $f_{w\perp} = \frac{F \cdot \sin\theta}{2LS}$	$\sqrt{3 q_{w//}^2 + f_{w\perp}^2} \leq 1.1(0.2F_U)$
5-Combined Shear, Tension & Bending Moment.		$Q = Q$ $M = M$ $N = N$ $q_{w//} = \frac{Q}{2h.S}$ $f_{w\perp} = \frac{M \cdot h/2}{I} + \frac{N}{\sum L.S}$	$\sqrt{3 q_{w//}^2 + f_{w\perp}^2} \leq 1.1(0.2F_U)$

* * * *



Example (3.1):

A truss member is composed of 2 angles b.to.b. (80x80x10) (Steel 37). The member is welded to a gusset plate 10 mms thick. Design the connection if the force is 37 tons. Geometrical length of the member is 250 cms.



Solution

$$F_1 = 37 \times \frac{2.34}{8} = 10.82 \text{ tons} \quad F_2 = 37 \times \frac{5.66}{8} = 26.18 \text{ tons.}$$

$$\text{Allowable stress for fillet weld} = 0.2 F_U = 0.72 \text{ t/cm}^2.$$

The thickness of angle being 10 mm.

$\therefore$ take size of weld $S = 10$ mms.

$$\therefore L_1 = \frac{F_1}{2 \times 0.72 \times 1} + 2S = \frac{10.82}{2 \times 0.72 \times 1.0} + 2 \times 1.0 = 9.5 \text{ cms.} < 70 \text{ s (o.k.)}$$

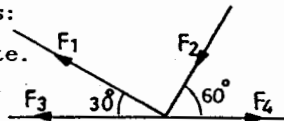
$$\therefore L_2 = \frac{F_2}{2 \times 0.72 \times 1} + 2S = \frac{26.18}{2 \times 0.72 \times 1.0} + 2 \times 1.0 = 20.18 \text{ cms.} < 70 \text{ s (o.k.)}$$

Example (3.2):

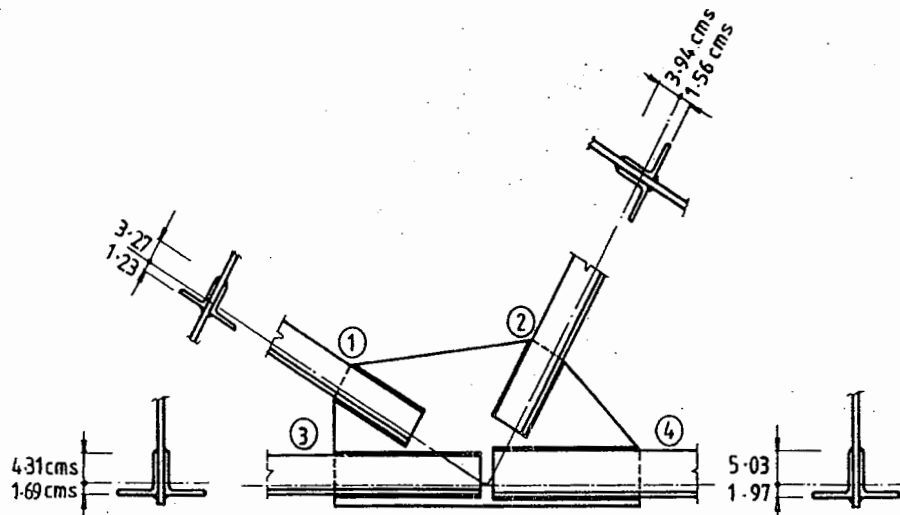
Design the given bottom chord connection (unloaded) using to two different concepts:

(I) Bottom chord members are separate.

(II) Bottom chord is continuous.



Member	1	2	3	4
D.F.	6.67 t (I)	3.85 t (I)	14.5 t (I)	22.2 t (I)
Cross.Section	2 < 45x45x5	2 < 55x55x6	2 < 60x60x6	2 < 70x70x7

SolutionA. Separate Connections(Unloaded Connection)Member (1):

$$F_1 = \frac{6.67 \times 1.23}{4.50} = 1.82 \text{ ton} \quad F_2 = \frac{6.67 \times 3.27}{4.50} = 4.846 \text{ tons}$$

Choose $S = 0.50$ cm (thickness of angle); $q = 0.2 \times 3.6 = 0.72$ t/cm²

$$L_1 = \frac{1.82}{2 \times 0.72 \times 0.50} + 2 \times 0.50 = 2.52 + 1.0 = 5.0 + 1.0 = 6.00 \text{ cms}$$

The effective length ≥ 5.0 cms or 4 s (o.k)

$$L_2 = \frac{4.846}{2 \times 0.72 \times 0.5} + 2 \times 0.50 = 6.72 + 1.0 = 7.72 \text{ cms}$$

Member (2):

$$F_1 = \frac{3.85 \times 1.56}{5.50} = 1.092 \quad F_2 = \frac{3.85 \times 3.94}{5.50} = 2.758 \text{ tons.}$$

Choose $S = 0.60$ cm.

$$L_1 = \frac{1.092}{2 \times 0.72 \times 0.60} + 2 \times 0.60 = 1.16 + 1.20 = 5.0 + 1.20 = 6.20 \text{ cms}$$

$$L_2 = \frac{2.758}{2 \times 0.72 \times 0.60} + 2 \times 0.60 = 3.18 + 1.20 = 5.0 + 1.20 = 6.20 \text{ cms}$$

Member (3):

$$F_1 = \frac{14.5 \times 1.69}{6.0} = 4.08 \text{ tons} \quad F_2 = \frac{14.5 \times 4.31}{6.0} = 10.41 \text{ tons}$$

Choose $S = 0.60$ cm.

$$L_1 = \frac{4.08}{2 \times 0.72 \times 0.60} + 2 \times 0.60 = 4.71 + 2 \times 0.60 = 5.0 + 1.20 = 6.20 \text{ cms.}$$

$$L_2 = \frac{10.41}{2 \times 0.72 \times 0.6} + 2 \times 0.60 = 12.04 + 2 \times 0.60 = 13.24 \text{ cms.}$$

Member (4):

$$F_1 = \frac{22.2 \times 1.97}{7.0} = 6.24 \text{ tons.} \quad F_2 = \frac{22.2 \times 5.03}{7.0} = 15.95 \text{ tons}$$

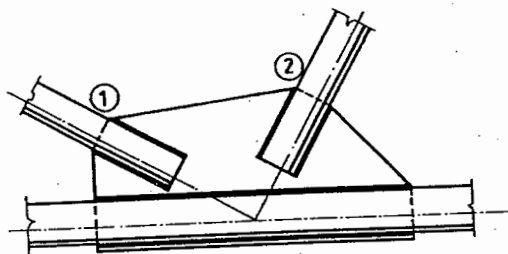
Choose $S = 0.70 \text{ cm.}$

$$L_1 = \frac{6.24}{2 \times 0.70 \times 0.72} + 2 \times 0.70 = 6.18 + 2 \times 0.70 = 7.58 \text{ cms.}$$

$$L_2 = \frac{15.95}{2 \times 0.70 \times 0.72} + 2 \times 0.70 = 15.81 + 2 \times 0.70 = 17.21 \text{ cms}$$

B. Lower Chord Continuous

(Unloaded Connection)



Lower Chord:

$$\begin{aligned} F (\text{lower chord}) &= F_4 - F_3 = 22.2 - 14.50 = 7.70 \text{ tons.} \\ &= F_1 \cos 30^\circ + F_2 \sin 60^\circ \\ &= 6.66 \times 0.86 + 3.85 \times 0.50 = 7.70 \text{ tons.} \end{aligned}$$

$$F_1 = \frac{7.70 \times 1.97}{7.0} = 2.167 \text{ tons.} \quad F_2 = \frac{7.70 \times 5.03}{7} = 5.533 \text{ tons}$$

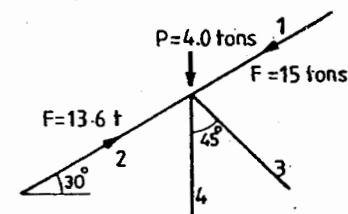
Choose $S = 0.70 \text{ cm.}$

$$L_1 = \frac{2.167}{2 \times 0.70 \times 0.72} + 2 \times 0.70 = 2.14 + 1.40 = 5.0 + 1.40 = 6.40 \text{ cms.}$$

$$L_2 = \frac{5.533}{2 \times 0.70 \times 0.72} + 2 \times 0.70 = 5.48 + 1.40 = 6.88 \text{ cms.}$$

Example (3.3):

Design the connection shown in figure where the top chord member is continuous (loaded connection).

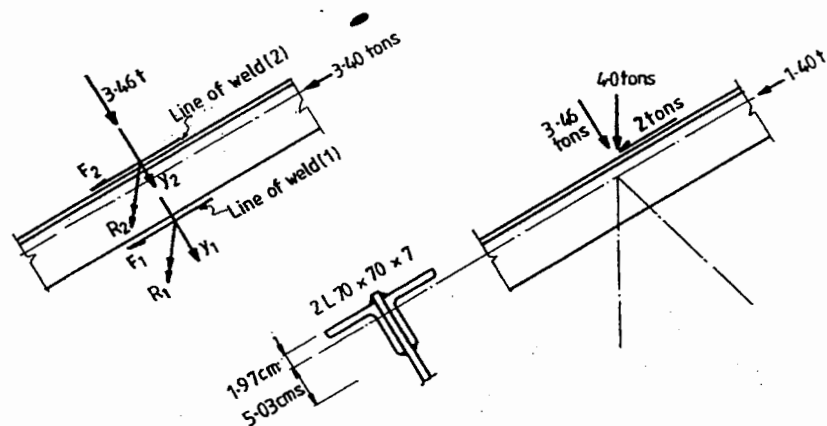


Assumptions:

1. Line of action of the load transmitted by the purlin ($P = 4.0 \text{ tons}$) will intersect the center lines of different members composing the connection.
2. The component of the load of the purlin (2 ton) acting parallel to the chord will induce a moment ($M = 2 \times 1.970 = 3.94 \text{ ton.cm}$) which will be neglected.
3. On the other hand the other component of 3.46 tons acting perpendicular to the chord is assumed to be transmitted equally by both members (1) & (2).



(A) Continuous Members



$$F_1 = \frac{3.40 \times 1.97}{7.0} = 0.956 \text{ ton}; \quad F_2 = \frac{3.40 \times 5.03}{7.0} = 2.44 \text{ tons.}$$

$$y_1 = y_2 = \frac{3.46}{2} = 1.73 \text{ ton}$$

$$R_1 = \sqrt{(0.956)^2 + (1.73)^2} = 1.97 \text{ ton.}$$

$$R_2 = \sqrt{(2.44)^2 + (1.73)^2} = 2.99 \text{ tons.}$$

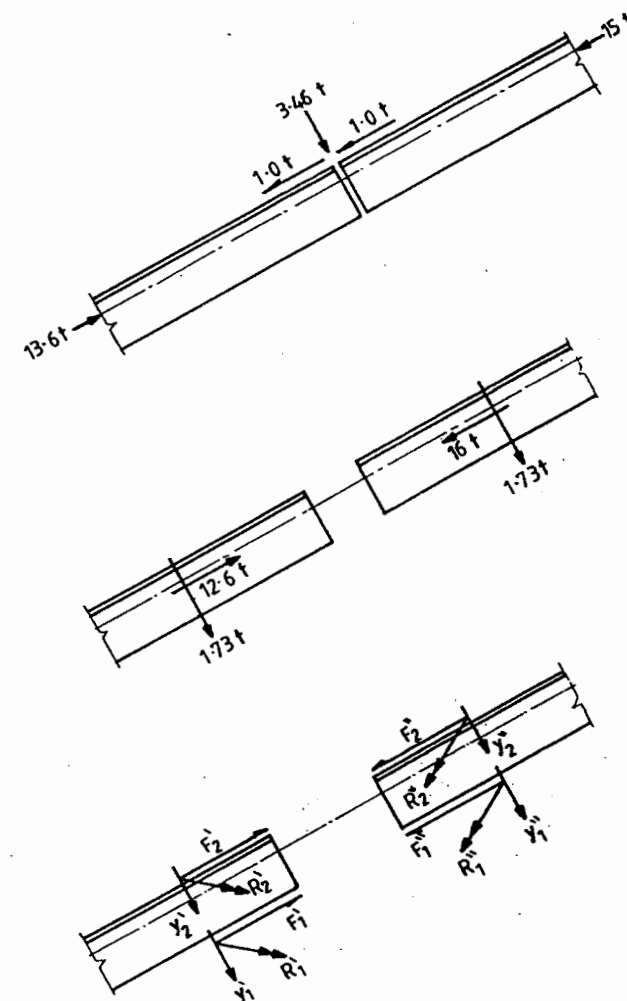
Choose $S = 0.70 \text{ cm.}$

$$L_1 = \frac{1.97}{0.70 \times 0.72 \times 2} + 2 \times 0.70 = 1.95 + 1.40 = 5.0 + 1.40 = 6.40 \text{ cms}$$

$$L_2 = \frac{2.99}{0.70 \times 0.72 \times 2} + 2 \times 0.70 = 2.96 + 1.40 = 5.0 + 1.40 = 6.40 \text{ cms.}$$



(B) Separate Members



$$F_1' = \frac{12.60 \times 1.97}{7.0} = 3.546 \text{ tons} \quad F_1'' = \frac{16 \times 1.97}{7} = 4.503 \text{ tons}$$

$$F_2' = \frac{12.6 \times 5.03}{7.0} = 9.054 \text{ tons} \quad F_2'' = \frac{16 \times 5.03}{7.0} = 11.497 \text{ tons}$$

$$y'_1 = y'_2 = \frac{1.73}{2} = 0.865 \text{ ton} \quad y''_1 = y''_2 = \frac{1.73}{2} = 0.865 \text{ ton}$$

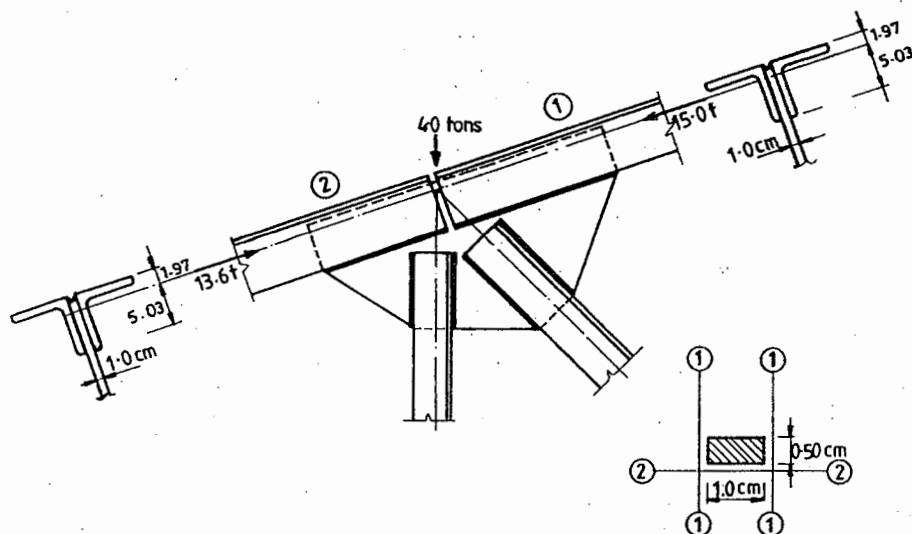
$$R'_1 = 3.65 \text{ tons} ; R'_2 = 9.09 \text{ tons} ; R''_1 = 4.585 \text{ tons} ; R''_2 = 11.53 \text{ tons}$$

Comparing the results of separate and continuous members we can conclude that the purlin load can be neglected for the separate case.

Example (3.4):

Design the connection given in example (3.3) using an embedded welding (loaded Connection).

(A) Separate Connection



Member (1):

As previously calculated $R''_2 = 11.53 \text{ tons}$, $R''_1 = 4.59 \text{ tons}$

Choose the thickness of the gusset plate = 1.0 cm.

Then failure will take place on either plane (1) or (2) due to direct shear.

$$L_2 = \frac{11.53}{2 \times 0.72 \times 0.50} + 2 \times 0.50 = 16.01 + 1.0 = 17.01 \text{ cms}$$

$$< 40 \times 0.50 \text{ o.k}$$

Thickness of angle = 0.70 cm, hence take $S_2 = 0.70 \text{ cm}$.

$$\therefore L_1 = \frac{4.59}{2 \times 0.72 \times 0.70} + 2 \times 0.7 = 4.55 + 1.40 = 5.0 + 1.40 = 6.40 \text{ cms.}$$

Member (2):

$$R'_1 = 3.65 \text{ tons} , R'_2 = 9.10 \text{ tons.}$$

$$L_2 = \frac{9.10}{2 \times 0.72 \times 0.5} + 2 \times 0.50 = 12.63 + 1.0 = 13.63 \text{ cms}$$

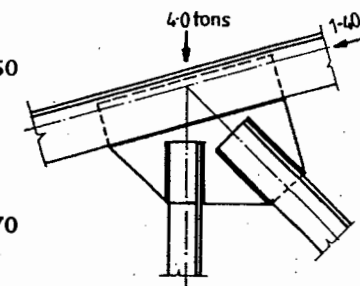
$$L_1 = \frac{3.65}{2 \times 0.72 \times 0.70} + 2 \times 0.70 = 3.87 + 1.40 = 5.0 + 1.4 = 6.40 \text{ cms.}$$

(B) Continuous Chord Members

$$R_1 = 1.97 \text{ ton} , R_2 = 2.99 \text{ t.}$$

$$\therefore L_2 = \frac{2.99}{2 \times 0.72 \times 0.50} + 2 \times 0.50 = 4.15 + 1.0 = 6.0 \text{ cms.}$$

$$\therefore L_1 = \frac{1.97}{2 \times 0.72 \times 0.70} + 2 \times 0.70 = 1.95 + 1.40 = 6.40 \text{ cms.}$$



Example (3.5):

Design the required weld lengths of the bracket shown in Fig. to resist a load $V = 21.0$ tons and $H = 1.60$ ton. The breaking force = 2.857 ton. Design the weld lines considering that the applied loads are static loads.

Solution

Preliminary Dimensioning:

The dimensions of the bracket can be assumed as shown. Where the thickness of the bracket is taken equal to the column flange.

The horizontal shock 1.6 tons is directly transmitted to the column.

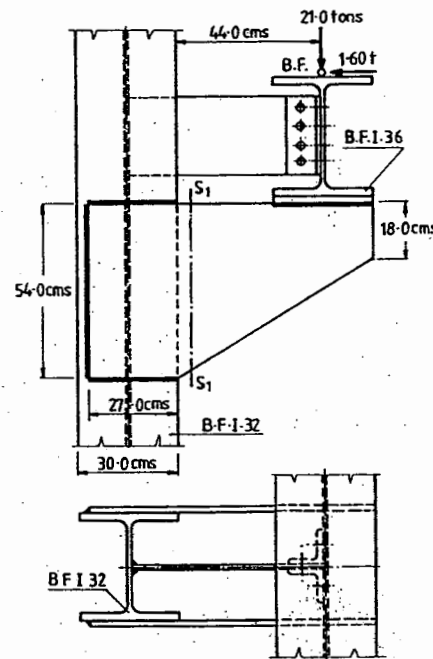
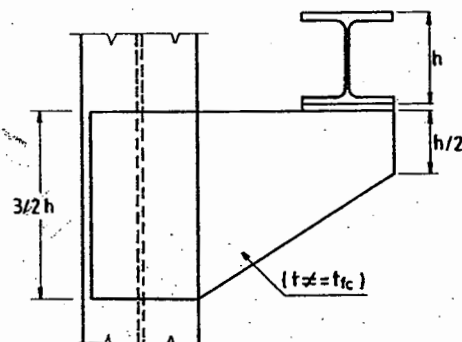
(A) With Braking Force

Case (II)

(1) Section (S_1-S_1):

$$M_x = 21 \times 44 = 924 \text{ t.cm.}$$

$$M_y = 2.857 \times 44 = 125.7 \text{ t.cm.}$$



$$M_t = 2.857 (36+1.0+27) = 182.85 \text{ t.cm.} ; Q_y = 21.0 \text{ tons.}$$

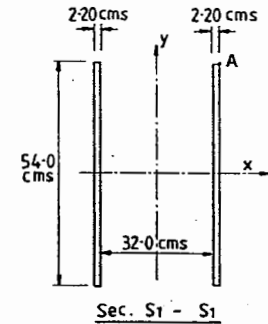
$$Q_x = 2.857 \text{ tons (Neglected for shear calculations only)}$$

Properties of Area:

$$A = 2 \times 54 \times 2.2 = 237.6 \text{ cm}^2 ; I_x = 2 \left[\frac{(54)^3 \times 2.2}{12} \right] = 57736.8 \text{ cm}^4$$

$$I_y = 2 \left[\frac{54 \times 2.2^3}{12} + 54 \times 2.2 \times 17.1^2 \right] = 69476.62 \text{ cm}^4$$

$$I_p = I_x + I_y = 127213.416 \text{ cm}^4$$



Point (A):

$$f_b = \frac{M_x}{Z_x} + \frac{M_y}{Z_y} = \frac{924 \times 27}{57736.8} + \frac{125.7 \times 18.20}{69476.62} = 0.465 \text{ t/cm}^2$$

$$q_{M_t} = \frac{M_t \cdot r_A}{I_p} = \frac{182.85 \times 32.56}{127213.42} = 0.047 \text{ t/cm}^2$$

$$f_e = \sqrt{(0.465)^2 + 3(0.047)^2} = 0.47 \text{ t/cm}^2$$

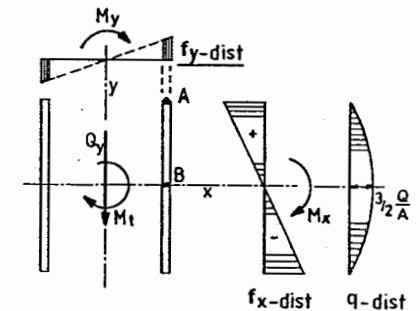
$$< 1.4 \times 1.1 \times 1.2 \text{ (o.k.)}$$

Point (B):

$$q_{Q_d} = \frac{3}{2} \times \frac{21}{237.60} = 0.133 \text{ t/cm}^2$$

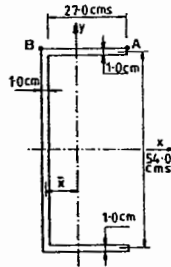
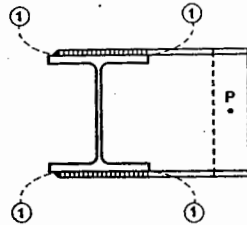
$$q_{M_t} = \frac{182.85 \times 18.20}{127213.42} = 0.026 \text{ t/cm}^2$$

$$q_{\text{total}} = 0.159 \text{ t/cm}^2 \text{ Safe}$$



The thickness of the gusset can be decreased to 1.0 cm.

(2) Fillet Weld Connecting Bracket to Column:



Section 1-1 connecting beam web to fillet weld ($s = 1.0$ cms) :-

$$A = 54 \times 1.0 + 2 \times 27 \times 1.0 = 108 \text{ cm}^2$$

$$\bar{X} = \frac{(27)^2}{2 \times 27 + 54} = 6.75 \text{ cms} \quad (\text{Table 3.6}) - \bar{X} = \frac{b^2}{(2b + h)}$$

$$I_x = \frac{54^3}{12} \times 1.0 + 2 \times 27 \times 1.0 \times 27^2 = 52488 \text{ cm}^4$$

$$I_y = 54 \times 6.75^2 + \frac{2 \times 27^3 \times 1.0}{12} + 2 \times 27 \times (13.5 - 6.75)^2 = 8201.25 \text{ cm}^4$$

$$\therefore I_p = I_x + I_y = 60689.25 \text{ cm}^4$$

Staining Actions:

$$M_x = 2.857 \times (36 + 1.0 + 27) = 182.84 \text{ t.cm.}$$

$$M_y = 2.857 \times (44 + 20.25) = 183.56 \text{ t.cm.}$$

$$M_t = 21 \times (44 + 20.25) = 1349.25 \text{ t.cm.}$$

$$Q_y = 21.0 \text{ tons.} ; N = 2.857 \text{ tons.}$$

At Point (A):

$$f_t = \frac{N}{A} + \frac{M_x \cdot y}{I_x} + \frac{M_y \cdot x}{I_y}$$

$$= \frac{2.857}{2 \times 108} + \frac{182.84 \times 27}{2 \times 52488} + \frac{183.56 \times 20.25}{2 \times 8201.25} = 0.287 \text{ t/cm}^2$$

$$q_{M_t}^{\rightarrow} = \frac{1349.25 \times 27.0}{2 \times 60689.25} = 0.3 \text{ t/cm}^2$$

$$q_{M_t}^{\downarrow} = \frac{1349.25 \times 20.25}{2 \times 60689.25} = 0.225 \text{ t/cm}^2$$

$$q_Q^{\downarrow} = \frac{21}{2 \times 108} = 0.097 \text{ t/cm}^2$$

$$q_t = \sqrt{(0.097 + 0.225)^2 + (0.3)^2} = 0.44 \text{ t/cm}^2$$

$$f_e = \sqrt{(0.287)^2 + 3(0.44)^2} = 0.814 \text{ t/cm}^2 \approx 1.1 \times 0.72 \times 1.2 \text{ (o.k.)}$$

At point (B):

$$f_t = \frac{2.857}{2 \times 108} + \frac{182.84 \times 27}{2 \times 52488} + \frac{183.56 \times 6.75}{2 \times 8201.25} = 0.135 \text{ t/cm}^2$$

$$q_Q^{\downarrow} = \frac{21.0}{2 \times 108} = 0.097 \text{ t/cm}^2$$

$$q_{M_t(x)}^{\rightarrow} = \frac{1349.25 \times 27}{2 \times 60689.25} = 0.3 \text{ t/cm}^2$$

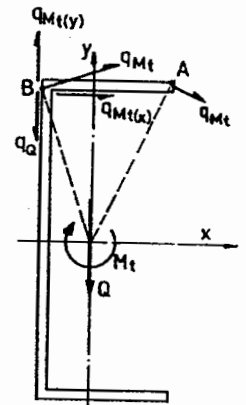
$$q_{M_t(y)}^{\uparrow} = \frac{1349.25 \times 6.75}{2 \times 60689.25} = 0.075 \text{ t/cm}^2$$

$$q_{\text{total}} = \sqrt{(0.3)^2 + (0.097 + 0.075)^2}$$

$$= 0.346 \text{ t/cm}^2$$

$$f_e = \sqrt{(0.135)^2 + 3(0.346)^2}$$

$$= 0.613 \text{ t/cm}^2 < 1.1 \times 0.72 \times 1.20 \text{ (o.k.)}$$



Section 2-2 outstanding face of weld with steel plate of bracket :-

$Q_y = 21.0$ tons will induce

$$f_{A_{\perp}} = 0.097 \text{ t/cm}^2$$

$$q_B = 0.097 \text{ t/cm}^2$$

$M_t = 1349.25$ t.cm will induce

$$q_{A_{\perp}} = 0.3 \text{ t/cm}^2 ; q_{A_{\perp}} = 0.225 ; q_B = 0.075 ; q_{B_{\perp}} = 0.3 \text{ t/cm}^2$$

$$N = 2.857 \text{ tons}$$

$$q_{A_{\perp}} = q_{B_{\perp}} = \frac{2.857}{2 \times 108} = 0.013 \text{ t/cm}^2$$

$M_x = 182.84$ t.cm (assume resisted by 2 horizontal lines by force F)

$$\frac{M_x}{(27 + 1)} = F = \frac{182.84}{28} = 6.53 \text{ tons}$$

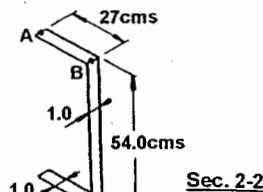
$$q_{A_{\perp}} = q_{B_{\perp}} = \frac{6.53}{2 \times 27 \times 1} = 0.121 \text{ t/cm}^2$$

$M_y = 183.56$ t.cm (assume resisted by the vertical line)

$$f_{B_{\perp}} = \frac{183.56 \times 27}{(2 \times 54^3)/12} = 0.189 \text{ t/cm}^2$$

$$f_B (e) = \sqrt{(0.189)^2 + 3 (0.097)^2} = 0.83 \text{ t/cm}^2 < 1.1 \times 0.72 \times 1.2$$

It is quite clear that for this type of I weld profile the stresses regarding section (1-1) are critical compared to section (2-2) (the difference is about 7%). Hence no need to achieve this tedious analysis regarding section (2-2).



(B) Without Braking Force - Case (I)

1. Section (S_1-S_1):

Refer to example (1.11).

2. Fillet Welds Connecting the Bracket to the Column (sec 1-1) :

2.1. Properties of Area:

$$A = 108 \text{ cm}^2 ; I_x = 52488 \text{ cm}^4 ;$$

$$I_y = 8201.25 \text{ cm}^4 ; I_p = 60689.25 \text{ cm}^4$$

$$\bar{X} = 6.75 \text{ cms}$$

2.2. Straining Actions:

$$Q_y = 21 \text{ tons.} ; M_t = 1349.25 \text{ t.cm}$$

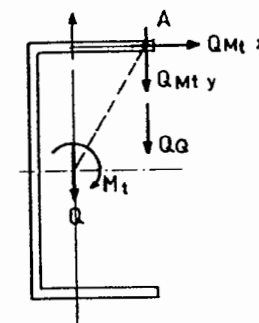
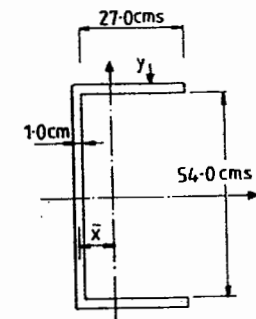
2.3. Stresses:

Point (A)

$$q_{Mt(x)} = 0.3 \text{ t/cm}^2 ; q_{Mt(y)} = 0.225 \text{ t/cm}^2$$

$$q_Q = 0.097 \text{ t/cm}^2$$

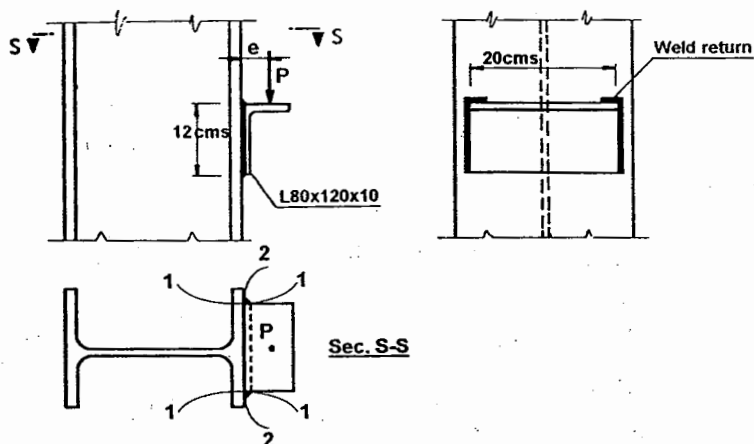
$$q_{total} = \sqrt{(0.225 + 0.097)^2 + (0.3)^2} = 0.439 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ (o.k.)}$$



Comparing the results of the two cases of loading we can conclude that the case of loading (II) is more critical. Here the braking force has been assumed to be totally transmitted to the column by one bracket which is very conservative, normally this braking force is to be resisted by at least four brackets and hence the resulting stresses devided by "4".



Example (3.6) (Neglect weld returns for the computation of stresses)



Design the fillet weld lines shown in Fig. Use E 60 electrodes and steel 37, knowing that $P = 6$ tons, $e = 4.50$ cms.

Solution

Straining Actions:

$$Q = P = 6 \text{ tons}$$

$$M = P \times 4.50 = 6 \times 4.50 = 27.0 \text{ t.cm.}$$

Section (1-1):

Neglect weld return at top of seat angle. Take $S = 1.0$ cm.

$$q_{[Q]}^{\downarrow} = \frac{6.0}{2 \times 12 \times 1.0} = 0.25 \text{ t/cm}^2.$$

$$q_{[M_t]}^{\rightarrow} = \frac{27 \times 6.0}{2 \times \frac{1 \times 12^3}{12}} = 0.563 \text{ t/cm}^2.$$

$$q_{(t)} = \sqrt{(0.25)^2 + (0.563)^2} = 0.61 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ (Safe)}$$



Section (2-2): (Considering Contact)

$$\frac{\bar{y}^2 \times 20}{2} = \frac{(12 - \bar{y})^2}{2} \times 2,$$

$$\text{Get } \bar{y} = 2.87 \text{ cms}$$

$$I_x = 20 \times \frac{(2.87)^3}{3} + 2 \times 1 \times \frac{(9.13)^3}{3} = 664.97 \text{ cm}^4.$$

$$f_t = \frac{27 \times 9.13}{664.97} = 0.37 \text{ t/cm}^2.$$

$$f_{eq.} = \sqrt{(0.37)^2 + 3(0.25)^2} = 0.569 \text{ t/cm}^2 < 0.72 \times 1.1 \text{ t/cm}^2.$$

Decrease size of weld (S) according to shear stresses (q_t) on Section (1-1).

$$S = 1.0 \times \frac{0.61}{0.70} = 0.87 \approx 0.90 \text{ cm.}$$

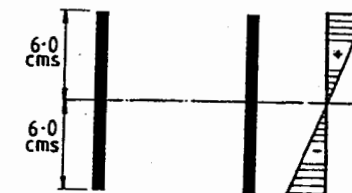
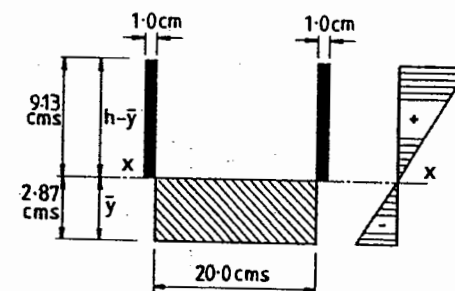
Section (2-2): Without Contact

$$I_x = 2 \times \frac{12^3 \times 1.0}{12} = 288 \text{ cm}^4.$$

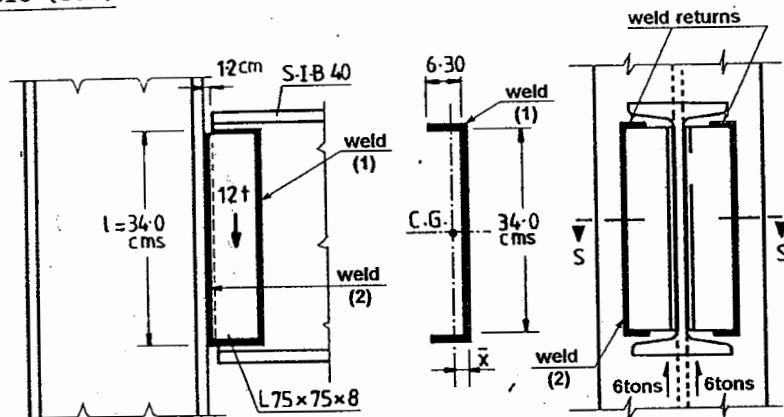
$$f_t = \frac{27 \times (12/2)}{288} = 0.5625 \text{ t/cm}^2.$$

$$f_{eq.} = \sqrt{(0.5625)^2 + 3(0.25)^2} = 0.7 \text{ t/cm}^2 < 1.1 \times 0.72 \text{ t/cm}^2 \text{ (o.k.)}$$

$$\text{Take the size of weld} = 1.0 \times \frac{0.7}{0.72 \times 1.1} = 0.88 \text{ cm} \approx 1.0 \text{ cm.}$$



Example (3.7):



Section 1-1 Faces of weld connecting angles to beam web :-

Assume size of weld $s = 0.4$ cms (min size)

$$\bar{X} = \frac{(6.3)^2}{2 \times 6.3 + 34} = 0.85 \text{ cms} - \bar{X} = \frac{b^2}{(2b + h)}$$

$$M_t = \frac{12}{2} \times (7.5 - 1.2 - 0.85) = 32.7 \text{ t.cms}$$

$$q_{\parallel} = \frac{12}{2 \times 18.64} = 0.321 \text{ t/cm}^2$$

$$I_x = 0.4 \left(\frac{34^3}{12} + 2 \times 6.3 \times 17^2 \right) = 2766.6 \text{ cm}^4$$

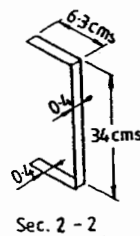
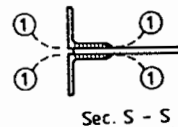
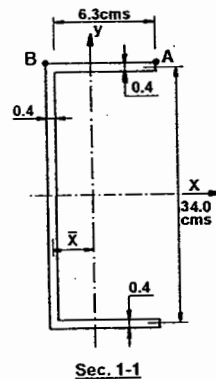
$$I_y = 0.4 [34 \times 0.85^2 + 2(6.3^2/12 + 6.3 \times 2.3^2)] = 53.15 \text{ cm}^4$$

$$I_p = I_x + I_y = 2819.75 \text{ cm}^4$$

$$q_{A(Mt)} \downarrow = (32.7 \times 5.45)/2819.75 = 0.063 \text{ t/cm}^2$$

$$q_{A(Mt)} \rightarrow = (32.7 \times 17.0)/2819.75 = 0.197 \text{ t/cm}^2$$

$$q_{t(A)} = \sqrt{(0.321 + 0.063)^2 + (0.197)^2} = 0.431 \text{ t/cm}^2$$



Section 2-2 outstanding face connected to angle leg :-

One can assume that the vertical shear = 6 tons is transmitted by shear "q" to the vertical line (34x0.4) and by normal stress $f_{\perp}$ to the two horizontal lines (6.7x0.4).

Regarding the moment = 32.7 the transmission of forces will be exactly as section (1-1).

$$f_{(A)\perp} = \frac{12}{2 \times 18.64} = 0.321 \text{ t/cm}^2$$

$$q_{A(Mt)} \downarrow = 0.063 \text{ t/cm}^2 ; \quad q_{A(Mt)} \rightarrow = 0.197 \text{ t/cm}^2$$

$$f_{(A)e} = \sqrt{(0.321)^2 + 3[(0.063)^2 + (0.197)^2]} = 0.48 \text{ t/cm}^2 < 0.72 \times 1.1$$

It clear here also that it will be sufficient for weld profile [to consider only section (1-1).

Column Flange Connection weld lines (no Contact):

Section (1)-(1): Assume size of weld = 0.40 cms

Neglect weld returns

$$q_{\parallel} = \frac{12}{2 \times 34 \times 0.80} = 0.44 \text{ t/cm}^2$$

$$\therefore M_t = 6 (7.5 - \bar{x}) = 39.9 \text{ t.cm.}$$

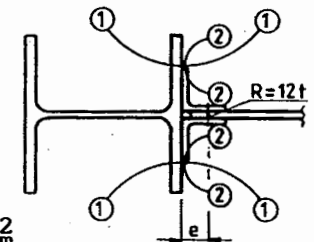
$$\therefore q_{Mt} = \frac{39.9 \times 34 / 2}{0.40 \times (34)^3 / 12} = 0.517 \text{ t/cm}^2$$

$$\therefore q_t = \sqrt{(0.44)^2 + (0.517)^2} = 0.68 \text{ t/cm}^2 < 0.72 \text{ O.K.}$$

Section (2-2):

$$q_{\parallel} = 0.44 \text{ t/cm}^2, \quad f_{\perp} = \frac{M.y}{I_x} = 0.517 \text{ t/cm}^2$$

$$f_{eq.} = \sqrt{(0.517)^2 + 3(0.44)^2} = 0.92 \text{ t/cm}^2 > 0.72 \times 1.1 \text{ Unsafe}$$



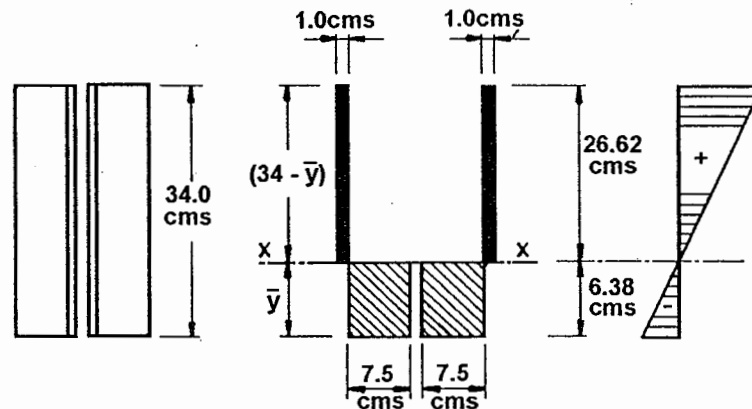


$$\text{Take } s = \frac{0.4 \times 0.92}{0.72 \times 1.1} = 0.461 \text{ cms} \quad \text{take } s = 0.5 \text{ cms}$$

Considering Contact Area Between Angles and Column Flange
Section (2-2):

Assume size of weld = 0.4 cms.

$$\frac{\bar{y}^2}{2} \times 15 = \frac{2 \times (34 - \bar{y})^2}{2} \times 0.4 \quad \text{Get } \bar{y} = 6.38 \text{ cms.}$$



$$I_x = \frac{15 \times (6.38)^3}{3} + \frac{2 \times 0.4 \times (27.62)^3}{3} = 6917 \text{ cm}^4$$

$$f_{\perp} = \frac{12 \times (7.5 - 0.85) \times 27.62}{6917} = 0.318 \text{ t/cm}^2$$

$$q_{//} = \frac{12}{2 \times 34.0 \times 0.4} = 0.44 \text{ t/cm}^2$$

$$f_{eq.} = \sqrt{(0.318)^2 + 3(0.44)^2} = 0.82 \text{ t/cm}^2 > 0.72 \times 1.1 \text{ unsafe}$$

Use size of weld $s = 0.5 \text{ cms}$ it will be safe.

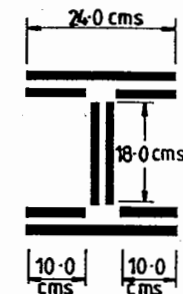
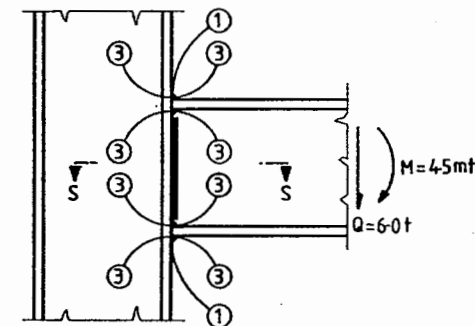


Example (3.8):

Design the rigid connection shown in the Fig. below The beam is a B.F.I.B. No. (24). The column is a B.F.I.B. No. (30).

Solution

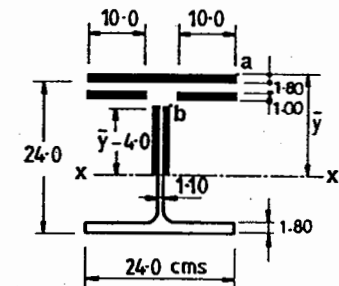
$$M = 4.50 \text{ mt} \quad Q = 6.0 \text{ tons}$$



Section (1-1): (With Contact)

Assume that moment will be resisted by the whole effective cross section and that the shearing force will be resisted by the vertical welds only.

Assume size of weld $S = 1.0 \text{ cm}$.



Taking first moment of area about x-x, we get;

$$(24 \times 1.80) (24 + 1.0 - \bar{y} - 0.90) + \frac{(24 + 1.0 - \bar{y} - 1.80)^2}{2} \times 1.10 =$$

$$(24 \times 1.0) (\bar{y} - 0.50) + 2(10.0 \times 1.0) (\bar{y} - 2.80 - 0.50) + \frac{2(\bar{y} - 4.6)^2 \times 1.0}{2}$$

$$\therefore \bar{y} = 12.67 \text{ cms.}$$



$$I_x = 24 \times 1.80 \times (11.43)^2 + \frac{(10.53)^3 \times 1.10}{3} + \frac{2 \times 1.0 \times (8.07)^3}{3} \\ + 2 \times 10 \times 1.0 \times (9.37)^2 + 24.0 \times 1.0 \times (12.17)^2 = 11732.83 \text{ cm}^4.$$

$$f_t \text{ (point a)} = \frac{4.50 \times 100 \times 12.67}{11732.83} = 0.48 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2.$$

$$f_t \text{ (point b)} = \frac{4.50 \times 100 \times (12.67 - 4.6)}{11732.83} = 0.31 \text{ t/cm}^2.$$

$$q_{//} = \frac{6}{2 \times 16.8 \times 1.0} = 0.178 \text{ t/cm}^2, \text{ (vertical lines only)}$$

$$f_{eq} \text{ (point b)} = \sqrt{(0.31)^2 + 3(0.178)^2} = 0.43 \text{ t/cm}^2 \\ < 0.72 \times 1.1 \text{ t/cm}^2.$$

s can be reduced to 0.7 cms it will be safe.

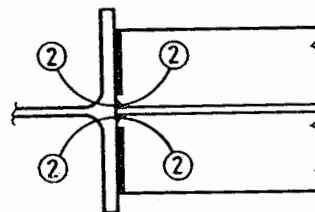
Section (2-2):

$$q_{//} = \frac{6.0}{2 \times 16.8 \times 1.0}$$

$$= 0.178 \text{ t/cm}^2$$

$$\text{Reduce } s = \frac{0.167}{0.720} \times 1.0$$

$$= 0.23 \text{ cm}$$



Sec. S - S

Section (3-3):

Assume that the moment is replaced by two forces acting on the flanges.

$$F = \frac{4.50 \times 100}{(24 - 1.80)} = 20.27 \text{ tons.}$$

$$q_{F \perp} = \frac{20.27}{1.0 (24 + 2 \times 10)} = 0.4607 \text{ ton/cm}^2 < 0.72 \text{ t/cm}^2$$

Safe

$$\text{Reduce } S = \frac{0.4607}{0.72} \times 1.0 = 0.64 \text{ cm} \approx 0.70 \text{ cm}$$

From Sections (1-1), (2-2) and (3-3) take $S = 0.70 \text{ cm}$.
(The greatest).

Section (1-1): (Without Contact)

The N.A. will lie on the C.G. of the lines of welds (i.e. at mid height). - size of weld $s = 1.0 \text{ cms}$

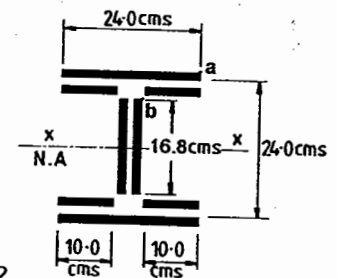
$$I_x = 2 \times 24 \times 1.0 (12 + 0.5)^2 + 4 \times 10 \times 1.0 (12 - 1.80 - 0.5)^2 \\ + 2 \times \frac{(16.8)^3 \times 1.0}{12} = 12054 \text{ cm}^4$$

$$f_t \text{ (point a)} = \frac{450 \times 13.0}{12054} \\ = 0.48 \text{ t/cm}^2 < 0.72$$

$$f_t \text{ (point b)} = \frac{450 \times 8.4}{12054} = 0.31 \text{ t/cm}^2$$

$$q_{[Q/\downarrow]} = \frac{6.0}{2 \times 16.8 \times 1.0} = 0.178 \text{ t/cm}^2$$

$$f_{eq} \text{ (point b)} = \sqrt{(0.31)^2 + 3(0.178)^2} = 0.43 \text{ t/cm}^2 \\ < 0.72 \times 1.10 \text{ t/cm}^2$$



Check stresses on Section (2-2) & Section (3-3) as before

We can reduce the size of weld according to stress at point

(a).

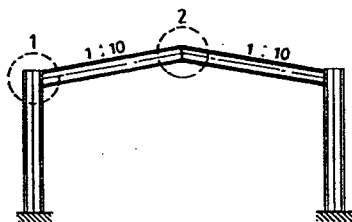
$$\therefore s = \frac{1.0 \times 0.48}{0.72} = 0.67 \approx 0.70 \text{ cm}$$

i.e. The size of weld for Sections (1-1), (2-2) and (3-3) will be taken $s = 0.70 \text{ cm}$.

Example (3.9):

Design the welded corner connection shown in figure for the following straining actions:

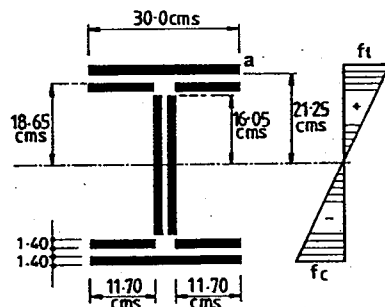
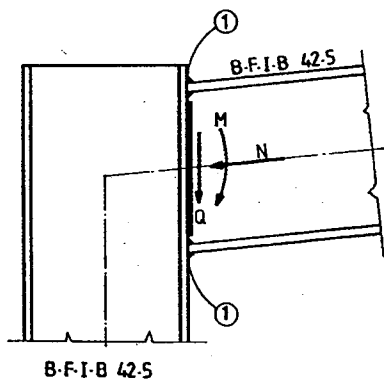
- $M = 43.13 \text{ mt.}$
- $N = - 6.3 \text{ tons.}$
- $Q = 7.0 \text{ tons.}$



Using the maximum size of welds:

$s = 2.6 \text{ cms. (around flanges)}$

$s = 1.4 \text{ cm. (around web)}$



For Weld Section (1-1):

$$I_x = 2 \times 30 \times 2.6 (21.25 + 1.3)^2 + 4 \times 11.7 \times 2.6 (18.65 - 1.3)^2 + 2 \times 1.40 \times \frac{(16.05 \times 2)^3}{12} = 123672.5 \text{ cm}^4$$

$$f_t(\text{point a}) = \frac{M.Y}{I} = \frac{4313 \times (21.25 + 2.6)}{123672.5} = 0.83 \text{ t/cm}^2 > 0.72 \text{ t/cm}^2$$

It is clear that this solution will never give safe stresses in welds.

To overcome this problem, we use either of the following;

- (1) K - Welds.
- (2) Haunched Connection.

Example (3.10):

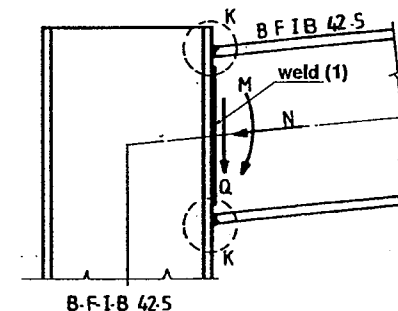
Design the corner welded connection of example (3.9) using K - Welds.

Solution

The K - Weld is considered safe as long as the cross section is safe.

Check of Weld Line (1):

Weld line (1) will carry only the shearing force Q .



$$\therefore L = 42.5 - 4 \times 2.60$$

$$= 32.10 \text{ cms.}$$

$$\therefore s = \frac{7.0}{2 \times 32.1 \times 0.72}$$

no. of lines $\uparrow$ $f_{\text{all.weld}}$

$$= 0.15 \text{ cm.}$$

$\therefore$ Take $s = 6.0 \text{ mms.}$

$s = 6 \text{ mms (min.)}$ as the thickness of the thicker plate (column web) = 1.4 cms (o.k.).

Design of Corner Connection:

(1) Crippling of Column Web at Comp. Flange:

$$K = 2 t_c = 2 \times 2.6 = 5.20 \text{ cms.}$$

Resisting area of column web

$$= [5k + (t_b / \cos \alpha)] \times t_{wc} = (5 \times 5.2 + 2.61) \times 1.40$$

$$= 40.05 \text{ cm}^2.$$

Applied area from flange

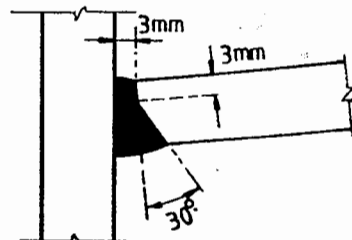
$$= t_b \times b_f \times \cos \alpha$$

$$= 2.6 \times 30 \times 0.995$$

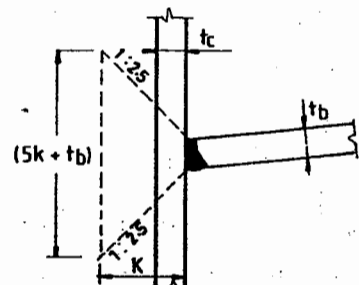
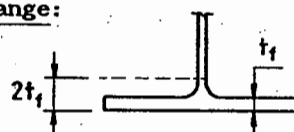
$$\approx 78 \text{ cm}^2 > 40.05 \text{ cm}^2$$

$\therefore$ Stiffeners must be used.

Area of one stiffener (inclined)



Detail K
(K-Weld)



t_b = th. of beam flange.

t_c = th. of column flange.

t_{wc} = th. of column web.

b_f = breadth of beam flange.

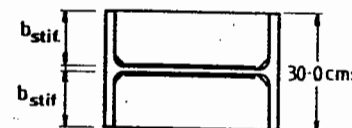
$$= [1/2 (78 - 40.05)] / \cos \alpha$$

$$= 18.98 \text{ cm}^2.$$

$$b_{\text{stiff.}} = \frac{30 - 1.40}{2} = 14.3 \text{ cms. ; } b_{\text{st}} / t_{\text{st}} \leq 25 / \sqrt{F_y} \leq 16$$

to prevent local buckling.

$$\text{take } b = 14 \text{ cms , } t = \frac{18.98}{14} \approx 1.4 \text{ cm}$$



$$A_{\text{stiff.}} = 1.40 \times 14 = 19.60 \text{ cm}^2$$

"Welds Connecting Stiffener to the Column Web"

$$F_{\text{st}} = [\frac{1}{2} (78 \times 1.4 - 40.05 \times 0.75 \times 2.4)] / \cos \alpha$$

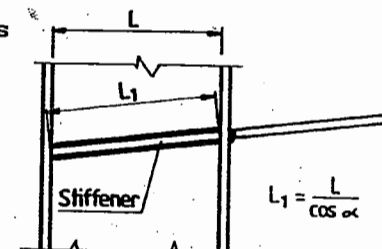
$$\approx 18.5 \text{ tons}$$

$$L \approx L_1 = 42.5 - 4 \times 2.60 = 32.1 \text{ cms}$$

$$s = \frac{18.5}{2 \times 32.1 \times 0.72} = 0.4 \text{ cm ,}$$

take $s = 0.5 \text{ cm}$

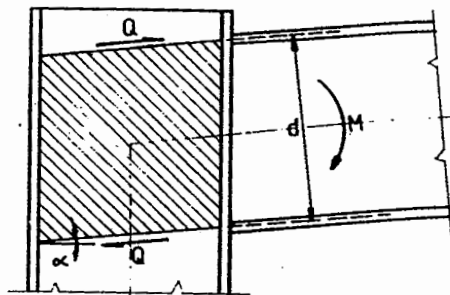
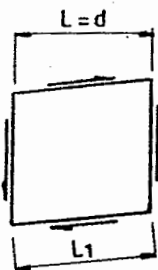
where : $L = 42.5 - 4 \times 2.60 = 32.1 \text{ cms}$



(2) Stability of the Corner:

$$d_b = 42.5 - 2.60 = 39.9 \text{ cms.}$$

$$Q = \frac{M}{d_b} = 108.10 \text{ tons.}$$



$$\text{Resistance} = \frac{42.5}{\cos \alpha} \times t_{wc} \times q_{all.}$$

$$(q_{all.} = 0.35 F_y = 0.84 \text{ t/cm}^2)$$

$$= \frac{42.5}{\cos \alpha} \times 1.40 \times 0.84 = 50.23 \text{ tons.}$$

Resistance < Q

Use one of the following two solutions:

(A) Increase (t_w): Using 2 additional cover plates.

$$(108.1 - 50.23) = \frac{(42.5 - 4 \times 2.6)}{\cos \alpha} \times 2 t_c \times 0.84$$

$$t_c = 0.949 \text{ cm.}$$

Choose $t_c = 1.0 \text{ cm.}$

Two additional corner plates each of thickness 1.0 cm are to be added.

(B) Use Diagonal Stiffener:

$$\text{Residual force} = 108.10 - 50.23$$

$$F = 57.87 \text{ tons.}$$

Taking the equilibrium in the horizontal direction we get:

$$R \sin \beta = F \cos \alpha$$

$$\text{Where } \alpha = 6^\circ; \quad \beta = \frac{1}{2} (90 + 6) = 48^\circ$$

$$R = \frac{57.87 \times 0.995}{0.743} = 77.49 \text{ tons}$$

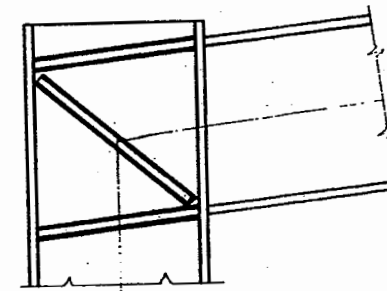
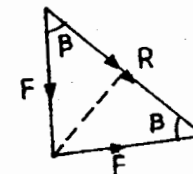
$$A_{\text{one stiff.}} = \frac{77.49}{2 \times 1.40} = 27.67 \text{ cm}^2$$

$$b_{\text{stiff.}} = 14 \text{ cms.}$$

$$\therefore t_{\text{stiff.}} = \frac{27.67}{14.0} = 1.97 \text{ cms.}$$

$$\therefore \text{Take } t_{\text{stiff.}} = 2.0 \text{ cms.}$$

$$\text{Check } \frac{b}{t} = 7.0 < 16 \text{ Safe}$$



(3) Safety Against Tension Flange:

$$t_c (\text{min.}) = 0.40 \sqrt{A_F \cos \alpha}$$

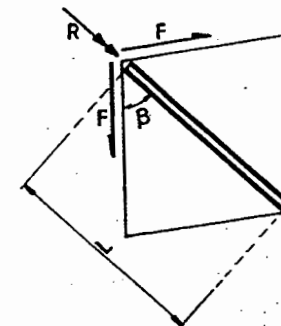
$$\approx 0.40 \sqrt{2.60 \times 3.0}$$

$$= 3.53 \text{ cms} > 2.60 \text{ cms}$$

(Unsafe)

$\therefore$ Use two stiffeners as used for compression flange.

$$[b = 14 \text{ cms}, t = 1.4 \text{ cm}]$$





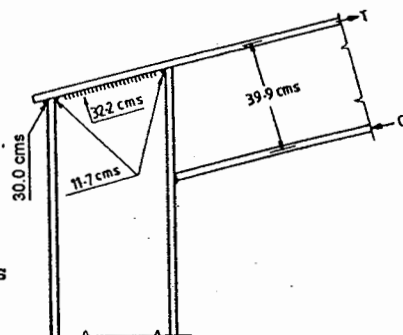
N.B.

The tension flange of the rafter may be extended to act as a stiffener. The tension force will be transmitted to the column via fillet weld lines whose length (L) is:

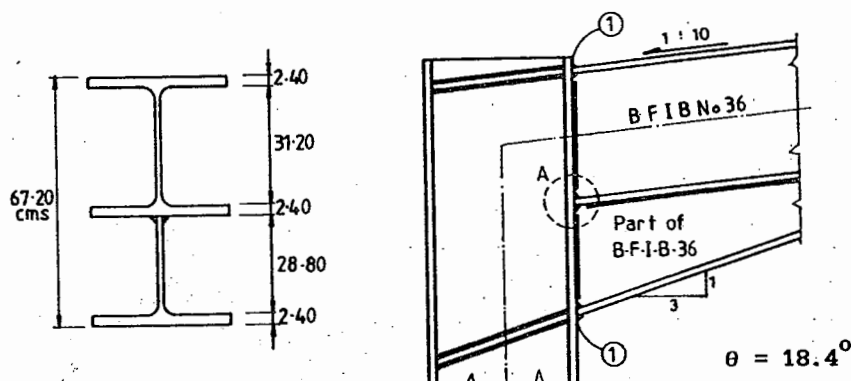
$$L = 30 + 4 \times 11.7 + 32.2 \times 2 = 141.2 \text{ cms}$$

$$T = C = \frac{43.13 \times 100}{(42.5 - 2.6)} = 108.1 \text{ tons}$$

$$s = \frac{108.1}{141.2 \times 0.72} = 1.06 \text{ cm} \quad \text{take } s = 11 \text{ mm}$$

**Example (3.11):**

Design the corner welded connection given in example (3.8) using a haunched connection.



For simplicity neglect fillet welds (A) around flange. Weld line will appear as shown in Sec. (1-1).



To determine the size of weld use the following approximate analysis:

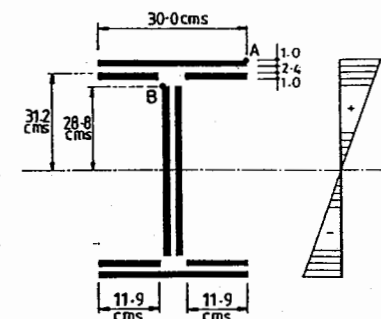
$$\text{Compression force} = \frac{43.13 \times 100}{(67.2 - 2.4) \cos \theta}$$

$$= 70.43 \text{ tons}$$

$$s = \frac{70.43}{(30 + 2 \times 11.9) \times 0.72} = 1.81 \text{ cm}$$

$$\text{Take } (s) \text{ around flanges} = 1.8 \text{ cm}$$

$$(s) \text{ around web } 1. = 1.4 \text{ cm}$$



$$I_x = 2 \times 1.80 \times 30 \times (33.6 + 0.9)^2 + 4 \times 1.8 \times 11.9 \times (31.2 - 0.9)^2 + 2 \times 1.40 \times \frac{(28.80 \times 2)^3}{12}$$

$$= 251800 \text{ cm}^4$$

$$f_t (\text{point A}) = \frac{4313 \times (33.6 + 1.80)}{251800} = 0.60 \text{ t/cm}^2 < 0.7 \text{ t/cm}^2 \quad (\text{o.k.})$$

$$f_t (\text{point B}) = \frac{0.60 \times 28.80}{(33.6 + 1.80)} = 0.49 \text{ t/cm}^2$$

Point (B) will also carry a shear stress $q \left[\frac{Q}{I} \right]$, [shearing force will be carried by vertical welds].

$$q \left[\frac{Q}{I} \right] = \frac{Q}{A_{\text{vert.weld}}} = \frac{7}{2 \times (28.8 \times 2) \times 1.4} = 0.043 \text{ t/cm}^2$$

$$f_e = \sqrt{(0.49)^2 + 3 (0.043)^2} = 0.495 \text{ t/cm}^2 < 0.72 \times 1.1 (\text{o.k.})$$

Check of Sec (2-2) of Weld:

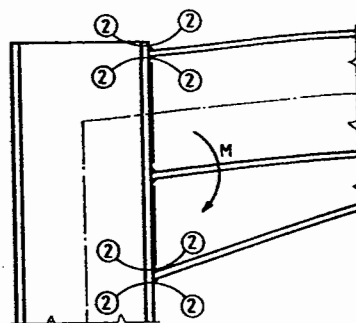
$$T = \frac{M}{d_b} = \frac{4313}{64.8} = 66.80 \text{ tons.}$$

$$L_w = 30 + 2 \times 11.90 = 53.80 \text{ cms.}$$

$$q_{[Q_1]}^{\rightarrow} = \frac{66.8/\cos \alpha}{(53.8 \times 1.80)} = 0.69 \text{ t/cm}^2$$

$$< 0.72 \text{ t/cm}^2$$

Safe



Design of Corner Connection:

(1) Crippling of Column Web at Comp. Flange: (At the End of Haunch)

$$K = 2 t_c = 2 \times 2.60 = 5.20 \text{ cms.}$$

$$\text{Applied inclined area} = 30 \times 2.4 \times \cos \theta = 68.3 \text{ cm}^2$$

$$\text{Resisting area} = t_{wc} (t_b / \cos \theta + 5k) = \left(\frac{2.6}{0.945} + 5 \times 5.2 \right) \times 1.4$$

$$= 40.25 \text{ cm}^2$$

Hence use 2 horizontal stiffeners

$$A_{st} = \frac{68.30 - 40.25}{2} = 14.03 \text{ cm}^2$$

$$\text{Choose } b_{st} = 14 \text{ cms ; } t_{st} = \frac{14.03}{14} = 1.0 \text{ cm}$$

$$\text{Take } t_{st} = 1.0 \text{ cm ; } b/t < 16 \text{ (o.k.)}$$

(2) Corner Stability:

$$\text{Applied shear} = \frac{M}{d_b \cos \alpha} = \frac{4313}{64.8 \times 0.995} = 66.9 \text{ tons}$$

$$\text{Resisting shear} = \frac{42.5}{\cos \alpha} \times 0.84 \times 1.4 = 50 \text{ tons}$$

unsafe.

Hence use two additional cover plates of thickness (t_c)

$$(66.9 - 50) = \frac{36 - 4 \times 2.4}{\cos \alpha} \times 0.84 \times 2 t_c$$

$$\text{use } t_c = 7 \text{ mms.}$$

(3) Bending of Column Flange:

$$t_{fc} \geq 0.4 \sqrt{30 \times 2.4 \times \cos \alpha} \geq 3.31 \text{ cms}$$

3.31 cms being greater than 2.4 cms, use 2 horizontal stiffeners opposite to the tension flange of the rafter (14.0 cms x 1.2 cm).

CHAPTER 4



CHAPTER 4

FLEXIBLE CONNECTIONS

4.1. Introduction

In conventional analysis, the beam-to-column connection is either assumed perfectly flexible, with complete freedom to rotate, or completely rigid, with no relative rotation between column and beam. These assumptions are not entirely consistent with actual conditions, in which the angle subtended between column and beam varies with the applied moment.

The American specifications provide for the design of multistory steel buildings three approaches concerning the type of connections: "Type 1 construction" which requires elastic frame analysis assuming rigid beam-to-column connections. "Type 2 construction" which assumes that the beam-to-column connections are free to rotate under gravity loads, while offering moment resistance under lateral loads. "Type 3 construction" which calls for elastic frame analysis accounting for the behavior of semi-rigid beam-to-column connections. The British Standard 449 allows for design by semi rigid as well as simple or fully rigid connections. While the Egyptian Code of Practice [2001] has not mentioned these classifications, leaving the choice to the designer.

However, a more economical design would result if the effect of the rigidity of the connections were included in the analysis of a frame. This has not received sufficient attention because of the lack of information about the actual connection behavior, as well as the greater amount of time required for the analysis, especially without the use of digital computers.



A basic description of flexural connection behavior is its moment rotation relationship, which expresses the moment transmitted by the connection as a function of the relative rotation of the slope lines at the point of intersection as shown in Fig. 4.1.

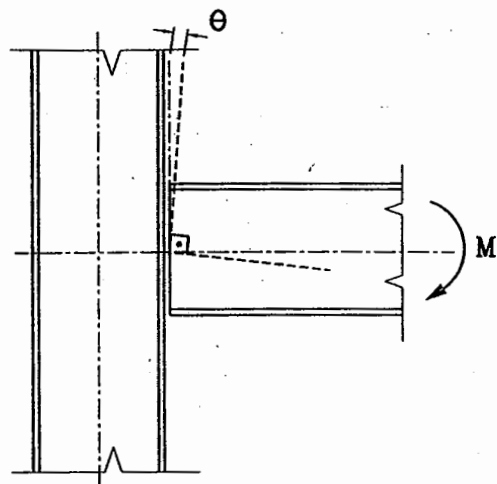


Fig. 4.1 Actual Moment-Rotation
Behaviour of Connection

Fig. 4.2 illustrates various moment-rotation curves of different types of connections. An ideally rigid connection is one that yields no relative rotation regardless of the moment acting on the joint. A rigid connection in practice allows for a small rotation as the applied moment increases. A flexible connection offers some resistance to rotation as the applied moment increases. An ideally flexible connection offers no resistance to rotation. Any connection in practice lies between the ideally rigid and ideally flexible, and may be called a semi-rigid connection.

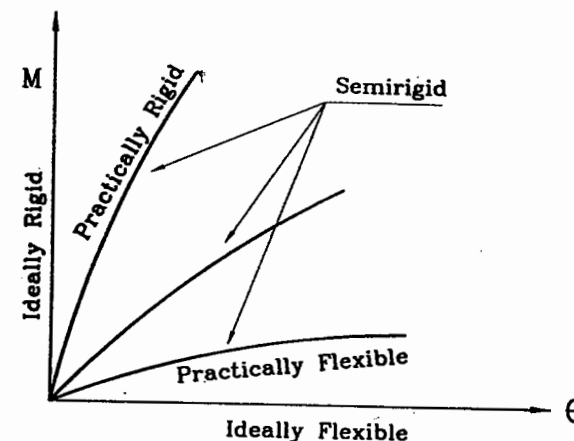


Fig. 4.2 Typical Moment-Rotation Diagram

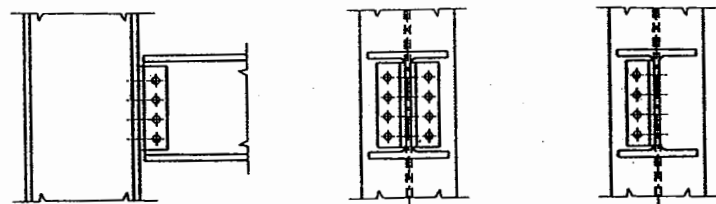
In this chapter all the available flexible connections are analyzed, while in chapters 5 and 6 the rigid and semi rigid connections will be investigated.

4.2. Flexible Beam-to-Column Connections:

Flexible beam-to-column connections are simple connections which transmit shear only. The reactions induced at the ends of the beams will go through welds or bolts to the columns. Some parasite moments due to the eccentricities may appear inducing either torsional or bending stresses. These stresses are to be added to the shear stresses specially for welded connections.

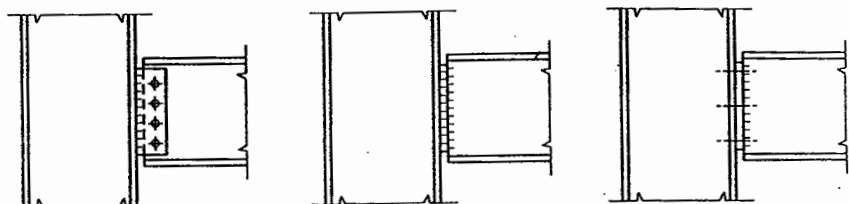
A wide variety of flexible connections is available as shown in Fig. 4.3. These connections can be classified as follows:

- (a) The double web framing angles connection.
- (b) The one-sided web framing angle connection.



(a) Double Framing.

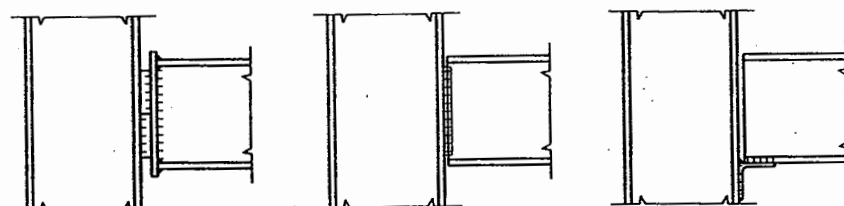
(b) Single Framing.



(c) Cleat Plate.

(d) Shear End Plate.
(Welded)

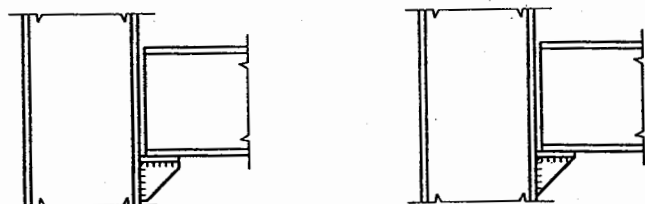
(d) Shear End Plate.
(Bolted)



(e) Bearing End Plate.

(f) Direct Welding.

(g) Seat Angle.



(h) Stiffened Seat.

(i) Triangular Seat Bracket.

Fig. 4.3 Flexible Beam-to-Column Connections

- (c) The cleat plate web-beam connection.
- (d) The end-plate web-beam connection.
- (e) The bearing end plate connection.
- (f) The directly welded web-beam connection.
- (g) The unstiffened seated beam connection.
- (h) The stiffened seat bracket connection.
- (i) The triangular bracket plate connection.

In the following sections a detailed analysis of each type is presented.

4.3. Double Web Framing Angles:

4.3.1. Welded Connections:

Fig. 4.4 illustrates a double welded web framing angles connection. The angles are shop welded to the beam's web with a hook weld extending along the whole leg leaving a set-back of 12 mm between the beam's edge and the column flange. While the other leg of the angles are field welded to the column flange with a hook weld not exceeding 12 mm so the beam end is not restraint from rotating under load.

In order to allow sufficient flexibility, it is recommended to choose the framing angles thinner as possible. For the analysis of this type of connections, one should consider the following:

- (i) The determination of the framing angles size.
- (ii) The analysis of the shop welds attaching the beam web to framing angles.
- (iii) The analysis of the field welds attaching the column flange to framing angles.



(i) Framing Angles Size:

Fig.4.4.b illustrates the expected bent position of the framing angles required to allow for sufficient flexibility.

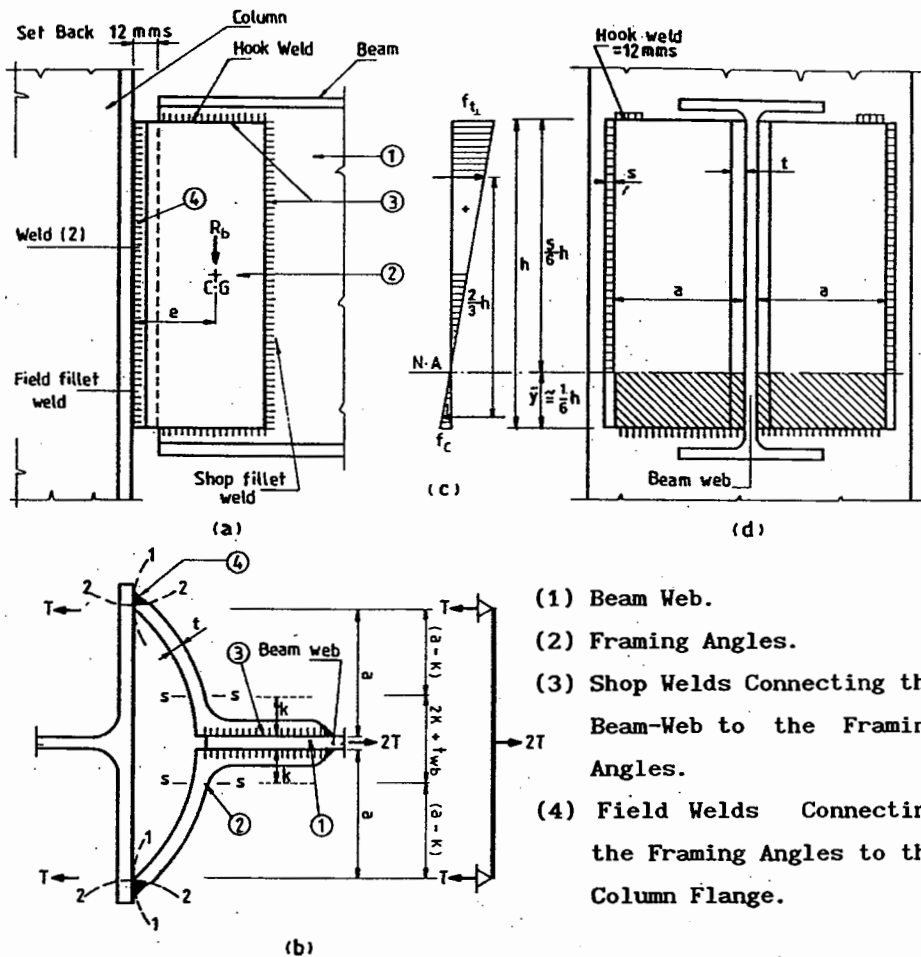


Fig. 4.4

The flexural moment $M = R_b \cdot e$ tends to press the framing angles against the column flange. The lines of welds on face 1-1 will be subjected to tensile stresses f_t (Fig.4.4.c) which can be determined by following the path of forces from one structural element to the other. The reaction R_b of the beam will be transmitted to the shop fillet welds connecting the beam's web to the framing angles by a concentric shear R_b passing through the C.G of I-shape fillet weld. Then to the column's flange by a concentric shear R_b and a torsional moment $M_t = R_b \cdot e$ through face 2-2 of the field fillet welds "4". Then to the column flange via face 1-1 of the fillet welds "4" by a concentric shear R_b and a flexural moment $M = R_b \cdot e$.

Concerning the analysis, one can follow either method "A" or method "B" as given below:

Method "A" - Neutral Axis Lying at $h/6$:

- Consider the neutral axis lying at a distance $\bar{y} = h/6$ from the lower edge of the framing angles.
- Draw the stress distribution as shown in Fig. 4.4.c.
- The external moment is given by the following:

$$M_e = R_b \cdot e \quad (a)$$

- While the internal moment can be computed from Fig. 4.4.c as follows :

$$M_i = \frac{1}{2} f_t \times 2s \times \frac{5}{6} h \times \frac{2}{3} h \quad (b)$$

- Equating (a) and (b) , the applied tensile stress on face (1-1) of weld (4) can be calculated and then checked :-

$$f_{t\perp} = (9 R_b e) / (5 h^2 s) \quad 4.1$$

Method "B" - Exact Position of Neutral Axis:

- Determine the exact position of the neutral axis by taking the first moment of area about the neutral axis. The following equation is to be solved to compute $\bar{y}$:-

$$2a\bar{y} \times \bar{y}/2 = 2s \frac{(h-\bar{y})^2}{2} \quad (c)$$

- The corresponding inertia will be as follows:

$$I = \frac{1}{3} \times 2a \times \bar{y}^3 + \frac{1}{3} \times 2s(h-\bar{y})^3 \quad (d)$$

- Apply in the stress equation ($f=M.y/I$), thus get $f_{t\perp}$:-

$$f_{t\perp} = R_b \cdot e \cdot (h-\bar{y})/I \quad 4.2.$$

The bending tensile stress of Eq. 4.1 or Eq. 4.2 will induce a tensile force 2T per centimeter length of h which can be computed using the stress distribution of Fig. 4.4.c.

Thus:

$$2T = f_{t\perp} \times 2s \times 1\text{cm} \quad (e)$$

This load 2T will bend the framing angles to the configuration shown in Fig. 4.4.b. For welded connections the behavior can be approximated to a simply supported beam. The critical section in flexure will be generally at the base of the fillet of the angle , a distance k from the back of the angle (section s-s of Fig. 4.4.b).

$$M = T (a-k) \quad (f)$$

Where a is the size of the framing angle leg. Stress along sec.s-s at a distance k from the back of the angle, get:

$$f_b = T (a-k)/z \quad (g)$$

$$\text{where } z = 1 \times t^2 / 6 \quad (h)$$

Framing angles bending stresses:

Combining Eqs.(e),(h) and 4.1 in Eq.(g), get the following:

$$f_b = 10.8 R_b e(a-k)/h^2 t^2 \quad 4.3$$

Replace f_b by the allowable bending stress $F_b = 0.58 F_y$, the thickness "t" of the framing angles can be computed.

One can say that the section along s-s is a solid rectangular section and hence $F_b = 0.72 F_y$, but we prefer to be conservative as the analysis is based on many unverified assumptions.

(ii) Shop Weld Lines Attaching the Beam to the Framing Angles:

The shop fillet welds connecting the beam web to the framing angles is an eccentric welded shear connection. The force R_b is the reaction of the floor deck, the superimposed loads and the own weight of steel of main and secondary beams from center to center of the neighbouring columns. These welds will transmitt the reaction R_b to the framing angles. Fig. 4.5 illustrates a typical configuration of this type of welds. For the design, proceed as follows:

- Choose the size of welds "s" equal to the thickness "t" of the framing angles previously deduced.
- Assume a set back of 12 mm from the edge of the beam to the column's flange.

- The reaction R_b and the torsional moment $M_t = R_b \cdot \bar{x}$ will create shear stresses along Section (1-1) as follows :-

$$\left. \begin{aligned} q_{A//} &= \frac{R_b}{[2(a-12\text{mm})+h] s} \\ q_{A(Mt)//} &= \frac{M_t \cdot (h/2)}{I_p} ; \quad q_{A(Mt)\perp} = \frac{M_t \cdot (a-12\text{mm}-\bar{x})}{I_p} \end{aligned} \right\} \quad (i)$$

where $I_p = I_x + I_y$ of the I weld shape.

Check that $q_t = \sqrt{q_{//}^2 + q_{\perp}^2} \leq 0.2 F_U$

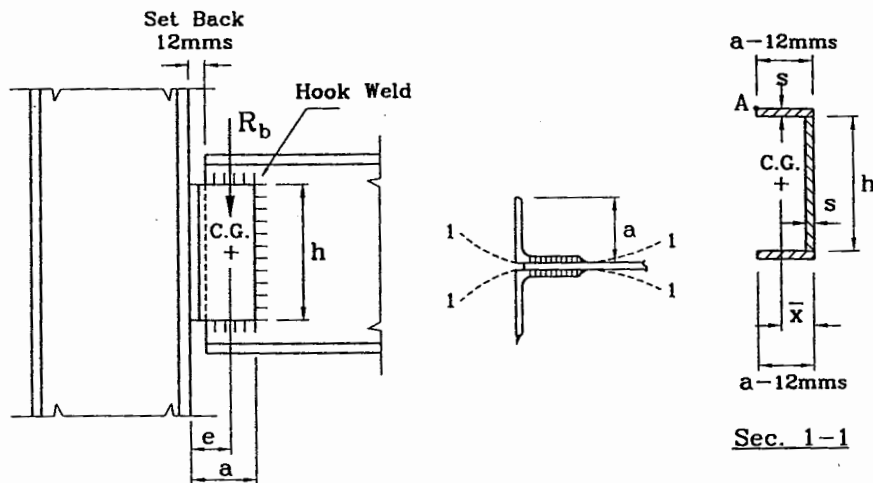


Fig. 4.5.

(iii) Field Fillet Welds Connecting Column to Framing Angles:

Referring to Fig. 4.4.b, two different planes (1-1) and (2-2) are to be analyzed. For plane (1-1) the induced stresses will be as follows:

- Tensile bending stresses as previously deduced in Equ. 4.1.
- Vertical shear stresses due to the concentric shear R_b are given by:

$$q_{\downarrow\downarrow} = R_b / 2.s.h \quad (j)$$

- The equivalent stresses are to be calculated and checked. Thus using Eqs. 4.1 and (j) we get:

$$f_e = \sqrt{f_{t\perp}^2 + 3 q_{\downarrow\downarrow}^2} \leq 1.1 (0.2 F_U)$$

On the other hand the induced stresses on section 2-2 will be as follows:

- Concentric shear stresses:

$$q_{\downarrow\downarrow} = R_b / 2.h.s \quad (k)$$

- Eccentric shear stresses due to $M_t = R_b \cdot e$:

$$q_{Mt\perp}^{\leftarrow} = 3 R_b e / s h^2 \quad (L)$$

- The resultant shear stresses are checked as follows:

$$q_{total} = \sqrt{q_{\downarrow\downarrow}^2 + q_{Mt\perp}^2} \leq 0.2 F_U$$

It is to be noticed that the vertical shear stress $q_{\downarrow\downarrow}$ for both planes (1-1) and (2-2) is induced by the vertical reaction R_b which is transmitted to these planes via an eccentricity ($e' = a = \text{leg size}$) of framing angle. This will induce an

additional torsional stress ($q_{Mt\perp}^{\rightarrow} = \frac{6 R_b \cdot a}{2 s h^2}$) and a bending normal stress ($f_{t\perp} = \frac{6 R_b \cdot a}{2 s h^2}$) regarding planes (1-1) and (2-2)

respectively. These additional stresses may be added to the previous computed stresses.

Refer to Example 4.1.



4.3.2. Bolted Connections:

The procedure previously outlined in Section 4.3.1, will be applied. One must differentiate between the behavior of ordinary and high strength bolted connections.

4.3.2.1. Ordinary Bolted Connections:

(i) The Framing Angles Size:

Fig. 4.6.d, and 4.6.b, illustrate the expected deflected shape of the framing angles as well as the corresponding bending stresses on group of bolts (2).

The reaction R_b will be transmitted to the framing angles via the group of bolts (1) and then to the column's flange via the group of bolts (2).

The induced moment ($M = R_b \cdot (a + t)/2$) will create tensile stresses (f_{tb}) on group of bolts (2) as shown in Fig. 4.6.b

The stress (f_{tb}) is computed as previously given in Chapter (1) in Section (1.10).

Once the tensile stresses (f_{tb}) are computed, the tensile force T can be determined as follows:

$$T = f_{tb} \cdot A_s \quad (a)$$

The framing angles will behave as a simple beam of span $(a + t + t_{wb})$, thus the induced moment at the critical Section (1-1) will be:

$$M' = T \cdot \left(\frac{a + t}{2} - K \right) \quad (b)$$

Applying in the stress equation ($f = \frac{M \cdot Y}{I}$) using for M and T Equations (a), (b) we get:

$$I = \frac{t^3 p}{12}; \quad Y = \frac{t}{2} \quad \text{We get:}$$

$$f_b = \frac{6 f_{tb} A_s (a/2 + t/2 - K)}{t^2 p} \quad 4.4$$

Where: P = pitch between bolts; the edge distance being taken equal to $p/2$;

A_s = stress cross sectional area of bolt.

Applying the allowable bending stress ($F_b = 0.58 F_y$) in Equ. 4.4, determine the required thickness (t) of the framing angles. One can say that the section is solid rectangular and $F_b = 0.72 F_y$, but we prefer to be conservative.

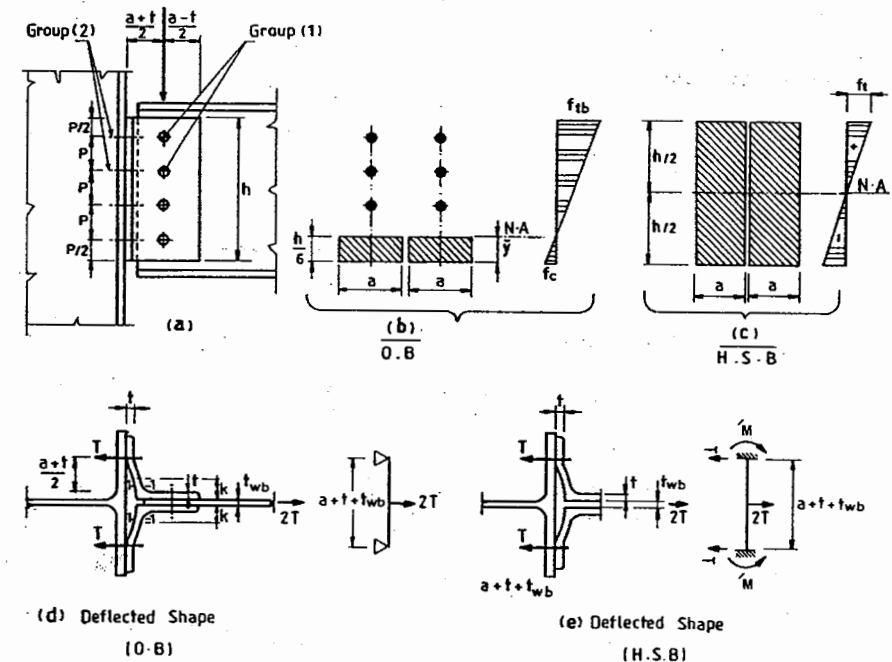


Fig. 4.6

ii) The Group of Bolts (1):

These bolts are subjected to concentric shear stresses

$$q = \frac{R_b}{n.A} \quad (c)$$

Where : n = total number of bolts. (4 bolts as shown in Fig. 4.6.a).

A = the bolt shank area where threads lie outside the two planes of shear.

iii) The Group of Bolts (2):

- These bolts are subjected to tensile stresses due to

$M = R_b \left(\frac{a+t}{2} \right)$ see Fig. 4.6.b and refer to Section 1.10 of

Chapter (1).

- Shear stresses induced by the reaction R_b (single shear)

$$(q_{\downarrow} = \frac{R_b}{2n.A}) \quad (d)$$

It is to be noticed that the induced shear stresses ($q_{\downarrow}$) are resulting from the vertical reaction R_b which is transmitted

to the group of bolts (2) via an eccentricity ($e' = \frac{a+t}{2}$).

This eccentricity will induce additional torsional stresses (q_{Mt}) on the group of bolts (2):

$$q_{Mt} = \frac{(R_b/2) \times (a+t)/2 \times (h/2 - p/2)}{A \times \sum_{i=1}^n y_i^2} \quad (e)$$

Where: a = the leg size of the framing angle.

y_i = the bolt height (i.e. the distance between the corresponding bolt and the mid-height axis).

The additional stress (e) is to be added to the previously computed vertical shear stress (d) and checked.

Refer to Example 4.2

4.3.2.2. High Strength Bolted Connections:

(i) The Framing Angle Size:

Fig. 4.6.e and 4.6.c, illustrate the expected deformed shape of the framing angles as well as the corresponding bending stresses on group of bolts (2).

The induced moment ($M = R_b \times (a+t)/2$) will create bending stresses on group of bolts (2) as shown in Fig. 4.6.c.

$$f_{tb} = \left[1.5 R_b (a+t) (h-p) \right] / ah^3 \quad (a)$$

This equation is deduced using Method (A) of Section 2.7.5 of Chapter 2.

Once the tensile stress (f_{tb}) is calculated, the tensile force T can be determined as follows:

$$T = f_{tb} \cdot p \cdot a \quad (b)$$

The framing angles will behave as a beam fixed at both ends as shown in Fig. 4.6.e. The clamping force induced between the column's flange and the framing angles will create this fixation. To be noticed that this phenomenon is only valid for high strength bolts where a pretension is applied. The induced moment due to T at support fixation is:

$$M' = \frac{2T}{8} (a + t + t_{wb})$$



$$M = \frac{T}{4} (a + t + t_{wb}) \quad (c)$$

Applying in the stress equation $f = \frac{M.y}{I}$:

Where: $I = \frac{t^3 p}{12}$; $y = t/2$; M , T , and f_{tb} from equations (c), (b), and (a) respectively we get:

$$f_b = \frac{2.25 R_b (a + t + t_{wb}) (a + t) (h - p)}{t^2 . h^3} \quad 4.5$$

Applying the allowable bending stress ($F_b = 0.58 F_y$) of the corresponding steel in Equation 4.5, the thickness of the framing angles can be computed.

(ii) The Group of Bolts (1):

These bolts are subjected to concentric shearing forces $R_{bolt} = R_b/n$ which must not exceed the frictional resistance P_s of Table (2.6) where two planes of friction are to be considered.

(iii) The Group of Bolts (2):

These bolts will be subjected to :

- Tensile external forces due to the induced moment.

$$M = R_b \left(\frac{a + t}{2} \right); \text{ Refer to equation (b).}$$

- Concentric shearing forces induced by the reaction R_b .

Refer to Example 4.2.

General Notes Regarding the Flexible Framing Angles Connections:

1- One may observe from the previous sections that the use of



thick angles will reduce the stresses (f_b), and consequently the deformation of the framing angles will decrease offsetting the required flexibility. Thus it is recommended to choose the thinnest angles as possible.

- 2- Also for flexibility purposes, the bolted connection is to be dimensioned so that the pitch (P) is chosen as greater as possible. This means that the height of the framing angles must be taken equal as possible to the height of the beams minus its breadth but may be a little deeper.
- 3- The use of high strength bolts is to be omitted if possible. The induced pretension creates clamping forces that will reduce the flexibility of the connection. (i.e. will reduce the rotation).
- 4- For welded connections, erection bolts are generally placed near the bottom of the angle, so that they do not restrain the beam end from rotating under the reaction R_b .
- 5- For deeper girders, these erection bolts may be placed near the top of the framing angles for stability requirements. These bolts can be removed after field welding.
- 6- The analysis previously mentioned for H.S. Bolted connections using Method (A) (Section 2.7.5) neglects the clamping effect, hence the deduced thickness (t) of the framing angles is very conservative.

Moreover, for the friction type H.S. Bolted connections the clamping forces will reduce considerably the tensile stresses (f_b), hence the size of the framing angles can be safely chosen according to the constructional requirements.

7- A common practice today is to shop weld the angles to the beam and field connect them to the columns with high strength bolts.

8- Leg of framing angles are usually from 8.0 cms till 10.0 cms. Sometimes unequal legs are necessary with the longer legs connected to the column flange.

4.4. Single Web Framing Angle:

Generally, the use of single web framing angles is not recommended: the induced tensile stresses on the high strength bolts connecting the framing angle to the column flange are relatively high and the corresponding thickness (t) of the angle will be extremely thick.

However, if this type of flexible connections is utilized, the following analysis can be adopted:

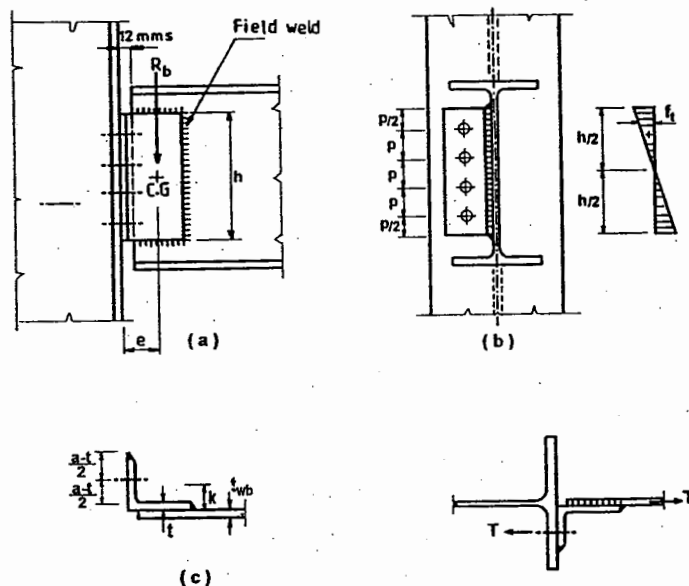


Fig. 4.7. Single Web Framing Angle

(i) The Framing Angle Thickness:

The deformed shape of the framing angles is somewhat rather complicated. The simplified assumption is to consider a cantilever beam behavior as shown in Fig. 4.7.c.

The induced moment about the center line of the high strength bolts chosen of the friction type is:

$$M = R_b \cdot e \quad (a)$$

The induced bolt bending stresses (f_{tb}) are to be computed according to the following relation:

$$f_{tb} = \frac{6 R_b \cdot e \cdot (h-p)}{a h^3} \quad (b)$$

The corresponding tension force T per high strength bolts is:

$$T = f_{tb} \times p \times a$$

where: p = the pitch between the high strength bolts

a = the angle leg size

Equating (b) and (c) get "T":

$$T = \frac{6 R_b \cdot e \cdot (h-p) \cdot p}{h^3} \quad (d)$$

This tension force "T" will induce bending stresses (f_b) along the high strength bolts center line regarding the angle leg. These bending stresses can be computed as follows:

$$f_b = \frac{6T \cdot (a/2 + t/2 - k)}{p \cdot t^2} \quad (e)$$

Replacing "T" in equation (e) using relation (d) we



get:

$$f_b = \frac{18 R_b \cdot e \cdot (h-p)(a+t-2k)}{h^3 \cdot t^2} \quad (f)$$

Replacing in relation (f), f_b by the allowable compressive stress ($F_b = 0.58 F_y$), the angle thickness "t" can be computed.

(ii) The Shop Weld Lines Attaching the Beam Web to the Framing Angle:

Concerning the shop fillet weld lines connecting the beam to the framing angle an eccentric shear will be induced as for the case of the double framing angles previously outlined in section 4.3.

(iii) The High Strength Bolts Attaching the Framing Angle to the Column Flange:

On the other hand the high strength bolts attaching the framing angle to the column flange will be subjected to the following straining actions:

(a) The eccentric shearing force $Q = R_b$

(b) The torsional moment $M_t = R_b \cdot \frac{(a+t)}{2}$

(c) The flexure moment $M = R_b \cdot e$

The corresponding induced forces on the most stressed high strength bolts are:

$$\begin{aligned} Q_b \downarrow &= \frac{R_b}{n} \\ \rightarrow Q_{Mt} &= \frac{R_b \cdot (a+t) \cdot 1.5p}{2 [(0.5p)^2 + (1.5p)^2]} \end{aligned} \quad \begin{array}{l} (g) \quad h \\ (h) \end{array} \left. \begin{array}{l} \\ \end{array} \right\} \begin{array}{l} \text{Resultant} \\ < P_s \end{array}$$



T = as previously outlined in equation (d). The tension force "T" is to be checked according to the prescriptions of sections 2.7.

It is to be observed that the computed thickness of the framing angle may be excessively thick. For the common connections, the thickness of relation (f) was never respected. Fortunately the framing angles had never shown any signs of failure.

However, it is suggested to allow for some yielded zones, which means that the allowable bending stress is allowed to reach the yield stress " F_y ".

4.5. The Cleat Plate Web-Beam Shear Connection:

As an alternative to web framing angles, plate cleats (called also fin plates) may be used to transmit the beam reaction to the column flange. They are specially used when attachments to the minor axis of small columns are required (see Fig. 4.8). The cleat plates are shop welded to the column web and then field bolted to the beam web. This configuration has the advantage that welding access problems are avoided.

Generally the beam flanges are to be notched, so that the beam can be dropped into position.

For this type of connections three aspects are to be safely analyzed:

(a) The Bolts will be Subjected to Concentric Shear:

$$R_{\text{bolt}} = \frac{R_b}{n} \quad (a)$$

Where n is the total number of bolts behaving in single shear.

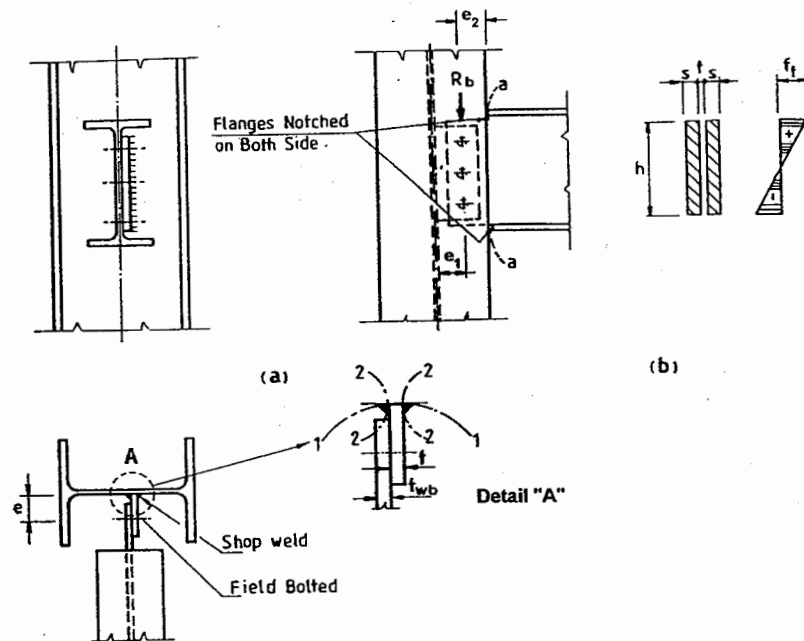


Fig. 4.8 Cleat Plate

The value obtained in equation (a) must not exceed either P_s for H.S.B or R_{LEAST} for ordinary bolts.

(b) The Stresses Induced in the Fillet Weld Lines are:

Plane (1-1):
$$f_{t\perp} = \frac{3R_b \cdot e}{h^2 \cdot s} ; \quad q_{\searrow} = \frac{R_b}{2 h s} ;$$

Hence calculate the equivalent stresses (f_e) and check :

$$f_e \leq 1.1 (0.2 F_U)$$

Plane (2-2):
$$q_{\searrow} = \frac{R_b}{2 \cdot h \cdot s} ; \quad q_{\nearrow} = \frac{3R_b \cdot e_1}{h^2 \cdot s} ;$$

Hence calculate the resultant shear stresses to be $\leq 0.2 F_U$



(c) The Thickness (t) of the Cleat Plate :-

$$t = \frac{6R_b \cdot e_1}{F_b \cdot h^2} \quad (b)$$

F_b in Equ. (b) is to be replaced by the allowable bending stresses ($F_b = 0.72 F_y$). Here the analysis is exact, the section being a solid rectangular section.

It is to be noticed that the beam web at section (a-a) is to be checked against the bending stresses as follows :-

$$f_b = \frac{6R_b \cdot e_2}{t_{wb} \cdot h^2} \leq 0.72 F_y \quad (c)$$

If the beam web thickness (t_{wb}) is not sufficient to overcome the allowable bending stresses, one or two horizontal stiffeners are to be utilized as given in Section 4.14.1.1.

It is to be noticed that the effect of shear stresses have been neglected for the determination of the web cleat thickness (t) as well as for the beam web thickness (t_{wb}).

In order to ensure that this type of connections is sufficiently flexible one can observe the following:

- Once again the use of ordinary bolts is recommended, while the high-strength-bolts show some rigidity created by the pretension force which induces clamping forces between the cleat plate and the beam web.

- The plate thickness must not exceed half of the diameter (0.5 d) of bolts for steel 37 and 0.4 d for steel 52 in order to ensure bearing deformation of the cleat plate.

- The edge distance and the pitches are to be chosen as greater as possible in order to allow sufficient flexibility.

- The used fillet weld lines must achieve a minimum capacity. This condition can be ensured if the designer chooses the size of welds $\sum s$ not exceeding the thickness (t) of the cleat plate.

Refer to Examples 4.4 and 4.5.

4.6. The Shear End Plate-Beam-to-Column Connection:

Fig.4.9, illustrates what we call the shear end-plate beam-to-column connections. Three different configurations are available where their corresponding behavior is as given below:

Configuration (a):

Fig. 4.9.a, shows an end plate welded to the beam web by a shop fillet weld subjected to concentric shear stresses for both planes (1-1) and (2-2). Thus:

$$q = \frac{R_b}{2.s.h} \quad (a)$$

The field fillet weld is subjected to the same concentric shear (R_b) where equation (a) remains valid.

Configuration (b):

Fig. 4.9.b, illustrates an end plate shop welded to the beam web. This type of connections has the advantage that all hole drilling in the web are eliminated. The bolts connecting the end plate to the column flange are subjected to a concentric

shear which must be safe against both single shear and bearing. Thus:

$$R_{bolt} = \frac{R_b}{n} \quad (b)$$

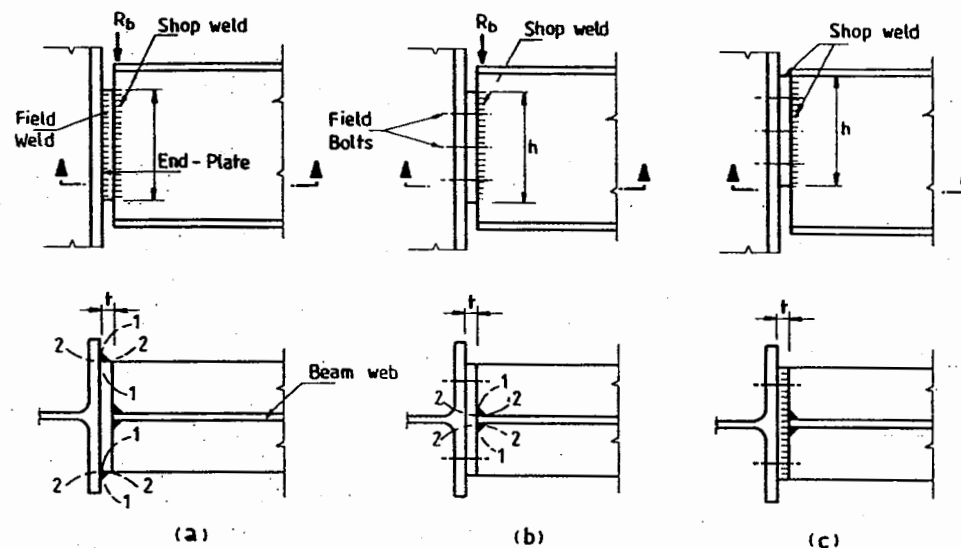


Fig. 4.9

Regarding the end plate thickness, one must check the safety against the shear capacity within the end plate on critical section passing through the bolts holes. Hence:

$$q = \frac{R_b}{2t(h - n\phi)} \quad (c)$$

Where n = the number of bolts in one row.

d = the bolt diameter.

$\phi = d + 2 \text{ mms.}$

t = thickness of end plate.



The induced shear stress q of Equation (c) must not exceed the allowable stress q_{all} as prescribed by the E.C.p. 2001. It is to be noticed that the existing eccentricity is so small to introduce flexural stresses.

Configuration (c):

The end plates may be extended on the full depth of the beam. For such a configuration the upper and lower edges of the end plates are fillet welded to the beam flanges. This will increase the deformation of the top region of the connection where the beam end rotation will be forced down to the bottom flange. Fig. 4.9.c, illustrates an end plate whose top is shop fillet welded to the upper beam flange. This end plate is not extended till the bottom flange. This extension is practically unnecessary where the top edge ensures the required beam end rotation.

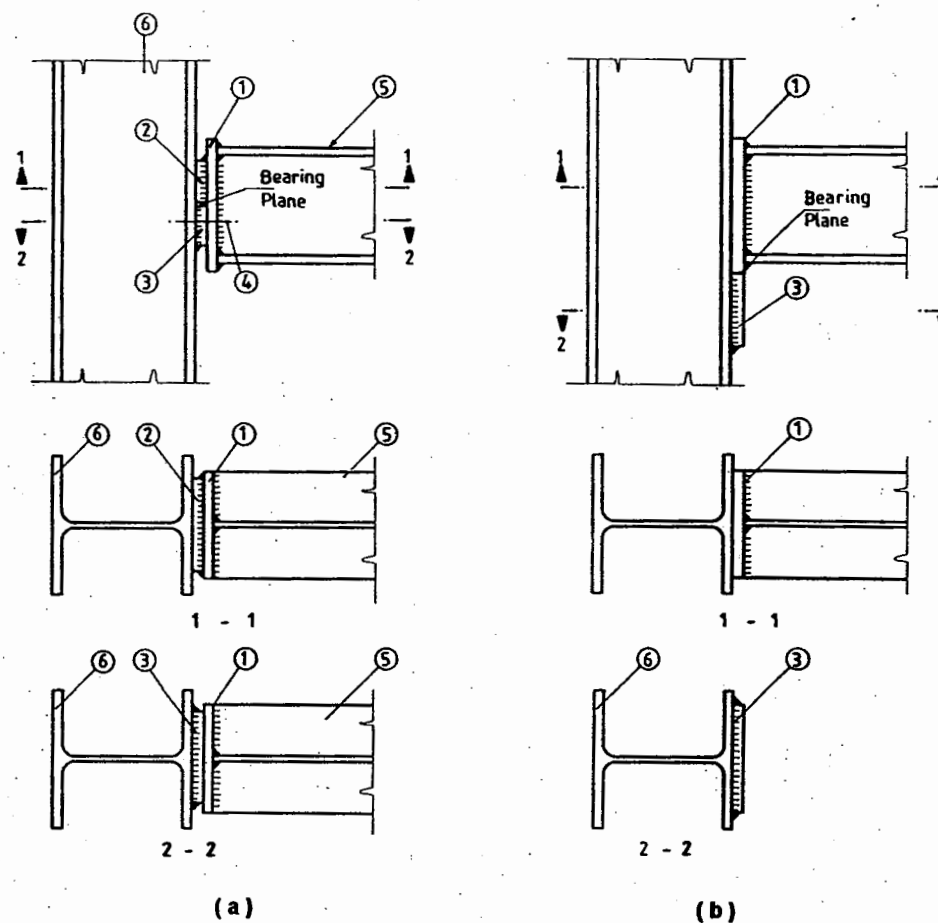
Equations (a), (b), and (c) of the preceding paragraphs are to be applied to check the safety against stresses.

It is to be noticed that in order to ensure the flexibility of such type of connections, one must choose the thickness of the end plate of the order of one third to half the bolt diameter. This choice will provide failure of bolts by shear rather than by bearing. Also the pitch between bolts must not be less than four times the bolt diameter. A double fillet weld lines is highly recommended rather than a single line. In order to ensure that the bottom flange of the beam will not bear on the face of the column, the height of the end plate is to be chosen half the beam depth.

Refer to Examples 4.6.



4.7. The Bearing End Plate Connections:



- (1) The welded beam end plate.
- (2) The bearing beam end plate.
- (3) The bearing column end plate.
- (4) Bolts.
- (5) The beam.
- (6) The column.

Fig. 4.10



Two types of the so-called bearing end plate web-to-beam connections are shown in Fig. 4.10. a, b. Both types are generally utilized when the shear transmitted from the beam to the column is relatively high.

Fig. 4.10.a, illustrates the first configuration where the end plate (1) is shop fillet welded to the beam flange and beam web. An end bearing plate (2) is fillet welded to the end plate (1) on its upper and two longitudinal sides while the lower side is resting on the upper edge of the bearing plate (3). The beam reaction is thus transmitted by bearing to the end plate (3) and then to the column flange via the fillet weld lines attaching the end plate (3) to the column flange.

Fig. 4.10.b, shows another configuration where the reaction of the beam is transmitted to the beam end plate (1) via the shop fillet welds and then to the end plate (3) by bearing. This configuration requires one less plate and less welding.

This Type of Connections Require: (a) excessive fabrication where adequate bearing area must be provided to transmit the vertical reaction of the beam. (b) it must be ensured that there are no possibility of the beam slipping off the column bearing end plate (3). (c) nominal bolts (two bolts) are to be positioned near the effective center of rotation. (d) The bearing end plates (2) and (3) are only welded on three sides and must be sufficient regarding the allowable shear stresses.

4.8. The Directly Welded Beam-Web Connection:

The web may be welded directly to the column. The weld lines must have sufficient height (h) in order to transmit safely the reaction R_b of the beam.



If there is a gap between the beam web and the column flange, the size (s) of the fillet weld is increased by the same amount (see Fig. 4.11).

Tests of this type of connections have shown initial signs of yielding adjacent to the lower ends of the weld lines at a moment beam less than the yield one. It is to be noticed that this type of connections does not ensure adequate rotation for flexibility requirements.

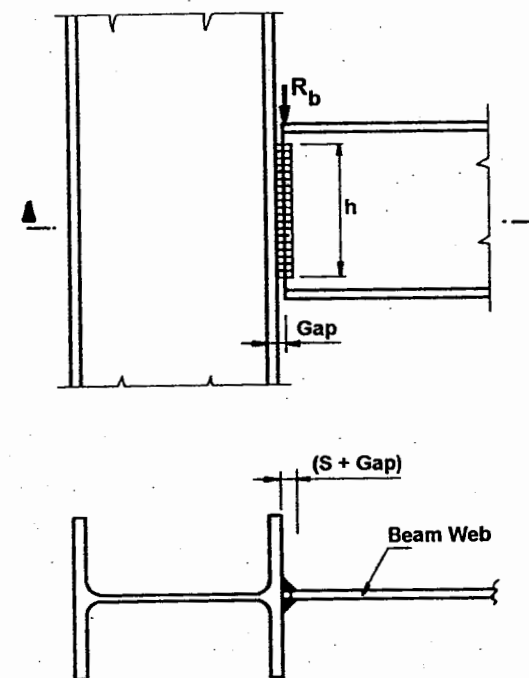


Fig. 4.11

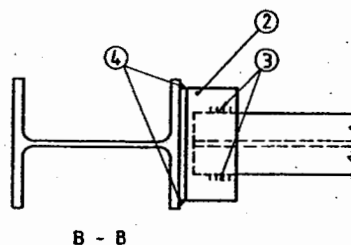
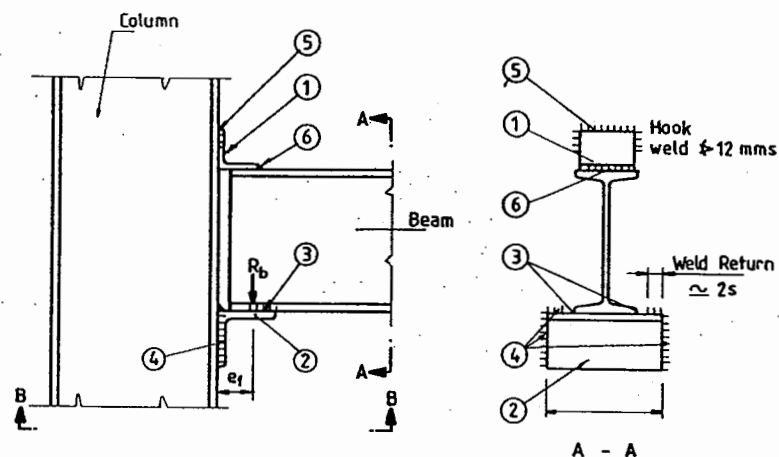
For all these reasons, this configuration is not recommended.



4.9. The Unstiffened Seated Beam-Connections:

4.9.1. Welded Unstiffened Seated Beam Connection:

Fig. 4.12, illustrates a typical welded seated-beam connection. The reaction of the beam R_b will be transmitted to the seat angle (2) by bearing. The seat angle tends to deflect downward which induces bending stresses. The behavior of the deflected seat angle is rather a complicated problem depending mainly on the thickness of the seat angle.



- (1) Top clip angle
- (2) Seat angle
- (3) Beam-to-seat angle weld (field)
- (4) Column-to-seat angle weld (shop)
- (5) Top clip-angle-to-column weld (field)
- (6) Top clip-angle-to-beam weld (shop)

Fig. 4.12



This phenomena is illustrated in Fig. 4.14 and will be discussed later in this section. Assuming a safe seat angle thickness regarding bending, the reaction R_b will be transmitted to the column flange by the shop weld (4) which is subjected to a concentric shear, a torsional moment ($M_t = R_b \cdot e_f$) and/or a flexural moment ($M = R_b \cdot e_f$). The beam is preferably attached to the seat angle by a beam-to-seat angle field weld (3). This will force the reaction R_b to act by bearing on the surfaces in contact. The critical seat angle section will probably be at the base of the fillet of the outstanding leg of the seat angle (Section a-a Fig. 4.13). The field weld (3) can be replaced by two bolts as shown in Fig. 4.13.b. If the beam is not attached to the seat angle, the critical section will be probably Section (b-b) as illustrated in Fig. 4.13.c.

As shown in Fig. 4.12, a top clip angle (1) is to be field welded to the top of the beam in order to ensure the stability of the beam against the lateral torsional buckling. Another configuration consists to use a top clip angle attached to the beam web.

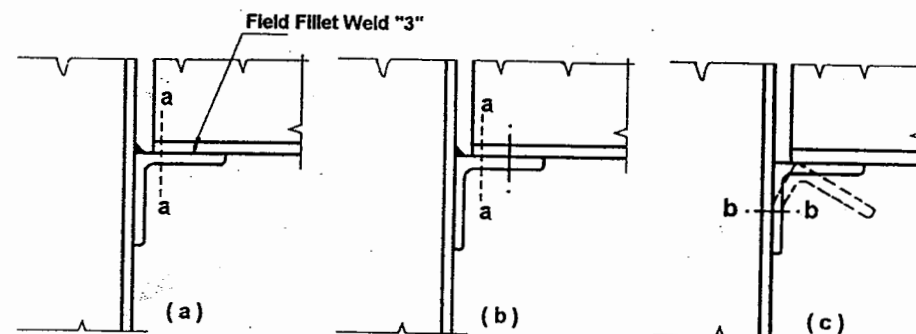


Fig. 4.13



As the thickness of the seat beam is a direct function of the induced bending moment, one must observe the following (see Fig. 4.14):

- (a) The conservative assumption considers the beam reaction R_b acting at the centroid of the bearing stress area (see Fig. 4.14.a).

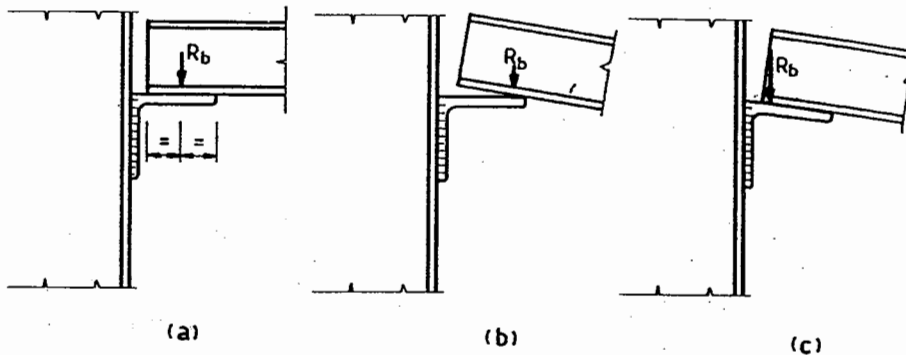


Fig. 4.14

- (b) The two other possibilities will follow the fact that when the beam deflects downward, its ends will rotate. Consequently the point of contact of the reaction R_b tends to move outward or inward for thick and thin seat angles as illustrated in Figs 4.14. b, c respectively.

Due to lack of accurate studies, we will consider the conservative assumption of Fig. 4.14.a in the analysis presented below.

For the seat angle connection of Fig. 4.12, four aspects are to be determined: (see Fig. 4.15)



- (a) The seat width (a)
- (b) The moment arms (e), (e_f)
- (c) The seat angle dimensions. (a , t)
- (d) The weld lines size (h , s)

(a) The Seat Width (a):

The seat width (a) depends on the bearing length (N) which is based on the crippling stress F_{crp} along Section (m-m). Referring to the E.C.P. (2001), clause (3.6.6) the following equation can be utilized:

$$\frac{R_b}{(N + K) t_{wb}} \leq F_{crp} \leq 0.75 F_y$$

$$\text{i.e. } N = \frac{R_b}{0.75 F_y \cdot t_{wb}} - K \geq K \quad (a)$$

Assuming a gap of 12 mms between the beam edge and the column flange, hence "a" can be computed:

$$a = N + 12 \text{ mms} \quad (b)$$

(b) The Moment Arms (e) and (e_f):

From Fig. 4.15 one can deduce:

$$\left. \begin{aligned} e_f &= \frac{N}{2} + 12 \text{ mms.} \\ \text{and } e &= e_f - t - 10 \text{ mms.} \end{aligned} \right\} \quad (c)$$

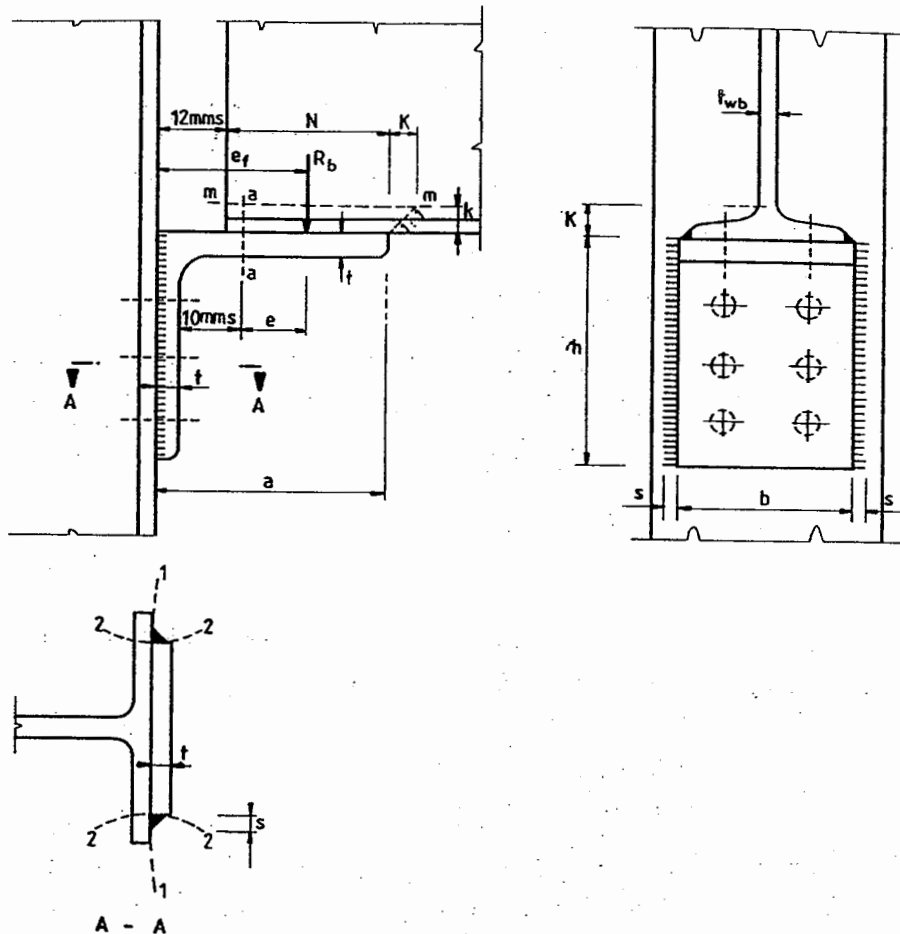


Fig. 4.15

(c) The Seat Angle Dimensions:

At the critical section (a-a), the induced moment (M) is:

$$M = R_b \cdot e \quad (d)$$

Applying in the stress equation

$$f_b = \frac{M \cdot y}{I}; \quad I = \frac{bt^3}{12}; \quad y = t/2 \text{ we get:}$$

$$t = \sqrt{\frac{6 M}{F_b \cdot b}} \quad (e)$$

Where: F_b = allowable bending stress.

Equating (d) in (e) we get:

$$t = \sqrt{\frac{6 R_b \cdot e}{F_b \cdot b}} \quad (f)$$

(d) The Fillet Weld Lines Size (s):

Regarding face (2-2) of the fillet welds, the induced stresses are:

$$q_{\downarrow\backslash} = \frac{R_b}{2 \cdot h \cdot s}; \quad q_{Mt_{\perp}} = \frac{3R_b(e_f)}{s \cdot h^2}$$

Both shear stresses are to be added together vectorially and checked against the allowable shear stresses of fillet welds:

While for face (1-1), the induced stresses are:

$$q_{\downarrow\backslash} = \frac{R_b}{2 \cdot h \cdot s}; \quad f_{t_{\perp}} = \frac{3R_b(e_f)}{s \cdot h^2}$$

The equivalent stresses $f_e = \sqrt{f_{t_{\perp}}^2 + 3 q_{\downarrow\backslash}^2}$ are to be calculated and checked against the allowable values.

N.B. The top clip angle is generally chosen 100 x 100 x 10 with a maximum hook weld of 12 mms.

4.9.2. Bolted Unstiffened Seated-Beam Connection:

In this configuration all welds of Fig. 4.12 are replaced by bolts. This means that:

(a) The vertical leg of the seat angle will be connected to the column flange by bolts (shown dotted in Fig. 4.15). These bolts will be subjected to concentric shear R_b and a flexural moment ($M = R_b \cdot e_f$), (Refer to Sections (1.10) and (2.7.5) for ordinary and high strength bolted connections respectively.

(b) The horizontal leg of the seat angle will be connected to the beam flange by two bolts shown dotted in the side view of the connection of Fig. 4.15.

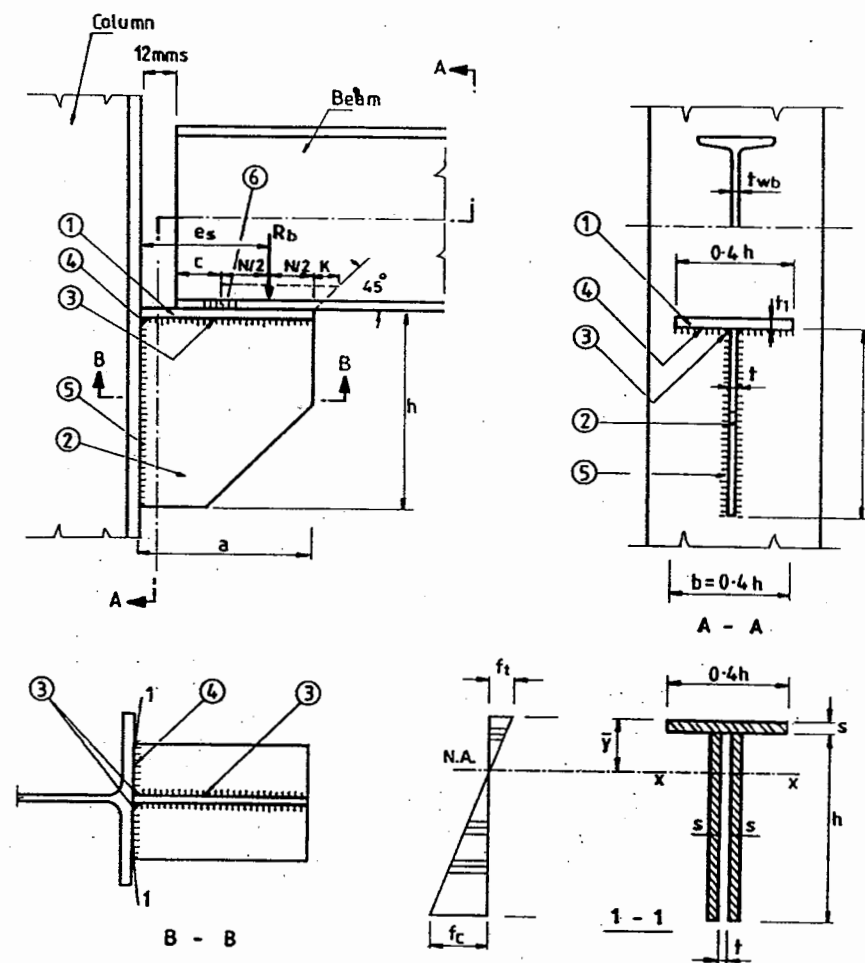
(c) Top clip angle will be connected to the column flange and to the beam top flange by four bolts two on each leg.

Refer to Examples 4.7 and 4.8.

4.10. Stiffened Rectangular Seat Bracket:

For relatively great beam reactions R_b , the seat angle thickness becomes excessive. An alternative configuration consists to use vertical stiffeners attached by fillet welds to a top bearing plate.

Fig. 4.16, illustrates this type of connections where the plane of the beam web transmitting the reaction R_b is located in the same plane as that of the rectangular vertical stiffener of the bracket.



- (1) Top plate of bracket.
- (2) Vertical stiffener of bracket;
- (3) Welds attaching top plate to vertical stiffeners;
- (4) Horizontal welds attaching top plate to column flange;
- (5) Vertical welds attaching stiffener to column flange;
- (6) Welds attaching top plate to beam flange.

Fig. 4.16

The reaction (R_b) is transmitted from the lower beam flange to the top plate by bearing along a distance (N). The vertical stiffener then transmits R_b via the weld lines (4) and (5) to the column flange.

The following steel structural elements are to be analyzed:

- The bearing length (N).
- The moment arm (e_s).
- The weld lines (4) and (5).
- The vertical stiffener thickness (t).
- The top plate thickness (t_1), the weld size of the horizontal weld line (3) and the dimensions of the weld lines (6).

(a) The Bearing Length (N) and the Moment Arm (e_s):

The analysis follows that of the seat angle connection.

Refer to Fig. 4.16 to compute the following:

$$\frac{R_b}{(N + K) \cdot t_{wb}} \leq F_{crp} \leq 0.75 F_y$$

Thus:

$$N \leq \frac{R_b}{t_{wb} \cdot (0.75 F_y)} - K \geq K \quad (a)$$

The beam is to be extended beyond the bearing length a convenient distance say 30 to 50 millimeters. Hence the moment arm e_s can be determined:

$$e_s = \frac{N}{2} + C + 12 \text{ mms.} \quad (b)$$

and

$$a = N + C + 12 \text{ mms.} \quad (c)$$

(b) The Weld Lines (4) and (5):

The stiffened seat bracket is shop welded to the column flange by a horizontal weld line (4) and two vertical weld lines (5) of height (h). Generally the horizontal weld line (4) is welded on the underside of the bracket top plate and for simplicity of analysis is assumed to be a percentage of the vertical weld length (h) (say 0.4 h).

The weld face along Section 1-1 will be subjected to the following stresses:

$$q = \frac{R_b}{2.4 h \cdot s} \quad (d)$$

$$f_{t\perp} = \frac{R_b \cdot e_s \cdot (h - \bar{y})}{I_x} \quad (e)$$

Where $\bar{y}$ and I_x are to be determined from Table 3.2 for a unit length size ($s=1$).

The equivalent stresses $f_e = \sqrt{f_{t\perp}^2 + 3 q^2}$ are to be equated to the corresponding allowable values (i.e 0.22 F_u) and hence h can be computed.

It is to be observed that the weld size (s) may be chosen equal to 3/4 the vertical stiffener thickness(t).

(c) The Stiffener Thickness (t):

The chosen stiffener thickness (t) must satisfy the following:

- The stiffener thickness (t) must be at least as thick as the beam web (t_{wb}).

$$t \geq t_{wb} \quad (f)$$

- In order to prevent the local buckling the thickness (t) must satisfy the constraints previously outlined in Chapter 5 - volume (1), where:

$$t \geq \frac{a}{16} \quad (g)$$

- The induced bending stresses must not exceed the allowable limit.

This means that (t) can be computed using the allowable bending stress f_b in the following relation:

$$f_b = \frac{R_b}{a \cdot t} + \frac{R_b (e_s - a/2)}{t \cdot a^2 / 6} \quad (h)$$

- If we assume that the maximum shear stresses will be created simultaneously in the vertical lines of welds (5) as well as in the stiffener (2), thus the following relation can be computed:

$$2 \cdot h \cdot s \cdot (0.2 F_U) = h \cdot t \cdot (0.58 F_y)$$

Hence: $t \geq 1.66 \cdot s \quad (i)$

Where: $(0.2 F_U)$ and $(0.58 F_y)$ are the allowable shear stresses for the fillet weld lines and the stiffener respectively.

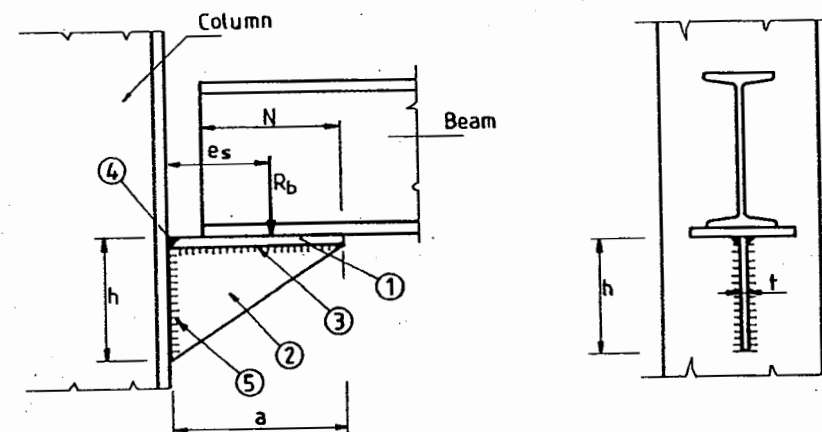
It is to be noticed that h, t and s must be in accordance with Equations (d), (e), (f), (g), (h), and (i).

On the other hand the top plate thickness t_1 is to be chosen equal to the beam flange thickness while the size of the weld lines (3) equal to the size (s) of welds (5). The weld lines (6) attaching the top plate to the beam flange is to be at least of 50 mms length and 6 mms size.

Refer to Example 4.9.

4.11. Triangular Stiffened Seat Brackets:

Triangular bracket plates are utilized as support brackets, as stiffeners and as buttresses in heavy frame joints.



- (1) Top plate bracket;
- (2) Triangular stiffener;
- (3) Weld attaching the top plate to the stiffener;
- (4) Weld attaching the top plate to the column flange;
- (5) weld attaching the stiffener to the column flange.

Fig. 4.17

The behavior of the triangular stiffeners differs from the rectangular one analyzed in the previous section. The stiffener behaves as a plate supported at two sides and free at the sloping edge. However plate buckling and stress conditions will determine the required thickness of the stiffener while the two supported side dimensions will be governed by the construction conditions.

Fig. 4.17, illustrates the configuration of a triangular stiffened seat bracket. It has been observed that the maximum stress occurs at the free edge and that the failure is governed by yielding rather than by buckling. An expression relating the average stress on the loaded edge ($f_{AV} = R_b / a.t$) to the maximum stress occurring at the free edge (f_{MAX}) was found to conform with the following relation:

$$m = \frac{f_{AV}}{f_{MAX}} = 1.39 - 2.2 \left(\frac{a}{h}\right) + 1.27 \left(\frac{a}{h}\right)^2 - 0.25 \left(\frac{a}{h}\right)^3 \quad (a)$$

The triangular stiffeners will be governed by either the buckling or the yielding conditions whichever is the most critical:

(a) Buckling Condition:

$$F_{cr} = \frac{K \pi^2 E}{12 (1-\mu^2) (h/t)^2} \quad (b)$$

where F_{cr} is the maximum stress on the free edge which must not exceed the yield stress (F_y).

Replacing in equation (b): $f_{cr} = f_y$, $\pi = 3.14$, $E = 2100$

and $\mu = 0.3$, Equation (b) becomes:

$$\frac{h}{t} \leq 43.54 \sqrt{\frac{K}{F_y}} \quad (c)$$

Using the appropriate theoretical values of $K = 1.23$, Equation (c) becomes:

$$\frac{h}{t} \leq \frac{48}{\sqrt{F_y}} \quad (d)$$

(b) Yielding Condition:

Using the average stress relation in conjunction with Equation (a) we get:

$$f_{AV} = \frac{R_b}{a.t} = m \cdot f_{MAX}$$

$$\text{i.e.} \quad f_{MAX} = \frac{R_b}{m.a.t} \leq 0.58 F_y \quad (0.58 F_y \text{ is the allowable stress})$$

$$\text{Thus:} \quad t \geq \frac{R_b}{0.58 F_y \cdot m.a} \quad (e)$$

The thickness (t) must satisfy both conditions (d) and (e) in order to overcome both buckling and yielding considerations. To be noticed that the top plate was considered in the analysis rigidly connected to the column flange, also it has been assumed that the reaction R_b was uniformly distributed along the side a and that it lies at a distance (e_s) closer as possible to the column flange.

Concerning the other steel structural elements one can proceed as follows:

(a) Choose the dimension "a" equal to the bearing length (N) as previously deduced without any extension of the beam flange over the bearing length. Only a set back of 12 mms is required to assure the rotation of the beam.

$$\text{i.e.} \quad \left. \begin{aligned} a &= N + 12 \text{ mms} \\ e_s &= \frac{N}{2} + 12 \text{ mms} \end{aligned} \right\}$$

(b) The welds connecting the top plate of the bracket to the column flange is chosen a butt weld in order to ensure the required rigidity previously assumed in the analysis.

(c) The weld lines connecting the stiffener to the column flange are designed exactly as previously outlined in Section 4.10.

It is to be noticed that the actual analysis is based on qualitative assumptions. A more accurate analysis may apply a three - dimensional finite elements approach considering the different weld lines as springs of appropriate rigidities. Thus both buckling and yielding will be automatically included.

Refer to Example 4.11.

4.12. Stiffened Seat Bracket Whose Plane is Normal to the Beam
Web Plane:

If the beam web is in a plane normal to the plane of the vertical stiffener (case of crane track girder), the analyses previously presented in Section 4.10 and 4.11 are valid for all the steel structural components. The vertical stiffener only will behave differently. The analysis must satisfy the following considerations:

Fig. 4.18.a, illustrates the configuration of a rectangular stiffened bracket. Section (1-1) passing through the abrupt change of inclination is considered to be the critical section where the maximum bending and shear stresses are to be computed in order to determine the thickness(t).

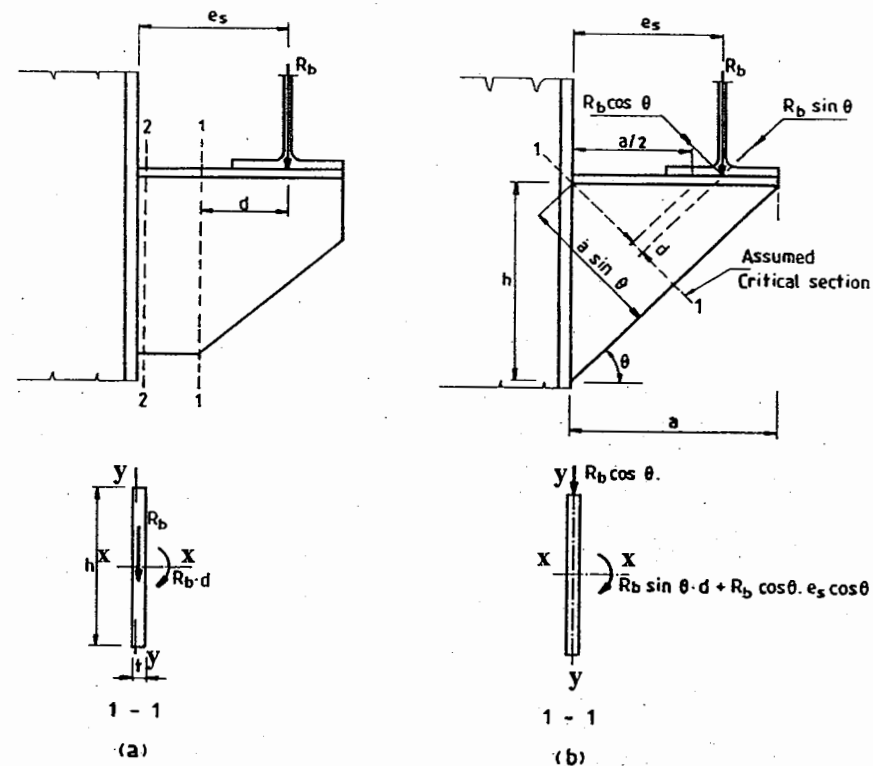


Fig. 4.18

For the rectangular bracket apply in the stress equation:

$$(f_{bc} = \frac{N}{A} + \frac{M_x \cdot y}{I_x} + \frac{M_y \cdot x}{I_y})$$

by the following:

$$N = \text{zero} ; \quad M_x = R_b \cdot d ; \quad M_y = \text{zero}$$



$$I_x = \frac{th^3}{12}; \quad y = \frac{h}{2}$$

Hence get:

$$f_{bc} = \frac{6 R_b \cdot d}{t \cdot h^2} \leq F_{bc} \quad (a)$$

On the other hand for the triangular stiffener shown in Fig. 4.18.b, the critical Section 1-1 is that section perpendicular to the free side. Having:

$$N = R_b \cdot \sin\theta$$

$$M_x = R_b \cdot \cos\theta \cdot e_s \cdot \cos\theta + R_b \cdot \sin\theta \cdot d$$

$$M_y = \text{zero}$$

$$\text{Where: } d = (e_s - \frac{a}{2}) \cdot \sin\theta$$

$$\text{Get: } f_{bc} = \frac{R_b \cdot \sin\theta}{a \cdot \sin\theta \cdot t} + \frac{6 R_b (e_s \cos^2\theta + d \cdot \sin\theta)}{t \cdot (a \cdot \sin\theta)^2} \leq F_{bc} \quad (b)$$

In Equation (a) and (b) one can utilize the allowable bending compressive stresses (F_{bc}) instead of f_{bc} to compute the required thickness (t).

It is to be noticed that for the rectangular bracket of Fig. 4.18.a, the following question arises: Why Section 2-2 was not considered as the critical section for the calculation of the bending stresses in Equation (a)? In fact this will lead to a higher moment ($M = R_b \cdot e_s$ instead $M = R_b \cdot d$) which consequently will induce higher stresses leading to an excessive thickness (t).



However for a more accurate analysis, one can consider Section 2-2 with the induced moment ($M = R_b \cdot e_s$) while the resisting cross section will be that of the Tee section comprising the top plate and the vertical stiffener.

Once again, these analyses as well as the assumptions concerning the pass of forces from one steel structural element to the other are approximate. For an accurate behaviour, a three dimensional finite elements approach is to be applied where the weld lines can be represented by springs of appropriate rigidities.

4.13. Stiffened Seat-Angle Connections:

Some designers prefer to utilize the seat-angle connection of Section 4.9 adding one, two or three triangular stiffeners as shown in Fig. 4.19.

Design considerations are the same as previously outlined in Sections 4.9, 4.11 and 4.12.

Compute the bearing length (N) using Equation (a) of Section 4.9, then add a set back of 12 mm for the calculation of the moment arm (e_s). For simplicity we can assume a seat angle of equal legs i.e. take $h = a = N + 12$ mm.

The critical Section 1-1, perpendicular to the free edge of the stiffener, will be subjected to the following straining actions:

$$N = R_b \sin\theta$$

$$M = R_b \cdot \cos\theta \cdot e_s \cdot \cos\theta + R_b \cdot \sin\theta \cdot \left[\left(\frac{N}{2} + 12 \text{ mm} \right) \cdot \sin\theta - \bar{y} \right]$$

The position of the neutral axis $\bar{y}$ is to be computed hence the inertia I_{xx} calculated. Applying in the stress equation by the allowable bending stress the thickness t can be computed.

$$\text{i.e. } f_{bc} = \frac{N}{A} + \frac{M.(h \cos \theta - \bar{y})}{I_{xx}} \leq F_{bc}$$

It is to be noticed that the resisting cross section is comprising in addition to the stiffeners the angle section intersecting Section (1-1)

(Refer to Fig. 4.19 Section 1-1).

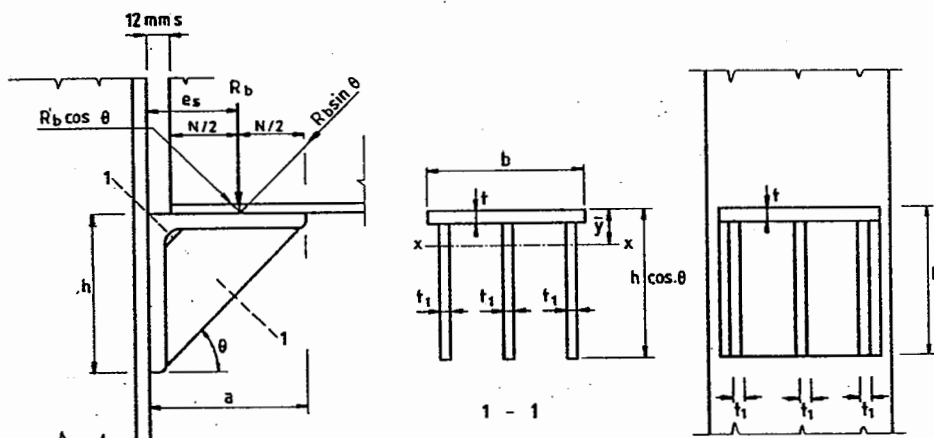


Fig. 4.19

Once again all these assumptions are not probably very accurate, but for lack of refined analysis one can follow the assumed previous behaviour.

Refer to Example 4.10.

4.14. Beam-to-Beam Flexible Connections:

The beam-to-beam connection is a typical connection of floor beam systems where the secondary beams are connected to the main beams. The connection is either a flexible connection transmitting only the vertical reaction (concentric shear) or a rigid connection ensuring the continuity of the secondary beams (i.e. transmitting a concentric shear and a flexural moment).

The behaviour of these connections is generally the same as the beam-to-column connections previously analyzed. Only the main features are discussed in the following paragraphs, while for the design of the different steel structural components refer to the previous sections.

4.14.1. Web Framing Angles:

4.14.1.1. Beams Of Different Heights:

The most common type of flexible beam-to-beam connection is the web framing angles connection. Naturally one can differentiate between: a framing angle using only the welding process to connect both webs of secondary and main beams by shop and field welds respectively, and a framing angle connected by bolts. A combination of welds and bolts is possible where the framing angles are shop welded to the secondary beam web while the outstanding leg is bolted to the main beam web.

Fig. 4.20, illustrates a secondary beam connected to the framing angles by bolts behaving in double shear. If the secondary beams are of the same height, the erection process is

somewhat difficult where say the left secondary beam is to be propped in some way until after the right secondary beam is placed and then the bolts are inserted. On the other hand if the two secondary beams are of different heights one can provide two additional bolts say to the left secondary beam of the greatest height in order to support this beam and then the right secondary beam is placed and the bolts inserted.

Concerning group of bolts (1) of Fig. 4.20 and 4.21 one can observe that these bolts will behave either in single shear when only one of the neighboring spans is subjected to superimposed loads while the behaviour of these bolts will be in double shear if the two neighboring spans are loaded.

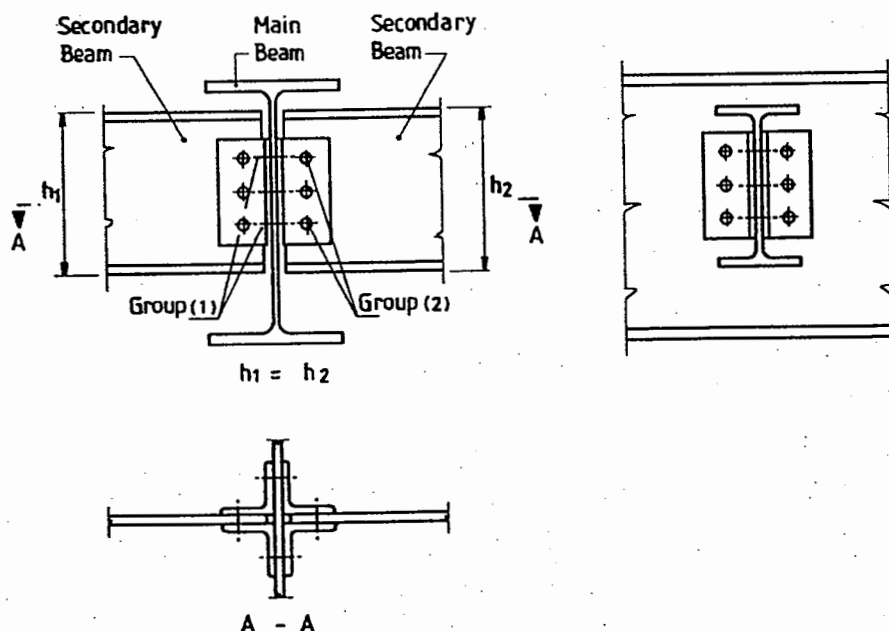


Fig. 4.20

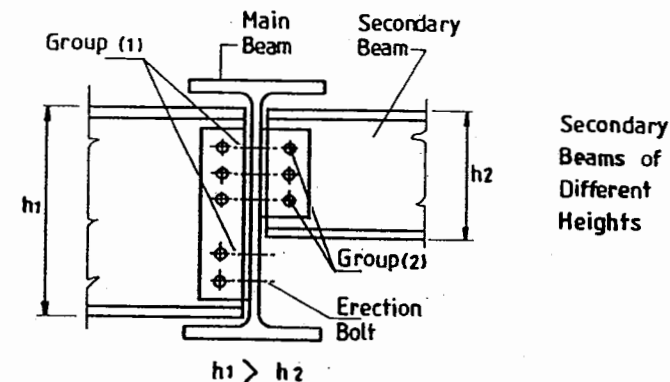


Fig. 4.21

For the determination of the thickness of the framing angles and the number of bolts for groups (1) and (2) refer, to Sections 4.3 and 4.4.

In the preceding paragraphs the flanges for both the main and secondary beams was not required to be coplanar where the secondary beams are allowed to be set a lower top flange level than the main beam. However, if the flanges are required to be coplanar one can utilize the configuration of Fig. 4.22.a, where the flanges of the secondary beams are notched.

The determination of the size of the framing angles, the number of group of bolts (1) and (2) is a straight forward matter. While the region of the secondary beam, where the top flange is coped (notched) with a small part of the web (Fig. 4.22.b), can show signs of failure. Section (1-1) passing through the web will be subjected to a concentric shear (R_{sb}) and a flexural moment $M = R_{sb} (L_o - e)$ thus inducing principal

stresses. This phenomenon is called the web tear-out shear. Check the situation by calculating the inertia of the cross section passing through (1-1) composed of the web and the lower flange of the secondary beam and then compute the compressive stress. The calculation of the induced shear stress on this section is also required. If the state of compressive stresses is critical, a stiffener is to be welded to the secondary beam web. Only a stiffener from one side is sufficient to respond to the over stressed situation. Concerning the size of the stiffener (shown dotted in Fig. 4.22.a,b) one can choose the same dimensions of a single outstanding secondary beam flange with an extended length (L_1) equal to the notched length (L_0).

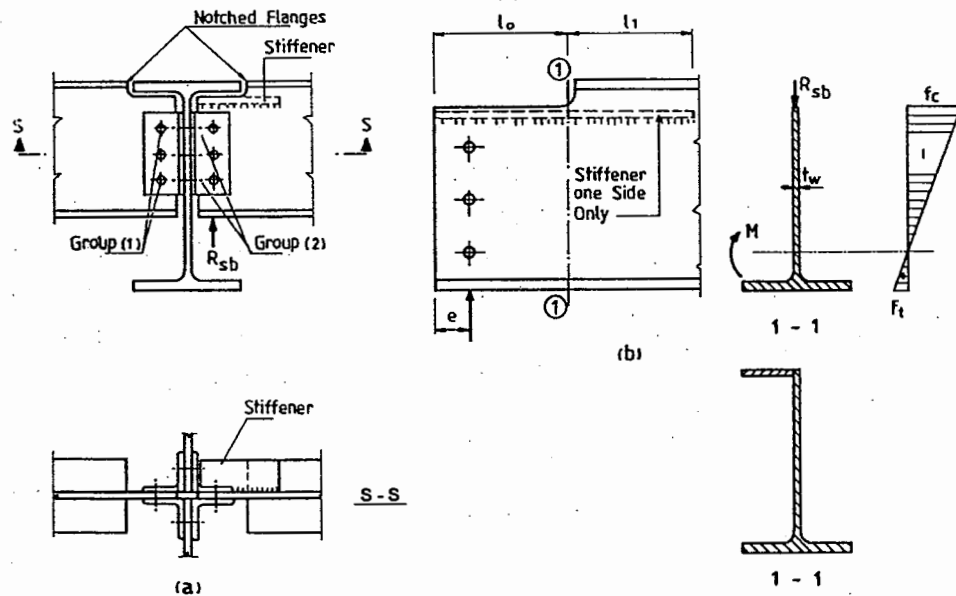


Fig. 4.22

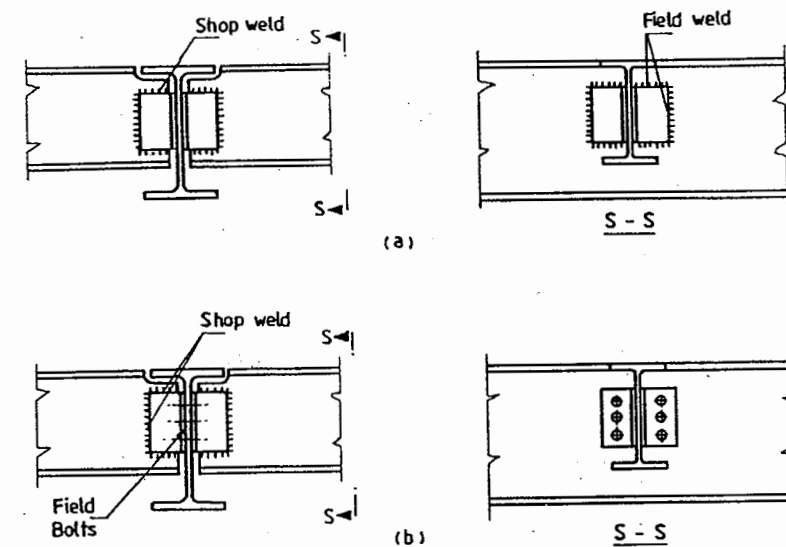


Fig. 4.23 Flexible Beam-to-Beam Welded Connections

As has been mentioned previously, one can use shop welding to connect the framing angle to the secondary beam web and then use field welding to connect the framing angle to the main beam web. This field welding is very popular in some countries, while in others (like in Egypt) the field welding is not recommended, the inspection being rather questionable. Fig. 4.23.a, illustrates a flexible beam-to-beam connection completely welded.

Field bolting being very rapid and requiring less skilled labor than welding, one can utilize the shop welding in conjunction with the field bolting as shown in Fig. 4.23.b.

Refer to Example 4.12.

4.14.1.2. Beams of Constant Heights:

If the cross section of the main and the secondary beams are of constant heights, we can choose one of the following flexible connections:

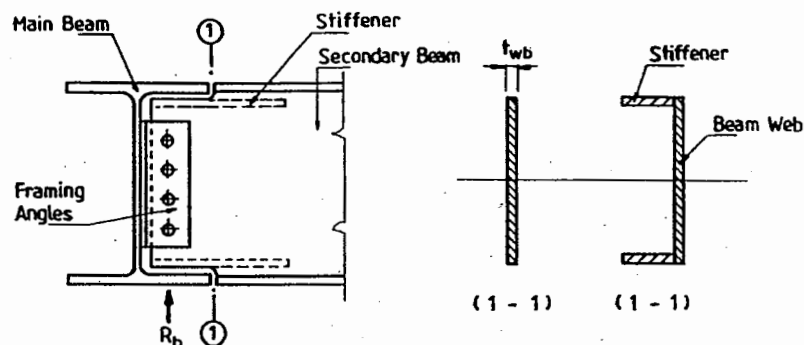


Fig. 4.24

(a) Fig. 4.24, illustrates a configuration similar to that given in Section 4.14.1.1 where the secondary beam is notched at the tension and compression flanges. The secondary beam web is to be checked against bending stresses along Section 1-1. If the web thickness is not sufficient an upper as well as a lower stiffener are to be provided in order to increase the resisting cross section.

(b) Fig. 4.25, shows an alternative configuration where notching the flange beam can increase the cost of fabrication, hence a cleat web plate is first welded to the secondary beam web and then welded to two framing angles. The advantage of this type of connections is that the web cleat plate thickness (t) can be chosen in accordance to a safe bending stress along Section (1-1). The induced moment along Section (1-1) is

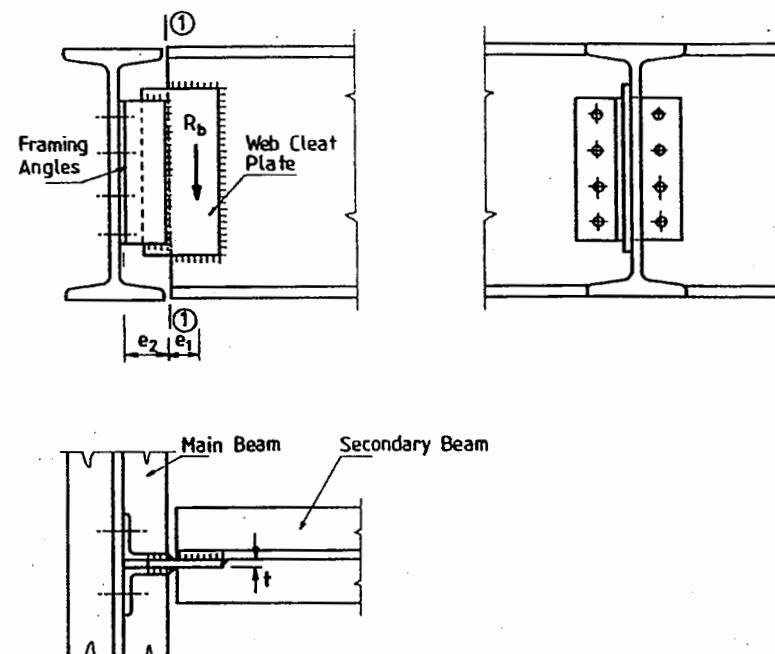


Fig. 4.25

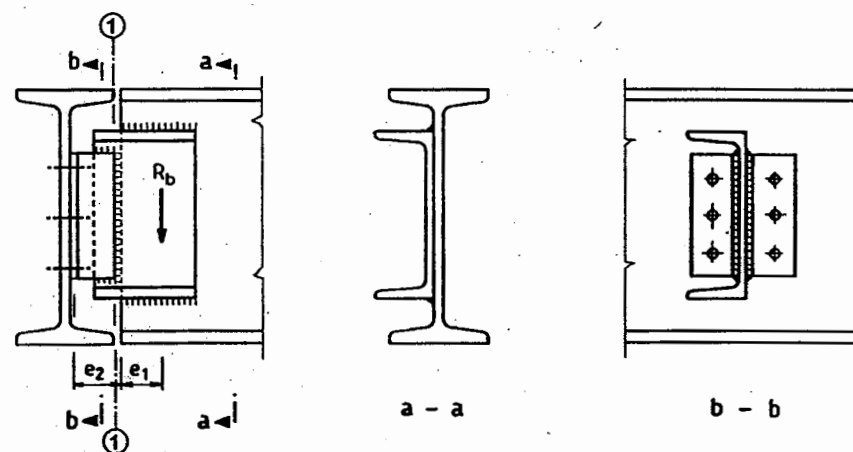


Fig. 4.26

$$M = R_b \cdot e_1$$

To be noticed that the group of bolts connecting the framing angles to the main beam web will be subjected to a vertical shear (R_b) as well as a flexural moment $M = R_b(e_1 + e_2)$.

(c) The third alternative is to replace the web cleat plate by a channel as shown in Fig. 4.26. The channel cross section may be chosen in accordance to the safe bending stress along Section (1-1) where $M = R_b \cdot e_1$.

4.14.2. Flexible Shear End Plate Beam-to-Beam Connections:

In order to avoid repetition, refer to Section 4.6. The same idea is utilized to provide a flexible beam-to-beam connection. Vertical fillet weld lines will transmit the secondary beam reactions to the beam shear end plates. These reactions will be transmitted to the main beam via the bolts behaving in double shear.

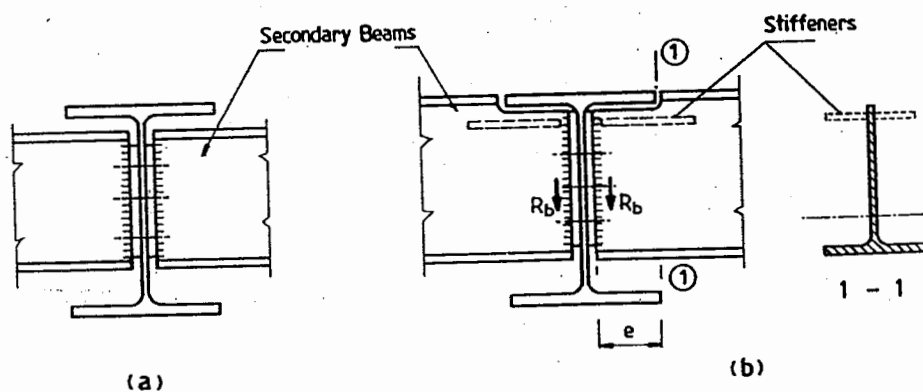


Fig. 4.27

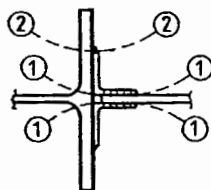
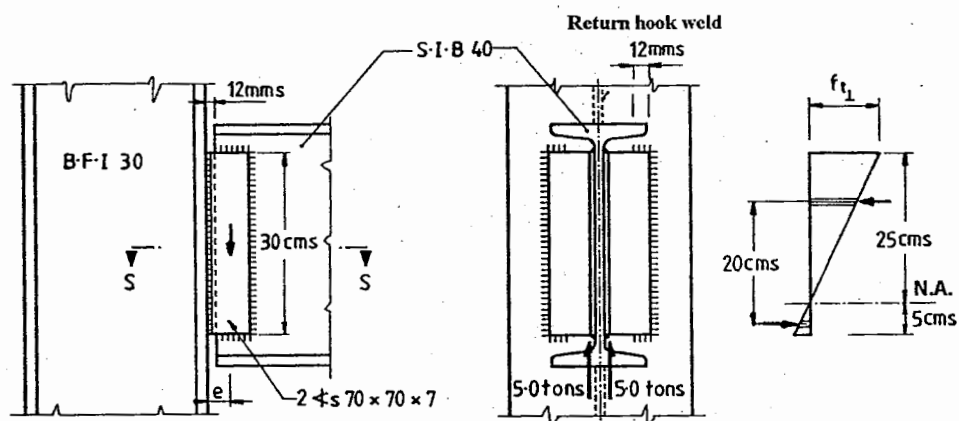
Fig. 4.27.a, illustrates a beam-to-beam connection where the secondary beam is set below the main beam top flange. While Fig. 4.27.b, shows a connection where a coplanar top flanges is required. For this last situation bending along Section 1-1 is to be checked where ($M = R_b \cdot e$).

Refer to Example 4.13.

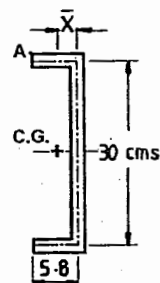
** **

**Example (4.1): Welded Double Web Framing Angles:**

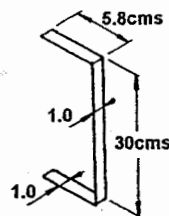
Design the flexible beam-to-column connection using a welded double web framing angles. The column and the beam are B.F.I No 30 and S.I.B. No 40 respectively. The beam reaction is 10.0 tons ($R_b = 10.0$ tons). Refer to section 4.3.1.

Solution

Sec. S-S



Sec. 1-1



Sec. 2-2

Framing angles to beam web**1. Welds Attaching the Framing Angles to the Beam Web:**

Assume the size of fillet weld $s = 1.0$ cms. to use Table 3.2.

$$\bar{X} = \frac{\bar{b}^2}{d + 2b} \quad (b = 5.8 \text{ cms; } d = 30 \text{ cms}); \quad \bar{X} = 0.81 \text{ cms.}$$



$$I_x = \frac{30^3}{12} + 2 \times 5.8 \times 15^2 = 4860 \text{ cm}^4$$

$$I_y = 30 \times 0.81^2 + 2 \times 5.8 \times (5.8/2 - 0.81)^2 = 70.3 \text{ cm}^4$$

$$I_p = I_x + I_y = 4930.3 \text{ cm}^4$$

$$e = 7 - 0.81 = 6.19 \text{ cms}; \quad M_t = 10 \times 0.81 = 8.1 \text{ t.cm}$$

$$q_{A_{\perp}} = \frac{10}{2 \times (30 + 2 \times 5.8) \times 1.0} = 0.12 \text{ t/cm}^2$$

$$q_{A(Mt)} = \frac{8.1 \times 30}{2 \times 4930.3} = 0.0246 \text{ t/cm}^2$$

$$q_{A(Mt)_{\perp}} = \frac{8.1 \times (5.8 - 0.81)}{2 \times 4930.3} = 0.04 \text{ t/cm}^2$$

$$q_t = \sqrt{(0.12 + 0.04)^2 + (0.0246)^2} = 0.161 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ O.K.}$$

Use $s = 0.161/0.72 = 0.22 \text{ cm}$ take $s = 0.4 \text{ cm (min)}$

No need to check section (2-2) it will be safe.

2. Framing Angles Thickness:

Neglect the effect of the end return hook weld (12 mms).

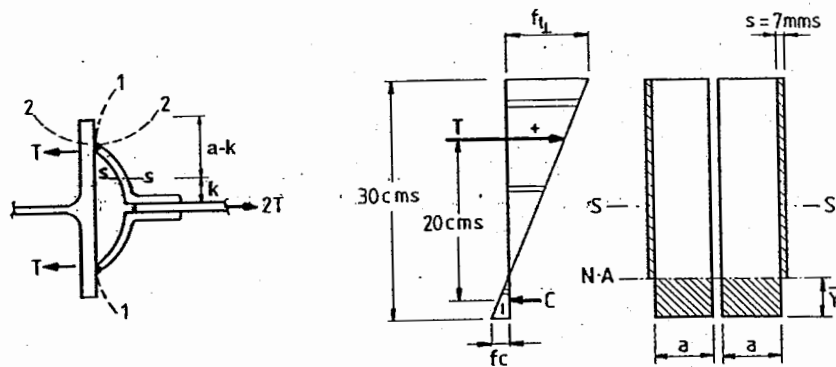
Assume the N.A to be at a distance $h/6 = 5.0$ cms.

Using Method (A) of Section 4.3.1:

$$M_{\text{external}} = 10 \times 6.19 = 61.9 \text{ t.cms.}$$

$$M_{\text{internal}} = \frac{1}{2} f_t \times 2s \times \frac{5}{6} h \times \frac{2}{3} h = 500 f_t \cdot s$$

Equate $M_{\text{ex}} = M_{\text{in}}$ and get: $f_t = 0.124/s \text{ t/cm}^2$.



Section (s-s):

$$T = 0.124/s \times s \times 1 \text{ cm} = 0.124 \text{ tons} ; (s = 0.4 \text{ cm}).$$

$$M = 0.124 \times (a-k) = 0.124 \times (7 - 2 \times 0.7) = 0.694 \text{ t.cm.}$$

$$f_b = \frac{M}{Z} = \frac{0.694 \times 6}{t^2 \times 1} \text{ put } F_b = 1.4 \text{ t/cm}^2 \text{ get } t:$$

$$t = 1.73 \text{ cm. (angle thickness).}$$

The same result can be obtained using Equation 4.3

$$t^2 = \left[10.8 \times R_b \times e \times (a-k) \right] / (h^2 \times F_b)$$

$$t^2 = 10.8 \times 10 \times 6.19 \times (7 - 2 \times 0.7) / 30^2 \times 1.4$$

$$t = 1.73 \text{ cm. (very excessive).}$$

Using Method (B) we get:

$$2 \times 7 \times \bar{y} \times \frac{1}{2} \bar{y} = 2s(30 - \bar{y}) \times \left(\frac{30 - \bar{y}}{2} \right) ; \bar{y} = 7.2 \text{ cms.}$$

$$I_{xx} = \frac{1}{3} \times \left[2 \times 7 \times 7.2^3 + 2 \times 0.7 \times (30 - 7.2)^3 \right] = 7273.1 \text{ cm}^4$$

$$f_{t\perp} = \frac{61.9 \times (30 - 7.2)}{10052.1} = 0.194 \text{ t/cm}^2$$

$$T = 0.194 \times s \times 1 \text{ cm} = 0.1358 \text{ tons.}$$



$$M' = 0.1358 \times (7 - 2 \times 0.7) = 0.76 \text{ t.cms.}$$

$$f_b = \frac{M'}{Z} = \frac{0.76 \times 6}{t^2 \times 1} \text{ put } f_b = 1.4 \text{ t/cm}^2$$

$$t = 1.8 \text{ cms. (still excessive)}$$

It may be allowed to reach the yield stress.

$$t = \sqrt{\frac{6 \times 0.76}{1 \times 2.4}} = 1.38 \text{ cms. ; use } 2 < s \text{ } 120 \times 120 \times 14.$$

3. Welds Attaching the Framing Angles to the Column Flange:

Choose $s = 0.7 \text{ cms.}$ (neglect hook weld)

Plane 1-1:

$$f_{t\perp} = 0.194 \text{ t/cm}^2.$$

$$q_{\downarrow\downarrow} = \frac{10}{2 \times 30 \times 0.7} = 0.238 \text{ t/cm}^2.$$

$$\vec{q}_{Mt\perp} = \frac{5 \times 7 \times 6}{0.7 \times 30^2} = 0.33 \text{ t/cm}^2.$$

$$f_e = \sqrt{0.194^2 + 3(0.238^2 + 0.33^2)} = 0.73 \text{ t/cm}^2$$

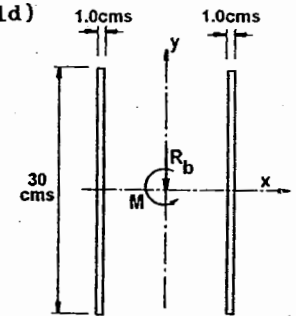
(o.k.).

Plane 2-2:

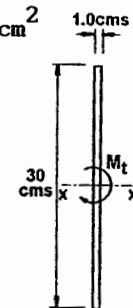
$$q_{\downarrow\downarrow} = 0.238 \text{ t/cm}^2 ; f_{\perp} = 0.33 \text{ t/cm}^2.$$

$$\vec{q}_{Mt\perp} = \frac{10 \times (7 - 0.81) \times 6}{2 \times 0.7 \times 30^2} = 0.295 \text{ t/cm}^2.$$

$$f_e = \sqrt{0.33^2 + 3(0.238^2 + 0.295^2)} = 0.73 \text{ t/cm}^2 \text{ (o.k.).}$$



Sec.1-1



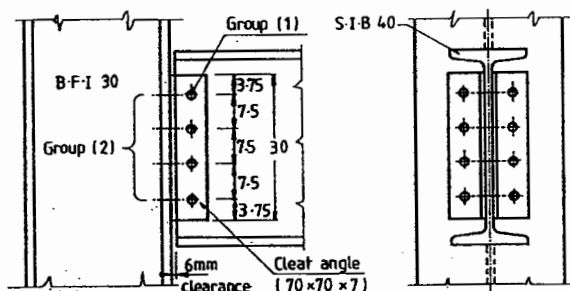
Sec.2-2

Framing angles to column flange

Example (4.2): Bolted Double Web Framing Angles Connection:

Design a flexible bolted double web framing angles connection. The beam is a S.I.B. # No (40) carrying a reaction of 10 tons. The column is a B.F.I. # No (30). Use Non pretensioned bolts (Grade 4.6). Refer to section 4.3.2. ($t_{web} = 1.4 \text{ cms}$)

Solution



- Use connecting angles 70 x 70 x 7 with height equals to 30 cms.

- From the construction condition

$$d_{max} = \frac{7 - 0.70}{3} = 2.10 \text{ cm.}$$

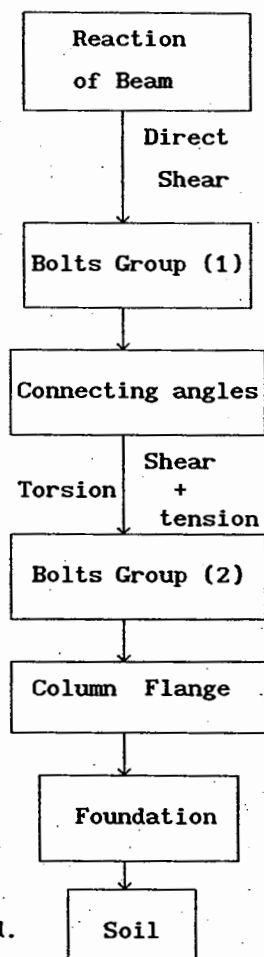
1. Group (1) of Bolts:

$$Q_{bolt} = \frac{10}{4} = 2.50$$

$$R_{d.sh.} = 2 \times 0.25 \times 4 \times \frac{\pi d^2}{4} = 1.57 d^2.$$

$$R_b = 0.80 \times 3.6 \times 1.40 \times d = 4.032 d.$$

$$Q_{bolt} = 2.50 = 1.57 d^2$$



Flow Chart of Reaction Path to Soil

$$\therefore d = 1.27 \text{ cm} \quad \dots \dots \dots (a)$$

$$Q_{bolt} = 2.50 = 4.032 d$$

$$\therefore d = 0.62 \text{ cm} \quad \dots \dots \dots (b)$$

From (a) & (b) use 4 ordinary bolts 12 mm diameter.

$$A = 1.13 \text{ cm}^2. ; A_s = 0.84 \text{ cm}^2.$$

$$S = 7.5 \text{ cms} ; S = 7.5/1.2 = 6.25 < 12 t = (12 \times 0.7 = 8.4 \text{ cms})$$

$$e_1 = 3.75 \text{ cms} ; e_1 = 3.75/1.2 = 3.125d > 3d ; F_b = 1.2 F_U$$

2. Group of Bolts (2):

Assume the number of bolts per column 4 bolts.

$$R_{sh} = 0.25 \times 4 \times 1.13 = 1.13 \text{ tons}$$

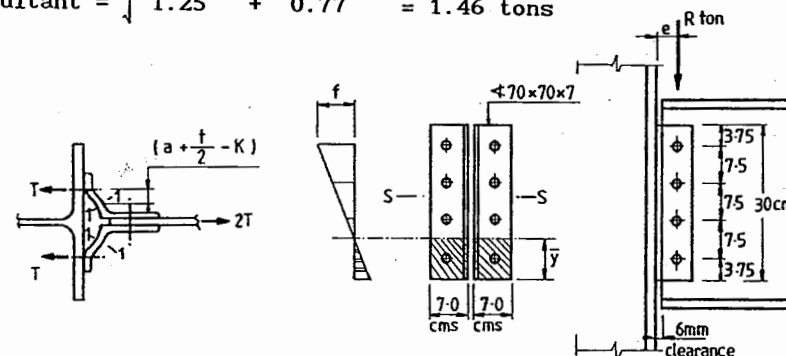
$$R_{sh.a} = 5.0 / 4 = 1.25 \text{ tons} > 1.13 \text{ tons} \quad \text{unsafe}$$

$$\text{Use M16} - R_{sh} = 0.25 \times 4 \times 2.0 = 2.0 \text{ tons} > 1.25 \text{ tons} \quad (O.K.)$$

$$R_{Mt.a} = \frac{5 \left[\frac{(7 - 0.7)}{2} + 0.7 \right] \times r_{max}}{\sum r_i^2} = 0.77 \text{ tons.}$$

$$\sum r_i^2 = 2 \times (3.75^2 + 11.25^2) = 281.2 \text{ cm}^2. ; r_{max} = 11.25 \text{ cms.}$$

$$\text{Resultant} = \sqrt{1.25^2 + 0.77^2} = 1.46 \text{ tons}$$





$$e = \frac{7 + 0.7}{2} = 3.85 \text{ cms.}$$

$$M_{\text{ext}} = 10 \times 3.85 = 38.5 \text{ t.cms.}$$

Take the first moment of area at the lower edge:

$$\therefore 14 \bar{y} \times \frac{\bar{y}}{2} = 2 A_s \left[(11.25 - \bar{y}) + (18.75 - \bar{y}) + (26.25 - \bar{y}) \right]$$

$$\therefore 7 \bar{y}^2 + 6.93 \bar{y} - 129.94 = 0.0$$

$$\therefore \bar{y} = 3.842 \text{ cm.}$$

$$\therefore I_x = \frac{3.842^3}{3} \times 14 + 2 \times 1.155 \left[7.408^2 + 14.908^2 + 22.408^2 \right]$$

$$I_x = 2064.711 \text{ cm}^4.$$

$$f_{tb} = \frac{38.50 \times 22.408}{2064.711} = 0.417 \text{ t/cm}^2$$

$$R_{t.a} = 0.417 \times 1.57 = 0.654 \text{ tons. } (A_s = 1.57 \text{ cm}^2)$$

N.B. Whenever bolts are subjected to combined shear and tension, they can be designed as follows:

Bolts should satisfy the following interaction equation:

$$\left[\frac{R_{\text{sh.a}}}{R_{\text{sh}}} \right]^2 + \left[\frac{R_{t.a}}{R_t} \right]^2 \leq 1.0$$

Where:

$$R_{\text{sh}} = 2.0 \text{ tons}$$

$$R_{\text{sh.a}} = 1.46 \text{ tons}$$



$$R_t = (0.33 \times 4) \times 1.57 = 2.072 \text{ tons}$$

$$R_{t.a} = 0.654 \text{ tons}$$

Then:

$$\left[\frac{1.46}{2.0} \right]^2 + \left[\frac{0.654}{2.072} \right]^2 = 0.795 < 1.0 \quad \therefore \text{safe}$$

3. The Framing Angles Thickness:

Referring to Section 4.3.2.1, regarding sec. (1-1) get the following:

$$T = f_{tb} \cdot A_s = 0.417 \times 1.57 = 0.655 \text{ tons.}$$

$$M' = T \cdot \left[\frac{a + t}{2} - K \right] = 0.655 \left[\frac{7 + 0.7}{2} - 2 \times 0.7 \right] \\ = 1.6 \text{ t.cm}$$

$$f_b = \frac{1.6 \times 6}{7.5 \times t^2}; \text{ put } f_b = F_b = 1.4 \text{ t/cm}^2$$

Hence:

$$t = 0.957 \text{ cms. (to get the same result use Equ. 4.4)}$$

If it is allowed to reach the yield stress, hence

$$t^2 = \frac{1.6 \times 6}{7.5 \times 2.4} = 0.73 \text{ cms. ; } t = 0.86 \text{ cms.}$$

Choose $2 < s \leq 70 \times 70 \times 10$ (yield not allowed).

or

$2 < s \leq 70 \times 70 \times 9$ (yield allowed).

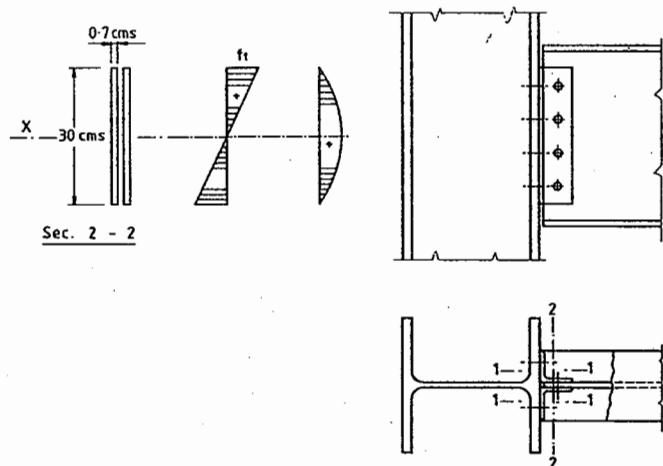
The other Section (2-2) can be checked according to the eccentricity of the beam reaction as follows:

$$R_b = 10 \text{ tons}$$

$$M_{2-2} = \left[\left(\frac{7 + 1.0}{2} \right) - 2 \times 1.0 \right] \times 10 = 20.0 \text{ t.cm.}$$

$$f_{bc} = \frac{20.0 \times 6}{2 \times 1.0 \times 30^2} = 0.067 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (o.k.)}$$

$$q_{\downarrow} = \frac{3 \times 10}{2 \times 2 \times 30 \times 1.0} = 0.25 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2 \text{ (o.k.)}$$



Example (4.3): Bolted Single Web Framing Angle Connection:

Design a bolted single web cleat connection. The beam is a S.I.B. 40 carrying a reaction of 10 tons and the column a B.F.I. # 30. Refer to section 4.4.

Solution

- Use connecting angles 80 x 80 x 8 with height equals to 32 cms.
- From the construction condition

$$d_{\max} = \frac{8 - 0.80}{3} = 2.40 \text{ cm.}$$

1. Group (1) of Bolts (Grade 4.6) :-

$$R_{sh.a} = \frac{10}{4} = 2.50 \text{ tons.}$$

$$R_{s.sh.} = 0.25 \times 4 \times \frac{\pi d^2}{4} = 0.785 d^2.$$

$$R_b = 0.6 \times 3.6 \times 0.80 d = 2.16 d. \quad (e_1 = 1.5 d)$$

$$Q_{bolt} = 2.50 = 0.785 d^2$$

$$\therefore d = 1.78 \text{ cm} \quad \dots \dots \dots (a)$$

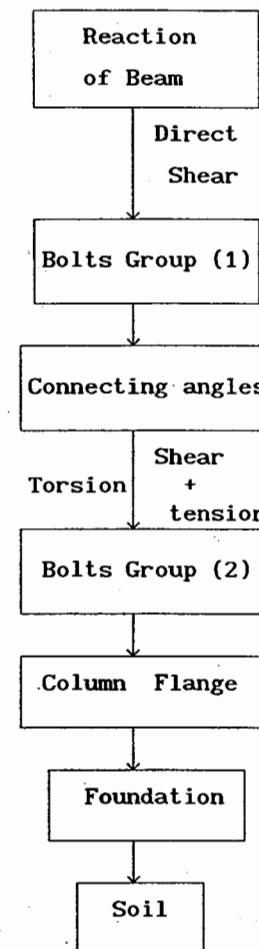
$$Q_{bolt} = 2.50 = 1.728 d$$

$$\therefore d = 1.44 \text{ cm} \quad \dots \dots \dots (b)$$

From (a) & (b) use 4 Non pretensioned bolts (Grade 4.6) 20 mm diameter.

2. Group of Bolts (2):

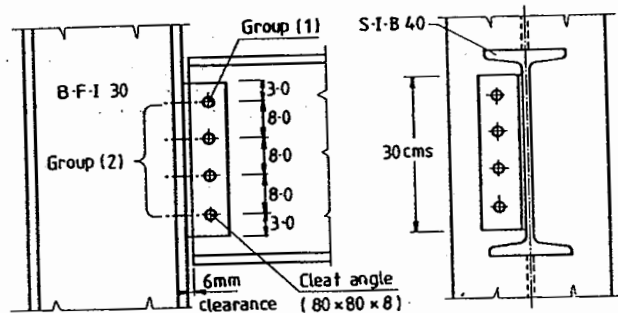
Assume to use 4 bolts d (20 mm)



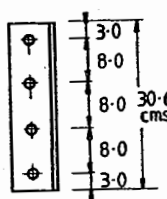
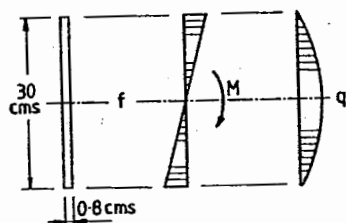
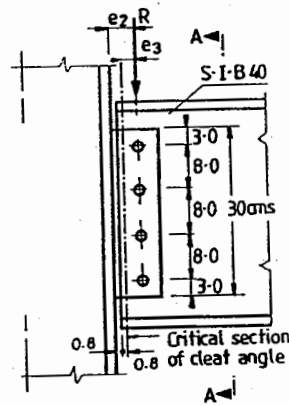
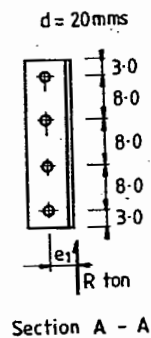
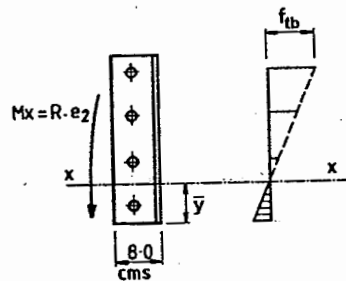
Flow Chart of Reaction Path to Soil



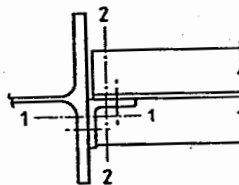
$$R_{sh.a} = \frac{10}{4} = 2.50 \text{ tons. } e_1 = \frac{8 + 0.80}{2} = 4.40 \text{ cms} = e_2$$



Section (1-1): $d = 20 \text{ mms}$



Pitch ($d=22$)



Cleat angle
critical section



$$M_t = R \cdot e_1 = 10 \times 4.40 = 44 \text{ t.cm.}$$

$$R_{Mt} = \frac{44.0 \times 12.0}{2 \left[\frac{4^2}{12^2} + 1 \right]} = 1.65 \text{ ton.}$$

$$\text{Resultant shearing force} = \sqrt{2.5^2 + 1.65^2} = 2.995 \text{ tons.}$$

$$R_{sh} = 0.25 \times 4 \times 3.14 \times (2)^2 / 4 = 3.14 \text{ tons.}$$

$$R_b = 0.6 \times 3.6 \times 2 \times 0.8 = 3.456 \text{ tons. } (e_1 = 1.5 d)$$

$$\text{Flexural moment "M}_x\text{"} = R \times e_2 = 44 \text{ t.cm.}$$

$$A_s = 2.45 \text{ cm}^2$$

Take first moment of area to compute $\bar{y}$:

$$4 \bar{y}^2 = 2.45 \left[(11 - \bar{y}) + (19 - \bar{y}) + (27 - \bar{y}) \right]$$

$$4 \bar{y}^2 + 7.08 \bar{y} - 134.52 = 0$$

$$\bar{y} = 4.981 \text{ cm.} \approx h/6 \approx 30/6 \approx 5.0 \text{ cms (can be utilized).}$$

$$I_x = \frac{4.981^3 \times 8}{3} + 2.36 \left[\frac{6.019^2}{12} + \frac{14.019^2}{12} + \frac{22.019^2}{12} \right]$$

$$I_x = 2023.07 \text{ cm}^4.$$

$$f_{tb} = \frac{44 \times 22.019}{2023.07} = 0.479 \text{ t/cm}^2$$

$$R_{ta} = 0.479 \times A_s = 0.479 \times 2.45 = 1.17 \text{ tons}$$

$$R_t = 0.33 \times 4 \times A_s = 3.23 \text{ tons}$$

$$\left[\frac{2.995}{3.14} \right]^2 + \left[\frac{1.17}{3.23} \right]^2 = 1.02 \approx 1.0 \quad (\text{O.K.})$$

3. Check of Cleat Angle:

Applying in equation (f) of Section (4.4) :-

$$f_b = [18 R_b \cdot e \cdot (h - p) \cdot (a + t - 2k)] / (h^3 \cdot t^2)$$

$$R_b = 10 \text{ tons}$$

$$e = 4.4 \text{ cms}$$

$$h = 30 \text{ cms}$$

$$p = 8.0 \text{ cms}$$

$$a = 8.0 \text{ cms}$$

$$t = 0.8 \text{ cms}$$

$$k = 2 \times 0.8 = 1.6 \text{ cms}$$

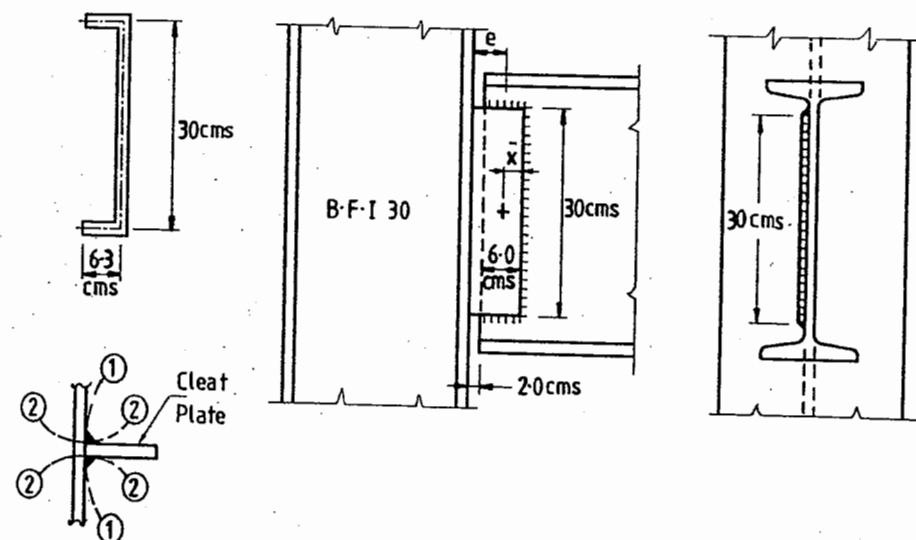
$$f_c = F_y = 2.4 \text{ t/cm}^2$$

get $t = 1.22 \text{ cms}$ choose angle $80 \times 80 \times 14$.

Example (4.4): Welded Cleat Plate Web-Beam Shear Connection (Section 4.5)

Design the connection previously given in Example 4.3 using a welded cleat plate.

Solution



1. Welds Attaching the Cleat Plate to the Beam Web:

Assume the following:

h = height of vertical weld = 30.0 cms.

b = breadth of horizontal weld = 6.0 cms.

s = size of weld lines = 10 mms.

Use Table 3.2 to compute the properties of the weld lines

$$\bar{x} = \frac{\bar{b}^2}{d + 2b} = \frac{6.0^2}{30 + 2 \times 6.0} = 0.86 \text{ cm.}$$

$$e = 6.0 + 2.0 - 0.86 \approx 7.14 \text{ cms.}$$

$$q_{\downarrow} = \frac{10}{(30 + 2 \times 6.0) \times 1.0} = 0.238 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ O.K.}$$

The torsional moment $M_t = 10 \times 0.86 = 8.6 \text{ t.cm}$ is small and can be neglected.

$$s = \frac{0.238 \times 1}{0.7} = 0.34 \text{ cms. , take } s = 0.4 \text{ cms (min).}$$

2. Welds Attaching the Cleat Plate to the Column Flange:

Plane 1-1: ($s = 1.0 \text{ cms}$)

$$q_{\downarrow//} = \frac{10}{2 \times 30 \times s} = \frac{0.166}{s} \text{ t/cm}^2$$

$$f_{\perp} = \frac{10 \times 7.14 \times 6}{2 \times s \times 30^2} = \frac{0.238}{s} \text{ t/cm}^2$$

$$0.72 \times 1.1 = \sqrt{\left(\frac{0.238}{s}\right)^2 + 3 \times \left(\frac{0.166}{s}\right)^2}$$

$$s = 0.42 \text{ cm. take } s = 5 \text{ mms.}$$

Plane 2-2:

$$q_{\downarrow//} = \frac{10}{2 \times 30 \times 0.5} = 0.33 \text{ t/cm}^2$$

$$q_{Mt_{\perp}} = \frac{10 \times 7.14 \times 6}{2 \times 0.5 \times 30^2} = 0.476 \text{ t/cm}^2$$

$$q_t = \sqrt{(0.33)^2 + (0.476)^2} = 0.578 \text{ t/cm}^2 < 0.72 \text{ (o.k.)}$$

3. Cleat Plate Thickness:

$$f_{bc} = \frac{10 \times 7.14 \times 6}{t \times 30^2} \text{ put } f_b = F_b = 0.72 \times 2.4 \text{ t/cm}^2 \text{ (solid rectangular section)}$$

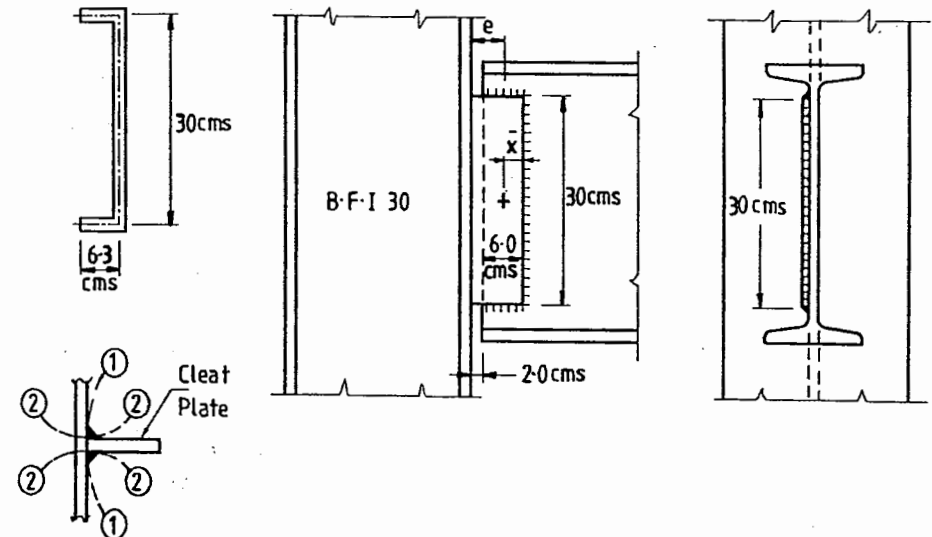
$$t = 0.52 \text{ cm. take } t = s = 0.6 \text{ cms.}$$

$$q = \frac{3}{2} \times \frac{10}{0.6 \times 30} = 0.83 \text{ t/cm}^2 < 0.84 \text{ O.K.}$$

Example (4.5):

Repeat Example 4.4 using Non-pretensioned (Grade 4.6) bolts to attach the cleat plate to the beam web.

Solution



The maximum length of cleat plate is

$$40 - 4 \times t_{fb} = 40 - 4 \times 2.16 = 31.36 \text{ cm.}$$

∴ Take length = 30 cm.

(1) Thickness of the Cleat Plate:

The cleat plate must withstand the total reaction as a shearing force in addition to the moment induced by the reaction eccentricity of 6 cm.

$$Q = R_b = 10.0 \text{ ton.}$$

$$M = R_b \cdot e = 10 \times 6 = 60 \text{ t.cm.}$$

$$f_b = \frac{6M}{th^2} = \frac{6 \times 60}{t \times 30^2} = \frac{0.4}{t} \text{ t/cm}^2$$

Put: $f_b = F_b = 1.4 \text{ t/cm}^2$, get:

$$t \geq 0.285 \text{ cm.} \quad (a)$$

$$q = \frac{3}{2} \frac{Q}{th} = \frac{3}{2} \times \frac{10}{t \times 30} = 0.84 \text{ t/cm}^2$$

$$t \geq 0.596 \text{ cm.} \quad (b)$$

From (a) & (b) take $t = 0.60 \text{ cm}$

(2) Bolts Attaching Beam Web to Cleat Plate:

Use Non pretension bolts M20 (Grade 4.6) :-

$$R_{s.sh} = 0.25 \times 4 \times \pi \frac{(2)^2}{4} = 3.14 \text{ tons}$$

$$R_b = 0.80 \times 3.6 \times 2.0 \times 0.60 = 3.45 \text{ tons (Table 1.3 - } e_1 = 2d)$$

$$R_{least} = 2.54 \text{ tons}$$

$$\text{number of bolts} = \frac{10}{3.14} = 3.18 \text{ bolts.}$$

Use 4 bolts

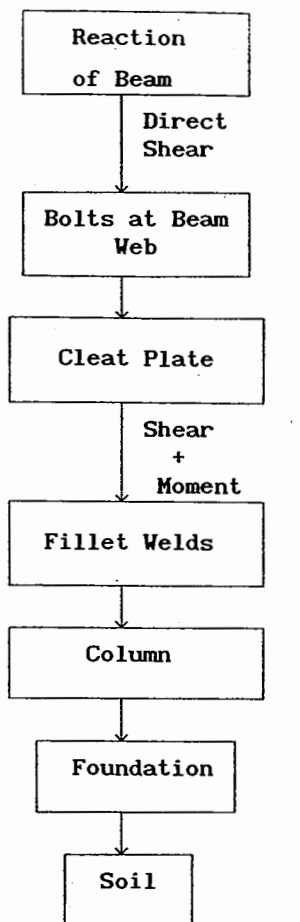
$$\text{pitch} = \frac{30}{4} = 7.50 \text{ cm} = 4d \quad (e_1 = p/2 = 3.75 \text{ cms})$$

$$e_1 = 3.75/2 = 1.875d \quad ; \quad F_b = 0.6 F_U$$

$$R_b = 0.6 \times 3.6 \times 2 \times 0.6 = 2.59 \text{ tons} \quad ; \quad n = 10/2.59 = 4 \text{ bolts}$$

3. Welds Attaching Cleat Plate to Column Flange:

$$Q = 10 \text{ tons} \quad ; \quad M = 10 \times 6 = 60 \text{ t.cms.}$$



Flow Chart of Reaction Transmission to Soil

Assume size of weld $s = 1 \text{ cms}$ and then compute the appropriate weld size.

Section (1-1): At point (A)

$$q = \frac{10}{2 \times 30 \times 1} = 0.17 \text{ t/cm}^2$$

$$q_{[Mt]_{\perp}} = \frac{6 \times 10 \times 6}{2 \times 1 \times (30)^2} = 0.20 \text{ t/cm}^2$$

$$q_R = \sqrt{(0.17)^2 + (0.2)^2} = 0.26 \text{ t/cm}^2$$

$$s_{(required)} = \frac{0.26}{0.72} = 0.36 \rightarrow s = 4 \text{ mm} \quad (a)$$

Section (2-2): At point (A)

$$f_{t_{\perp}} = \frac{M}{Z} = \frac{6M}{2sh^2} = \frac{6 \times 10 \times 6}{2 \times 1.0 \times 30^2} = 0.20 \text{ t/cm}^2$$

$$q_{[Q_{//}]_{\downarrow}} = \frac{Q}{2sh} = \frac{10}{2 \times 1.0 \times 30} = 0.17 \text{ t/cm}^2$$

Equivalent Stresses:

$$f_{eq} = \sqrt{f_{t_{\perp}}^2 + 3q_{//}^2} \leq 1.10 \times 0.72 \text{ t/cm}^2$$

$$f_{eq} = \sqrt{(0.2)^2 + 3(0.17)^2} = 0.36 \text{ t/cm}^2$$

$$s_{(req.)} = \frac{1.0 \times 0.36}{1.1 \times 0.72} = 0.45 \text{ cm.}$$

$$s = 5 \text{ mms} \quad (b)$$

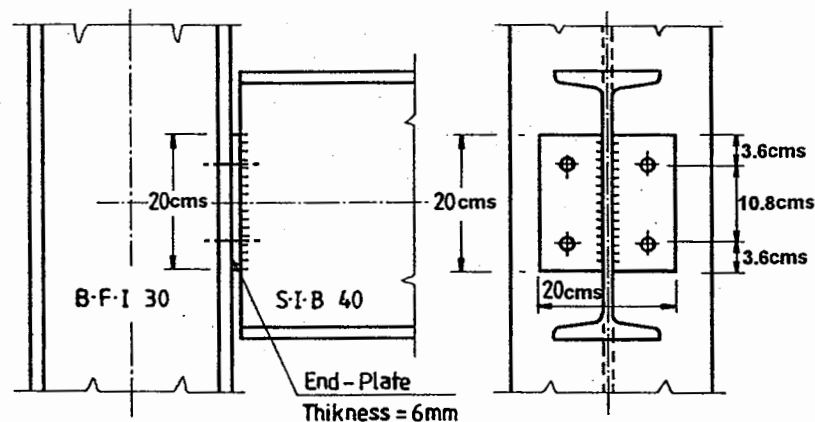
From (a) & (b) $\rightarrow$ use weld size of 5 mm at both sides of cleat plate.



Example (4.6): Welded Flexible End-Plate Connections Refer to Section 4.2.4

Redesign Example 4.3 using an end-plate welded to the beam web and bolted to the column flange by ordinary bolts.

Solution



(1) Bolts of End-Plate to Column Flange:

* Use end-plate with thickness 6 mm.

* IF we use four bolts

$$\text{Force per bolt } R = \frac{10}{4} = 2.50 \text{ ton.}$$

Use Non pretensioned bolts of Grade (4.6) :-

$$\text{Single shear resistance } R_{s.sh} = 0.25 \times 4 \times \frac{\pi d^2}{4} = 0.785 d^2.$$

$$\text{Bearing resistance } R_{bearing} = 0.8 \times 3.6 \times d \times 0.60 = 1.728 d.$$

end distance = 2d



$$0.785 d^2 = 2.50 \rightarrow d = 1.78 \text{ cm.}$$

$$1.728 d = 2.50 \rightarrow d = 1.44 \text{ cm.}$$

$$d = 18 \text{ mms.}$$

Use 4 Non-pretensioned bolts Grade (4.6) M20

(2) Fillet Welds Attaching Beam-Web to End-Plate:

$$\begin{aligned} \text{Height of end-plate} &= 2 \times 2d + 6d = 10d \\ &= 20 \text{ cms.} \end{aligned}$$

Weld Size:

$$s = \frac{10}{2 \times 20.0 \times 0.72} = 0.347 \text{ cm.}$$

$$\text{Take } s = 4 \text{ mms.}$$

(3) Check of Shear Stresses on End Plate:

$$q = \frac{3 \times 10}{2 \times 2 [20 - 2(2.0 + 0.2)] \times 0.6} = 0.8 \text{ t/cm}^2 < 0.35 \times F_y$$

$$< 0.35 \times 2.4$$

(O.K.)

Example (4.7): Unstiffened Seat-Beam Connection:

Refer to Section 4.9

Design an unstiffened seated beam connection of a floor beam having a span 6.0 ms. The spacing between beams is 4.0 ms.

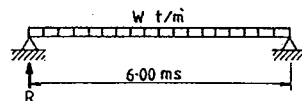
Assume: own weight of beam = 100 kg/m'.

Floor is R.C. slab = 250 kg/m².

Floor covering = 150 kg/m².

Live loads = 300 kg/m².

Solution



$$W = 0.10 + (0.25 + 0.15 + 0.30) \times 4$$

$$\therefore W = 2.90 \text{ t/m'}$$

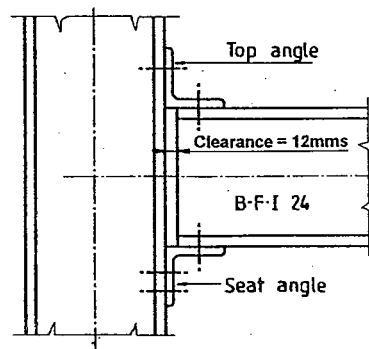
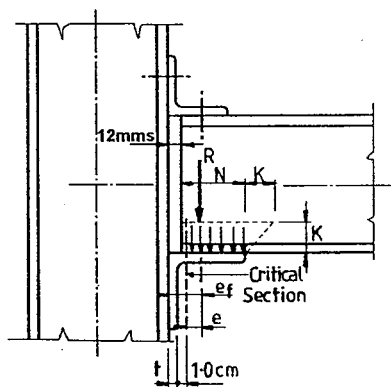
$$\therefore M_{\max} = \frac{W L^2}{8} = \frac{2.90 \times 6^2}{8} = 13.05 \text{ mt.}$$

$$\therefore Z_{x \text{ req.}} = \frac{13.05 \times 100}{1.40} = 932 \text{ cm}^3 \rightarrow \text{choose B.F.I. No(24).}$$

$$\text{Reaction of the floor beam} = \frac{2.90 \times 6}{2} = 8.70 \text{ ton.}$$

(1) Bearing Length of Seat Angle:

$$N + K = \frac{R}{F_{\text{crp}} \times t_w} ; K = 2 t_{\text{bf}} = 2 \times 1.80 = 3.6 \text{ cms.}$$



$$N = \frac{8.70}{1.80 \times 1.10} - 3.60 = 0.79 \text{ cm.}$$

$$N \geq K ; \text{ Take } N = K = 3.60 \text{ cms.}$$

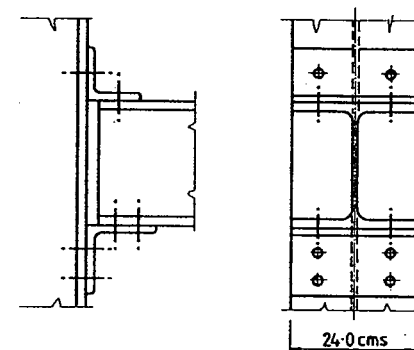
Required seat angle width > 3.60 cms.

Choose a seat angle (100 x 100 x 12).

$$\text{From Fig. : } e_f = \text{Clearance} + \frac{N}{2} = 1.2 + \frac{3.6}{2} = 3.0 \text{ cms.}$$

$$e = e_f - t - 1.0 = 3.0 - 1.2 - 1.0 = 0.80 \text{ cm.}$$

Use angle width = 24 cms = breadth of beam flange.



(2) Design of Seat Angle:

$$8.70 \times 0.80 = 6.96 \text{ t.cm}$$

$$t_{\text{req}} = \sqrt{\frac{6 \times 6.96}{24 \times 1.40}} = 1.12 \text{ cm} < 1.20 \text{ cm} \quad \text{O.K.}$$

While conservative the allowable bending stresses $F_b = 0.58 F_y$, one can say the critical section of the steel angle is rectangular and hence $F_b = 0.72 F_y$. The utilized assumptions being somewhat approximate we prefer to be conservative.

Example (4.7): Unstiffened Seat-Beam Connection:

Refer to Section 4.9

Design an unstiffened seated beam connection of a floor beam having a span 6.0 ms. The spacing between beams is 4.0 ms.

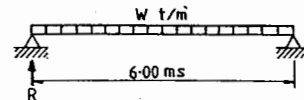
Assume: own weight of beam = 100 kg/m'.

Floor is R.C. slab = 250 kg/m².

Floor covering = 150 kg/m².

Live loads = 300 kg/m².

Solution



$$W = 0.10 + (0.25 + 0.15 + 0.30) \times 4$$

$$\therefore W = 2.90 \text{ t/m'}$$

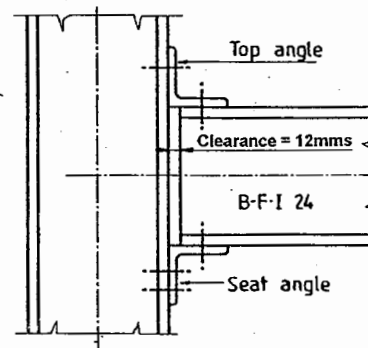
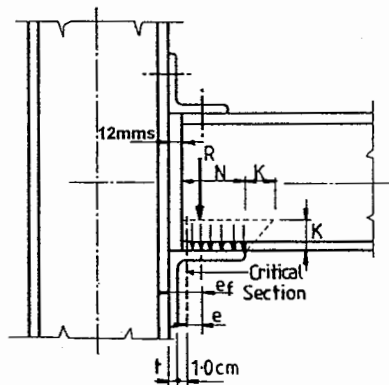
$$\therefore M_{\max} = \frac{W L^2}{8} = \frac{2.90 \times 6^2}{8} = 13.05 \text{ mt.}$$

$$\therefore Z_{x \text{ req.}} = \frac{13.05 \times 100}{1.40} = 932 \text{ cm}^3 \rightarrow \text{choose B.F.I. No(24).}$$

$$\text{Reaction of the floor beam} = \frac{2.90 \times 6}{2} = 8.70 \text{ ton.}$$

(1) Bearing Length of Seat Angle:

$$N + K = \frac{R}{F_{crp} \times tw} ; K = 2 t_{bf} = 2 \times 1.80 = 3.6 \text{ cms.}$$



$$N = \frac{8.70}{1.80 \times 1.10} - 3.60 = 0.79 \text{ cm.}$$

$$N \geq K ; \text{ Take } N = K = 3.60 \text{ cms.}$$

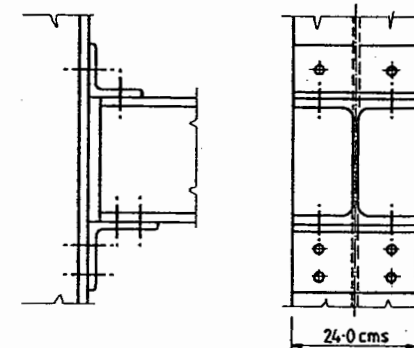
Required seat angle width > 3.60 cms.

Choose a seat angle (100 x 100 x 12).

$$\text{From Fig. : } e_f = \text{Clearance} + \frac{N}{2} = 1.2 + \frac{3.6}{2} = 3.0 \text{ cms.}$$

$$e = e_f - t - 1.0 = 3.0 - 1.2 - 1.0 = 0.80 \text{ cm.}$$

Use angle width = 24 cms = breadth of beam flange.



(2) Design of Seat Angle:

$$8.70 \times 0.80 = 6.96 \text{ t.cm}$$

$$t_{\text{req}} = \sqrt{\frac{6 \times 6.96}{24 \times 1.40}} = 1.12 \text{ cm} < 1.20 \text{ cm} \quad \text{O.K.}$$

While conservative the allowable bending stresses $F_b = 0.58 F_y$, one can say the critical section of the steel angle is rectangular and hence $F_b = 0.72 F_y$. The utilized assumptions being somewhat approximate we prefer to be conservative.



(3) Bolts Attaching the Seat Angle to Column Flange:

Use four high strength bolts of friction type as shown.
The diameter of bolts is governed by the construction condition.

$$\text{i.e. } (10 - 1.2) > 2 \times 3 d$$

$$\therefore d < 1.50 \text{ cm.}$$

$$M_1 = R \cdot e_f = 8.70 \times 3.00 = 26.1 \text{ t.cm.}$$

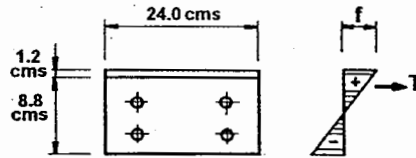
$$f = \frac{My}{I} = \frac{6 M}{b t^2} = \frac{6 \times 26.1}{24 \times (10)^2} = 0.0652 \text{ t/cm}^2$$

$$T_{\text{bolt}} = \frac{f}{2} \times 12 \times \frac{10}{2} = 1.956 \text{ ton.}$$

$$T_{i \text{ req}} = \frac{1.956}{0.80} = 2.44 \text{ tons} \rightarrow \text{Choose H.S.B. M16 (10.9)}$$

$$Q_{\text{bolt}} = \frac{R}{4} = \frac{8.70}{4} = 2.175 \text{ tons. } (P_s = 3.16 \text{ ton safe})$$

Use 4 H.S.B. M 16 (10.9)



(4) Top Cleat:

From the construction condition.

$$(a - t) > 3 d > 4.80 ; \text{choose } 1 < 55 \times 55 \times 6$$

* If We Use Non pretensioned bolts of Grade (4.6) M20 :-

$$\bar{y} \times 24 \times \frac{\bar{y}}{2} = 2 A_b \times \left[(2.20 - \bar{y}) + (6.60 - \bar{y}) \right]$$

$$\text{Use bolts } d=18 \rightarrow A_s = 0.75 \times \pi \frac{(1.8)^2}{4} \therefore A_s = 1.91 \text{ cm}^2.$$

$$12 \bar{y}^2 + 7.64 \bar{y} - 33.60 = 0$$

$$\therefore \bar{y} \approx 1.40 \text{ cm.}$$

$$I_x = \frac{1.40^3 \times 24}{3} + 2 \times$$

$$1.91 \left[0.80^2 + 5.20^2 \right]$$

$$\therefore I_x = 121.33 \text{ cm}^4.$$

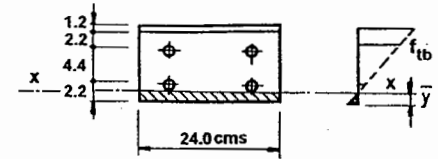
$$\therefore f_{tb} = \frac{26.10 \times (2.2 + 4.4 - 1.4)}{121.33} = 1.11 \text{ t/cm}^2$$

$$R_{t.a} = 1.11 \times 2.45 = 2.72 \text{ tons}$$

$$R_t = 0.33 \times 4 \times 2.45 = 3.23 \text{ tons}$$

$$R_{sh.a} = 8.7 / 8 = 1.080 \text{ tons}$$

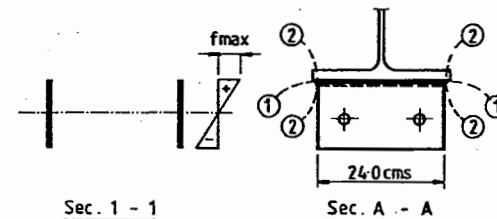
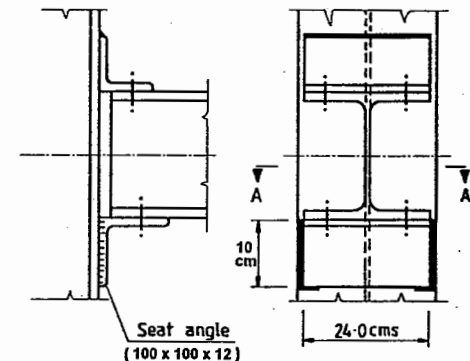
$$R_{sh} = \frac{\pi (2.0)^2}{4} \times 0.25 \times 4 = 3.14 \text{ tons}$$



Check by interaction equation :-

$$\left[\frac{1.08}{3.14} \right]^2 + \left[\frac{2.72}{3.23} \right]^2 = 0.91 < 1.0 \therefore \text{safe}$$

* If Welds are Used to Connect Seat Angle to the Column:



$$M = R \cdot e_f = 8.70 \times 3.00 = 26.10 \text{ t.cm.}$$

$$Q = R = 8.70 \text{ ton.}$$

Assume size of weld to be unity.

Weld at Sec(1-1):

$$f_{t\perp} = \frac{M \cdot y_{\max}}{I_x} = \frac{26.10 \times (10/2)}{2 \left[\frac{1 \times (10)^3}{12} \right]}$$

$$f_{t\perp} = 0.781 \text{ t/cm}^2.$$

$$q_{\downarrow\downarrow} = \frac{Q}{A_w} = \frac{8.70}{2 \times 10 \times 1} = 0.435 \text{ t/cm}^2$$

$$f_{eq} = \sqrt{f_{t\perp}^2 + 3 q_{\downarrow\downarrow}^2} = 1.08 \text{ t/cm}^2 > 1.1 \times 0.70 \text{ Unsafe}$$

$$s_{req} = \frac{1.08}{1.1 \times 0.70} \cong 1.4 \text{ cms} > \text{Thickness of seat angle.}$$

Hence either to increase the size of the seat angle or to reject the welded seat angle solution.

Example (4.8):

Redesign Example 4.3 using an Unstiffened Seated Angle Connection. Refer to section 4.9.

Solution

(1) Bearing Length on Seat Angle:

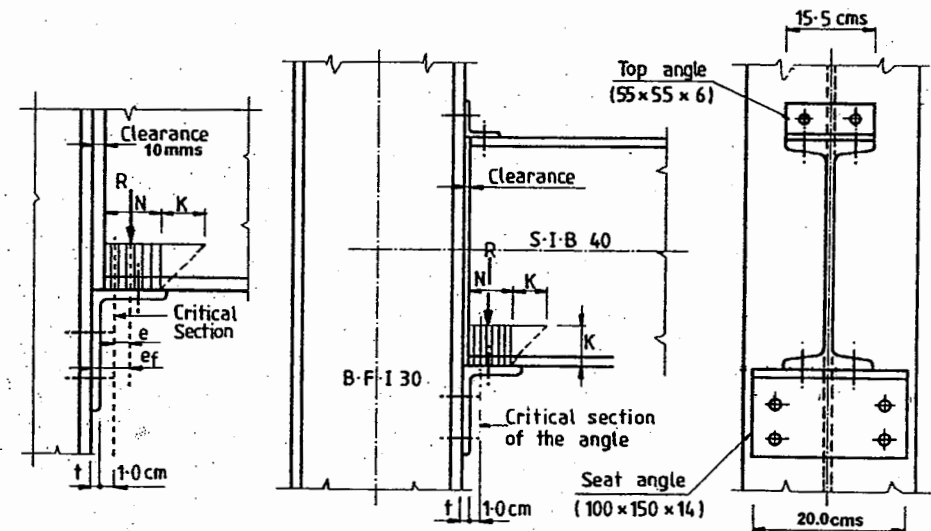
$$N + K = \frac{R}{t_w \cdot F_{crp}}, \quad K = 2 t_{bf} = 2 \times 2.16 \text{ cm.}$$

Where " t_w " is the beam web thickness, " F_{crp} " is the allowable crippling stress which equals 1.80 t/cm^2 (E.C.P. 2001).

$$N = \frac{10}{1.44 \times 1.80} - (2 \times 2.16) = -0.46 \text{ cm.}$$

$$\therefore N \geq K \quad \text{Take } N = K = 2 \times 2.16 \cong 4.5 \text{ cms.}$$

End clearance of beam from face of column allow 10 mm clearance = 1.0 cm.



(2) Design of Seat Angle:

Try seat angle 80 x 120 x 12

Assume critical section of angle to be at 1.0 cm from the inner face of angle leg (as shown in Fig.).

From Fig. :

$$e_f = \frac{N}{2} + \text{Clearance} = \frac{4.50}{2} + 1.0 = 3.25 \text{ cms.}$$

$$e = e_f - t - 1.0$$

$$e = 3.25 - 1.20 - 1.0 = 1.05 \text{ cm.}$$

Moment at root of angle:

$$= 10 \times 1.05 = 10.5 \text{ t.cms.}$$

Assume that the angle width is 20 cm.

$$\text{Req. thickness} = \sqrt{\frac{6 \times 10.5}{20 \times 1.40}} = 1.5 \text{ cms} > 1.2 \text{ cms} \text{ Unsafe}$$

Try an unequal angle [100 x 150 x 14]

$$e_f = \frac{N}{2} + \text{Clearance} = \frac{4.50}{2} + 1.0 = 3.25 \text{ cms.}$$

$$e = e_f - t - 1.0 = 3.25 - 1.40 - 1.0 = 0.85 \text{ cms}$$

Moment at the angle root = $10 \times 0.85 = 8.5 \text{ t.cms.}$

$$\text{Req. thickness} = \sqrt{\frac{6 \times 8.5}{20 \times 1.40}} = 1.35 \text{ cms} < 1.4 \text{ cms (O.K.)}$$

(3) Bolts Attaching the Seat Angle to the Column Flange:

Use high strength bolts (10.9) type.

* Construction condition allows a maximum bolt diameter of

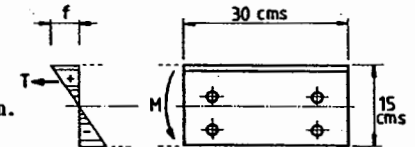
$$d_{\max} = \frac{15 - 1.40}{6} = 2.26 \text{ cm.}$$

$$M = R \cdot e_f$$

$$= 10 \times 3.25 = 32.50 \text{ t.cms.}$$

$$f = \frac{6 \times 3.25 \times 10}{30 \times (15)^2} = 0.03 \text{ t/cm}^2$$

$$T = \frac{0.03}{2} \times \frac{15}{2} \times 30 = 3.375 \text{ ton.}$$



Use two bolts per row

$$\therefore T_{\text{bolt ext, M}} = \frac{3.375}{2} = 1.6875 \text{ ton.}$$

* Minimum allowable diameter of bolt from construction condition:

$$d_{\min} = \frac{15 - 1.40}{2 \times 6} = 1.13 \text{ cm.}$$

$$\therefore \text{Use 4 H.S.B. M(12) (10.9). } \left[T_{\text{ext, M all.}} = 0.80 \times 5.30 \right]$$

$$= 4.24 \text{ ton}$$

$$> 1.687 \text{ t safe}$$

Check of Shear on Bolts:

$$Q_{\text{bolt}} = \frac{R}{4} = \frac{10}{4} = 2.5 \text{ tons.}$$

$$P_s \text{ (M 12)} = 1.92 \text{ ton} < 2.5 \text{ tons} \therefore \text{Unsafe}$$

Increase the bolt diameter to 16 mm $\left[P_s (M 16) = 3.58 \text{ tons} \right]$
 $> 2.5 \text{ t}$
 Safe

Use 4 H.S.B. M 16 [(10.9) type]

(4) Top Cleat:

From the construction condition where $d = 16 \text{ mm}$,

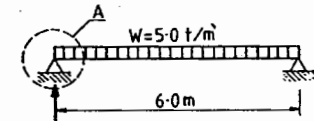
$(a - t) < 3 d < 4.8 \text{ cms}$; Choose 1 $\times$ 55 $\times$ 55 $\times$ 6

Example (4.9): Stiffened Seat Beam Connection:

Refer to Section 4.10

If the total load on a floor beam of an industrial building is 5 t/m , span of the beam is 6.0 ms . Design a stiffened seated beam connection.

Solution



(1) Design of the Beam Cross Section:

$$M = \frac{W L^2}{8} = \frac{5 \times 6^2}{8}$$

$$M = 22.50 \text{ mt.}$$

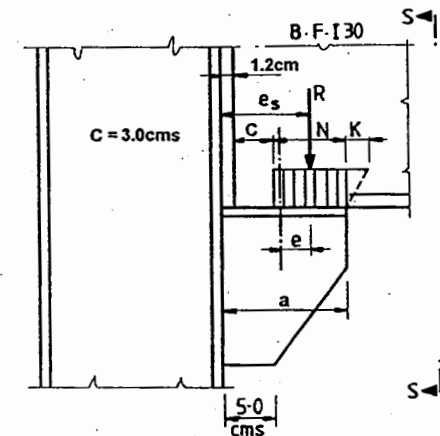
$$Z_{x \text{ req}} = \frac{22.50 \times 100}{1.40}$$

$$= 1607 \text{ cm}^3$$

Choose BFI # (30)

$$\text{Reaction} = R = \frac{5 \times 6}{2}$$

$$= 15.0 \text{ tons.}$$



Detail "A"

(2) Determination of Seat Width "W":

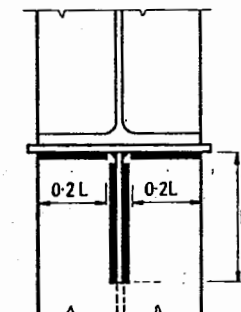
* Check Against Crippling of Beam Web:

$$N = \frac{R}{f_{all} \times t_w} - K, \quad K = 2 \times t_{fb}$$

$$= 2 \times 2 \text{ cms.}$$

$$N = \frac{15.0}{1.80 \times 1.20} - 2 \times 2 = 2.90 \text{ cms}$$

$$N \geq K ; N = 4.0 \text{ cms.}$$



Sec. S - S



Seat width $a = N + \text{Clearance} + C = 4.0 + 1.2 + 3.0 = 8.2$ cms.

where C is as shown in figure.

Hence take $a = 8.2$ cms.

(3) Stiffener Thickness " t_s ":

The stiffener thickness " t_s " is chosen such that several conditions are satisfied:

A- $t_s \geq t_{wb} \geq 1.20$ cm.

Where t_{wb} = thickness of beam web.

B- Local Buckling Condition:

$$\frac{a}{t_s} < 16 \quad \therefore t_s > \frac{8.2}{16} > 0.48 \text{ cm.}$$

C- Bearing Stress on Stiffener:

The reaction from the beam will be concentrated on the far edge of the seat bracket.

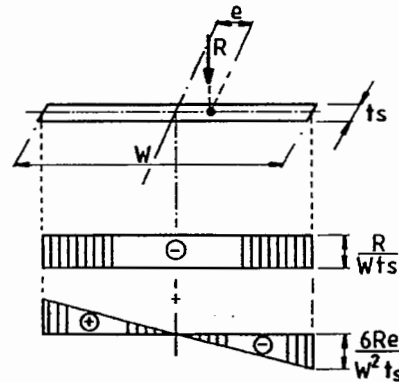
From Fig. :

$$e_s = a - \frac{N}{2} = 8.2 - \frac{4}{2} = 6.20 \text{ cms.}$$

$$e = e_s - \frac{a}{2} = 6.20 - \frac{8.2}{2} = 2.10 \text{ cms.}$$

$$f_{bc} = \frac{R}{a t_s} + \frac{6R.e}{a^2 t_s}$$

$$\leq f_{\text{bearing}} = 1.2 F_U = 4.32 \text{ t/cm}^2.$$



$$4.32 = \frac{15}{8.2 t_s} + \frac{6 \times 15 \times 2.10}{8.2^2 \times t_s}$$

$$t_s = 1.075 \text{ cm. (a conservative design uses } f_{\text{max}} = f_{bc})$$

D- Weld Condition:

$$2(s) \times 0.72 = 0.84 t_s \quad (s = \text{size of fillet weld})$$

$$t_s = \frac{2s \times 0.72}{0.84} = 1.71 s$$

If we used weld of size 1.0 cm.

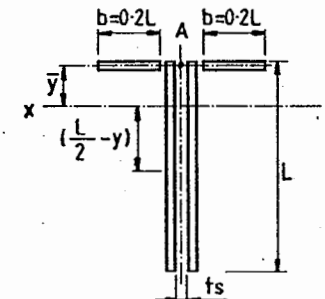
$$t_s = 1.71 \text{ cms.}$$

From all the previous conditions, the stiffener thickness " t_s " can be estimated as; $t_s = 1.80$ cm.

(4) Determination of the Stiffener Length:

A practical trend is to take the top-plate width as equal to ($\cong 0.40 L$); the stiffener plate length. The stiffener length is chosen to satisfy the stress conditions of fillet welds.

Take $s = 1.0$ cm.



Section in Weld Lines.



$$\bar{y} = \frac{2 L \times L / 2}{(0.40 L + 2 L)} = \frac{L}{2.40}$$

$$I_x = \frac{2 L^3}{12} + 2 L \left[\frac{L}{2} - \frac{L}{2.4} \right]^2 + 0.40 L \times \left[\frac{L}{2.4} \right]^2$$

$$I_x = \frac{L^3}{4} \quad ; \quad Z_{x_A} = \frac{I_x}{y_A} = \frac{L^3 / 4}{L / 2.40} = 0.60 L^2$$

$$\bar{f}_{A_{\perp}} = \frac{R \cdot e_s}{Z_{x(A)}} = \frac{15 \times 7.50}{0.60 L^2}$$

$$q_{A_{\downarrow\downarrow}} = \frac{R}{A_w} = \frac{15}{2.4 L} = \frac{6.25}{L}$$

$$f_{eq.} = \sqrt{\bar{f}_{A_{\perp}}^2 + 3 q_{A_{\downarrow\downarrow}}^2} = \sqrt{\left(\frac{187.50}{L^2}\right)^2 + 3 \left(\frac{6.25}{L}\right)^2}$$

$$< 0.72 \times 1.1 \text{ t/cm}^2.$$

This equation can be best solved by trial and error.

$$\text{Assume } L = 15 \text{ cm} \rightarrow f_{eq.} = 1.10 \text{ t/cm}^2 \text{ (Unsafe).}$$

$$L = 21 \text{ cm} \rightarrow f_{eq.} = 0.70 \text{ t/cm}^2 \text{ (Safe).}$$

$$\text{Width } b = 0.20 L = 0.40 \times 21 = 8.40 \text{ cms.}$$

But the width of the top plate should not be less than b_{fb} , which means that top plate width is to be taken 32 cm. The weld width "b" can be taken 12 cm. The weld length "L" can thus be decreased.



$$\bar{y} = \frac{2 L \times L / 2}{(2 b + 2 L)}$$

$$= \frac{L^2}{2 (b + L)}$$

$$Z_{x(A)} = \frac{I_x}{y_A} = \frac{I_x}{\bar{y}} = \frac{L (L + 4 b)}{3}$$

$$Z_{x(A)} = \frac{L (L + 4 \times 12)}{3}$$

$$= \frac{L (L + 48)}{3}$$

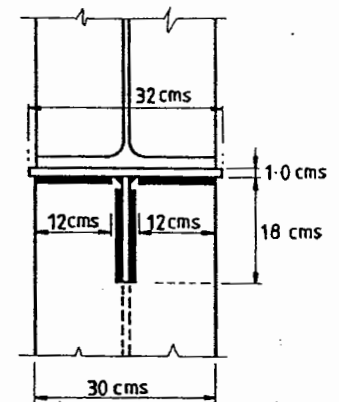
$$\bar{f}_{A_{\perp}} = \frac{15 \times 7.5 \times 3}{L (L + 48)} = \frac{337.50}{L (L + 48)}$$

$$q_{\downarrow\downarrow} = \frac{15}{2 L} = \frac{7.50}{L}$$

$$f_{eq.} = \sqrt{\left(\frac{337.5}{L (L + 48)}\right)^2 + 3 \left(\frac{7.5}{L}\right)^2}$$

$$\text{By trial and error} \rightarrow L = 18 \text{ cms. } f_{eq.} \approx 0.79 \text{ t/cm}^2. \text{ (O.K.)}$$

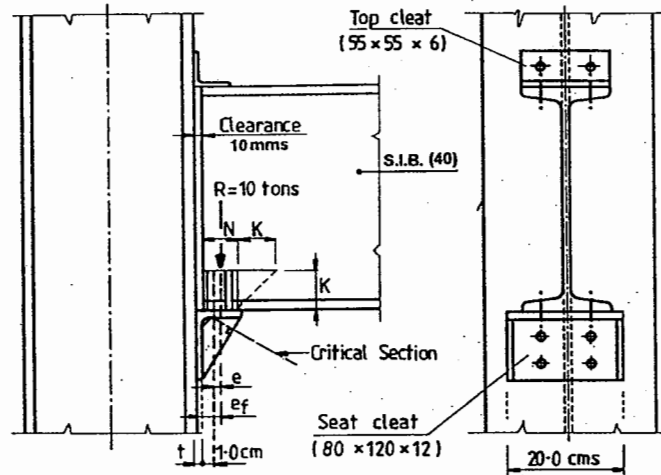
N.B. The steel T-Section will be safe against bending and equivalent stresses.



Example (4.10): Stiffened Seat-Angle Connection Section 4.13

Redesign Example 4.8 using a stiffened seat-angle connection.

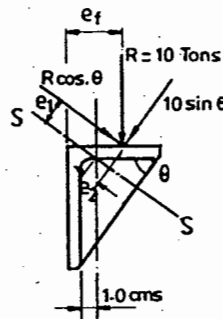
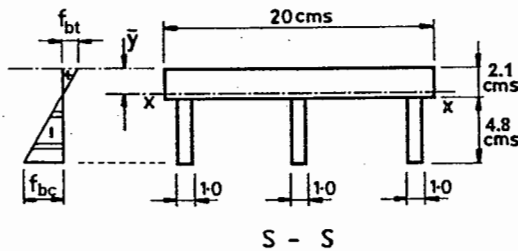
Solution



1. Geometrical configuration:

* Choose a seat angle (80 x 120 x 12) with 3 stiffeners of thickness 1.0 cms each and of width 20.0 cms.

* The critical section (s-s) is assumed at 1.0 cm from the inner face of the angle leg.



2. Properties of Critical Section (s-s):

$$\bar{y} = \frac{20 \times 2.1 \times 1.05 + 3 \times 4.8 \times 1 \times 4.5}{20 \times 2.1 + 3 \times 4.8 \times 1.0} = 1.93 \text{ cms}$$

$$I_x = 170 \text{ cm}^4$$

From figure get $e_1 = 2.65 \text{ cm}$; $e_2 = 2.2 \text{ cm}$

3. Straining Actions and Stresses:

$$M = 10 \sin \theta \times e_2 + 10 \cos \theta \times e_1$$

$$M \approx 32.98 \text{ t.cm} \quad (\cos \theta = 0.554 ; \sin \theta = 0.832)$$

$$f_{bt} = \frac{32.98 \times 1.93}{170} = 0.374 \text{ t/cm}^2 \text{ (o.k.)}$$

$$f_{bc} = \frac{32.98 \times 4.95}{170} = 0.96 \text{ t/cm}^2 \text{ (o.k.)}$$

.... safe which means that we can decrease the thickness of the stiffeners.

4. Bolts Attaching the Seat Angle to the Column flange:

Proceed as previously outlined in example (4.8).

5. Top Cleat:

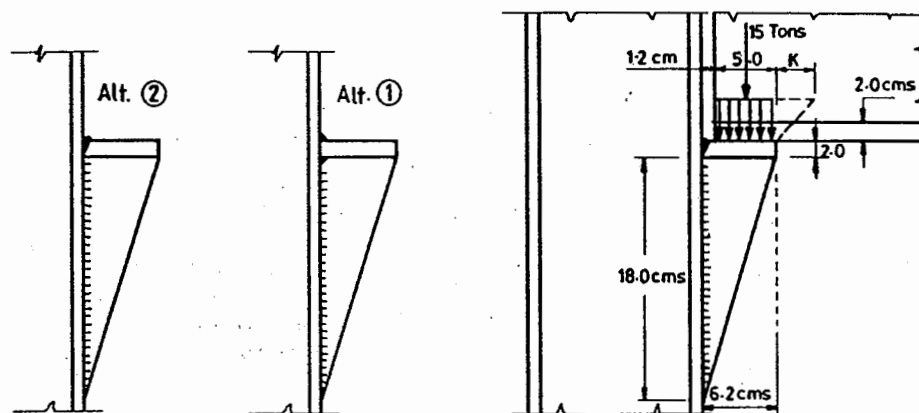
Choose an angle 55 x 55 x 6 with 4 high strength bolts (d = 16 mm). Two bolts are to be arranged on each leg.

Example (4.11): Triangular Stiffened Seat Brackets:

Refer to Section 4.11

Redesign the connection given in Ex. 4.9 using a triangular stiffened seat bracket.

Solution



○ Referring to the results previously computed in Ex. 4.9,
 $N = 5.0 \text{ cms.}$; $W = a = 1.2 + 5.0 = 6.2 \text{ cms.}$

○ The height "h" = 18.0 cms.

○ The width of the top plate = 30 cms. (flange beam width)

(1) Triangular Stiffener Thickness:

(a) Buckling Condition:

$$h / t \leq 48 / \sqrt{F_y}$$

$$\text{hence } t \geq \frac{h \sqrt{F_y}}{48} \geq \frac{18 \sqrt{2.4}}{48} \geq 0.58 \text{ cm.}$$

(b) Yielding Condition:

$$t \geq \frac{15.0}{0.583 \times 2.4 \times m \times 6.2}$$

$$m = 1.39 - 2.2 \left(\frac{6.2}{18} \right) + 1.27 \left(\frac{6.2}{18} \right)^2 - 0.25 \left(\frac{6.2}{18} \right)^3$$

$$m = 0.763$$

$$\text{hence } t_s \geq 2.26 \text{ cms. Take } t = 23 \text{ mms.}$$

(2) Welds Attaching the Triangular Stiffener to the Column Flange:

If alternative (1) is utilized hence refer to Example 4.9 the required size of welds is 20 mms.

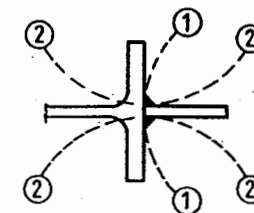
If alternative (2) is chosen, consider only that the two vertical lines of welds are transmitting the beam reaction to the column flange; thus:

$$q_{\perp} = \frac{15}{2 \times 1.0 \times 18} = 0.416 \text{ t/cm}^2.$$

Plane 1-1:

$$q_{\perp} = 0.416 \text{ t/cm}^2.$$

$$f_{t\perp} = \frac{15 \times 3.7 \times 6}{2 \times 1 \times (18)^2} = 0.51 \text{ t/cm}^2.$$



$$f_e = \sqrt{(0.51)^2 + 3 \times (0.416)^2} = 0.88 \text{ t/cm}^2 > 0.79 \text{ t/cm}^2$$

Hence $s = 1.15 \text{ cm}$. Take $s = 12 \text{ mms}$.

Plane 2-2:

$$q_{\downarrow//} = \frac{0.416}{1.2} = 0.347 \text{ t/cm}^2$$

$$q_{\rightarrow \perp} = \frac{0.51}{1.2} = 0.425 \text{ t/cm}^2$$

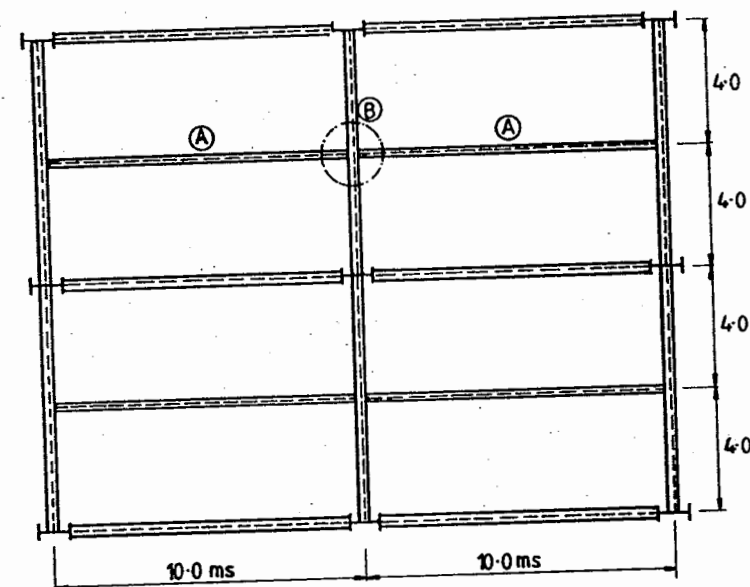
$$q_t = \sqrt{(0.347)^2 + (0.425)^2} = 0.548 \text{ t/cm}^2 < 0.72 \text{ (o.k.)}$$

Example (4.12): Flexible Beam-to-Beam Connection:

Refer to Section 4.3.1.1

It is required to design the web framing angles beam-to-beam connection shown by the dotted circle in the Fig. below. Use Non pretensioned bolts of Grade (4.6). Top of beams are to be coplanar.

Solution



Layout

Applied Loads:

Assume own weight of beam (A) = 100 kg/m .

Floor (R.C.) slab 12 cms thick = 300 kg/m^2 .

Floor covering = 150 kg/m^2 .

Superimposed loads = 300 kg/m^2 .

$$R_{A(D.L)} = \frac{0.1 \times 10}{2} + (0.3 + 0.15) \times 4.0 \times \frac{10}{2} = 9.5 \text{ tons.}$$

$$R_{A(L.L)} = 0.3 \times 4.0 \times \frac{10}{2} = 6 \text{ tons.}$$

$$M_{A(D.L+L.L)} = \left[0.1 + (0.3 + 0.15 + 0.3) \times 4 \right] \times \frac{10^2}{8} = 38.75 \text{ t.m.}$$

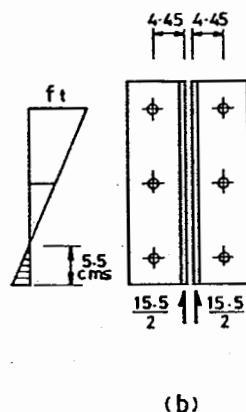
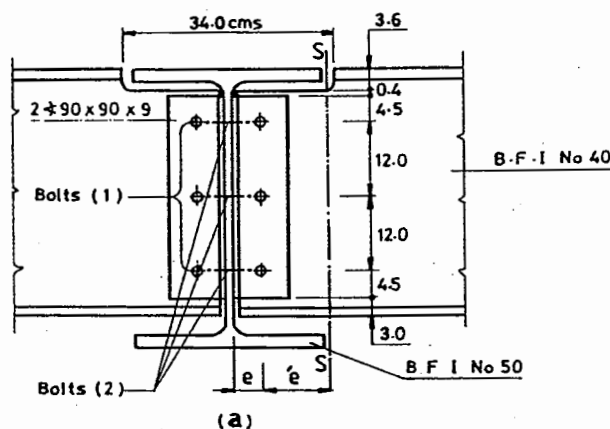
$$Z_{x(\text{required})} = \frac{38.75 \times 100}{1.4} = 2767.8 \text{ ton}^3.$$

$$\text{Choose a B.F.I No 40 (} f = \frac{38.75 \times 100}{3030} = 1.28 \text{ t/cm}^2 < 1.4 \text{) o.k}$$

$$M_{B(DL+L.L)} = \frac{2(9.5 + 6) \times 8}{4} + \frac{0.2 \times 8^2}{8} = 63.6 \text{ t.m.}$$

$$\text{Choose a B.F.I. N } \cong 55 \text{ (} f_b = \frac{6360}{4530} = 1.4 \text{ t/cm}^2 < 1.4 \text{ (o.k).}$$

Group of Bolts (1):



Assume diameter of ordinary bolts $d = 20 \text{ mms.}$

$$R_{sh} = 2 \times 0.25 \times 4 \times \frac{\pi \times 2^2}{4} = 6.28 \text{ tons.}$$

$$R_b = 0.8 \times 3.6 \times 1.4 \times 2 = 8.064 \text{ tons. (where edge dist.} = 2.25d \text{)}$$

$$\text{Number of bolts} = \frac{9.5 + 6}{6.28} = 2.46 \text{ take 3 bolts}$$

Pitch = 12 cms and edge distance 4.5 cms.

Group of Bolts (2):

Right span of beam (A) loaded by L.L. in addition to D.L.

$$R_{D.L+L.L}^{\downarrow} = 9.5 + 6.0 = 15.5 \text{ tons.}$$

$$R_{sh}^{\downarrow} = 0.25 \times 4 \times 1.4 \times \frac{\pi \times 2^2}{4} = 3.14 \text{ tons}$$

$$R_b^{\downarrow} = 0.8 \times 3.6 \times 0.9 \times 2 = 5.184 \text{ tons. (where framing } < s \text{ } 90 \times 90 \times 9 \text{)}$$

$$\text{Number of bolts} = \frac{15.5}{3.14} = 4.93 \text{ bolts, take 2 columns of 3 bolts each.}$$

These bolts will be subjected to a torsional moment

$$M_t = \frac{15.5}{2} \times \frac{(9 + 0.9)}{2} = 38.36 \text{ t.cms. (see Fig. b)}$$

$$R_{(Mt)}^{\rightarrow} = \frac{38.36 \times 12}{2 \times 12^2} = 1.60 \text{ tons}$$

In addition the eccentricity $e = \frac{9 + 0.9}{2} = 4.45 \text{ cms}$ will induce a bending normal stress (i.e tensile force) on this group of bolts due to the live load reaction only ($R_{L.L} = 6.5 \text{ t}$)



Assume contact between the framing angles and the main beam web. Either compute exactly the position of the N.A. or assume its location at $h/6 = 33/6 = 5.5$ cms from the lower edge of the framing angles.

$$A = 3.14 \text{ cm}^2.$$

$$A_s = 2.45 \text{ cm}^2.$$

$$I_{xx} = \frac{1}{3} \times 2 \times 9 \times 5.5^3 + 2 \times 2.45 \times (11^2 + 23^2) \\ = 2529 \text{ cm}^4.$$

$$f_{tb} = \frac{15.5 \times 4.95 \times 23}{2529} = 0.69 \text{ t/cm}^2.$$

$$R_{t.a} = 0.69 \times 2.45 = 1.69 \text{ tons}$$

$$R_t = 0.33 \times 4 \times 2.45 = 3.23 \text{ tons}$$

$$R_{sh.a_v} = 15.5 / 6 = 2.583 \text{ tons}$$

$$R_{(Mt)} = 1.60 \text{ tons}$$

$$\text{Resultant} = \sqrt{(2.853)^2 + (1.6)^2} = 3.038 \text{ t/cm}^2.$$

Check using the interaction equation:

$$\left[\frac{3.038}{3.14} \right]^2 + \left[\frac{1.69}{3.23} \right]^2 = 1.2 > 1.0 \text{ (unsafe)}$$

Use 3 bolts Grade 5.8 will be safe.



Framing Angles Thickness:

$$T = 0.266 \times 2.355 = 0.626 \text{ ton.}$$

$$M = 0.626 \times \left(\frac{a+t}{2} - K \right) = 0.626 \times 3.15 \approx 2.0 \text{ t.cm.}$$

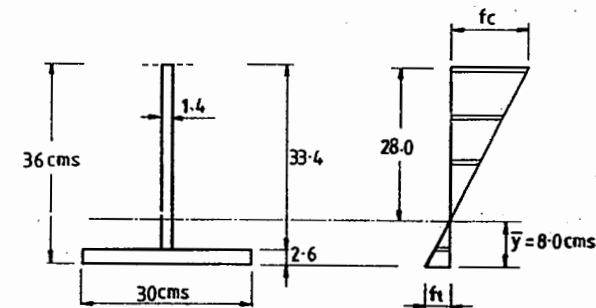
$$t = \sqrt{\frac{2.0 \times 6}{1.4 \times (4.5 + 6)}} = 0.903 \text{ cm.}$$

Choose 2 $\times$ s 90 $\times$ 90 $\times$ 11.

Check of Notch: (Section S-S Fig. a)

$$e = \left(\frac{34 - 1.6}{2} \right) - \left(\frac{9 + 1.1}{2} \right) = 11.15 \text{ cms.}$$

$$M = 15.5 \times 11.15 = 172.825 \text{ t.cms.}$$



$$A = 30 \times 2.6 + 33.4 \times 1.4 = 124.76 \text{ cm}^2.$$

$$\bar{y} = \frac{30 \times 2.6 \times 1.3 + 33.4 \times 1.4 \times (2.6 + \frac{33.4}{2})}{124.76} = 8.0 \text{ cms.}$$

$$I_{xx} = \frac{1}{3} \times 1.4 \left[(33.4 + 2.6 - 8)^3 + (8 - 2.6)^3 \right] + 30 \times 2.6 \\ \times (8 - 1.3)^2 = 13819 \text{ cm}^4.$$



$$f_{bd} = \frac{172.825 \times 28}{13819} = 0.35 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (o.k.)}$$

$$q_{\max} = (15.5 \times s_x) / (13819.1 \times 1.4)$$

$$s_x = 1.4 \times 28 \times 14 = 548.8 \text{ cm}^3$$

$$q = 0.44 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2 \text{ (o.k.)}$$



Example (4.13): Flexible Beam-to-Beam Connection Using End-Plates. Refer to Section 4.14.2.

Redesign the connection previously given in Example 4.12 using end-plates instead of the framing angles.

Solution

1. Bolts Connecting the End Plates to the Main Beam Web:

$$\circ R_{D.L+L.L} = 15.5 \text{ tons. ; } d = 20 \text{ mms.}$$

$$R_{s.s} = 3.14 \text{ tons.}$$

$$R_b = 0.8 \times 3.6 \times 1.0 \times 2 = 5.76 \text{ tons.}$$

Where the thickness of the end-plate has been chosen 1.0 cms

$$\text{Number of ordinary bolts} = \frac{15.5}{3.14} = 4.93 \text{ bolts}$$

Take two columns of 3 bolts each.

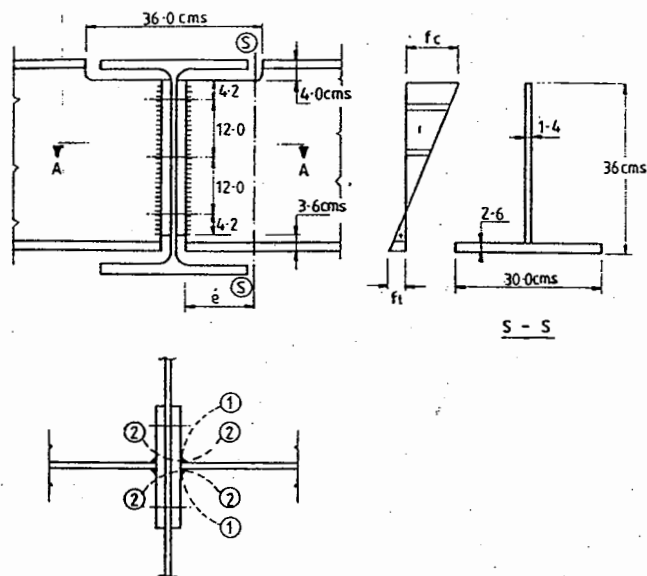
$$\circ 2 (R_{D.L+L.L} + R_{D.L}) = 15.5 \times 2 = 31 \text{ tons.}$$

$$R_{d.s} = 6.28 \text{ tons}$$

$$R_b = 0.8 \times 3.6 \times 1.6 \times 2 = 9.216 \text{ tons.}$$

$$\text{Number of bolts} = \frac{31}{6.28} = 4.93 \text{ bolts}$$

Hence two columns of 3 bolts each is sufficient.



2. Fillet Welds Attaching the End-Plate to the Secondary-Beam

Web:

Assume $s = 1.0$ cms.

Plane (1-1) and (2-2):

$$q_{\downarrow} = \frac{15.5}{2 \times 1.0 \times (40 - 4.0 - 3.6)} = 0.239 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2.$$

$$s = 1.0 \times \frac{0.239}{0.72} = 0.33 \text{ cms.}$$

Hence use the minimum allowed size $s = 4$ mms.

3. End Plate Thickness:

Thickness of end plate = 1.0 cms. ; shear at bolts section is:

$$q = \frac{3 \times 15.5}{2 \times 2(40 - 4.0 - 3.6 - 3 \times 2.2)} = 0.435 \text{ t/cm}^2 < 0.35 F_y \text{ (o.k.)}$$



4. Notch Stresses: Section S-S

$$e' = \left(\frac{36 - 1.6}{2} \right) - 1.0 = 16.2 \text{ cms.}$$

$$M = 15.5 \times 16.2 = 251.1 \text{ t.cms.}$$

$$\bar{y} = 8.0 \text{ cms.} ; I_{xx} = 13819.1 \text{ cm}^4.$$

$$f_b = \frac{251.1 \times (36 - 8)}{13819.1} = 0.51 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (o.k.)}$$

$$q_{\max} = 0.44 \text{ t/cm}^2 \text{ (o.k.) (Refer to Ex. 4.12).}$$

CHAPTER 5



CHAPTER 5

RIGID CONNECTIONS

Rigid connections are those transmitting the induced moments, shearing forces and normal forces from one structural element to the other. The conventional rigid connections are the beam-to-column connections either fully field welded or partially field bolted generally used for multi-storey frames. Two other rigid common connections are: the corner (eave) and the apex connections either welded or bolted. On the other hand if for a grid beam system it is required to achieve the continuity of the beams, one must proceed using a rigid connection. Naturally, the design intent is to achieve no relative rotation of members within the joint.

In the following sections four group welded and bolted beam-to-column connections as outlined below are analyzed:

Group (I) Welded beam - to - column Connections :-

- (a) Horizontal stiffener configuration. (Section 5.1)
- (b) Vertical Tee-stub stiffener configuration. (Section 5.2)
- (c) Vertical plate stiffener configuration. (Section 5.3)
- (d) Connecting plate configuration. (Section 5.4)

Group (II) Bolted beam - to - column Connections :-

- (a) Bolted Tee-stub moment connection. (Section 2.8.2)
- (b) Bolted end-plate moment connections (flushed, extended and haunched). (Sections 2.8.3.1, 2.8.3.2 and 2.8.3.3)
- (c) Bolted seat angle moment connection. (Example 5.12)
- (d) Connecting plate moment connections. (Example 5.13)

Group (III) Different types of welded Knee Connections :-

- (a) The square corner connection. (Section 5.6.3)
- (b) The corner knee with bracket. (Section 5.6.4)
- (c) The haunched tapered knee connection. (Section 5.6.5)
- (d) The haunched curved knee connection. (Section 5.6.6)

Group (IV) Eave and Apex Connections :-

- (a) The direct welded eave connection. (Section 5.6.3.1)
- (b) The end plate eave connection. (Section 5.7.1.2)
- (c) The splice plate eave connection. (Section 5.7.1.3)
- (d) The direct butt welded apex connection. (Section 5.7.2.1)
- (e) The splice plate apex connection. (Section 5.7.2.3)
- (f) The end plate apex connection. (Section 2.8.3.2)

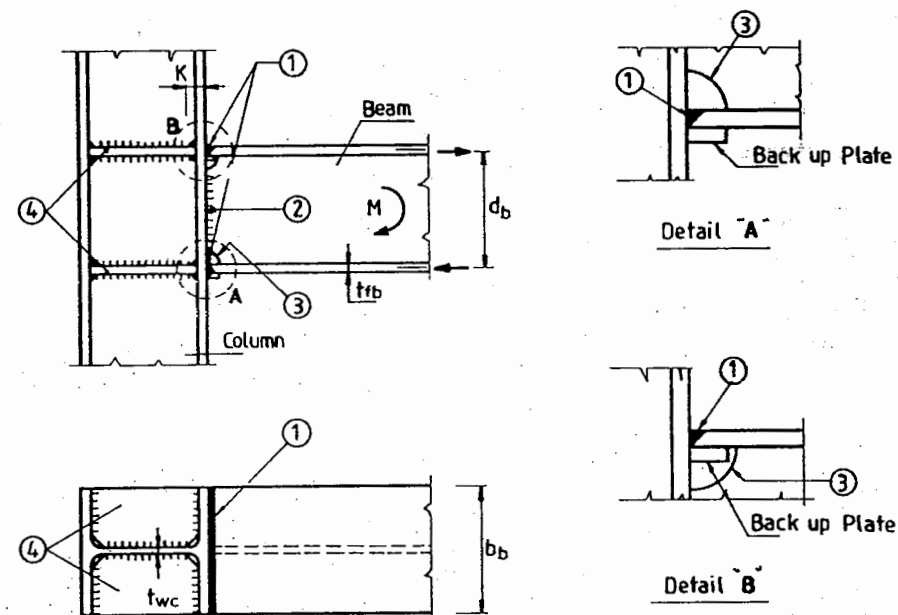
Group (V) Beam - to - Beam rigid Connections :-

The continuous grid beam connections are classified as rigid connections where the angle of rotation is to be maintained. The following configurations are also analyzed:

- (a) The direct butt welded beam-to-beam connection. (Sec. 5.8.1)
- (b) The direct fillet welded beam-to-beam connection. (Sec. 5.8.1)
- (c) The splice plate beam-to-beam connection either welded or bolted. (Section 5.8.2)
- (d) The end plate beam-to-beam connection. (Section 5.8.2)
- (e) The splice end plate connection. (Section 5.8.2)

Group (I) - Sections 5.1, 5.2, 5.3, 5.4 and 5.5 :-5.1. Welded Beam-to-Column Connections:Horizontal Stiffener Configuration:5.1.1. Behaviour:

The straight forward welded rigid beam-to-column connection is that shown in Fig. 5.1. Fully field welds are needed for flanges in order to transmit the moment from the beam flanges to the column flange either by butt or fillet welds. The analysis of this type of welded connections has been analyzed in Section 3.9, where three phenomenon are to be considered: (a) crippling of the column web opposite to the beam compression flange; (b) the bending of the column flange opposite to the beam tension flange; (c) the shear resistance of the corner column web.



(1) Beam flanges butt welds.

(2) Beam web Fillet welds.

(3) Cope holes.

(4) column web stiffeners.

Fig. 5.1



In such connections fitted column web stiffeners (shown by item "4" in Fig. 5.1) are generally required. Cope holes should be provided to allow access for down hand welding. Normally a back up plate is used to ensure a complete-penetration groove weld.

5.1.2. The Stiffener Choice Configuration:

As has been outlined in Section 3.9, the horizontal pair of plate stiffeners (shown by item "4" in Fig. 5.1) at (A) must provide the following cross sectional area in order to prevent the crippling of the column web:

$$A_{st(A)} \geq b_b \cdot t_{fb} - t_{wc} (t_{fb} + 5K) \quad (a)$$

Concerning the stiffener configurations, Fig. 5.2, illustrates three different possibilities:

(a) If the stiffener is only required to overcome the local over stress close to the compression beam flange, one can utilize a pair of plate stiffeners fitted and close to the loaded location (see Fig. 5.2.a). Their length (L_{st}) must be sufficient for fillet weld lines transmitting the excess of load at the compression beam flange i.e:

$$L_{st} = \frac{C - t_{wc} \cdot (t_{fb} + 5K) \cdot F_{crp}}{4 \cdot q_w \cdot s} \quad (b)$$

where F_{crp} = the allowable crippling stress = $0.75 F_y$

q_w = the allowable fillet shear stress = $0.2 f_u$

$c = M / d_b$; d_b = beam height - t_{fb}

s = size of fillet weld.



It is to be observed that a torsional moment (M') will result on the fillet weld lines connecting the stiffeners to the column web. This moment can be computed as follows:

$$M' = \frac{1}{2} F_r \times \frac{b_b - t_{wc}}{4} \quad (c)$$

where: F_r = the remaining compression force.

$$F_r = C - t_{wc} \cdot (t_{fb} + 5K) \cdot F_{crp} \quad (d)$$

The induced torsional shear stresses $q_{Mt \perp}$ are to be added to the concentric shear stresses ($q_{//} = F_r / 4 \cdot L_{st} \cdot s$) and hence the resultant shear stress are to be checked against the allowable values. These additional stresses are to be considered only for short stiffeners.

(b) Fig. 5.2.b, illustrates a long, singly fitted stiffener, where the web column crippling is to be prevented. The stiffeners are extended but are not necessarily fitted to the other flange. It is generally cheaper to curtail these stiffeners short of the unloaded flange a distance not exceeding 5 times the web column thickness ($5 \cdot t_{wc}$)

(c) The fully fitted stiffeners shown in Fig. 5.2.c, must satisfy the relation (a) and in addition the choice of the dimensions (b_{st}, t_{st}) must satisfy the local buckling requirements:

$$\text{i.e.} \quad \frac{b_{st}}{t_{st}} \leq \left(\frac{25}{\sqrt{F_y}} = 16.1 - \text{St37} \right) \quad (e)$$

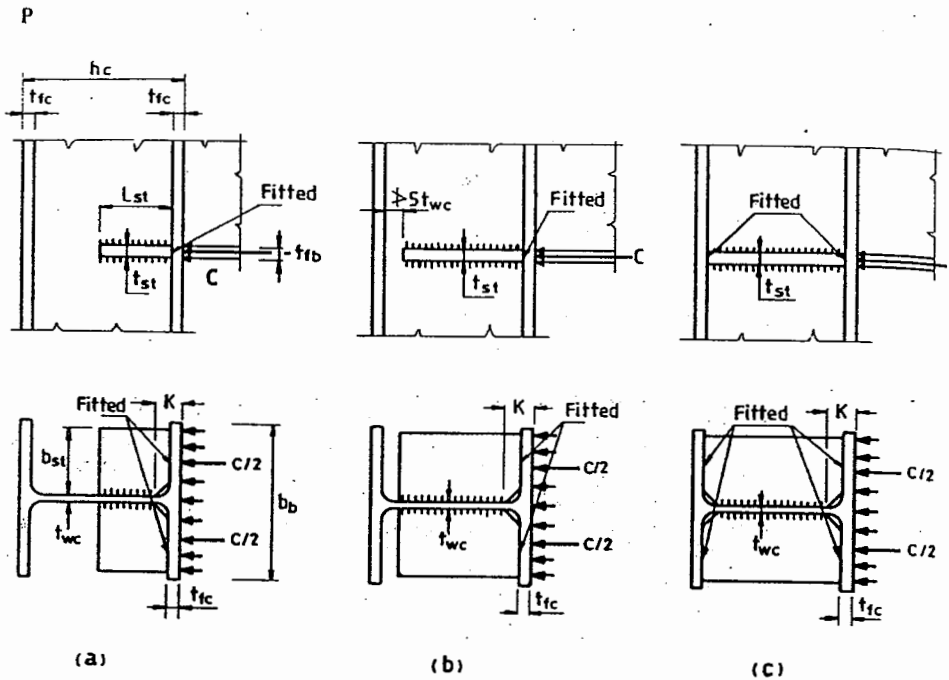


Fig. 5.2

For an interior beam-to-column connection, if the beams are of different heights on the opposite flanges of the column, one can choose one of the following configurations:

(a) Fig. 5.3.a, illustrates the straight forward solution where a stiffener (A) is utilized to overcome the bending of the column flange on the tension side, while opposite to the compression beam flanges, two stiffeners (B) and (C) are provided to respond to the remaining compression force F_r computed as in equation (d) to prevent the crippling of the column web.

(b) The other possible configuration is shown in Fig. 5.3.b where opposite to the compression flanges an inclined stiffener (B) is utilized. This stiffener will be designed on the remaining of the compression force ($F_r/\cos\theta$), where F_r can be computed as in Equation (d).

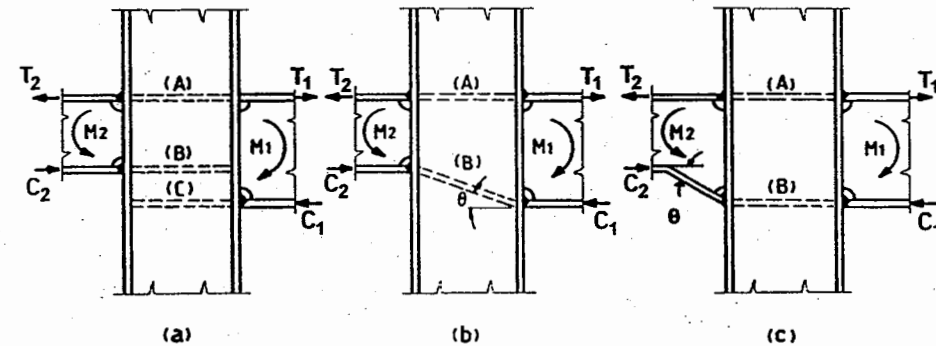


Fig. 5.3

(c) Fig. 5.3.c, illustrates a third configuration where the flange of the beam of the smallest height is forced by an inclination (θ) in order to achieve the same height at the opposite side of the column. This will allow the use of one horizontal stiffener (B). The excessive fabrication cost tends to reject this configuration, while configuration (a) remains the best one.

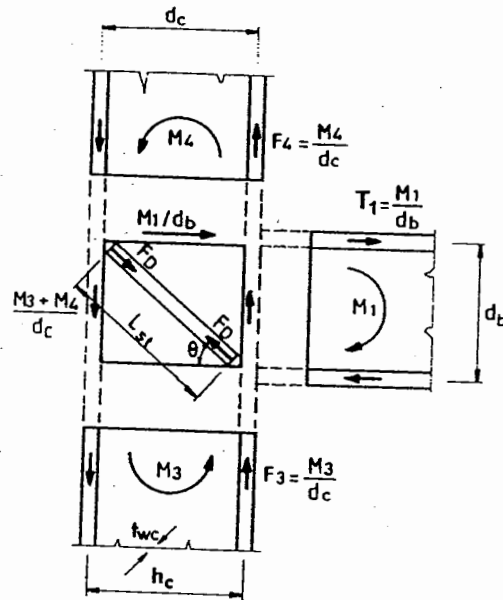
Refer to Examples 5.1 and 5.2.

5.1.3. The Use of Diagonal and Inclined Stiffeners:

Traditionally, horizontal fitted stiffeners are provided on the lines of the compression and/or tension beam flanges (see Fig. 5.4.a).

where:

F_D = the induced force in the diagonal pair of stiffeners.



$$M_1 = M_3 + M_4$$

Neglect Q_1, Q_3, Q_4

t_{wc} = Web column thickness

$$d_c = h_c - t_{fc}$$

$$d_b = h_b - t_{fb}$$

F_D = Remaining diagonal Force

Fig. 5.5

Replacing (q_a) of equation (f) in (g) we get:

$$F_D = \left[\frac{M_1}{d_b} - q \cdot h_c \cdot t_{wc} \right] / \cos \theta \quad (h)$$

Hence the dimensions of the diagonal stiffeners can be computed:

$$2b_{st} \cdot t_{st} = F_D / F_c \quad (i)$$

Where F_c is the allowable compressive stress = $0.58 F_y$

For the choice of the dimensions b_{st} and t_{st} relation (e) is to be satisfied in order to prevent the local buckling.

5.1.4. Additional Information Regarding Stiffeners:

(a) Horizontal Compression Stiffener Cancelled:

The lower horizontal stiffener at the compression beam flange, can be omitted if the diagonal stiffener is designed to carry the resultant of the forces (C_1) and (C_3) shown in Fig. 5.6.a.

(b) MORRIS Stiffener:

Fig. 5.6.b, illustrates a recent stiffening configuration namely MORRIS stiffener where a diagonal tension stiffener is utilized. The different components of this type are to be designed according to the following:

(i) The horizontal portion (1-2) of the MORRIS stiffener opposite to the beam tension flange must have a length not less than 10.0 cms to allow accessibility for welding. The corresponding cross section will carry the remaining beam tension flange force.

(ii) The diagonal portion (2-3) must be designed to resist the remaining shearing force if the column web thickness is not sufficient. The induced diagonal force will follow equation (h) with the only difference that for the MORRIS type the induced diagonal force is tension.

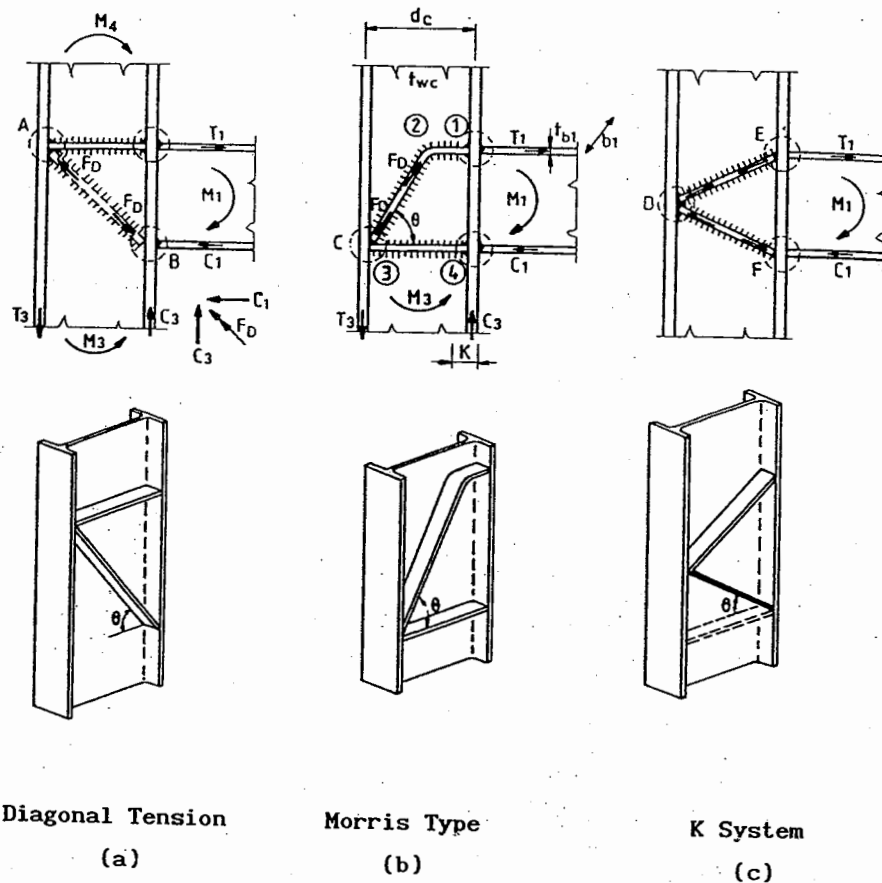


Fig. 5.6

Hence the dimensions b_{st} and t_{st} are computed making use of relation (i), F_c is replaced by the allowable tensile stress F_t .

(iii) It is to be observed that the MORRIS stiffener is a continuous stiffener. Hence the horizontal portion (1-2) is to be chosen as the diagonal one (2-3).

(iv) A horizontal stiffener (3-4) opposite to the

compression beam flange is to be provided in accordance with Equation (a).

(c) K Stiffener:

If the column depth is considerably less than the beam depth, one can use two diagonal stiffeners as shown in Fig. 5.6.c. This type is named the K stiffener where the inclination with the vertical lies between 30° and 60° . The analysis and the design will follow the equilibrium conditions using the computed relations outlined in the previous paragraphs.

(d) Welding Details:

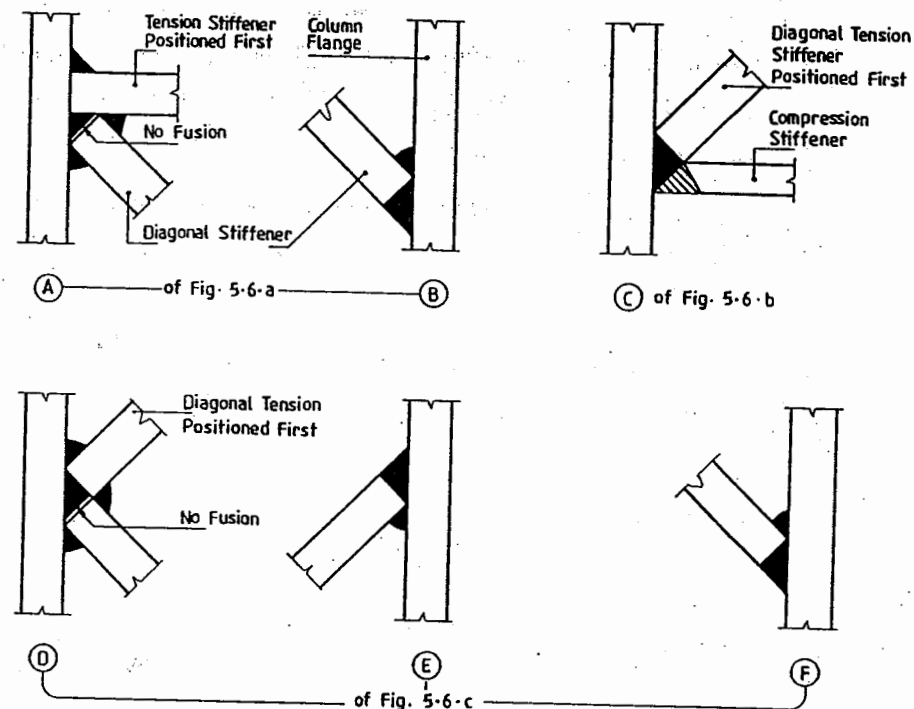


Fig. 5.7

The use of the diagonal stiffeners may lead to complications in the welding process. It is recommended to put first in position the tension stiffeners and after that the compression ones. Fig. 5.7, illustrates details of the common weld situations.

Refer to Examples 5.1 and 5.2.

5.2. Vertical Tee Stiffeners Connected Either to Column Web or to Column Flange:

Another configuration of fully welded beam-to-column connections consists to use either vertical plate stiffeners or vertical tee stiffeners as shown in Fig. 5.8.

Usually the vertical stiffeners are chosen as Tee sections. Figs. 5.8.a, b, illustrate a one way slab where beams (A) in one direction only are attached to the connection. Fig. 5.8.c shows a two way slab where beams in the two orthogonal directions (A-A) and (B-B) are attached to the connection. However both situations must consider the following four design aspects:

(a) The analysis of welds (1) attaching the stiffener to the column flange.

(b) The analysis of welds (2) attaching the stiffener Tee stem to the column web.

(c) The analysis of the vertical Tee stiffeners.

(d) The analysis of the weld lines (3) and (4) attaching the beam flange to the column flange or to the stiffener flange.

In the following sections, some assumptions are considered in order to facilitate the approach.

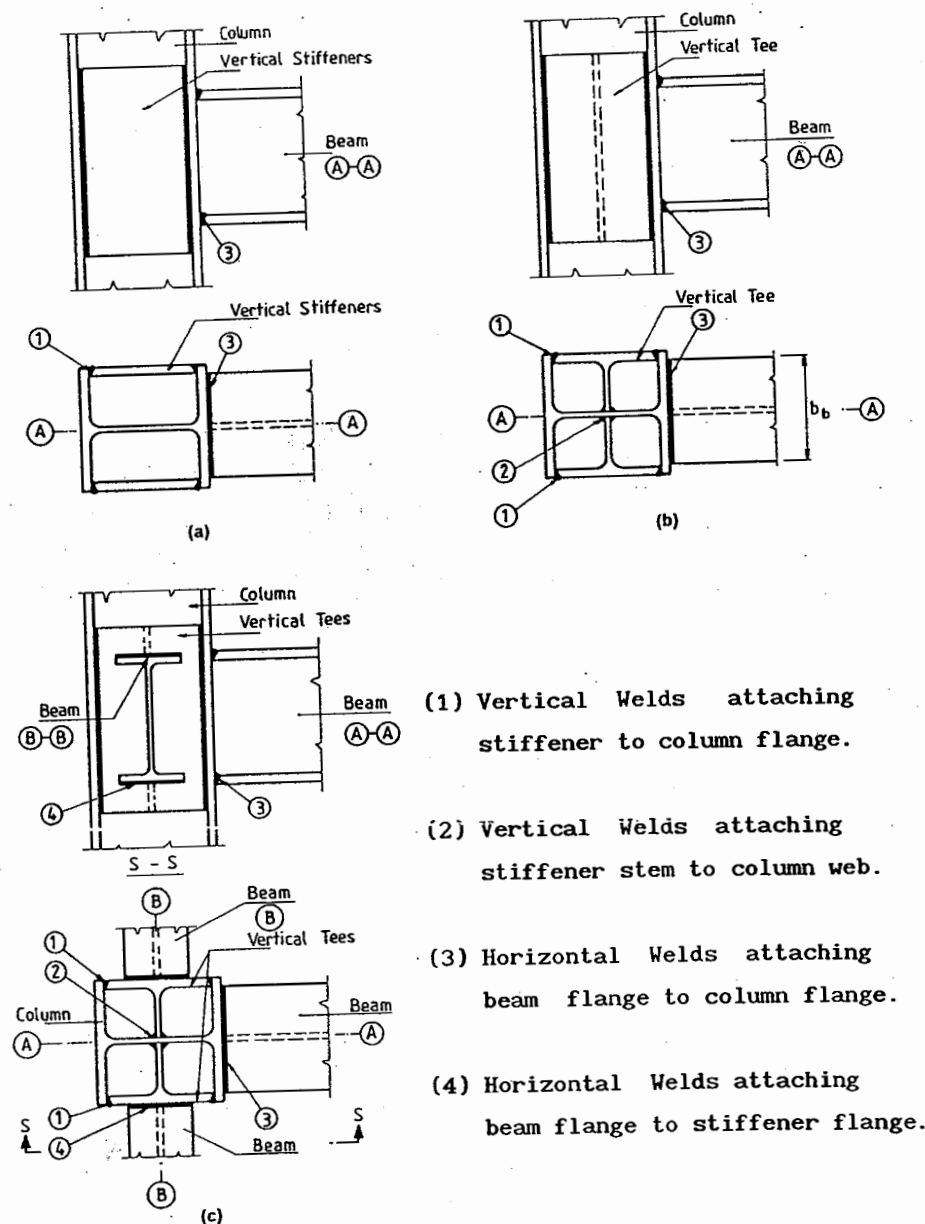


Fig. 5.8

5.2.1. Path Flow of Forces:

Fig. 5.9, illustrates the path flow of forces from one steel structural element to the other. One can replace the beam moment by two equal and opposite forces ($F = M/d_b$). These forces are transmitted to the butt weld lines connecting the beam flanges to the Tee flange. The Tee stem and the two lines of weld (1) will act as supports to the Tee flange. Experiments have shown that the force transmitted to the Tee stem is $5/8 F$ and that resisted by each line of weld (1) is $3/16 F$.

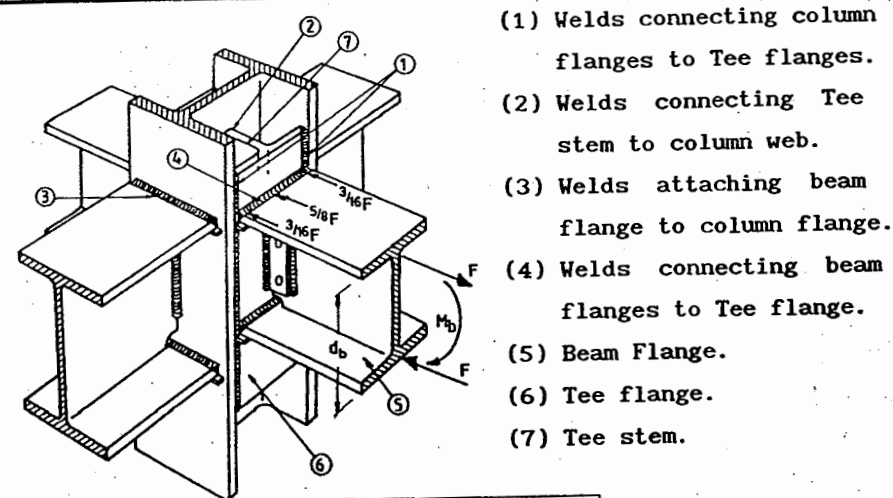
The force carried by the weld (1) will be transmitted by direct contact to the column flange. On the other hand the force resisted by the Tee stem will be transmitted to the column web via weld lines (2) behaving either in bending or in shear depending on the face of weld lines under consideration.

5.2.2. Analysis of Weld Lines Connecting the Column Flange to the Tee Flange:

A conservative approach assumes, only one third of the beam moment transmitted to each of the weld lines connecting the column flange to the Tee flange.

Fig. 5.10.a, illustrates the portions of the beam flanges neighboring the weld lines transmitting each one third of the force F (i.e. $1/3 F = M/(3d_b)$). Two configurations are possible for the choice of the weld lines connecting the Tee flange to the column flange: either a butt weld or a fillet weld as shown in Fig. 5.10.b.

Concerning the butt weld configuration, the force $1/3 F$ will induce normal compressive or tensile stresses as well as



- (1) Welds connecting column flanges to Tee flanges.
- (2) Welds connecting Tee stem to column web.
- (3) Welds attaching beam flange to column flange.
- (4) Welds connecting beam flanges to Tee flange.
- (5) Beam Flange.
- (6) Tee flange.
- (7) Tee stem.

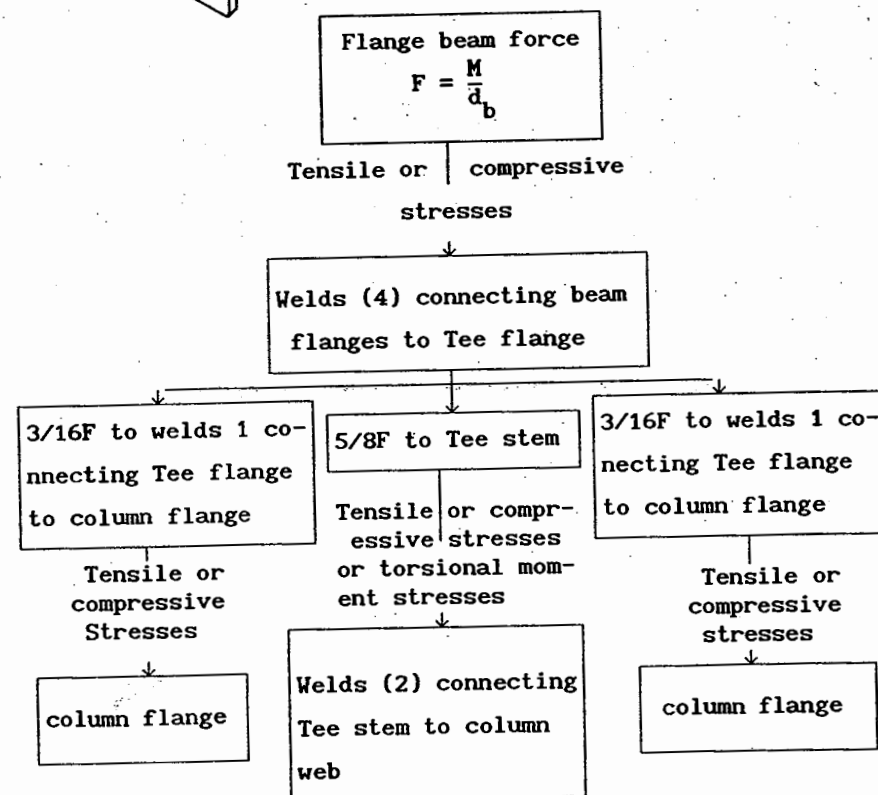


Fig. 5.9

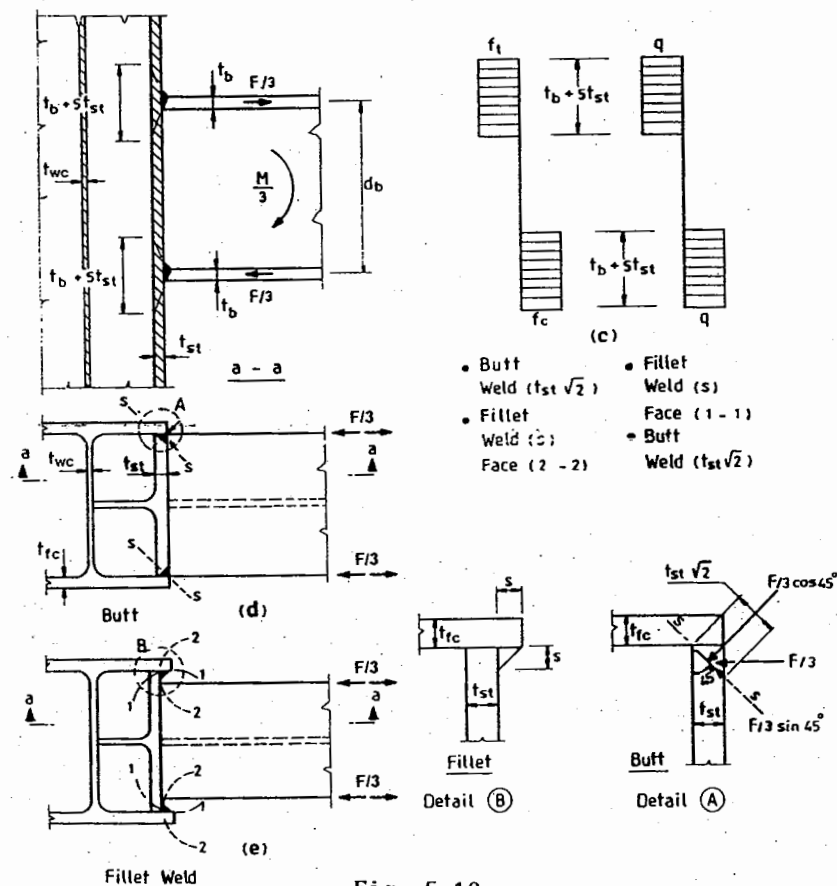
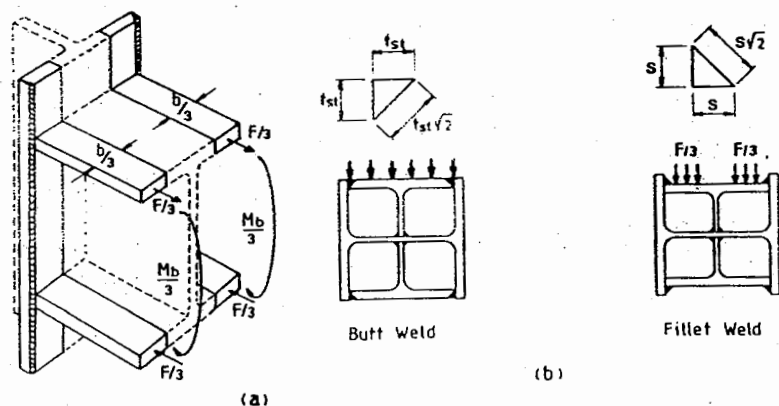


Fig. 5.10

shear stress on Section (s-s) as shown in Fig. 5.10.c. It has been tested that a spread along lines 1:2 1/2 are acceptable as illustrated by Fig. 5.10.d. Applying these assumptions (see detail (A) of Fig. 5.10), compute the following:

$$f_w = \frac{M \cos^2 45}{3 d_b (t_b + 5 t_{st}) t_{st}} \quad (a)$$

$$q_w = \frac{M \cos^2 45}{3 d_b (t_b + 5 t_{st}) t_{st}} \quad (b)$$

where:

f_w and q_w are the induced normal and shear stresses on Section (s-s). It is to be observed that the equivalent stresses are to be computed and checked against the allowable stresses.

Regarding the fillet weld configuration detail (B), the previous assumptions remain unchanged. Faces (1-1) and (2-2) will be subjected to concentric shear stresses and normal stresses respectively (Refer to Fig. 5.10.e). Hence:

$$q_{w(1-1)} = \frac{M}{3 d_b (t_b + 5 t_{st}) s} \quad (c)$$

$$f_{w(2-2)} = \frac{M}{3 d_b (t_b + 5 t_{st}) s} \quad (d)$$

The stresses of Equations (c) and (d) must not exceed the permissible stresses (i.e. $0.2 F_u$).

5.2.3. Analysis of Weld Lines Connecting the Tee Stem to the Column Web:

Once again some assumptions are necessary in order to get the design possible, otherwise a sophisticated analysis is necessary. These assumptions are:

(a) Only three quarters of the beam moment is transmitted to the Tee stem and then to the column web via the weld lines.

(b) These weld lines are assumed to be spreaded on lines inclined 1:2.5.

Making use of these assumptions and referring to Fig. 5.11 the fillet weld lines connecting the Tee stem to the web column are subjected to $3/4 M$.

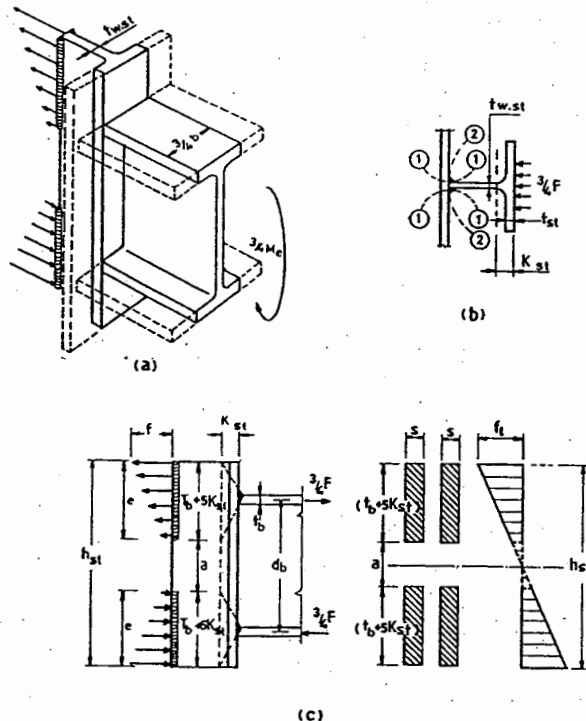


Fig. 5.11

Differentiation is to be made, if one is calculating the induced stresses on face (1-1) or on face (2-2). However the straining actions will remain the same only the stress nature will change.

Referring to Fig. 5.11.b, c, the tensile stresses f_w induced on face (2-2) Based on a unit length for the weld size S will be as follows :-

$$f_w = \frac{3 M}{4 Z} \quad \left. \begin{array}{l} \\ \text{Where: } Z = \frac{2 (h_{st}^3 - a^3) \times 2}{12 \times h_{st}} = \frac{h_{st}^3 - a^3}{3 \cdot h_{st}} \end{array} \right\} \quad (e)$$

$$\text{i.e. } f_w = \frac{9 \cdot M \cdot h_{st}}{4 (h_{st}^3 - a^3)} \quad (f)$$

While on face (1-1) the transmitted moment ($3/4 M$) will act as a torsional moment inducing shear stresses (q_w):

$$\text{i.e. } q_w = \frac{9 \cdot M \cdot h_{st}}{4 (h_{st}^3 - a^3)} \quad (g)$$

5.2.4. Analysis and Proportioning of the Tee Stiffener:

The Tee stiffener dimensions is governed by two considerations:

- (a) Strength considerations.
- (b) Practical or geometrical considerations.

(a) Strength Considerations:

The beam tension flange tends to bend the stiffener Tee flange following the same behavior as in Section 3.9.



Equation (3.10) of Section 3.9, can be utilized. This relation remains unchanged except that the applied tension force will be reduced to $3/4 F$ which means consequently that the area transmitting the tension force will be reduced also to $3/4 b_b \cdot t_b$. Hence the stiffener flange thickness must satisfy:

$$t_{st} \geq 0.4 \sqrt{(3/4) \cdot b_b \cdot t_b} \quad (h)$$

As an approximate estimate of the Tee stem thickness ($t_{w.st}$), distribution lines 1:2.5 will spread the force $3/4 F$ on a length $(t_b + 5.K_{st})$. The crippling stress relation can be used as a guide for the determination of $t_{w.st}$:

$$t_{w.st} (t_b + 5.K_{st}) \geq 3/4 \cdot b_b \cdot t_b$$

i.e.

$$t_{w.st} \geq \frac{3/4 \cdot b_b \cdot t_b}{(t_b + 5.K_{st})} \quad (i)$$

(b) Practical or Geometrical Considerations: (see Fig. 5.12)

- The stiffener Tee width b_{st} must be sufficient to reach the column flanges i.e. :

$$b_{st} \geq h_c - 2 \cdot t_{fc} \quad (j)$$

- The stiffener Tee depth (d_{st}) must be sufficient to extend from the column web till the column outer edges:

$$d_{st} = \frac{b_c - t_{wc}}{2} \quad (k)$$

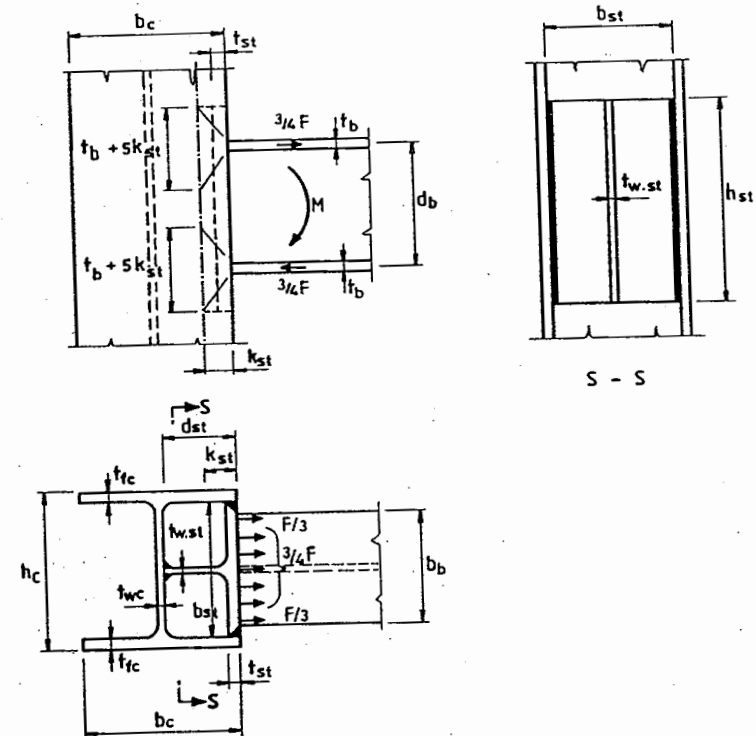


Fig. 5.12

- It will be of good estimate to choose the stem tee thickness ($t_{w.st}$) equal to the beam web thickness (t_{wb}):

$$t_{w.st} \geq t_{wb} \quad (l)$$

Refer to Example 5.3.

5.2.5 Additional Observations:

First Observation:

It is to be noticed that the relations (a), (b), ..(i) are



deduced for a beam flange width (b_b) approximately equal to the Tee flange width (b_{st}). While if the beam flange width (b_b) is significantly less than the Tee flange width (say, more than 4.0 cms), one must assume that the stem portion of the Tee stiffener will carry the entire moment (M_b) and consequently relations (f), (g), (h) and (i) will be as follows:

$$\left. \begin{aligned} f_w &= \frac{3.M.h_{st}}{(h_{st}^3 - a^3)} ; & q_w &= \frac{3.M.h_{st}}{(h_{st}^3 - a^3)} \\ t_{st} &\geq 0.4 \sqrt{b_b \cdot t_b} ; & t_{w.st} &\geq \frac{b_b \cdot t_b}{t_b + 5.K_{st}} \end{aligned} \right\} \quad (m)$$

Second Observation:

If two orthogonal systems of beams are connected to the column, and if a vertical Tee stiffener is attached to the column web as shown in Fig. 5.13, the analysis becomes a combination of the assumptions previously taken in Section 3.9 and those considered in the previous paragraphs. The analysis of this configuration can be proceeded as follows:

(a) Beams (1) and (2) will transmit their corresponding moments M_1 and M_2 to the column flange by the forces F_1 and F_2 ($F_1 = M_1/d_1$ and $F_2 = M_2/d_2$). The flanges of the Tee stiffeners and the column web will act as supports to the column flange. The assumptions of the preceding paragraphs are always valid. This means that the column web will carry $(3/4) F_1$ while the flanges of the Tee stiffener will carry $F_1/3$ as shown in Fig. 5.13.a.

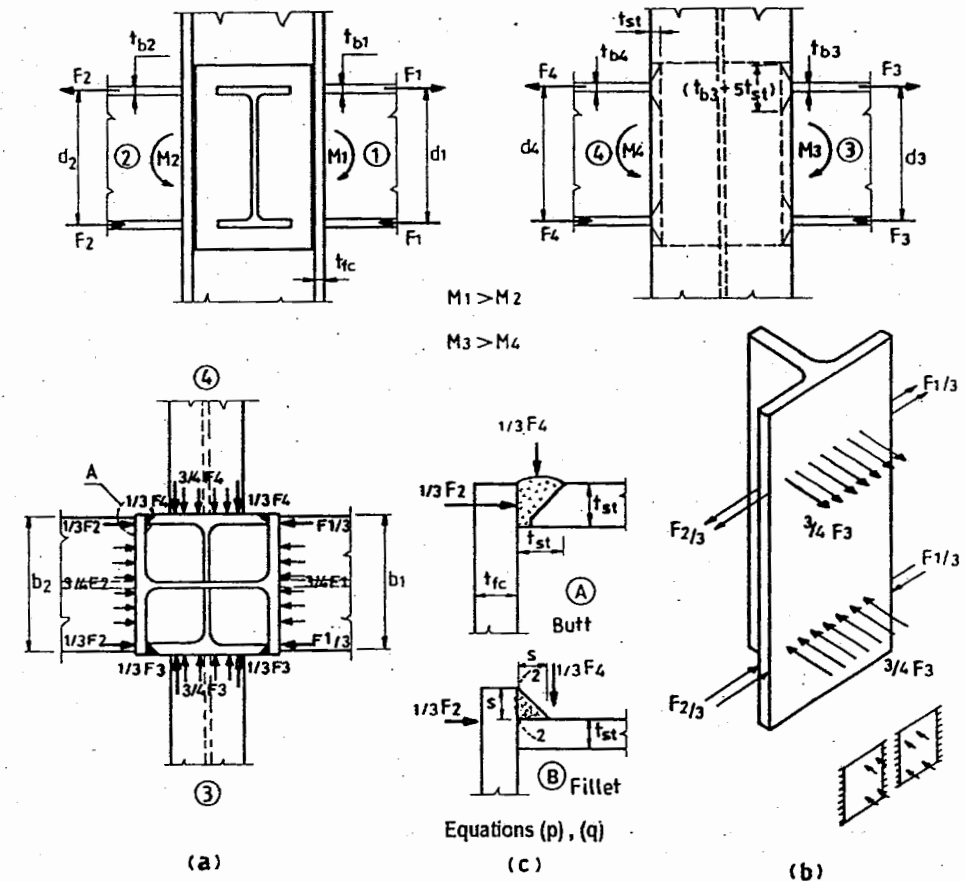


Fig. 5.13

Based on these assumptions the following relations are computed:

- Crippling of the column web:

$$t_{wc} \geq \frac{3.b_1 \cdot t_{b1}}{4(t_{b1} + 5.K)} \quad (n)$$

- The bending of the column flange:

$$t_{fc} \geq 0.4 \sqrt{(3/4) b_1 \cdot t_{b1}} \quad (o)$$

- The flange of the Tee stiffener is designed to carry $F_1/3$ as illustrated in Fig. 5.13.b.

(b) Beams (3) and (4) will transmit their corresponding moments exactly as in section 5.1.2. The flange Tee thickness is to be checked against bending. i.e $t_{st} = 0.4 \sqrt{(3/4) b_3 \cdot t_{b3}}$ Where b_3 and t_{b3} are the breadth and thickness of beam flange (3) respectively.

(c) As shown in Fig. 5.13.b the Tee stiffener will be subjected to bending due to $3/4 F_3$. In addition the forces $F_1/3$ and $F_2/3$ will induce additional normal stresses. An exact analysis is somewhat complicated, one must use a plate model with springs of appropriate rigidities along the vertical groove weld lines. The Tee flange is considered to be relatively stiff, and thus for simplicity the additional stresses caused by the forces $F_1/3$ and $F_2/3$ can be neglected. Their magnitude will be nill if the column web thickness is sufficient to respond to the crippling phenomena based on the whole beam cross sectional area ($b_1 \cdot t_{b1}$) instead of $(3/4) b_1 \cdot t_{b1}$.

(d) Regarding the fillet weld lines attaching the Tee flange to the column flange (Detail (B) of Fig. 5.13.c) the forces $F_1/3$ and $F_2/3$ will induce on face 2-2 tensile or compressivestresses while the forces $F_4/3$ and $F_3/3$ will induce shear stresses. The equivalent stresses are to be computed.

As has been mentioned before the effective weld length can be assumed equal to $(t_{b3} + 5 \cdot t_{st})$ or $(t_{b4} + 5 \cdot t_{st})$.

Thus:

$$f_{\perp} = \frac{F_2/3}{s(t_{b4} + 5 \cdot t_{st})} \quad (p)$$

and

$$q_{//} = \frac{F_4/3}{s(t_{b4} + 5 \cdot t_{st})} \quad (q)$$

It is of good practice to choose the size $s = t_{st} = t_{fc}$

Using Equations (p) and (q), deduce the equivalent stresses.

For butt weld detail (A) deduce the equivalent stresses the following equations (p) and (q) :

$$f_w = \frac{1/3 (F_2 + F_4) \cos^2 45}{t_{st}(t_{b4} + 5 \cdot t_{st})} \quad ('p)$$

$$q_w = \frac{1/3 (F_2 + F_4) \cos^2 45}{t_{st}(t_{b4} + 5 \cdot t_{st})} \quad ('q)$$

Third Observation:

The effect of the applied shearing force on the behaviour of the different steel elements was neglected.

Refer to Examples 5.4.

5.3. Welded Vertical Plate Stiffeners Configurations:

Two vertical plate stiffeners configurations are analysed according to the following:

5.3.1 Configuration "A":

Fig. 5.14.a, illustrates the different components. The applied beam moment (M) is replaced by two equal and opposite forces ($F = M/d_b$).

Proceeding as in section 5.2.4 the following assumptions can be utilized:

(a) The beam flange compression force $(3/4)F$ is transmitted to the column web via the column web length $(t_b + 5K)$ as illustrated in Fig. 5.14.b; hence:

$$t_{wc} \geq \frac{3 b_b \cdot t_b}{4(t_b + 5K)}$$

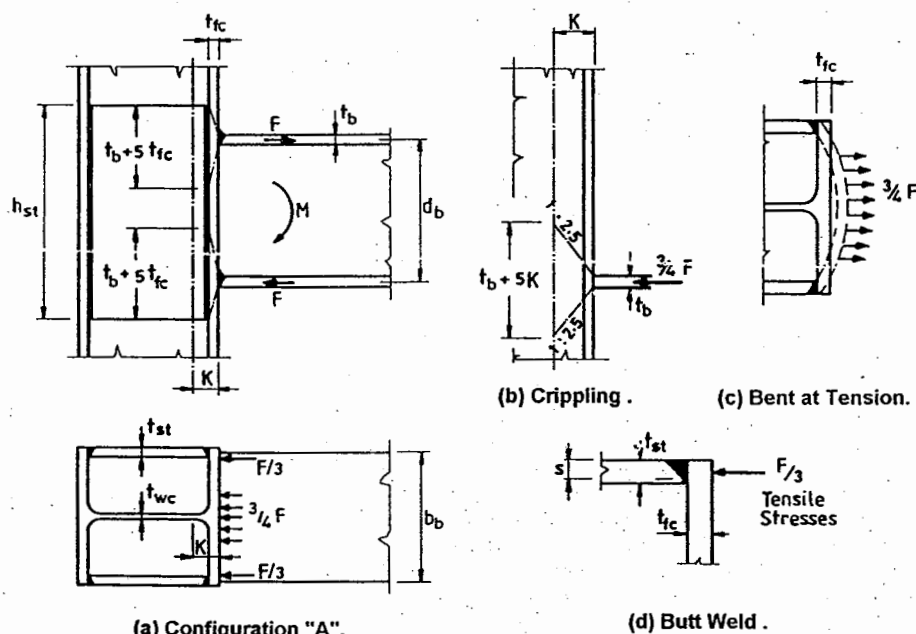


Fig. 5.14

(b) The beam flange compression force $F/3$ will be transmitted to the butt weld lines via a length $(t_b + 5t_{fc})$, thus inducing compressive and shear stresses on the welds (see Fig. 5.14.d). This means that:

$$q_w = f_w = \frac{F \cos^2 45}{3(t_b + 5t_{fc}) \cdot s} \quad (b)$$

(c) The beam flange tension force $3/4 F$ is transmitted to the column flange trying to pull it out as shown in Fig. 5.14.c; hence the column flange thickness can be computed according to the following relation:

$$t_{fc} \geq 0.4 \sqrt{(3/4) b_b \cdot t_b} \quad (c)$$

(d) The stiffener thickness (t_{st}) can be chosen equal to the beam flange thickness (t_b). The normal stresses are computed using the following relation:

$$f_t = f_c = \frac{F}{3 t_{st} (t_b + 5t_{fc})} \quad (d)$$

The geometrical considerations previously outlined in Section 5.2.4 are always valid.

Refer to Example 5.5.

5.3.2. Configuration "B":

Fig. 5.15.a, illustrates the different steel components, for this configuration. The different aspects regarding the

connection behaviour are as given below:

If an exact solution is required a three dimensional finite elements analysis is necessary, while for simplicity the plate model shown in Fig. 5.15.b is sufficient.

For design purposes and due to lack of advanced studies, one can proceed using the following assumptions:

(a) At the vicinity of the beam tension flange Fig. 5.15.c illustrates the expected deformed shape of the stiffener element. Only the tensile force (F) is taken into consideration. In reality the compressive force at the lower flange of the beam is applied simultaneously. However if the tension force is taken

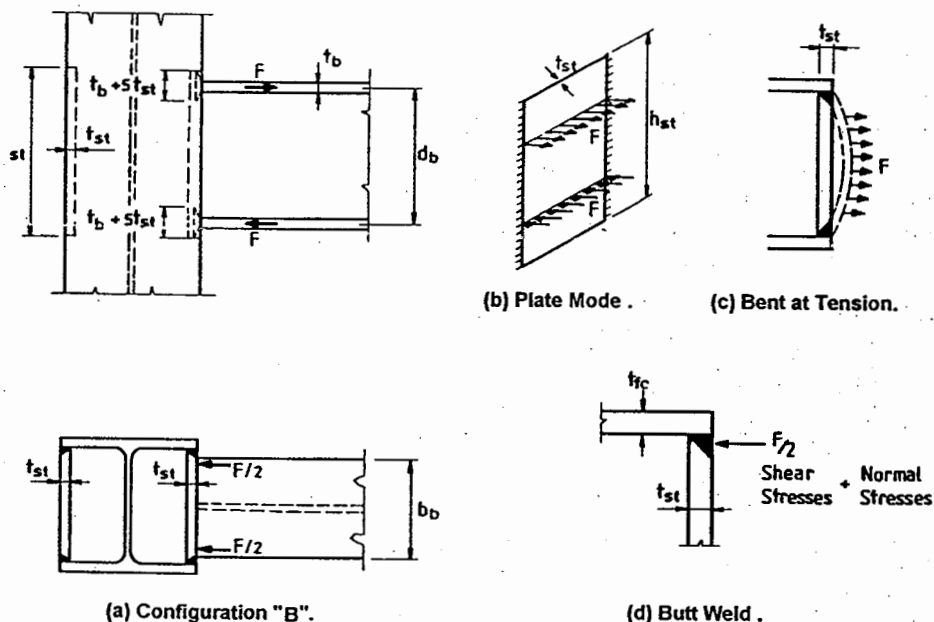


Fig. 5.15

acting separately, the results will be conservative and the following equation can be utilized:

$$t_{st} \geq 0.4 \sqrt{b_b \cdot t_b} \quad (e)$$

(b) The force (F) will be transmitted to the column flanges via the butt weld lines as shown in Fig. 5.15.d, thus causing normal and shear stresses.

It is to be noticed that the induced stresses will be relatively high. Hence it is not recommended to use the vertical plate stiffeners configuration, the horizontal stiffeners arrangements will satisfy the weld stress requirements.

Refer to Example 5.5.

5.4. Connecting Plates Welded to Either the Column Flange or the Column Web:

This type of connections consists to use a connecting plate either fillet or butt welded to the beam flanges. This connecting plate is then welded to the column in order to transmit the beam moment to the column. Regarding the beam reaction, one of the configurations previously outlined in Chapter (4) can be utilized (framing angles, brackets ... etc).

In the following sections the common configurations are analysed. The transmission of forces from one steel element to the other and the dimensioning are simple. To avoid repetition of the analysis refer to the previous sections.

4.1. Connecting Plates Welded to the Column Flange:

Configuration(A):

Fig. 5.16.a, illustrates a beam-to-column connection with top connecting plate. The transmission of forces and the design of the steel elements can be proceeded as follows:

(a) The reaction of the beam (R_b) is transmitted via fillet weld lines (3) to the leg of the framing angle (2). While the other angle leg transmits this reaction to the column flange via the fillet welds (4). The design and dimensioning is as previously outlined in Section 4.1.1.

(b) The beam moment is replaced by two equal and opposite forces $T = C = M/d_b$. The tensile force (T) will be transmitted via the fillet weld lines (7) to the upper connecting plate, subjected to either tension or shear. Hence :-

$$\text{For Fillet welds(7): } q_w = \frac{T}{(w + 2L_1).s} \quad (a)$$

$$\text{For Connecting plate(6): } f_t = \frac{T}{w.t_c} \quad (b)$$

here:

w, t_c = the width and thickness of the connecting plate respectively

L_1 = loaded length of the connecting plate.

Fig. 5.16.b, shows different configurations for the choice of the connecting plate. The breadth W_1 at the right edge is to be chosen less than the beam breadth (b_b) by 3.0 to 5.0 mm, while the left edge breadth W_2 is to be chosen according to the condition $b \geq W_2 \geq (1 \frac{2}{3} W_1)$.

In order to determine the required safe thickness (t_c) apply in Equation (b) using the allowable tensile stresses. The

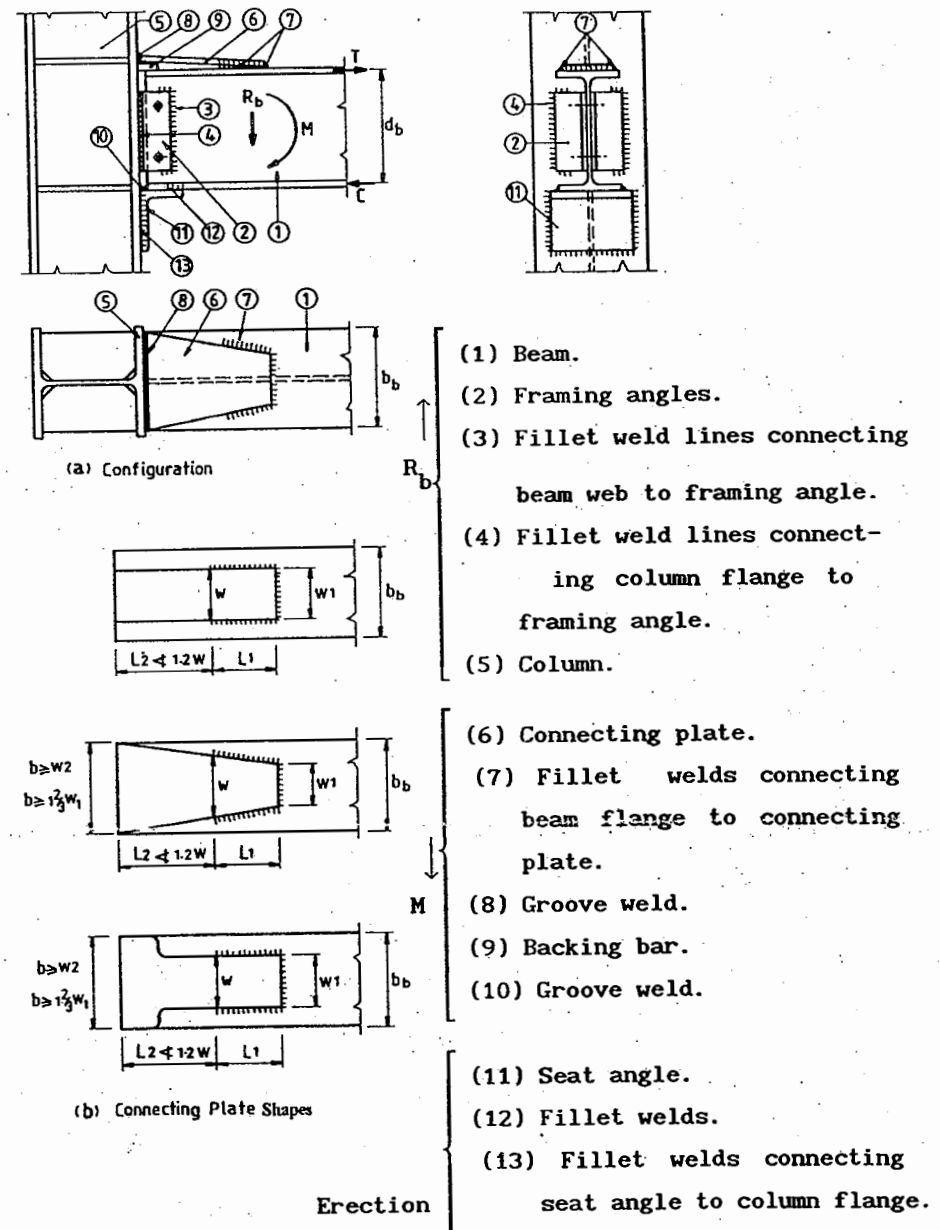


Fig. 5.16 Configuration (A)



unloaded length L_2 of the connecting plate must be equal at least to 1.2 times the breadth W_1 .

It is to be observed that the connecting plate is shop fillet welded to the beam flange.

The field groove weld (8) will transmit the tensile force T to the column. A backing bar (9) of 4 mm thick is to be utilized.

(c) The beam flange compression force is transmitted directly to the column flange via the groove butt weld (10).

(d) The reaction being already transmitted, the seat angle (11) will serve as a backing bar to the groove weld (10). Dimensions of the seat angle are to be chosen of minimum size and are attached to the column flange by the shop fillet weld (13).

Refer to Example 5.6.

Configuration (B):

Fig. 5.17, illustrates a second beam-to-column configuration using connecting plates.

The transmission of the beam moment is exactly as configuration (A). The analysis and design of the connecting plate, the corresponding fillet and groove welds remain unchanged.

The reaction R_b will be transmitted to the column flange via a bracket. This reaction is transmitted by bearing to the top plate (2) of the bracket and then to the column flange via the vertical stiffener (3) and the shop fillet weld lines (4).

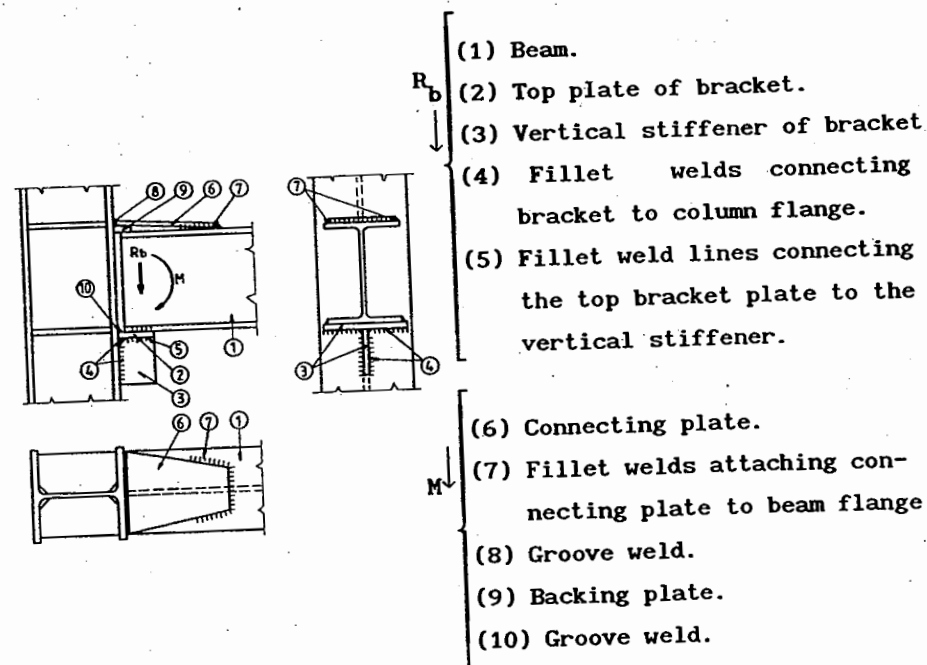


Fig. 5.17. Configuration (B)

For the analysis and the design of the bracket steel components refer to Chapter 4.

Refer to example 5.7.

Configuration (c):

Fig. 5.18, illustrates configuration (c).

The transmission of the vertical beam reaction R_b follows configuration (B) of Fig. 5.17. The tension force T is transmitted to the column flange as in configuration (A) of Fig. 5.16.

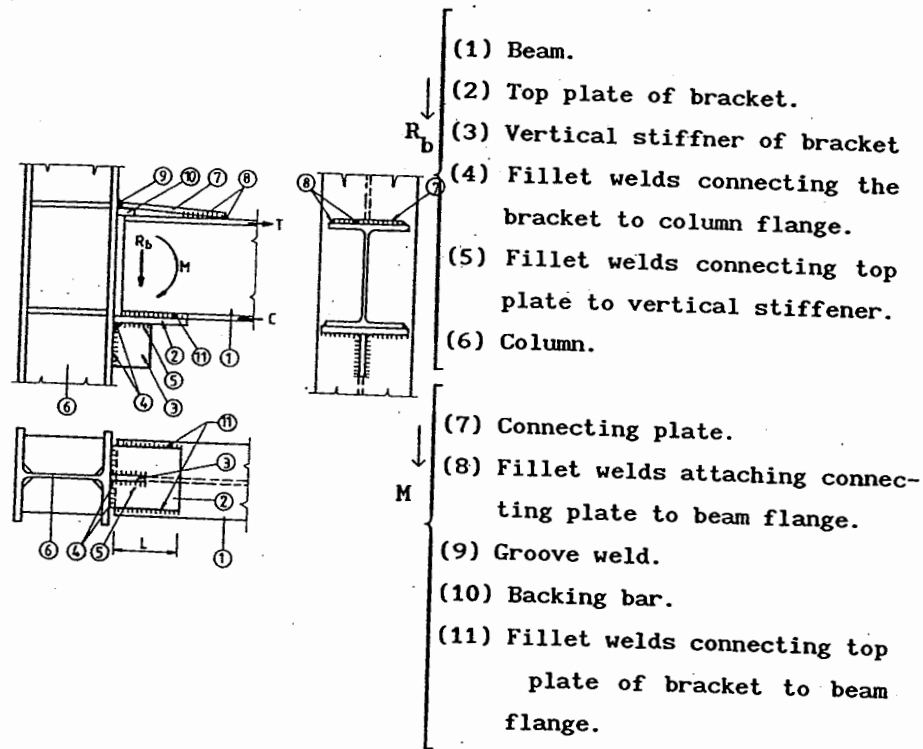


Fig. 5.18. Configuration (c)

While the force C is transmitted to the top plate of the bracket (2) by means of the field fillet weld lines (11) carrying the following weld shear stresses:

$$q_{//} = \frac{C}{2 \cdot L \cdot s} \quad (c)$$

the bracket top plate is utilized as a connecting plate.

Refer to example 5.8.



5.4.2. Connecting Plates Welded to the Column Web:

Configuration (D): (Fig. 5.19)

The beam reaction R_b is transmitted by bearing to the top plate of the bracket (2), then to the vertical stiffener (3), then to the shop fillet weld lines (4) and finally to the column flange. For the analysis of the bracket refer to Chapter 4.

The tensile force T is transmitted to the upper connecting plate (9) via the field groove weld (7) where a backing bar (8) is added. This tensile force T is then transmitted to the column flange via the shop fillet weld lines (10). Referring to Section (b-b) and (d-d) of Fig. 5.19, we notice that the connecting plate (9) can be welded to the column flange and column web along the upper and lower faces of welds (10). The following weld shear and tensile stresses are induced:

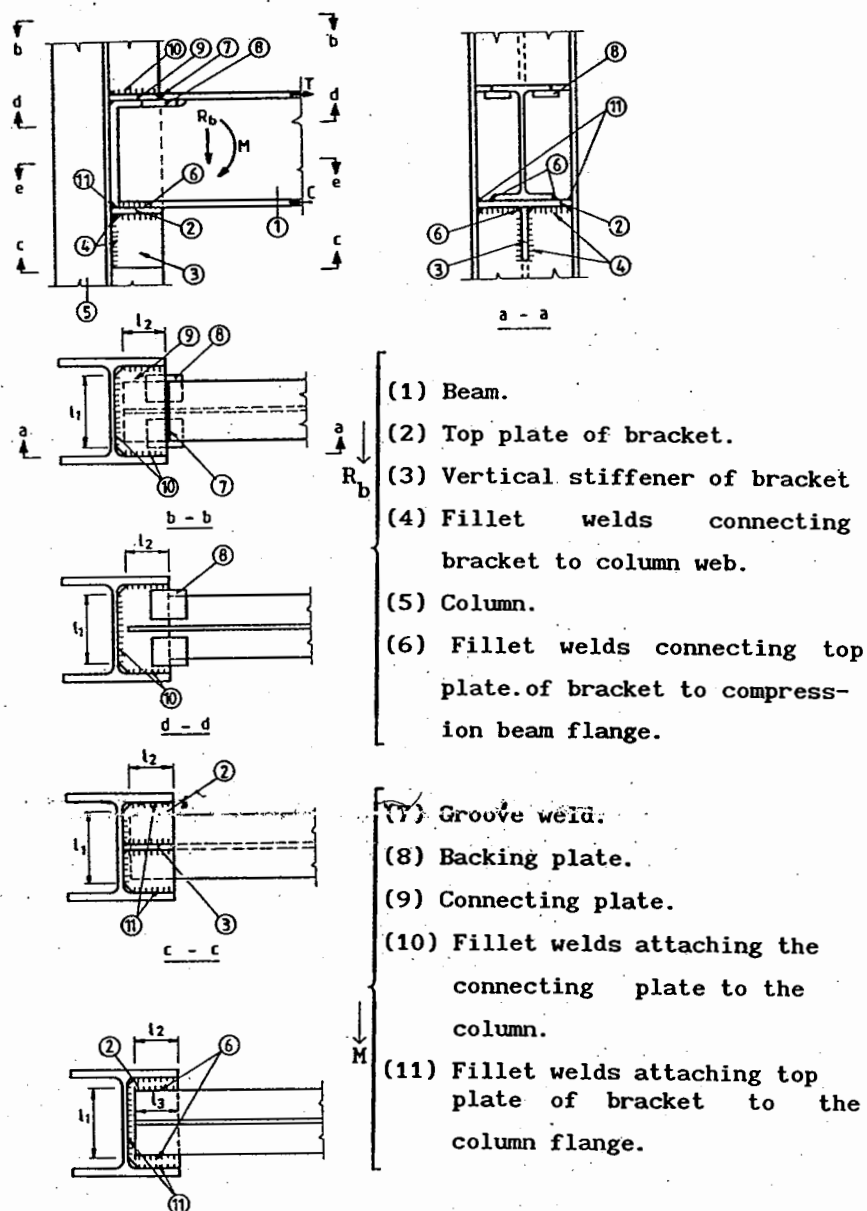
$$q_w = \frac{T}{2(L_1 + 2L_2) \cdot s} \quad (a)$$

It is to be noticed that if the upper face of welds (10) is sufficient, the weld lines on the lower face can be omitted.

The compression force C is transmitted from the compression beam flange to the top plate of the bracket (2) via the field weld lines (6). The following shear stresses ($q_{//}$) are induced: (refer to the Section e-e of Fig. 5.19)

$$q_{//} = \frac{C}{2 \cdot L_3 \cdot s} \quad (b)$$

If the top bracket plate (2) thickness is the same as the beam flange, the induced compressive stresses will be consequently within the allowable limits.



e - Fig. 5.19. Configuration (D)

The force C is then transmitted to the shop fillet weld lines (11). Referring to Section (e-e) of Fig. 5.19 we notice that the effective weld lengths (11) are $L_1 + 2L_2$ (Section e-e). R_b will be transmitted by weld lines (4) to the column as shown by section (e-e). Thus the induced shear and tensile stresses in the fillet weld lines (11) will follow:

$$q_w = \frac{C}{(2L_2 + L_1) \cdot s} \quad (c)$$

Once again if the weld lines (11) of the top plate upper face are not sufficient, we can add two weld lines of length L_2 on the lower face of the top plate. If the upper connecting plate and the bracket top plate thicknesses are chosen equal to the beam flange thickness, the induced stresses due to the forces T and C will be consequently within the allowable limits.

Refer to Example 5.9.

Configuration (E): (Fig. 5.20)

The reaction R_b is transmitted exactly as Configuration (D) of Fig. 5.19.

The compressive force C is also transmitted to the column as Configuration (D).

The force T is transmitted from the beam upper flange to the two upper connecting plates (7) inducing tensile stresses (f_t):

$$f_t = \frac{T}{2 \cdot w \cdot t_c} \quad (d)$$

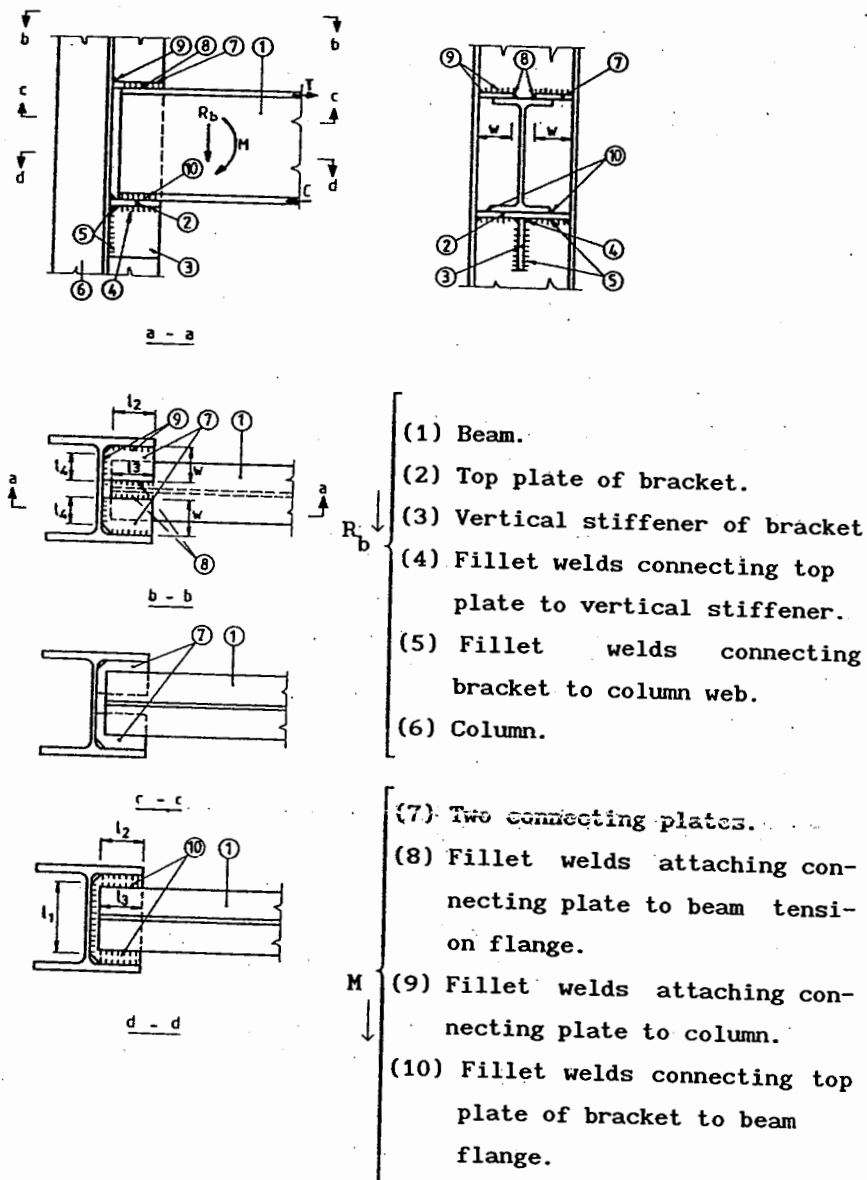


Fig. 5.20. Configuration (E)



where t_c = the thickness of the upper connecting plates.

w = the width of each of the two upper connecting plate.

This tensile force is transmitted to the the plate (7) via the field fillet weld lines (8) and then to the column via welds (9). Referring to Section (b-b) of Fig. 5.20, we can compute the induced shear stresses in fillet welds (8) and (9) as follows:

$$q_{w(8)} = \frac{T}{2(L_2 + L_4)} \quad ; \quad q_{w(9)} = \frac{T}{2L_3} \quad (e)$$

Refer to Example 5.10.

Configuration (F) : (Fig. 5.21):

The beam reaction R_b is transmitted to the cleat web plate (2) via the shop fillet weld lines (3), then to the column web via the field fillet weld lines (4). For the analysis refer to chapter 4.

The tension force T will be transmitted to the column in a similar way as in Configuration (D) of Fig. 5.19.

On the other hand, the compressive force C will be transmitted to the lower connecting plate (10) via the shop groove weld (9), then to the column via the field fillet weld lines (11). To be noticed that the backing bars (12) are needed for the groove welds (6) and (9).

Once again if the thickness of the top and bottom connecting plates are chosen equal to the beam flange thickness, hence the induced stresses will be within the allowable limits.

Refer to Example 5.11.

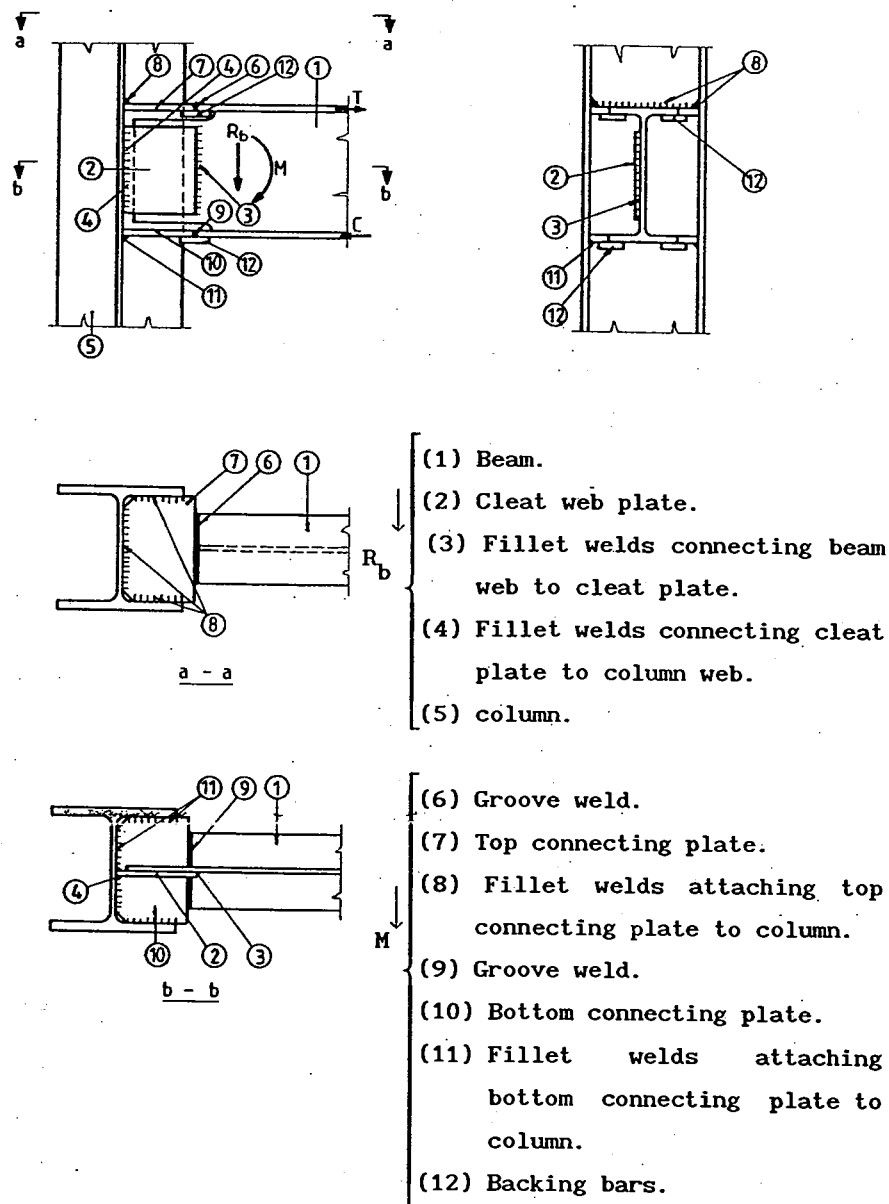


Fig. 5.21. Configuration (F)

5.5. Different Comments:

One can observe that in Section 5.1. the distribution of compression forces was assumed spreaded on lines sloping 1:2.5. On the other hand the bending stresses on the column flange was deduced by assimilating part of it as a plate subjected to line loads with assumed end conditions.

While in Sections 5.2 and 5.3 the analysis of the vertical Tee-stub stiffener was based on some experimental results concluding that the Tee-stem is carrying 5/8 of the applied beam flange force F while each of the two edge tips of the Tee-stub flange will carry the remaining force $3/16 F$.

Unfortunately for design considerations doubt concerning test results has forced the designers to consider a force equal to $3/4 F$ carried by the Tee-stub stem and a force $F/3$ carried by each edge tip of the Tee-stub flange.

This means that the steel structural elements are designed to carry a total force ($F = 3/4 F + 2/3 \times F = 1.4 F$) which is greater than the applied beam force ($F = \frac{M}{d} = T = C$).

This situation seems to be somewhat in contradiction, but the main reason is the lack of exact analyses which can cover the different configurations previously outlined.

In the Structural Steel Division of the Faculty of Engineering of Cairo University, a research study (12) has been conducted in 1991 where part of it has concerned the welded beam-to-column connections. The main objectives were:

(a) To determine the real behaviour as well as the state of stresses of the different steel elements comprising the



beam-to-column connection (i.e. the beam flanges, the column flange, the column web and the stiffeners).

(b) To check the existing available design equations for the web column crippling, the tensile bending stresses of the column flange and the stiffener cross section.

For the analysis, the finite element approach has been utilized where a plate bending element was chosen to represent the flanges and the web. While for the representation of welds, linkage elements of the spring type were utilized.

The stiffness values of the butt weld in the different directions were chosen such that the relative displacement between the two connected nodes is nearly zero where the strength of a butt weld is considered as the parent material. Fillet welds being weaker than butt welds, the relative displacement between nodes joined by fillet welds will be larger. The determination of the fillet weld stiffness was based on experimental test results conducted in 1983 in the Structural Steel Division of Cairo University (13) where a correlation was made between load-deflection curves for different fillet weld sizes

Three beam-to-column connections have been analyzed. The beam cross section was maintained constant (i.e. B.F.I.30), while the column cross section was varying (i.e. B.F.I.30, 42 1/2 and 50). Horizontal stiffeners at beam compression and tension flanges were added, their cross sections corresponded to the allowable normal stresses as specified by the E.C.P. of steel structures. A fourth beam-to-column connection was chosen according to the actual design rules concerning the stiffeners. This fourth configuration was necessary for comparison.



Fig. 5.22, illustrates the normal stress results at column fillet depth for tension and compression beam flange sides. One can observe that:

(a) The stress distribution in both tension and compression zones is not uniform, the shape is more or less triangular rather than rectangular.

(b) The peak stress value for the majority of the cases exceeds the allowable stress limit, but is always less than the yield stress.

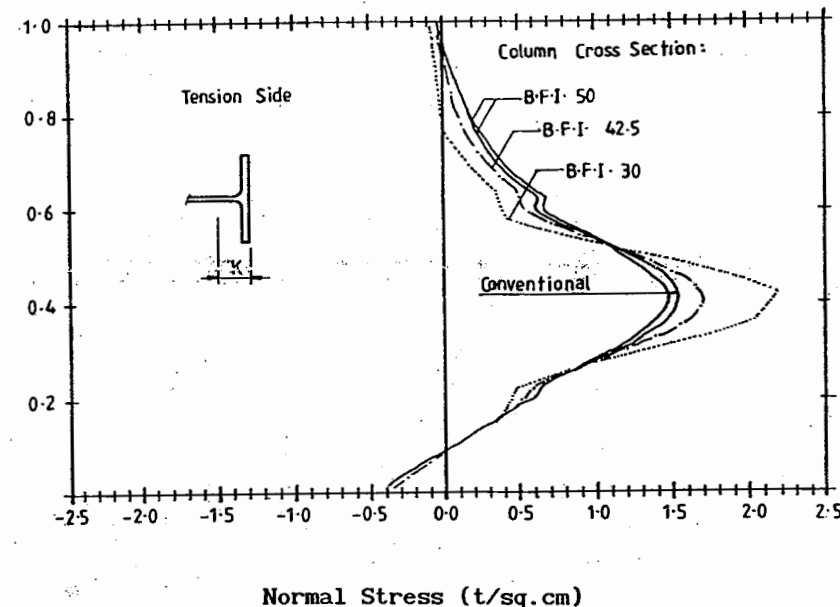


Fig. 5.22.a Horizontal Normal Stresses At Column Fillet Depth

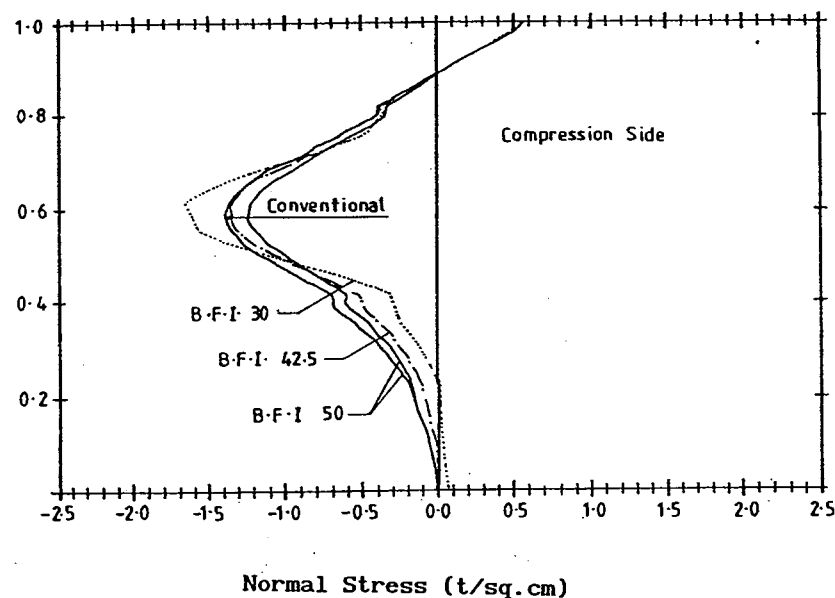


Fig. 5.22. Horizontal Normal Stresses At Column Fillet Depth

N.B. Regarding the results deduced and illustrated in figures 5.22 and 5.23 the thickness of the stiffeners at neighbouring of the tension and compression beam flanges was 1.4 cms respectively .

* Tension stiffener thickness = 1.4 cms

* Compression stiffener thickness = 1.6 cms

(c) The stress dispersion in the column web is different from the proposed value. The inclination was varying from 1:2.13 to 1:2.83.

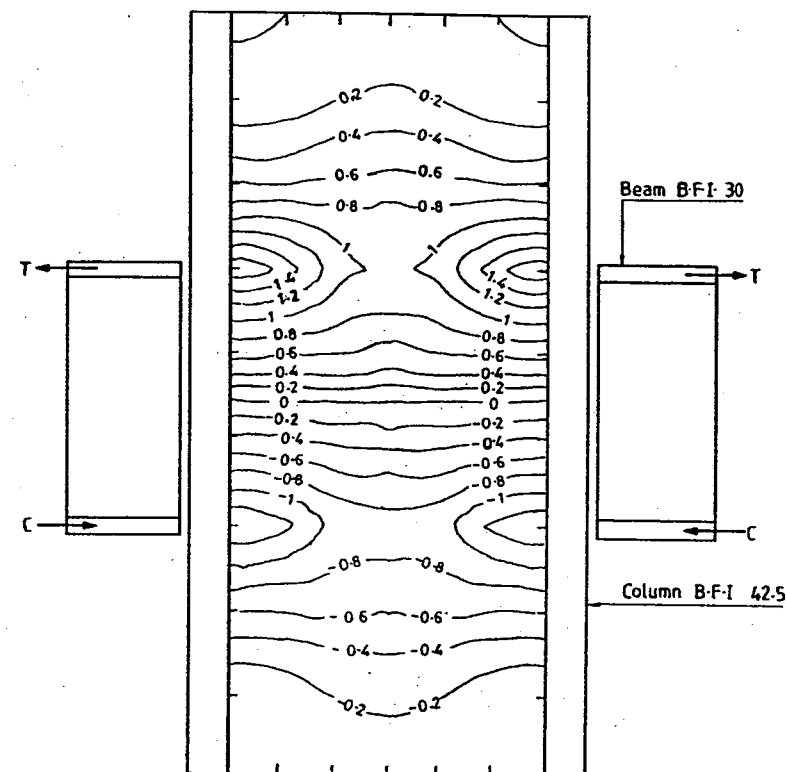


Fig. 5.23. Principal Normal Stress in Column Web with Stiffeners

(d) The length along which stresses are spreaded was greater compared to that calculated using the conventional inclination 1:2.5.

Fig. 5.23 shows the distribution of normal principal stresses in the column web. A stress concentration at the neighborhood of the beam flanges can be observed. This concentration is more pronounced at tension than at compression region.

This can be attributed to the fact that the tension is applied via the fillet welds while the compression is assumed to be transmitted by direct contact via the compression beam flange.

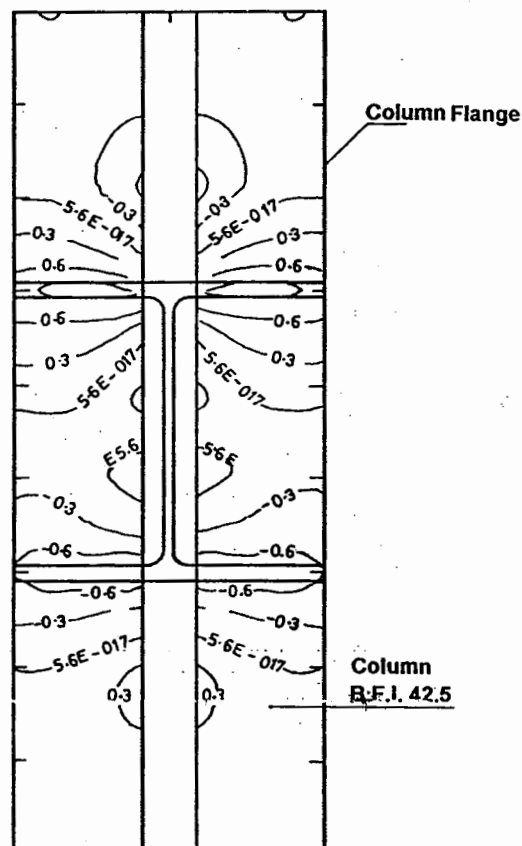


Fig. 5.24. Principal Normal Stresses
in Column Flange

Fig. 5.24 illustrates the contour lines of the principal stresses in the column flange. Stress concentrations are observed around the beam flanges, being higher at the vicinity of the beam tension flange. The obtained stresses have proved to be within the allowable stress limit.

Concerning the stiffeners, the study has shown that the resulting stresses, for all the configurations, were exceeding the allowable limits by about 30% which means that the stiffener cross section must be increased.

The aim of the above discussion was to notify the reader that we may use a similar mathematical model for the welded Tee-stub vertical stiffener configuration in order to obtain more representative design rules.

Group (II) Bolted beam - to - column Connections :-

This group of connections have been analysed in Chapter 2 . The corresponding sections as well as the example are as mentioned in the following paragraphs :-

(a) Bolted Tee-Stub moment Connections :-

Refer to Section 2.8.2.

Refer to Example 2.6.

(b) Bolted End-Plate moment Connections (flushed, extended and haunched) :-

The Flushed End Plate

Refer to Section 2.8.3.1.

Refer to Example 2.7.

The Extended End Plate

Refer to Section 2.8.3.2

Refer to Examples 2.8 and 2.9.

The Haunched End Plate

Refer to Section 2.8.3.3

Refer to Examples 2.10, 2.11 and 5.14.

(c) Bolted Seat- Angle moment Connections :-

Refer to Example 5.12.

(d) Connecting plate moment Connections :-

Refer to Example 5.13.

Group (III) Welded knee connections :-

5.6. Welded Knee Connections:

The corner beam-to-column connection is frequently named a knee connection. These connections are either field welded or bolted. In the following sections the analysis of the different types of welded knee connections as well as their behaviour is presented.

The following sections represent a great importance for the designers. The discussed knee connections (either bolted or welded) are utilized in a largest scale and represent more than 50 percent of structural steel connections.

5.6.1. Different Configurations:

Fig. 5.25 illustrates the main types of connections which can be classified as follows:

(a) The square corner connection, mainly used for beam and column weld sections. As the critical moment for a gable frame occurs generally at the eaves, one can conclude that the rafters (beams) will not be efficient, the steel at the vicinity of the apex is always subjected to low stresses. For this type of connections the fabrication cost is minimum while the weight of steel is excessive. (see Fig. 5.25.a)

(b) The square corner with bracket of Fig. 5.25.b is an improvement of the square type where a bracket (haunch) is added to the corner in order to withstand the critical moment at the eave. This haunch may be cut from the beam rolled section. Inclination of the haunch flange goes from 1:2 till 1:5 depending on the geometrical dimensions and the bending moment distribution.

(c) The tapered haunch connection illustrated in Fig. 5.25.c is composed of a haunched built up section which allows to overcome the critical eave moment. Beams and columns may be designed either rolled for common spans or built up for large spans. This type of connections will decrease the structural steel weight while the fabrication cost is somewhat elevated.

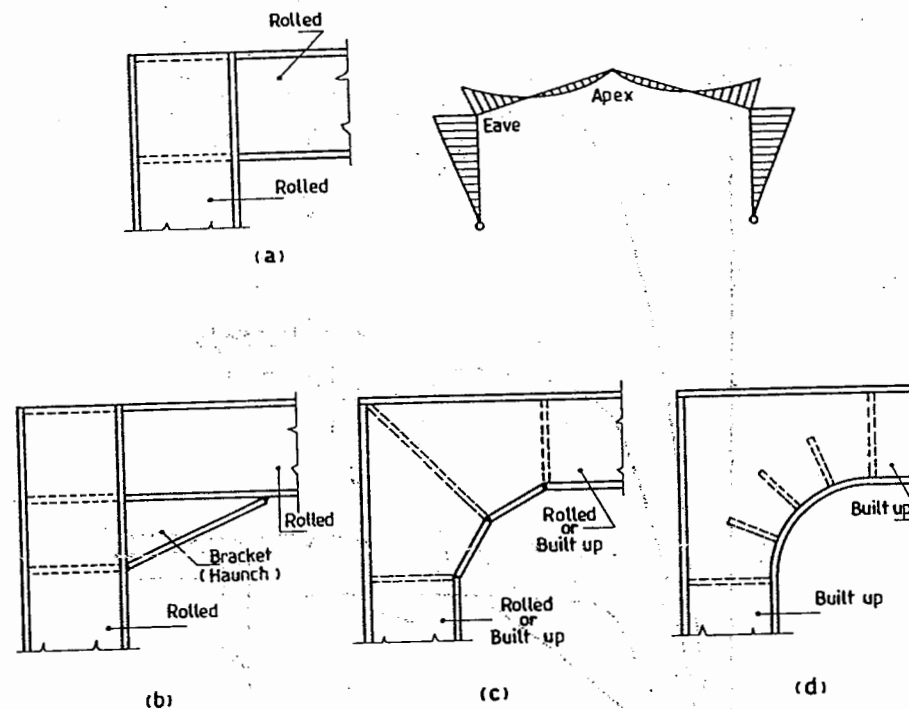


Fig. 5.25

(d) The curved haunch connection shown in Fig. 5.25.d is mainly utilized for large spans where the beam and the column sections are chosen built up. The fabrication cost is elevated. This type can resist higher moments if compared to the previous types.

5.6.2. The Moment-Rotation Diagram:

As our concern is to design a rigid connection, one can ask: What is the type of connections presented in Fig. 5.25 that will respond close to an ideal rigid connection?

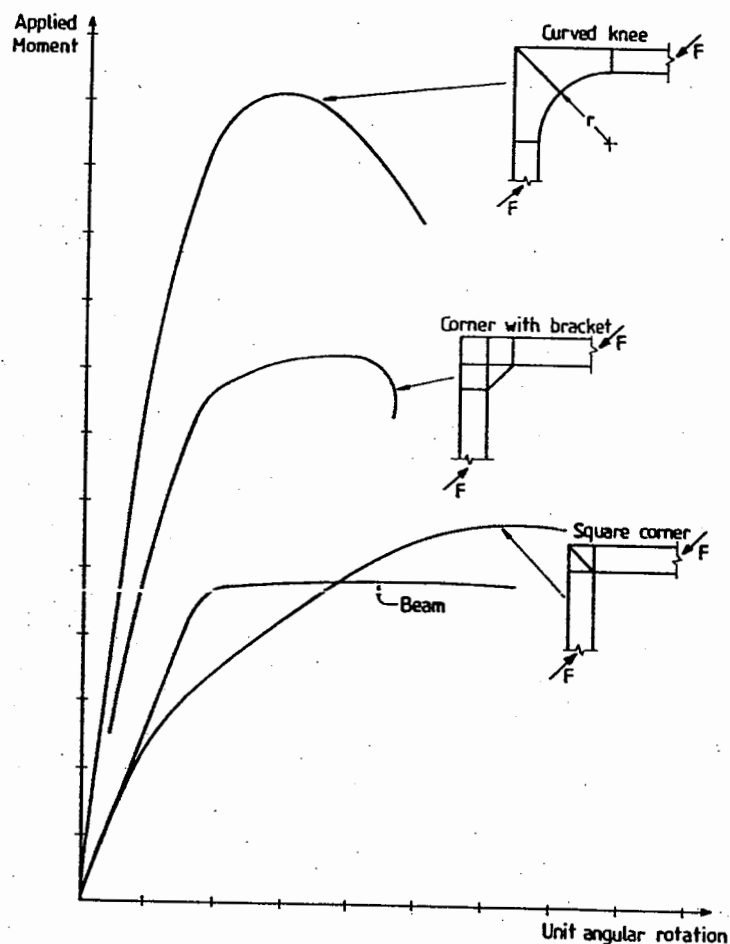


Fig. 5.26

Test results of Fig. 5.26 can answer to this question where the moment-rotation curves of different knee connections are illustrated.

The square corner connection shows to be the most flexible one. It possesses the greatest rotation capacity but on the other hand its strength is somewhat close to the beam resistance. The slope of the straight portion of the curve represents the stiffness of the connection, thus we can conclude that the square corner type induces the smallest stiffness. This can be attributed to the fact that a square corner connection is a continuation of the beam section, thus behaving as rigid as the connected members.

The square corner with bracket (which is similar to the tapered haunch) shows to be more rigid. The stiffness and the strength of this type are increased due to the bracket (or the haunch) added to the square corner type. The rotation capacity, however is smaller compared to the square corner connection.

The curved haunch connection is the most rigid one which possesses the highest strength, while the rotational capacity is relatively small compared to the square corner type.

It is to be noticed that for an elastic analysis, the rotation capacity is not of great importance. Our main objective for such an analysis is the initial stiffness and the strength of the connection. Hence we can conclude that all types of Fig. 5.25 are suitable for an elastic design. However the curved and tapered haunch connections have the advantage of moving the beam section into a region of lower moment. Hence beam cross section will be fully utilized in positive as well as in negative bending regions.

5.6.3. The Corner Connection:

Analysis and Design:

The analysis and the design considerations have been outlined in Sections 3.9 and 5.1. In order to avoid repetition, only the missing features in the previous sections are discussed in order to have a clear understanding of the behavior. The corner connections can be classified as follows:

(a) Configuration (A):

Fig. 5.27.a illustrates a field welded corner connection where the column is extended. The beam will be either butt or fillet welded to the column flange. If stiffeners are required to overcome the crippling of the column web, the bending of the column flange, and the shear of the corner web one can add them as shown by the dotted lines of Fig. 5.27.a. From equilibrium the induced force (F_D) of the diagonal stiffener can be computed as follows:

(Neglect effect of inclination of roof α)

$$F_D = \left(\frac{M}{s_b} - h_c \cdot t_{wc} \cdot q \right) / \cos \theta \quad (a)$$

Hence:

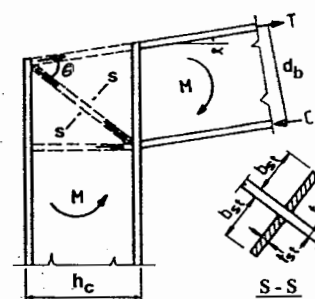
$$2 b_{st} \cdot t_{st} = \frac{F_D}{F_c} \quad (b)$$

Where:

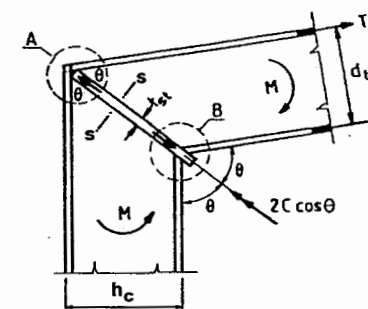
q = the allowable shear stress = $0.35 F_y$

F_c = the allowable compressive stress = $0.58 F_y$

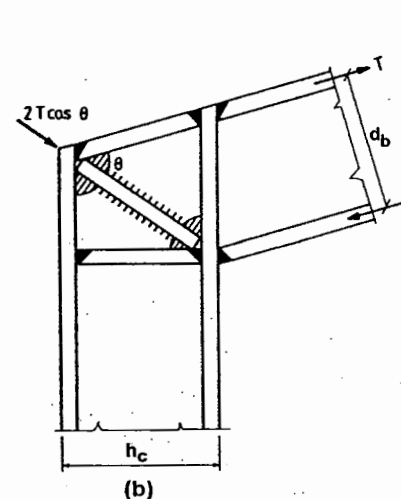
$$b_{st} / t_{st} \leq 25 / \sqrt{F_y} \quad (\text{Local buckling condition})$$



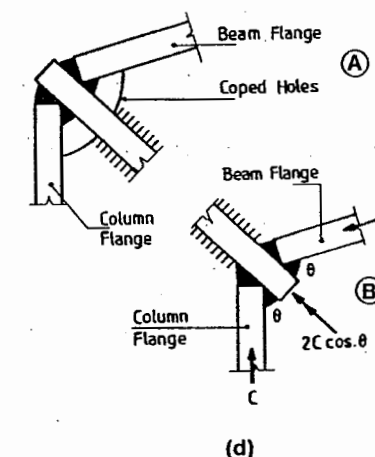
(a) Configuration (A).



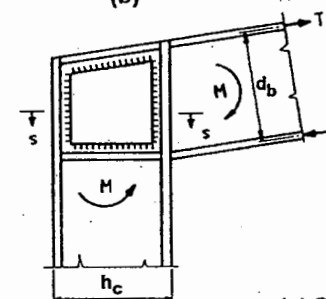
(c) Configuration (B).



(b)



(d)



(e) Configuration (C).

Fig. 5.27



Fig. 5.27.b illustrates the details of welds required to connect the different structural steel components.

The size of the fillet weld (s) attaching the diagonal stiffener to the corner web must be sufficient to transmit the force F_D of Equation (b).

Hence:

$$s = \frac{F_D}{4 [(h_c - 2t_{wc})/(\cos\theta)] \cdot q_w} \quad (c)$$

Where

q_w = the allowable shear stress of fillet welds = $0.2 F_U$

t_{wc} = the column web thickness.

N.B. The inclination of the roof (α) is neglected.

(b) Configuration(B):

Fig. 5.27.c shows an alternative configuration where the column and beam sections are cut diagonally meeting together with a diagonal stiffener.

From equilibrium, the induced force F_D in the diagonal stiffener is:

$$F_D = \left(\frac{M}{d_b} + \frac{M}{d_c} \right) \cdot \cos \theta \quad (d)$$

N.B. The inclination of the roof (α) is neglected, $\theta = 45^\circ$

If the beam and the column are of the same height equation (e) becomes:

$$F_D = 2M \cos \theta / d_b \quad (e)$$



Hence the size of the fillet weld (s) required to attach the diagonal stiffener to the corner web is:

$$s = \frac{F_D}{4 [(h_c - 2t_{fc})/(\cos\theta)] \cdot q_w} \quad (f)$$

Fig. 5.27.d, illustrates details of welds for Configuration (B).

(c) Configuration (C):

However, if the analysis of the corner web shows that the thickness is insufficient, the other alternative instead of using a diagonal stiffener is to provide the corner with a cover plate fillet welded to the web as shown in Fig. 5.27.e. The thickness of the cover plate t_c can be computed as follows:

$$\frac{M}{d_b} = [h_c \cdot t_{wc} + (h_c - 4t_{fc}) \cdot t_c] \cdot q \quad (g)$$

Where:

t_{wc} = thickness of the web of the corner.

t_c = thickness of the additional cover plate.

q = the allowable shear stress. ($q = 0.35 F_y$)

(d) Other Configurations:

Two other alternatives for the corner type are illustrated in Fig. 5.28.

Configuration (D):

If the flanges of one of the rolled sections is extended on through the connection, they can act as stiffeners. This extension may be done either by extending the beam tension flange (Fig. 5.28.a) or by inserting a plate (A) fillet welded to both the column tension flange, the beam compression flange and the beam tension flange (Section 1-1). Only a shaped stiffener (B) will be required as shown by (Section 2-2) of Fig. 5.28.b.

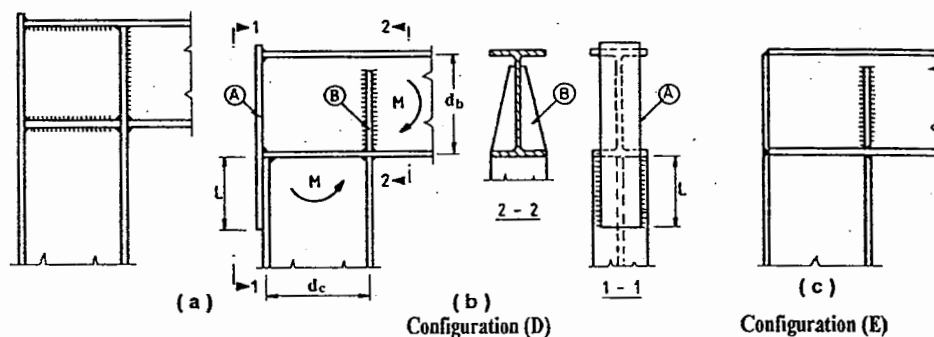


Fig. 5.28

The length of plate (A) is such that the fillet weld lines are capable to transmit the corresponding strength.

$$L = \frac{M}{2 d_c \cdot s \cdot q_w} \quad (h)$$

This configuration is recommended for light sections where the thickness of the steel plates allows to develop the strength by fillet welds.

Configuration (E):

Fig. 5.28.c, illustrates a typical connection attaching members whose flanges are thick. Thus it is essential to transmit the corresponding straining actions by butt welds.

Refer to Example 5.28.

5.6.4. Corner Connection with Bracket: (Refer to Fig. 5.29.a)

As has been outlined in Section 5.8.3 a bracket (haunch) can be provided to the common corner connection to resist the critical eave moment. It was found that the optimum configuration, in order to ensure a minimum weight of steel, can be achieved using a haunch length of approximately one-tenth of the span. The practical situation consists to use a haunch cut from the same section as the beam.(rafter).

In order to avoid repetition, we must notice that the main features of the analysis are exactly the same as Section 5.6.3.

However, the following aspects are to be considered:

(a) Path of Forces:

Fig. 5.29.b, illustrates the diagram of the path of forces. The moments are replaced for simplicity by two equal and opposite forces.

$$T_c = C_c = \frac{M_c}{d_c} ; \quad T_b = C_b = \frac{M_b}{d_b} ; \quad T_h = C_h = \frac{M_h}{(d_b + d_h)}$$

Hence, one can conclude the following:

- Just to the right of joint (f) at the abrupt change of the cross section, the beam compression flange is subjected to the force C_b .

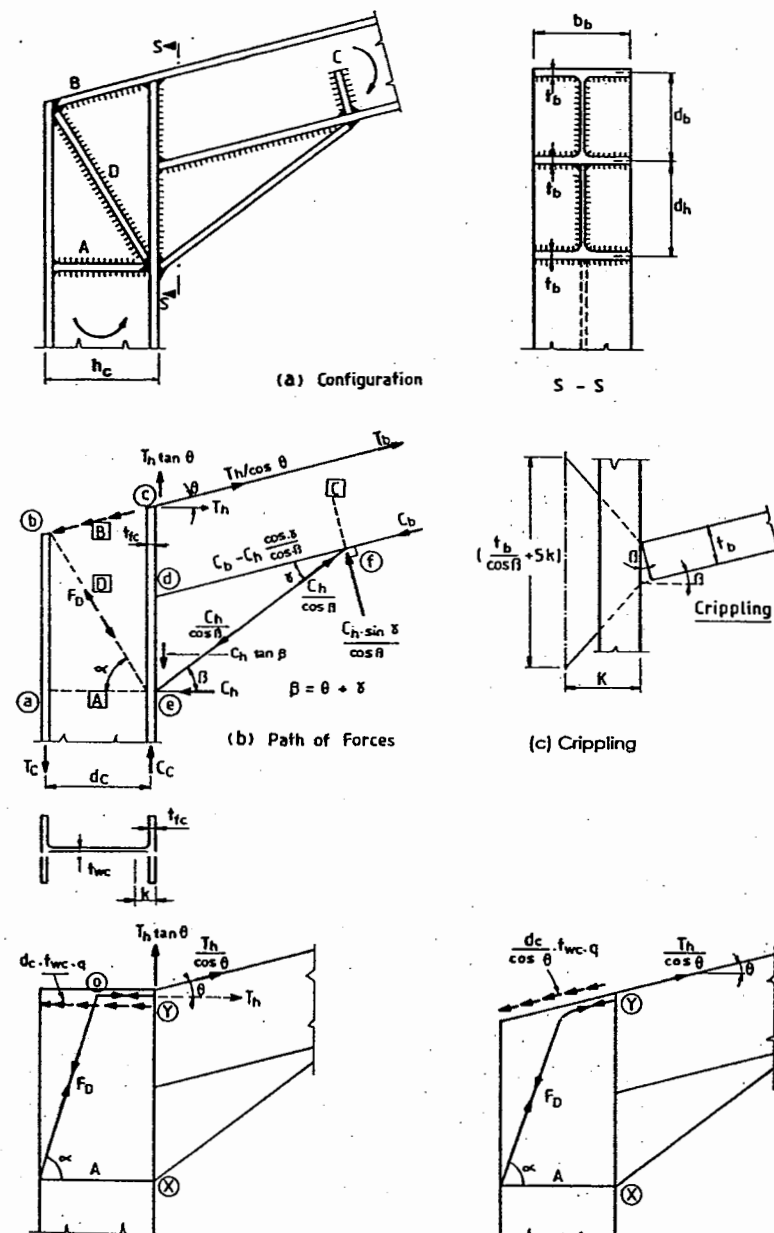


Fig. 5.29

- The haunch flange at joint (e) is subjected to the force ($C_h / \cos \beta$).

- From equilibrium of forces at joint (f), the extension of the beam flange just to the left of the joint (f) will be subjected to the force ($C_b - C_h \cdot \frac{\cos \gamma}{\cos \beta}$):

(b) Stiffeners:

Different stiffeners are to be utilized. Their location is as follows:

(i) A Stiffener (A) is to be provided opposite to the haunch flange (i.e. at joint "e"). This stiffener will prevent the crippling of the column web (see Fig. 5.41.b and c).

The cross section of stiffener (A) is:

$$A_{st(A)} = \left[b_b \cdot t_b \cdot \cos \beta - t_{wc} \left[\frac{t_b}{\cos \beta} + 5K \right] \right] \quad (a)$$

Hence, the size of the fillet welds attaching the stiffener to the column web is:

$$s = \frac{C_h - t_{wc} \left(\frac{t_b}{\cos \beta} + 5K \right) \times (F_{crp})}{4 q_w \cdot (h_c - 4 t_{fc})} \quad (b)$$

Where:

s = the fillet weld size.

q_w = the allowable shear stress of the fillet weld = $0.2 F_u$

F_{crp} = the allowable crippling stress = $0.75 F_y$

(ii) A Stiffener (B) is to be provided opposite to the beam tension flange if the column flange thickness is not capable to prevent the bending. The location of stiffener (B) is at joint (c) and is to be utilized if the following condition is not satisfied:

$$t_{fc} \geq 0.4 \sqrt{b_b \cdot t_b \cdot \cos \theta} \quad (c)$$

It must be noticed that it is preferable to extend the beam tension flange. The extended flange length will be fillet welded to the nearest column flange and to the column web. While the farther weld detail can be chosen as previously shown in Fig. 5.27.

(iii) Stiffener (C) must be added at joint (f) to overcome the abrupt change of the compressive forces. This stiffener is preferably chosen normal to the beam flange and is subjected to a force (F_c) which can be computed from the equilibrium of forces at joint (f) in a direction normal to the beam flange.

$$F_c = C_h \cdot \frac{\sin \gamma}{\cos \beta} \quad (d)$$

Accordingly, the fillet weld length required to attach this stiffener to the beam web can be computed if the size of the weld is conveniently assumed.

(iv) The Diagonal Stiffener (D) is subjected to a compression force F_D as shown in Fig. 5.27.b. In order to determine the force F_D , let us proceed as follows:

The applied force on the column web at the outer

$$\text{side } b-c = \frac{T_h}{\cos \theta} \quad (e)$$

While the shear resistance of the column web at this outer side $b-c = \left(\frac{d_c}{\cos \theta} \right) \times t_{wc} \times q \quad (f)$

Where q = the allowable shear stress. ($q = 0.35 F_y$).

If the applied force of equation (e) exceeds the shear resistance (Eq. (f)) a diagonal stiffener (D) is to be provided.

Applying the equilibrium of horizontal forces at joint (b), the following relation can be computed:

$$\frac{T_h}{\cos \theta} \times \cos \theta = \left(\frac{d_c}{\cos \theta} \right) \cdot t_{wc} \cdot q \cdot \cos \theta + F_D \cdot \cos \alpha$$

$$\text{Hence: } F_D = \frac{T_h - d_c \cdot t_{wc} \cdot q}{\cos \alpha} \quad (g)$$

It is to be noted that the use of (d_c) in Eqs. (f) and (g) is conservative.

The MORRIS Tension stiffener (Fig. 5.41.d, e):

Once again, the MORRIS' tension stiffener is a continuous stiffener combining the function of the horizontal column tension stiffener (B) and the compression diagonal stiffener (D).

The design of the different steel components comprising this MORRIS stiffener has been previously outlined in Section 5.1.4. The analysis can be proceeded as follows:



- The upper Horizontal Portion Of The MORRIS Stiffener as shown in Fig. 5.41.d will be subjected to the remaining tension force ($T_u - P_u$) where:

T_u is the maximum tension capacity of the beam tension flange.

P_u is the maximum capacity of the column flange. Their values can be computed from the yield-line theory as previously outlined in Section 3.9.

Hence:

$$T_u = b_b \cdot t_b \cdot F_y \cdot \cos\theta \quad (h)$$

$$P_u = 0.8 (7 F_y \cdot t_{fc}^2 + 0.15 b_b \cdot t_b \cdot F_y \cdot \cos\theta) \quad (i)$$

If T_u exceeds P_u the upper portion of the MORRIS stiffener will be subjected to the remaining tension force T_r ,

$$T_r = F_y (b_b \cdot t_b \cdot \cos\theta - 5.6 t_{fc}^2 - 0.12 b_b \cdot t_b \cdot \cos\theta) \quad (j)$$

- The Diagonal Portion of the MORRIS Stiffener will be subjected to the force F_D :

$$F_D = \frac{T_h - d_c \cdot t_{wc} \cdot q}{\cos\alpha} \quad (k)$$

The MORRIS stiffener being a continuous stiffener, we conclude that the governing stiffener force will be the greater of Equation (j) and Equation (k).

It is to be noticed that in addition to the horizontal and diagonal portions, a horizontal stiffener (A) is to be



provided opposite to the haunch compression flange following Equation (a).

Refer to Examples 5.10 and 5.14.

5.6.5. Tapered Haunch Connection:

The tapered and curved haunch connections of Figs. 5.25.c,d may extend over one tenth of the span. These types are not really connections in the proper sense of the term. The actual design for most of the specifications is based on the elastic behavior which was extensively studied. In addition recent developments of the behavior in the inelastic range are utilized for plastic analysis.

However, in the E.C.P. of steel structures (2001), these types of connections are left to the judgement of the designers. One can make use of the design rules available in the literature.

In the following paragraphs as well as in Section 5.6.6, elastic analysis is conducted and the corresponding restrictions are deduced.

As has been stated in Section 5.6.1, the tapered haunch connections are utilized to improve the esthetic appearance of the structure. These connections placed at the maximum eaves moments, the size of the rolled shapes required for the rafters is reduced. Thus a considerable saving of steel is obtained. For the analysis and the design, the following aspects are to be considered:

(a) The strength of the steel elements comprising the connection must be such that they can withstand the induced

straining actions due to the applied loads.

(b) The location of the neutral axis for the tapered portion is to be determined in a safe manner. This will allow the calculation of each section's modulus and hence the corresponding stresses.

(c) The haunch is to be proportioned in a manner that will provide resistance against the lateral buckling. At each joint of abrupt change bracing are to be provided.

(d) Stiffeners are to be utilized at each joint of abrupt change to overcome the concentration of compressive forces.

5.6.5.1. Strength and Location of the Neutral Axis:

(a) As a first step we must select the general proportions of the haunch. Its length along the girder (beam or rafter) can be chosen one-tenth of the span and the angle (β) between the flanges not less than 12 degrees (see Fig. 5.42.a). This limitations of inclination will probably provide safe bending stresses regarding Section (2-2).

The breadth and thickness of the tension haunch flange as well as the haunch web thickness may be chosen as the girder (rafter) dimensions.

The compression lower haunch flange thickness may be increased by the ratio ($1/\cos\beta_1$) due to the following reasons:

(i) Section 1-1 just to the right in the rafter cross section may be stressed to the maximum allowable compressive stress (f). (ii) Regarding Section 1-1 just to the left in the haunch cross section, the lower compressive haunch flange will

be stresses by ($f/\cos\beta_1$). (iii) Hence the compressive haunch flange thickness is to be increased by the same ratio (i.e. $1/\cos\beta_1$).

This means that:

$$t_h = t_b / \cos\beta \quad (a)$$

(b) The straining actions are to be computed using a theorem of structural analysis. The ratio of column to girder inertia is to be conveniently assumed, while for the haunch an increased inertia is to be selected along its length.

The bending moment diagram is to be drawn perpendicular to the girder axis as shown in Fig. 5.30.a.

If the points of inflection are known, one can compute the bending moment at the different required sections as shown in Fig. 5.42.b.

$$\begin{aligned} \text{i.e.} \quad M_1 &= Q.X_1 \\ M_2 &= Q.X_2 \\ M_2' &= Q.X_2' \\ M_3 &= Q.X_3 \end{aligned} \quad (b)$$

(c) Regarding the location of the neutral axis necessary to compute the modulus section at the critical sections, a simple but conservative method is to extend the center lines of the beam and the column to intersect in the knee at point (0) as shown in Fig. 5.30.a. Perpendicular sections to these center



lines are used to determine the section modulus with respect to the axes lying on the corresponding center lines. This assumption is conservative and will lead to higher stresses in the sloping flanges.

Another method assumes that the neutral axis is located on the axis passing through the center of gravity of sections normal to the axes of the beam and the column. The section modulus is then computed with respect to the axis passing along the center of gravity of the corresponding section.

(d) The stresses are to be checked along Sections (1-1), (2-2) and (3-3). If these stresses exceed the allowable limits, it is suggested to increase only the haunch compression flange thickness while the other haunch dimensions remain unchanged. The other possibility is to increase the haunch depth and recheck the stresses.

5.6.5.2. The Stability Requirements:

As has been previously mentioned, the tapered haunch can suffer from instability. The haunch compression flange at joints (1), (2) and (3) can buckle laterally. IN order to overcome this instability phenomenon, let us consider the following assumptions:

(a) Provide bracing at the extremities and at the common intersection points of the compression flanges. (i.e. at joints (1), (2) and (3)). These bracings ensure that the compression flange will not buckle laterally at these joints.

(b) Then consider the haunch compression flange between the bracing points acting as a column simply supported at the ends.



The web is assumed to provide restraint in the direction perpendicular to the haunch compression flange. Hence only the flange cross section is assumed to withstand the lateral buckling.

In order to deduce a safe limiting condition, the buckling strength of the compression flange acting as a column is given by the following:

$$F_c = \frac{\pi^2 E}{(L/i)^2} = F_y \quad (a)$$

Where F_c = the Euler buckling stress.

L = either L_{12} or L_{23} as shown in Fig. 5.30.a

F_y = the yield stress.

$i = b/\sqrt{12}$ (for a rectangular cross section of breadth b)

Put in Equation (a): $\pi = 3.14$, $E = 2100 \text{ t/cm}^2$, $F_y = 2.4 \text{ t/cm}^2$, $i = b/\sqrt{12}$ and use a factor of safety F.S ≈ 3.0 .

Get:

$$\frac{L}{b} \leq 100 \quad (b)$$

If Equation (b) is satisfied, and adequate bracing is utilized at joints (1), (2) and (3) the lateral buckling of the haunch compression flange is prevented.

(c) The other instability phenomenon consists to ensure that the local buckling of the compression flange is prevented.



One can make use of the results for stocky sections where the following condition is to be satisfied:

$$\frac{C}{t_b} \leq \frac{1}{\sqrt{F_y}} \quad (c)$$

5.6.5.3. Stiffeners:

A diagonal stiffener (B) as well as two transversal stiffeners (A) and (C) are to be provided at the intersection of the haunch flanges and at the abrupt change of cross sections. The induced forces and the corresponding cross sectional areas can be computed as follows:

(i) Diagonal Stiffener (B):

Fig. 5.30.c, illustrates the compressive flange forces, induced in the diagonal stiffener (B):

Taking the equilibrium of forces in the horizontal direction we can compute F_B from the following relation:

$$F_B \cdot \cos \theta + F_{23} \cdot \sin \beta_2 = F_{21} \cos (\beta_1 + \gamma) \quad (d)$$

$$\text{Where: } F_{21} = \frac{M_2'}{d_2' \cos \beta_1}; \quad F_{23} = \frac{M_2''}{d_2'' \cos \beta_2} \quad (e)$$

The computed force (F_B) (Equation (d)) is the resulting compressive force. An induced compressive force F_B can result also from the equilibrium of the forces in the outer flanges. For the computation of this force we can proceed as follows:

- The web of the haunch will resist a shearing force on the outer inclined edge of length $d_2' / \tan(\gamma + \theta)$:

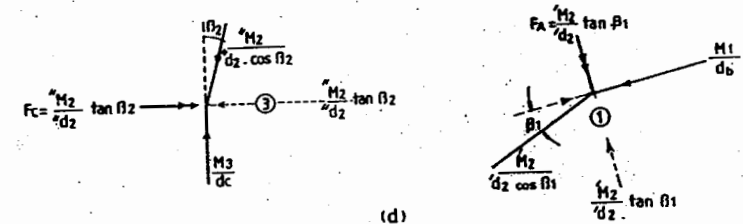
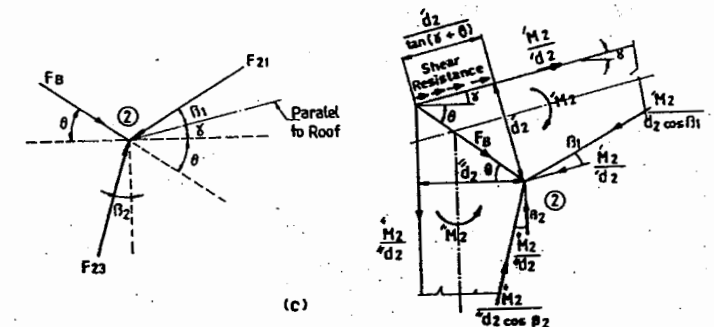
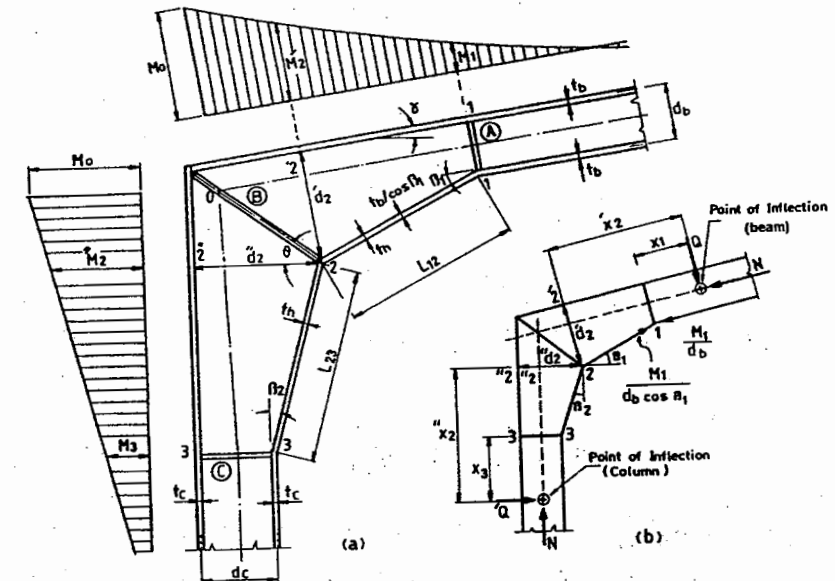


Fig. 5.30



$$\text{Resistance} = \left[\frac{d'_2}{\tan(\gamma + \theta)} \right] \cdot q \cdot t_{wh}$$

where q = allowable shear stress = $0.035 F_y$

t_{wh} = thickness of the haunch web.

Taking the equilibrium of forces in the horizontal direction at the outer corner, the following relation can be deduced:

$$\frac{M'_2}{d'_2} \cos \gamma = \frac{d'_2}{\tan(\gamma + \theta)} \times q \cdot t_{wh} \cdot \cos \gamma + F_B \cdot \cos \theta \quad (f)$$

Hence calculate F_B and then choose the greater of Equations (e) and (f) to determine the required cross sectional area ($2b_{st} \cdot t_{st}$) with $b_{st}/t_{st} \leq 16$.

(ii) Transversal Stiffeners:

At the abrupt change of cross sections i.e. at joints (3) and (1), Fig. 5.30.d, illustrates the equilibrium of forces. Hence the force in the corresponding stiffeners (A) and (C) can be computed.

From Fig. 5.30.d one can deduce the following:

$$\left. \begin{aligned} F_A &= \frac{M'_2}{d'_2} \cdot \tan \beta_1 \\ F_C &= \frac{M''_2}{d''_2} \cdot \tan \beta_2 \end{aligned} \right\} \quad (g)$$

It is to be noticed that stiffeners (A) and (C) are normal to the axes of the rafter and the column respectively.



5.6.5.4. Bracing Requirements:

As has been mentioned previously in Section 5.6.5.2 bracings are to be provided at joints (1), (2) and (3) to prevent the lateral buckling of the haunch compression flange.

It will be in accordance with the test results to take the magnitude of the restraining force in any restraining structural element (bracing element) of the order of 2 percent of the flange compressive force. Hence from Fig. 5.31 we can deduce:

$$F_{\text{Bracing}} = \pm \frac{0.02 \text{ b.t. } F_b}{2 \cos \eta} \quad (h)$$

Where: b.t = the area of the compression flange of either the beam or the column for joints (1) and (3) respectively

F_b = the allowable bending stress = $0.58 F_y$.

η = the angle of inclination of the bracing element, usually taken 45 degrees.

Generally the induced force in the bracing element is relatively small. The stiffness limitation is at least as important as the strength criterion. Due to the lack of sufficient information, a limiting slenderness ratio of 100 is recommended for diagonal bracing elements i.e:

$$L/i \leq 100 \quad (i)$$

Where: L = length of the diagonal bracing element.

i = the least radius of gyration.

Fig. 5.31, illustrates the details of the two connections at joints (1) and (3) where the upper purlin was chosen a S.I.B section while the side purlin was a channel section.

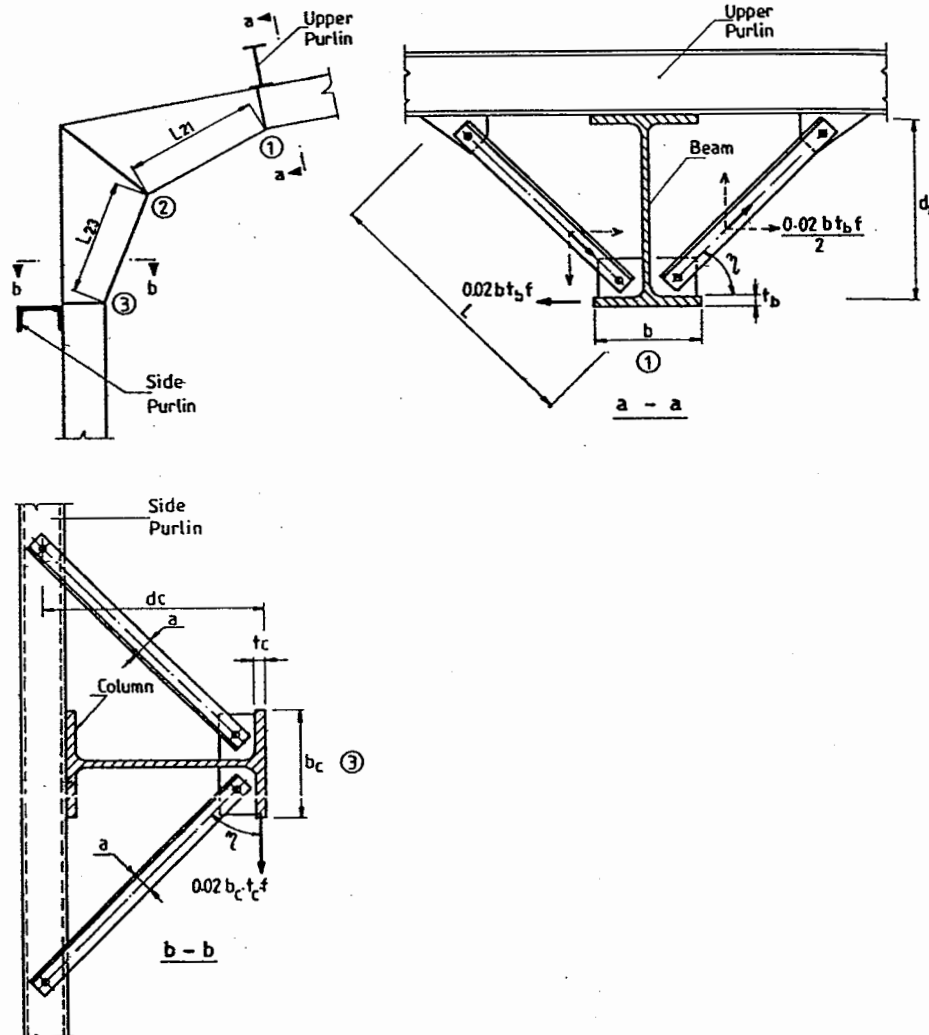


Fig. 5.31

It is of common practice to standardize the size of the bracing elements. As the column flange area is generally larger



than that of the beam (rafter), the design of the bracing element will be based on the conditions appropriate to the restraint column haunch intersection. Hence if one angle cross section is chosen to respond to the compression force of Equation (h), the leg angle size "a" is to be chosen according to the following:

$$a \geq \frac{d_c \sqrt{2}}{20} \quad (j)$$

Where: d_c = the column depth.

Then check the induced stress with respect to the allowable buckling stress for compression members.

Similarly a bracing member of the same size is to be provided at joint (2) and attached to either an upper purlin or a side purlin.

5.6.5.5. Welding Details:

In order to avoid repetition, refer to Figs. 5.7 and 5.27 which shows the details of the different configurations of butt and fillet weld lines.

It is to be noticed that the transversal stiffeners are to be fitted with the haunch compression flange, while the fillet weld lengths are to be calculated according to the appropriate induced compression forces.

Refer to Example 5.15.



5.6.6. The Curved Haunch Connections:

The main aspects of a curved haunch connection are the same as a tapered one. The curvature will cause additional cross bending stresses. In the following sections the analysis and the design considerations are presented:

5.6.6.1. The Strength and the Location of the Neutral Axis:

(a) The design starts with the selection of the geometrical configuration. For simplicity choose a symmetrical location of the diagonal stiffener (B) with respect to the inclined roof and the vertical side (i.e. same inclination α) as shown in Fig. 5.32.a.

(b) The length L_{12} may be selected one tenth of the span, thus the center of the curved lower haunch compression flange can be determined and consequently the length L_{23} .

(c) The neutral axis is assumed to be located at the center lines of the rolled shapes of the girder and the column as has been adopted for the tapered haunch.

(d) Apply one of the theorems of the theory of structures to calculate the induced straining actions. The ratio of the column inertia to the girder inertia as well as the knee inertia are to be reasonably assumed.

The bending moment diagram may be drawn as shown in Fig. 5.32.a. If the points of inflection are determined and the corresponding applied normal and shearing forces are known, hence the moments can be computed.

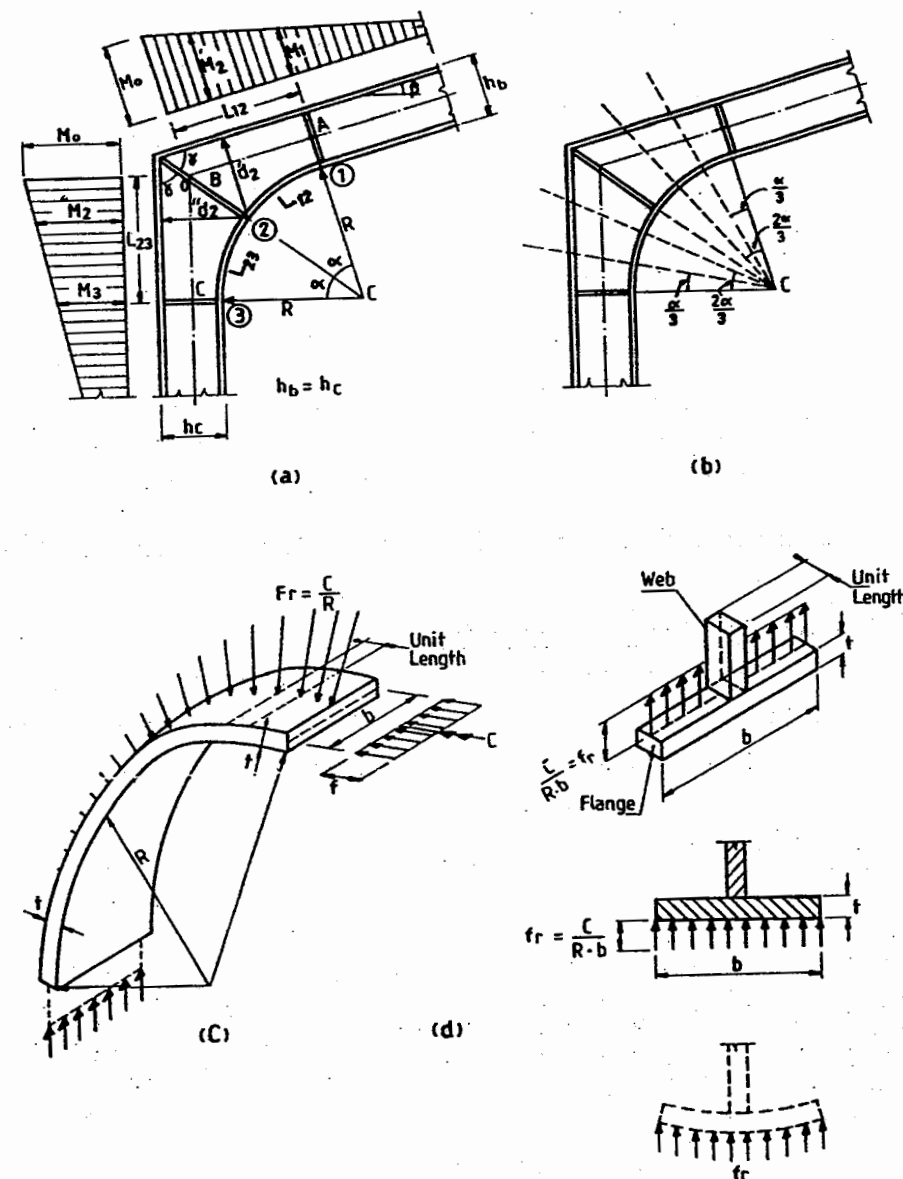


Fig. 5.32



(e) For the calculation of the critical stresses it is to be observed that the critical section is unknown as where the moment increases the modulus section is also increasing. Hence it is suggested to compute the section modulus at Sections (1), (2) and (3) and those which correspond to the angles $(\alpha/3)$, $(2\alpha/3)$ as shown in Fig. 5.32.b.

The haunch flange breadth and thickness as well as the haunch web thickness are chosen equal to those corresponding to the girder dimensions.

If the computed stresses are within the allowable limits, the chosen dimensions are sufficient. Otherwise the thickness of the curved compression flange is to be increased. Generally the critical section was found to be along an angle equal to 12° .

5.6.6.2. The Cross Bending Effect:

As has been mentioned previously if an axial force is changing its direction, suitable steel components are required to overcome the equilibrium of the components of forces. As an example, the tapered haunch of Fig. 5.32.a was demanding three stiffeners (A), (B) and (C) to overcome the abrupt change at joints (1), (2) and (3).

For the curved haunch, the change in direction of the axial force is uniform along the curved edge of the flange. This will induce radial compressive forces in the haunch web as shown in Fig. 5.32.c.

Replacing the applied moment by two equal and opposite forces ($C = T = \frac{M}{d}$), the applied compression force (C) in the

compressive flange will induce a radial compressive force F_r in the web as shown in Fig. 5.32.c. The force F_r acting radially per linear unit length of the web is computed as given below:

$$F_r = \frac{C}{R} \quad (a)$$

The axial compressive force being assumed to be uniformly distributed across the width (b), hence the radial pressure (f_r) can be written as follows:

$$f_r = \frac{C \times \text{unit length}}{R \times b} = \frac{C}{R.b} \quad (b)$$

As shown in Fig. 5.32.d the radial pressure (f_r) tend to bend the flange which behaves as a cantilever beam, since it is supported only along its center line by the web. The induced bending moment M' is:

$$M' = \frac{1}{2} \times \frac{C}{R.b} \times \left[\frac{b}{2} \right]^2 = \frac{C.b}{8R} \quad (c)$$

The corresponding critical stress is M' / Z , hence:

$$f_t = \frac{3 C.b}{4 R.t^2} \quad (d)$$

Where: $Z = 1.t^2 / 6$

f_t = the tensile cross bending stress or the tensile transverse stress.

Replace C in Eq. (d) by the compressive stress (f) times the compression flange area of the haunch (bxt) i.e ($c = f.b.t$):

$$f_t = \frac{3 f.b^2}{4 R.t} \quad (e)$$

In order to hold the transverse tensile stress (f_t) to a value not exceeding the flange axial longitudinal compressive stress (f) compute the following relation:

$$\frac{b^2}{R \cdot t} \leq \frac{4}{3} \quad (f)$$

If relation (f) is not satisfied, stiffeners attaching the haunch compression flange to the neighbouring web are to be utilized. For the computation of the forces acting on these stiffeners, refer to Section 5.6.6.4.

5.6.6.3. Concentration of Stresses:

(a) Concentration of Stresses at the Haunch Web-To-Flange Attachment:

In the preceeding Section the axial compressive stress of the flange (f) was assumed to be uniform along the flange breadth (b). But unfortunately this assumption is not a correct one where the cross bending action will tend to induce higher stresses at the vicinity of the attachment of the web with the flange while at the tips of the flange these stresses are reduced.

In order to clarify what happens, let us consider a portion of the curved flange ($d\phi$) as illustrated in Fig. 5.32.a. Due to the cross bending effect of Section 5.6.6.2, the induced radial pressure (f_r) will force the flange to deflect as a cantilever beam. Because of the cantilever type deflection the axis of the member longitudinal fibers at points such as (a) will strain less than fibers directly below the web such as point (b). Fig. 5.33.b, illustrates the corresponding strains Δa and Δb . Since the strain at (b) is greater than that at (a), the resulting stress at (b) will be greater than at (a). This will

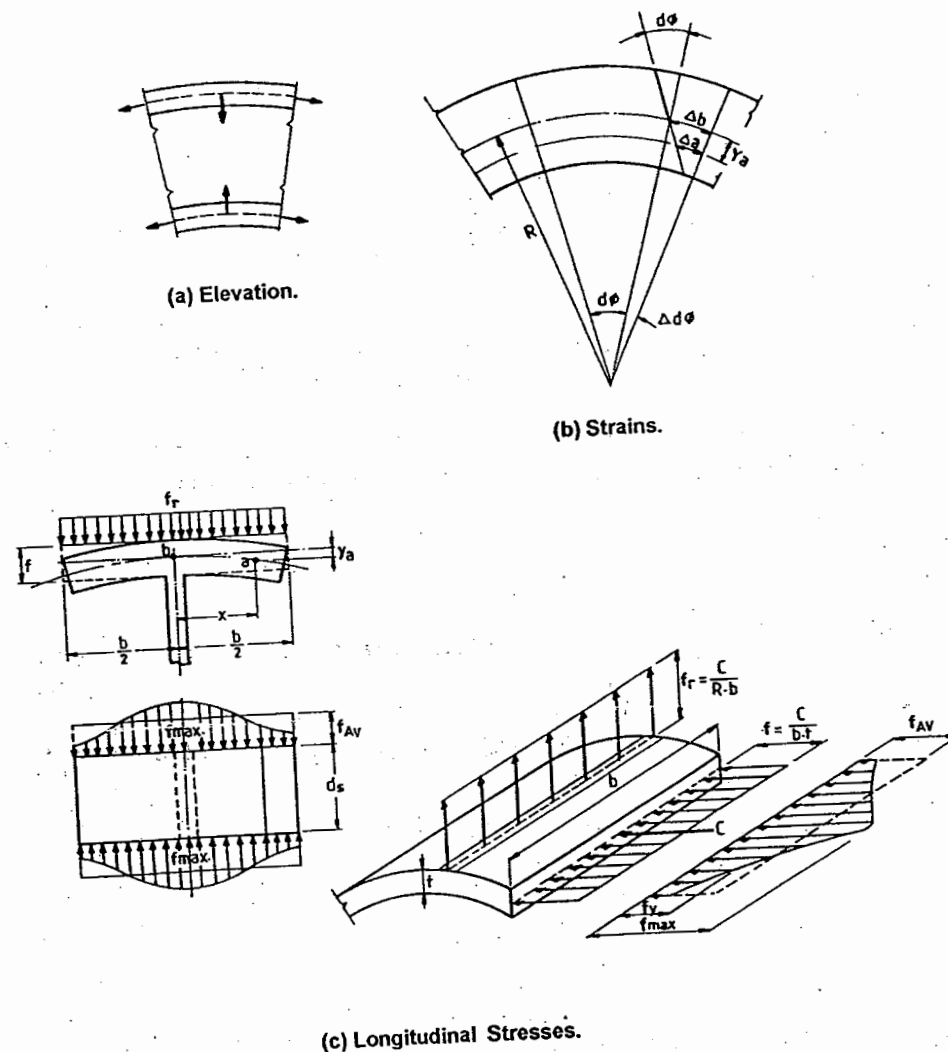


Fig. 5.33

force consequently the longitudinal stress distribution to be nonuniform as shown in Fig. 5.33.c.

Bleich (10) has studied this situation and has carried an analysis leading to the following:

$$f_{MAX} = \frac{f}{\alpha} \quad (g)$$

Where: f_{MAX} = the longitudinal compressive stress in line with the web.

α = a factor affected by the value of $(b^2/R.t)$

The maximum transverse tensile bending stress (cross bending stress) is found using the following relation:

$$f_{t.MAX} = \beta \cdot f_{MAX} \quad (h)$$

Where: $f_{t.MAX}$ = the maximum transverse bending tensile stress.

β = a factor which depends on the value $(b^2/R.t)$

For the determination of the values of α and β we can refer to Fig. 5.34 (Bleich - Reference 10).

From Fig. 5.46 we can observe that for values of $(b^2/R.t)$ less than $(4/3)$ as in Equation (f), the values of α and β are slightly influenced. The maximum longitudinal compressive stress as well as the transverse tensile stresses are affected by about 15 per cent. Neglecting this change has been judged to be tolerable.

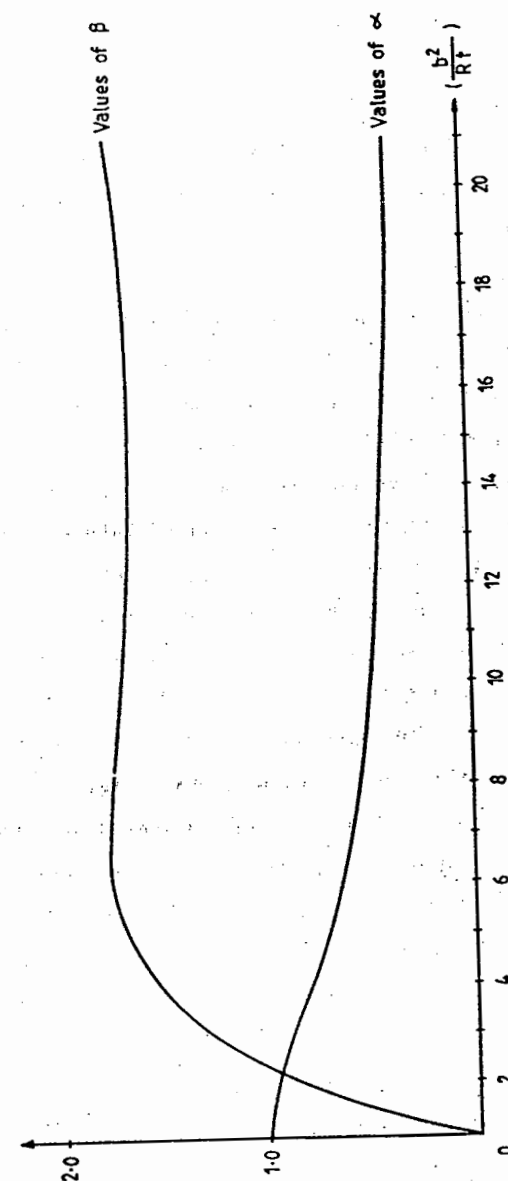


Fig. 5.34 Ref. (10)

(b) Concentration of Stresses at the Reentrant Compression Haunch Flange:

Due to the variation in cross section within the haunch, the ordinary beam theory relations for the computation of stresses can lead to some error. However the amount of error depends on the inclination of the haunch. If the haunch is not too sharp the results obtained are sufficiently accurate for design purposes.

In order to clarify the real behavior, let us refer to Sections 5.6.5.1 and 5.6.6.1 where the neutral axis was assumed to be located on the center lines of both the girder and the column. In reality a more accurate method is to use sections normal to the axis of either the beam or the column as shown in Fig. 5.35.a. The section modulus is then determined about an axis through the center of gravity of the section. The resulting stress in the inner flange is increased by the factor $(1/\cos^2 \phi)$, while the bending stress in the outer flange decreases. This behavior is illustrated in Fig. 5.35.b. For this section, the neutral axis will be located at about one fifth of the length of the diagonal from the inside corner for a tapered haunch connection, while for a curved haunch connection (as shown by the dotted line) the neutral axis is located at one fourth of the diagonal length. The stress distribution may be considered to take the form of a second degree parabola on the tension side and of a triangle on the compression side.

From this discussion, it can be observed that the ineffectiveness of the outer flanges may induce high tensile stresses in the web. However this situation can be checked using the effective cross section shown in Fig. 5.35.c assuming the neutral axis to be located at one fifth or one fourth of the

diagonal length for the tapered and curved haunch respectively. Hence the stresses can be computed using the equation of the beam theory.

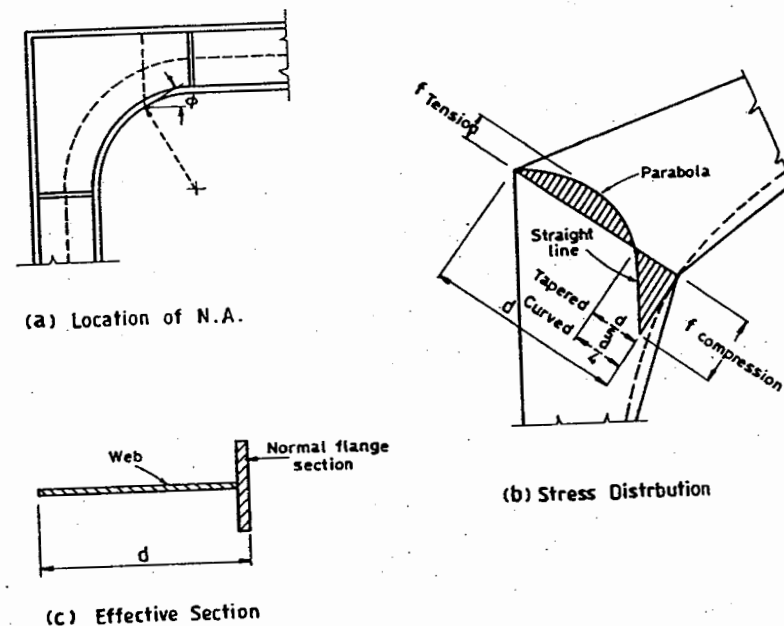


Fig. 5.35

5.6.6.4. The Stiffeners:

(i) The Diagonal Stiffener:

The same procedure, as for the tapered haunch, will be adopted for the determination of the force F_B applied on the diagonal stiffener.

For the computation of the compressive force F_B as a result of the compressive forces in the curved inner flange at joint (2), one can treat the curved haunch as being a tapered one as shown in Fig. 5.36.a.



The force F_B is computed by taking the equilibrium of forces at joint (2) in the horizontal direction. The following relation can be deduced: (see Fig. 5.36.b)

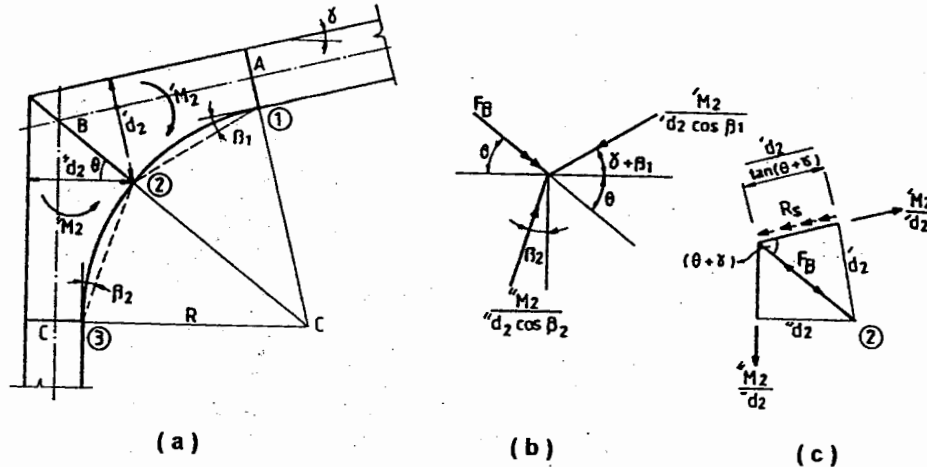


Fig. 5.36

$$F_B \cos \theta + \frac{M_2''}{d_2} \tan \beta_2 = \frac{M_2' \cdot \cos(\gamma + \beta_1)}{d_2 \cdot \cos \beta_1} \quad (i)$$

While if one will consider that the diagonal stiffener will resist the remaining force in shear, the following can be computed (see Fig. 5.36.c):

$$\text{Shear Resistance} = R_s = \frac{d_2}{\tan(\theta + \gamma)} \cdot q \cdot t_{wh} \quad (j)$$

Hence taking the equilibrium in the horizontal direction:



$$F_B = \left(\frac{M_2'}{d_2} - R_s \right) \cdot (\cos \gamma / \cos \theta) \quad (k)$$

Once the induced force F_B is calculated the corresponding cross sectional area and the required fillet weld length can be computed.

(ii) The Transverse Stiffeners:

Applying the equilibrium of forces in a direction normal to the axes of the beam and of the column, we can compute the induced forces F_A and F_C as follows:

$$\left. \begin{aligned} F_A &= \frac{M_2' \cdot \sin \beta_1}{d_2 \cdot \cos \beta_1} = \frac{M_2'}{d_2} \tan \beta_1 \\ F_C &= \frac{M_2'' \cdot \sin \beta_2}{d_2 \cdot \cos \beta_2} = \frac{M_2''}{d_2} \tan \beta_2 \end{aligned} \right\} \quad (l)$$

(iii) The Radial Stiffeners:

Radial stiffeners are to be utilized to support the lower compression flange of the haunch if the condition of Section 5.6.6.2, (Equation (f)) is not satisfied. This means that the radial stiffeners are utilized if $\left(\frac{b^2}{R \cdot t} \right)$ exceeds $(4/3)$.

In order to determine the location and the cross section of these radial stiffeners, we can proceed as follows:

(a) Assume to use a radial stiffener at mid-distance between stiffeners (A) and (B) and another one at mid-distance between stiffeners (B) and (C).



(b) Calculate the bending moments at the location of these two radial stiffeners say M_4 and M_5 . Hence calculate the corresponding compression forces, acting normal to the curved flange of the haunch (see Fig. 5.37.a):

$$C_4 = M_4 / d_4 \cdot \cos \beta_4 \text{ and } C_5 = M_5 / d_5 \cdot \cos \beta_5$$

(c) Refer to the first principles of Section 5.6.6.2 and use Equation (d) to calculate the transverse bending stress ($f_t = \frac{3 C \cdot b}{4 R \cdot t^2}$) which will be somewhat greater than the allowable bending stress say f_{all} . This step is to be calculated twice for both Sections (4-4) and (5-5) (i.e. f_{t4} and f_{t5} are to be calculated).

(d) Calculate the percent of increase of these transverse bending stresses with respect to the allowable one i.e:

$$P_4 = \frac{f_{t4} - f_{all}}{f_{all}}$$

$$P_5 = \frac{f_{t5} - f_{all}}{f_{all}}$$

(m)

Where : P_4 and P_5 are the percent of stress increase at Sections (4-4) and (5-5).

(e) Compute the force to be resisted by the radial stiffeners using equations (a) and (m). Hence the following can be determined:



and

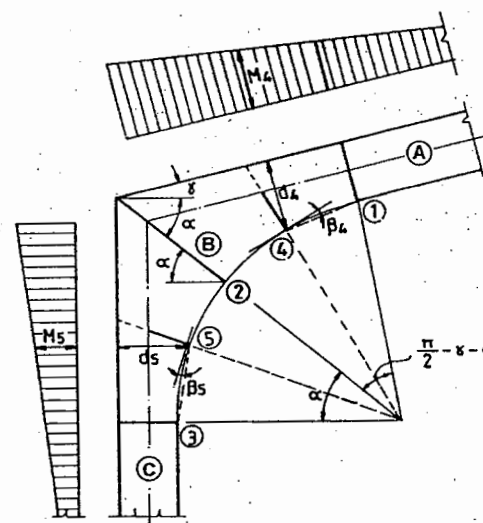
$$F_4 = P_4 \cdot \frac{C_4}{R} \cdot L_{12}$$

$$F_5 = P_5 \cdot \frac{C_5}{R} \cdot L_{23}$$

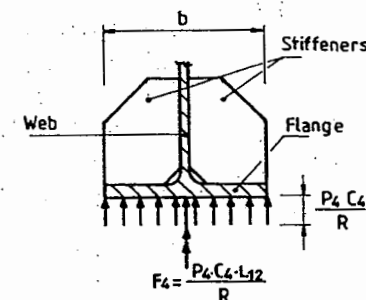
(n)

Where:

L_{12} and L_{23} are the arc lengths between 1,2 and 2,3 respectively



(a)



(b)

Fig. 5.37

(f) The corresponding area of each pair of stiffeners can be computed using relation (n) in accordance with the allowable compressive stress ($F_{all} = 0.58 F_y$).

$$\text{i.e. } A_{st(4)} = \frac{P_4 \cdot C_4 \cdot L_{12}}{R \cdot 0.58F_y}$$

$$A_{st(5)} = \frac{P_5 \cdot C_5 \cdot L_{23}}{R \cdot 0.58F_y}$$

(o)

It is to be noticed that no need to extend these radial stiffeners till the outer flange of the haunch. Their length will correspond to the length required to transmit their forces by fillet welds to the web of the haunch. The lower edge of these stiffeners must be fitted to the compression flange of the haunch.

5.6.6.5. The stability Requirements:

The curved haunch connection is to be braced laterally at joints (1), (2) and (3). Hence as for the tapered haunch relation (b) of Section 5.6.5.2 is to be utilized (i.e. $L/b \leq 100$).

$$\text{Where } L = L_{12} \text{ or } L_{23} = R \left(\frac{\pi}{2} - \gamma - \alpha \right) = R \alpha \quad (p)$$

γ and α are in radians.

If the unbraced length L_{12} or L_{23} exceeds this limit, the thickness of the inner compression flange should be increased. The other alternative would be to increase the inner flange breadth. In any case, b/t must not exceed 16 in order to prevent the local buckling phenomenon.

5.6.6.6. Weld Detailing:

Referring to Fig. 5.7 and 5.27 one can choose the

appropriates welds to attach together the different steel components comprising the curved haunch connection.

Refer to Example 5.16.

Group (IV) Eave and Apex connections :-

5.7. The Portal Frame Connections:

The structural steelwork is most competitive in industrial buildings. Portal frames represent the most widely used form of frames. In addition to the strength and the stability requirements the designer is obliged to take care about the cost of fabrication and the erection procedure.

Fig. 5.38, illustrates a typical portal frame where the field connections are moved to the eaves and apex positions. Due to transportation considerations, field connections are usually located at positions of high moments. This means that the connections tend to occur at the interface of the column and the rafter (eave connections) and also between the rafter members at the ridge (apex connections).

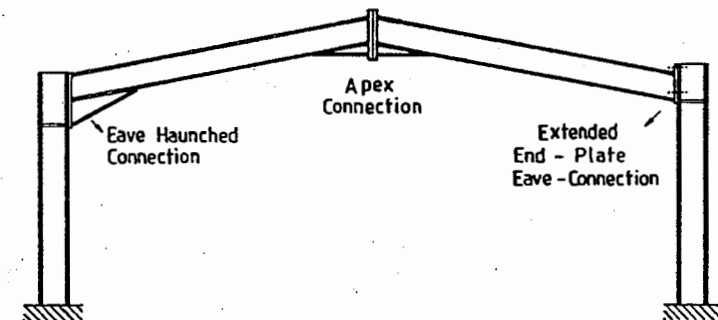


Fig. 5.38

Generally it is impossible to develop sufficient strength by means of bolts located within the normal depth of the rafter section. Hence a haunched rafter is usually utilized to increase the lever arm of the bolt group. This will allow consequently to use efficiently the rafters.

However in the following sections the analyses of the different eave and apex connections are presented.

5.7.1. Eave Connections:

Different eave connection configurations are available and can be classified as follows:

- (a) The welded eave connections.
- (b) The bolted end-plate eave connections.
- (c) The splice-plate eave connections.

5.7.1.1. The Welded Eave Connections:

The welded eave connections will require field welding which is rarely utilized in industrial building constructions. However, there exists different configurations of eave welded connections as presented in the paragraph below:

As has been previously outlined in Section 5.6 for the corner connection (namely here eave connection) five different configurations were presented. In order to avoid repetition, one can refer to Section 5.6.3.1 as well as to example 5.15.

5.7.1.2. The Bolted End Plate Eave Connections:

The most popular eave connection is the bolted end plate eave connection. Fig. 5.39 shows different configurations of these end-plate eaves connections.

Configurations (A) and (B) of the flushed and the extended end plates have been previously analyzed in Sections 5.8.1 and 5.8.2 where the plane of connection is vertical. Similar configurations would result with a horizontal connection by extending the beam rather than the vertical column. The vertical plane connection is recommended where this permits standardization for single and multi-bays frames.

As has been previously outlined in Section 5.8.1 the flushed end plate configuration has the main disadvantage that the short lever arm of the bolt group will be insufficient to develop the full beam capacity. While the extended end plate configuration will accommodate this situation where the increased lever arm of the bolt group can provide for the common spans the required beam strength. The edge purlin (eave purlin) must be displaced away from the face of the column in order to prevent interference with the roof covering material (shown by the dotted line of Fig. 5.39.b).

Configuration (C) namely the haunched-end-plate eave connection is the most popular one. Fig. 5.39.c, illustrates a typical haunched connection where a short haunch was employed to produce an adequate lever arm of the bolt group. The haunch configuration will consequently reduce the maximum moment in the rafter and hence its size. The common practice is to make the haunch at the connection interface approximately twice the depth of the basic rafter section. For the analysis refer to Section 5.8.3. where the strength, stability and stiffening requirements were presented.

Refer to examples 5.17 and 5.18.

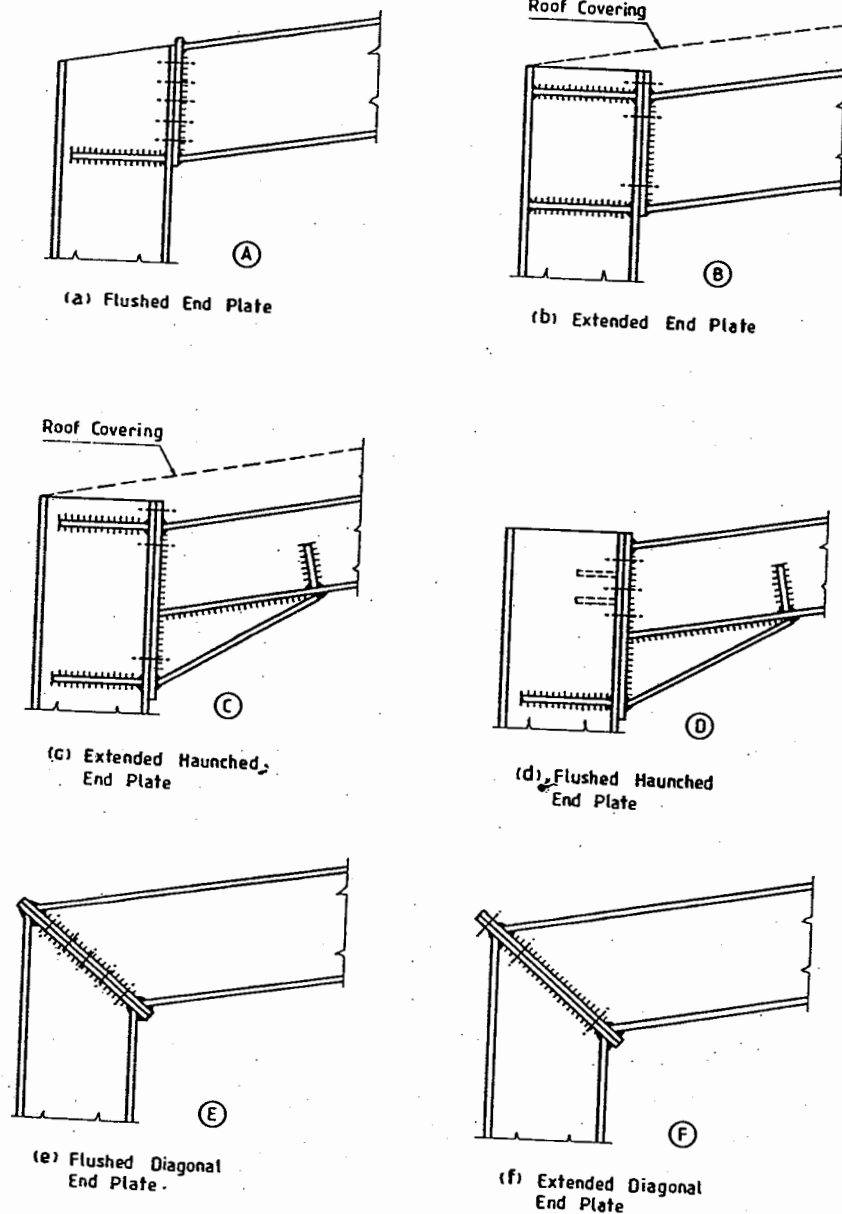


Fig. 5.39

It is to be noticed that due to the greater depth available for the haunched end plate type, it is possible to transfer the tension force of the rafter down. The traditional outer row of bolts can be omitted. Tests have shown that for such a detail as shown in Fig. 5.39.d, the second row of bolts away from the tension flange may approximately reach the same load as the first row. This usually means that the tension of the rafter will be spreaded on a greater distance, thus the upper tension stiffener can be canceled. However small stiffeners (shown dotted in Fig. 5.39.d) may be necessary to increase the flexural capacity of the column flanges.

Figs. 5.39.e, f, illustrate a flushed and an extended end-plate connections having a diagonal plane of connection. The end plates will act as diagonal stiffeners which seems apparently to be the most economical configuration. But unfortunately these types have the main disadvantage that the inner corner tend to buckle laterally. Having no upper or side purlins to attach the inner corner, we must check this region against the lateral torsional buckling considering the unbraced length equal to twice the distance between this inner corner and the adjacent upper purlin.

In addition this type of connection is difficult to erect. Thus it is not recommended to use these two last configurations due to the stability and erection difficulties.

Refer to Examples 5.19 and 5.20.

5.7.1.3. The Splice Plate Eave Connections:

Fig. 5.40, illustrates the common eave connection that makes use of a splice plate. The beam moment is replaced by two



opposite and equal forces ($T = C = \frac{M_b}{d_b}$). The beam tension force is transmitted to the upper splice plate "1" via the group of H.S.B "3" behaving in friction. Then this tension force is transmitted to the column section via the fillet lines of welds "4". While the compression force "c" will be transmitted by bearing to the column web. To prevent the crippling of the column web a stiffener opposite to the beam compression flange is to be provided if necessary.

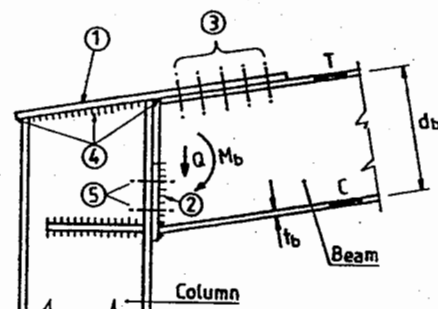


Fig. 5.40

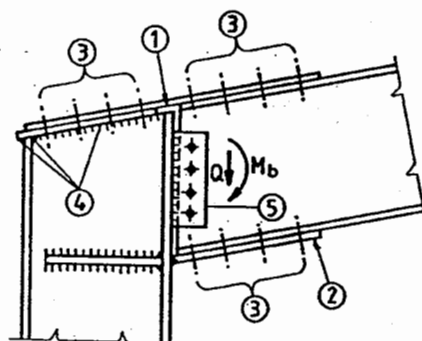


Fig. 5.41

- (1) Splice Plate.
- (2) End Plate.
- (3) High strength bolts.
- (4) Fillet welds connecting the splice plate to the column section.
- (5) High strength bolts resisting the shear Q

- (1) Upper splice plate.
- (2) Lower splice plate.
- (3) H.S.B transmitting the applied moment "M".
- (4) Fillet welds connecting tension stiffener to column section.
- (5) Cleat plate transmitting the shear Q.



Fig. 5.41 shows another configuration of splice plate eave connections. The beam tension force is transmitted to the upper splice plate "1" via group of bolts (3) acting in friction. This tension force is then transmitted to the column section via the tension stiffener and the group of H.S.B "3". On the other hand the beam compression flange will be transmitted to the lower splice plate "2" via the group of H.S.B "3". The butt weld attaching the lower splice plate to the column flange will transmit the beam compression force to the column web. The cleat web plate "5" will transmit the shearing force "Q". This detail increases the number plates and of bolts transmitting the moment "M" and hence makes the connection more expensive.

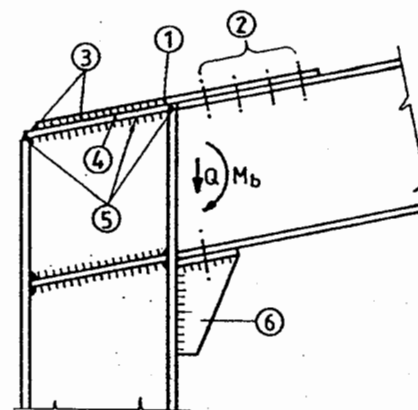


Fig. 5.42

- (1) Upper splice plate.
- (2) H.S.B transmitting the applied moment "M".
- (3) Welds connecting the splice plate to the tension stiffener.
- (4) Tension stiffener.
- (5) Welds connecting the tension stiffener to the column section.
- (6) Bracket transmitting the shear "Q".

Fig. 5.42, illustrates a splice plate configuration where the beam tension force is transmitted to the splice plate "1" via the group of H.S.B "2" acting in friction. Then the tension force is transmitted to the tension stiffener (4) via the fillet weld lines "3". The tension stiffener "4" is fillet welded to



the column section by the line of welds "5". This tension stiffener can be replaced by extending the beam tension flange. Regarding the beam compression force, this configuration assumes the transmission by direct contact. The lower bracket, "6" will transmit the shearing force "Q" to the column flange.

The design and analysis of the connections shown in Fig. 5.40, 5.41 and 5.42 are simple. To avoid repetition refer to:

Refer to Examples 5.21 and 5.22.

The other alternative is to extend the beam. The moment and shear of the column will be transmitted to the beam via the splice plate "1" and the group of H.S.B "2" as shown in Fig. 5.43 and 5.44.

Generally, for purposes of standardization the designers prefer to extend the columns where for multi-bay frames the planes of connections will be vertical for the different locations of connections.

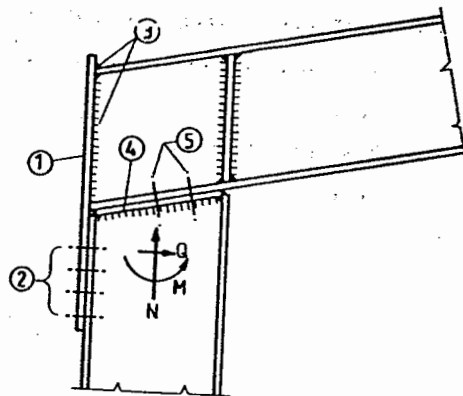
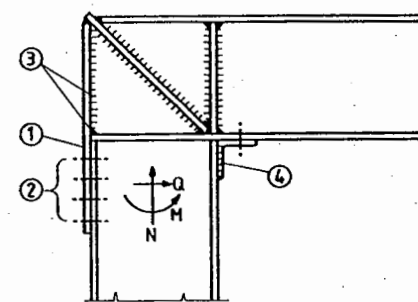


Fig. 5.43

- (1) Splice plate.
- (2) H.S.B transmitting the applied moment "M".
- (3) Fillet welds connecting the splice plate to the column section.
- (4) End-plate.
- (5) Bolts transmitting the components of "Q" and "N" to the beam.



- (1) Splice plate.
- (2) H.S.B transmitting the applied moment "M" and "N".
- (3) Fillet welds connecting the splice plate to the beam section.
- (4) Seat angle.

Fig. 5.44

5.7.2. The APEX Connections:

The classification of the apex connections can follow a logic similar to the eave connections as given below:

- (a) The butt-welded apex connections.
- (b) The end-plate apex connections.
- (c) The splice-plate apex connections.

5.7.2.1. The Butt or Fillet Welded Apex Connections:

As has been mentioned before, the field welding is rarely utilized. However, if this configuration is chosen by the designer, attention is to be concentrated with regard to the severe situation at the tension flange weld.

Fig. 5.45.a, illustrates the general arrangement of a welded apex connection where a vertical stiffener is provided. This vertical stiffener is fillet welded at both sides to the beam webs. The size "s" of this fillet weld must be capable to transmit the induced force "F" :

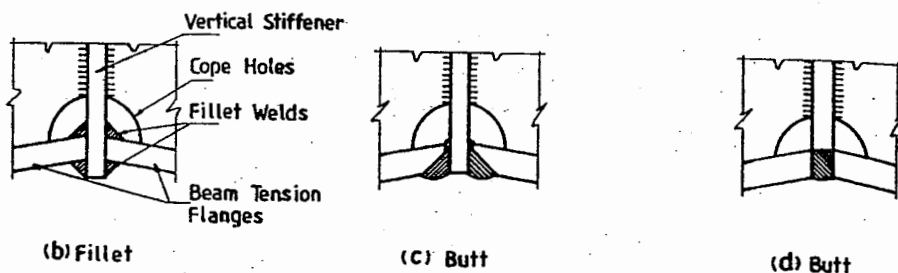
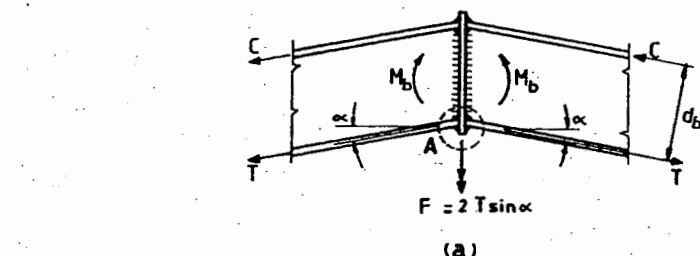


Fig. 5.45

$$F = 2 T \sin \alpha \quad (a)$$

where:

$$T = \frac{M_b}{d_b} \quad (b)$$

Hence:

$$s = \frac{M_b \cdot \cos \alpha \cdot \sin \alpha}{d_b \cdot d_b \cdot q_w} \quad (c)$$

Where: $q_w = 0.2 F_U =$ allowable fillet weld stress.

On the other hand the beam flanges can be either fillet welded to the vertical stiffener as shown in Fig. 5.45.b or butt welded to it as illustrated in Fig. 5.45.c, d.

For the computation of the fillet weld size "s" of Fig. 5.45.b, we can apply in the following:

$$S = \frac{F}{2 b_b \cdot q_w} \quad (d)$$

Where: $b_b =$ width of the rafter flange.

5.7.2.2. The End-plate Apex Connections:

The performance requirements of the apex connections are identical to those of the eave connections, that of elastic strength and stiffening elements. However an additional fact, the designer must bear in mind that the deformation occurring within the apex connection can produce a magnified downward deflection relative to the eaves. The control of these local deformations are depending on the design details of the connection in the tension zone.

Fig. 5.46.a, illustrates a typical detail of an apex connection which seems on first appearances to be similar to the extended end-plate of Section 5.8.2. However the similarity of analysis is true if the end plate is thick enough. Unfortunately, tests have shown that at ultimate loads, the resulting bolt load distribution is quite different compared to the elastic analysis previously outlined in Section 5.8.2 in which the bolts located beyond the tension flange are constrained from moving towards the tension flange by the stiff column flange on which the bolts bear while for the configuration shown in Fig. 5.46.a this movement is not prevented. Initially this type of connections behaves in an elastic manner producing a similar bolt load distribution to that depicted by the extended end-plate of Section 5.8.2. But as the moment increases the more flexible extended portion of the end-plate bends are in double curvature causing a gap to open up between the end-plates, local to the tension flanges and hence precautions are to be provided.

In order to prevent the separation happening between the connected end-plates two stiffeners connecting the end plates to the beam tension flanges can be utilized as shown in Fig. 5.46.b. By this method part of the tension force in the rafter flange is transferred via the stiffening plate to the end-plate and then to the bolts, hence reducing the flexural action in the end-plate. Nevertheless, the load still has to pass from the wide tension flange into the thin stiffening plate element and then by double curvature bending of the end-plate into the bolts. The weld sizes and thickness of stiffener are crucial due to extremely large tension forces being transmitted.

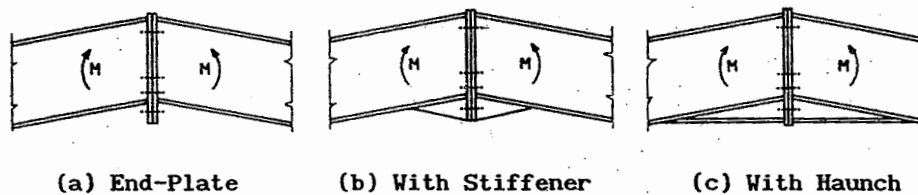


Fig. 5.46. Apex Configurations With End Plates

The best configuration of an apex connection is that shown in Fig. 5.46.c. The beam tension flange is reinforced by a small haunch usually cut from the same rolled section as the rafter. The tension force will pass via the haunch to the outermost bolts, whose distribution tends to be elastic as previously outlined in Section 5.8.2.

Refer to Example 5.23, 5.24 and 5.25.

5.7.2.3. The Splice-Plate Apex Connections:

These types of connections are rarely utilized, they have shown to be less economic than the end-plate configurations of Section 5.7.2.2.

Fig. 5.47.a shows an apex connection using two lower splice plates to transfer the tension force via a group of high strength bolts (n_1) where:

$$n_1 = \frac{M_b}{(d_b / \cos \alpha + d_h) \cdot P_s} \quad (e)$$

Where:

d_h = the haunch depth.

P_s = the friction resistance of a H.S.B.

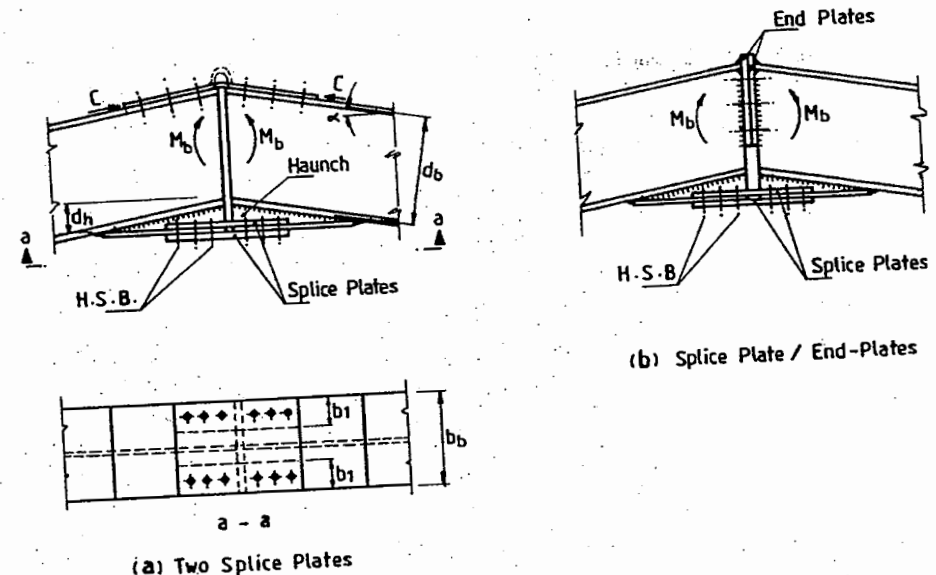


Fig. 5.47. Apex Using Splice Plates

The splice plate thickness "t" can be computed using the following relation:



$$F_t = \frac{M}{(d_b / \cos \alpha + d_h) \cdot (b_b + 2 b_1) \cdot t} \quad (f)$$

Where:

F_t = the steel allowable tensile stress.

Regarding the path of the beam compression force "c" the use of an upper splice plate is not recommended where the compressive load can force this splice plate to bend as shown by the dotted lines in Fig. 5.47.a. The compressive load should preferably be transferred by bearing via two end plates as shown in Fig. 5.47.b.

Refer to Example 5.26.

Group (V) Beam -to- Beam connections :-

5.8. Continuous Grid Beam Connections:

Continuous grid beam connections will behave exactly as a rigid beam-to-column connection. The connection must be capable to transmit the reactions of the secondary beams to the main beam and also must ensure the development of the moment continuity between the two secondary beams meeting at the connection.

In the following sections a brief presentation of the common types of connections is provided.

5.8.1. Welded Rigid Grid Beam Connections:

Actually it is rare to use field welded connections, but if the designer is sure that the inspection provisions are provided, hence one of the connections shown in Fig. 5.48 and 5.49 may be chosen.



Fig. 5.48.a, illustrates a direct butt weld configuration where the secondary beam tension force is transferred from the right secondary beam to the left one via the butt welds. If the butt weld is of the excellent type along the breadth " b_1 " of the secondary beam, hence we are sure that the tension force is transmitted. On the other hand the secondary beam compression force is transferred via the lower butt welds and the reactions via the vertical lines of fillet welds.

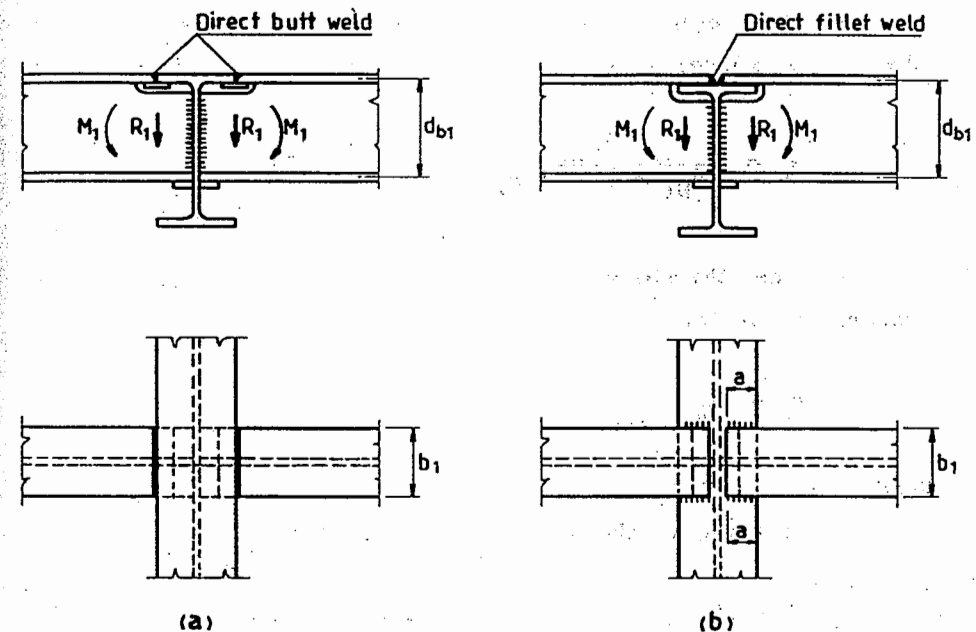


Fig. 5.48. Continuity of Beam-To-Beam by Direct Welding

Fig. 5.48.b shows a direct fillet weld configuration. The secondary beam tension force is transferred to the upper flange of the main beam via three lines of fillet welds ($2a + b_1$)



behaving in concentric shear. The size (s) of these welds can be chosen equal to the flange thickness of the secondary beams. The reaction are transmitted in a similar way as for the connection of Fig. 5.48.a. It is to be noticed that backing bars are provided for the butt welds as shown in Fig. 5.48.a and b.

Fig. 5.49.a shows a common grid connection using an upper splice plate. Two aspects are to be considered:

(a) The fillet weld lengths connecting the splice plate to the secondary beam tension flange must be capable to transfer the tension force; i.e:

$$q_w = \frac{M_1}{d_{b1} \times (2L_1 + b_1) \cdot s} \quad (a)$$

(b) The thickness of the splice plate must ensure the transmittion of the tension force:

$$f_t = \frac{M_1}{d_{b1} \cdot t_1} \quad (b)$$

Where:

t = the thickness of the splice plate.

f_t = the actual steel tensile stress. ($\leq 0.58 F_y$)

One can determine first the required safe thickness of the splice plate (t) and then choose (s = t).

A lower seat angle of minimum size can be used as a backing bar.

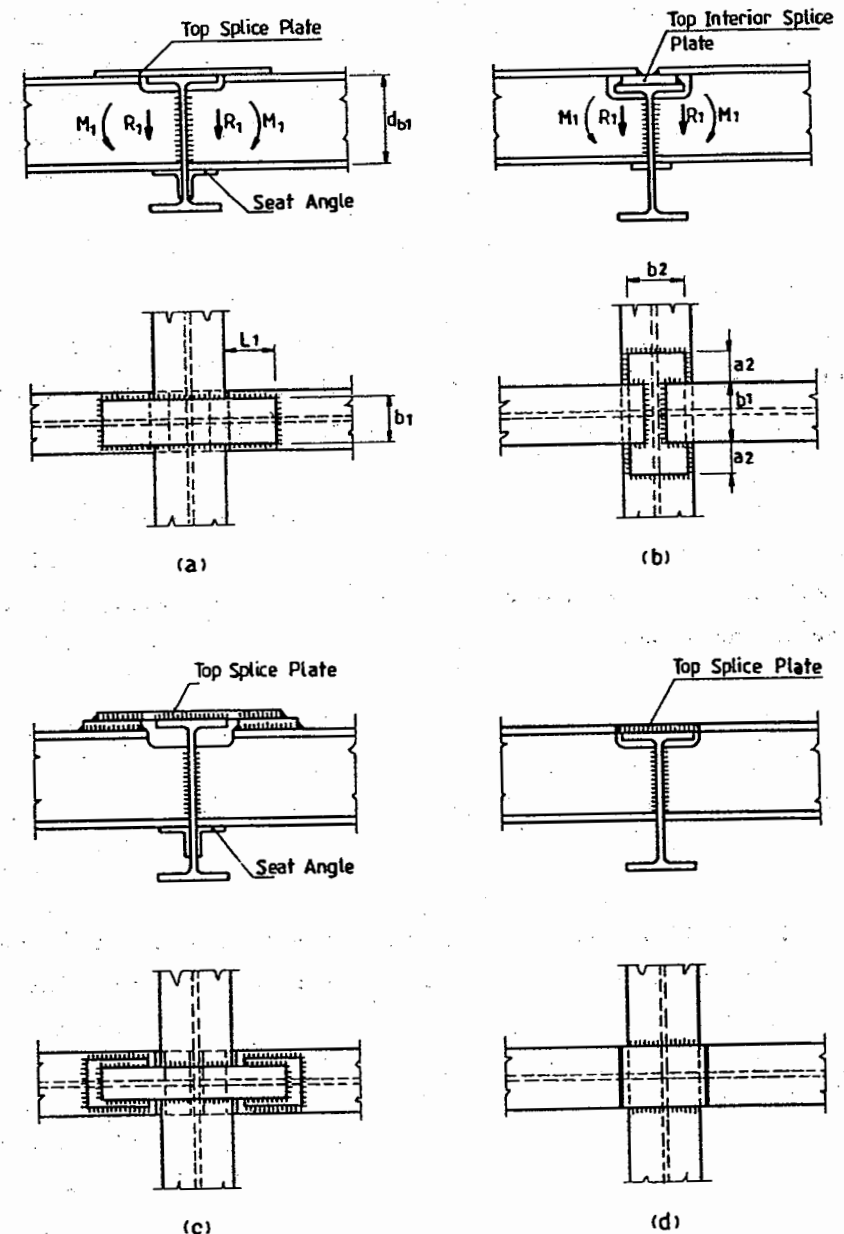


Fig. 5.49. Continuity of Beam-To-Beam
by Welded Splice Plates



Fig. 5.49.b, illustrates another configuration of grid connections using a splice plate. The tension force of the secondary beam is transmitted to the top plate via a fillet weld line of length " b_1 " and then to the flange of the main beam via three fillet weld lines ($b_2 + 2a_2$). The principles of design remain unchanged.

Fig. 5.49.c shows a configuration of a splice-plate detail where the tension force is transmitted via two splice-plates fillet welded to the corresponding steel structural elements. The transmission of forces is indirect for this detail.

Fig. 5.49.d, illustrates a top splice-plate butt welded to the tension flange of the secondary beam. For butt welds of excellent type, the tension force is transferred safely from one side to the other.

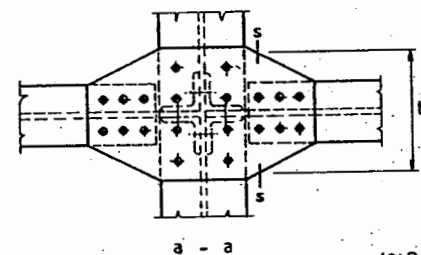
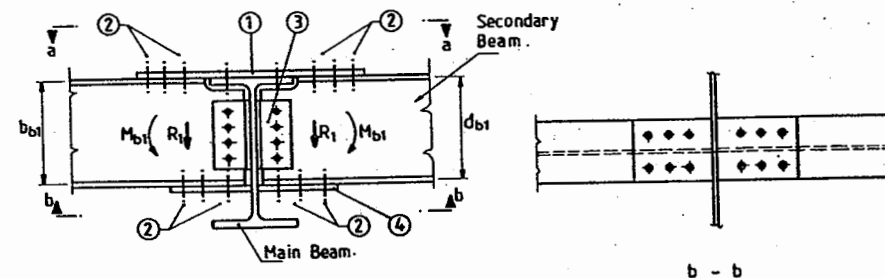
5.8.2. Bolted Rigid Grid Beam Connections:

Three typical rigid grid beam connections are illustrated in Fig. 5.50. Details (A) and (B) are utilized if coplanar top flanges are required.

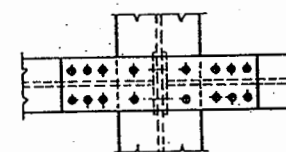
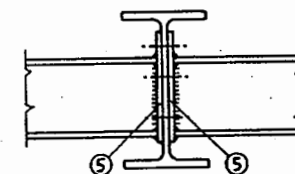
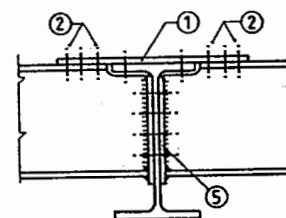
Fig. 5.49.a shows a typical connection where an upper splice plate is used to transfer the secondary beam tension force via the group of ordinary bolts "2". Two aspects are to be analyzed:

(a) The thickness of the upper splice plate (t) must ensure the transmission of the tension force using ($F_t = 0.58 F_y$)

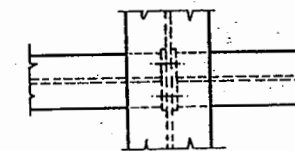
$$F_t = \frac{M_{b1}}{d_{b1} \times (b_s - n\phi) \times t} \quad (c)$$



(a) Detail (A)
Two Splice Plates.



(b) Detail (B)
Splice Plate / End-Plate



(c) Detail (C)
Two End Plates

- (1) Upper Splice Plate.
- (3) Framing Angles.
- (5) End-Plate.

- (2) Group of H.S.B or O.B.
- (4) Lower Splice Plate.

Fig. 5.50. Rigid Grid Beam Connections

Where f_t = the actual steel tensile stress. ($\leq 0.58 F_y$)

b_s = the breadth of the splice plate at Section s-s.

n = the number of ordinary bolts passing through s-s

= 2 bolts as shown in Fig. 5.62.a.

ϕ = diameter of bolts "d" + 2 mms.

t = the thickness of the upper splice plate.

(b) The group of bolts (n_1 ordinary bolts) must transmit the tension force:

$$n_1 = \frac{M_{b1}}{d_{b1} \cdot R_{least}} \quad (d)$$

Where: n_1 = the total number of ordinary bolts.

R_{least} = the shear or bearing resistance of one ordinary bolt or the friction resistance (P_s).

For the transfer of the compression force a lower splice plate is provided. Opposite to the lower splice plate, the web of the main beam is cut in order to permit the extension of this splice plate. The lower splice plate being subjected to a compression force, hence the gross area is to be considered. This means that if the thickness and the breadth are chosen equal to the corresponding secondary beam dimensions, no additional checks are required.

If high strength bolts of the friction type are utilized, the gross area is to be considered for both the tension and the compression zones and hence the breadth "b" and the thickness "t" of the upper splice plate can be chosen similar to the



corresponding secondary beam dimensions. The transmission of the secondary beam vertical reactions are ensured by two framing angles "3" as illustrated in Fig. 5.50.a.

This configuration is rarely used since the number of bolts is excessive. In addition the erection and fabrication costs are relatively high.

Fig. 5.50.b, illustrates a coplanar top flange connection where a combined splice-plate/end-plate are used. The required number of high strength bolts "2" as well as the computation of the splice-plate thickness will follow exactly detail (A). Two end plates fillet welded to the secondary beams will ensure the transfer of the vertical reactions by friction. While the secondary beam compression force is transmitted via the end plates by bearing.

If the secondary beams may be set below the top flange of the main beam, the most economic connection is to utilize extended end plates as shown in Fig. 5.50.c (Detail "C"). The design of the end plates will follow exactly Section 5.8.2.

5.9. General Comments Regarding the Analysis of the Moment End Plate High-Strength Bolted Connections:

In 1991 two different studies concerning the flushed and the extended end plate moment connections were conducted in the Steel Division of the Faculty Of Engineering at Cairo University. The first study was analyzing the rigid high strength bolted connections and will be presented in the following paragraphs. While the second study was a parametric analysis dealing with Semi-Rigid connections and will be presented in Chapter 6.

In the first research study namely << Stress Analysis of



Steel Moment Connections (12) the moment connections were analyzed using a three dimensional finite element approach. Rectangular plate bending elements were utilized having four corners as the primary nodes. Six degrees of freedom were established at each node, to allow for space interaction between the adjacent elements. High strength bolts, welds, and contact between any two plates were assimilated by linkage elements. These linkage elements were joining two nodes by a set of linear springs in the three space translations and the corresponding rotations. Each stiffness spring was deduced for the corresponding linkage. In fact a model representing each linkage element was proposed in order to estimate a realistic value for the stiffness.

The stiffness of the butt and fillet welds were computed as previously outlined in Section 5.5, while the normal stiffness of the bolt shaft ($K_x = K_b$) was computed considering the governing parameters: the modulus of elasticity "E", the total bolt length "L" and the bolt stress area " A_b ". In order to ensure very small relative displacements between the connected nodes in the shear direction, K_y , K_z and $k_{\theta x}$ namely the shear stiffnesses were chosen very high.

On the other hand the plate stiffness was computed considering that clamping forces are induced due to the tightening of the bolts and hence cause a fit contact between areas. Assuming that the bolt head and the nut distribute the applied forces at a reasonable slope to the plates in contact, the stiffness of the plates in contact " K_p " was easily computed.

Different beam-to-column connections were analyzed using the extended and the flushed end plate configurations. The following parameters were governing the analysis:



- (a) The end plate thickness.
- (b) The location of the stiffeners at the compression and at the tension beam flanges.
- (c) The bolts diameter.
- (d) The geometrical configuration of the bolts location.

The main results of this study can be summarized as follows:

Prying Force Effect:

The prying forces were found to generate on the vertical side edges of the end plate coinciding with the column flange. Hence, these prying forces are governed by the deformations of both the column flange and the end plate. Fig. 5.51, illustrates a deformation pattern of one of the studied connections where the lateral deformation of the column flange caused by the bolts tension forces is causing a contact compressive zone at the edge between the column flange and the end plate (prying edge) and hence is creating the prying forces.

The prying force was found to be unequally distributed over the two rows at the neighborhood of the beam tension flange with the greater portion shared by the lower row.

The external induced tension force in these two rows of bolts due to an external applied moment can be estimated as a ratio of the beam tension flange and the total prying force. An acceptable share of 60 and 40 percent for these two rows can be applied. It was also suggested to calculate the prying force as a percentage of the beam tension force as 20 and 10 percent for connections prevented or supplied with stiffeners opposite to the beam tension flange respectively.

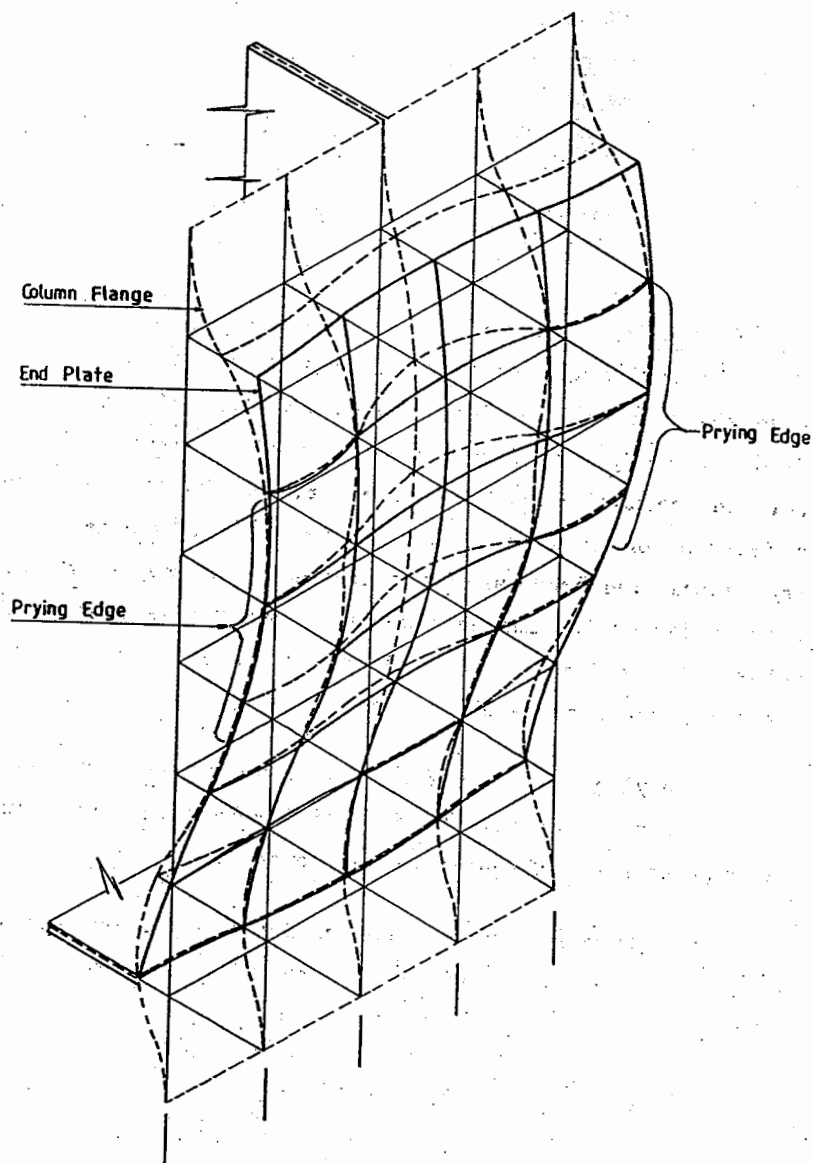


Fig. 5.51. Deformation Pattern of the End Plate With the Column Flange (Detail 2)

End Plate Thickness:

The end plates were categorized to two types according to the end plate thickness: "thin" and "thick" end plates. The beam tension force is distributed differently for each category. In the thin end plate category, the upper two rows of bolts adjacent to the beam tension flange share equally the total flange force. As for the thick end plate connections, the third row as well as the two upper rows was sharing the tension force.

It was found that the increase of the end plate thickness leads to a corresponding increase in the external tension forces induced in the outer and the third rows while for the inner row adjacent to the beam tension flange the induced force decreases. This was attributed to the configuration of the deformed shape of Fig. 5.51.

Fig. 5.52, illustrates the induced principal stresses in an end plate of one of the studied configurations. The end plate thickness was designed using an elastic analysis where the bending stresses were chosen equal to the permissible stress. Fig. 5.52 showed a concentration of stresses exceeding the allowable stress at the vicinity of the beam tension flange. Hence, it was suggested for an approximate analysis to consider the end plate behaving as a beam simply supported at the outer two rows.

Geometrical Configuration of Bolts:

It was suggested to use the maximum number of bolts per row with the least diameter in order to create a convenient rigidity at the bolts location. This will consequently decrease the induced bending stresses in the end plate.

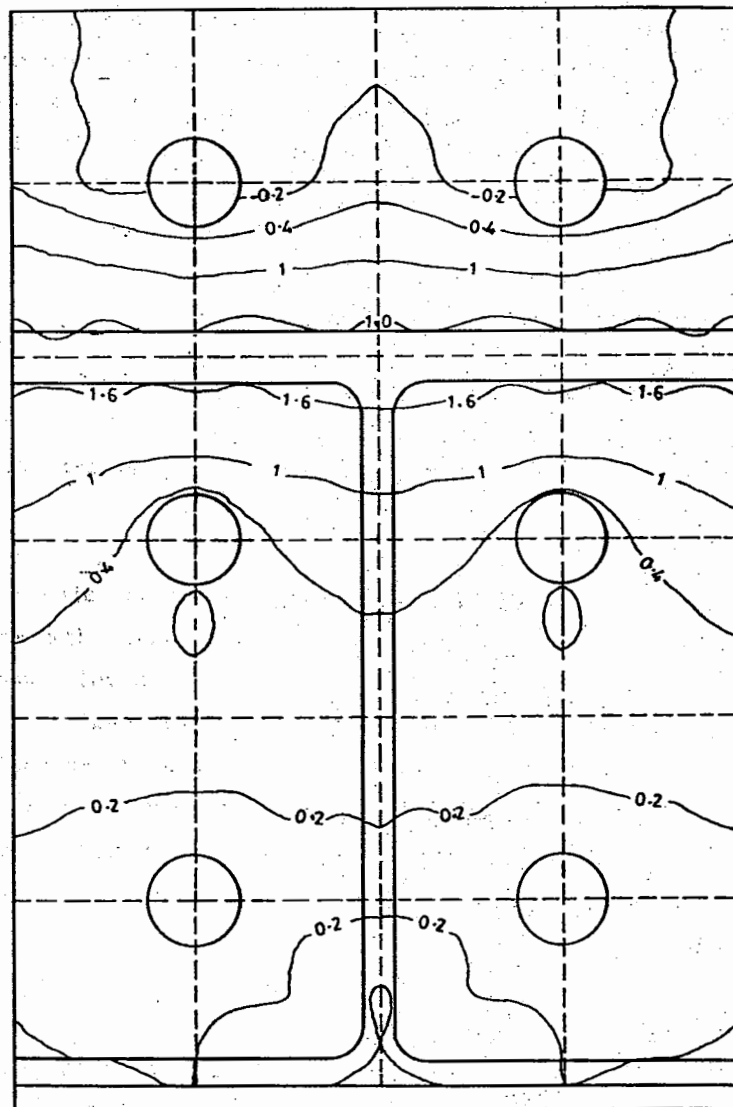


Fig. 5.52. Maximum Principal Stresses
in the End Plate (Detail 1)

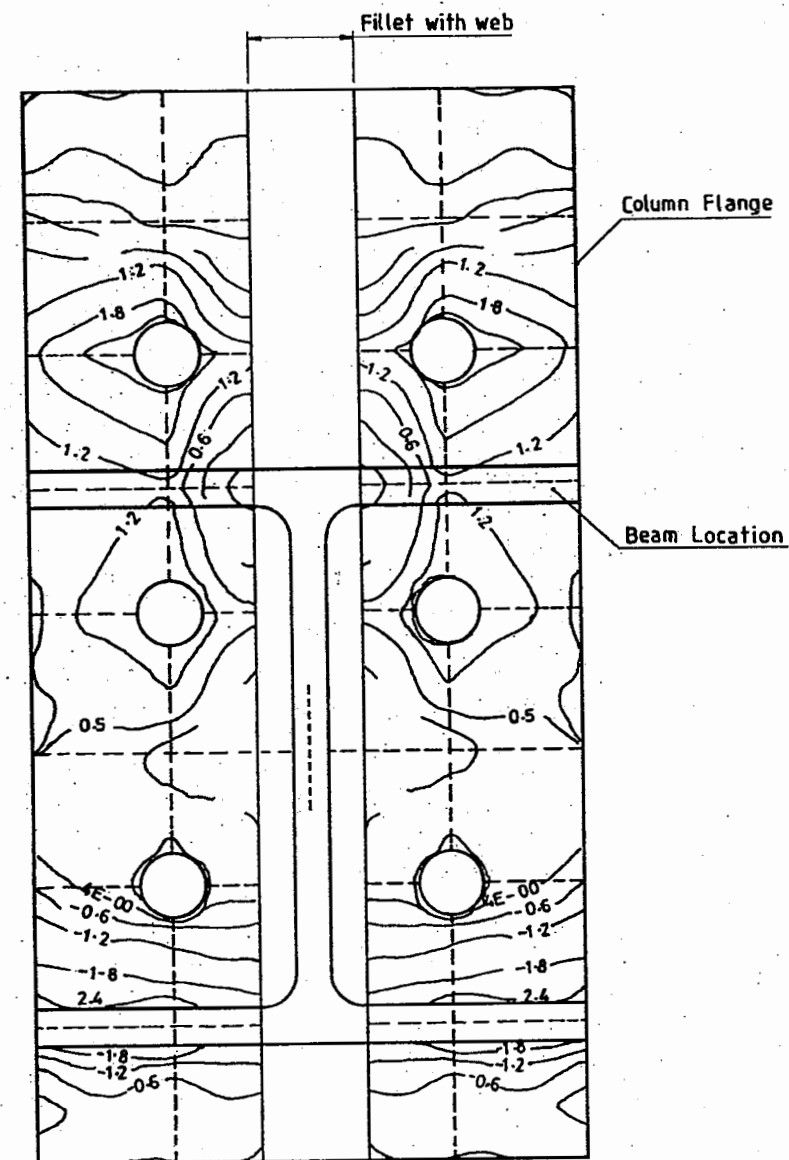


Fig. 5.53. Principal Bending Stresses
in Column Flange (Detail 1)

Stiffener's Location:

The study has proved that the use of tension stiffeners opposite to the beam tension flange reduces greatly the prying force. Hence it was suggested to use these stiffeners even if the column flange is safe against bending. Fig. 5.53, illustrates the contour lines of the principal stresses of the column flange where two stiffeners opposite to the beam flanges are utilized. It can be observed that the induced stresses exceed the permissible stress at the vicinity of the beam compression flange.

The aim of this discussion is to notify that still this type of connections needs additional research work. We possess actually in the Steel Division of Cairo University a refined mathematical model and a computer program based on realistic steel structural elements behavior. We do believe that the lack of definite design rules can be accommodated by an extensive analysis of a large number of connections including the different parameters affecting the behavior.

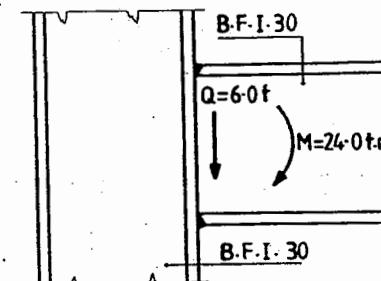
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Example (5.1):

Design the welded beam-to-column connection shown in the Fig. below using butt welds of the excellent type.

SolutionDesign of Flange Welds:

Using excellent butt welds the size of welds being equal to the flange thickness, thus no checks are required.

Design of Web Welds :

In order to transmit the shearing force $Q = 6.0$ tons from the web beam to the column flange we can use one of the following configurations:

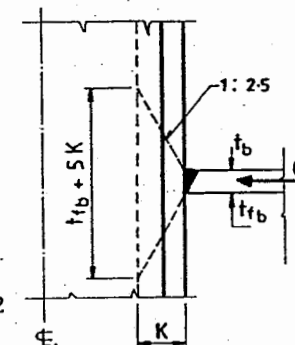
- (i) Framing angles welded to the beam web and bolted to the column flange.
- (ii) Cleat plate bolted to the beam web and welded to the column flange.

For the design of the different structural components, refer to Chapter 4.

Crippling of the Column Web:

$$\begin{aligned} \text{Area of applied force} &= b_b \times t_b \\ &= 30 \times 2 = 60 \text{ cm}^2 \end{aligned}$$

$$\begin{aligned} \text{Resisting area} &= t_{wc} (t_{fb} + 5k) \\ &= 1.2 (2 + 5 \times 2 \times 2) = 26.4 \text{ cm}^2 \end{aligned}$$



N.B. The notation of the beam thickness (t_{fb}) is exactly the same as (t_b).



Hence 2 horizontal stiffeners are to be utilized.

$$b_{st} \times t_{st} = \frac{60 - 26.4}{2} = 16.8 \text{ cm}^2$$

$$b_{st} \leq \frac{30 - 1.2}{2} \leq 14.4 \text{ cms}$$

$$\text{Take } b_{st} = 14 \text{ cms} ; t_{st} = \frac{16.8}{14} = 1.2 \text{ cms}$$

Check of local buckling $b_{st}/t_{st} = 14/1.2 = 11.6 < 25/\sqrt{F_y}$

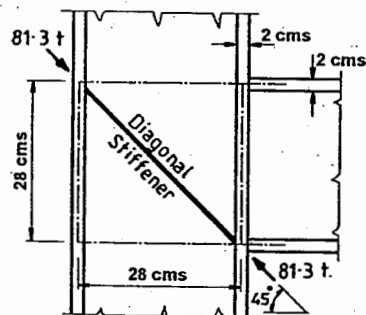
Bending of Column Flange:

To prevent the bending of the column flange ; the column flange thickness must at least be equal to $t_{bc} = 0.4 \sqrt{b_b \cdot t_b}$

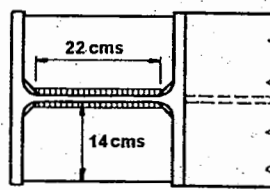
$$t_{fc} \geq 0.4 \sqrt{b_b \cdot t_b} \geq 0.4 \sqrt{30 \times 2} \geq 3.09 \text{ cm} > t_{fc} \text{ (Unsafe)}$$

Hence use 2 horizontal stiffeners having the same dimensions as for crippling i.e ($b_{st} = 14 \text{ cms} ; t_{st} = 1.2 \text{ cm}$).

Shear of the Corner Web (Stability):



Diagonal Stiffeners



Cover Plates



$$\text{Applied shear} = \frac{M}{d_b} = \frac{24 \times 100}{(30-2)} = 85.71 \text{ tons.}$$

$$\text{Resisting force} = t_{fc} \cdot d \cdot q = 1.2 \times 30 \times 0.84 = 30.24 \text{ tons.}$$

$$\text{Remaining force} = 85.71 - 30.24 = 55.47 \text{ tons.}$$

If diagonal stiffeners are utilized:

$$F_D = \frac{55.47}{\cos 45} = 78.45 \text{ tons.}$$

$$A_{st(D)} = b_{st} \cdot t_{st} = \frac{78.45}{2 \times 1.4} = 28.02 \text{ cm}^2.$$

$$\text{Choose : } b_{st} = 14 \text{ cms.} ; t_{st} = 2.0 \text{ cms.}$$

If 2 cover plates of thickness "t" are utilized:

$$t = \frac{55.47}{2 \times (30 - 4 \times 2) \times 0.84} = 1.5 \text{ cms} \quad \text{choose } t = 1.5 \text{ cms.}$$

Example (5.2):

Design the welded connection shown in the figure below using the following different configurations:

- A diagonal stiffener.
- A Morris stiffener.
- K stiffener.

Solution

- Use butt welds of the excellent type to attach the beam flanges to the column flange.

- Use a framing angle configuration to transmit the shear $Q = 6.0 \text{ t}$ from the beam web to the column flange.

- Crippling of column web:

$$\text{Area transmitting the applied force} = 30 \times 2 = 60 \text{ cm}^2.$$

$$\text{Resisting area} = 1.5(2+5 \times 2 \times 2.8) = 1.5 \times 30 = 45 \text{ cm}^2.$$

Hence use two horizontal stiffeners "B":

$$A_{st} = \frac{60 - 45}{2} = 7.5 \text{ cm}^2;$$

(B)

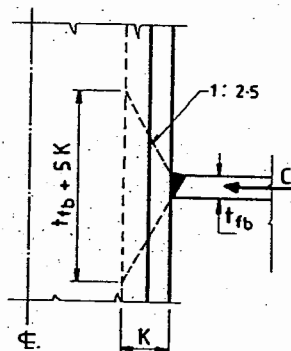
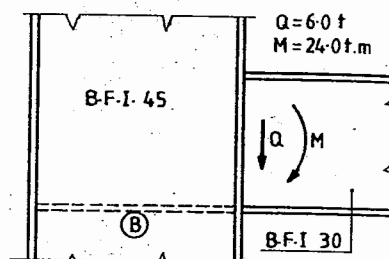
$$\text{choose } b_{st} = \frac{30 - 1.5}{2} \leq 14.25 \text{ cms}$$

$$\text{Take } b_{st} = 14 \text{ cms.}$$

To prevent the local buckling:

$$t_{st} = \frac{14}{16.1} = 0.87 \text{ cm} \quad \text{choose } t_{st} = 1.0 \text{ cm}$$

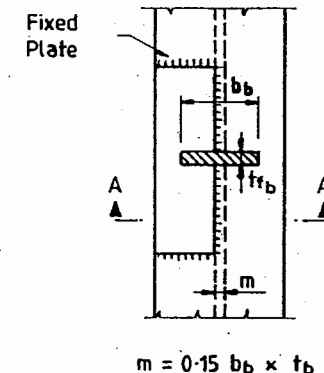
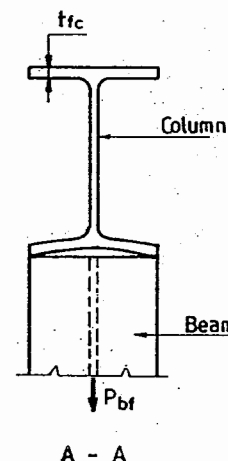
Bending of Column Flange:



$$t_{fc} \geq 0.4 \sqrt{b_b \cdot t_b} \geq 0.4 \sqrt{30 \times 2} \geq 3.09 \text{ cms.}$$

Unsafe, hence use 2 horizontal stiffeners whose dimensions are those opposite to the compression beam flange.

It is to be noticed that the common practice using stiffeners to prevent the bending of the column flange as those utilized opposite to the compression flange is an approximate approach.



The exact analysis must follow the first principles i.e.:

$$T_{ultimate} = 30 \times 2 \times 2.4 = 144 \text{ tons.}$$

$$\begin{aligned} \text{Resisting force} &= 0.8 (7 \times t_{fc}^2 \times F_y + 0.15 b_b \cdot t_{fb} \cdot F_y) \\ &= 0.8 (7 \times 2.8^2 \times 2.4 + 0.15 \times 30 \times 2 \times 2.4) \\ &= 122.65 \text{ tons.} \end{aligned}$$

$$\text{Remaining force} = 144 - 122.65 = 21.35 \text{ tons.}$$

Hence : $A_{st} = \frac{21.35}{2 \times 1.4} = 7.62 \text{ cm}^2$.

Choosing $b_{st} = 14 \text{ cms}$; $t_{st} = 1.0 \text{ cm}$ will satisfy the exact ultimate analysis.

Shear resistance of Corner Web (Stability):

Applied shear = $\frac{M}{d_b} = \frac{24 \times 100}{(30 - 2)} = 85.71 \text{ tons.}$

Resisting force = $1.5 \times 45 \times 0.35 \times 2.4 = 56.7 \text{ tons.}$

Remaining force = $85.71 - 56.7 = 29.01 \text{ tons.}$

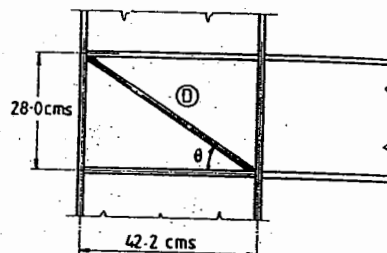
(a) Using a Compressive Diagonal Stiffener "D"

$F_D = \frac{29.01}{\cos\theta} = 34.82 \text{ tons. (}\cos\theta = 0.833\text{)}$

$A_{st(D)} = \frac{34.82}{2 \times 1.4} = 12.43 \text{ cm}^2$.

Take $b_{st} = 14 \text{ cms}$; $t_{st} = 0.9 \text{ cms.}$

$b_{st} / t_{st} = 14 / 0.9 = 15.55 < 16.1$



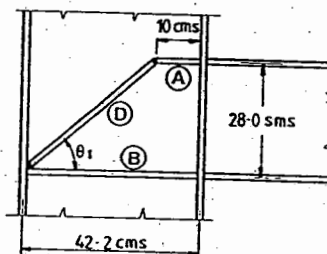
(b) Using a Tension Diagonal Stiffener

"MORRIS TYPE"

Use stiffeners "B" as previously outlined (2 stiffeners $14.0 \times 0.9 \text{ cms.}$).

Stiffener D is dimensioned to resist the remaining shearing force as previously computed. Hence use the following relation:

$F_D = \frac{29.01}{\cos\theta_1} = 38.47 \text{ tons. (}\cos\theta_1 = 0.754\text{)}$



$A_{st} = \frac{38.47}{2 \times 1.4} = 13.74 \text{ cm}^2$

Take $b_{st} = 14 \text{ cms}$; $t_{st} = \frac{13.74}{14} = 1.0 \text{ cm}$; $b_{st}/t_{st} = 14 < 16 \text{ O.K.}$

$F_A = 21.35 \text{ tons}$; hence F_D governs the design.

As portion "A" and "D" are forming one plate unit, hence choose for portion "A" the same dimensions as "D".

In reality, these values are conservative due to the high value of the remaining tension which was based on an ultimate analysis.

If allowable stress values are utilized we get:

$F_A = \frac{21.35 \times 1.4}{2.4} = 12.45 \text{ tons}$; F_D governs the design.

$A_{st(A)} = A_{st(D)} = \frac{38.47}{2 \times 1.4} = 13.74 \text{ cm}^2$.

Choose: $b_{st} = 14.0 \text{ cms.}$; $t_{st} = \frac{13.74}{14} = 1.0 \text{ cm.}$

$b_{st} / t_{st} = 14 / 1 = 14.0 < 16.1 \text{ (O.K.)}$

(c) Using a "K" Stiffener

* Based on ultimate stress analysis :-

Remaining force to proportion stiffener "A" based on ultimate stress analysis (opposite to beam tension flange) :-

$F_{st(A)} = 21.35 / \cos\theta_2 = 22.47 \text{ tons. ----- (a)}$

Remaining Force in Stiffener "B":opposit to beam compression flange Based on Ultimate Approach:

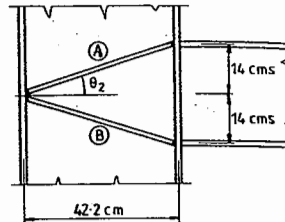
Remaining compression force = $(60 - 45) \times 2.4 = 36 \text{ tons}$

$b_b \cdot t_b \quad t_{wc} (t_b + 5k) \quad f_y$

$$F_B = \frac{36}{\cos \theta_2} = 37.9 \text{ tons} \text{ ----- (b)}$$

Remaining shearing force = 32.53 tons.

$$F_A = \frac{29.01}{\cos \theta_2} = 30.52 \text{ tons} \text{ ----- (c)}$$



Hence (b) governs the design of stiffeners (A) and (B)

* Based on allowable Stresses:

Remaining shearing force = 29.01 tons.

$$\begin{aligned} \text{Remaining compression force} &= \frac{M}{d_b} - F_{crp} \cdot t_{wc} \cdot (t_b + 5k) \\ &= 85.71 - 1.8 \times 1.5 \times (2 + 5 \times 2 \times 2.8) = 4.71 \text{ tons.} \end{aligned}$$

$$\begin{aligned} \text{Remaining Force opposit to beam tension flange} &= 21.35 \times (F_c / F_y) \\ &= 21.35 \times (1.4 / 2.4) = 12.45 \text{ tons} \end{aligned}$$

Remaining shearing force governs = 29.01 tons (governs the design)

$$\text{Hence } F_B = F_A = \frac{29.01}{\cos \theta_2} = 30.52 \text{ tons.}$$

$$\text{Hence } A_{st} = \frac{30.52}{2 \times 1.4} = 10.9 \text{ cm}^2.$$

$$\text{Choose } b_{st} = 14, \quad t_{st} = \frac{10.90}{14} = 0.8 \text{ cms.}$$

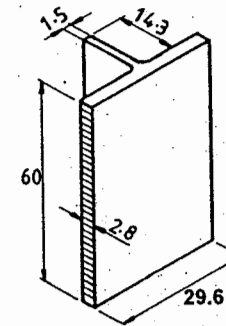
$$b_{st} / t_{st} = 14 / 0.8 = 17.5 < 16.1 \text{ unsafe ; take } t_{st} = 0.9 \text{ cms}$$

N.B. It is to be noticed that for the ultimate stress analysis F_{crp} was taken equal to F_y . On the other hand for the computation of the remaining shearing force for both the ultimate and allowable analysis the applied force was equal to (M/d_b) and the allowable shear stress was " $q = 0.35 F_y$ ". These utilized assumptions was due to lack of accurate known behaviour for such type of connections.

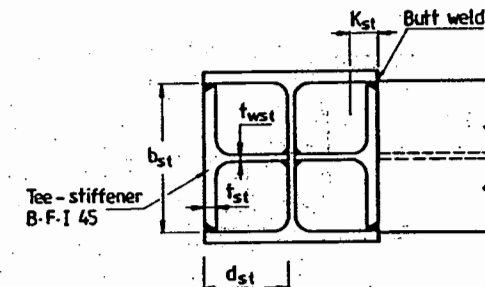
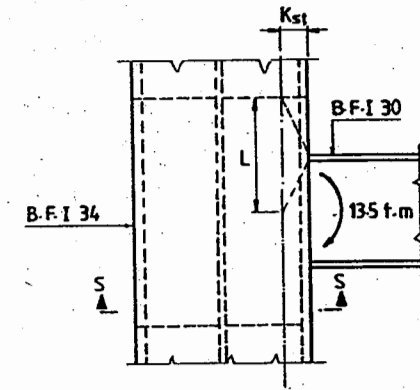
Example (5.3):

Design the welded beam-to-column connection shown in the figure below using a Tee-stub vertical stiffener.

Solution



Tee - Stub Stiffener



Sec. S - S

Replace the moment $M = 13.5 \text{ t.m}$ by two equal and opposite forces:

$$F = T = C = \frac{13.5 \times 100}{(30 - 2)} = 48.2 \text{ tons.}$$

1. Design of the Tee-Stub Stiffener: (Refer to Section 5.2.4)

1.1. Dimensioning:

$$b_{st} = 34 - 2 \times 2.2 = 29.6 \text{ cms.}$$



$$d_{st} = \frac{1}{2} (30 - 1.3) = 14.35 \text{ cms.} ; t_{wst} \geq t_{w.b} \geq 1.2 \text{ cm}$$

$$h_{st} \geq 30 + 5 K_{st} = 30 + 5 (2 \times 2.8) = 58 \text{ cms}$$

1.2. Bending Tee-Stub Flange:

$$t_{st} \geq 0.4 \sqrt{0.75 b_b \cdot t_b} \geq 0.4 \sqrt{0.75 \times 30 \times 2} \geq 2.68 \text{ cms.}$$

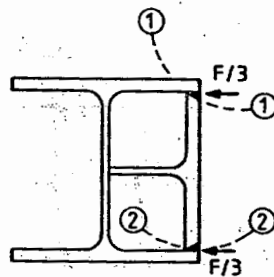
Choose a Tee-Stub stiffener whose flange is 29.6 x 2.8 and web 14.3 x 1.5 (use a part of B.F.I. No. = 45)

1.3. Crippling of Tee-Stub Web:

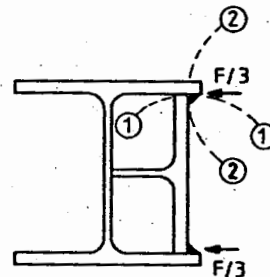
$$\text{Area of applied force} = 0.75 \times 30 \times 2 = 45 \text{ cm}^2.$$

$$L = 2 + 2 \times 2.5 \times 2 \times 2.8 = 30 \text{ cms.}$$

$$\text{Resisting area} = 1.5 \times 30 = 45 \text{ cm}^2 \text{ o.k}$$



Butt weld



Fillet weld

2. Design of Welds: (Refer to Section 5.2.2):

$$\text{Effective length of welds} = 2 + 5 \times 2.8 = 16 \text{ cms.}$$

2.1. Welds Attaching Tee-Stub Flange to Column Flange:

Butt Welds "Good Weld"

Section 1-1:

$$\text{Size of weld} = \text{flange thickness} = 2.8 \text{ cms.}$$



$$f_t = q = \frac{(48.2/3) \cdot \cos 45}{(2.8 \times 16)/\cos 45} = 0.18 \text{ t/cm}^2.$$

$$f_{equi} = \sqrt{0.18^2 + 3(0.18)^2} = 0.35 \text{ t/cm}^2 < 1.1 \times 1.4$$

Section 2-2:

$$q = \frac{48.2/3}{2.8 \times 16} = 0.35 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2.$$

Fillet Welds

$$q_{1-1} = f_{2-2} = \frac{48.2/3}{2.2 \times 16} = 0.456 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ safe}$$

$s_{\max}$ weld length

2.2. Welds Attaching Tee-Stub Web to Column web (Section 5.2.3):

$$M = 0.75 \times 13.5 = 10.125 \text{ t.m.}$$

$$L = t_b + 2 \times 2.5 k_{st} = 30 \text{ cms.}$$

Plane 1-1:

$$f_t = \frac{M}{Z}$$

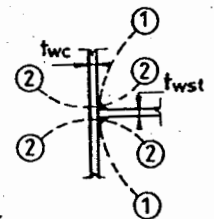
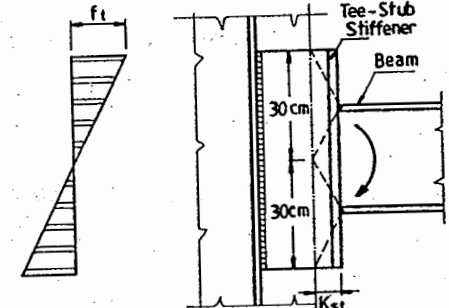
$$Z = \frac{2 \times 60^3 \times 1.3}{12 \times (60/2)} = 1560 \text{ cm}^3$$

$$f_t = \frac{1012.5}{1560} = 0.64 \text{ t/cm}^2$$

$$< 0.72 \text{ t/cm}^2 \text{ o.k}$$

Plane 2-2:

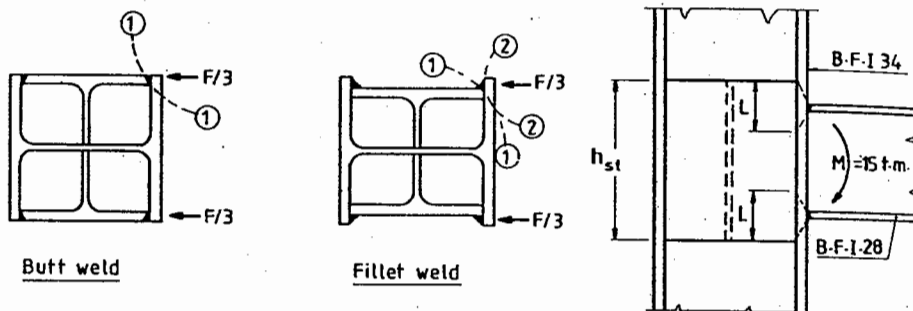
$$q_2 = 0.64 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ o.k}$$



Example 5.4:

Design the welded beam-to-column connection shown in the figure below. Use a vertical Tee-stub stiffener.

Solution

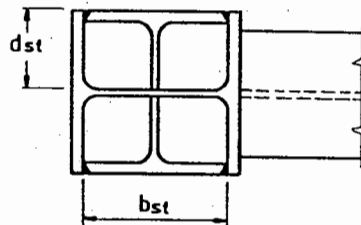


1. Dimensions of the Tee-Stub:

$$b_{st} = 34 - 2 \times 2.2 = 29.6 \text{ cms.}$$

$$d_{st} = \frac{1}{2} (30 - 1.3) = 14.35 \text{ cms.}$$

$$h_{st} = 28 + 2 \times 2.5 \times t_{fc} = 28 + 5 \times 2.2 = 39.0 \text{ cms.}$$



Take the Tee-stub as a part of a B.F.I. No. = 30

$$\text{Replace the moment by two forces: } F = \frac{M}{d} = \frac{1500}{26} = 57.7 \text{ tons}$$

2. Welds Attaching Tee-stub Flange to Column Flanges:

Butt Welds « Good Welds »

$$\text{Size of butt welds} = t_{st} = 2.0 \text{ cms.}$$

$$L = \text{Length of weld} = t_b + 5 t_{fc} = 2 + 5 \times 2.2 = 13 \text{ cms.}$$

Section 1-1:

$$f_t = q = \frac{(57.7/3) \cos 45}{(2 \times 13)/\cos 45} = 0.37 \text{ t/cm}^2$$

$$f_{equi} = \sqrt{0.37^2 + 3 \times (0.37)^2} = 0.546 \text{ t/cm}^2 < 1.4 \times 1.1$$

Fillet Welds

$$q_1 = f_1 = \frac{(57.7/3)}{2 \times 13} = 0.74 \text{ t/cm}^2 > 0.72$$

s_{max}

∴ unsafe

In order to use safe fillet welds the Tee-stub must be replaced by a portion of a B.F.I N = 32.

3. Crippling of Column Web:

$$\text{Area of applied force} = \frac{3}{4} \times 2 \times 28 = 42 \text{ cm}^2.$$

$$\text{Resisting area} = 1.3 (2.0 + 5 \times 2 \times 2.2) = 31.2 \text{ cm}^2 \text{ Unsafe}$$

Either we omit this type of configuration or we apply an allowable stress approach as follows:

$$\text{Applied force} = \frac{3 \times 15 \times 100}{4 \times (28 - 2)} = 43.26 \text{ tons.}$$

$$\text{Resisting force} = 1.8 \times 1.3 \times (2.0 + 5 \times 2 \times 2.2)$$

$$F_{crp} \quad t_{wc} \quad t_b \quad t_{fc}$$

$$= 56.16 \text{ tons} > 43.26 \text{ safe.}$$

4. Bending of Column Flange:

$$t_{fc} \geq 0.4 \sqrt{\frac{3}{4} \times 28 \times 2} \geq 2.59 > 2.3 \text{ unsafe.}$$



This approach is too conservative, if we apply an allowable stress design approach we get:

Applied force = 43.26 tons.

$$\text{Resisting force} = 0.8 (7 \times 1.4 \times \overbrace{2.2}^{\substack{\checkmark \\ F_c}} + 0.15 \times 28 \times 2 \times \overbrace{1.4}^{\substack{\checkmark \\ t_{fc}}})$$

$$= 50.85 \text{ tons} \rightarrow \text{43.26 tons (o.k.)}$$

N.B. The Tee-stub web is attached to the column web using fillet welds of minimum size $s = 0.4$ cms.

5. Shear Resistance of Column Web:

Applied force = $0.75 \times 57.7 = 43.275$ tons.

$$\begin{aligned} \text{Resistance} &= 1.3 \times 34 \times 0.84 \\ &= 37.128 \text{ t} \quad \text{unsafe.} \end{aligned}$$

$$\begin{aligned} \text{Remaining force} &= 43.275 - 37.128 \\ &= 6.14 \text{ tons} \end{aligned}$$

Use a cover plate of thickness t :

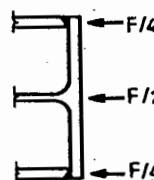
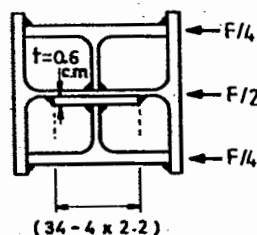
$$6.14 = (34 - 4 \times 2.2) \times t \times 0.84$$

get $t = 0.3$ cm ; take $t = 0.6$ cm.

To be noticed that this approach is somewhat conservative, a realistic analysis must consider that the applied force is $F/2$ instead of $\frac{3}{4} F$.

i.e. Applied force = $0.5 \times 57.7 = 28.85$ tons

< 34.72 t o.k safe



Example 5.5:

Design the connection of Example 5.4 using vertical plate stiffeners.

Solution

1. Dimensions of the Plate Stiffener and the Corresponding Welds:

$$b_{st} = 34 - 2 \times 2.2 = 29.6 \text{ cms.}$$

$$\begin{aligned} h_{st} &= 28 + 5 t_{fc} = 28 + 5 \times \\ &2.2 = 39 \text{ cms.} \end{aligned}$$

Vertical plate stiffener

$$\text{Take } t_{st} = t_{fc} = 2.2 \text{ cms}$$

$$I_x = 2.2 \left(\frac{39^3}{12} - \frac{13^3}{12} \right) = 10472.3 \text{ cm}^4$$

$$f_b = \frac{15 \times 100 \times (39/2)}{3 \times 10472.3} = 0.926 \text{ t/cm}^2$$

O.K.

Butt Welds «Good Weld»

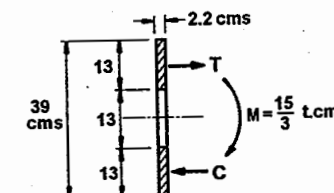
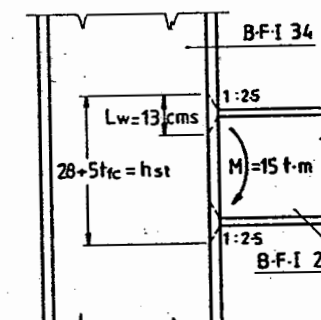
$$\text{Size of butt welds} = t_{st} ;$$

$$\begin{aligned} L_w &= t_b + 5 t_{fc} = 2 + 5 \times 2.2 \\ &= 13 \text{ cm.} \end{aligned}$$

Section 1-1: (Butt welds) :-

$$f_t = q = \frac{(57.7/3) \times \cos 45}{(t_{st} \times 13) / \cos 45} = \frac{0.74}{t_{st}}$$

$$f_{equi} = \sqrt{\left(\frac{0.74}{t_{st}} \right)^2 + 3 \left(\frac{0.74}{t_{st}} \right)^2} \leq 1.1 \times 1.4 \text{ t/cm}^2$$



Put: $f_{\text{equi}} = 1.1 \times 1.4$;

$t_{st} = 1.0$ cms. take 2.2 cms

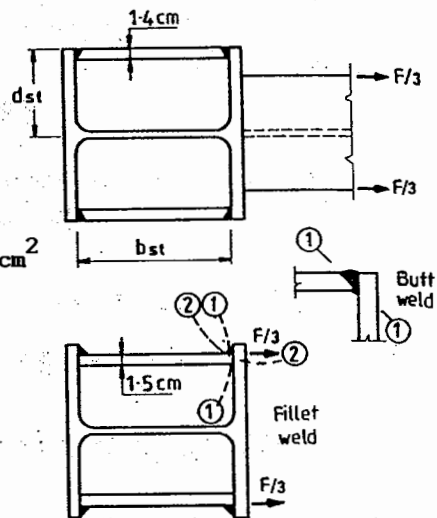
Fillet Welds

$$f_{1-1\perp} = q_{2-2} = \frac{57.7/3}{s \times 13} \leq 0.72 \text{ t/cm}^2$$

Put: $f_{\perp} = q_{\perp} = 0.72 \text{ t/cm}^2$.

Get: $s = 2.05 \text{ cms.}$;

Take $s = 2.2$ cms.



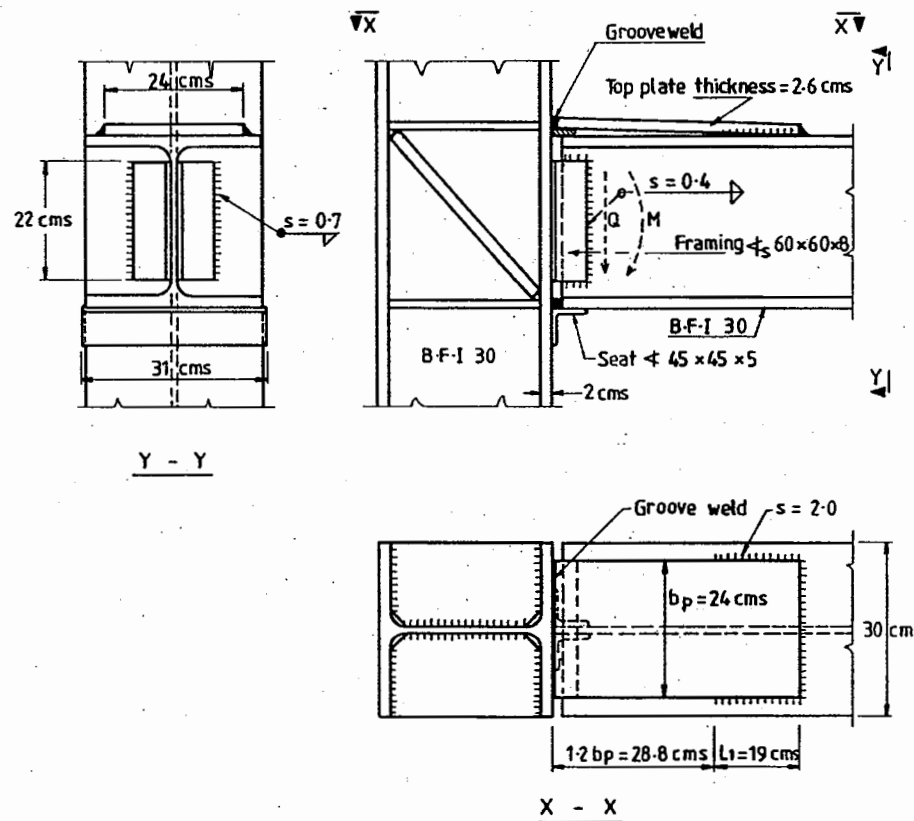
2. Crippling of Column Web, Bending of Column Flange, Shear
Resistance of Column Web:

Exactly as Example 5.4.

Example (5.6):

Design the moment connection shown in the Fig. below. The connection corresponds to configuration "A" of Section 5.4.1.
 $M = 24.0 \text{ t.m.}$, $R_b = 10.0 \text{ t.}$

Solution



1. Design of the Top Plate:

$$\text{Applied force} = \frac{24 \times 100}{(30 - 2)} = 85.71 \text{ tons.}$$



Choose width of top plate $b_p = 24.0$ cms.

Hence, thickness of top plate " t_p " is:

$$t_p = \frac{85.71}{24 \times 1.4} = 2.55 \text{ cms, take } t_p = 2.6 \text{ cms.}$$

- Use fillet weld lines to attach the top plate to the beam flange. The size of fillet welds is chosen equal to the beam flange $s = 2.0$ cms.

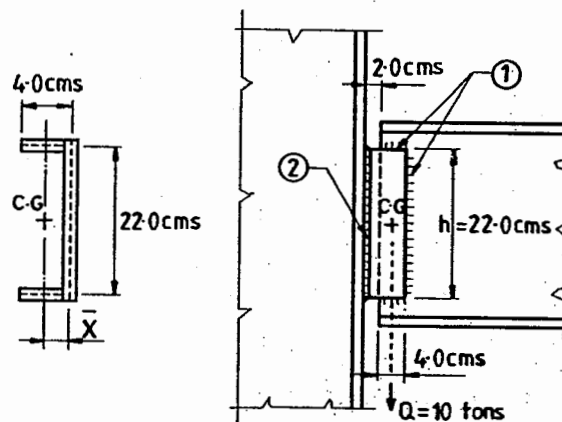
Applying in the shear stress equation we get " L_1 ":

$$q = 0.72 = \frac{85.71}{2(24 + 2L_1)}$$

$$L_1 = 17.76 \text{ cms, take } L_1 = 18.0 \text{ cms.}$$

2. Design of the Framing Angles:

Choose 2 $\times$ 60 $\times$ 60 $\times$ 8 with a height " h " = 30 - 4 $\times$ 2 = 22 cms.



The Size of Group of Welds "1" is:

$$s_1 = \frac{10}{0.72 \times 2(22 + 2 \times 4)} = 0.227 \text{ cms (take } s_1 = 4 \text{ mms.)}$$

$$\bar{X} = \frac{4^2}{22 + 2 \times 4} = 0.53 \text{ cms (Refer to Table 3.6)}$$

The induced torsional moment $M_t = 10 \times 0.53 = 5.3$ t.cm is of minor effect and hence the induced shear stresses can be neglected. Also the face of weld joining the welds to the framing angles are safe if the other face of weld is safe.

The Size of Group of Welds "2" is:

$$R_b = 10 \text{ tons. ; } M = 10(6 - 0.53) = 54.7 \text{ t.cms}$$

Plane (1-1):

$$q_{//} = \frac{10}{2 \times 22 \times s_2} = \frac{1}{4.4 s_2}$$

$$f_{\perp} = \frac{10 \times (6 - 0.53) \times 6}{2 \times s_2 \times 22^2} = \frac{1}{2.95 s_2}$$

$$f_{eq} = \sqrt{f_{\perp}^2 + 3 q_{//}^2} < 0.79 \text{ t/cm}^2.$$

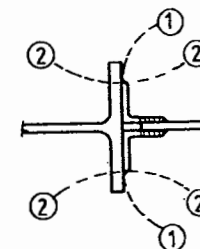
$$\text{Get } s_2 = 0.65 \text{ cms. , choose } s_2 = 7 \text{ mms.}$$

Plane (2-2):

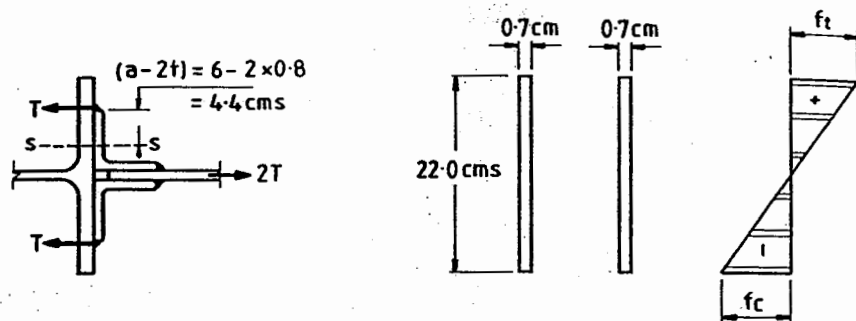
$$q_{//} = \frac{1}{4.4 s_2} ; q_{\perp} = \frac{1}{2.95 s_2}$$

$$q_{total} = 0.72 = \sqrt{q_{\perp}^2 + q_{//}^2}$$

$$\text{i.e. } s_2 = 0.55 \text{ cms. , hence the governing size is 7 mms.}$$



Angles:



$$f_t = f_c = \frac{1}{2.95 \times s_2} = \frac{1}{2.95 \times 0.7} = 0.484 \text{ t/cm}^2$$

$$T = 0.484 \times 0.7 \times 1 \text{ (one centimeter height)} = 0.339 \text{ tons.}$$

$$M_{ss} = 0.339 (6 - 2 \times 0.8) = 1.49 \text{ t.cm.}$$

If plastification is allowed the thickness of the framing angles is:

$$t = \sqrt{\frac{1.49 \times 6}{2.4 \times 1}} = 1.93 \text{ cms.} > 0.8 \text{ cms}$$

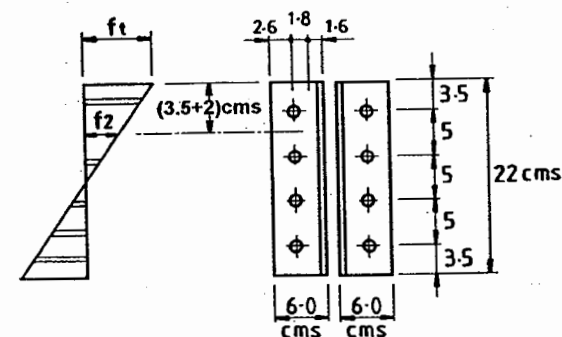
one centimeter height

This value is high and conservative.

If high strength bolts 10.9 are utilized to attach the framing angles with the column flange, the angle thickness is computed as follows:

Use 4 M₁₆ (10.9)

$$f_t = \frac{10 \times (6 - 0.53) \times 6}{2 \times 6 \times 22^2} = 0.0565 \text{ t/cm}^2$$



$$f_2 = \frac{0.0565 \times (11 - 3.5 - 2.5)}{11} = 0.0256 \text{ t/cm}^2$$

$$T = \left(\frac{0.0565 + 0.0256}{2} \right) \times 6 \times (3.5 + 2.5) = 1.4 \text{ tons.}$$

$$M = 1.4 \times (a - t)/2 = 1.4 \times (6 - 0.8)/2 = 3.64 \text{ t.cm}$$

$$t = \sqrt{\frac{3.64 \times 6}{2.4 \times 6}} = 1.23 \text{ cms. (yield is allowed)}$$

Hence use 2 < s 120 x 120 x 14

Bolts:

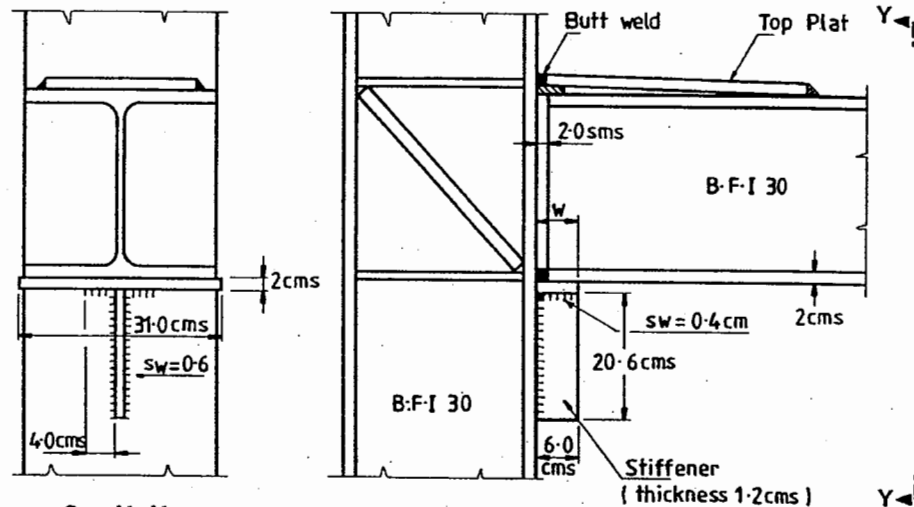
$$Q_b = \frac{10}{8} = 1.25 \text{ tons} < 3.16 \text{ o.k.}$$

$$T_{ext} = 1.4 \text{ tons} < 0.8 \times 9.89 \text{ o.k.}$$

Example (5.7):

Solve example 5.6 using configuration "B" of Section 5.4.1.

Refer also to Section 4.2.7.2 for the design of the bracket. $M = 24.0 \text{ t.m}$, $R_b = 10 \text{ tons}$.

SolutionSec. Y-Y1. Top Plate:

The design is exactly as Example 5.6.

2. Crippling of Column Web, Bending of Column Flange, Stability of Corner Web:

As Examples 5.1 and 5.2.

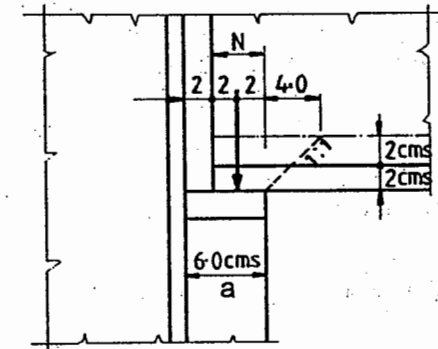
3. Design of the Bracket:(a) The Bearing Length "N":

$$\frac{R_b}{(N + K) t_{w.b}} = 0.75 F_y ; R_b = 10 \text{ tons} , t_{w.b} = 1.2 \text{ cms} ,$$

$$K = 2 \times 2 = 4.0 \text{ cms.}$$

$$\text{Hence get } N = 0.63 \text{ cms} , N_{\min} = K = 4.0 \text{ cms.}$$

$$\text{The width } a = N + C = 4.0 + 2.0 = 6.0 \text{ cms.}$$

(b) Stiffener Thickness:

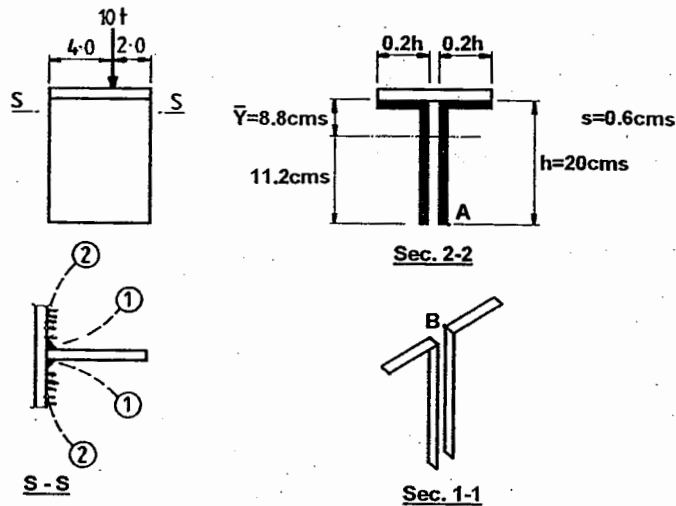
The stiffener thickness t_{st} must satisfy the following conditions:

$$- t_{st} \geq t_{w.b} \geq 1.2 \text{ cms.}$$

- To avoid the local buckling.

$$a / t_{st} \leq 16$$

$$\text{Hence } t_{st} \geq 6 / 16 \geq 0.375 \text{ cms.}$$



- The shear stress in the vertical stiffener = shear stress in the weld

$$\text{i.e. } 0.35 \times 2.4 \times t_{st} \times h = 2 \times 0.72 \times s \times h$$

$$t_{st} \geq 1.71 \text{ s}$$

If "s" is taken equal to 0.6 cms get $t_{st} \geq 1.0 \text{ cms}$.

- Bending stress of stiffeners must be safe.

$$R_b = 10 \text{ tons.}, \quad M = 10 (3 - 2) = 10 \text{ t.cm.}$$

$$f_c = \frac{10}{6 t_{st}} + \frac{10 \times 6}{t_{st} \times 6^2}$$

$$\text{Put } f_c = 1.4 \text{ t/cm}^2, \quad \text{get } t_{st} = 2.38$$

Hence choose $t_{st} = 2.4 \text{ cms}$.

(c) Welds Connecting Stiffeners to Column Flange:

Assume $b = 0.2 h$; $h = 20 \text{ cms}$; $s = 0.6 \text{ cms}$.

Hence compute the following properties of weld lines:

$$\text{Area} = 28.8 \text{ cm}^2$$

$$\bar{y} = 8.88 \text{ cms} ; \quad I_{xx} = 1224.36 \text{ cm}^4$$

Applying in the stress equation:

$$M = 10 \times 4 = 40 \text{ t.cm.}$$

Plane (2-2):

$$q_{A//} = \frac{10}{28.8} = 0.347 \text{ t/cm}^2$$

$$f_{A\perp} = \frac{40 \times 11.2}{1224.36} = 0.366 \text{ t/cm}^2$$

$$f_e = \sqrt{(0.366)^2 + 3(0.347)^2} = 0.703 \text{ t/cm}^2 < 0.72 \times 1.1 \text{ t/cm}^2$$

O.K.

Plane (1-1):

$Q = 10 \text{ tons}$ will be resisted by shear on the vertical lines and by normal stresses on the two horizontal lines.

$$f_{B\perp} = 0.347 \text{ t/cm}^2 ; \quad Q = 10 \text{ tons}$$

Assume $M_t = 40 \text{ t.cm}$ to be resisted by the vertical lines :

$$q_{B\perp}^{(Mt)} = \frac{40 \times 20}{2 \times 0.6 \times 20^3} = 0.083 \text{ t/cm}^2 \quad (\text{Neglected})$$

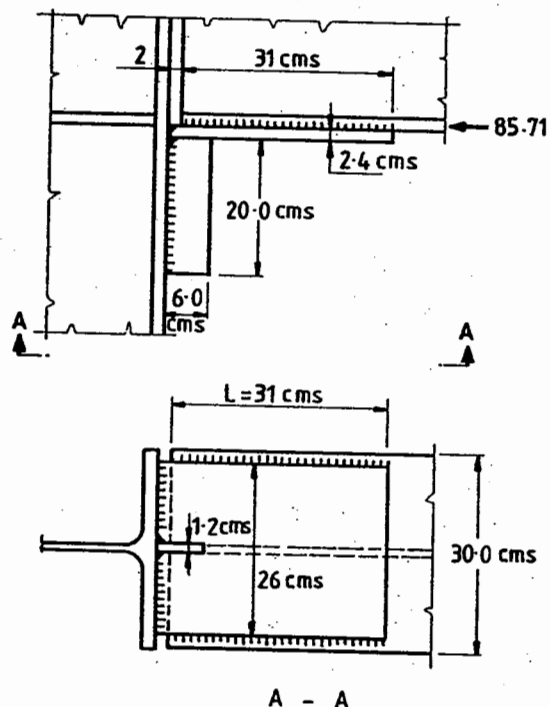
No need to check this plane 1-1

Example (5.8):

Solve Example 5.6 using configuration "C" of Section 5.4.1.

Solution

All steel structural components of the configuration "c" are the same as Example 5.6 except for the lower bracket where its top plate is to be designed as follows:

Length of Bracket Top Plate for the Lower Bracket :-

Transmitted force to top plate of bracket = 85.71 tons.

Take $s = 2.0$ cms.



Hence: $L = \frac{85.71}{0.72 \times 2 \times 2} = 29.76$ cms. Take $L = 30$ cms.

The total length of this top plate = $30 + 2 = 32$ cms.

Width and Thickness of Top Plate:

Take width of top plate of bracket = $30 - 2 \times 2 = 26$ cms.

Required thickness $t = \frac{85.71}{26 \times 1.4} = 2.35$ cms.

Take $t = 2.4$ cms.

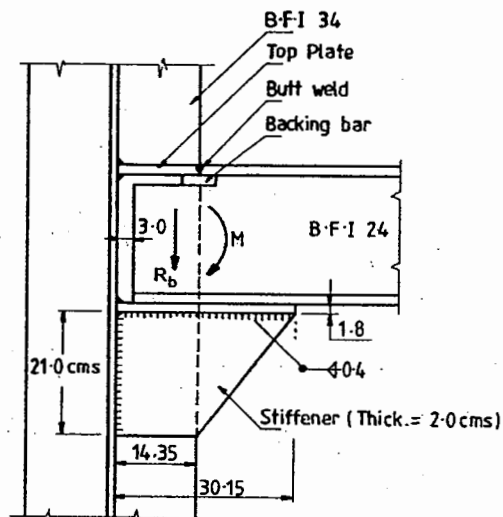
The design of the vertical stiffener and the fillet weld lines connecting this stiffener to the column flange are exactly as Example 5.7.

Example (5.9):

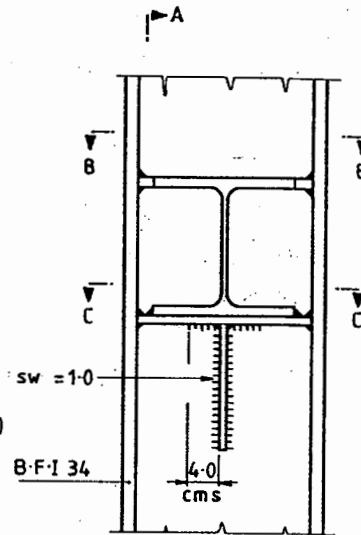
Design the connection shown in the Fig. below. Refer to Section 5.4.2 "Configuration "D".

$$M = 13.5 \text{ t.m.}, \quad R_b = 6.0 \text{ tons.}$$

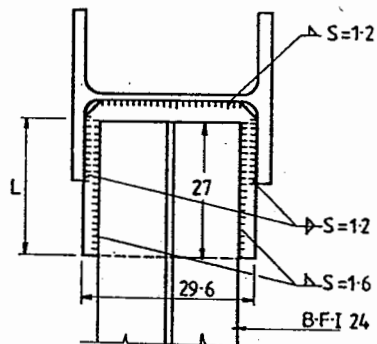
Solution



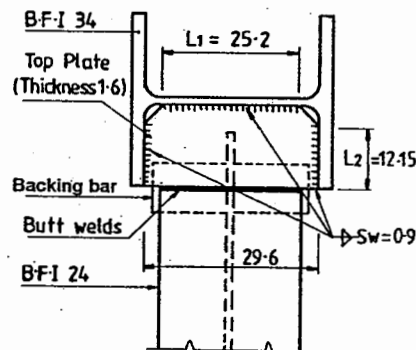
A - A



B - B



C - C



B - B

(A) Tension Side

1. Top Plate:

$$\text{Tension force} = \frac{1350}{(24 - 1.8)} = 60.81 \text{ tons.}$$

$$\text{Maximum width} = 34 - 2 \times 2.2 = 29.6 \text{ cms.}$$

$$\text{Thickness} = \frac{60.81}{29.6 \times 1.4} = 1.47 \text{ cms.}$$

Take 1.8 cms as beam.

2. Fillet Welds Attaching Top plate to Column Section:

From Section B-B:

$$L_1 = 34 - 4 \times 2.2 = 25.2 \text{ cms.}$$

$$L_2 = \frac{30 - 1.3 - 2 \times 2.2}{2} = 12.15 \text{ cms.}$$

$$s = \frac{60.81}{2(25.2 + 2 \times 12.15) \times 0.72} = 0.85 \text{ cms. Take 9 mms.}$$

3. Butt Welds Attaching Top Plate to Beam Flange:

Use butt welds to connect the top plate to the beam flange. No checks are required for excellent weld. Use 2 backing bars as shown by dotted lines in Section B-B.

(B) Compression Side

1. Fillet Welds Attaching the Beam Bottom Flange to the Bracket

Top Plate: Section (C-C)

Take $s = 1.6 \text{ cms.}$

$$L = \frac{60.81}{2 \times 1.6 \times 0.72} = 26.39 \text{ cms. Take 27.0 cms.}$$

**2. Dimensions of the Bracket Top Plate:**

Total length = 27.0 + 3.0 clearance = 30.0 cms.

Width = 34 - 2 x 2.2 = 29.6 cms.

Thickness = 1.8 cms as beam flange.

3. Fillet Welds Attaching Top Plate of Bracket to Column

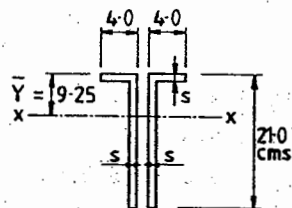
Section:

lower face

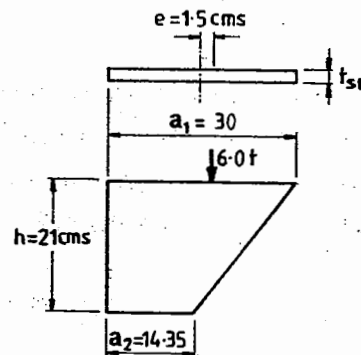
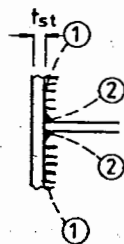
upper face

$$\begin{aligned} \text{Total length of fillet weld} &= (2 L_2) + (L_1 + 2 L_2) \\ &= 25.2 + 4 \times 12.15 = 73.8 \text{ cms.} \end{aligned}$$

$$s = \frac{60.81}{0.72 \times 73.8} = 1.14 \text{ cms. Take } s = 1.2 \text{ cms.}$$

4. Bracket Dimensions:

Sec. 1-1



Sec. 2-2



$$a_1 = 27.0 + 3.0 = 30.0 \text{ cms.}$$

$$a_2 = \frac{30 - 1.3}{2} = 14.35 \text{ cms.}$$

$$e = \frac{30.0 - 27.0}{2} = 1.5 \text{ cms.}$$

s to be taken 1.0 cms (s = 10 mms).

$$\text{The bearing length: } \frac{R_b}{(N + K) t_{w.b}} = 0.75 f_y$$

$$R_b = 6.0 \text{ tons}, K = 2 \times 1.8 = 3.6 \text{ cms}, t_{w.b} = 1.1 \text{ cms.}$$

$$N = -ve \text{ hence } N_{\min} = K = 3.6 \text{ cms.} < 27.15 \text{ cms. o.k.}$$

- Stiffener thickness t_{st} :

$$t_{st} \geq t_{w.b} \geq 1.1 \text{ cms.}$$

$$t_{st} \geq 30.15 / 16 \geq 1.88 \text{ cms.}$$

$$t_{st} \geq 1.77 s \geq 1.77 \text{ cms (where } s = 1.0 \text{ cms).}$$

Get t_{st} from the normal stress equation

$$f_b = \frac{N}{30.15 t_{st}} + \frac{6 M}{t_{st} \times 30.15^2}$$

$$\text{Where: } N = 6.0 \text{ tons}, M = 6 \times 1.5 = 9 \text{ t.cms},$$

$$\text{Put } f_b = 1.4 \text{ t/cm}^2$$

$$\text{Hence } t_{st} = 0.2 \text{ cms.}$$

$$\text{Choose } t = 2.0 \text{ cms.}$$

- Welds connecting stiffeners to column web:

$$R_b = 6.0 \text{ tons}, M = 6 \left(\frac{27.0}{2} + 3 \right) = 99.0 \text{ t.cms.}$$



$$s = 1.0 \text{ cms.}$$

Properties of Fillet Welds:

$$\bar{y} = 9.25 \text{ cms.}$$

$$\text{Area} = 48 \text{ cm}^2.$$

$$I_{xx} = 2068.33 \text{ cm}^4 ; Z_x = 176.03 \text{ cm}^3.$$

Plane (1-1):

$$q_{//} = \frac{6}{48} = 0.125 \text{ t/cm}^2, f_{\perp} = \frac{99.0}{176.03} = 0.565 \text{ t/cm}^2$$

$$f_{\text{equ}} = \sqrt{0.565^2 + 3 \times 0.125^2} = 0.605 \text{ t/cm}^2 < 0.79 \text{ (o.k.)}$$

Plane (2-2):

No need to check this section (2-2) it will be safe as mentioned in the previous examples.



Example (5.10):

Design the connection of Example 5.9 using Configuration "E" of Section 5.4.2.

$$M = 13.5 \text{ t.m.}$$

$$R_b = 6.0 \text{ tons.}$$

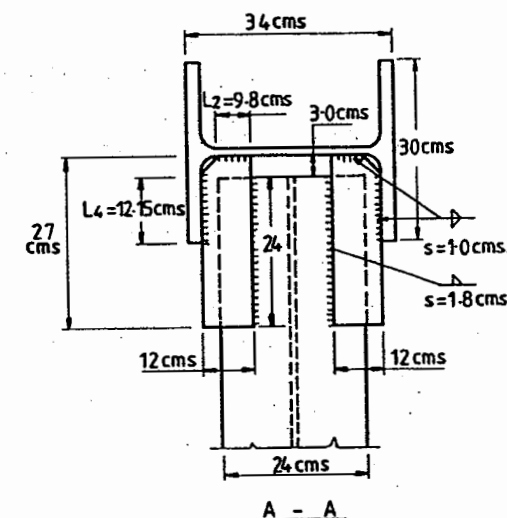
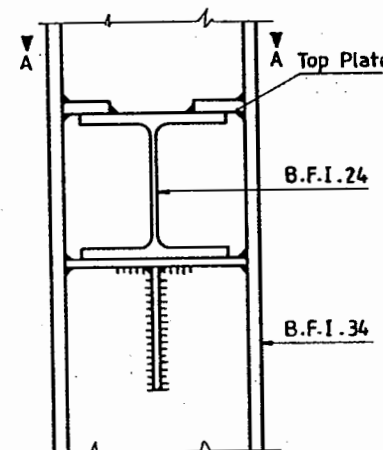
Solution

1. Design of the Top Plates:

$$\text{Tension force} = 60.81 \text{ tons.}$$

Take thickness of top plate = thickness of beam flange
= 1.8 cms.

$$\text{Width} = \frac{60.81}{2 \times 1.4 \times 1.8} = 12.06 \text{ cms. take } 12.0 \text{ cms}$$



(a) Tension Side2. Fillet Welds Attaching Top Plates to Beam Tension Flange:

Take $s = 1.8$ cms.

Length of welds = $\frac{60.81}{2 \times 0.72 \times 1.8} = 23.46$ cms. Take 24.0 cms.

Total length = 24.0 + 3 clearance = 27.0 cms.

3. Fillet Welds Attaching Top Plates to Column Section:

Total length of welds = 4 (12.15 + 9.8) = 87.8 cms.

Where: $L_4 = \left[\left(\frac{30 - 1.3}{2} \right) - 2.2 \right] = 12.15$ cms.

$L_2 = 12 - 2.2 = 9.8$ cms.

$s = \frac{60.81}{87.8 \times 0.72} = 0.96$ cms. Take $s = 10$ mms.

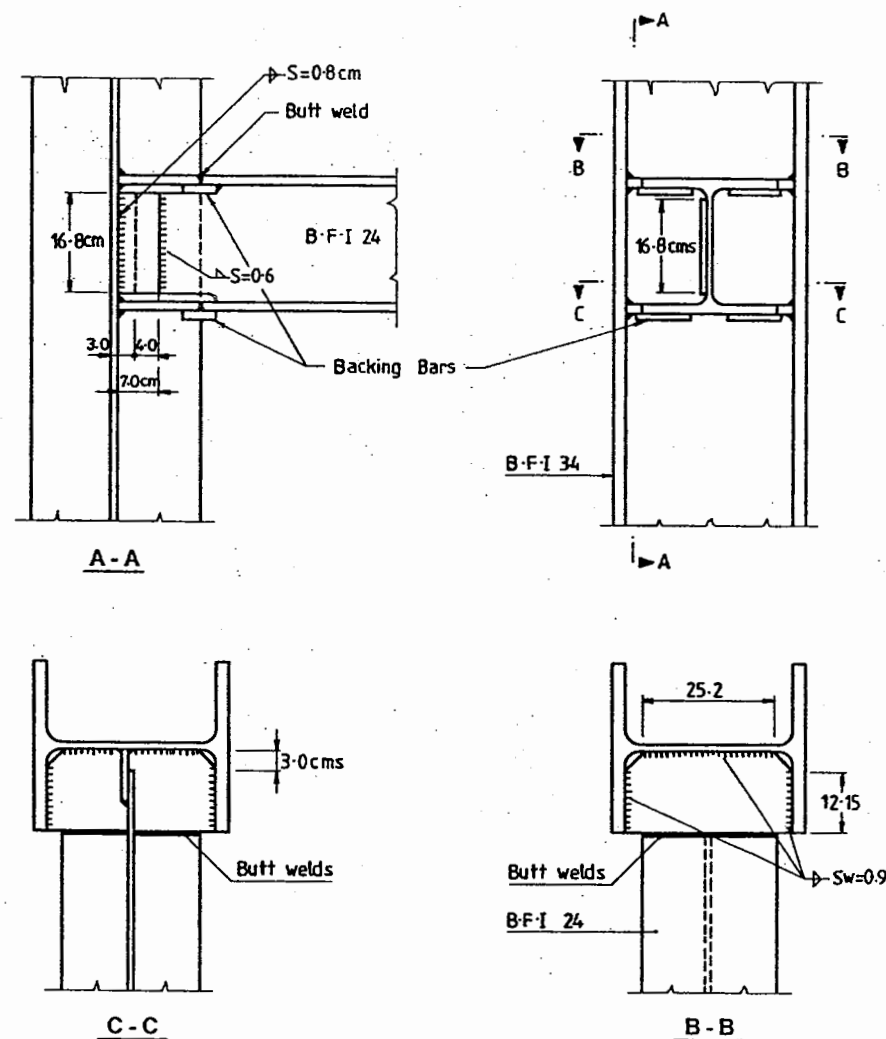
(b) Compression Side

Exactly as Example 5.9.

Example (5.11):

Redesign the connection of Example 5.10 using configuration "F" of Section 5.4.2.

Refer also to Section 4.2.3. $M = 13.5$ t.m , $R_b = 6.0$ tons.

Solution



1. Tension and Compression Sides are to be designed exactly as the tension side of Example 5.9. (B-B Section).

2. Design of the Cleat Plate:

(a) Fillet Welds Attaching Cleat Plate to the Beam Web:

Maximum height of fillet weld lines = $24 - 4 \times 1.8 = 16.8$ cms.

$$s = \frac{6.0}{16.8 \times 0.72} = 0.50 \text{ cms} \quad \text{Take } s = 6 \text{ mms.}$$

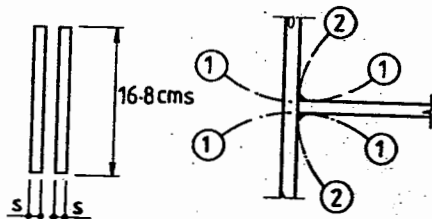
(b) Fillet Welds Attaching Cleat Plate to Column Web:

$$R_b = 6.0 \text{ tons}, \quad M = 6 \times (3 + 4) = 42 \text{ t.cms.}$$

$$q_{//} = \frac{6}{2 \times 16.8} = \frac{1}{5.6} \text{ t/cm}^2$$

$$Z_x = \frac{2 s h^2}{6} = 94.08 s \text{ cm}^3 \quad (h = 16.8 \text{ cms})$$

$$f_{\perp} = \frac{42}{94.08 s} = \frac{1}{2.24 s} \text{ t/cm}^2$$



Plane (1-1):

$$q_{//} = \frac{1}{5.6 s}, \quad q_{\perp} = \frac{1}{2.24 s}$$

$$q_t = 0.72 = \frac{1}{s} \sqrt{\left(\frac{1}{2.24}\right)^2 + \left(\frac{1}{5.6}\right)^2} \quad \text{get } s = 0.66 \text{ cms.}$$



Plane (2-2):

$$q_{//} = \frac{1}{5.6 s}, \quad f_{\perp} = \frac{1}{2.24 s}$$

$$f_{\text{equ}} = 0.79 = \frac{1}{s} \sqrt{\frac{3}{(5.6)^2} + \frac{1}{(2.24)^2}} \quad \text{get } s = 0.68 \text{ cms.}$$

Hence take $s = 8 \text{ mms.}$

(c) Thickness of Cleat Plate:

height = 16.8 cms

width = 7.0 cms

take thickness = 0.8 cms.

$$q_{\text{av}} = \frac{3 \times 6}{2 \times 16.8 \times 0.8} = 0.67 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2 \text{ o.k.}$$

$$f_t = \frac{42 \times 6}{0.8 \times 16.8^2} = 1.12 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ o.k.}$$

**Example (5.12):**

Design the bolted seat angles connection shown in the Fig. below. Where $M = 3.0 \text{ t.m}$ and $R_b = 5.0 \text{ tons}$.

Solution

(1) Assume that the moment is replaced by two equal forces of opposite sense:

$$F = T = C = \frac{M}{h} = \frac{3 \times 100}{(22-1.6)}$$

$$= 14.70 \text{ tons}$$

Group of Bolts (1):

These bolts are subjected to shear.

$$Q_b = \frac{14.7}{8} = 1.84 \text{ tons}$$

Choose H.S.B (10.9) M 16

Use $80 \times 80 \times 8$

$$P_s = 3.16 \text{ tons}; T_1 = 9.89 \text{ tons}$$

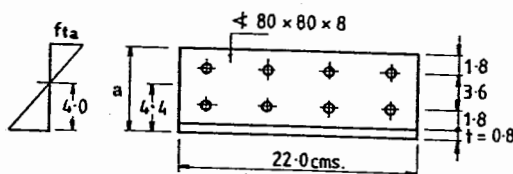
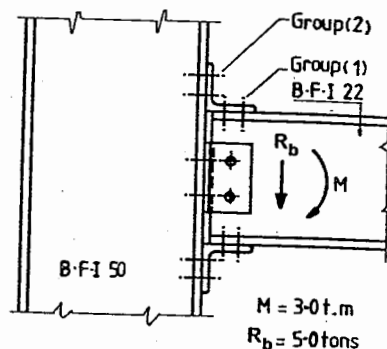
Group of Bolts (2):

$$T = 14.70 \text{ tons.}$$

$$M = 14.70 \times 4.4 = 64.68 \text{ t.cm}$$

$$T_{\text{ext, b}} = \frac{14.70}{8} = 1.84 \text{ tons}$$

bolts



$$I_{xx} = \frac{22 \times (8)^3}{12} = 938.66 \text{ cm}^4$$

$$f_{ta} = \frac{64.68 \times 4}{938.66} = 0.276 \text{ t/cm}^2$$

$$T_{b,M} = \left[\frac{1}{2} \times 0.276 \times 4 \times 22 \right] \times \frac{1}{4} = 3.0 \text{ tons.}$$

bolts

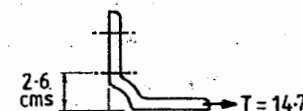
Hence:

$$1.84 + 3.0 = 4.84 \text{ tons} < 0.8 \times 9.89 \text{ (O.K.)}$$

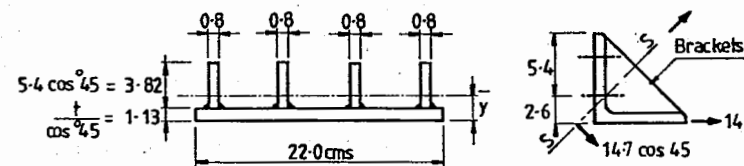
(2) Thickness of Seat Angle:

$$M = 14.70 \times 2.6 = 38.22 \text{ t.cm.}$$

$$t = \frac{6 \times 38.22}{22 \times 1.4} = 2.72 \text{ cms.}$$



Excessive thickness, hence this type of configuration is to be modified as follows:

Add Stiffeners:

$$A = 4 \times 3.82 \times 0.8 + 22 \times 1.13 = 37 \text{ cm}^2$$

$$\bar{y} = \frac{22 \times 1.13 \times 0.565 + 4 \times 3.82 \times 0.8 \times 3.04}{37} \approx 1.38 \text{ cms}$$

$$I_{xx} = 22 \times 1.13 \times \left(1.38 - \frac{1.13}{2} \right)^2 + 4 \times \frac{0.8 \times (3.82)^3}{12} + 4 \times 0.8 \times$$



$$3.82 \times \left(\frac{3.82}{2} + 1.13 - 1.38 \right)^2 = 16.5 + 14.86 + 33.7 = 65 \text{ cm}^4$$

$$f_c = \frac{14.7 \cos 45^\circ}{37} \pm \frac{14.7 \cos 45^\circ (e)}{65} (y_{u,1})$$

$$e = (2.6 / \cos 45^\circ) - 1.13 + 1.38 = 3.9 \text{ cms}$$

$$y_u = 3.82 + 1.13 - 1.38 = 3.57 \text{ cms} ; y_l = 1.38 \text{ cm}$$

$$f_c = 1.96 \text{ t/cm}^2 \text{ Unsafe}$$

Still with the use of stiffeners the configuration is unsafe. Naturally if the dimensions of the seat angle and those of the stiffeners are increased we can reach a safe induced compressive stress i.e. with (200 x 200 x 16).

N.B. We don't recommend to use this type of configuration for a rigid Beam-to-Column connection.

(3) Bolts Connecting Beam Web to Framing Angles:

$$R_b = 5.0 \text{ tons.}$$

$$Q_b = \frac{5}{2} = 2.5 \text{ t} < 2 \times P_s < 2 \times 1.69$$

bolts

Use 2 H.S.B.(10.9) M_{12} to connect the beam web to the framing angles and 4 bolts to connect the other leg of the framing angle to the column flange.

Compute the required framing angles thickness as has been previously outlined in Chapter "4".



Example (5.13):

Design a beam-to-column connection using a top and a bottom plate. The connection is subjected to:

$$M = 20.0 \text{ t.m.} \quad \text{and} \quad R_b = 16.8 \text{ tons.}$$

Use ordinary and high strength bolts to attach the beam flanges to the plates and butt welds to attach the plates to the column flanges.

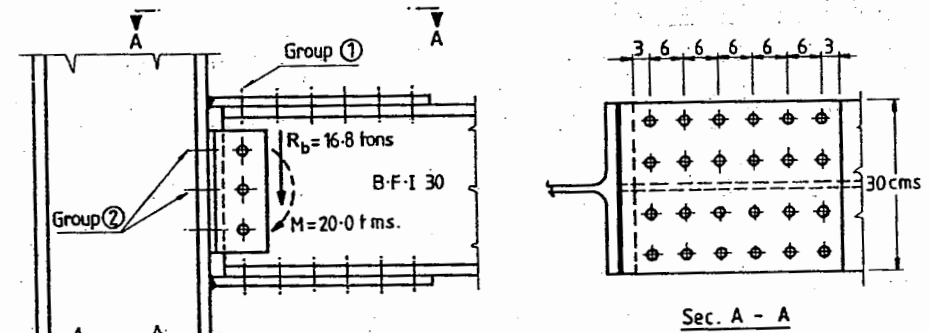
Solution

Ordinary Bolts:

Use ordinary bolts of diameter $d = 20 \text{ mms.}$ (Grade 4.6)

$$R_{ss} = 0.25 \times 4 \times \frac{\pi (2)^2}{4} = 3.14 \text{ tons.}$$

For the computation of the required thickness of the tension top plate the holes area must be deducted.



$$\text{Tension force} = \frac{M}{d} = \frac{20 \times 100}{(30 - 2)} = 71.42 \text{ tons.}$$

$$n_1 = \frac{71.42}{3.14} = 22.7 = 24 \text{ bolts (6 rows - 4 bolts/row)}$$



Assume breadth of plate = width of beam flange = 30.0 cms.

$$\text{Net width of top plate} = [30 - 4 \times (2 + 0.2)] = 21.2 \text{ cms}$$

$$\text{Thickness} = \frac{71.42}{1.4 \times 21.2} = 2.4 \text{ cms.}$$

$$n_2 = \frac{16.8}{3.14} = 5.35 = 6 \text{ bolts (3 bolts at each side).}$$

For the check of the beam tension flange it is sufficient to deduct only the holes exceeding 15% of the gross flange area.

$$\frac{\text{Area of bolts holes}}{\text{Area gross}} = \frac{4 \times 2 \times 2.2}{30 \times 2} = 29.33 \% > 15 \%$$

Hence only 14.33% of the flange tensile capacity is to be deducted, that is to say:

$$\begin{aligned} \text{Net area of beam tension flange} &= (1 - 0.1433) \times 30 \times 2 \\ &= 51.4 \text{ cm}^2. \end{aligned}$$

$$f_t = \frac{71.42}{51.4} = 1.389 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2.$$

High Strength Bolts:

Use high strength bolts (10.9) M_{16} ($P_s = 3.16$ tons).

$$n_1 = \frac{71.42}{3.16} = 22.6 = 24 \text{ bolts (6 rows} \times 4 \text{ bolts).}$$

$$n_2 = \frac{16.8}{3.16} = 5.30 \text{ use 6 bolts (3 bolts at each side).}$$

The high strength bolts of the friction type are pretensioned and hence a clamping force will allow to use the gross area.

$$\text{The required thickness of the top plate} = \frac{71.42}{1.4 \times 30} = 1.7 \text{ cms}$$

Hence use $t = 2.0$ cms as the beam flange.



Example (5.14):

Design the welded corner with bracket connection of the two hinged gable frame shown in the figure below.

Solution

1. Geometrical Configuration:

(a) Use a bracket whose length is equal to one tenth the span i.e. $\frac{30.0}{10} = 3.0$ ms. This length is measured on the center line of the beam from point "O" which corresponds to the intersection point of the beam and the column center lines.

(b) The bracket is to be cut from the beam rolled section.

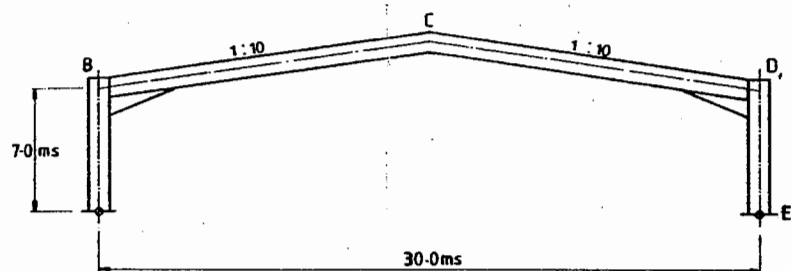
(c) Compute the induced straining actions using a method of structural analysis. The bending moment diagram is illustrated in figure b.

(d) Compute the bending moment at the following sections:

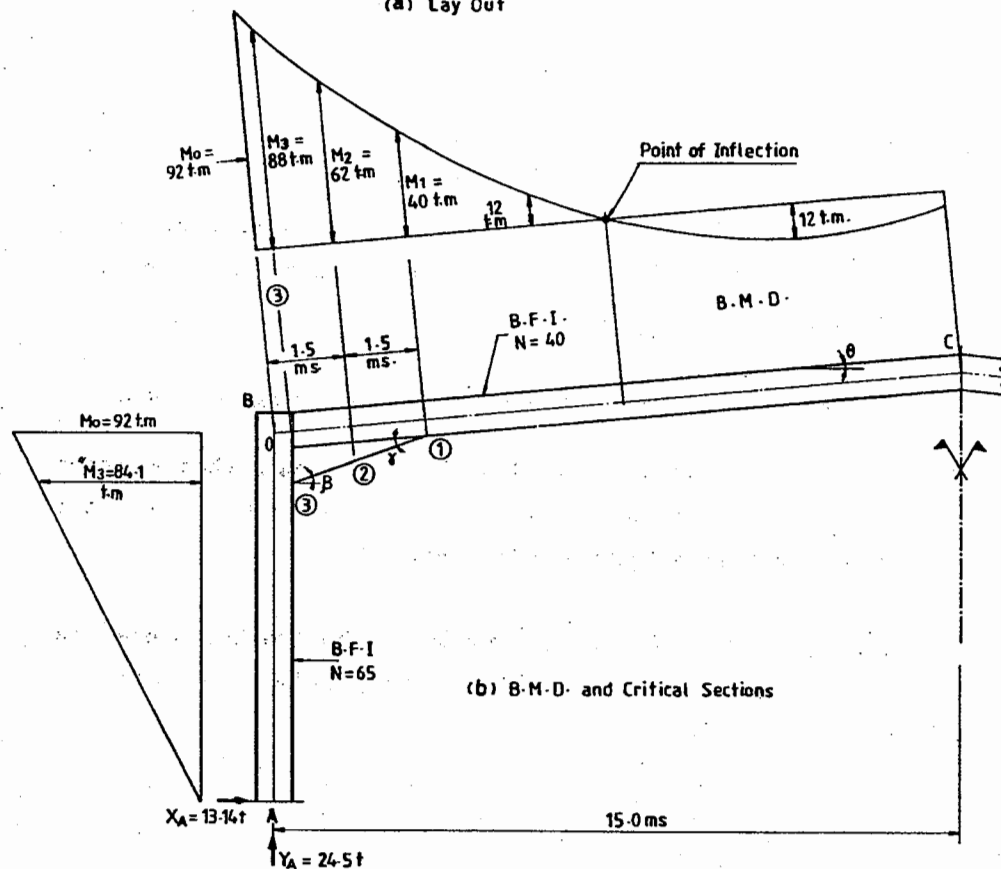
- Section (1-1) just before the abrupt change cross section, the corresponding moment $M_1 = 40.0$ t.m will govern the design of the beam outside the bracket.

- Section (2-2) at mid-length of the bracket where the induced stresses due to $M_2 = 62$ t.m are to be less than the allowable stresses.

- Section (3' - 3') belonging to the beam, the corresponding moment $M_3 = 88.0$ t.m will induce stresses along the vertical section at the beam and the bracket connection to the column flange.



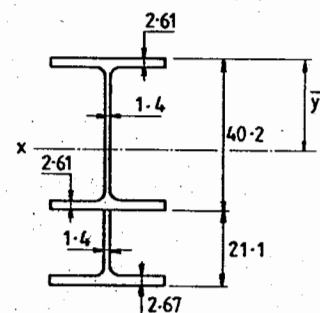
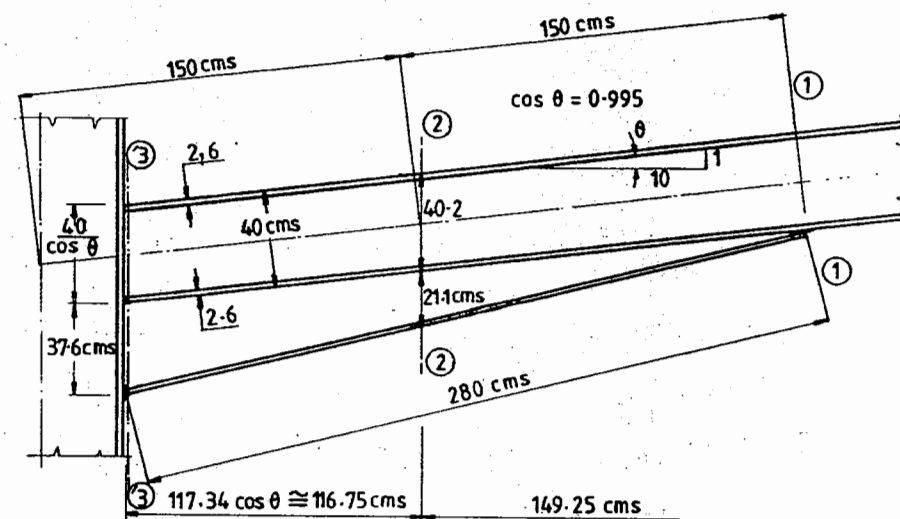
(a) Lay Out



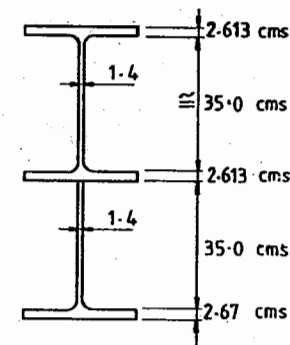
(b) B-M-D and Critical Sections

- Section (3'' - 3'') belonging to the column, the moment $M_3 = 84.1$ t.m will govern the design of the beam-column below the bracket.

2. Strength Requirements:



② - ②



③ - ③



(a) Beam Cross Section (Section 1-1):

$$M_1 = 40 \text{ t.m} \quad Z_x = \frac{40 \times 100}{1.4} = 2857.1 \text{ cm}^3$$

Choose a B.F.I No. 40

$$f_c = \frac{40 \times 100}{3030} = 1.32 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (o.k.)}$$

(b) Beam-Column Cross Section (Section 3'' - 3''):

$$M_{3''} = 84.1 \text{ t.m} \quad Z_x = \frac{84.1 \times 100}{1.4} = 6007.1 \text{ cm}^3$$

Choose a B.I.B No. 65

$$f_c = \frac{84.1 \times 100}{6670} = 1.26 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (o.k.)}$$

(c) Mid-Length of Bracket (Section 2-2):

$$M_2 = 62 \text{ t.m}$$

$$\bar{y} \approx 32.65 \text{ cms} \quad I_{xx} \approx 161800 \text{ cm}^4$$

$$f_c = \frac{62 \times 100 \times 32.65}{161800} = 1.25 \text{ t/cm}^2 \text{ (o.k.)}$$

(d) Bracket End - Section 3' - 3' Passing along the Beam to Column Vertical Plane Connection:

$$M_3 = 88 \text{ t.m}$$

$$I_{xx} \approx \frac{1.4 \times (72.61)^3}{12} + 2 \times 302.61 \times (37.6)^2 \approx 266056 \text{ cm}^4$$

$$f_c = \frac{80 \times 100 \times 39}{266056} = 1.29 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (o.k.)}$$

3. Stability Requirements:

Assuming that adequate bracing is utilized at joints (1) and (3), thus the unbraced length L_{13} is equal to 280 cms.



Equation (b) of section 5.11.5.2 may be utilized to check the safety of the unbraced length against lateral buckling. This means that:

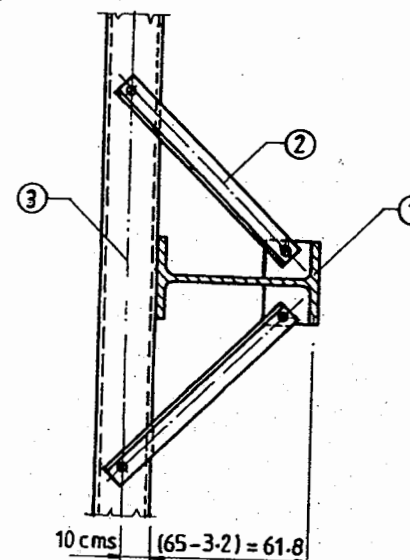
$$\frac{L_{13}}{b} = \frac{280}{30} = 9.33 < 100 \text{ (o.k.)}$$

The bracing structural elements at joints (1) and (3) may be chosen a one angle of size (a). Thus equation (j) of section 5.11.5.4 may be used for the computation of the angle size (a):

$$a \geq \frac{(61.8 + 10)\sqrt{2}}{20} \geq 5.07 \text{ cms}$$

Use one angle 55 x 55 x 6

The bracing elements are assumed to be inclined 45° as shown in the figure below.



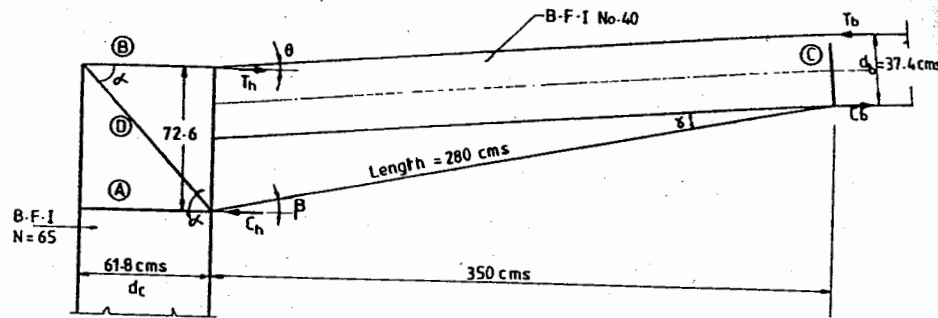
(1) Beam-Column B.F.I No 65.

(2) Bracing Element.

(3) Side Purlin [No 20].



4. Design and Dimensioning of Stiffeners:



$$\theta = 5.8^\circ$$

$$\gamma = 8.2^\circ$$

$$\beta = 14^\circ = \gamma + \theta$$

$$\alpha = 49.6^\circ$$

$$T_h = C_h = \frac{88 \times 100}{72.6} = 121.2 \text{ tons}$$

$$T_b = C_b = \frac{40 \times 100}{37.4} = 106.95 \text{ tons}$$

$$\beta = \gamma + \theta$$

$$\cos \theta = 0.995 ; \sin \theta = 0.101 ; \tan \theta = 0.1$$

$$\cos \gamma = 0.989 ; \sin \gamma = 0.142 ; \tan \gamma = 0.145$$

$$\cos \beta = 0.97 ; \sin \beta = 0.242 ; \tan \beta = 0.25$$

Stiffener (A):

$$\text{Applied force area} = 30 \times 2.6 \times \cos \beta = 75.66 \text{ cm}^2$$

$$\text{Resisting area} = \left[\frac{2.6}{\cos \beta} + 2 \times 2.5 \times 2 \times 3.2 \right] \times 1.7 = 57.08 \text{ cms}$$

Use 2 stiffeners (A) opposite to the bracket lower flange



$$A_{2st A} = 75.66 - 57.08 = 18.58 \text{ cm}^2$$

$$b_{st A} = \frac{30 - 1.7}{2} = 14.15 \text{ cms, use } b_{st A} = 14 \text{ cms}$$

$$t_{st A} = \frac{18.58}{2 \times 14} = 0.66 \text{ cm take } t_{st A} = 0.7 \text{ cm}$$

$$\frac{b_{st A}}{t_{st A}} = \frac{14.0}{0.7} = 20 > 16.1 \text{ unsafe, hence use } t_{st A} = 0.9 \text{ cms}$$

The corresponding size of weld will be governed by the length of the stiffener $L_{st A}$, where:

$$L_{st A} = 65 - 4 \times 3.2 = 52.2 \text{ cms}$$

$$\text{Force resisted by stiffeners A} = C_h - 57.08 \times \frac{F_{crp}}{0.75 \times 2.4} = 18.456 \text{ tons}$$

$$s = \frac{18.456}{2 \times 2 \times 52.2 \times 0.2 \times 3.6} = 0.116 \text{ cm}$$

two 2 weld $L_{st A}$ q_w
stiffeners lines

$$\text{Take } s_{min} = 0.4 \text{ cm}$$

Stiffeners (B):

The bending of the column flange opposite to the beam tension flange is to be checked for simplicity according to the following (Eq. (c) of section 5.3.4):

$$t_{fc} = 0.4 \sqrt{30 \times 2.6 \times \cos \theta} = 3.52 \text{ cms} > 3.2 \text{ cms unsafe}$$

Hence use two horizontal stiffeners (B) opposite to the beam tension flange as in the previous paragraph (i.e. 14 x 0.9 cm).



N.B. 1. The exact required cross sectional areas of stiffeners (B) can be computed using the first principle of the yield line theory as example 5.2.

2. The tension beam flange may be extended to act as a stiffener.

Stiffeners (C):

Apply in equation (d) of section 5.11.4

$$\text{Force resisted by stiffeners (C)} = C_h \frac{\sin \gamma}{\cos \beta} = 121.2 \times \frac{0.142}{0.97} = 17.74 \text{ tons}$$

$$A_{st-c} = \frac{17.74}{1.4} = 12.67 \text{ cm}^2 ; b_{st A} = 14 \text{ cms}$$

$$t_{st c} = \frac{12.67}{2 \times 14} = 0.45 \text{ cm use } t_{st} = 1.0 \text{ cm to prevent local buckling}$$

$$L_{st} = \frac{17.74}{2 \times 2 \times 0.2 \times 3.6 \times 0.4} = 15.39 \text{ ms take } 17 \text{ cms}$$

$\swarrow \quad \searrow \quad \quad \quad \swarrow \quad \searrow$
 2 stiff. 2 welds q_w s

Diagonal Stiffeners (D):

$$\text{Force resisted by diagonal stiffeners} = F_D = \frac{C_h - Q_{horiz}}{\cos \alpha}$$

$$C_h = 121.2 \text{ tons}$$

$$Q_{horiz} = (65 - 3.2) \times 1.7 \times 0.84 = 88.25 \text{ tons}$$

$$\cos \alpha = 0.635$$

$$F_D = 51.9 \text{ tons} ; b_{st D} = 14.0 \text{ cms}$$

$$t_{st D} = \frac{51.9}{2 \times 1.4 \times 14} = 1.32 \text{ cm take } t = 1.4 \text{ cm}$$



Morris Type Stiffener:

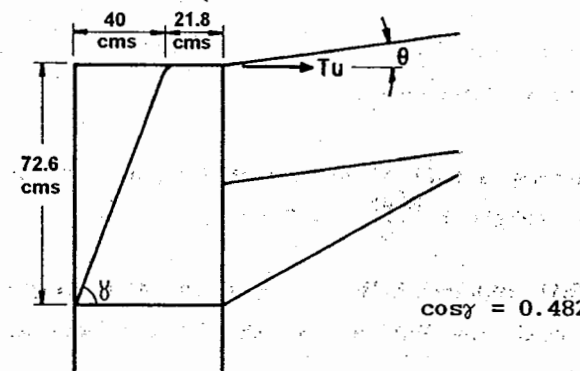
$$\text{Horizontal portion force} = T_u - P_u \text{ (first principles)}$$

$$T_u = 30 \times 2.6 \times 2.4 \times \cos \theta = 186.26 \text{ tons}$$

$$P_u = 0.8 (7 \times 2.4 \times 3.2^2 + 0.15 \times 30 \times 2.6 \times 2.4) = 160 \text{ tons}$$

$$\text{Hence Horizontal portion force} = 26.26 \text{ tons}$$

$$\text{Diagonal portion force} = (121.2 - 88.25) / \cos \gamma = 68.36 \text{ tons}$$



As the Morris type is a continuous stiffener the diagonal portion will govern its dimensioning (i.e. Force = 68.36 tons)

Regarding Stiffeners (B) and (C) the previous design remains unchanged .

$$\text{i.e. take } b_{st} = 14 \text{ cms} ; t_{st} = 1.8 \text{ cms.}$$

Redesign the different steel components of the gable rigid frame of example 5.21 using a tapered connection.

Solution

Refer to section 5.11.5.

(1) Geometrical Configuration:

(a) Draw the B.M.D on the center lines of the rafter and the beam-column. For simplicity the bending moment diagram for the tapered frame configuration may be taken as for the corner with bracket frame configuration.

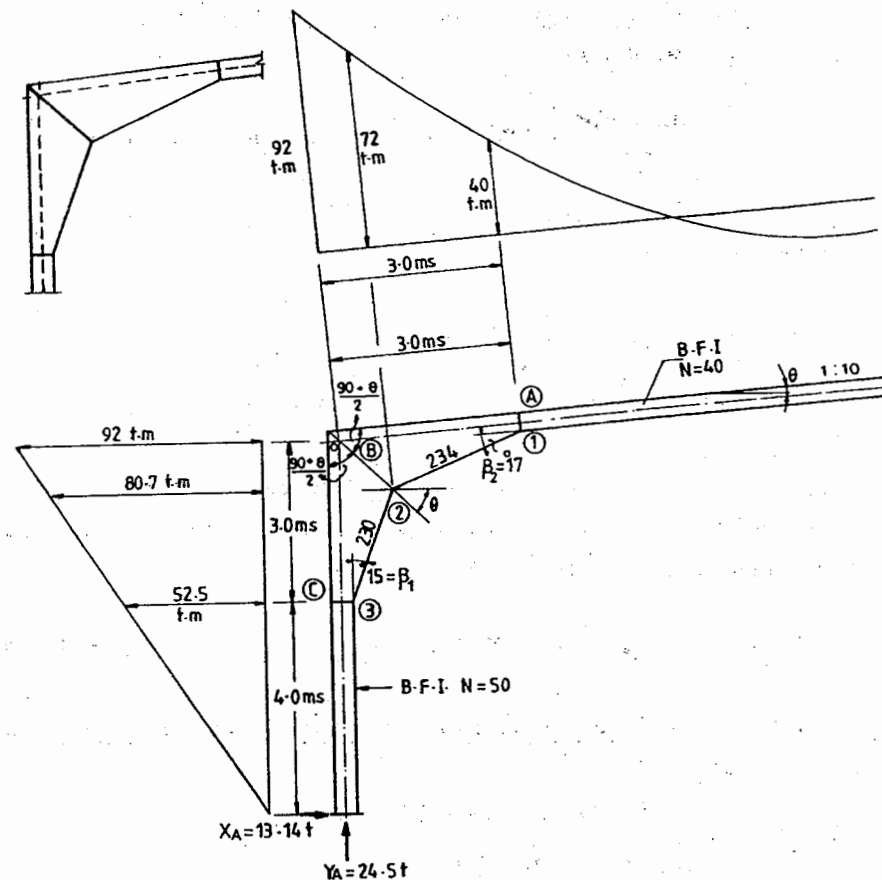
(b) Assume a tapered haunch of length equal to one tenth the span (i.e. length = $\frac{\text{SPAN}}{10} = 3.0$ ms).

(c) This tapered haunch length is to be taken on the center lines of both the rafter and the beam-column as shown in the figure below.

(d) Draw the bisectrice of the angle corresponding to the intersection of the exterior beam-column flange and the upper rafter flange. The induced angle is equal to $(\frac{90^\circ + \theta}{2})$.

(e) From joint (3) belonging to the end beam-column haunch compose the flange 3-2 whose inclination β_1 with respect to the vertical axis is equal to 15° .

(f) Join joints (2) and (1) to get the haunch flange 1-2 whose inclination β_2 with respect to the beam axis is 17° .



(g) Compute the haunch flange lengths

$$L_{32} = 230 \text{ cms} \quad ; \quad L_{21} = 234 \text{ cms.}$$

(h) Compute the following moments:

$$M_3 = 52.5 \text{ t.m} \quad ; \quad M_1 = 40 \text{ t.m}$$

$M_2 = 80.7$ t.m belonging to the beam-column.

$M_2 = 72.0 \text{ t.m}$ belonging to the rafter.

(2) Strength Requirements:(a) Section (1-1) belonging to the rafter constant depth:

$$M_1 = 40.0 \text{ t.m}$$

$$Z_x = \frac{40 \times 100}{1.4} = 2857.1 \text{ cm}^3, \text{ choose a B.F.I No. 40.}$$

$$f_c = \frac{40 \times 100}{3030} = 1.32 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (o.k)}$$

The compression rafter flange just to the left of joint (1) will be subjected to the following compressive stress:

$$f_c = \frac{1.32}{\cos \beta_2} = \frac{1.32}{\cos 17^\circ} \cong 1.4 \text{ t/cm}^2 \text{ (o.k)}$$

Hence no need to increase the haunch flange thickness. Consequently the haunch dimensions may be chosen equal to those which corresponds to the rafter.

$$\text{Hence: } t_{hf} = 2.6 \text{ cms ; } t_{hw} = 1.4 \text{ cm ; } b_h = 30 \text{ cms}$$

(b) Section (3-3) belonging to the beam-column constant depth:

$$M_3 = 52.5 \text{ t.m ; } N = 24.5 \text{ tons}$$

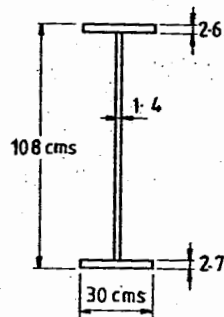
Choose a B.F.I No. 50

Compute the induced stresses and compare with the allowable limits (safe) o.k

(c) Section (2-2) belonging to the beam-column:

$$M_2 = 80.7 \text{ t.m}$$

Neglect the increase of the tapered haunch flange due to the inclination.



Hence:

$$I_x \cong 558800 \text{ cm}^4$$

$$f_c = \frac{80.7 \times 100 \times 54}{558800} = 0.78 \text{ t/cm}^2 < 1.4 \text{ (o.k)}$$

(d) Section (2-2) belonging to the rafter:

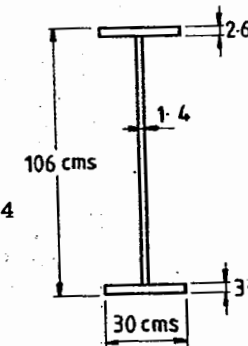
$$M_2 = 72 \text{ t.m}$$

Neglect the variation of thicknesses

Hence:

$$I_x = 530600 \text{ cm}^4$$

$$f_c = \frac{72 \times 100 \times 53}{530600} = 0.72 \text{ t/cm}^2 < 1.4$$

(3) Stability Requirements:

$$\frac{L_{12}}{b_h} = \frac{234}{30} = 7.8 < 100 \text{ (o.k)}$$

$$\frac{L_{23}}{b_h} = \frac{230}{30} = 7.66 < 100 \text{ (o.k)}$$

$$\frac{b_h}{t_b} = \frac{30 - 1.4}{2 \times 2.6} = 5.5 < 16.1 \text{ (o.k)}$$

Provided that adequate bracing is utilized at joints (1), (2) and (3) the stability requirements are satisfied.

For the design of the bracing elements refer to equation (j) of section 5.11.5.4. Choose a one angle section with a leg size

(a) computed as follows:

$$a \geq \frac{[(50 - 3) + 10] \sqrt{2}}{20} \geq 4.03 \text{ cms}$$

Choose a one angle 45 x 45 x 5

(a) Diagonal Stiffener (D):

$$F_D \cos 42.1^\circ + 79.1 \times \sin 15^\circ = 72.8 \cos (17^\circ + 5.8^\circ)$$

Get $F_D = 62.85$ tons

$$\frac{(106 - 2.6)}{\tan(5.8 + 42.1)} \times 0.84 \times 1.4 = 112.63 \text{ tons}$$
$$\frac{72 \times 100}{(106 - 2.6)} \cos 5.8^\circ = 112.63 \cos 5.8^\circ + F_D \cos 42.1^\circ$$

Get $F_D = 58.43$ tons

$$F_D = 62.85 \text{ tons}$$

Choose 14.0 x 1.6 cms

$$F_{st(A)} = (M'_2 / d'_2) \tan \beta_2 = [72.0 / (106 - 2.6)] \tan 17^\circ$$

$$F_{st(B)} = (M_2'' / d_2'') \tan \beta_1 = [80.7 / (108 - 2.6)] \tan 15^\circ$$

Choose for stiffeners (A) and (B) (14.0 x 0.9) cms

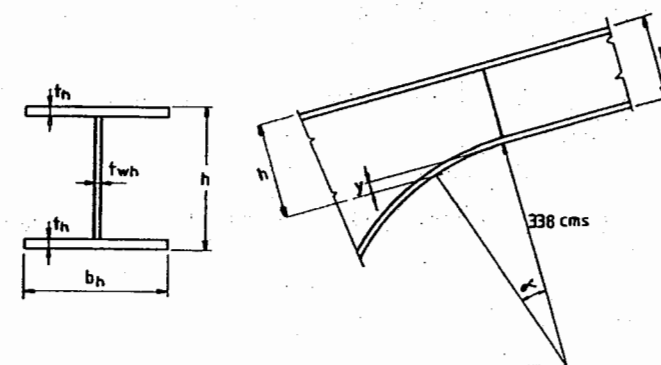
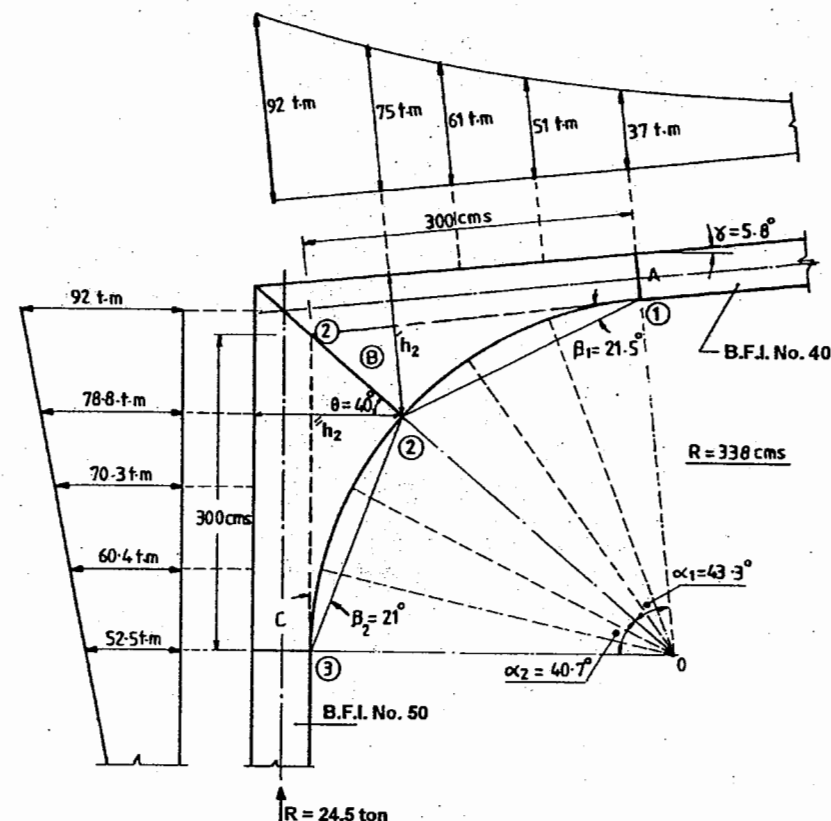
Example (5.16):

Design the connection previously outlined in example 5.21 using a curved haunched connection.

Solution

1. Geometrical Configuration:

- For simplicity use the moment diagram of ex. 5.22.
- The rafter and the beam-column are chosen a B.F.I No. 40 and No. 50 respectively.
- Draw the inner faces of the rafter and the beam-column (shown by the dotted lines).
- From the intersection point (2') of these inner flange faces take the lengths (2'-1) and (2'-3) equal to 300 cms.
- Draw perpendiculars from (1) and (3) to the rafter and to the beam-column respectively to intersect at the center of the curved flange (0).
- Construct the curved haunch 1-3, where the corresponding radius R is equal to 338 cms.
- Construct the diagonal stiffener (B) and get its intersection point (2) with the curved haunch flange. Compute the moments belonging to the rafter and the column.
- Get the inclinations $\alpha_1 = 40.7^\circ$ and $\alpha_2 = 43.3^\circ$.
- Divide α_1 and α_2 into three regions and get the corresponding moments where:
 - The sections regarding α_2 belong to the beam-column.
 - The sections regarding α_1 belonging to the rafter.



$$h = h_b + (1 - \cos \alpha) R$$



2. Strength Requirements:

(a) Section (1-1) - Rafter:

$$M_1 = 37 \text{ t.m} - \text{B.F.I No. 40} \quad Z_x = 3030 \text{ cm}^3$$

$$f_1 = \frac{37 \times 100}{3030} = 1.22 \text{ t/cm}^2$$

Due to the abrupt change in compression flange at joint (1), a conservative approach is to consider the tapered flange inclination $\beta_1 = 21.5^\circ$.

$$\text{Hence: } f_1 \text{ (haunch flange)} = \frac{1.22}{\cos 21.5^\circ} = 1.31 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \quad (\text{o.k.})$$

(b) Section (3-3) Beam Column:

$$M_3 = 52.5 \text{ t.m} \quad Z_x \text{ (B.F.I No. 50)} = 4530 \text{ cms}$$

$$A = 255 \text{ cm}^2$$

$$f_3 = \frac{52.5 \times 100}{4530} + \frac{24.5}{255} = 1.25 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \quad (\text{o.k.})$$

$$\beta_2 = 21^\circ$$

$$\text{Hence: } f_3 \text{ (haunch flange)} = \frac{1.25}{\cos 21^\circ} = 1.34 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \quad (\text{o.k.})$$

(c) Section (2-2) belonging to the rafter and the beam-column:

Choose for the tension flange, the compression flange and the web of the curved haunch the same dimensions of the rafter.

Hence:

$$b_h = b_b = 30 \text{ cms} ;$$

$$t_h = t_b = 2.6 \text{ cms} ;$$

$$t_{wh} = t_{wb} = 1.4 \text{ cm}$$



$$h_2 = h_b + (1 - \cos \alpha_1) R = 40 + (1 - \cos 43.3^\circ) 338 \approx 132 \text{ cms.}$$

$$I_{xx} = \frac{(132 - 2 \times 2.6)^3}{12} \times 1.4 + 2 \times 30 \times 2.6 \times \left[\left(\frac{132}{2} - 1.3 \right) \right]^2$$

$$I_{xx} = 890880 \text{ cm}^4 ; \quad A = 333.52 \text{ cm}^2$$

$$f_2 \text{ (rafter)} = \frac{75 \times 100 \times 66}{890880} = 0.55 \text{ t/cm}^2$$

$$h_2 = 50 + (1 - \cos 40.7^\circ) 338 = 131.8 \approx 132$$

$$f_2 \text{ (beam-column)} = \frac{24.5}{333.52} + \frac{78.8 \times 100 \times 66}{890880} = 0.65 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2$$

(d) Sections Corresponding to $(\alpha_1/3)$ and $(2\alpha_1/3)$ Belonging

to the Rafter:

$$\alpha_1/3 \quad h = 40 + (1 - \cos \frac{43.3^\circ}{3}) 338 = 50.66 \text{ cms}$$

$$I_x = 101040 \text{ cm}^4$$

$$f_2 = \frac{51 \times 100 \times 50.66}{101040 \times 2} = 1.28 \text{ t/cm}^2 \quad (\text{o.k.})$$

$$(2\alpha_1/3) \quad h = 40 + (1 - \cos \frac{2 \times 43.3^\circ}{3}) 338 = 82 \text{ cms}$$

$$I_x = 298718 \text{ cm}^4$$

$$f_2 = \frac{61 \times 100 \times 82}{298718 \times 2} = 0.83 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \quad (\text{o.k.})$$

(e) Sections Corresponding to $(\alpha_2/3)$ and $(2\alpha_2/3)$ Belonging

to the Beam-Column:

$$\alpha_2/3 \ h = 50 + (1 - \cos \frac{40.7}{3}) 338 = 59.43 \text{ cms}$$

$$I_x = 545745 \text{ cm}^4$$

$$A = 231.92 \text{ cm}^2$$

$$f_2 = \frac{24.5}{231.92} + \frac{60.4 \times 100 \times 59.43}{545745 \times 2} = 0.43 \text{ t/cm}^2 \text{ (o.k.)}$$

$$2 \alpha_2/3 \ h = 50 + (1 - \cos \frac{2 \times 40.7}{3}) 338 = 87.2 \text{ cms}$$

$$I_x = 1215420 \text{ cm}^4$$

$$A = 270.8 \text{ cm}^2$$

$$f_2 = \frac{24.5}{270.8} + \frac{70.3 \times 100 \times 87.2}{1215420 \times 2} = 0.34 \text{ t/cm}^2 \text{ (o.k.)}$$

3. Stability Requirements:

$$\text{Arc length } 1 - 2 = \frac{43.3 \times \pi}{180} \times 338 = 255 \text{ cms}$$

$$\frac{\text{Arc } 1-2}{b} = \frac{255}{30} = 8.5 < 100 \text{ (o.k.)}$$

$$\text{Arc length } 2 - 3 = \frac{40.7 \times \pi}{180} \times 338 = 239.9 \text{ cms}$$

$$\frac{\text{Arc } 2-3}{b} = \frac{239.9}{30} = 8.0 < 100 \text{ (o.k.)}$$

4. Cross Bending Effect:

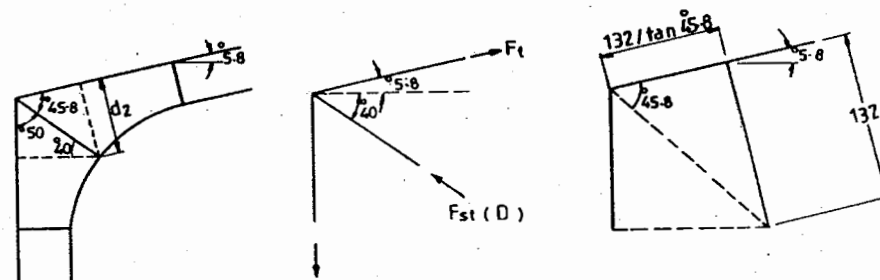
Apply in equation (f) of section 5.11.6.2

$$\frac{b^2}{R t_h} = \frac{(30)^2}{338 \times 2.6} = 1.024 < 1.33 \text{ (o.k.)}$$

5. The Stiffeners:

(a) Diagonal Stiffener:

Based on Tension Force



$$F_t = \frac{75 \times 100}{(132 - 2.6)} = 57.95 \text{ tons}$$

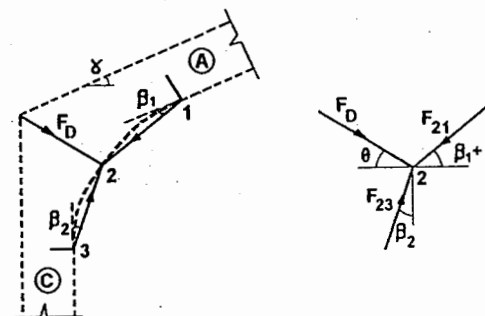
$$\text{Resisting force} = \frac{132}{\tan 45.8} \times t_{wh} \times (0.6 \times 1.4) = 151 \text{ tons}$$

$$151 \text{ tons} > 57.95 \text{ tons (safe)}$$

Based on Compression Force

$$F_{21} = \frac{75 \times 100}{(132 - 2.6) \cos 21.5} = 62.29 \text{ tons}$$

$$F_{23} = \frac{78.8 \times 100}{(132 - 2.6) \cos 21} = 65.22 \text{ tons}$$



$$\begin{aligned} \gamma &= 5.8^\circ \\ \beta_1 &= 21.5^\circ \\ \beta_2 &= 21^\circ \\ \theta &= 40^\circ \end{aligned}$$



$$F_{st(D)} \cos 40^\circ + F_{23} \sin 21^\circ - F_{21} \cos 27.3^\circ = 0$$

$$F_{st(D)} = 42.4 \text{ tons}$$

$$b_{st} = \frac{30 - 1.4}{2} = 14.3 \text{ cms take } b_{st} = 14.0 \text{ cms}$$

$$A_{st} = \frac{42.4}{1.4} = 30.3 \text{ cm}^2$$

$$t_{st} = \frac{30.3}{2 \times 14} = 1.08 \text{ take } t_{st} = 1.1 \text{ cm}$$

$$b_{st}/t_{st} = \frac{14}{1.4} = 13 < 16.1 \text{ (o.k.) for local buckling.}$$

(b) Transverse Stiffeners (A) and (C):

$$F_{21} = 62.29 \text{ tons}$$

$$F_{st(A)} = \frac{62.29 \times \sin \beta_1}{\cos \beta_1} = 58.25 \tan 21.5^\circ = 23.0 \text{ tons}$$

$$F_{23} = 65.22 \text{ tons}$$

$$F_{st(c)} = 65.22 \tan \beta_2 = 65.22 \tan 21^\circ = 25.0 \text{ tons}$$

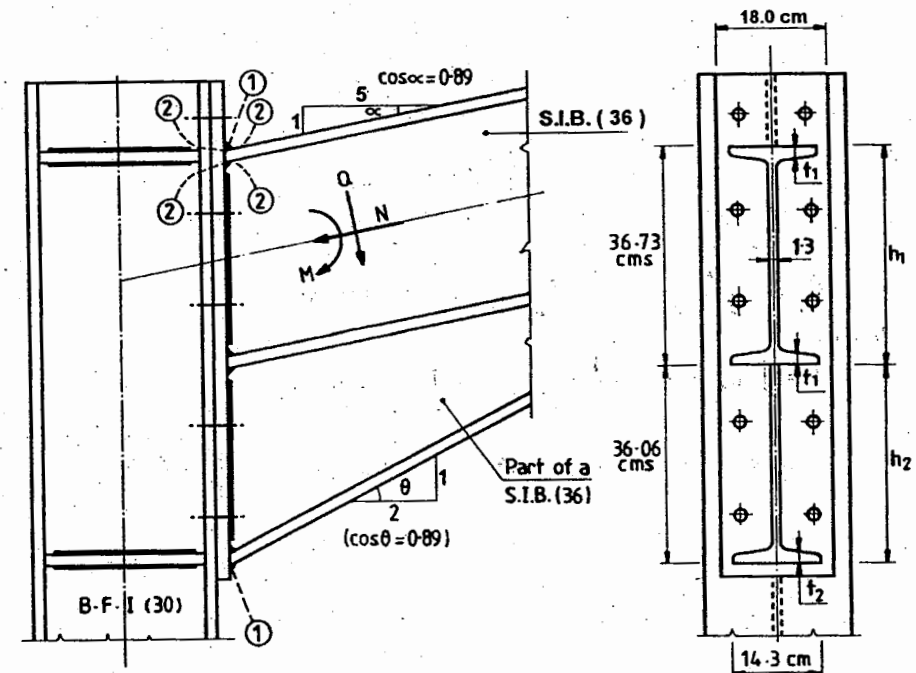
Choose for stiffeners (A) and (c) 14.0 x 0.9 cms



Example (5.17): Rigid Frame Haunched Connection:

Design a rigid frame connection using a haunch. The connection is subjected to a moment "M" = 21.0 mt and shear "Q" = 7.50 ton. The axial thrust in the beam is "N" = -4.3 ton. The beam is a S.I.B.# 36 and the column is B.F.I.# (30).

Solution



The haunch will be a part of a S.I.B.# 36 with a height equals to $(36 - 2 \times 1.95) = 32.10 \text{ cm}$.

$$t_1 = 1.95/\cos \alpha = 1.989 \cong 2.0 \text{ cms; } t_2 = 1.95/\cos \theta = 2.19 \text{ cms} \cong 2.2 \text{ cms.}$$

$$h_1 = 36/\cos \alpha = 36.73 \text{ cms; } h_2 = (36 - 2 \times 1.95)/\cos \theta = 36.06 \text{ cms.}$$

$$h = 36.73 + 36.06 = 72.79 \text{ cms.}$$

Approximate Analysis:

High Strength Bolts:

$$\text{Tension} = \frac{21 \times 100}{72.79 - \left(\frac{2 + 2.2}{2}\right)} - \frac{4.3}{2} = 27.55 \text{ tons.}$$

$$\text{External force on each bolt} = \frac{27.55}{4} = 6.88 \text{ tons.}$$

$$\text{Required Pretension} = \frac{6.88}{0.8} = 8.6 \text{ tons.}$$

Use H.S.B. M₂₀ (8.8), where T = 15.43 x 0.7 = 10.8 tons.

(1) Welds:

(a) Fillet Welds Around the Beam Flanges:

Plane (2-2):

$$F_w = \frac{M}{dx \cos \alpha} = \frac{21 \times 100}{71.69 \times 0.99} = 29.89 \text{ tons.}$$

Where:

$$d = \left[h_1 + h_2 - \frac{1}{2} (t_1 + t_2) \right] = 71.69 \text{ cms.}$$

$$\text{Length of horizontal weld lines} = 14.3 + 2 \left[\frac{(14.3 - 1.3)}{2} - 1.95 \right] = 23.4 \text{ cms.}$$

$$s = \frac{29.89}{23.4 \times 0.72} = 1.77 \text{ cms. , Take } s = 1.8 \text{ cms.}$$

(b) Fillet Welds Around the Web:

$$\text{Web vertical lines of weld} = 36.73 - 4 \times 2 + 36.06 - 2 \times 2.2 = 60.39 \text{ cms.}$$

$$s = \frac{7.5}{60.39 \times 0.72} = 0.16 \text{ cms; Take } s = 0.4 \text{ cms.}$$

For the exact calculation of stresses, the fillet welds around the beam lower flange are neglected for simplicity.

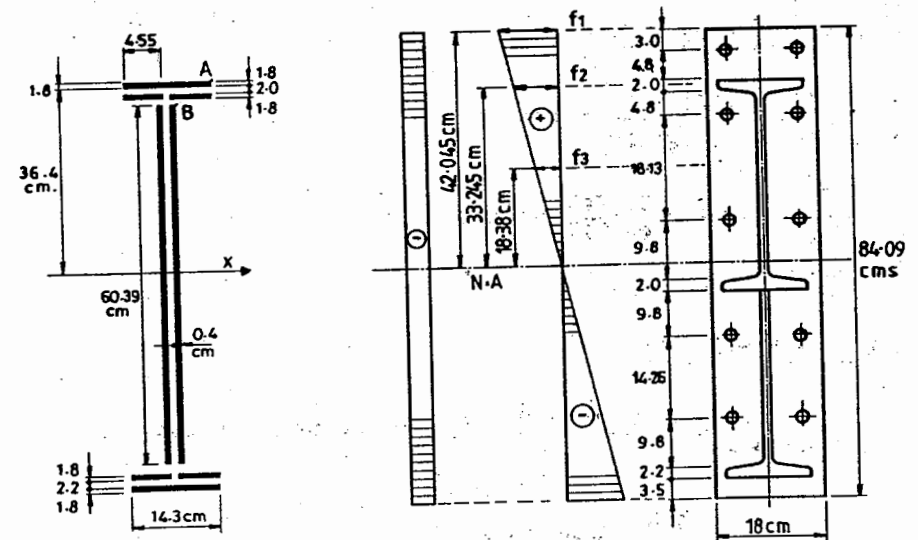
$$I_{xx} = 2 \times 0.4 \times \frac{60.39^3}{12} + 2 \times 14.3 \times 1.8 \times \left(\frac{72.79 + 1.8}{2} \right)^2 + 4 \times 4.55 \times 1.8 \times \left(\frac{72.79 - 2 - 2.2 - 1.8}{2} \right)^2 = 122821.83$$

$$f_A = \frac{2100 \times \left(\frac{72.79}{2} + 1.8 \right)}{122821.83} = 0.653 \text{ t/cm}^2 = 0.65 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ o.k.}$$

$$f_B = 0.653 \times \frac{30.595}{38.275} = 0.521 \text{ t/cm}^2.$$

$$q_B = \frac{7.5}{2 \times 0.4 \times 60.39} = 0.15 \text{ t/cm}^2.$$

$$q_{eq(B)} = \sqrt{(0.521)^2 + 3(0.15)^2} = 0.58 \text{ t/cm}^2 < 0.79 \text{ t/cm}^2 \text{ (o.k.)}$$



Sec. (1 - 1)

End Plate Thickness:

$$L = 4.8 + 2.0 + 4.8 \\ = 11.6 \text{ cms.}$$

$$T = \frac{M}{d'} - \frac{N}{2} = \frac{2100}{70.69} - \frac{4.3}{2} = 27.55 \text{ tons.}$$

Where:

$$d' = \left[h_1 + h_2 - \frac{1}{2} (t_1 + t_2) \right] = 70.69$$

$$M = \frac{27.55 \times 11.6}{8} = 40.0 \text{ t.cm.}$$

$$t_{\text{required}} = \sqrt{\frac{6 \times 40}{18 \times 1.4}} = 3.1 \text{ cms.}$$

Take $t = 3.5 \text{ cms.}$

Exact Analysis:

High-Strength Bolts:

$$f_1 = \frac{2100 \times 6}{18 \times 84.09^2} = 0.099 \text{ t/cm}^2$$

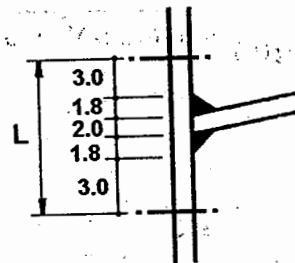
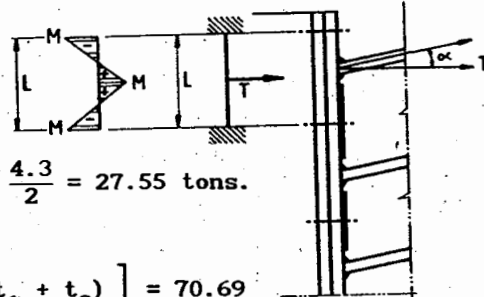
$$f_2 = 0.099 \times \frac{33.245}{42.045} = 0.078 \text{ t/cm}^2$$

Group of H.S.B "1":

$$T_{\text{ext, b, M}} = \frac{1}{2} \left(\frac{0.099 + 0.078}{2} \right) \times 8.8 \times 18 = 7.0 \text{ tons.}$$

$$T_{\text{ext, b}} = \frac{4.3}{10} = 0.43 \text{ tons.}$$

$$T_{\text{total}} = 7 - 0.43 = 6.579 \text{ tons} < 0.8 \times 15.43 \times 0.7 \text{ (o.k.)}$$



Check of Prying force in Bolts:

$$P = \left[\frac{\frac{1}{2} - \frac{w t^4}{30 a b^2 A_s}}{\frac{3a}{4b} \left(\frac{a}{4b} + 1 \right) + \frac{w t^4}{30 a b^2 A_s}} \right] T_{\text{total}}$$

$$a = 3.0 \text{ cms.}, b = 3.0 \text{ cms.}, w = 9 \text{ cms.}, A_s = 2.45 \text{ cm}^2.$$

$$P = 0.017 \times 6.579 = 0.11 \text{ tons.}$$

$$\text{Total force carried by each bolt} = 6.579 + 0.11 = 6.68 \text{ tons} \\ < 10.8 \text{ t o.k.}$$

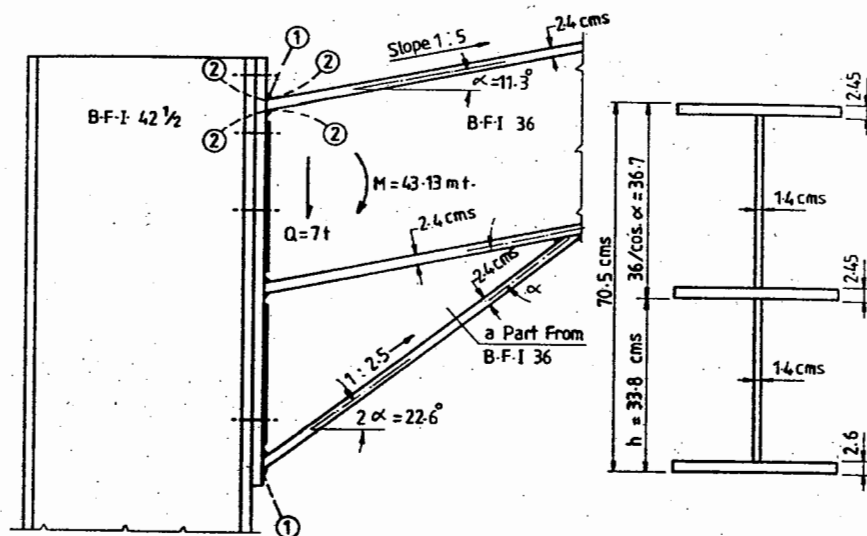
$$M_1 = 3 \times 0.11 = 0.33 \text{ t.cm.}$$

$$M_2 = 6.579 \times 3 - 0.33 = 19.4 \text{ t.cms.}$$

$$t_{\text{req}} = \sqrt{\frac{6 \times 19.4}{1.4 \times 9}} = 3.03 \text{ cms.} < 3.5 \text{ cms o.k.}$$

N.B. The crippling of column web, the tension of the column flange and the stability of the corner web are to be checked as Example 5.13.

Example (5.18):



Height of the haunch (h)

$$h = \frac{36 - 2 \times 2.4}{\cos 2\alpha} = 33.8 \text{ cms.}$$

(1) Design of Welds: (Fillet

Welds):

Sec (2-2):

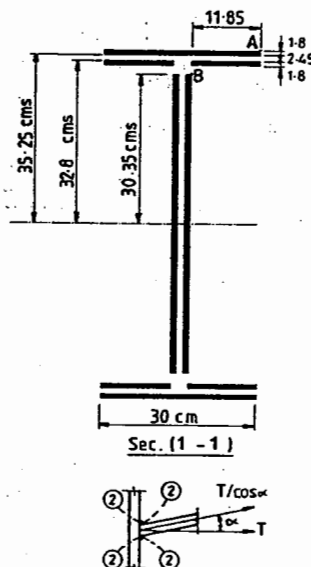
$$\frac{T}{\cos \alpha} = \frac{M}{d \cos \alpha} = \frac{4313}{(70.0 - 2.45) \cos \alpha}$$

$$= 64.6 \text{ t}$$

Length of welds around

tension flange = $30 + 2 \times 11.85$

$$= 53.7 \text{ cms.}$$



$$s = \frac{64.6}{53.7 \times 0.72} = 1.66 \text{ cms} \quad \text{Take } s = 1.8 \text{ cms.}$$

Weld Around Web:

$$q = \frac{7}{2 \times h \times s} \quad \text{Where } h = 70.5 - 4 \times 2.45 - 2 \times 2.6 = 55.5 \text{ cms}$$

with $q = 0.72 \text{ t/cm}^2$ Get $s = 0.087$, Take $s = 4 \text{ mms.}$

Sec (1-1) (Exact):

∴ Take s around flanges = 1.8 cms. & s around web = 0.4 cms.

$$I_{xx} = \frac{1}{12} (55.5)^3 \times 0.4 \times 2 + 4 \times 11.85 \times 1.8 \times (32.8 - 0.9)^2$$

$$+ 2 \times 30 \times 1.8 (35.25 + 0.9)^2 = 242860 \text{ cm}^4.$$

$$f_{tB} = \frac{4313}{I_{xx}} \times 30.35 = 0.54 \text{ t/cm}^2.$$

$$q_B = \frac{7.0}{2 \times 60.7 \times 0.4} = 0.14 \text{ t/cm}^2.$$

$$f_{eq.B} = \sqrt{(0.54)^2 + 3(0.14)^2} = 0.6 \text{ t/cm}^2 < 0.79 \text{ t/cm. (Safe).}$$

$$f_{tA} = \frac{4313}{I_{xx}} (35.25 + 1.8) = 0.66 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2. \quad (\text{safe})$$

(2) Design of the H.S.B:

(2-a) Approximate Analysis:

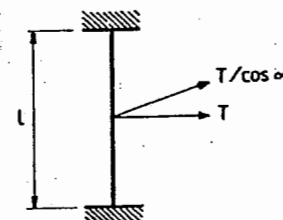
$$T = \frac{4313}{[70.5 - (2.45/2) - (2.6/2)]} = 63.4 \text{ t}$$

Assume we have 2 rows each 4 bolts

$$\therefore \text{Tension /bolt} = \frac{63.4}{8} = 7.92 \text{ tons.}$$

$$\text{Try M(20) (10.9)} \quad T_{all} = 15.43 \times 0.8 = 12.4 \text{ t}$$

$$P_s = 4.93 \text{ t}$$





(2-b) Design of End Plate:

$$L = 3d + 2s + \frac{t_{fb}}{\cos \alpha}$$

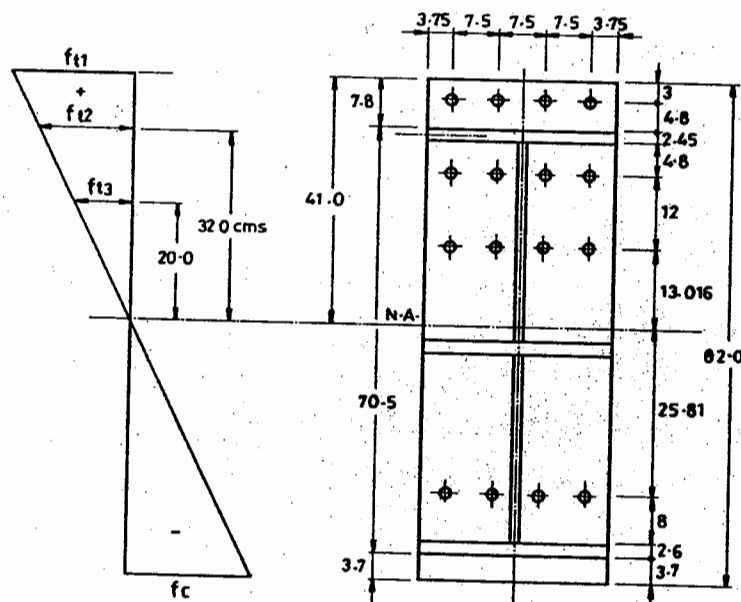
$$L = 3 \times 2 + 2 \times 1.8 + \frac{2.4}{\cos \alpha} = 12.05 \text{ cms.}$$

$$M = \frac{TL}{8} = \frac{63.4 \times 12.05}{8} = 95.49 \text{ t/cm}$$

$$t = \sqrt{\frac{6M}{b \times 1.4}} = \sqrt{\frac{6 \times 95.49}{30 \times 1.4}} = 3.7 \text{ cms.}$$

Take $t = 3.7$ cms.

(2-c) Exact Analysis of H.S.B:



$$I_x = \frac{1}{12} (82.0)^3 \times 30 = 1378420 \text{ cm}^4.$$

$$f_{t1} = \frac{M}{I} y_1 = \frac{4313}{1378420} \times 41.0 = 0.128 \text{ t/cm}^2.$$

$$f_{t2} = \frac{M}{I} y_2 = \frac{4313}{1378420} \times 32.0 = 0.1 \text{ t/cm}^2.$$

$$f_{t3} = \frac{M}{I} y_3 = \frac{4313}{1378420} \times 20.0 = 0.062 \text{ t/cm}^2.$$

Group (1):

$$T_{\text{ext},b,M} = \left(\frac{0.128 + 0.1}{2} \right) 30 \times \frac{9.025}{4} = 7.72 < 15.43 \times 0.8 \text{ t} \quad (\text{Safe}).$$

Group (2):

$$T_{\text{ext},b,M} = \left(\frac{0.1 + 0.062}{2} \right) 30 \times \frac{12.025}{4} = 7.3 \text{ t} < 15.43 \times 0.8 \text{ t} \quad (\text{Safe}).$$

Group (3):

$$T_{\text{ext},b,M} = \frac{1}{2} 0.062 \times 30 \times \frac{19.0165}{4} = 4.42 \text{ t} < 15.43 \times 0.8 \text{ t Safe}.$$

(3) Check of Shear on Bolts:

$$Q/\text{bolt} = \frac{7.0}{4} = 1.750 \text{ tons} < P_s < 4.93 \text{ t (Safe).}$$

(4) Check the Effect of Prying Force:

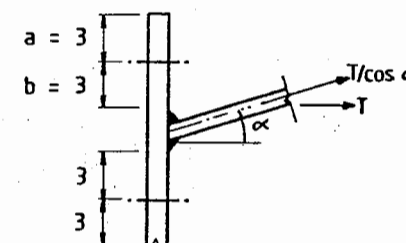
$$a = 3.0 \text{ cms.}, \quad b = 3.0 \text{ cms.}$$

$$w/\text{bolt} = 7.5 \text{ cms.}$$

$$A_{p/2} = 2.45 \text{ cm}^2.$$

$$t = 3.8 \text{ cms.}$$

$$\text{Force/bolt} = 8.07 \text{ tons}$$



$$P = \left[\frac{\frac{1}{2} - \frac{w t^4}{30 a b^2 A_s}}{\frac{3a}{4b} \left(\frac{a}{4b} + 1 \right) + \frac{w t^4}{30 a b^2 A_s}} \right] \cdot T_{\text{ext}, b, M}$$

$$P = \left[\frac{\frac{1}{2} - \frac{7.5 (3.8)^4}{30 \times 3 \times 9.0 \times 2.45}}{\frac{3 \times 3}{4 \times 3.0} \left(\frac{3}{4 \times 3.0} + 1 \right) + \frac{7.5 (3.8)^4}{30 \times 3 \times 9 \times 2.45}} \right] \cdot T_{\text{ext}, b, M}$$

Q/bolt = - ve no prying force.

Moment = $8.07 \times 3 = 24.21$ ton/cm

$$t_p = \sqrt{\frac{24.21 \times 6}{7.5 \times 1.4}} = 3.72 \text{ cms} > 3.7 \text{ cms.}$$

$\therefore t_{ep} = 3.8 \text{ cms.}$

(5) Check of Crippling:

If areas are compared:

k = column fillet length

$$k = 2 t_{fc} = 2 \times 2.6 = 5.2 \text{ cms.}$$

$$L = 2 (2.5 k + t_{ep}) + 2.45 / \cos 2\alpha$$

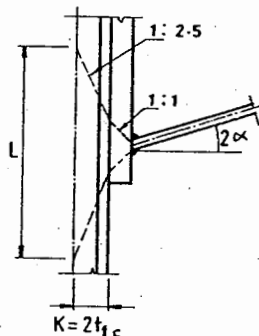
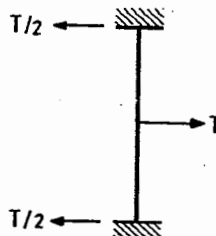
$$L = (2.6 + 2 \times 3.8 + 2 \times 2.5 \times 5.2) = 36.20 \text{ cms.}$$

$$\begin{aligned} \text{Resisting area} &= L \times t_{wc} \\ &= 36.20 \times 1.4 \\ &= 50.68 \text{ cms.} \end{aligned}$$

$$\begin{aligned} \text{Applied area} &= \text{Area of beam flange} \\ &= 30 \times 2.4 \times \cos 2\alpha \\ &= 66.47 \text{ cm}^2. \end{aligned}$$

Applied area > Resisting area

(Unsafe).



Use compression stiffeners:

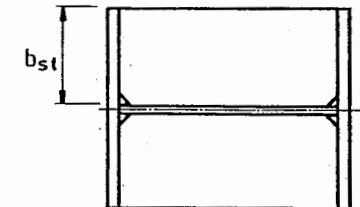
$$\begin{aligned} A_{\text{one stiff}} &= \frac{66.47 - 50.68}{2} \\ &= 3.95 \text{ cm}^2. \end{aligned}$$

$$b_{st} = \frac{b_{fc} - t_{wc}}{2} = 14.3 = 14.0$$

$$(1) t_{st} = \frac{3.95}{14.0} = 0.3 \text{ cms.}$$

$$(2) t_{st} = \frac{14.3}{16} = 0.894 \text{ cms.}$$

From (1) & (2), take $t_{st} = 0.9 \text{ cms.}$



(6) Check Bending of Tension Column Flange:

Min. thickness required to column flange:

$$t_{fc} \geq 0.4 \sqrt{b_b \times t_b \cos \alpha} \quad \alpha = \tan^{-1} \frac{1}{5}$$

$$t_{fc} \geq 0.4 \sqrt{30 \times 2.4 \times 0.98} = 3.36 \text{ cms. (Unsafe).}$$

Use 2 tensioning stiffeners having the same cross section as chosen before (14.3 x 0.9).

(7) Stability Check of the Corner Web:

$$\text{Force acting} = T_{\text{horiz}} = 63.4 \text{ tons}$$

$$\begin{aligned} \text{Resistance} &= 42.5 \times 1.4 \times 0.84 \\ &= 49.98 \text{ tons.} \end{aligned}$$

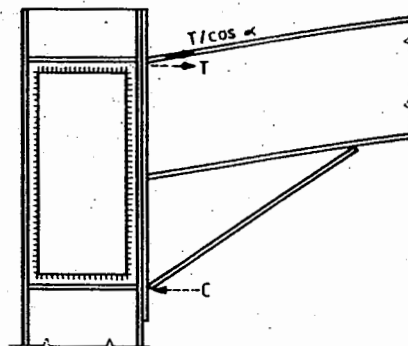
$$\text{Remaining Shear} = 63.4 - 49.98 = 13.43 \text{ tons.}$$

- (a) Choose 2 Cover Plates to Resist the Remaining Force:

$$13.42 = 0.84 \times 2 t' \times (42.5 - 4 \times 2.6)$$

$$t' = 0.248 \text{ cms.}$$

Take $t' = 0.3 \text{ cms.}$



- (b) We Use 2 Diagonal Stiffeners (D) Instead of the 2 Plates:

$$F_D = \frac{13.42}{\cos \gamma} = 26.84 \text{ tons}$$

$$b_{st} \cdot t_{st} = \frac{26.84}{1.4 \times 2} = 9.58 \text{ cms.}^2$$

$$b_{st} = \frac{30 - 1.4}{2} = 14.3 \text{ cms}$$

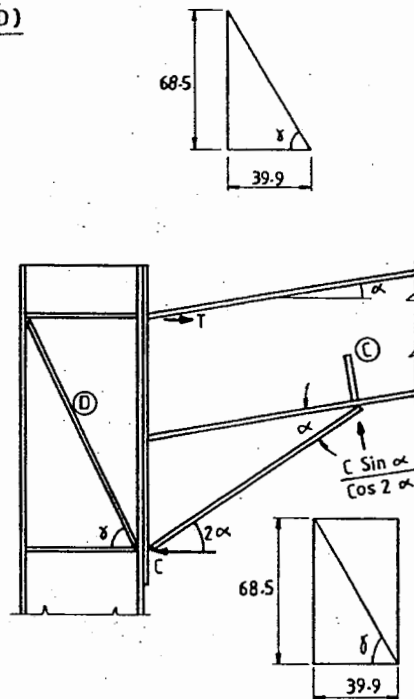
Take $b_{st} = 14.0 \text{ cms}$

(1) $t_{st} = 0.7 \text{ cms.}$

(2) $t_{st} = \frac{b}{16.1} = 0.869 \text{ cms.}$

From (1) & (2) $t_{st} = 0.9 \text{ cms.}$

∴ Use 2 diagonal stiffeners (14 x 0.9)



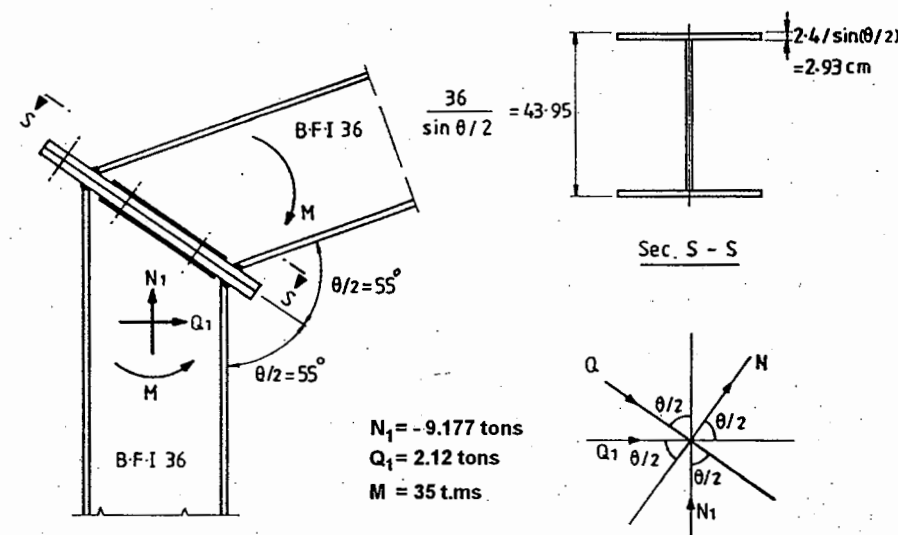
- (8) Stiffener (c):

$$F_c = \frac{63.4 \times 0.196}{0.92} = 13.48 \text{ tons choose } 14.0 \times 0.9 \text{ cms}$$

Example (5.19):

Design the extended end plate moment connection shown in the figure below.

Solution



Straining Actions:

$$N = Q_1 \cos \theta / 2 + N_1 \sin \theta / 2 = 9.177 \text{ tons.}$$

$$Q = Q_1 \sin \theta / 2 - N_1 \cos \theta / 2 = 2.121 \text{ tons.}$$

$$M = 35.0 \text{ mt.}$$

(1) Design of Welds:

- Use butt welds of the excellent type to connect flanges to the end plate.
 - Use fillet welds to connect web to the end plate.
- $$h = 43.95 - 4(2.93) = 32.23 \text{ cms}$$

$$s = \frac{2.121}{2 \times 32.23 \times 0.72} = 0.045 \text{ cms.}$$

$$\text{take } s = s_{\min} = 4 \text{ mms.}$$

(2) Design of H.S.B:

(2.a) Approximate Analysis:

$$\text{Tension force} = \frac{M}{d} = \frac{3500}{43.95 - 2.93} = 85.324 \text{ tons.}$$

Assume we have 2 rows of bolts each 4 bolts.

$$\therefore \text{Tension / bolt} = \frac{85.33}{8} = 10.67 \text{ tons.}$$

$$\text{Use H.S.B M(22) (10.9)} \quad T_{\text{all}} = 0.8 \times 19.08 = 15.26 \text{ tons.}$$

(2.b) Design of the End Plate:

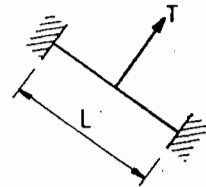
$$L = 3 d + \frac{t_f}{\sin \theta / 2} = 3 \times 2.2 + 2.93$$

$$L = 6.6 + 2.93 = 9.53 \text{ cms.}$$

$$\text{Moment} = \frac{TL}{8} = \frac{85.324 \times 9.53}{8} = 101.642 \text{ ton.cm.}$$

$$t_p = \sqrt{\frac{6M}{b \times 1.4}} = \sqrt{\frac{6 \times 101.642}{30 \times 1.4}} = 3.811 \text{ cms.}$$

$$\text{let } t_p = 4.0 \text{ cms.}$$



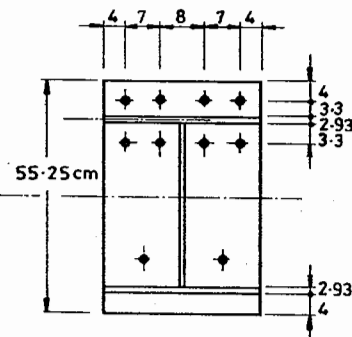
(2.c) Exact Analysis for H.S.B:

$$I_x = \frac{1}{12} \times 55.25^3 \times 30 f_{t1}$$

$$I_x = 421635.1 \text{ cm}^4.$$

$$f_{t1} = \frac{3500}{421635.1} \times 27.625 = 0.23 \text{ t/cm}^2$$

$$f_{t2} = \frac{3500}{421635.1} \times 18.86 = 0.157 \text{ t/cm}^2$$



Neglect the effect of N (compression).

Group (1):

$$T_{\text{ext,b,M}} = \left(\frac{0.23 + 0.157}{2} \right)$$

$$\times \frac{30 \times 8.765}{4} = 12.72 \text{ t} < 0.8 \times 19.08 \text{ tons. O.K.}$$

Group (2):

$$T_{\text{ext,b,M}} = \frac{1}{2} \times 0.157 \times 30 \times \frac{18.86}{4} = 11.1 \text{ tons.}$$

$$< 0.8 \times 19.08 \text{ tons. O.K.}$$

(3) Check of Shear on Bolts:

$$Q / \text{bolt} = \frac{2.121}{2} = 1.06 \text{ tons} < P_s$$

$$< 4.93 \text{ tons (Safe).}$$

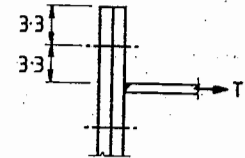
(4) Check Effect of Prying Force:

$$W / \text{bolt} = 7.5 \text{ cms.}$$

$$T / \text{bolt} = 12.72 \text{ tons.}$$

$$a = b = 3.3 \text{ cms.}$$

$$A_s = 3.03, \quad t_p = 4.0 \text{ cms.}$$



$$P = \left[\frac{\frac{1}{2} - \frac{w t^4}{30 a b^2 A_s}}{\frac{3a}{4b} \left(\frac{a}{4b} + 1 \right) + \frac{w t^4}{30 a b^2 A_s}} \right] \cdot T_{\text{ext,b,M}}$$

$$Q = -ve \quad - \text{no prying effect.}$$

**Example (5.20):**

Repeat the previous example using a flushed end plat.

Solution**(1) Design of Welds:**

as before.

(2) Design of the H.S.B:**(2-a) Approximate Analysis:**

$$\text{Tension force} = \frac{M}{d'}; \quad d' = 0.85 d$$

$$d = 43.95 - 2.93 = 41.02 \text{ cms}$$

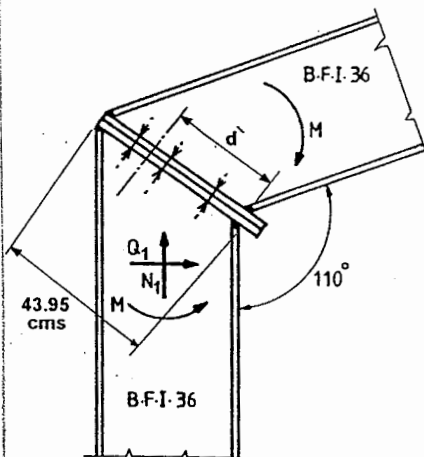
$$d' = 0.85 \times 41.02 \cong 35 \text{ cms}$$

$$\text{Tension force} = \frac{3500}{35} \cong 100 \text{ tons.}$$

Assume we have 2 rows of bolts each 4 bolts.

$$\text{Tension / bolt} = \frac{100}{8} = 12.5 \text{ tons.}$$

$$T_i = \frac{12.5}{0.8} \cong 15.6 \text{ tons.}$$

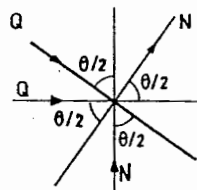
**Straining Action:**

$$M = 35.0 \text{ m t}$$

$$N_1 = -9.177 \text{ tons (comp.)}$$

$$Q_1 = 2.121 \text{ tons}$$

(Computed as Ex. 5.19)



Use H.S.B. M(22) (10.9)

$$T_{all} = 0.8 \times 19.08 = 15.26 \text{ tons.}$$

$$P_s = 6.91 \text{ tons.}$$

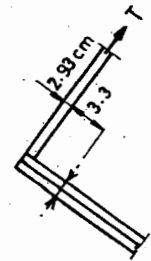
(2.b) Design of the end Plate:

$$T = 3500 / 41.02 = 85.324 \text{ tons}$$

$$\begin{aligned} \text{Moment} &= 85.324 \left(\frac{2.93}{2} + 3.3 \right) \\ &= 406.57 \text{ t.cm} \end{aligned}$$

$$t_{ep} = \sqrt{\frac{6 \times 406.57}{30 \times 1.4}} = 7.62 \text{ cms.}$$

$$\text{Take } t_{ep} = 7.7 \text{ cm}$$

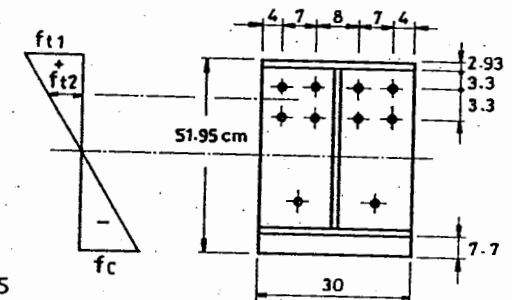
**(2.c) Exact Analysis:**

$$I_{xx} = \frac{1}{12} (30) (51.95)^3$$

$$I_{xx} = 350506.97 \text{ cm}^4$$

$$\begin{aligned} f_{t1} &= \frac{3500}{350506.97} \times 25.975 \\ &= 0.2594 \text{ t/cm}^2. \end{aligned}$$

$$\begin{aligned} f_{t2} &= \frac{3500}{350506.97} \times 18.095 \\ &= 0.18 \text{ t/cm}^2. \end{aligned}$$

**Group (1):**

$$\begin{aligned} T_{ext, b, M} &= \frac{1}{2} (0.2594 + 0.18) 30 \times \frac{7.88}{4} \\ &= 12.98 \text{ tons} < 15.26 \text{ tons (Safe)} \end{aligned}$$

Group (2):

$$T_{\text{ext, b, M}} = \frac{1}{2} 0.1656 \times 30 \times \frac{18.095}{4}$$

= 11.23 tons

< 15.26 tons Safe

(3) Check of Shear on Bolts:

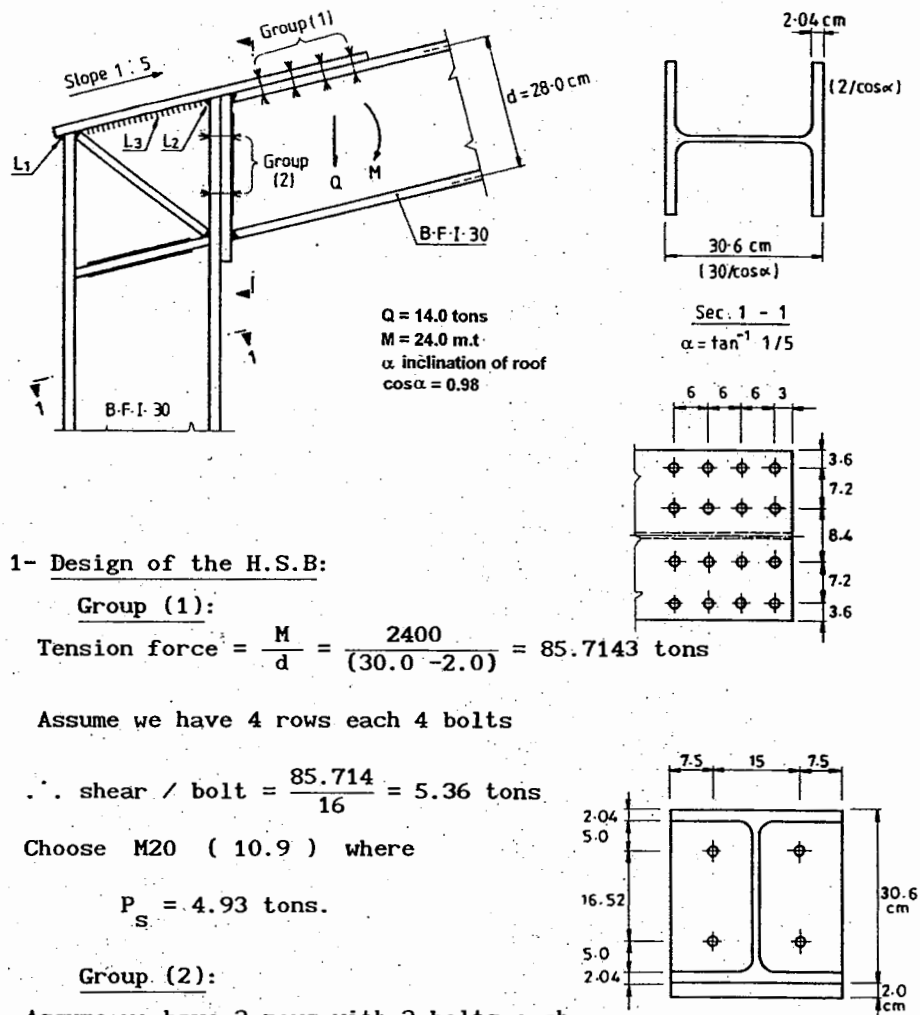
$$Q / \text{bolt} = \frac{2.121}{2} = 1.06 \text{ tons} < 6.1 \text{ tons (Safe).}$$

Comparing the extended end plate configuration Example 5.19 W.r.T. the flushed end plate, we can observe that the thickness of the end plate is becoming 7.7 cms instead of 5.0 cms, thus there are no benefit to use the flushed end plate.

Example (5.21):

Design the rigid connection shown in the Fig. below.

Solution



1- Design of the H.S.B:

Group (1):

$$\text{Tension force} = \frac{M}{d} = \frac{2400}{(30.0 - 2.0)} = 85.7143 \text{ tons}$$

Assume we have 4 rows each 4 bolts

$$\therefore \text{shear / bolt} = \frac{85.714}{16} = 5.36 \text{ tons}$$

Choose M20 (10.9) where

$P_s = 4.93$ tons.

Group (2):

Assume we have 2 rows with 2 bolts each.

$$\therefore \text{shear / bolt} = \frac{14}{4} = 3.50 \text{ tons}$$

Choose M20 (10.9) where $P_s = 4.93$ tons

2- Design of Welds:

- Use k welds to connect the beam flanges with the end plate.
- Use fillet welds around the beam web to carry the shear force.

$$L_w \max = \frac{1}{\cos \alpha} [30 - 4(2)] = 22.40 \text{ cms let it} = 22.0 \text{ cms.}$$

$$s = \frac{14}{2 \times 22 \times 0.72} = 0.43 \text{ cm Use } s = 0.5 \text{ cm.}$$

3- Design of the Splice Plate:

For H.S.B, the gross area of splice plate is utilized.

$$A_{\text{gross}} = 30 \times t ; \quad t = \text{thickness of plate}$$

$$\therefore 30 \times t \times 1.4 = 85.714$$

$$\text{Get } t = 2.04 \text{ cms let it} = 2.2 \text{ cms.}$$

4- Design of Welds Connecting Splice Plate with Column:

$$L_1 = 30 \text{ cms}$$

$$L_2 = \frac{30 - 1.2 - 2(2.04)}{2}$$

$$= 12.36 \text{ cms let it} = 12.0 \text{ cms}$$

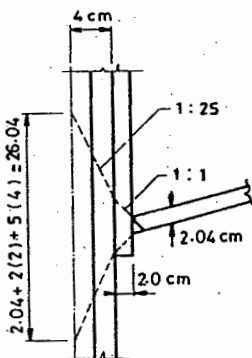
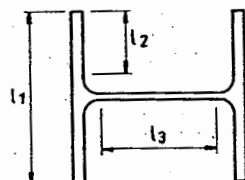
$$L_3 = 30.6 - 4(2.04)$$

$$= 22.44 \text{ cms let it} = 22.0 \text{ cms}$$

$$\therefore L_w \max = 2 \times 22 + 30 + 4(12) = 122 \text{ cms}$$

$$\text{Get } s_w = \frac{85.714}{122 \times 0.72} = 0.97 \text{ cm}$$

5- Crippling, shear resistance of corner web, as previous examples.



Example (5.22):

Design the rigid corner frame connection shown in the figure below.

Solution

1- Design of the Top Tie Plate: (C)

For H.S.B the gross area of plate is utilized.

$$\text{Tension force} = \frac{2400}{30 - 2} = 85.714 \text{ tons.}$$

$$t_p = \frac{85.714}{30 \times 1.40} = 2.04 \text{ cms}$$

$$\text{let it} = 2.2 \text{ cms.}$$

Take the same thickness for plates a, b and c.

2- Design of H.S.B:

Group (1):

Using 4 rows of bolts each 4 bolts.

$$Q / \text{bolt} = \frac{85.714}{16} = 5.36 \text{ tons.}$$

$$\text{Choose M22 (10.9) } P_s = 6.1 \text{ tons} > 5.36 \text{ O.K.}$$

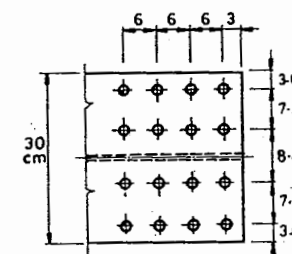
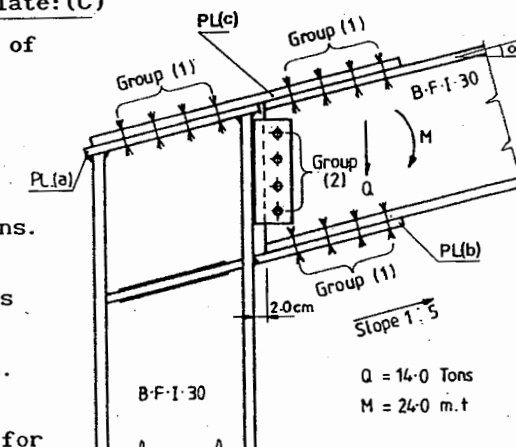
Group (2):

Assume that the shearing force Q (14 tons) acts at center line of the H.S.B. group (2).

Using 4 bolts:

$$Q / \text{bolt} = \frac{14.0}{4} = 3.5 \text{ tons.}$$

$$\text{Choose M20 (10.9) } P_s = 4.93 \text{ tons} > 3.5 \text{ t Safe.}$$





3- Design of the Cleat Web Plate:

a- Straining Action:

$$Q = 14.0 \text{ tons}$$

$$M = 14(2 + 2.4) = 61.6 \text{ t.cm}$$

$$\text{Max. height of plate} = \frac{30 - 2(4)}{\cos \alpha}$$

$$(\alpha = \tan^{-1} 1/5) = 22.44^\circ$$

b- Check of Shear:

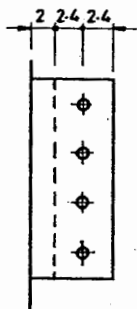
$$q_{\text{all}} = 0.84 \text{ t/cm}^2$$

$$\therefore 0.84 = \frac{3}{2} \times \frac{Q}{\text{Area}}$$

$$0.84 = \frac{3}{2} \times \frac{14}{22 \text{ t}}$$

$$\text{Get } t_p = 1.136 \text{ cm}$$

$$\text{let } t_p = 1.2 \text{ cm}$$



(a)

c- Check of Bending:

$$F_b = 1.4 \text{ t/cm}^2$$

$$1.4 = \frac{M}{Z_x}, \quad Z_x = \frac{1}{12} t_p \frac{(22)^3}{11} = 80.67 t_p \text{ cm}^3$$

$$1.4 = \frac{61.6}{80.67 t_p} \quad \text{Get } t_p = 0.55 \text{ cm}$$

$$\text{let } t_p = 0.6 \text{ cm} \quad \text{(b)}$$

From (a) & (b) take $t_p = 1.2 \text{ cm}$

4- Design of Fillet Welds:

a- Straining Action:

$$Q = 14.0 \text{ tons} \quad M = 61.6 \text{ t.cm}$$



b- Properties of Area:

$$A_{\text{welds}} = 2 \times 22 \times s; \quad Z_x(\text{welds}) = \frac{s \times (22)^2}{3}$$

$$A_w = 44 s \text{ cm}^2, \quad Z_x = 161.33 s \text{ cm}^3$$

c- Check of Stresses:

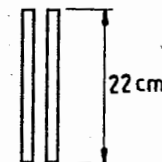
Sec (1-1)

$$q_{//} = \frac{14}{44 s} = \frac{0.318}{s} \text{ t/cm}^2$$

$$q_{\perp} = \frac{61.6}{161.33 s} = \frac{0.382}{s} \text{ t/cm}^2$$

$$q_{\text{total}} = 0.72 = \frac{1}{s} \sqrt{(0.382)^2 + (0.318)^2}$$

$$\text{Get } s = 0.69 \text{ cm} \quad \text{(a)}$$



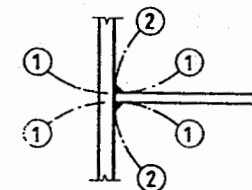
Sec (2-2)

$$q_{//} = \frac{0.318}{s} \text{ t/cm}^2$$

$$f_{\perp} = \frac{0.382}{s} \text{ t/cm}^2$$

$$f_e = \frac{1}{s} \sqrt{3(0.318)^2 + (0.382)^2}$$

$$= 0.79 \text{ t/cm}^2 \quad \text{Get } s = 0.848 \text{ cm} \quad \text{(b)}$$



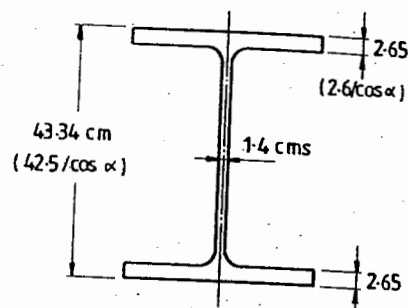
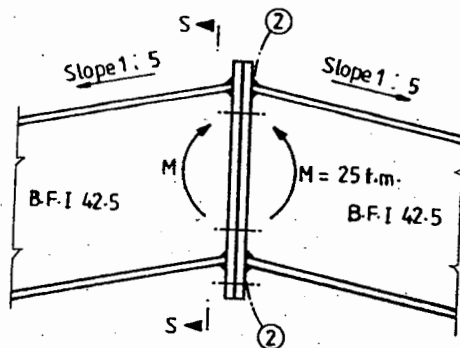
Hence: take $s = 0.9 \text{ cm}$

5- Check of crippling and shear resistance of corner web as previous Examples.

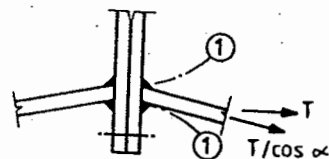
Example (5.23):

Design the rigid apex frame connection shown in Fig.

Solution



Séc. S - S



(1) Design of the Welds:

let s around web = 14 mm

Sec (1-1)

Tension force (Horizontal)

$$= \frac{2500}{43.34 - 2.65}$$

= 61.44 tons.

$$\begin{aligned} \text{Length of welds around flange} &= 30 + 2 \left(\frac{30 - 1.4 - 2 \times 2.65}{2} \right) \\ &= 30 + 2 \times 11.65 = 53.3 \text{ cms.} \end{aligned}$$

$$s_{\text{required}} = \frac{61.44 / \cos \alpha}{53.3 \times 0.72} = 1.63 \text{ cm}$$

∴ Take $s = 1.8 \text{ cm}$

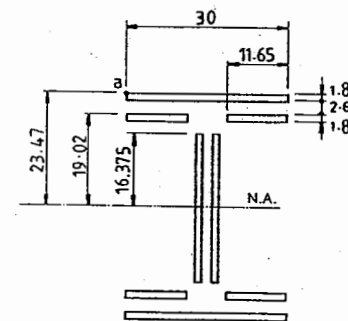
Sec (2-2)

s around web = 1.4 cm

s around flanges = 1.8 cm

$$\begin{aligned} I_{xx} &= \frac{1}{12} (32.74)^3 \times 1.4 \times 2 \\ &\quad + 30 \times 1.8 \times 2 \times (22.57)^2 \\ &\quad + 11.65 \times 1.8 \times 4 \times (18.12)^2 \\ I_{xx} &= 90745.1 \text{ cm}^4 \end{aligned}$$

$$f_{tA} = \frac{2500}{I_{xx}} \times (23.47) = 0.647 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \quad (\text{Safe})$$



(2) Design of H.S.B.:

(2.a) Approximate Analysis:

Assume we have 2 rows of 4 bolts each.

Tension force (Horizontal) = 61.44 t.

$$\text{Force / bolt} = \frac{61.44}{8} = 7.68$$

Try H.S.B. M_{16} (10.9) $T_{i_{req}} = 9.9$ tons.

$$T_{all} = 0.8 \times 9.89 = 7.91 \text{ tons.} \quad \text{O.K.}$$

(2.b) Design of the End Plate:

Tension (H.z.) = 61.44 tons.

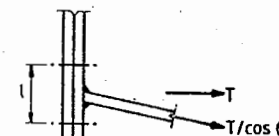
$$\text{Moment} = \frac{TL}{8}$$

$$L = 3d + 2s + \frac{t_f}{\cos \alpha}$$

$$= 3 \times 1.6 + 2 \times 1.8 + 2.65$$

$$= 11.05 \text{ cms.}$$

$$M = \frac{61.44 \times 11.05}{8} = 84.864 \text{ ton cm}$$



$$t_p = \sqrt{\frac{6M}{b \times 1.4}} = \sqrt{\frac{6 \times 84.864}{30 \times 1.4}}$$

$$= 3.482 \text{ cms. Take } t_p = 3.5 \text{ cms}$$

(2.c) Exact Analysis for H.S.B.:

$$I_{xx} = \frac{1}{12} (53.44)^3 \times 30$$

$$= 381539.37 \text{ cm}^4$$

$$f_{t1} = \frac{2500}{381539.37} \times 26.72$$

$$= 0.1751 \text{ t/cm}^2$$

$$f_{t2} = \frac{2500}{381539.37} \times 18.795 = 0.1232 \text{ t/cm}^2$$

Group (1):

$$T_{\text{ext},b,M} = \left(\frac{0.1751 + 0.1232}{2} \right) \times 30 \times \frac{7.925}{4} = 8.865 \text{ t} > 7.91$$

Unsafe

Group (2):

$$T_{\text{ext},b,M} = \frac{0.1232 \times 18.795}{2} \times \frac{30}{4} = 8.68 > 7.91 \text{ Unsafe.}$$

Use M20 / 10.9

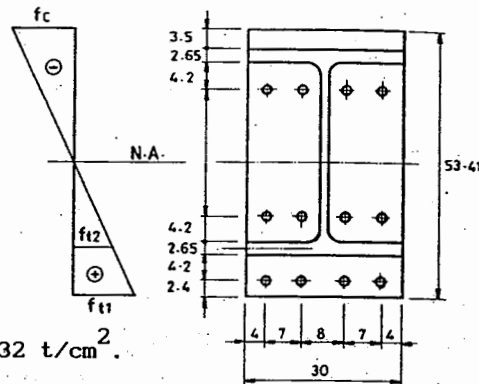
(2.b) Design of the End Plate:

$$L = 3d + 2s + t_f / \cos\theta = 3 \times 2 + 2 \times 1.8 + 2.65 = 12.25 \text{ cms}$$

$$M = \frac{61.44 \times 12.25}{8} = 94.08 \text{ t.cm}$$

$$t = \sqrt{\frac{6 \times 94.08}{8}} = 3.67 \text{ cm}$$

Take $t = 3.7 \text{ cm}$



(2.c) Exact Analysis for

H.S.B.:

$$I_{xx} = ((54.84)^3 \times 30) / 12$$

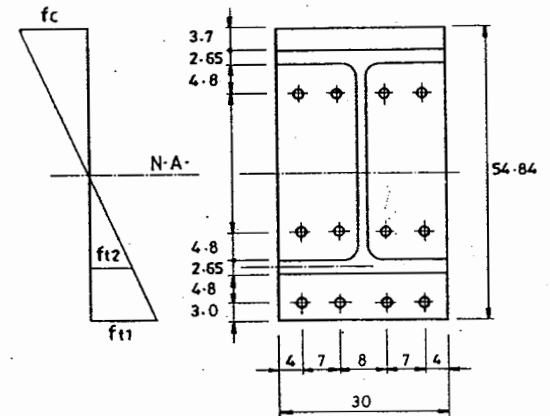
$$I_{xx} = 412318.05 \text{ cm}^4$$

$$f_{t1} = \frac{2500}{412318.05} \times 27.42$$

$$= 0.1663 \text{ t/cm}^2$$

$$f_{t2} = \frac{2500}{412318.05} \times 18.295$$

$$= 0.11093 \text{ t/cm}^2$$



Group (1):

$$T_{\text{ext},b,M} = \left(\frac{0.1663 + 0.11093}{2} \right) \times 30 \times \frac{9.125}{4} = 9.4865 \text{ t}$$

$$< 12.34 \text{ tons}$$

Group (2):

$$T_{\text{ext},b,M} = \frac{0.11093}{2} \times \frac{18.295}{4} \times 30 = 7.6105 \text{ t} < 12.34 \text{ t}$$

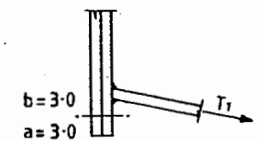
(3) Check the Effect of Prying Force:

$$a = 3.0 \text{ cm} \quad b = 3.0 \text{ cm} \quad P/\text{bolt} = 9.49 \text{ tons}$$

$$A_s = 2.45 \text{ cm}^2 \quad W/\text{bolt} = 7.5 \text{ cm} \quad t = 3.7 \text{ cm}$$

$$P = \left[\frac{\frac{1}{2} - \frac{w t^4}{30 a b^2 A_s}}{\frac{3a}{4b} \left(\frac{a}{4b} + 1 \right) + \frac{w t^4}{30 a b^2 A_s}} \right] \cdot T_{\text{ext},b,M}$$

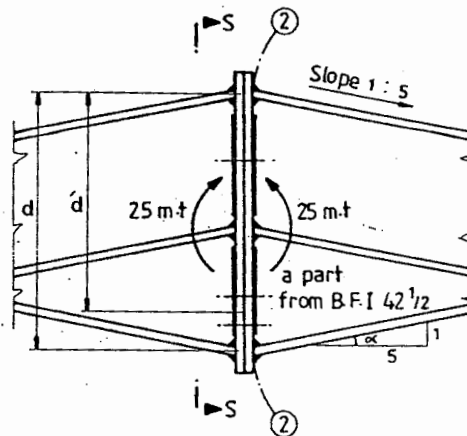
$P = -ve$ no prying force.



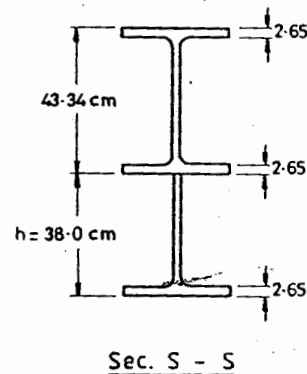
Example (5.24):

Design a rigid apex frame connection using haunches. The connection is subjected to Moment = 25.0 m.t.

Solution



$$\begin{aligned} d &= 0.85 d \\ d &= 43.34 + 38 - 2.65 = 78.9 \end{aligned}$$



The Height of Haunch:

$$h = \frac{42.5 - 2 \times 2.6}{\cos \alpha} = \frac{37.3}{\cos \alpha} = 38.04 \text{ cm}$$

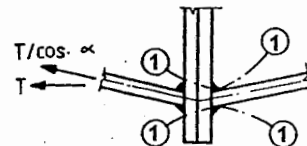
Take $h = 38.0 \text{ cm}$

(1) Design of Welds:

Sec (1-1)

Length of welds around the tension

$$\begin{aligned} \text{Flange} &= 30 + 2 \left[\frac{30 - 1.4 - 2 \times 2.65}{2} \right] \\ &= 53.3 \text{ cm} \end{aligned}$$



$$\text{Tension force (horizontal)} = \frac{2500}{(43.34 + 38 - 2.65)} = 31.8 \text{ tons}$$

$$s = \frac{31.8 / \cos \alpha}{0.72 \times 53.3} = 0.84 \text{ cm} \quad \text{let } s = 1.0 \text{ cm}$$

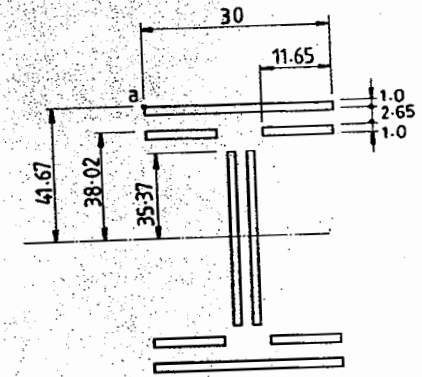
Sec (2-2)

s around the web = 6 mm

s around the flanges = 10 mm

$$\begin{aligned} I_{xx} &= \frac{1}{12} (70.74)^3 \times 2 \times 0.6 \\ &\quad + 30 \times 1.0 \times (41.17)^2 \\ &\quad \times 2 + 11.65 \times 1.0 \times \\ &\quad (37.52)^2 \times 4 \\ &= 202698.64 \text{ cm}^4 \end{aligned}$$

$$\begin{aligned} f_{tA} &= \frac{M}{I_{xx}} y = \frac{2500}{202698.64} \\ &\quad \times 41.67 = 0.514 \text{ t/cm}^2 \\ &< 0.72 \end{aligned}$$



(2) Design of the H.S.B.:

(2.a) Approximate Analysis:

$$T = \frac{2500}{0.85 \times 78.69} = 37.37 \text{ tons}$$

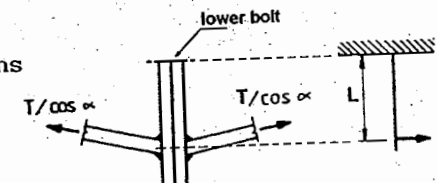
$$T / \text{bolt} = \frac{37.37}{4} = 9.34 \text{ tons}$$

Use H.S.B. M20 (10.9) where :

$$T_{\text{req}} = 9.34 / 0.8 = 11.68 \text{ tons}$$

(2.b) Design of End Plate:

$$L = 1.5 d + s + \frac{t}{2 \cos \alpha}$$





$$L = 1.5 \times 2 + 1.0 + \frac{2.6}{2 \cos 11.31^\circ}$$

$$= 5.326 \text{ cm}$$

$$M = T.L = 31.8 \times 5.326 = 169.37 \text{ ton cm}$$

$$t = \sqrt{\frac{6M}{b \times 1.4}} = \sqrt{\frac{6 \times 169.37}{30 \times 1.4}} = 4.92 \text{ cms Take } t = 5.0 \text{ cms}$$

(2.c) Exact Analysis:

$$I_{xx} = \frac{1}{12} (88.34)^3 \times (30)$$

$$= 1723503.6 \text{ cm}^4$$

$$f_{t1} = \frac{2500}{1723503.6} \times 44.17$$

$$= 0.06407 \text{ t/cm}^2$$

$$f_{t2} = \frac{2500}{1723503.6} \times 30.52$$

$$= 0.0443 \text{ t/cm}^2$$

Group (1):

$$T_{\text{ext}, b, M} = \frac{1}{2} (0.06407 + 0.0443) \times \frac{13.65 \times 30}{2}$$

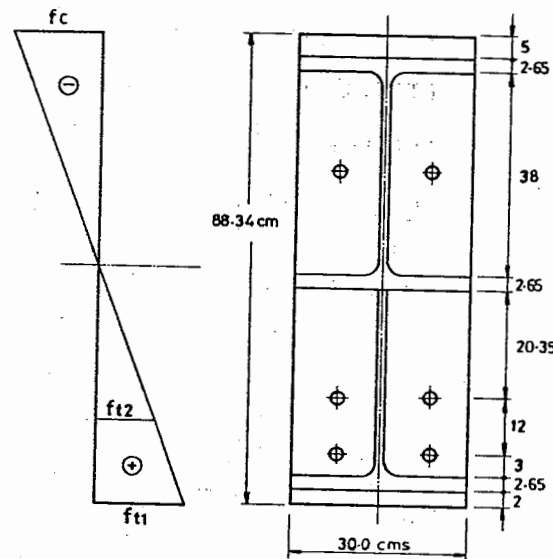
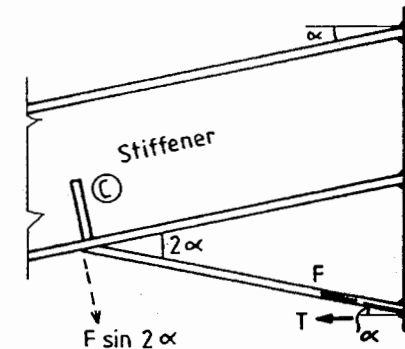
$$= 11.1 \text{ tons}$$

$$< 15.43 \times 0.8 = 12.34 \text{ tons O.K.}$$

Group (2):

$$T_{\text{ext}, b, M} = \frac{1}{2} \times 0.0443 \times 30 \times \frac{30.52}{2} = 10.1403 \text{ tons} < 12.34 \text{ tons}$$

Safe

(3) The Use of TransverseStiffeners:

Horiz. Tension applied force = 37.37 tons.

$$F = \frac{37.37}{\cos \alpha} = 38.13 \text{ tons.}$$

Force acting normal to the beam = $F_c = F \sin 2 \alpha$

= Force in Stiffener (c)

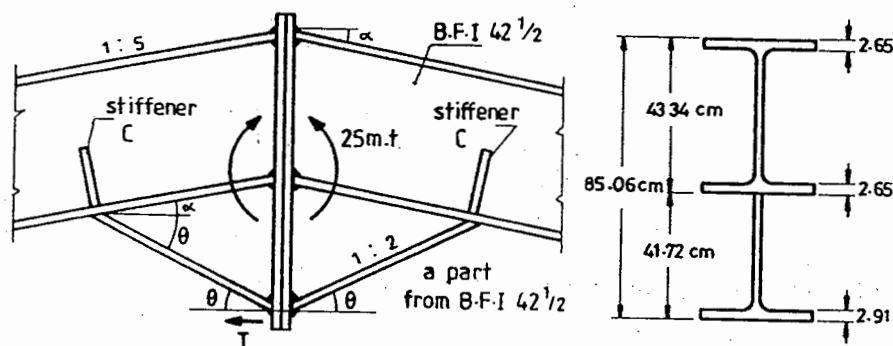
$$F_c \cong 16.45 \text{ tons}$$

Hence choose 2 stiffeners (c) 14.0 x 0.9 cms to overcome totally the induced force ($F = 16.45 \text{ tons}$).

Example (5.25):

Design the connection of Example 5.31.a using a haunch of slope 1:2.

Solution



(1) Design of Welds:

Take the same size of welds as Example 5.31.a.

Size of welds around the flanges = 10 mms.

Size of welds around the web = 6 mms.

(2) Design of H.S.B.:

(2.1) Approximate Analysis:

$$\text{Horiz. Tension force} = T = \frac{25 \times 100}{\left(85.06 - \frac{2.65}{2} - \frac{2.91}{2}\right)} = 30.38 \text{ tons.}$$

$$\cos \theta = 0.89$$

$$\cos \alpha = 0.98$$

No need to continue the design, the number of bolts and their configuration is exactly the same as example 5.31. Only the end plate will have the following dimensions:

$$h = 92.06 \text{ cms, } b = 30 \text{ cms, } t = 5.0 \text{ cms.}$$

(3) The Transverse Stiffener "c":

At the abrupt change of the beam cross section the lower flange of the beam will be subjected to the force:

$$F = \frac{30.38}{\cos \theta} = 33.98 \approx 34 \text{ tons.}$$

The component acting normal to the beam flange is:

$$\text{Applied force} = F \sin (\alpha + \theta) = 34 \times 0.77 = 26.25 \text{ tons.}$$

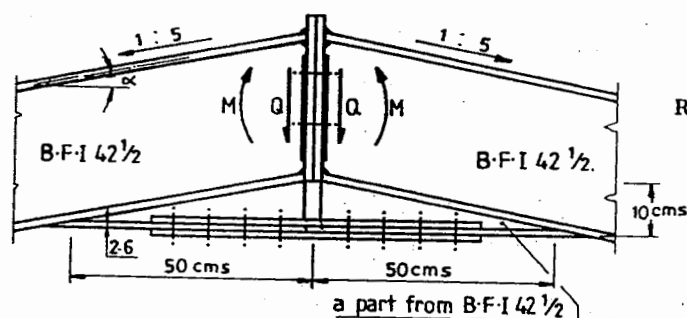
$$\text{Hence } F_{(c)} = 26.25 \text{ tons.}$$

Use 2 stiffeners 14.0 x 0.9 cms , to overcome the total induced force $F_{(c)} = 26.25$ tons.

Comparing examples 5.31 and 5.32, it is evident that for apex connections it is sufficient to use a haunch with a slope 1:2.

**Example (5.26):**

Design the rigid apex frame connection shown in the figure below using haunches, and splice plates.

Solution

$$M = 25.0 \text{ t.m.}$$

$$R_b = 14.0 \text{ t.}$$

(1) Design of Welds:**a- Fillet Welds Around Web:**

It is designed to transmit the shearing force to the end plate.

$$l_{\text{weld max}} = \frac{42.5 - 4 \times 2.6}{\cos \alpha} ; \alpha = \tan^{-1} \frac{1}{5}$$

$$= 32.74 \text{ cms}$$

$$s = \frac{14}{2 \times 32.74 \times 0.72} = 0.296 \text{ cm let it} = 4 \text{ mm.}$$

b- Fillet Welds Around Comp Flanges:

The comp force is transmitted from the flange to end plate by contact. For conservative solution let this transmission be through the welds.

$$l_{w \text{ max}} = 30 + 2 \left(\frac{30 - 1.40 - 2 \times 2.65}{2} \right) (2.6 / \cos \alpha)$$

$$= 53.3 \text{ cms}$$



$$C \cong T = \frac{2500}{42.5 / \cos \alpha + 10 - \frac{(2.65+2.6)}{2}} = 50.74 \text{ tons}$$

$$s = \frac{50.74 / \cos \alpha}{53.3 \times 0.72} = 1.34 \text{ cm let it} = 1.40 \text{ cm}$$

(2) Design of Splice Plates:

For H.S.B. the gross area of the splice plate is utilized.

$$\text{Total width of the 3 plates} = 30 + 2 \left(\frac{30 - 1.4 - 2 \times 2.6}{2} \right)$$

$$= 30 + 2 \times 11.7 = 53.40 \text{ cms}$$

$$t_p = \frac{50.74}{53.4 \times 1.4} = 0.68 \text{ cm Where Tension} = 50.74 \text{ tons.}$$

$$\text{let } t_p = 0.8 \text{ cm.}$$

(3) Design of H.S.B.:

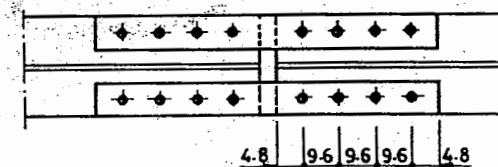
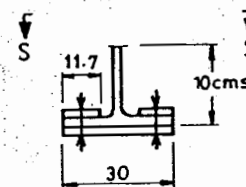
Using 4 rows each 2 bolts M20

$$Q / \text{bolt} = \frac{50.74}{8} = 6.34 \text{ tons}$$

Use M16 (10.9) Where $P_s = (\text{double shear}) = 4.93 \times 2$

$$= 9.86 \text{ tons.}$$

$$Q / \text{bolt} < P_s \text{ (Safe).}$$



Sec. S - S

CHAPTER 6



CHAPTER 6

SEMI-RIGID CONNECTIONS

6.1. Introduction:

In chapters 4 and 5 the ideally flexible and rigid connections have been analyzed. The fastening elements were either subjected to concentric and/or eccentric shear and/or flexural moment. The behavior was ideal and thus the interaction of the structural members and the stability phenomenon correspond to the behavior previously outlined in Volume (1).

A realistic structural analysis must include the performance of flexural connections. The real behavior of a connection will probably influence the straining actions and the stability of the different structural elements comprising the structure under consideration.

In fact, the conventional analysis of steel frames, considers that the beam-to-column as well as the base connections are either perfectly hinged (ideal flexible connection) or perfectly rigid. Although these assumptions are not consistent with the actual connection behavior, but their use will probably simplify the analysis.

Extensive experimental and theoretical research have been carried out on beam-to-column and base connections. Test results have shown that even the most flexible connections possess some ability to resist some rotational movement, while rigid connections have slight flexibility. This leads to the recognition of a class of connections termed "Semi-Rigid Connections".

6.2. The Moment-Rotation Curve:

The maximum strength, the stiffness and the deformation capacity of a connection can be obtained if its moment-rotation curve is available. The term rotation is used to designate the relative rotation (θ) between the elastic lines of the connected structural elements as illustrated in figure 6.1.a.

Fig. 6.1.b shows the classification of the connections regarding the moment-rotation curve. The classification is comprising the following three types:

Type (1) Corresponds to:

The ideally rigid connection where the relative rotation θ is zero ($\theta = 0$), while the induced end moment corresponds to the fixing end moment (M_f) of an ideally fixed structural beam element.

Type (2) Corresponds to:

The ideally flexible connection where the end moment is zero ($M_f = 0$) while the induced relative rotation θ corresponds to the beam characteristics (i.e. span, loading, inertia, ...etc).

Type (3):

In practice, nevertheless the two previous ideal types exist. A rigid connection is always exerting varying degrees of partial restraint while a flexible connection is to some degree a moment resistant connection. These intermediate situations lead to what we call "Semi-Rigid-Connections".

However, a typical rigid connection (Type 1) would carry an end moment (M_1) about 80 percent or more of the conventional fixing end moment (M_f). On the other hand, a typical flexible connection may have to resist about 20 percent or less of the

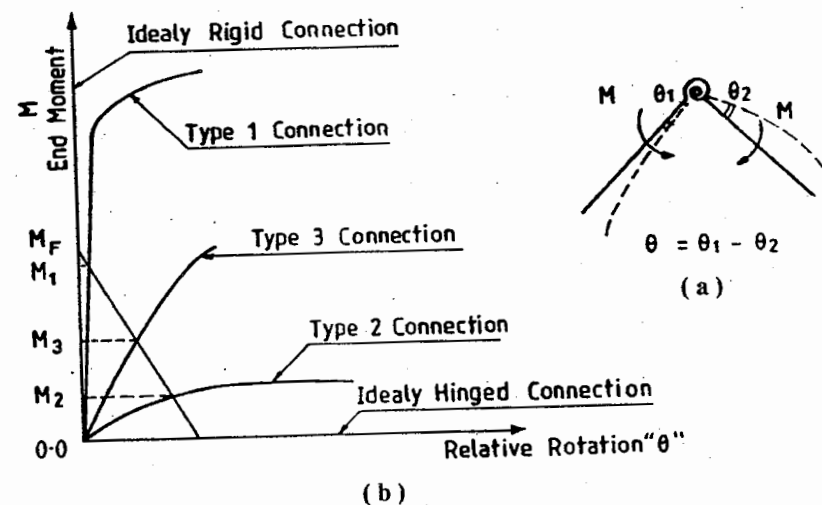


Fig. 6.1. Moment-Rotation Relationship

fixing moment M_f as indicated by the moment (M_2). The semi-rigid connection is an intermediate situation where the induced end moment M_3 is depending on the degree of restraint (i.e. degree of rigidity) of the connection.

6.3. Moment-Rotation Curves of Typical connections:

For the design of a semi-rigid connection the moment rotation curves are needed. Figs. 6.2 and 6.3 show a typical range of moment-rotation curves for a variety of welded and bolted connections respectively.

The ($M-\theta$) curves are generally non-linear over most of the range. However, stiffer connections have often an initial phase which can be approximated to a straight line.

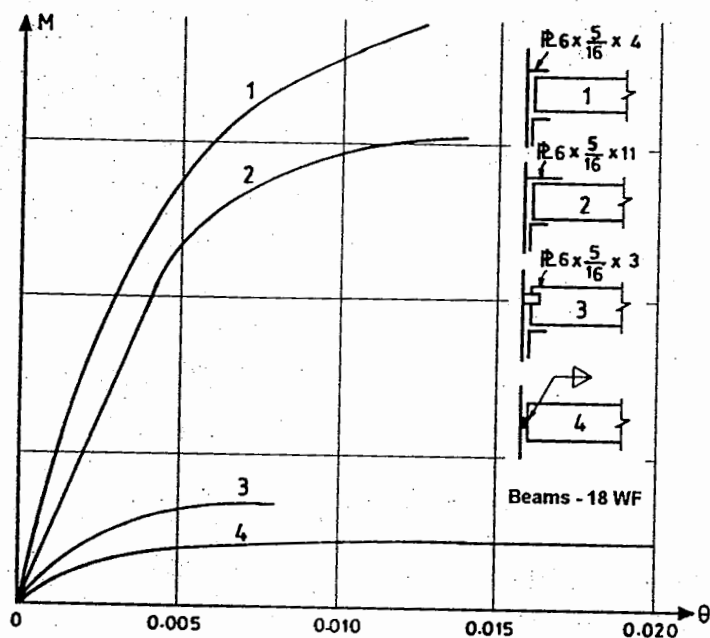


Fig. 6.2. (M-θ) Curves for Typical Welded Connections

Stiff connections such as Tee-stub, extended end-plate and fixed base connections are those where the moment is resisted by components carrying axial loads. The corresponding axial deformations under service loads are generally small. On the other hand, flexible connections such as flanges cleats, web cleats, flexible end-plates and hinged base connections resist forces by components subjected to bending and thus they possess higher rotational capacity.

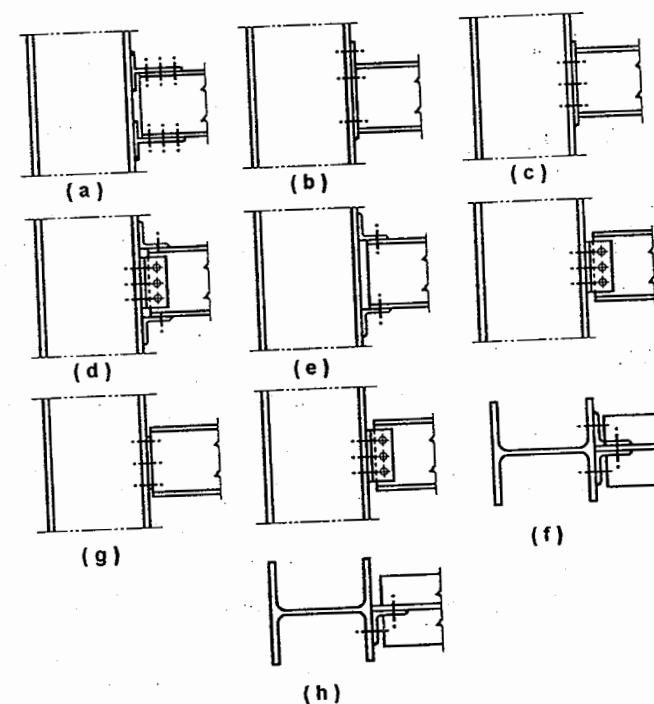
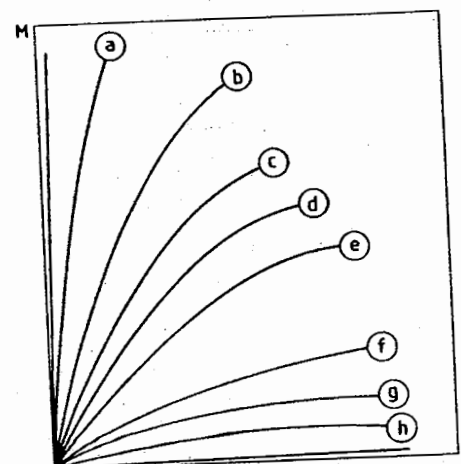


Fig. 6.3. (M-θ) Curves for Typical Bolted Connections

6.4. Methods of Prediction of (M-θ) Curve:

6.4.1. Prediction Using Mathematical Expressions:

Different attempts have been made to describe the (M-θ) curves. the experimental results have been utilized to compute mathematical expressions capable to recognize the general features of the (M-θ) curve. The different forms of the (M-θ) curve can be classified as follows:

(a) The Linear Representation:

Packer (16) represented the (M-θ) curve by the initial tangent slope which leads to the notation of the semi-rigid connection by the connection factor (K_s), defined as the angle change per unit moment:

$$\left. \begin{aligned} M &= K_s \cdot \theta \\ \text{i.e. } K_s &= \frac{M}{\theta} \end{aligned} \right\} \quad 6.1$$

This model overestimates the connection stiffness at finite rotations.

(b) The Polynomial Function:

Frye and Morris (17) proposed a polynomial function to describe the (M-θ) curve of several types of connections. The method of least squares has been utilized to compute the polynomial constants which were written in terms of dimensionless factors. These constants depend on the size parameters of the connection.

The main disadvantage of this model is that the first derivative indicating the connection stiffness, may be discontinuous and/or possibly negative, which is physically impossible.

The polynomial has the form:

$$\theta = C_1(aM) + C_2(aM)^3 + C_3(aM)^5 \quad 6.2$$

where C_1, C_2 and C_3 = curve fitting constants.

a = the standardization factor.

(c) Ramberg and Osgood Function:

Ang and Morris (18) replaced the polynomial function of Frye and Morris by a form of a function given by Ramberg and Osgood. The moment rotation expression had the following form:

$$\frac{\theta}{\theta_0} = \frac{aM}{[aM]_0} \left[1 + \left(\frac{aM}{[aM]_0} \right)^{n-1} \right] \quad 6.3$$

where: θ_0 and $[aM]_0$ are as indicated in Fig. 6.4, while the value of n controls the shape of the curve.

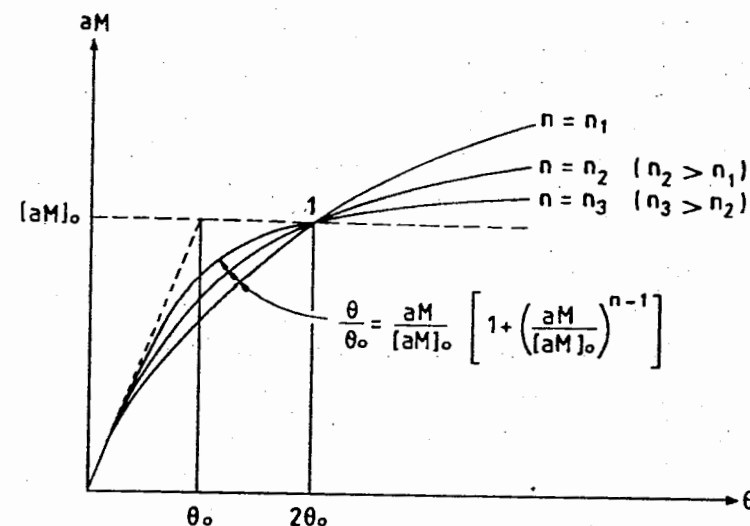


Fig. 6.4. Ramberg-Osgood Function for Moment-Rotation Curves



6.5. Behaviour and Analysis of Nonrotating Supports Structural Systems with Semi-Rigid Connections:

Complexities attendant upon finding and using the appropriate $M-\theta$ curves have made the methods of analysis too cumbersome for routine design. Whether savings can be achieved by using semi rigid rather than rigid or flexible connections is a question that has never been settled. In the following paragraphs an analysis of semi-rigid connections is presented in order to understand the general connection behaviour.

Fig. 6.5, illustrates the simplest application which consists to consider a prismatic uniformly loaded beam element framing into columns which are so stiff relative to the beam that they may be considered nonrotating.

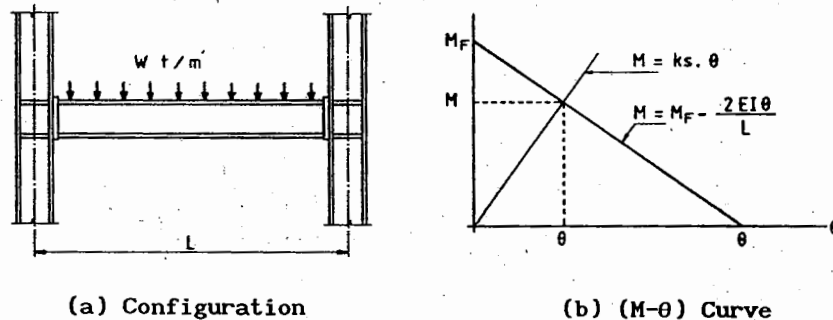


Fig. 6.5 A Semi-Rigid Connection with Nonrotating Supports

If the beam is rigidly connected to the vertical columns, the induced elastic end moment is twice the mid span moment $\left[\frac{M_{END}}{M_{MID}} = \left(\frac{wl^2}{12} \right) / \left(\frac{wl^2}{24} \right) \right]$. Similarly for flexible beam-to-column connections the mid span moment is maximum while the end one is zero. Both extreme situations will lead to



excessive material, while a semi-rigid connection would result probably in maximum moments allowing a reduction in beam material.

The optimum situation will be that of a semi-rigid connection permitting a sufficient end rotation to balance end and mid span moments to $wl^2/16$ as shown in Fig. 6.6.

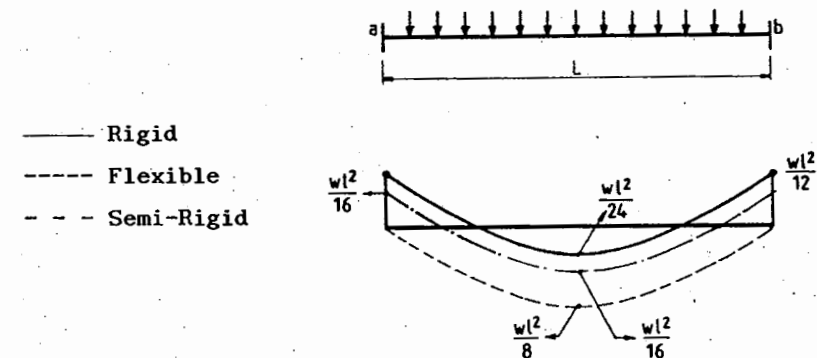


Fig. 6.6. Moment Diagrams

The question which arise now: « What is the percent of rigidity of the semi-rigid connection which will allow some support rotation to change the end moment ».

To answer this question the term Rigidity ratio "R" will be used, it is 100times the ratio obtained by dividing the fixed-end moment into the actual moment capable to minimize the structural element material.

In order to present an analysis sufficient to extend a useful theory for the system, consider as a first approximation that the $(M-\theta)$ curve for the connection is linear as has been previously outlined in equation 6.1. (i.e. $M = K_s \cdot \theta$)



Using the slope-deflection method, the beam line equation of the beam structural element shown in Fig. 6.5.a can be written in the following form:

$$M = M_F - \frac{2EI\theta}{L} \quad 6.4$$

where:

$$\theta_a = -\theta_b = \theta ; \Delta = 0.0$$

M_F = the end moment of a beam fixed at both ends.

For an ideally rigid connection $\theta = 0$, thus $M = M_F$, while for an ideally flexible connection $M = 0$ and hence the end rotation can be computed as follows:

$$\theta = \theta_s = \text{simple beam end slope}$$

$$\theta = \frac{M_F \cdot L}{2EI} \quad 6.5$$

The actual end moment "M" must satisfy equations (6.1) and (6.4). Eliminating θ from these two equations we get:

$$M = \frac{M_F}{\left(1 + \frac{2EI}{K_s}\right)} \quad 6.6$$

Using the definition previously utilized for the rigidity ratio (R), the following relation can be computed:

$$M = R \cdot M_F \quad 6.7$$

Hence equating (M) from equation (6.7) into equation (6.6), the rigidity ratio (R) can be written in the following form:



$$R = \frac{K_s}{\left(K_s + \frac{2EI}{L}\right)}$$

6.8

$$\text{Hence: } K_s = \frac{R \cdot 2EI}{1-R \cdot L}$$

It is to be noticed that three different situations arise according to the following:

(a) The Optimum Configuration where the end and mid span moments are balanced to $wl^2/16$ is corresponding to a rigidity ratio "R" equal to 0.75 (i.e. $0.75 \times \frac{wl^2}{12} = \frac{wl^2}{16}$), while the connection rigidity K_s can be computed using equation (6.8) as follows:

$$K_s = \frac{R}{(1-R)} \times \frac{2EI}{L} = \frac{6EI}{L}$$

(b) The Configuration where the End Moment Governs the Design:

This means that the end moment is greater than $wl^2/16$ and the corresponding rigidity ratio "R" exceeds consequently 0.75.

For a given beam element whose properties are known (i.e. I, Z, L and w), the corresponding end moment M is:

$$M = Z_x \cdot F_b \quad (a)$$

where: F_b = allowable bending stress.

Z_x = modulus section.

Hence, using equation 6.7, get the corresponding rigidity ratio R:

$$R = \frac{M}{M_F} \quad (b)$$

where: $M_F = \frac{wl^2}{12} \cdot M$ as computed using equation (a).

The corresponding connection stiffness (k_s) can be computed using equation (6.8). Thus the moment-curve and the beam line equations will follow equations (6.1) and (6.4) respectively.

(c) The Configuration where the Mid-Span Moment Governs the Design:

Using equation (6.7) the mid span moment is:

$$M_{mid} = \frac{wl^2}{8} - R \frac{wl^2}{12} = F_b Z_x \quad (c)$$

$$\text{i.e. } R = \frac{3}{2} - \frac{F_b Z_x}{(wl^2/12)} \quad (d)$$

$$\text{Hence: } M = R \cdot \frac{wl^2}{12} \quad (\text{using Eq. 6.7})$$

$$K_s = \frac{R}{1-R} \cdot \frac{2EI}{L} \quad (\text{using Eq. 6.8})$$

$$M = K_s \cdot \theta \quad (\text{using Eq 6.1})$$

Refer to example (6.1).

6.6. Design of the Steel Components of Welded Semi-Rigid Connections:

6.6.1. Configuration:

Fig. 6.7, illustrates a typical welded semi-rigid connection. The flexural end moment (M) is replaced by two equal and opposite forces ($T = C = \frac{M}{h}$). The flange beam tensile

force (T) is transmitted to the column flange via a top splice plate, while the flange beam compressive force (C) is transmitted via a seat angle. On the other hand the end reactions (shearing forces) are transmitted via the seat angle.

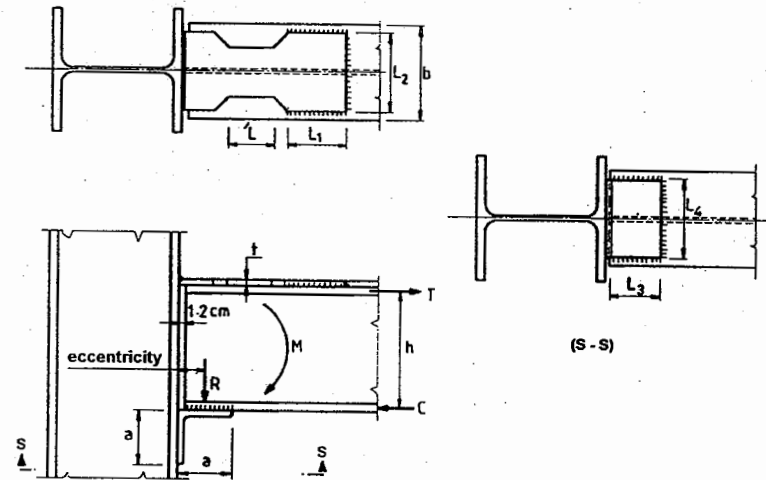


Fig. 6.7

6.6.2. Flexibility of the Connection:

In order to obtain a semi-rigid connection, the top plate must be capable to elongate such that to provide the corresponding end beam rotation θ . Fig. 6.8, illustrates the deformed shape of the semi-rigid connection. The beam is assumed to rotate about the top of the seat angle as follows:

$$\theta = \frac{h}{e} \quad 6.9$$

were:

θ = the angle of rotation.

h = the beam height.

e = the corresponding top plate elongation.

If the assumption of linear connection behaviour is satisfied, the top plate must remain elastic.

Hence:
$$e = \frac{TL}{AE} = \frac{M \cdot L}{hAE} \quad 6.10$$

where: A = cross sectional area of top plate = bt

$$T = \frac{M}{h}$$

E = young's modulus.

Equating (6.10) in (6.9) to eliminate "e" we get:

$$A = \frac{M \cdot L}{E\theta h^2} \quad 6.11$$

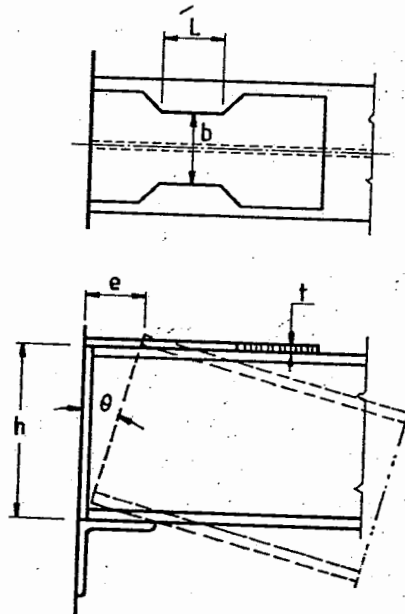


Fig. 6.8

Refer to example (6.2).

6.7. Design of Semi-Rigid Connections with Rotating Supports:

The analysis of a general structural system with semi-rigid connections need a matrix approach. This matrix analysis is presented in section 6.8.

Here, a simple situation is discussed to illustrate some aspects of semi-rigid connections incorporated in a symmetrical structural system with rotating supports. For purpose of simplicity consider the following assumptions:

- (a) The structural beam element AB is symmetrically loaded.
- (b) The semi-rigid connections at (A) and (B) are identical.
- (c) The upper and lower ends of column sections are assumed to rotate the same angle.

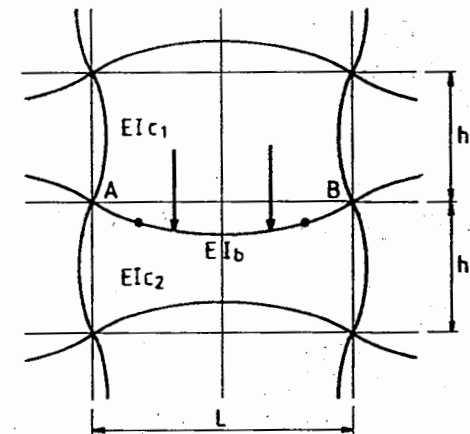


Fig. 6.9. Case of Rotating Supports

Fig. 6.9, illustrates a part of a structural frame system with the corresponding properties (EI_{c1} , EI_{c2} , EI_b) and whose dimensions are (L , h_1 , h_2).

Fig. 6.10.a shows the slope of the left end of the beam elastic line (θ_b), with respect to the horizontal, while the slope of the column elastic line with respect to the vertical is θ_c . The rotation within the semi-rigid connection will be $(\theta_b - \theta_c)$.

Applying as has been previously outlined in section 6.5, the linear moment-rotation curve may be written in the following form:

$$M = K_s (\theta_b - \theta_c) \quad 6.12$$

The beam line equation of section 6.5 will remain the same, only θ is to be replaced by θ_b since it is the total end beam rotation that controls its end moment. Hence:

$$M = M_F - \frac{2EI\theta_b}{L} \quad 6.13$$

The actual end moment (M) must satisfy both relations (6.12) and (6.13) as illustrated in Fig. 6.10.b. Eliminating θ_b from these two equations we get:

$$M = M_F - 2EK_b \left(\frac{M}{K_s} + \theta_c \right) \quad 6.14$$

where: $K_b = \frac{I_b}{L}$

The column rotation (θ_c) can be computed in terms of M and the column properties. Fig. 6.10.c shows the corresponding columns ($\frac{M}{EI}$) diagrams where θ_c can be deduced according to the following:

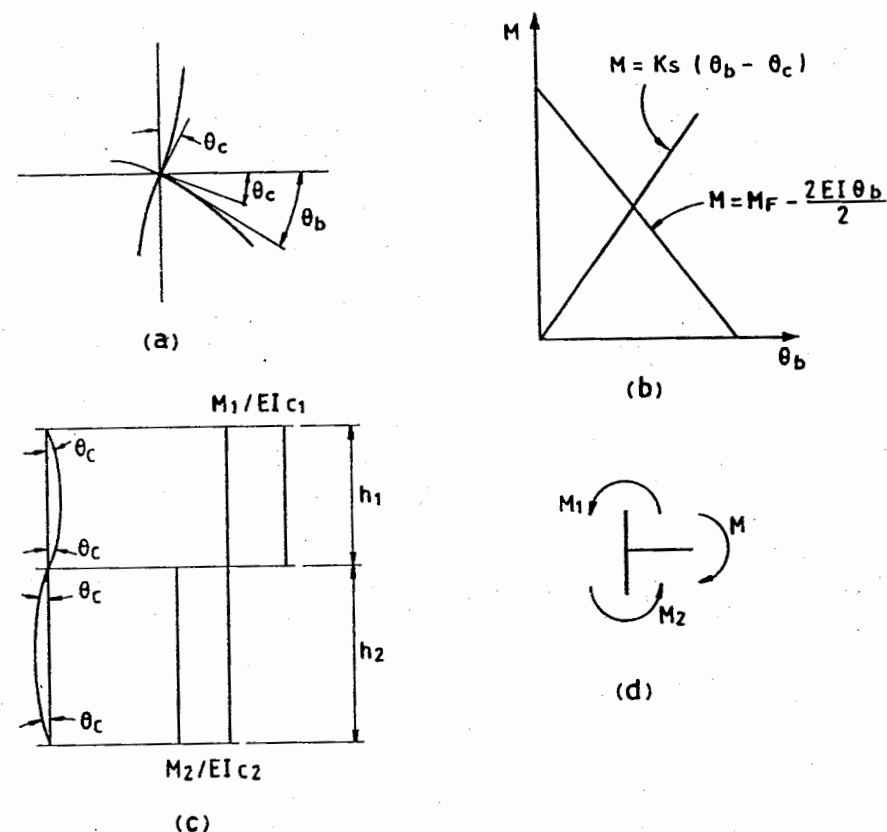


Fig. 6.10

$$\theta_c = \frac{M_{c1} \cdot K_{c1}}{2E} = \frac{M_{c2} \cdot K_{c2}}{2E} \quad 6.15$$

where: $K_{c1} = \frac{I_{c1}}{h_1}$; $K_{c2} = \frac{I_{c2}}{h_2}$



Using relation 6.15, the columns moments can be written in the following form:

$$M_{c1} = 2EK_{c1} \cdot \theta_c ; M_{c2} = 2EK_{c2} \cdot \theta_c \quad 6.16$$

From the joint equilibrium as shown in fig. 6.10.d get:

$$M = M_{c1} + M_{c2}$$

$$M = 2E\theta_c (K_{c1} + K_{c2})$$

Hence: $M = 2E\theta_c \sum K_c \quad 6.17$

Eliminating θ_c from equations (6.14) and (6.17) we get:

$$M = M_f - 2EK_b M \left[\frac{1}{K_s} + \frac{1}{2E \sum K_c} \right] \quad 6.18$$

Substituting the value of (K_s) of equation 6.8 into equation (6.18) and solving for (M) we get:

$$M = \left[\frac{1}{(1/R) + (K_b \sum K_c)} \right] M_f \quad 6.19$$

Refer to examples (6.3) and (6.4).

6.8. Analysis of Plane Frames with Semi-Rigid Connections using the Matrix Approach:

In the previous paragraphs simplified methods of analyses have been utilized for symmetrically loaded structures. However for more complicated structural systems the matrix approach is to be utilized.

However since 1978 an extensive program of research work was conducted by the steel division of the Faculty of Engineering Cairo University regarding the analysis of the different



structural systems with semi-rigid connections. In references [19], [20], [21], [22], [23] and [24] the different results regarding this program have been published.

In the following sections a matrix approach is presented. The analysis is capable to take the effect of rigidity of the connection into consideration.

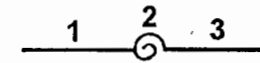
6.8.1. Definition and Assumptions:

1. The semi-rigid connection is represented by a torsional spring having a stiffness " K_s " as shown in Fig. 6.11.a.

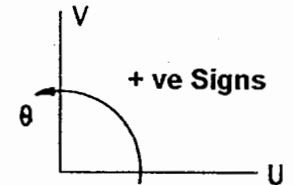
where: K_s is as has been previously given in Equ. 6.1:

$$K_s = \frac{M}{\theta} \quad 6.1$$

θ = Relative rotation between arms (1-2) and (2-3) as illustrated in Fig. 6.11.a.



(a) Idealization of Semi-rigid Connections



(b) Sign Convention

Fig. 6.11

2. Regarding (U,V) the connection is assumed to be rigid, i.e (U,V) just to the left of node "2" are the same as those just to the right.



3. The sign convention for U, V , and θ is as shown in Fig. 6.11.b.

4. The $(M-\theta)$ curve of the semi-rigid connection is assumed to be linear (i.e. $M = K_s \cdot \theta$)

5. The rigidity ratio " R " of a connection is computed by dividing the actual end moment by the fixed end moment as previously outlined in section 6.5, where:

$$M = R \cdot M_f \quad 6.7$$

6. The stiffness " K_s " is computed according to section (6.5) where:

$$K_s = \frac{R}{1-R} \times \frac{2EI}{L} \quad 6.8$$

6.8.2. Shape Functions of a Frame Structural Element with Semi-Rigid Ends:

Ignoring the shear deformations, the differential equation governing the static bending of a uniform structural element is:

$$\frac{d^4 \phi}{dx^4} = 0 \quad 6.20$$

The general solution is:

$$\phi(x) = ax^3 + bx^2 + cx + d$$

The corresponding displacements are illustrated in Fig. 6.12.

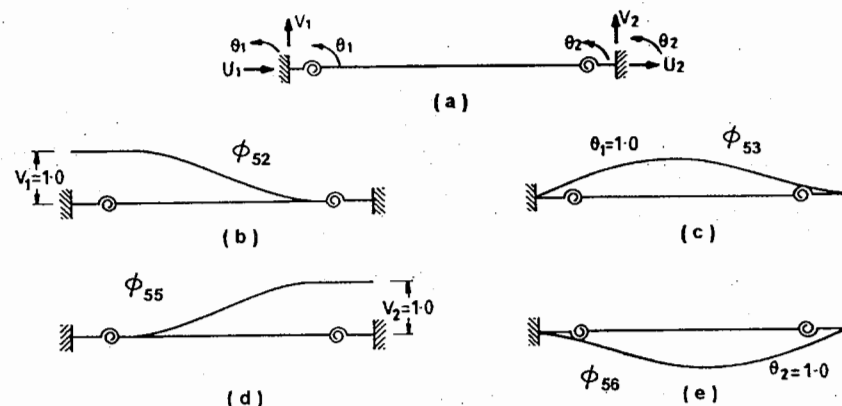


Fig. 6.12

Table 6.1

	θ_1	θ_2
W.R.T θ_1	$\frac{R_1 (2 - R_2)}{3 - R_1 - R_2} \theta_1$	$\frac{R_1 (1 - R_2)}{3 - R_1 - R_2} \theta_1$
W.R.T V_1	$-\left(\frac{3}{L}\right)\left(\frac{1 - R_1}{3 - R_1 - R_2}\right)V_1$	$-\left(\frac{3}{L}\right)\left(\frac{1 - R_2}{3 - R_1 - R_2}\right)V_1$
W.R.T θ_2	$-\frac{R_2 (1 - R_1)}{3 - R_1 - R_2} \theta_2$	$\frac{R_2 (2 - R_1)}{3 - R_1 - R_2} \theta_2$
W.R.T V_2	$\left(\frac{3}{L}\right)\left(\frac{1 - R_1}{3 - R_1 - R_2}\right)V_2$	$-\left(\frac{3}{L}\right)\left(\frac{1 - R_2}{3 - R_1 - R_2}\right)V_2$

If we define by " R_1 " and " R_2 " the rigidity ratios of the semi-rigid connections at both ends, then the relative rotations

θ_1 , θ_2 and the end displacements V_1 , θ_1 , θ_2 according to [19] are as given in table (6.1).

The shape functions $\phi_i(x)$ [$i = 2, 3, 5, 6$] can be computed using the expressions of table 6.1 and applying the following boundary conditions:

- Regarding $V_1 = 0$; the boundary conditions are:

$$\left. \begin{aligned} \phi_{s2}(0) &= 1; & \frac{d\phi_{s2}}{dx}(0) &= \theta_1' \\ \phi_{s2}(L) &= 0; & \frac{d\phi_{s2}}{dx}(L) &= \theta_2' \end{aligned} \right\} \quad 6.21.a$$

- Regarding $\theta_1 = 0$; the boundary conditions are:

$$\left. \begin{aligned} \phi_{s3}(0) &= 0; & \frac{d\phi_{s3}}{dx}(0) &= \theta_1' \\ \phi_{s3}(L) &= 0; & \frac{d\phi_{s3}}{dx}(L) &= \theta_2' \end{aligned} \right\} \quad 6.21.b$$

- Regarding $V_2 = 1.0$; the boundary conditions are:

$$\left. \begin{aligned} \phi_{s5}(0) &= 0; & \frac{d\phi_{s5}}{dx}(0) &= \theta_1' \\ \phi_{s5}(L) &= 1; & \frac{d\phi_{s5}}{dx}(L) &= \theta_2' \end{aligned} \right\} \quad 6.21.c$$

- Regarding $\theta_2 = 1.0$; the boundary conditions are:

$$\left. \begin{aligned} \phi_{s6}(0) &= 0; & \frac{d\phi_{s6}}{dx}(0) &= \theta_1' \\ \phi_{s6}(L) &= 0; & \frac{d\phi_{s6}}{dx}(L) &= \theta_2' \end{aligned} \right\} \quad 6.21.d$$

Using the above boundary conditions, the shape functions are as follows:

$$\theta_{s2}(x) = \frac{x^3}{L^3} \left[\frac{(R_1 + R_2)}{(3-R_1-R_2)} \right] - \frac{3x^2}{L^2} \left[\frac{(R_1)}{(3-R_1-R_2)} \right] - \frac{3x}{L} \left[\frac{(1-R_1)}{(3-R_1-R_2)} \right] + 1 \quad 6.22.a$$

$$\theta_{s3} = \frac{x^3}{L^3} \left[\frac{(R_1)}{(3-R_1-R_2)} \right] + \frac{x^2}{L^2} \left[\frac{R_1(3-R_2)}{(3-R_1-R_2)} \right] + x \left[\frac{R_1(2-R_2)}{(3-R_1-R_2)} \right] \quad 6.22.b$$

$$\theta_{s5} = - \frac{x^3}{L^3} \left[\frac{(R_1 + R_2)}{(3-R_1-R_2)} \right] + \frac{3x^2}{L^2} \left[\frac{(R_1)}{(3-R_1-R_2)} \right] + \frac{3x}{L} \left[\frac{(1-R_1)}{(3-R_1-R_2)} \right] \quad 6.22.c$$

$$\theta_{s6} = \frac{x^3}{L^3} \left[\frac{(R_2)}{(3-R_1-R_2)} \right] - \frac{x^2}{L^2} \left[\frac{(R_1)(R_2)}{(3-R_1-R_2)} \right] - x \left[\frac{R_2(1-R_1)}{(3-R_1-R_2)} \right] \quad 6.22.d$$

6.8.3. The Stiffness Matrix:

To form the stiffness matrix the corresponding stiffness coefficients can be computed using the following relation:

$$k_{ij} = \int_0^1 EI \theta_{Si}''(x) \theta_j''(x) dx \quad 6.23$$

where:

k_{ij} = the stiffness influence coefficient which is defined as; the force corresponding to coordinate "i" due to a unit displacement of coordinate "j".

$\theta_{Si}''(x)$ = the second derivative of the shape function for a beam element whose ends are semi-rigid Eq. (6.22).

$\theta_j''(x)$ = the second derivative of the shape function for a beam element with rigid ends.

E = Material modulus of elasticity.

I = Cross-sectional second moment of area.

On the other hand the stiffness coefficients associated with axial deformations in a beam element can be computed as follows:

$$k_{ij} = \int_0^1 AE \phi_i''(x) \phi_j''(x) dx \quad 6.24$$

Hence the stiffness matrix for a beam element whose ends are semi-rigid connections taking the axial deformations into consideration will take the following form:

$$[SM]_{6 \times 6} = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} \gamma_{12} & \frac{6EI}{L^2} \gamma_1 & 0 & -\frac{12EI}{L^3} \gamma_{12} & \frac{6EI}{L^2} \gamma_2 \\ 0 & \frac{6EI}{L^2} \gamma_1 & \frac{4EI}{L} \gamma_{11} & 0 & -\frac{6EI}{L^2} \gamma_1 & \frac{2EI}{L} \beta \\ -\frac{EA}{L} & 0 & 0 & \frac{EA}{L} & 0 & 0 \\ 0 & -\frac{12EI}{L^3} \gamma_{12} & -\frac{6EI}{L^2} \gamma_1 & 0 & \frac{12EI}{L^3} \gamma_{12} & -\frac{6EI}{L^2} \gamma_2 \\ 0 & \frac{6EI}{L^2} \gamma_2 & \frac{2EI}{L} \beta & 0 & -\frac{6EI}{L^2} \gamma_2 & \frac{4EI}{L} \gamma_{22} \end{bmatrix} \quad 6.25$$

Where the constants β , γ_1 , γ_2 , γ_{11} , γ_{22} , γ_{12} are as defined below.

$$\left. \begin{aligned} \beta &= \frac{(R_1) \cdot (R_2)}{3 - R_1 - R_2} \\ \gamma_1 &= \frac{(R_1)}{3 - R_1 - R_2} ; \gamma_2 = \frac{(R_2)}{3 - R_1 - R_2} \\ \gamma_{11} &= \frac{(R_1) \cdot (3 - R_2)}{3 - R_1 - R_2} ; \gamma_{22} = \frac{(R_2) \cdot (3 - R_1)}{3 - R_1 - R_2} \\ \gamma_{12} &= \frac{R_1 + R_2}{2(3 - R_1 - R_2)} \end{aligned} \right\} \quad 6.26$$



6.8.4. The Stiffness Matrix in Global Coordinates:

The element stiffness matrix previously mentioned in section 6.8.3 is referenced to the local coordinate axes, while the coordinate axes for the whole structure are known, as global axes. The transformation between the global and local coordinates according to Fig. 6.13 can be generated using the classical transformation matrix $[T]$ according to the following:

$$[P] = [T] [\bar{P}] \quad 6.27$$

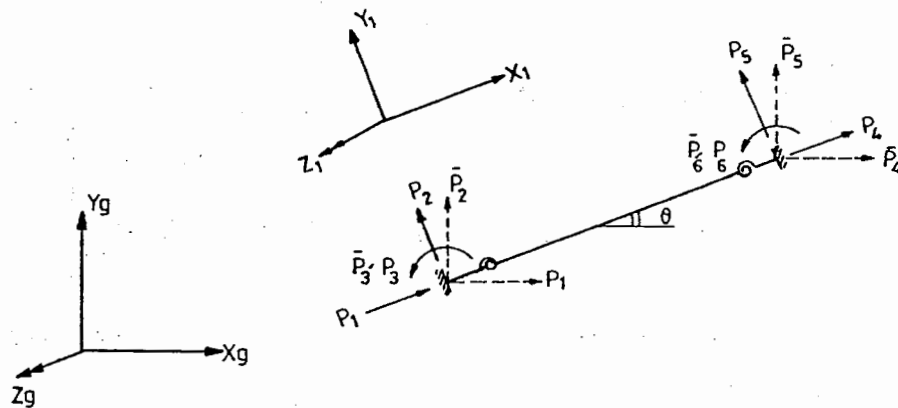


Fig. 6.13. Frame Element Showing the Nodal Forces in Local (X_1, Y_1, Z_1) and Global (X_g, Y_g, Z_g) Axes

Where $[P]$ and $[\bar{P}]$ are the vectors of the element nodal forces in local and global coordinates respectively.

Similarly we get:

$$[\delta] = [T] [\bar{\delta}] \quad 6.28$$

where $[\delta]$ and $[\bar{\delta}]$ are the nodal displacements in local and global coordinates respectively.

Substituting $[P]$ and $[\delta]$ from Eqs. (6.27) and (6.28) into the stiffness matrix referred to local axes we get:

$$[\bar{P}] = [T]^{-1} [SM] [T] [\bar{\delta}] \quad 6.29$$

where:

$$[P] = [SM] [\delta]$$

It can be verified that the transformation matrix $[T]$ is an orthogonal matrix, that is $[T]^{-1} = [T]^T$.

Hence we get:

$$[P] = [T]^T [SM] [T] [\bar{\delta}]$$

$$\text{i.e. } [P] = [\bar{SM}] [\bar{\delta}] \quad 6.30$$

where

$$[\bar{SM}] = [T]^T [SM] [T]$$

= the stiffness matrix referenced to the global system of coordinates.

6.8.5. Fixing End Actions for a Structural element with Semi-Rigid Ends:

The fixing end actions for the structural element with semi-rigid ends shown in Fig. 6.14 are derived in [20] and are as follows:

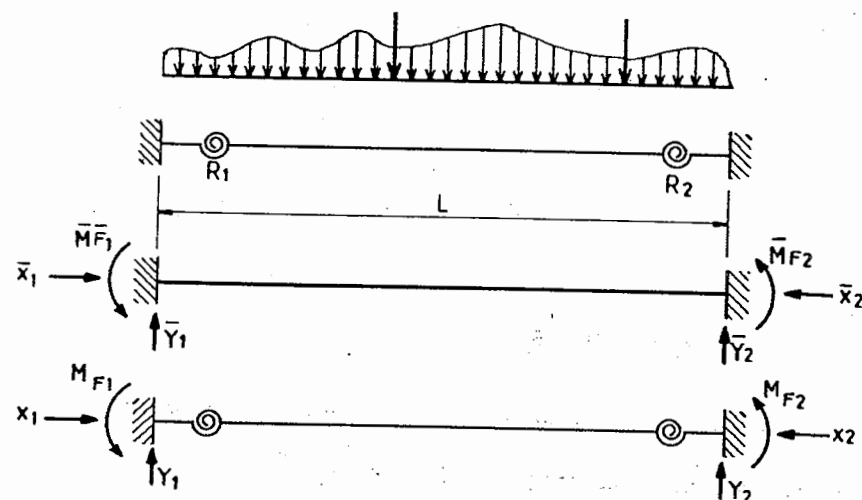


Fig. 6.14

$$\begin{aligned}
 M_{F1} &= \beta (\bar{M}_{F1} (4r_2 + 1) - \bar{M}_{F2} (2r_2)) \\
 M_{F2} &= \beta (\bar{M}_{F2} (4r_1 + 1) - \bar{M}_{F1} (2r_1)) \\
 y_1 &= \bar{y}_1 - \frac{6\beta}{L} (\bar{M}_{F1} \times r_1 \times \alpha_1 + \bar{M}_{F2} \times r_2 \times \alpha_2) \\
 y_2 &= \bar{y}_2 + \frac{6\beta}{L} (\bar{M}_{F1} \times r_1 \times \alpha_1 + \bar{M}_{F2} \times r_2 \times \alpha_2) \\
 x_1 &= \bar{x}_1 \\
 x_2 &= \bar{x}_2
 \end{aligned} \quad \left. \vphantom{\begin{aligned} M_{F1} \\ M_{F2} \\ y_1 \\ y_2 \\ x_1 \\ x_2 \end{aligned}} \right\} 6.31$$



where:

$$r_1 = \frac{1 - R_1}{2 R_1}$$

$$r_2 = \frac{1 - R_2}{2 R_2}$$

$$\alpha_1 = 2 r_2 + 1$$

$$\alpha_2 = 2 r_1 + 1$$

$$\beta = \frac{1}{1 + 4(r_1 + r_2) + 12 r_1 r_2}$$

6.8.6. Assembly of the Overall Stiffness and Load Matrices of the Structure:

For each member of the structure we can partition its global stiffness matrix $[SM]$ as well as its vector of end displacements and its vector of internal end actions, into sub matrices with dimensions equal to the number of degrees of freedom at each end of the element. Hence equation (6.30) will become:

$$\begin{Bmatrix} \bar{P}_1 \\ \bar{P}_2 \end{Bmatrix} = \begin{bmatrix} \overline{SM}(1,1) & \overline{SM}(1,2) \\ \overline{SM}(2,1) & \overline{SM}(2,2) \end{bmatrix} \begin{Bmatrix} \bar{\delta}_1 \\ \bar{\delta}_2 \end{Bmatrix} \quad 6.32$$

where the sub matrix $[\overline{SM}(1,1)]$ is of dimension (3×3) and the sub vectors $\{\bar{P}_1\}$ and $\{\bar{\delta}_1\}$ of dimension (3×1) as there are 3 degrees of freedom at each end of the element.

For a member connecting the two joints J and K of the structure, the process of expanding its global stiffness matrix to the system (overall) stiffness matrix $[S]$ of dimension equals to the total number of degrees of freedom in the system, is done by allocating the different sub matrices in their corresponding positions referred to the global number of the whole structure. This will be in the following form:

$$[S] = \begin{matrix} & \begin{matrix} J & & K \end{matrix} \\ \begin{matrix} J \\ K \end{matrix} & \begin{bmatrix} \dots & SM(1,1) & \dots & SM(1,2) & \dots \\ \dots & SM(2,1) & \dots & SM(2,2) & \dots \end{bmatrix} \end{matrix}$$

A similar expansion can be done to assemble the overall load vector for each element. Applying for all the elements of the structure, the final overall stiffness matrix and load vector are formed.

6.8.7. Boundary Conditions Regarding the Stiffness and Load Matrices:

The external equilibrium cannot be fulfilled unless all rigid body motions are prohibited by sufficient number of supports, that will ensure equilibrium of the external applied loads. The boundary conditions may occur at points of non-zero specified displacements (non-homogeneous). The process of considering the boundary conditions into the system of equilibrium equations of the whole assemblage can be inserted by partitioning the system of equilibrium equations as follows:

$$\begin{bmatrix} [S_{uu}] & [S_{ur}] \\ [S_{ru}] & [S_{rr}] \end{bmatrix} \begin{Bmatrix} \{r_u\} \\ \{r_r\} \end{Bmatrix} = \begin{Bmatrix} \{R_u\} \\ \{R_r\} \end{Bmatrix}$$

where: $\{r_u\}$ is the vector of unrestrained or free displacements and $\{r_r\}$ is that for specified displacements given as:

$$\{r_r\} = \{r_o\} \quad 6.33$$

$\{R_u\}$ and $\{R_r\}$ are the load vectors at the unrestrained and at the restrained displacements, respectively.

The problem then reduces to the solution of the first set of equations:

$$[S_{uu}] \{r_u\} + [S_{ur}] \{r_r\} = \{R_u\}$$

or

$$[S_{uu}] \{r_u\} = \left\{ \{R_u\} - [S_{ur}] \{r_r\} \right\} \quad 6.34$$

in which $[S_{uu}]$ is no longer singular and the problem can be solved to obtain the unknown vector of free displacements $\{r_u\}$. The second set of equations can be used to calculate the reaction components $\{R_r\}$ such that:

$$\{R_r\} = [S_{ru}] \{r_u\} + [S_{rr}] \{r_r\} \quad 6.35$$

A more practical way of this process of modification can be seen by merging the two sets of equations (6.34) and (6.35) in the following form:

$$\begin{bmatrix} [S_{uu}] & [0] \\ [0] & [I] \end{bmatrix} \begin{Bmatrix} \{r_u\} \\ \{r_r\} \end{Bmatrix} = \begin{Bmatrix} \{R_u\} - [S_{ur}] \{r_o\} \\ \{r_o\} \end{Bmatrix}$$

This process of modification is done by modifying first the load vector in case we have non-homogeneous boundary conditions. Hence all elements of the row and column of $[S]$ corresponding to that constraint except the element on the diagonal are made zeros. The diagonal element will be made unity while the

prescribed value of that displacement will be inserted in the appropriate position of the load vector.

6.8.8. Steps of analysis:

The steps involved in analyzing a structure can be summarized in the following:

1. Prohibit all joint displacements and find the system of fixed end actions at the two ends of each of the members of the structure (case a). For each member formulate the vector of end forces $\{P\}_a$ corresponding to case (a).
2. Apply the system of end actions with reversed signs at the joints of the structure to get case (b).
3. Formulate the global stiffness matrix for each member of the structure, then assemble the overall stiffness matrix of the structure.
4. Formulate the load vector $\{R\}$ from the resultant of all external forces at the different joints in case (b). The sub vector of $\{R\}$ corresponding to any of the joints will be the resultant of all end reactions of all members connected at that joint with reversed signs. All elements of $\{R\}$ are referred to the global system of axes.
5. Modify the stiffness and load matrices corresponding to the specified values of geometrical constraints.
6. Solve the system of equilibrium equations to get the displacement vector $\{r\}$ referred to the global system of axes.
7. For any member connecting the two joints J and K formulate the global vector of end displacements:

$$\{\bar{\delta}\} = \left\{ \left\{ \bar{\delta}_1 \right\}^T : \left\{ \bar{\delta}_2 \right\}^T \right\}^T = \left\{ \left\{ \bar{\delta}_J \right\}^T : \left\{ \bar{\delta}_K \right\}^T \right\}^T$$

To find the end forces of the member referred to the local system of axes, calculate:

$$\{\delta\} = [T]^T \{\bar{\delta}\}$$

8. Calculate the vector of end-forces of each member corresponding to case (b), as

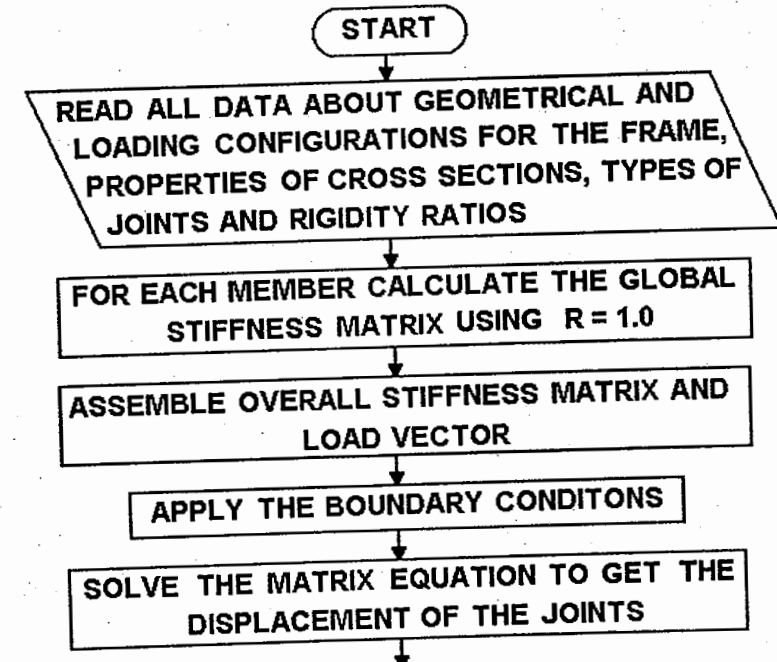
$$\{P\}_b = [SM] \{\delta\}$$

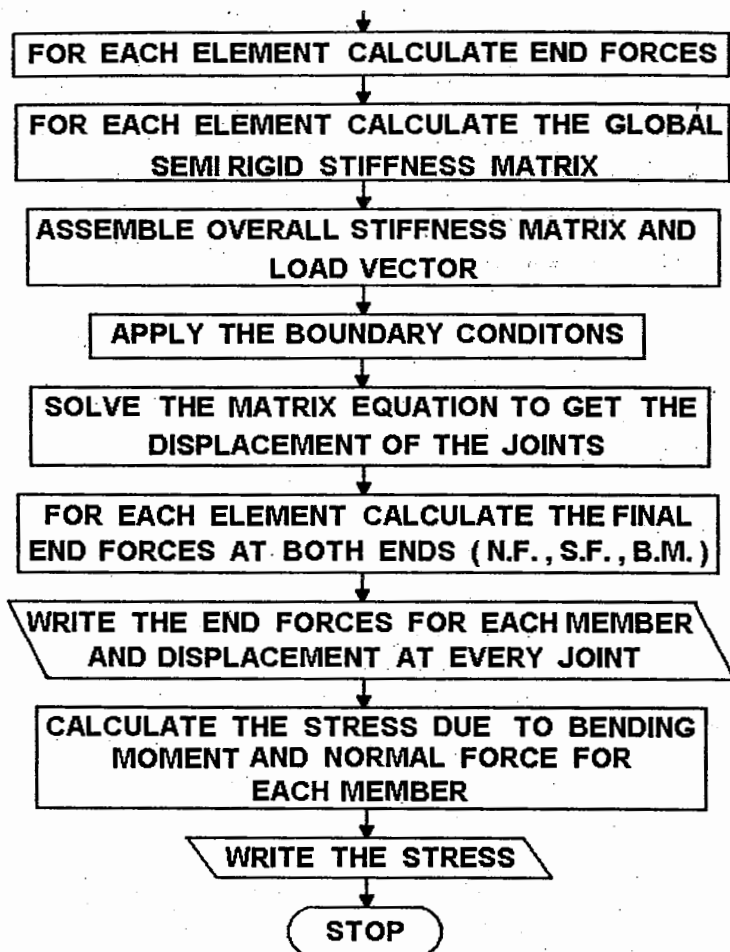
9. The total end-forces of any of the members:

$$\{P\} = \{P\}_a + \{P\}_b$$

$\{P\}_a$ and $\{P\}_b$ should be referred to the same system of axes.

6.8.9. Structure of the Computer Program:





The computer program is written in FORTRAN and Was adapted on the V.M.E - 2900 computer of Cairo University.

** **

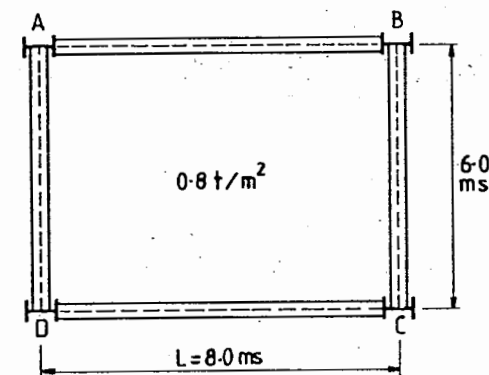


Example (6.1):

Find the suitable semi-rigid connection at (A) and (B) capable to redistribute the end and mid span moments of the over stressed beam element AB whose section is a B.F.I No. 22.

Consider the following two cases:

- The end moment governs the design.
- The mid span moment governs the design.



Solution

1. Analysis as Rigid Connections:

$$W = 0.8 \times 3 = 2.4 \text{ t/m'}$$

$$M_F = M_A = \frac{2.4 \times 8^2}{12} = 12.8 \text{ t.m}$$

$$\text{B.F.I N} = 22 ; Z_x = 732 \text{ cm}^3, I_x = 8050 \text{ cm}^4$$

$$f_c = \frac{12.8 \times 100}{732} = 1.75 \text{ t/cm}^2 > 1.4 \text{ the beam is over stressed.}$$

2. Analysis as Semi Rigid Connections (End Moments Governs):

$$M = Z_x \times F_c = 732 \times 1.4 = 1024.8 \text{ t.cm}$$



$$R = \frac{1024.8}{1280} = 0.8 \quad (\text{Eq. 6.7})$$

$$\text{Hence: } M = 0.8 \times 1280 = 1024 \text{ t.cm}$$

$$f = \frac{1024.8}{732} = 1.4 \text{ t/cm}^2 \quad (\text{o.k.})$$

Using Eq. (6.8) determine the connection stiffness K_s :

$$K_s = \frac{0.8}{0.2} \times \frac{2 \times 2100 \times 8050}{800} = 169 \times 10^3 \text{ t.cm/rad}$$

The (M- θ) curve is:

$$M = 169 \times 10^3 \theta \quad (\text{Eq. 6.1})$$

The corresponding end slope θ of the semi-rigid connection is:

$$\theta = \frac{M}{K_s} = \frac{1024}{169 \times 10^3} = 0.006059 \text{ rad}$$

The mid span moment is:

$$M_{\text{mid}} = \frac{2.4 \times 8}{8} - 10.24 = 8.96 \text{ t.m}$$

The beam line equation is:

$$M = 1280 - \frac{2 \times 2100 \times 8050}{800} \theta \text{ t.cm} \quad (\text{Eq. 6.4})$$

$$M = 12.8 - 422.62 \theta \quad (\text{t.m})$$

3. Analysis as a Semi Rigid Connection (Mid span governs):

Using equation (d) of section 6.5 case (c); get (R):



$$R = \frac{3}{2} - \frac{1.4 \times 732}{1280} = 0.7$$

$$\text{Hence: } M = 0.7 \times 1280 = 896 \text{ t.cm}$$

$$K_s = \frac{0.7}{0.3} \times \frac{2 \times 2100 \times 8050}{800} = 98.6 \times 10^3 \text{ t.cm/rad} \quad (\text{Eq. 6.8})$$

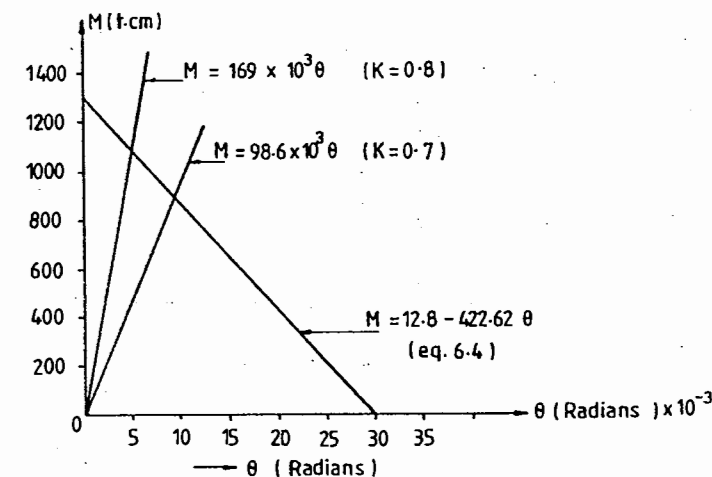
$$M = 98.6 \times 10^3 \theta \quad (\text{Eq. 6.1})$$

The corresponding end slope θ of the semi-rigid connection

$$\theta = \frac{896}{98.6 \times 10^3} = 0.0091 \text{ radians}$$

$$M_{\text{mid}} = \frac{wL^2}{8} - 896 = 1024 \text{ t.cm}$$

The corresponding beam line equation and the moment curve diagrams are as shown in the figure below.



**Example (6.2):**

Design the corresponding welded semi-rigid connection of example 6.1.

Solution

(a) Case of Maximum Moment at the End ($R = 0.8$):

(1) Top Plate Dimensions:

$$M = 1024.8 \text{ t.cm} ; \theta = 0.006059$$

$$h = 22 \text{ cms} ; b = 22 \text{ cms}$$

$$T = \frac{1024.8}{22} = 46.58 \text{ tons}$$

Using for the top plate steel 37 we get:

$$A = \frac{46.58}{1.4} = 33.27 \text{ cm}^2$$

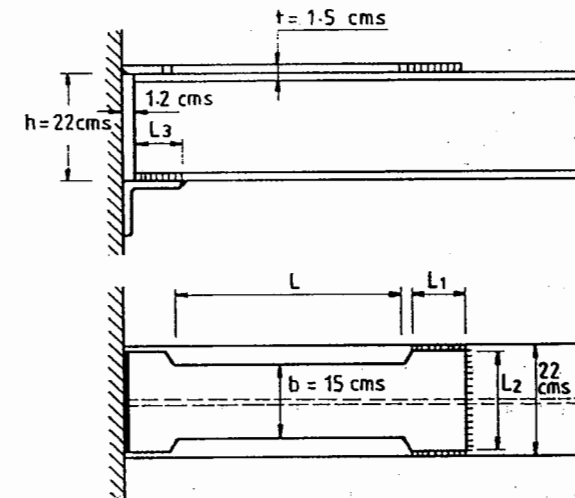
$$\text{Take } b = 22 - 2 \times 3.5 = 15 \text{ cms}$$

$$\text{Hence: } t = \frac{33.27}{15} = 2.218 \text{ cms} , \text{ take } t = 2.3 \text{ cms}$$

Applying in Eq. 6.11 get $\sqrt{L}$:

$$\begin{aligned} \sqrt{L} &= \frac{A.E.\theta.h^2}{M} = \frac{15 \times 2.3 \times 2100 \times 0.006 \times 22^2}{1024.8} \\ &= 205.3 \text{ cms (excessive)} \end{aligned}$$

The required length is excessive and generally is considered impracticable. The alternative is to use high strength steel for the top plate.



Using steel 52 for the top plate ($f = 2.1 \text{ t/cm}^2$), the required length (L) is:

$$A = \frac{46.58}{2.1} = 22.18 \text{ cm}^2 \text{ (} b = 15 \text{ cms, } t = 1.5 \text{ cm)}$$

$$L = \frac{15 \times 1.5 \times 2100 \times 0.006 \times 22^2}{1024.8} = 133.89 \text{ cms}$$

If special steel alloys are utilized for the top plate ($f = 6.4 \text{ t/cm}^2$), hence the length (L) is reduced as follows:

$$A = \frac{46.58}{6.4} = 7.27 \text{ (} b = 10 \text{ cms, } t = 0.8 \text{ cm)}$$

$$L = \frac{10 \times 0.8 \times 2100 \times 0.006 \times 22^2}{1024.8} = 47.6 \text{ cms (take 48 cms)}$$



- (2) To compute the required fillet welds attaching the top plate to the tension beam flange, proceed as follows:

$$T = 46.58 \text{ tons}$$

$$0.72 = \frac{46.58}{1.0 (2L_1 + L_2)} ; L_2 = 22 - 2 \times 2 = 18 \text{ cms} ; s = 1.0 \text{ cm}$$

Hence:

$$L_1 = 23.34 \text{ cms (take } L_1 = 23.34 + 2 \times 1.0 \cong 27 \text{ cms).}$$

On the other hand the top plate is attached to the column flange by a butt weld as shown in figure.

- (3) The compression beam flange is attached to a lower seat angle by 3 lines of weld ($s = 1.6 \text{ cm} = \text{beam flange thickness}$).

$$\sum \text{weld lengths} = \frac{46.58}{0.72 \times 1.6} = 40.43 \text{ cms}$$

$$L_4 = \text{The transversal weld length} = 22 - 2 \times 2 = 18 \text{ cms}$$

$$\text{Hence, the longitudinal weld length} = L_3 = \frac{40.43 - 18}{2} = 11.2 \text{ cms}$$

The required angle size (a) is:

$$a = 11.2 + 1.2 = 12.3 \text{ cms}$$

Hence, choose a seat angle $200 \times 200 \times 20$

The seat angle is to be analyzed as has been previously outlined in section (4.9), while the fillet weld lines attaching the seat angle leg to the column flange will be subjected to:

Weld (1)

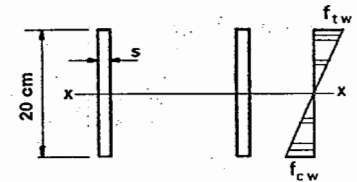
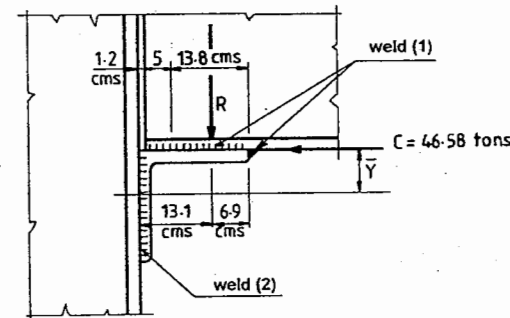
$$s = \frac{46.58}{[2(20-1.2)+22]0.72} = 1.04 \text{ cms} \quad \text{take } s = 1.1 \text{ cms}$$

Weld (2)

$$Q = R = \frac{2.4 \times 8}{2} = 9.6 \text{ tons}$$

$$M = R \cdot e = 9.6 \times 13.1 = 125.76 \text{ t.cm}$$

Choose a seat angle $200 \times 200 \times 20$ with fillet welds $s = 20 \text{ mm}$



$$I = \frac{2 \times 2 \times 20^3}{12} = 2666.66 \text{ cm}^4$$

$$f_{tw} = \frac{125.76 \times 10}{2666.66} = 0.47 \text{ t/cm}^2$$

$$q_{//} = \frac{9.6}{2 \times 2 \times 20} = 0.12 \text{ t/cm}^2$$

$$f_c = \sqrt{(0.47)^2 + 3(0.12)^2} = 0.513 \text{ t/cm}^2 < 0.79 \quad \therefore \text{ Safe}$$

(b) Case of Maximum Moment at Mid span ($R = 0.7$):

$$M = 896 \text{ t.cm} ; \theta = 0.0091$$

Using for the top plate steel 52 we get:

$$T = \frac{896}{22} = 40.72 \text{ tons}$$

$$A = \frac{40.72}{2.1} = 19.39 \text{ cm}^2 \text{ (b = 15 cms, t = 1.3 cm)}$$

$$L = \frac{15 \times 1.3 \times 2100 \times 0.0091 \times 22^2}{896} = 201.2 \text{ cms (excessive)}$$

Hence if an alloy of high strength is utilized for the top plate we get:

$$A = \frac{40.72}{6.4} = 6.36 \text{ cm}^2 \text{ (b = 12 cms ; t = 0.6 cm)}$$

$$L = \frac{12 \times 0.6 \times 2100 \times 0.0091 \times 22^2}{896} = 74.3 \text{ cms}$$

(Take $L = 75 \text{ cms}$)

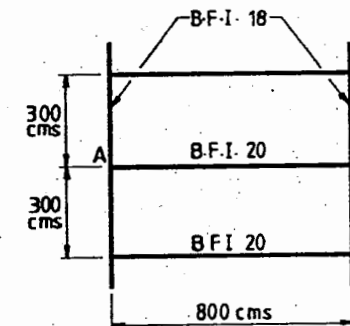
Design the required fillet weld lines connecting the top plate flange as case (a).

Similar to case (a) design the seat angle.

Example (6.3):

The figure below shows a part of a frame structural system. Connections (A) and (B) are rigid connections. The beam element AB is over stressed.

It is required to choose a semi-rigid connection at (A) and (B) to rend the beam element safe.



intensity $w = 2.4 \text{ t/m'}$
as Ex. (6.1).

Solution

$$K_{c1} = K_{c2} = \frac{3830}{300} = 12.76 \text{ cm}^3$$

B.F.I N = 18

$$K_b = \frac{5950}{800} = 7.4375 \text{ cm}^3$$

B.F.I N = 20

$$M_f = 1280 \text{ t.cm (Ex. 6.1)}$$

For ideally rigid connection $R = 1.0$ get:

$$M = \frac{1}{1/1 + (7.4375/2 \times 12.76)} \times M_f = 0.775 \times 1280 = 992 \text{ t.cm}$$

$$M_{c.L} = \frac{2.4 \times 8^2}{8} 992 = 928 \text{ t.cm} < 992 \text{ t.cm}$$



Hence, the end moment governs the design:

$$f = \frac{M}{Z_x} = \frac{992}{595} = 1.66 \text{ t/cm}^2 > 1.4 \text{ t/cm}^2 \text{ Unsafe}$$

The allowable end safe moment is:

$$M = F_c \times Z_x = 1.4 \times 595 = 833 \text{ t.cm}$$

Applying in Eq. (6.19):

$$833 = \frac{1}{(1/R) + 0.291} \times 1280 \quad (\text{Eq. 6.19})$$

Hence: $R = 0.803$

Applying in equation 6.8 get K_s :

$$K_s = \frac{0.803}{1-0.803} \times \frac{2EI}{L} = 127 \times 10^3 \text{ t.cm/rad}$$

Applying equation 6.17 get θ_c :

$$833 = 2 \times E \theta_c \sum K_c = 107184 \theta_c$$

$$\theta_c = 0.00777 \text{ radians}$$

Applying in equation 6.12 get θ_b :

$$833 = 127 \times 10^3 (\theta_b - 0.00777)$$

$$\theta_b = 0.0143 \text{ radians}$$

The same result can be obtained graphically as follows:

$$M = K_s (\theta_b - \theta_c) \quad (\text{Eq. 6.12})$$

$$\theta_c = \frac{M}{2E \sum K_c} \quad (\text{Eq. 6.17})$$

Eliminating (θ_c) we get:



$$M = \left(\frac{2E \sum K_c}{1 + 2E \sum K_c / K_s} \right) \theta_b$$

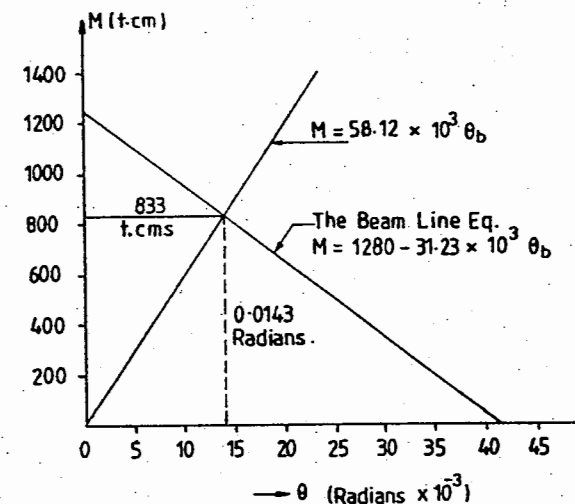
For $R = 0.8$; $K_s = 127 \times 10^3$

Hence: $M = 58.12 \times 10^3 \theta_b$

The beam line equation is:

$$M = M_F - \frac{2EI\theta_b}{L} ; \theta_b = 0 \rightarrow M = M_F = 1280 \text{ t.cm}$$

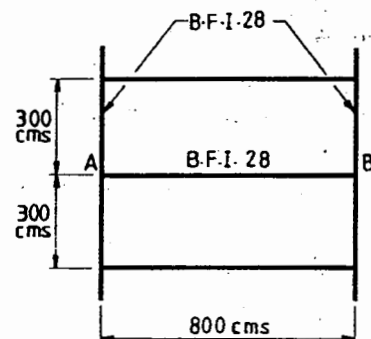
$$M = 0 \rightarrow \theta = 40.9 \times 10^3 \text{ radians}$$



Example (6.4):

The figure below shows a part of a structural frame system. The beams are attached to the columns by flexible supports.

It is required to choose the appropriate rigidity ratio capable to balance the positive and negative moments.



$$W = 2.4 \text{ t/m}$$

$$I_x = 20720 \text{ cms}^4$$

Solution

The beam element being with flexible connections at both ends hence:

$$M_{c.L} = \frac{2.4 \times 8^2}{8} = 19.2 \text{ t.m} = 1920 \text{ t.cm}$$

$$M_f = 1280 \text{ t.cm}$$

$$M (+ve) = M (-ve) = 960 \text{ t.cm}$$

$$K_b = \frac{20720}{800} = 25.9 \text{ cm}^3$$

$$K_c = \frac{20720}{300} = 69.06 \text{ cm}^3 \quad \sum K_c = 2 \times 69.06 = 138.12 \text{ cm}^3$$

$$K_b / \sum K_c = 25.9 / 138.13 = 0.1875$$

Applying in Eq. 6.19

$$960 = \frac{1}{1/R + 0.1875} \times 1280$$

Hence: $R = 0.872$

$$\theta_c = \frac{960}{2 \times 2100 \times 138.13} = 0.00165 \text{ radians (Eq. 6.17)}$$

$$K_s = \frac{0.872}{1 - 0.872} \times \frac{2 \times 2100 \times 20720}{800} = 74.1 \times 10 \text{ t.cm/rad (Eq. 6.8)}$$

$$960 = 74.1 \times 10^3 (\theta_b - 0.00165) \text{ (Eq. 6.12)}$$

$$\theta_b = 0.0146 \text{ radians}$$

Hence use a semi-rigid connection allowing for an angle of rotation $\theta_b = 0.0146$ radians.

For the design of the welded semi-rigid proceed as in example 6.2.

Example (6.5):

1. Proportion a top and bottom plate connection for a B.F.I. No. 20 beam framed to very rigid columns. End moment and reaction are equal to 11.3 t.m and 9.0 t respectively.

2. Estimate the rigidity ratio of the connection designed in (1) when used to support a load of 2.25 t/m' on a 8.0 ms span.

Solution1. Design:

$$\text{Force in top and bottom plates} = \frac{11.3 \times 100}{20} = 56.5 \text{ tons.}$$

Assume to use three fillet weld lines whose size $s = 1.6$ cms (as the beam flange thickness) for the top plate.

$$L_2 = 20 - 2 \times 1.6 = 16.8 \text{ cms.}$$

$$2 L_1 + L_2 = \frac{56.5}{0.72 \times 1.6} = 49.03 \text{ cms}$$

$$\text{Hence: } L_1 = \frac{49.03 - 16.8}{2} = 16.11 \text{ cms; take } L_1 = 17 \text{ cms.}$$

Assuming breadth = 16.8 cms, the thickness of top plate "t", can be calculated as :

$$t = \frac{56.5}{1.4 \times 16.8} = 2.4 \text{ cms}$$

$$\text{Shear stress in web plate} = \frac{9}{12 \times 1} = 0.75 \text{ t/cm}^2 < 0.84 \text{ (o.k.)}$$

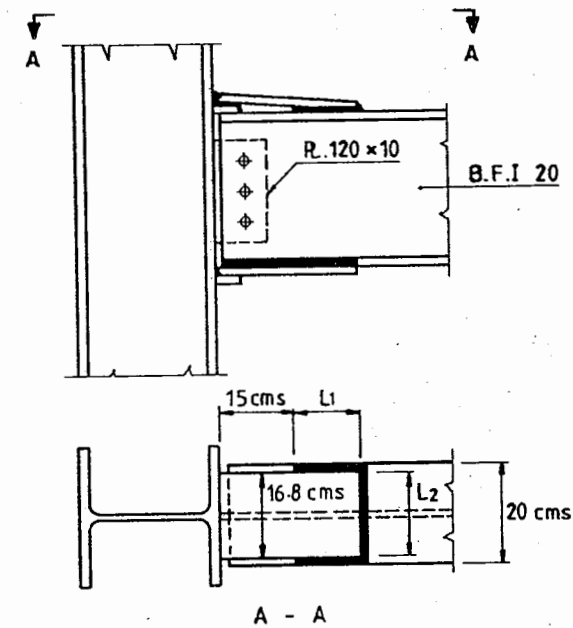
$$\text{Thickness of vertical welds} = \frac{9}{2 \times 12 \times 0.72} = 0.515 \text{ cm}$$



Take $s = 0.6$ cm

$$\text{Force per bolt} = \frac{9}{3} = 3 \text{ tons}$$

Choose 3 bolts M (10.9).

2. Rigidity Ratio of the Connection:

2.1. The following assumptions are to be considered:

- The rotation will occur about the bottom of the beam.
- All the connection rotation will result from the elongation of the 15 cms unwelded length of the top plate.
- The behavior is belonging to the perfect elasticity.

$$\text{Having } M = \frac{AE (h)^2 \theta}{L}$$



Where:

$$A = 16.8 \times 2.4 = 40.32 \text{ cm}^2$$

$$h = 20 \text{ cms; } E = 2100 \text{ t/cm}^2$$

$$L = 15 \text{ cms (assumed).}$$

Hence:

$$M = 2258 \times 10^3 \theta \quad (a)$$

Beam line for 2.25 t/m' on 8.0 ms span is:

$$M = \frac{2.25 \times (8)^2 \times 100}{12} - \frac{2 \times 2100 \times 5950}{800} \theta$$

$$M = 1200 - 31.2 \times 10^3 \theta \quad (b)$$

Solving (a) and (b) we get:

$$\theta = 0.524 \text{ radians}$$

$$M = 1183.6 \text{ t.cms}$$

$$R = \frac{1183.6}{1200} = 0.986$$

2.2 If the rotation of the connection is assumed to occur about the midheight of the beam, hence we get:

$$M = \frac{2258 \times 10^3}{2} \theta = 1129 \times 10^3 \theta \quad (c)$$

Hence solving (b) and (c) we get:

$$M = 1167.73 \text{ t.cm}$$

$$R = \frac{1167.73}{1200} = 0.973$$



3. Comments:

1. Whether one assumes rotation about the middle or the bottom of the beam makes very little difference; the calculated rigidity ratio varies by only 1.3 per cent.

2. It is interesting to note that, if the free length of the top plate were doubled, that is, increased from 15 to 30 cms, the decrease in calculated rigidity would not be large. Assuming rotation about mid-depth we get:

$$M = 1129 \times 10^3 \theta / 2 = 564.5 \theta \quad (d)$$

Solving (b) and (d) we get:

$$M = 1137.15 \text{ t.cms}$$

$$R = \frac{1137.15}{1200} = 0.947$$

3. A similar tested specimen has shown that the rigidity ratio is 0.91. This means that the designed connection is somewhat more rigid than the tested joint.

CHAPTER 7



CHAPTER 7

BEAM, COLUMN AND BEAM-COLUMN SPLICES

7.1. Why a Splice is Required ? :

The main reasons regarding the use of a structural element splice can be presented as follows:

(a) The required structural element length may exceed that available from the mill.

(b) The structural element section may vary along the available mill length according to the design considerations.

(c) For erection considerations, the fabricator may find it economical to splice the structural element, even when the required full length is available.

7.2. Splice Location:

As a general rule, a splice is to be located away from the most stressed sections. This may provide a considerable economy regarding the splice structural components whose function is to satisfy the continuity.

Column as well as truss structural element splices require little attention. The corresponding axial loads do not vary within the truss members while for columns the axial loads variation is not significant within a particular story.

For long continuous beams, when splices are required, they must be located in regions of low bending moment where the structural requirements are not severe.

For beam-columns of a multi-story frame, the splice location will follow the prescriptions of continuous beams.



It is to be noticed that the splice cost is not a linear function with respect to the section capacity percentage. However, the maximum splice cost occurs as the connection design strength approaches the section capacity. Hence, to reach a significant economy, it is recommended to move a splice away to a location whose straining actions are even twenty percent less than the maximum stressed section.

Finally, it is always preferable to locate the splice near a section braced laterally in order to avoid the instability phenomenon regarding the splice structural components. Generally this situation can be achieved easily, otherwise special requirements are to be provided in order to satisfy the continuity.

7.3. Analysis and Strength Requirements:

7.3.1. Truss Structural Members:

The design of a splice of an axially loaded truss member is a straight forward matter. The only question that arises is the appropriate magnitude of the design force for the splice connection.

However regarding this question, diverse recommendations are available which are as presented below:

(a) The full splice connection is to be designed so that it develops the actual stress, but not less than 50 percent of the effective strength of the member.

(b) The full splice connection shall be designed for the average of the calculated stress, but they shall also be designed for not less than 75 percent of the strength of the member.

(c) Full splice connections shall have a strength not less than that of the member, based on the allowable unit stress in the member.



From these different opinions, the third one is utilized, by the E.C.P. 2001 Clause 9.2.2.4 above.

Hence for the design of a full splice connection, we can proceed as follows:

(i) Compute the required fasteners using the maximum strength for tension as well as for compression axially loaded members using the allowable stresses (F_t and F_c).

(ii) Compute the required splice plate dimensions so that the induced stresses in each splice plate do not exceed the allowable stresses as prescribed by the E.C.P. 2001.

Only if splices are located along a zero structural member, the splice design force must correspond to 50 percent of the member strength. The zero members may be visualized as a tension structural element.

Refer to Examples 7.1, 7.2, 7.3.

7.3.2. Column Splices:

Column splices are splices in axially loaded compression members. The applied loads being concentric, no splice is theoretically required. The compression force is transmitted from one structural element to the other by direct contact (i.e. direct bearing) where the column sections could be rested one on top of each other. Unfortunately this situation does not never exist for the following reasons:

(a) The load is never truly axial.

(b) The bearing surfaces of the adjacent sections, must be carefully machined. This will never be perfect.



(c) All real columns will resist small bending moments induced from laterally applied loads where the actual wind bracing connections provide generally some eccentricities.

Hence, the steel components must be spliced adequately. The required fasteners as well as the splice plates dimensions should correspond to the maximum strength of the member according to the prescriptions of the E.C.P. 2001 as stated in Clauses 8.7 and 9.2.2.4 as follows :-

a) For tension members :-

$$\left. \begin{aligned} N_{\max.t} &= A_{tn} \cdot F_t \\ N_{wt} &= N_{\max.t} \cdot (A_{wn} / A_{tn}) \\ N_{ft} &= N_{\max.t} \cdot (A_{fn} / A_{tn}) \end{aligned} \right\} 7.1$$

Where :-

F_t = The allowable tensile stress ($0.58 F_y$).

$N_{\max.t}$ = Maximum strength of the tension member.

A_{tn} = The effective net cross sectional area of the member.

A_{wn}, A_{fn} = The effective net cross sectional area of the web and the flanges respectively.

N_{wt}, N_{ft} = The maximum strength of the web and the flanges respectively.



b) For compression members :-

$$\left. \begin{aligned} N_{\max.c} &= A_{tg} \cdot F_c \\ N_{wc} &= N_{\max.c} \cdot (A_{wg} / A_{tg}) \\ N_{fc} &= N_{\max.c} \cdot (A_{fg} / A_{tg}) \end{aligned} \right\} 7.2$$

Where :

F_c = The allowable buckling compressive stress.
(Clause 2.6.4 - E.C.P. 2001)

$N_{\max.c}$ = Maximum strength of the compression member.

A_{tg} = The total gross sectional area of the member.

A_{wg}, A_{fg} = The gross sectional area of the web and the flange respectively.

N_{wc}, N_{fc} = The maximum strength of the web and the flange respectively.

Refer to Example 7.4.

7.3.3. Beam Splices:

Just as in truss members (Section 7.3.1), practice varies with respect to the magnitude of the moments and shear for which a beam splice should be designed. At one extreme, one might design the splice to resist the actual calculated moment and shear at the point of the splice, even though this may be much less than the full capacity of the section. At the other extreme, the splice might be designed to develop the full

resistance of the section in both flexure and shear even though, at the splice location, the actual moment and shear will not attain these values. Some opinions adopt a middle situation between these extremes.

In the following paragraphs, the analysis is based on the maximum bending resistance of the girder section ($M_{\max}$) and the actual shearing force (Q_{act}) at the splice location according to the E.C.P. 2001 Clause 7.5. Accordingly the following expressions can be deduced :-

$$\left. \begin{aligned} M_{\max} &= Z_x \cdot F_b \\ M_w &= M_{\max} \cdot (I_w / I_t) \\ M_f &= M_{\max} \cdot (I_f / I_t) \end{aligned} \right\} \quad 7.3$$

Where :

$M_{\max}$ = The maximum bending resistance of the girder section.

F_b = $0.58 F_y$ for noncompact sections.

= $0.64 F_y$ for compact sections.

Z_x = The elastic modulus section.

I_t = The moment of inertia of the whole section w.r.t the neutral axis.

I_w, I_f = The moment of inertia of the web and the flanges w.r.t. the neutral axis respectively.

M_w, M_f = The maximum bending resistance of the web and the flanges respectively.

Each component of the splice is designed to achieve the work which the material underlying the splice plate could do if it was uncut. Plates on the flange do the work of the flange, and plates over the web do the work of the web according to partition of moments as given in Equ. 7.3.

In this case, the web splice is to be designed to resist in addition to M_w a moment arising from the local eccentricity (e) of the actual shear Q_{act} as well as the actual shearing force at the location of the splice which means that :-

$$\left. \begin{aligned} Q_w &= Q_{\text{act}} \\ M_{wt} &= M_{\max} \cdot \frac{I_w}{I_t} + Q_{\text{act}} \cdot e \end{aligned} \right\} \quad 7.4$$

Where :-

$Q_w = Q_{\text{act}}$ = The actual shearing force at splice location.

M_{wt} = The total web moment at C.G. of the bolts.

e = The local eccentricity between the splice location and the C.G of the group of bolts (see Fig. 7.1).

Fig. 7.1. shows a beam splice where the splice flange bolts are subjected to a concentric shear according to the following :

$$\left. \begin{aligned} Q_{bf} &= F / n_1 \\ F &= \frac{M_f}{d_b} \end{aligned} \right\} \quad 7.5$$

where :



Q_{bf} = The shear induced in the flange splice bolts.

F = Force carried by flanges due to (M_f) .

d_b = Distance between C.G. of flanges = $h - t_f$.

n_1 = The number of bolts in one side of the flange splice.

Regarding the web splice bolts they will be subjected to a concentric shear Q_{bw} due to the applied shearing force at splice location as well as to a shear due to the deduced moment (M_w) as given by Eqs. 7.3 and 7.4. This can be showed according to the following relations :-

$$\left. \begin{aligned} Q_{bw} &= Q_{act} / n_2 \\ Q_{bw.M_{wt}} &= \frac{M_w \cdot r_i}{\sum r_i^2} \end{aligned} \right\} \quad 7.6$$

Both Q_{bw} and $Q_{bw.M_{wt}}$ are to be added vectorially and checked against the shear and bearing resistance for non-pretensioned bolts or P_s for H.S.B.

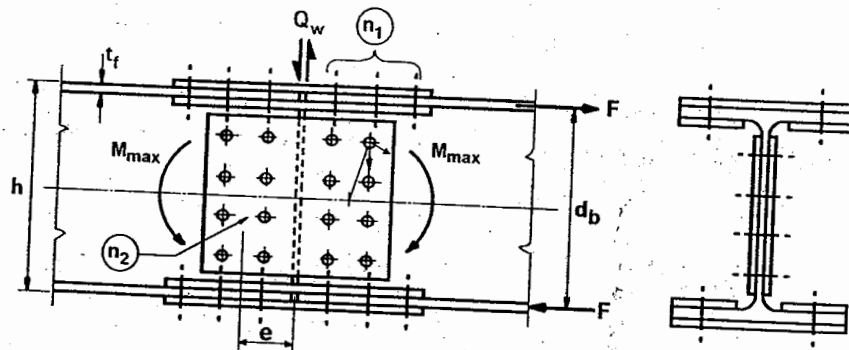


Fig. 7.1. Beam Splice

As the splice moments are usually less than the section capacity there is no danger to obtain any overstress in the section. Thus, no need to check the state of stresses in the net section where holes are provided for bolts installation.

Refer to Examples 7.5, 7.6, and 7.7.

7.4. Beam-Column Splices:

The design of a beam-column splice will be in accordance to the prescriptions previously outlined in Sections 7.3.2 and 7.3.3.

Refer to Example 7.8.

7.5. Different Types of Beam Splices:

7.5.1. Butt-Weld Splice Connections:

For welded plate girders, and frequently for rolled beams, the splice may be accomplished by a full-penetration groove weld. Splices made entirely in the shop are nearly always groove welded. From a design point of view this type does not present any problem, since full-penetration welds are as strong as the base material.

Only some practical considerations are to be provided to ensure an adequate butt-weld splice connection:

(a) A staggered location of the flange/web welds is to be provided as illustrated in Fig. 7.2.a. This is helpful for the welding process but unfortunately will be more expensive.

(b) Cope holes are to be utilized to facilitate the flange weldings. These cope holes are not filled.

(c) It will be of good practice to carry individual runs of weld on the three elements in sequence in order to balance the shrinkage between these elements. Alternatively, the flange welds are to be completed before carrying out the web welds.

(d) To facilitate the alignment, a short distance "x" can be omitted without weld prior to assembly and then completed after the main welds have been carried out. (see Fig. 7.2.a)

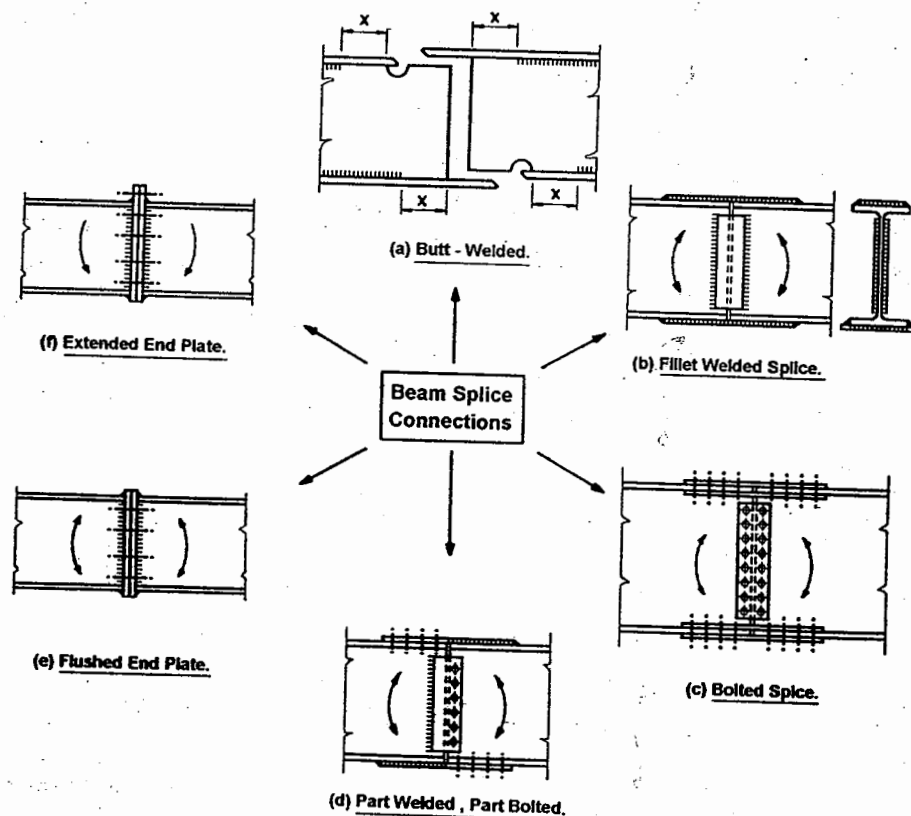


Fig. 7.2

7.5.2. Fillet Welded Splice Plate Connections:

Field splices are becoming more frequently all welded. Though, the use of splice plates fillet welded to the beam components instead of butt-welds will probably overcome the very costly requirements of good fit which are necessary for butt-welded connections.

Fig. 7.2.b, illustrates a single splice beam connection where the use of a double-splice plates is not easy. The design considerations follow the prescriptions of concentric and eccentric connections previously outlined in Chapter 3 (volume 3).

7.5.3. Bolted Splice Plate Connections:

The most common bolted splice plate connection shown in Fig. 7.2.c is that one with high strength bolts of the friction type. Its main advantage is that the only existing deformations are those which correspond to the plate steel elements, the bolts slip being omitted.

Usually a double-splice plates are utilized, this leads to a considerable economy where the number of bolts is consequently reduced.

The bolts are subjected to a concentric shear ($F = M_f / d$), to rend the assumption of uniform distribution of forces valid, it is recommended to achieve the following considerations:

- (a) Double-splice plates are to be used.
- (b) The pitch distance between bolts is to be chosen minimum as possible.
- (c) In no case, the number of bolts in one row must not exceed six bolts, otherwise the bolt diameter is to be increased.



7.5.4. Welded / Bolted-Splice Plate Connection:

Fig. 7.2.d, illustrates at splice-plate Connection where half of the splice is fillet welded in the shop, while the other half is bolted in the site.

Generally it is not easy to use a double-plate splices. Hence the scale of application is limited for rolled beams whose flange breadth is less than 20 cms. The use of high strength bolts of the friction type is essential to provide adequate stiffness. The main disadvantage of this type of connections is the expected damage of the welded splice during transportation.

7.5.5. The End-Plate Splice Connections:

Fig. 7.2.e, illustrates a flushed end-plate splice connection which can be utilized only when the beam is subjected to modest moments.

Fig. 7.2.f shows an extended end-plate splice connection where high moments of one sign occurs. However, if the beam is subjected to full reversal of moments, a doubly extended end plate is to be used.

The behavior, the analysis and the design will follow exactly the prescriptions previously outlined in chapter 5. regarding the apex portal frame connections.

High strength bolts of the friction type are to be utilized. The prying force effect is to be computed and added to the applied external tension force induced by the existing moment.



7.6. The Different Types of Column-Splice Connections:

Column splice connections are generally straight forward. The absence of tensile forces makes the matter easier. The forms of column splice connections are generally as for beam splices. The different common forms are as presented below.

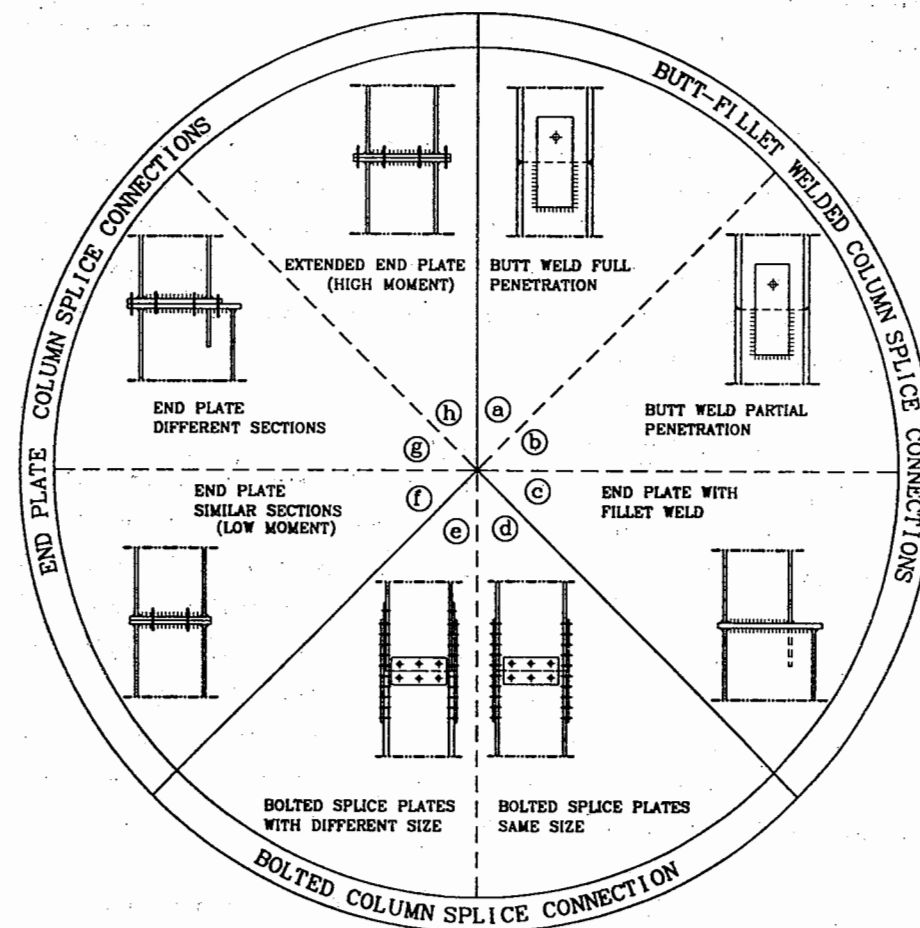


Fig. 7.3. Different Types of Column-Splice Connections



7.6.1. Butt-Welded Splices:

Fig. 7.3.a shows a full-penetration butt-weld configuration where the upper column compression force is transmitted totally to the lower column by butt-welds.

Fig. 7.3.b, illustrates a partial penetration butt weld configuration. A portion of the axial load is transmitted by bearing of the adjacent column sections, while the remaining portion via the butt welds.

Fig. 7.3.c shows a splice connection at a location where a change in columns size exists. If the exterior face is to be maintained flush, a division plate as well as a stiffener (shown by the dotted lines) are to be utilized. The lower column is shop fillet welded to the division plate while the upper column is fillet welded in the site.

7.6.2. Bolted Splice Plate Connections:

Fig. 7.3.d, illustrates the most common type of bolted splice plate connections. If the columns are only subjected to compressive axial loads, hence the bearing between the adjacent column sections will be sufficient. Only a minimum number of bolts is to be utilized to connect the splice plates with the corresponding column flange to maintain the alignment of the column elements. However, if the columns are subjected to moments, the splice plates are designed to transmit the outlined straining actions of Sections 7.3.3 and 7.3.4.

Fig. 7.3.e shows a bolted splice column connection at a location where a change in column size exists. In this configuration packs are to be utilized.



On the other hand if the change in columns size is great, a division plate as well as packs are to be provided.

7.6.3. The End-Plate Column Splice Connections:

Figs 7.3.f, g and h shows the different common end-plate column splice configurations for low moments, change in columns size and high moments respectively.

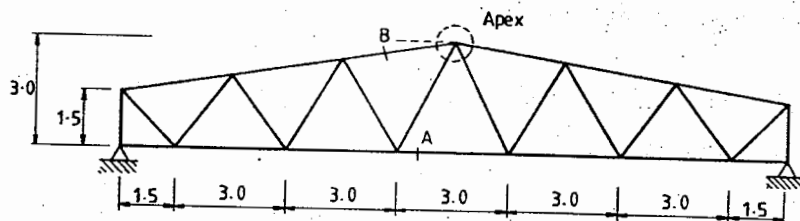
The behavior, analysis and design will follow exactly Sections 7.3.3 and 7.3.4.

This connection is generally located within the floor slab, hence it should not cause any architectural difficulties.

** **

Example (7.1):

Design a field bolted splice for member (A) at the marked location of the figure below as well as the apex splice regarding the apex connection. Thickness of gusset PL is 8 mm. For field splices the number of bolts is increased by 15 %. Use bolts of the non-pretensioned Grade (4.6) - $d = 20$ mm - angles are of st.37.



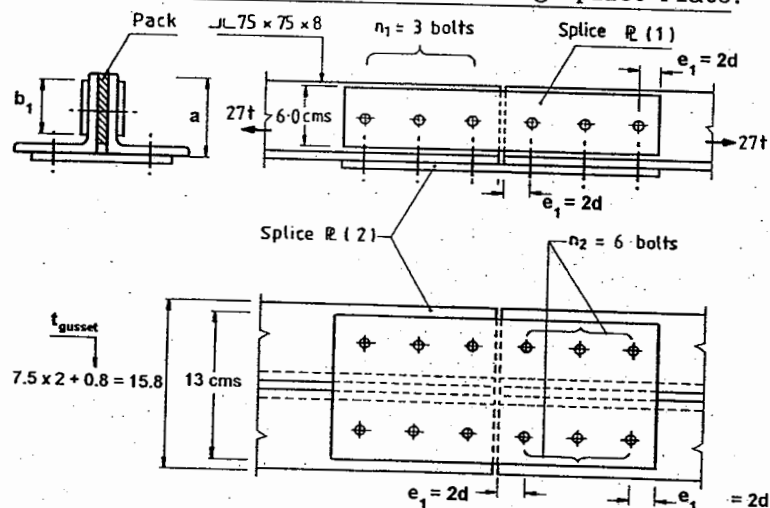
2 Ls

Design force of Member (A) = + 27 tons (case (I)) (75 x 75 x 8)

Design force of Member (B) = - 25.5 tons (case (I)) $\frac{2 \text{ Ls}}{(100 \times 100 \times 10)}$

Solution

(1) Splice of Bottom Chord Member (A) Using Splice Plate:



Maximum strength of Member (A) = $F_t \cdot A_{net}$

where:

$$F_t = 1.4 \text{ t/cm}^2 \quad (\text{Case I}).$$

$$A_{\text{net}} = 2 [A_g - (d + 2\text{mm}) t] = 2 [11.5 - (2+0.2)0.8] = 19.48 \text{ cm}^2$$

For $< 75 \times 75 \times 8$ ($A_g = 11.5 \text{ cm}^2$) - $d = 2 \text{ cms}$

M.S. = Maximum Strength = $1.4 \times 19.48 = 27.272$ tons

(a) Splice Plate (1) - Two plates :-

Assume that half the design force 27.272 tons is transmitted from the left side to the right one via the splice plate (1). Hence the required number of Non pretensioned bolts (Grade 4.6) - ($d = 20$ mm) is:

$$R_{s.s} = 0.25 \times 4 \times \frac{\pi \times (2)^2}{4} = 3.14 \text{ tons}$$

Take edge distance $e_1 = 2d$; $\alpha = 0.8$

$$R_b = 0.8 \times 3.6 \times 2 \times 0.8 = 4.608 \text{ tons}$$

$$n_1 = \frac{27.272 / 2}{2 \times 3.14} \times 1.15 = 2.49 \text{ bolts ; take } n_1 = 3 \text{ bolts}$$

Two splice plates field splice

Take $b_1 = a - 2t_c = 7.5 - 2 \times 0.8 = 5.9$ cms take 6.0 cms

The thickness of splice PL (1) $t_1 = \frac{27.272 / 2}{2 \times 1.4 \times (6.0 - 2.2)} = 1.28 \text{ cm}$

(b) Splice Plate (2) - One plate :-

The other half of the design force 27.272 t :-

$$n_2 = \frac{27.272}{2 \times 3.14} \times 1.15 = 4.99 \text{ bolts take 6 bolts}$$

Take $b_2 = 13 \text{ cms}$



The thickness of splice PL (2) $t_2 = \frac{27.272 / 2}{1.4 \times (13 - 2.2 \times 2)} = 1.13 \text{ cm}$

(c) Check of Stresses for Member (A) at Bolt Location:

$$2 L_S 75 \times 75 \times 8 ; (A_g = 11.5 \text{ cm}^2).$$

$$A_{\text{net}} = 2 (11.5 - 0.8 \times 2.2) = 19.48 \text{ cm}^2$$

$$f_{t.a} = \frac{27.272}{19.48} = 1.4 \text{ t/cm}^2 \quad (\text{o.k.})$$

(2) Splice of Bottom Member (A) using Splice Angles:

Use 2 splice angles $65 \times 65 \times 11$ ($A = 13.2 \text{ cm}^2$)

Use bolts M 16 for the condition $a \geq (3d + t)$.

$$R_{s.s} = 0.25 \times 4 \times \frac{\pi \times (1.6)^2}{4} = 2.0 \text{ tons}$$

$$R_b = 0.8 \times 3.6 \times 1.6 \times 0.8 = 3.68 \text{ tons } (e_1 = 2d)$$

$$n_1 = \frac{27.272 / 2}{2 \times 2.0} \times 1.15 = 3.92 \text{ bolts ; take } n_1 = 4 \text{ bolts}$$

Check of stresses of splice angles:

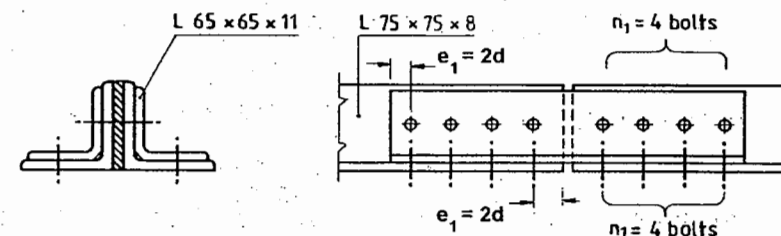
$$A_{\text{net}} = 2 (13.2 - 2 \times 1.8 \times 1.1) = 18.48 \text{ cm}^2$$

$$f_{t.a} = \frac{27}{18.48} = 1.46 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \quad (\text{o.k.})$$

While unsafe but it can be accepted where only the excess beyond 15 % of the hole areas can be deducted.

$$A_{\text{net}} = 2 (13.2 \times 1.15 - 2 \times 1.8 \times 1.1) = 22.44 \text{ cm}^2$$

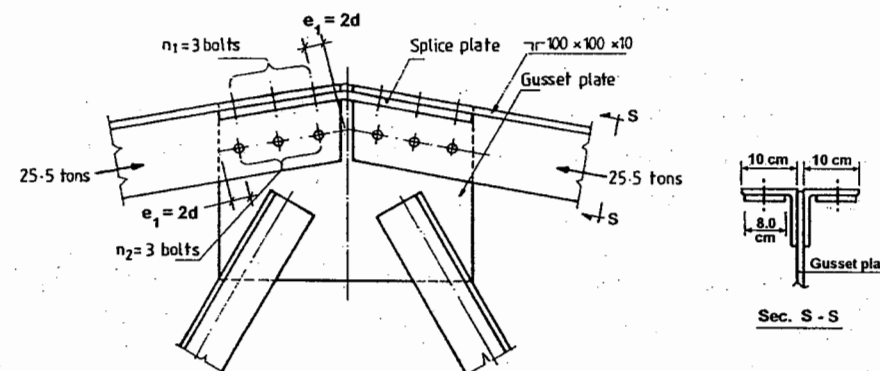
$$f_{t.a} = \frac{27}{22.44} = 1.2 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \quad (\text{o.k.})$$



(3) Splice of the Apex Connection:

The apex connection is not a full splice connection hence the design of the fasteners is based on the actual force.

Assume half of the force of the upper chord is transmitted via the gusset plate and that the other half is transmitted via the splice plate.



(a) Splice Plate:

Use Non-pretensioned bolts Grade (4.6) - diameter 20 mms

$$R_{s.s} = 3.14 \text{ tons} ; R_{ds} = 6.28 \text{ tons}$$



$$R_b = 0.8 \times 3.6 \times 2 \times 1 = 5.76 \text{ tons } (e_1 = 2d)$$

$$n_1 = \frac{25.5 / 2}{2 \times 3.14} \times 1.15 = 2.33 \text{ take } n_1 = 3 \text{ bolts}$$

$$\text{width of splice plate} = b_1 = 10 - 2 \times 1.0 = 8.0 \text{ cms}$$

$$\text{thickness of splice plate} = \frac{25.5 / 2}{1.4 \times (8 - 2.2)} = 1.57 \text{ cms}$$

take 1.6 cms

(b) Gusset Plate:

$$R_{s.s} = 3.14 \text{ tons} ; R_{ds} = 6.28 \text{ tons}$$

$$R_b = 0.8 \times 3.6 \times 2 \times 0.8 = 4.608 \text{ tons } (e_1 = 2d)$$

$$n_2 = \frac{25.5}{2 \times 4.608} = 2.76 \text{ bolts take 3 bolts}$$

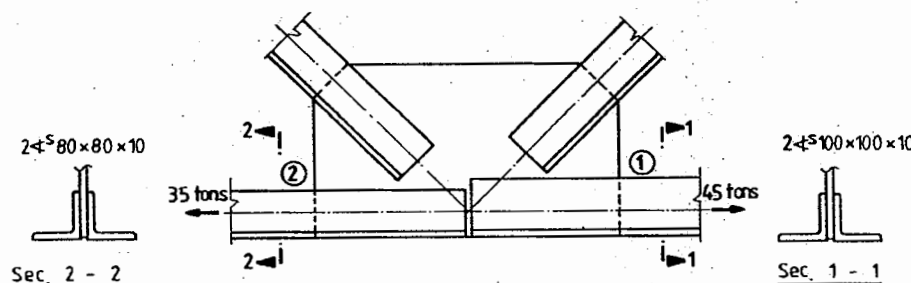
(c) Diagonal Members:

The diagonal members are to be attached to the gusset by fillet weld lines or bolts depending on the design process of the structural system.

The weld lines or the required number of ordinary bolts are computed according to the corresponding design force as has been given in chapters (1) & (2).

**Example (7.2):**

Design the shown bolted connection using bolts of the Non-pretensioned type of diameter 20 mm (Grade 4.6). Thickness of gusset plate is 8 mm - Angles St.37.

**Solution**

$$R_{d.s} = 0.25 \times 4 \times 2 \times \frac{\pi d^2}{4} = 6.28 \text{ tons}$$

$$R_b = 0.8 \times 3.6 \times 0.8 \times 2 = 4.608 \text{ tons}$$

$$\text{Number of bolts required for Member (1)} \quad n_1 = \frac{45.0}{4.608} = 9.76$$

$$= 10 \text{ bolts} > 6 \text{ bolts}$$

unsafe

Alternative (1) :-

The number of bolts required to transmit the D.F. = 45 tons to the gusset plate being 11 bolts which means that the distance between the two far bolts is :

$$L_i = 3d \times 9 \text{ (spacing)} = 27d > 15d$$

$$\text{Then a reduction factor } \beta_L = 1 - \frac{(L_i - 15d)}{200} \leq 1.0$$

$$\beta_L = 1 - \frac{(27d - 15d)}{200} = 0.94$$

Hence $R_{d.s} = 0.94 \times 6.28 = 5.9$ tons

$$R_b = 0.94 \times 4.608 = 4.33 \text{ tons}$$

$$n_1 = 45 / 4.33 = 10.4 \text{ bolts} - \text{take 11 bolts.}$$

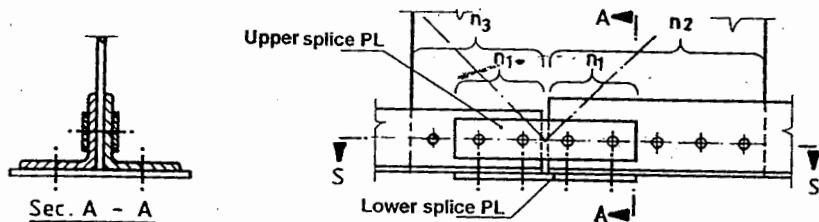
The dimension of the gusset plate will be huge.

Alternative (2) :-

Use splice plates to transmit part of the force from the right structural member to the left one.

(1) Using Splice Plates (Bolted):

The design force are to be utilised as the splice is not full.



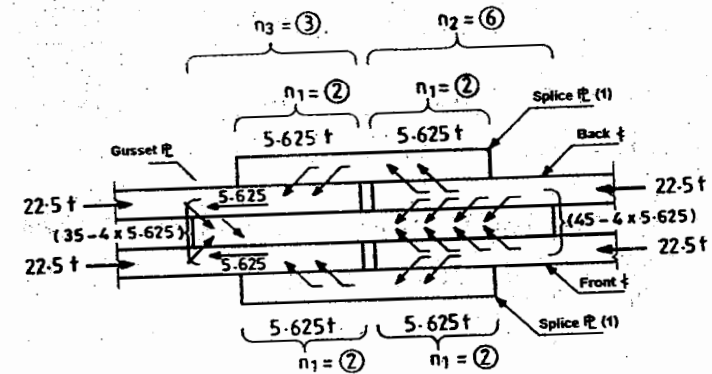
Assume half of the largest force is transmitted via two upper and a lower splice plates i.e. $45/2 = 22.5$ tons.

(A) Upper Splice Plate (1) :-

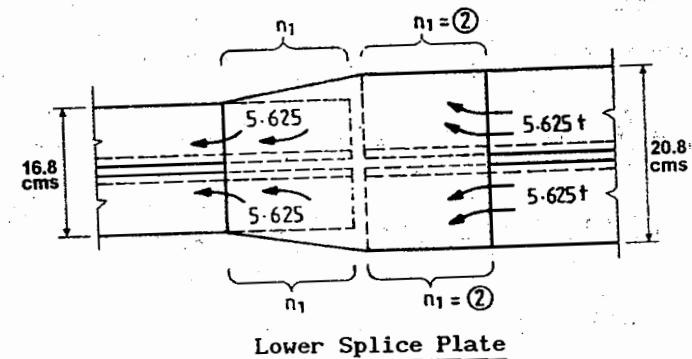
Assume that both the upper and the lower splice plates will transmit equally half of the largest force. Hence each splice plate will transmit:

$$F_{\text{upper splice}} = F_{\text{lower splice}} = \frac{22.5}{2} = 11.25 \text{ tons}$$

The upper splice connection being composed of two plates, hence each plate will transmit in single shear a force equal to $11.25/2 = 5.625$ tons as shown by the section s-s :



Upper Splice Plates (Section s-s)



Lower Splice Plate

$$R_{s.s} = 0.25 \times 4 \times \pi \frac{d^2}{4} = 3.14 \text{ tons}$$

$$R_b = 0.8 \times 3.6 \times 0.8 \times 2 = 4.608 \text{ tons} \quad (e_1 = 2 d)$$

$$n_1 = \text{number of bolts attaching each portion of splice plates} = \frac{5.625}{3.14} = 1.79 = 2.0 \text{ bolts}$$

Force transmitted directly from member (1) to the gusset plate = $\frac{45}{2} = 22.50$ tons

n_2 = number of bolts attached to gusset plate (member 1)



$$n_2 = \frac{22.50}{R_b} = \frac{22.50}{4.608} = 4.88 = 5 \text{ bolts}$$

Force transmitted directly from member (2) to gusset plate:

$$F = 35 - 45/2 = 12.50 \text{ tons}$$

n_3 = number of bolts attached to gusset plate (member 2)

$$n_3 = \frac{12.50}{R_b} = \frac{12.50}{4.608} = 2.7 = 3 \text{ bolts}$$

Thickness of Splice Plates:

Upper Splice Plates:

$$\text{Assume } b_1 = a - 2t_c = 8.0 - 2 \times 1 = 6.0 \text{ cms}$$

$$A_{\text{net}} = [6 - (d + 0.2)] t = (6 - 2.2) t = 3.8 t \text{ cm}^2$$

$$A_{\text{net}} = \frac{\text{force of splice plate}}{1.40} = \frac{5.62}{1.40} = 4.014 \text{ cm}^2$$

$$t_{\#1} = 4.014 / 3.8 = 1.05 \text{ cm} \quad \text{Take } 11 \text{ mms}$$

Lower Splice Plate:

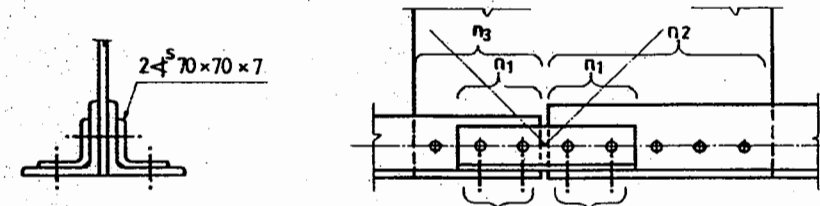
Take $b = 16.8 \text{ cms}$ while conservative.

$$A_{\text{net}} = [16.8 - 2(d + 0.2)] t = 12.4 t$$

$$A_{\text{net}} = \frac{\text{force of splice plate}}{1.40} = \frac{2 \times 5.62}{1.40} = 8.028 \text{ cm}^2$$

$$t_{\#2} = \frac{8.028}{12.4} = 0.64 \text{ cm} \quad \text{Take } 7 \text{ mms}$$

(2) Using Splice Angles: ($2 \angle^S 70 \times 70 \times 7$)



Assume that half of the largest force n_1 is transmitted through the splice angles. Hence the number of bolts required is:

$$n_1 = \frac{45 / 2}{4 \times 3.14} = 1.79 = 2.0 \text{ bolts}$$

number of plane of single shear $R_{s.s}$

$$n_2 = \text{number of bolts attached to gusset plate (Mem.1)} = \frac{45/2}{R_b} = 4.88 = 5 \text{ bolts}$$

$$n_3 = \text{number of bolts attached to gusset plate (Mem.2)} = \frac{12.50}{R_b} = 2.7 = 3 \text{ bolts}$$

$$F_{\text{through splice angle}} = 45 / 2 = 22.50 \text{ tons}$$

$$f_{t.a} = \frac{22.50}{2(9.40 - 0.7 \times 2.2)} = 1.43 \text{ t/cm}^2 \approx 1.4 \text{ (O.k.)}$$

For all cases the edge distance $e_1 = 2d$

Alternative (3) :-

If the edge distance (e_1) is taken (3d) the coefficient α regarding the bearing strength of the bolt will be :-



$$R_b = 1.2 \times F_u \times d \times t = 1.2 \times 3.6 \times 2.0 \times 0.8 = 6.912 \text{ tons}$$

$$R_{d.s} = 0.25 \times 4 \times 2 \times \frac{\pi d^2}{4} = 6.28 \text{ tons}$$

The shear strength governs the design.

$$\text{Number of bolts required for Member (1)} = \frac{45}{6.28} = 7.16$$

Take 8 bolts > 6 bolts unsafe

$$\text{Use the reduction factor } \beta_L = 1 - \frac{(L_i - 15d)}{200} \leq 1.0$$

$$\beta_L = 1 - \frac{(21d - 15d)}{200} = 0.97$$

$$\text{Where } L_i = 7 \text{ spacing} = 7 \times 3d = 21d$$

$$R_{d.s} = 0.97 \times 6.28 = 6.09 \text{ tons}$$

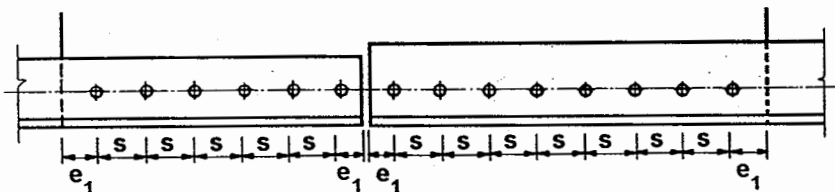
$$\text{Number of bolts of Member (1)} = 45 / 6.09 = 7.38 \text{ bolts.}$$

Hence use 8 bolts.

$$\text{Number of bolts required for Member (2)} = 35 / 6.28 = 5.57 \text{ bolts.}$$

= 6 bolts

(O.K.) no reduction

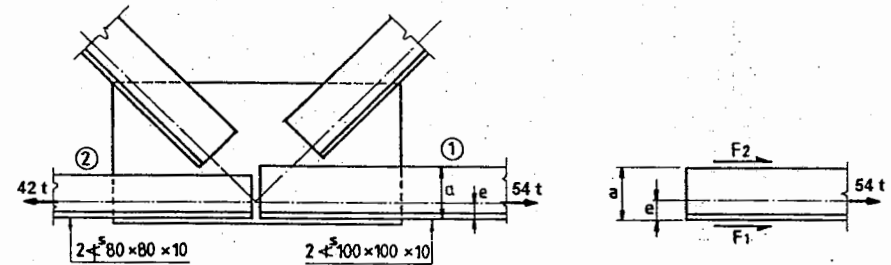


Example (7.3):

Design the shown fillet welded truss connection.

Gusset PL 8 mm.

Maximum $s = 8 \text{ mm}$.



Solution

Member (1):

$$F_1 = D.F \times \frac{(a-e)}{a} = 54 \times \frac{7.18}{10} = 38.77 \text{ tons.}$$

The maximum size "s" of weld will be the thickness of the gusset plate $s = 0.8 \text{ cms}$.

$$L_1 = \frac{F_1}{2 s_w \times q} = \frac{38.77}{2 \times 0.8 \times 0.72} = 33.65 \text{ cms}$$

$$< 70 s \text{ (O.K.)}$$

$$70 s = 70 \times 0.80 = 56 \text{ cms.}$$

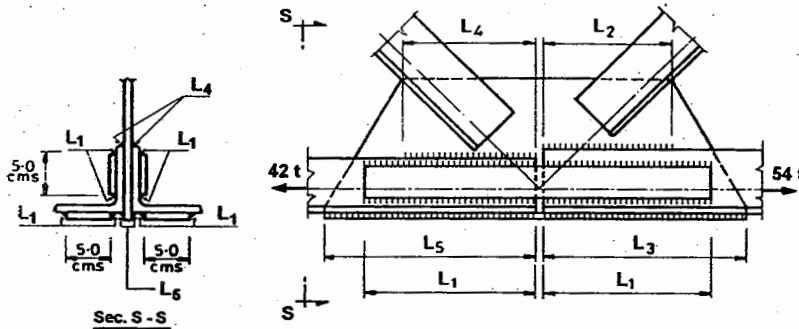
As the length of fillet weld to connect Member (1) to the gusset plate is $(33.65 + 2s) = 35.25 \text{ cms}$ will require a relatively huge gusset, one of the following two alternatives can be utilised.

Alternative (1) :-

Use 4 splice plates (welded).

Design using the D.F. as splice is not full.

Assume that half of the largest force is transmitted through the splice plates of width 5.0 cms.



$$\text{Force/4 splice plates} = \frac{54}{2} = 27.0 \text{ tons. (half the largest D.F.)}$$

$$\text{Force/one splice plate} = \frac{27.0}{4.0} = 6.75 \text{ tons.}$$

Assume thickness of splice plates = 10 mms. (as angles)
and breadth of splice plates = 5.0 cms.

$$\text{Stress in splice plate} = \frac{6.75}{b \times t} = \frac{6.75}{5 \times 1.0} = 1.35 \text{ t/cm}^2$$

Safe.

Assume size of fillet weld $s = 8$ mms.

$$L_1 = \frac{6.75}{2 \times 0.80 \times 0.72} + 2 \times 0.80 = 7.45 \text{ cms.}$$

Member (1) - Remaining force transmitted to the gusset plate :-

$$F = \frac{54}{2} = 27.0 \text{ tons.}$$

$$F_2 = \frac{54}{2} \times \frac{2.82}{10} = 7.614 \text{ tons.}$$

$$L_2 = \frac{7.614}{2 \times 0.80 \times 0.72} + 2 \times 0.80 = 8.20 \text{ cms.}$$

$$F_3 = \frac{54}{2} \times \frac{7.18}{10} = 19.38 \text{ tons.}$$



$$L_3 = \frac{19.38}{2 \times 0.8 \times 0.72} + 2 \times 0.80 = 18.40 \text{ cms.}$$

Member (2):

Force transmitted to gusset plate through weld lines:

$$F = 42 - \frac{54}{2} = 15.00 \text{ tons.}$$

$$F_4 = 15.00 \times \frac{2.34}{8} = 4.38 \text{ tons.}$$

$$L_4 = \frac{4.38}{2 \times 0.8 \times 0.72} + 2 \times 0.80 = 5.4 \text{ cms. (min)}$$

$$F_5 = 15.00 \times \frac{5.66}{8} = 10.61 \text{ tons.}$$

$$L_5 = \frac{10.61}{2 \times 0.8 \times 0.72} + 2 \times 0.8 = 10.81 \text{ cms.}$$

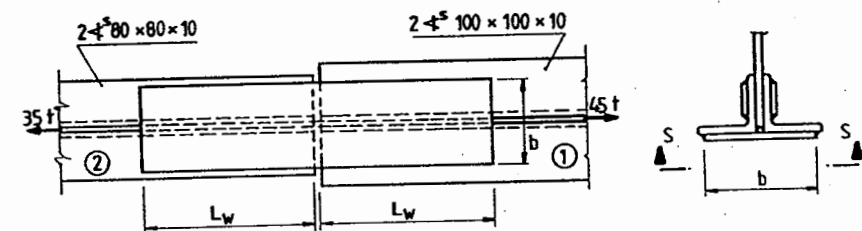
Alternative (2) :-

We can use one lower splice plate as shown:

Assume $b = 14.0$ cms.

$$t_{\neq} = \frac{6.75 \times 2}{14 \times 1.40} = 0.688 \text{ cm} \approx 0.70 \text{ cm.}$$

$$L_w = \frac{6.75 \times 2.0}{2 \times 0.80 \times 0.72} + 2 \times 0.80 = 13.31 \text{ cms.}$$





$$L_3 = \frac{19.38}{2 \times 0.4 \times 0.72} + 2 \times 0.4 = 35.24 \text{ cms (embedded).} < 70 \text{ s (O.K.)}$$

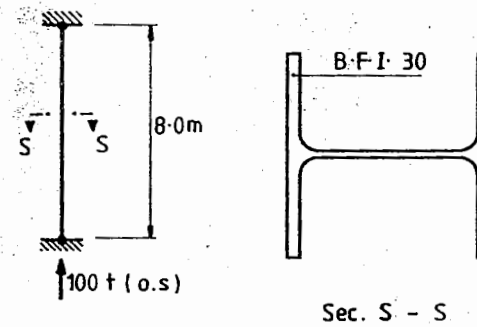
$$L_5 = \frac{10.61}{2 \times 0.4 \times 0.72} + 2 \times 0.4 = 19.74 \text{ cms (embedded).}$$

This solution is not recommended as the gusset plate will be relatively huge.

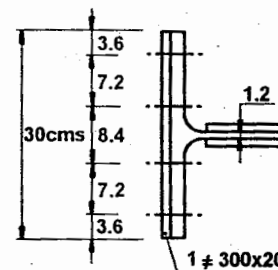


Example (7.4):

Design a field splice in the shown column using Non-pretensioned bolts Grade (4.6) M 20 mms. Given that; $N = 100 \text{ t}$ - Column St. 37.



Solution



$$A = 154 \text{ cms}^2 ; i_x = 12.9 \text{ cms} ; i_y = 7.65 \text{ cms}$$

$$l_x = l_y = 800 \text{ cms.}$$

$$\frac{l_y}{i_y} = \frac{800}{7.65} = 104.60 > 100$$

$$F_c = \frac{7500}{(104.60)^2} = 0.686 \text{ t/cm}^2.$$

$$f_c = \frac{N}{A} = \frac{100}{154} = 0.65 \text{ ton / cm}^2 < F_c \text{ (O.K.)}$$

The splice being a full splice, the design of the connection is based on maximum strength.

$$\text{Max. Strength} = F_c \times A_t = 0.686 \times 154 = 105.644 \text{ tons.}$$

$$A_f = 2 \times 30 \times 2 = 120 \text{ cm}^2. \quad A_w = 154 - 120 = 34 \text{ cm}^2.$$

$$\text{Force carried by web} = N \times \frac{A_w}{A_t} = 105.644 \times \frac{34}{154} = 23.32 \text{ tons.}$$

Force carried by flanges = $105.64 - 23.32 = 82.324$ tons.

Flange Splices:

$$F/\text{flange} = \frac{82.324}{2} = 41.16 \text{ tons. ;}$$

Use ordinary bolts $d = 20$ mms. (Grade 4.6)

$$R_{S.S} = 0.25 \times 4 \times \pi \frac{(2)^2}{4} = 3.14 \text{ tons.}$$

$$R_b = 0.8 \times 3.6 \times 2 \times 2 = 11.52 \text{ tons} \quad (e_1 = 2d)$$

$$\text{Number of bolts req.} = \frac{41.16}{3.14} \times 1.15 = 15.07 \text{ bolts}$$

Use 16.0 bolts

Use 4 rows with 4 bolts at each row .

$$t = \frac{41.16}{(30 - 4 \times 2.2) \times 1.4} = 1.38 \text{ cms}$$

Take $t = 20$ mm as flange column.

Web Splices:

$$R_{d.s} = 3.14 \times 2 = 6.28 \text{ tons.}$$

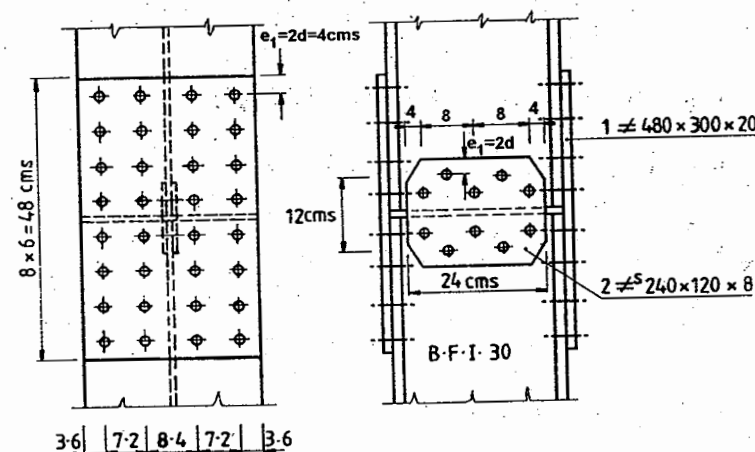
$$R_b = 0.8 \times 3.6 \times 2 \times 1.20 = 6.91 \text{ tons } (e_1 = 2d)$$

$$\text{Number of bolts required} = \frac{23.32}{6.28} \times 1.15 = 4.27 \text{ bolts.}$$

Use 5 bolts as shown in figure below.

$$t = \frac{23.32}{2 \times (24 - 3 \times 2.2) \times 1.4} = 0.47 \text{ cms (see figure below)}$$

Take 2 splice plates 240 x 6 mms.



**Example (7.5):**

Design the main girder (M.G) of a floor system given that;

- The floor is covered by R.C. slab of 10 cms thickness.
- The live loads are taken according to E.C.P. for residential buildings. Spacing of girders is 6.0 ms. (one way slab).

Design a field splice at 3.0 m from the support knowing that the girder span is 15.0 ms and is braced laterally at intervals of 5.0 ms.

Solution

$$W_{C.M} = \frac{10 \times 25 \times 6}{1000} = 1.50 \text{ t/m}'; \quad W_{L.L} = \frac{200 \times 6}{1000} = 1.20 \text{ t/m}'$$

Assume the own weight of beam = 225 kgm/m

$$W_t = 1.50 + 1.20 + 0.225 = 2.925 \text{ t/m}'$$

$$M_x = \frac{W_t \times L^2}{8} = \frac{2.925 \times 15^2}{8} = 82.26 \text{ mt.}$$

$$Q = \frac{W_t \times L}{2} = \frac{2.925 \times 15}{2} = 22.0 \text{ tons}$$

$$Z_{x \text{ req.}} = \frac{82.26 \times 100}{1.4} = 5876 \text{ cm}^3$$

Choose B.F.I. # 60, $Z_x = 6030 \text{ cm}^3$, $t_w = 1.7 \text{ cms}$, $t_f = 3.2 \text{ cms}$.

$$f_{bt} = f_{bc} = \frac{8226}{6030} = 1.36 \text{ t/cm}^2 < 1.40 \text{ t/cm}^2 \text{ (o.k.)}$$

$$q = 22 / 60 \times 1.7 = 0.215 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2 \text{ (o.k.)}$$

$$L_u = 20 b / \sqrt{F_y} = 20 \times 30 \sqrt{2.4} = 387 \text{ cms} < 500 \text{ cms}$$

$$L_u = (1380 A_f / d.F_y).C_b = (1380 \times 30 \times 3.2 / 60 \times 2.4) 1.0 = 920 > 500 \text{ cms}$$

The section is noncompact ($F_b = 1.4 \text{ t/cm}^2$)

**Straining Actions at Field Splice:**

$$M_x = 22 \times 3 - 2.925 \times \frac{3^2}{2} = 52.65 \text{ mt.}$$

$$Q_{act} = 22 - 3 \times 2.925 = 13.16 \text{ tons.}$$

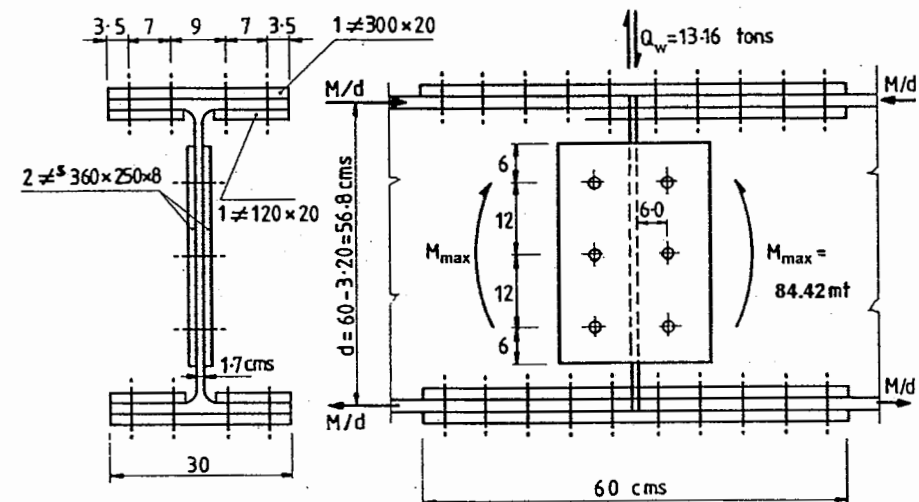
As the situation is a full splice connection, the design of the fasteners is based on the maximum moment strength and the actual shearing force.

$$M_{max} = \text{Max. Moment} = Z_x \cdot F_b = \frac{6030 \times 1.4}{100} = 84.42 \text{ t.ms.}$$

$$Q_{act} = \text{Actual shearing force} = 13.16 \text{ tons}$$

A. Approximate Analysis**1. Using H.S.B. in Field Splice M 20 mms Grade (10.9) -****Friction Type :-**

Assume that the moment " M_t " is totally carried by the flange splices and the shear by the web splices.





1.1. Flange Splices:

$$\text{Flange force due to moment} = \frac{8442}{(60 - 3.2)} = 148.62 \text{ tons.}$$

As H.S.B. of friction type are utilised use gross area

$$F_t = 1.4 = \frac{148.62}{(30 + 2 \times 12) t}; \quad t = 1.96 \text{ cms.}$$

Take 3 splice plate of thickness 2.0 cms each

one plate 30 x 2.0 cms ; two plate 12 x 2.0 cms

$$\begin{aligned} \text{Number of bolts} &= \frac{148.62}{P_s} = \frac{148.62}{2 \times 4.93} \times 1.15 \quad (\mu = 0.4 - \text{Case I}) \\ &= 17.33 \text{ bolts.} \approx 20 \text{ bolts.} \end{aligned}$$

Use 5 rows with 4 bolts / row. or 4 rows x 4 + 2 = 18 bolts

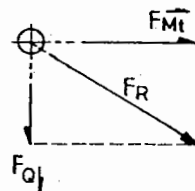
1.2. Web Splice:

$$\text{Moment due to eccentricity} = M_t = 13.16 \times 6 = 78.96 \text{ t.cm.}$$

$$Q_w = 13.16 \text{ tons.}$$

$$F_{\downarrow} / \text{bolt} = \frac{13.16}{3} = 4.40 \text{ tons}$$

$$\vec{F}_{M_t} / \text{bolt} = \frac{78.96 \times 12}{2 \times 12^2} = 3.29 \text{ tons.}$$



$$F_R = \sqrt{(4.40)^2 + (3.29)^2} = 5.49 \text{ tons.} < 2 \times 4.93.$$

Safe (double shear)

Assume thickness of web splice plates = 8 mms.

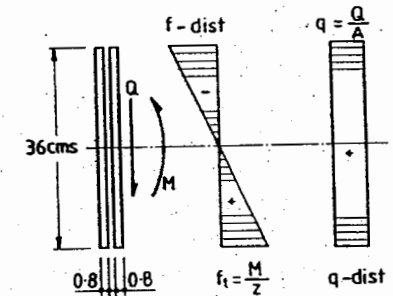


$$f_{bt} = \frac{78.96 \times 18}{2 \times 0.80 \times 36^3 / 12}$$

$$= 0.228 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (O.K.)}$$

$$q = \frac{13.16}{2 \times 0.8 \times 36} = 0.23 < 0.84 \text{ (O.K.)}$$

$$f_{eq} = \sqrt{(0.228)^2 + 3(0.23)^2} = 0.46 \text{ t/cm}^2 < 1.54 \text{ t/cm}^2 \text{ (Safe)}$$



2. Using Bolts of the non-pretensioned type in Field Splice M 20 mms - Grade (4.6).

Assuming that all the moment is totally carried by the flange splices and the shear by the web splices.

A double-plate is to be utilized to decrease the number of bolts.

2.1. Flange Splices:

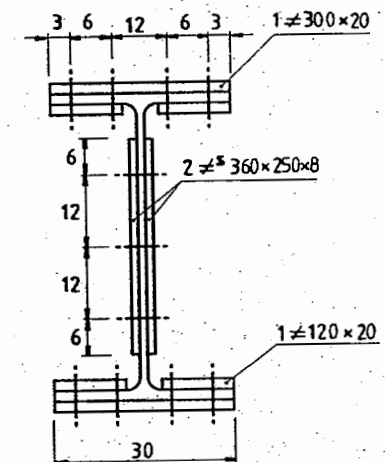
$$\begin{aligned} \text{Flange force} &= \frac{84.42 \times 100}{(60 - 3.2)} \\ &= 148.62 \text{ tons.} \end{aligned}$$

$$\begin{aligned} R_{d.sh.} &= 2 \times 0.25 \times 4 \times \pi \times \frac{(2)^2}{4} \\ &= 6.28 \text{ tons} \end{aligned}$$

$$\begin{aligned} R_b &= 0.8 \times 3.6 \times 2 \times 3.2 \\ &= 18.43 \text{ tons (} e_1 = 2d \text{)} \end{aligned}$$

$$\text{Number of bolts} = \frac{148.62}{6.28} \times 1.15 = 27.2 \text{ bolts}$$

Use 7 rows with 4 bolts / row = 28 bolts.





In each row $n = 7 > 6$; $\beta_L = 1 - \frac{(18d - 15d)}{200} = 0.985$

$$R_{d.s} = 0.985 \times 6.28 = 6.186 ; n = \frac{148.62}{6.186} \times 1.15 = 27.62, \text{ bolts}$$

Use 4 rows in each 7 bolts = 28 bolts.

No need to check the safety of tensile stresses at bolts section as the actual moment is 52.65 t.ms while the design moment is 84.42 t.ms. For the net section deduction of holes > 15 % of flange area is applied.

$$t_{\text{splice plates}} = \frac{148.62}{1.40 \left[1.15(30 + 2 \times 12) - 8 \times 2.2 \right]} = 2.38 \text{ cms}$$

$\swarrow$
 A_{net} of the 3 splices plates

$$t = 2.4 \text{ cms}$$

2.2. Web Splices:

Assume the same number and arrangements of bolts as in the H.S.B. concept.

$$F_R = 5.49 \text{ tons} < 6.28 \text{ tons (Safe)}$$

Use the same splice plates as in H.S.B.
(thickness = 8 mm).

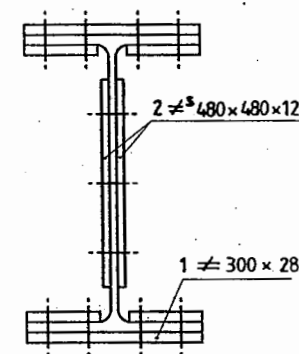
B. Exact Solution: Using H.S.B's M 20 mm Grade (10.9)

$$\text{Friction type - } P_s = 4.93 \text{ tons}$$

$$I_t = 180800 \text{ cm}^4 \text{ (B.F.I. No. 60)}$$

$$I_f = 2 \left[30 \times 3.2^3 / 12 + 30 \times 3.20 (30 - 1.6)^2 \right] = 155 \times 10^3 \text{ cm}^4$$

$$I_w = I_t - I_f = 25.8 \times 10^3 \text{ cm}^4$$



$$M_w = M_{\text{max}} \times \frac{I_w}{I_t} = \frac{84.42 \times 100 \times 25.8 \times 10^3}{180.8 \times 10^3}$$

$$= 1204.66 \text{ t.cms.}$$

$$M_f = M_{\text{max}} - M_w = 84.42 - 12.0466 = 72.373 \text{ t.ms.}$$

1. Flange Splice:

$$\text{Flange force} = \frac{72.373 \times 100}{(60 - 3.2)} = 127.41 \text{ tons.}$$

$$\text{Number of bolts} = \frac{127.41}{2 \times 4.93} \times 1.15 = 14.86 \text{ bolts}$$

→ take 16 bolts

Use 4 rows with 4 bolts at each row.

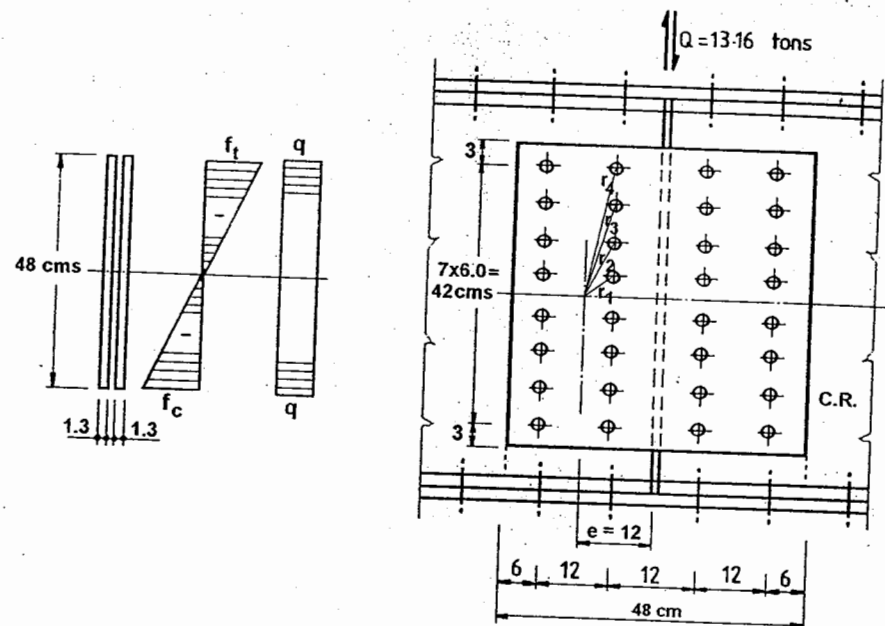
$$A_{\text{splice plates}} = 30 \times t + 2 \times 11 \times t = 52 t \text{ cm}^2$$

$$\text{Thickness } t = \frac{127.41}{52 \times 1.4} = 1.75 \text{ cms}$$

Take $t = 1.8 \text{ cms}$



2. Web Splice:



$$\sum \bar{r}_i^2 = 4 [(3)^2 + (9)^2 + (15)^2 + 4 \times (6)^2] = 3600 \text{ cm}^2$$

$$M_t = 1204.66 + 13.16 \times 12 = 1362.58 \text{ t.cm}$$

$$F_{\downarrow} / \text{bolt} = \frac{13.16}{8} = 1.645 \text{ tons. (shear)}$$

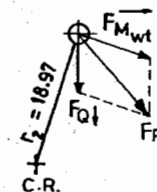
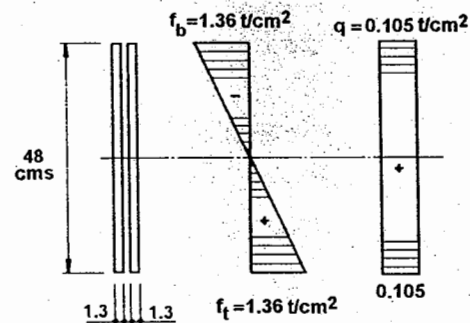
$$\vec{F}_{M_{wt}} / \text{bolt} = \frac{1362.58 \times 21}{3600} = 7.94 \text{ tons.}$$

$$F_{M_{wt}}^{\downarrow} / \text{bolt} = \frac{1362.58 \times 6.0}{3600} = 2.27 \text{ tons.}$$

$$F_R = 8.85 \text{ tons} < 2 \times 4.93 \text{ tons} \quad \text{O.K.}$$



Assume thickness of each splice web plate (12 mms).



$$f_t = \frac{1362.58 \times 24}{2 \times 1.2 \times 48^3 / 12} = 1.48 \text{ t/cm}^2 > 1.40 \text{ t/cm}^2.$$

Increase thickness $t = 13 \text{ mms}$

$$\therefore f_t = 1.36 \text{ t/cm}^2$$

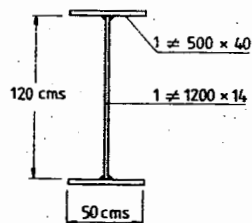
$$q = \frac{13.16}{2 \times 1.3 \times 48} = 0.105 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2.$$

$$f_{eq.} = \sqrt{(1.36)^2 + 3(0.105)^2} = 1.37 \text{ t/cm}^2$$

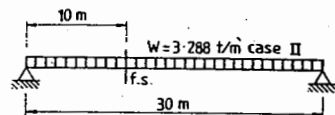
$$< 1.1 \times 1.40 \text{ t/cm}^2 \text{ (Safe)}$$

Example (7.6):

Design a field splice for the shown beam at 10 ms. from the support. The beam is carrying a uniformly distributed load of 3.288 t/m. Lateral supports at intervals 7.5 ms.



Beam X-sec.



Solution

$$M_x = \frac{3.288 \times (30)^2}{8} = 369.90 \text{ mt.}$$

$$I_x = \frac{1.4 \times (120)^3}{12} + 2 \times 50 \times 4 \times (62)^2 = 1739200 \text{ cm}^4$$

$$f_{bc} = f_{bt} = \frac{369.90 \times 100 \times 64}{1739200} = 1.36 \text{ t/cm}^2 < 1.4 \times 1.2 \text{ t/cm}^2 \text{ O.K.}$$

$$I_f = 2 \times 50 \times 4 \times (60 + 2)^2 = 153.76 \times 10^4 \text{ cm}^4$$

$$I_w = (173.92 - 153.76) 10^4 = 20.16 \times 10^4 \text{ cm}^4$$

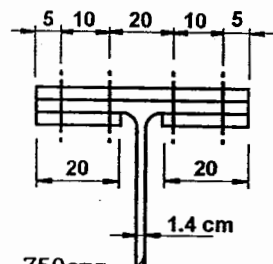
$$\frac{I_w}{I_{x_t}} = 0.116$$

$$\frac{I_f}{I_{x_t}} = 1 - 0.116 = 0.884.$$

$$L_u = 20 b / \sqrt{F_y} = 20 \times 50 \sqrt{2.4} = 645 \text{ cms} < 750 \text{ cms}$$

$$L_u = (1380 A_f / d.F_y).C_b = (1380 \times 50 \times 4 / 128 \times 2.4) \times 1.0 = 898 > 750 \text{ cms}$$

The section is noncompact.



1. At the Position of the Field Splice:

$$M_x = 3.288 \times 15 \times 10 - 3.288 \times \frac{(10)^2}{2} = 328.8 \text{ t.m.}$$

$$Q = 3.288 \times (15 - 3) = 39.456 \text{ tons.}$$

2. Flange Splices:

As the splice is a full splice the design is based on the maximum moment strength of the section and the actual shearing force.

$$\text{Max. Moment Strength} = \frac{1739.2 \times 10^3}{64} \times 1.4 \times 1.2 = 456.54 \text{ t.m.}$$

$$M_f (\text{Flanges}) = 0.884 \times 456.54 = 403.58 \text{ t.m.}$$

$$\text{Flange force} = \frac{403.58 \times 100}{124} = 325.46 \text{ tons}$$

Number of H.S.B. (10.9) - M22 - Friction type - $P_s = 6.615 \text{ tons}$

$$n = \frac{325.46}{2 \times 6.615} \times 1.15 = 28.29 \text{ bolts} - \text{take } 30 \text{ bolts}$$

Use 4 rows with 7 bolts / row + 2 bolts.

Maximum force carried by flange is to be calculated based on the gross area as we are using H.S.B of the friction type.

$$\begin{aligned} \text{Max. force of flange} &= 50 \times 4 \times 1.4 \times 1.2 \\ &= 336 \text{ tons.} > 325.46 \text{ tons} \quad (\text{O.K.}) \end{aligned}$$

Thickness of Splice Plates = t ($50 \times t + 2 \times 20 \times t$)

$$t = \frac{325.46}{(50 + 2 \times 20) \times 1.4 \times 1.2} = 2.15 \text{ cms} ; \text{ take } t = 2.2 \text{ cms.}$$



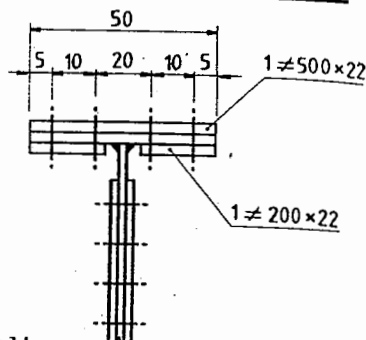
$$\beta_L = 1 - \frac{(L_i - 15d)}{200} \leq 1.0$$

$$\beta_L = 1 - \frac{(18d - 15d)}{200} = 0.985$$

$$P_s = 0.985 \times 6.615 = 6.515$$

$$n = \frac{325.46}{2 \times 6.515} \times 1.15 = 28.7 \text{ bolts} < 30 \text{ bolts}$$

O.K.



3. Web Splice:

Use two columns of bolts with 9 bolts at each column.

$$M_{wt} = 0.116 \times 456.54 \times 100 + 39.456 \times 12 = 5769.33 \text{ t.cms.}$$

$$\sum \bar{r}_i^2 = 4 [(12)^2 + (24)^2 + (36)^2 + (48)^2 + 5 \times (6)^2]$$

$$= 18000 \text{ cm}^2$$

$$F_{M_t} = 15.38 \text{ tons.}$$

$$F_{M_t} = 1.923 \text{ tons.}$$

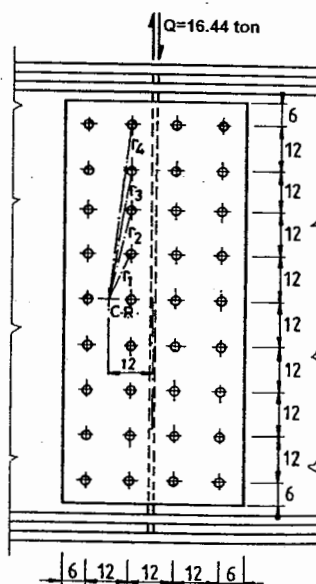
$$F(Q) = \frac{39.456}{9 \times 2} = 2.19 \text{ tons.}$$

$$F_R = 15.92 \text{ tons.} > 2 \times 6.93 \text{ tons.}$$

unsafe

Increase number of H.S.B to 10 / column.

Assume Thickness of Splice Plates = 8 mms.



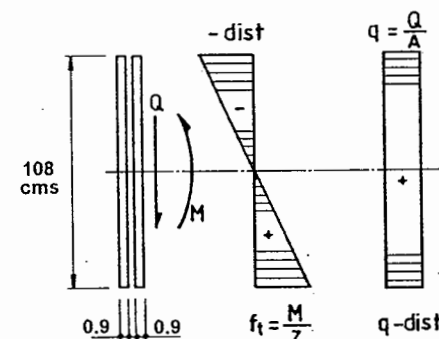
$$f_b = \frac{5769.33 \times 6}{2 \times 0.8 \times (108)^3} = 1.85 > 1.4 \times 1.2 \quad \text{unsafe}$$

$$\text{Use thickness} = 9 \text{ mms} \quad - \quad f_{bc} = 1.605 \text{ t/cm}^2$$

$$q = \frac{39.456}{2 \times 0.9 \times 108} = 0.202 \text{ t/cm}^2$$

$$f_{eq} = \sqrt{(1.605)^2 + 3(0.202)^2}$$

$$= 1.64 \text{ t/cm}^2 < 1.1 \times 1.2 \times 1.40 \text{ t/cm}^2. \quad (\text{O.K.})$$



**Example (7.7): Beam-Column Splice**

(a) Design the beam column (AB) of the gable frame shown, given that the frame is subjected to the following loads:

- Covering material corrugated sheets of intensity 15 kg/m^2
- Own weight of steel 40 kg/m^2 of covered area.
- Live load is to be taken according to the E.C.P. (2001).
- Neglect the effect of wind.

(b) Design a field splice in the column (AB) at a distance 4.0 ms from the base.

Solution

(1) Loads:

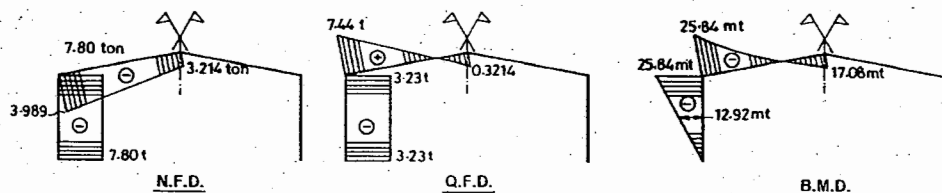
$$I_{D.L.} = \frac{15}{\cos \alpha} + 40 = 55.075 \text{ kg/m}^2$$

$$W_{D.L.} = \frac{55.075 \times 6}{1000} = 0.33 \text{ ton/m'}$$

$$I_{L.L.} = 53.33 \text{ kg/m}^2 \quad (\text{E.C.P 99}) \quad (\tan \alpha = 0.10)$$

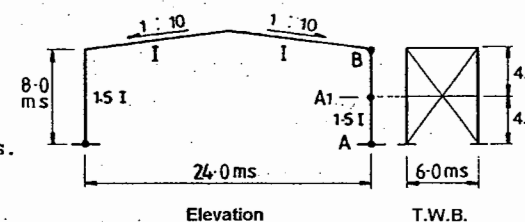
$$W_{L.L.} = \frac{53.33 \times 6}{1000} = 0.32 \text{ ton/m'}$$

$$W_{\text{total}} = W_{D.L.} + W_{L.L.} = 0.65 \text{ t/m'}$$

(2) Straining Actions: Due to Dead Load and Live Load Combined**(3) Design of the Beam Column (AB):****1. Straining Actions:**

$$M_{\text{max.}} = 25.84 \text{ mt.}$$

$$N_{\text{corr.}} = 7.80 \text{ tons.}$$

**2. Preliminary Design:**

$$\text{Assume that } f_{bx} = 1.20 \text{ t/cm}^2.$$

$$Z_{x \text{ req.}} = \frac{25.84 \times 100}{1.20} = 2153.3 \text{ cm}^3.$$

Choose B.F.I.B. No. (34)

3. Actual Stresses:

$$G_A = 10 \text{ (Hinged)} \quad G_B = \frac{1.50 I / 8}{I / 24.12} = 4.523 \quad (I_c = 1.50 I_g)$$

From the charts get $k = 2.50$

$$l_x = 2.50 \times 8 = 20.0 \text{ ms.} \quad l_y = 4.00 \text{ ms.} \quad (\text{Transversal Bracing})$$

$$\frac{l_x}{i_x} = 137.93 \quad \frac{l_y}{i_y} = 52.9 \quad \therefore f_c = f_{Ex} = 0.3942$$

$$f_{ca} = \frac{7.80}{174} = 0.045 \quad f_{bx} = \frac{25.84 \times 100}{2170} = 1.1908 \text{ t/cm}^2$$

4. Equivalency and Amplification Factor:

$$C_M = 0.85 \quad (\text{case of sides way permitted})$$

$$A = \frac{1}{1 - \frac{0.045}{0.3942}} = 1.1288$$



$$A_1 = C_M \cdot A = 0.85 \times 1.1288$$

$$= 0.960$$

5. Lateral Torsional Buckling:

Part (A₁-B):

$$M_{A1} = -12.92 \text{ mt}$$

$$M_B = -25.84 \text{ mt}$$

$$\alpha = \frac{M_{A1}}{M_B} = -0.5 \quad (-\text{ve used for the same curvature})$$

$$C_b = 1.75 - 1.05 \alpha + 0.3 \alpha^2 = 1.3$$

$$L_u = 20 b / \sqrt{F_y} = 20 \times 30 \sqrt{2.4} = 387.3 \text{ cms} < 400 \text{ cms}$$

The section is noncompact.

$$F_{L.T.B.} (A1-B) = \frac{800 \times 30 \times 2.2}{400 \times 34} \times 1.3 = 5.04 \text{ t/cm}^2$$

$$> 1.4 \text{ O.K.}$$

Part (A1-A):

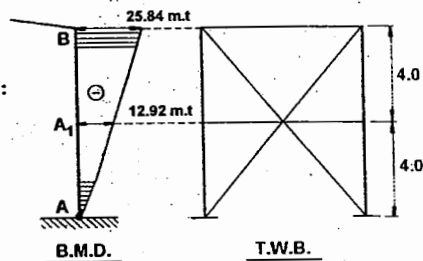
$$M_{A1} = -12.92 \text{ mt} \quad M_A = 0.0$$

$$\alpha = \frac{M_A}{M_{A1}} = 0.0 \quad C_b = 1.75 > 1.3 \text{ (no need to continue)}$$

6. Check on Stresses:

$$\frac{f_{ca}}{F_c} = \frac{0.045}{0.394} = 0.1142 < 0.15 \quad ; \text{ Take } A1 = 1.00$$

$$\frac{f_{ca}}{F_c} + \frac{f_{bx} \times 1.0}{F_{bc}} \leq 1.00$$



$$\frac{0.045}{0.394} + \frac{1.190 \times 1.0}{1.40} = 0.964 < 1.0$$

(4) Design of the Field Splice:

1. Straining Actions at the Field Splice:

$$M = 12.92 \text{ mt} \quad N = 7.80 \text{ tons}$$

$$Q = 3.23 \text{ tons.}$$

Use bolts of the pretensioned type

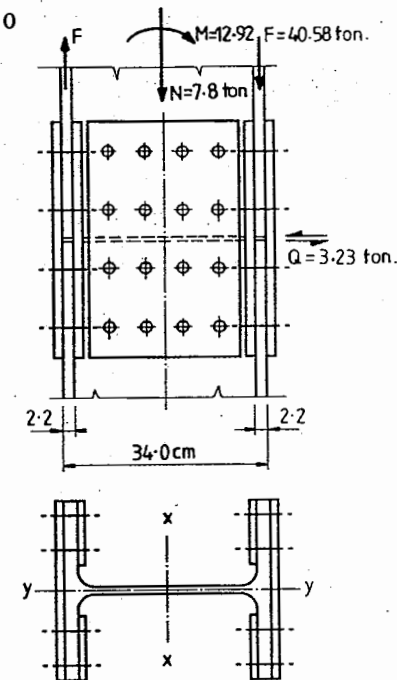
Grade (4.6) - M 20

$$R_{d.sh.} = 2 \times 0.25 \times 4 \times \Pi \frac{(2)^2}{4}$$

$$= 6.28 \text{ tons.}$$

$$R_b = 0.6 \times 3.6 \times 2.2 \times 2$$

$$= 9.504 \text{ tons}$$



Flange Splices:

The splice is to be designed on the full maximum moment, the actual shearing force and the actual normal force. While conservative because the corresponding stresses will exceed the allowable stresses ($0.58 F_y$), the design will be on the safe side.

$$\text{Max. Moment Strength} = Z_x \times 1.4 = 2170 \times 1.4 = 3038 \text{ t.cm.}$$

$$I_f = 2 \times 30 \times 2.2 \times (17 - 1.1)^2 = 33.37 \times 10^3 \text{ cm}^4$$

$$I_w = I_t - I_f = (36.94 - 33.37) \times 10^3 = 3.57 \times 10^3 \text{ cm}^4$$

$$M_f = \frac{3038 \times 33.37 \times 10^3}{36.94 \times 10^4} = 2744.39 \text{ t.cm.}$$



$$M_w = 3038 - 2744.39 = 293.6 \text{ t.cm.}$$

$$A_f = 2 \times 30 \times 2.2 = 132 \text{ cm}^2$$

$$A_w = 174 - 132 = 42 \text{ cm}^2$$

$$A_f/A_t = 0.759 \quad A_w/A_t = 0.241$$

$$N_f = 7.8 \times 0.759 = 5.92 \text{ tons}$$

$$N_w = 1.88 \text{ tons}$$

$$\text{Normal force transmitted by flanges} = \frac{2744.39}{34 - 2.2} + \frac{5.92}{2} = 89.26 \text{ tons}$$

$$\text{Number of bolts} = \frac{89.26}{6.28} \times 1.15 = 16.34 \text{ bolts} \quad \text{take 18 bolts}$$

Use 4 rows x 4 bolts per row + 2 bolts

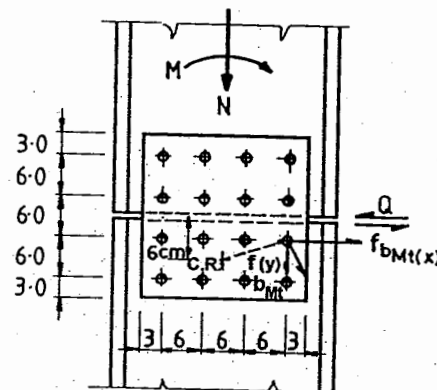
Web Splices:

Try two rows of 4 bolts at 60 mm. horizontal pitch and 60 mm. vertical pitch as shown in Fig.

Moment due to shear eccentricity = $3.23 \times 6 = 19.38 \text{ t.cm}$

$$\begin{aligned} \text{Total Moment carried by web splice} &= 293.6 + 19.38 \\ &= 312.98 \text{ cm.t} \end{aligned}$$

Normal force transmitted to web = 1.88 tons

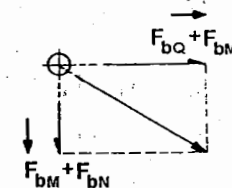


$$F_{b \downarrow N} = \frac{1.88}{2 \times 4.0} = 0.235 \text{ tons.}$$

$$F_{b \downarrow M_t} = \frac{312.98 \times 9}{4 \times (4.24^2 + 9.49^2)} = 6.5 \text{ tons.}$$

$$F_{b \rightarrow M_t} = \frac{312.98 \times 3}{4 (4.24^2 + (9.49^2))} = 2.17 \text{ tons.}$$

$$F_{b \rightarrow Q} = \frac{3.23}{2 \times 4.0} = 0.404 \text{ tons.}$$



$$\begin{aligned} F_{\text{bolt (resultant)}} &= \sqrt{(0.235 + 6.5)^2 + (2.17 + 0.404)^2} \\ &= 7.3 \text{ tons.} > R_{d.s} \quad \text{unsafe} \end{aligned}$$

Use M20 - Grade (5.8)

$$R_{d.s} = 2 \times 0.25 \times 5 \times 3.14 \times 2^2 / 4 = 7.85 \text{ tons} \quad \text{O.K.}$$

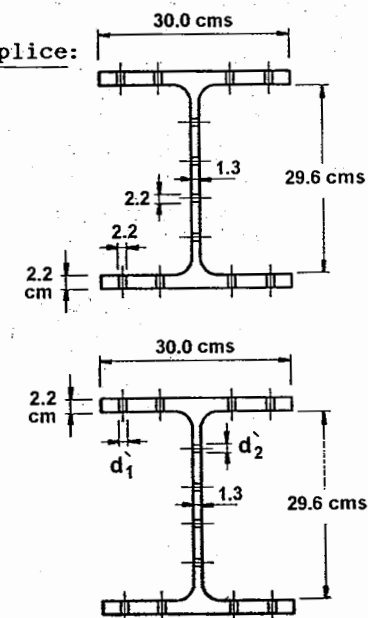
Check on the Section of the Column At Splice:

1st Solution (Conservative) :-

$$\begin{aligned} A_{\text{net}} &= 174 - 8 \times 2.2 \times 2.20 - 4 \times 2.2 \times 1.30 \\ &= 123.84 \text{ cm}^2. \end{aligned}$$

$$\begin{aligned} I_{x \text{ net}} &= 36940 - 8 \times 2.2 \times 2.2 \times (15.9)^2 \\ &\quad - 2 \times 2.2 \times 1.30 \times (9)^2 - 2 \times 2.2 \times \\ &\quad 1.3 \times (3)^2 = 26636.397 \text{ cm}^4. \end{aligned}$$

$$f_{ca} = \frac{N}{A_{\text{net}}} = \frac{7.80}{123.84} = 0.063 \text{ t/cm}^2$$





$$f_{bx} = \frac{M_{x,y}}{I_{x,net}} = \frac{12.92 \times 100 \times 17}{26636.397} = 0.825 \text{ t/cm}^2$$

$$\frac{f_{ca}}{F_c} + \frac{f_{bx} \cdot C_M \cdot A}{F_{bc}} = 0.1599 + \frac{0.825 \times 1.0}{1.40} = 0.749 < 1.00 \text{ Safe}$$

2nd Solution (Allowed) :-

$$A_{holes} = 8 \times 2.2 \times 2.2 + 4 \times 2.2 \times 1.30 = 50.16 \text{ cm}^2$$

$$A_{gross} = 174 \text{ cm}^2 ; \quad \frac{A_{holes}}{A_{gross}} = 28.8 \% > 15 \%$$

The reduction of holes from the section area and inertia should be done for that exceed only 15% of area.

$$A_{net} = A_{gross} - A_{holes}$$

$$= 174 \times 1.15 - 50.16 = 149.94 \text{ cm}^2$$

Reduced hole diameter for $I_{x,net}$:

$$\text{Also } I_{x,holes} / I_x = 10304 / 36940 = 0.279 \cong 0.28 > 0.15$$

$$I_{x,net} = (1.15 - 0.28) \times 36940 = 32137 \text{ cm}^4$$

$$f_{ca} = \frac{7.80}{149.94} = 0.052 \text{ t/cm}^2$$

$$f_{bx} = \frac{12.92 \times 100 \times 17}{32137} = 0.683 \text{ t/cm}^2$$

$$\frac{f_{ca}}{F_c} + \frac{f_{bx} \cdot C_M \cdot A}{F_{bc}} = 0.132 + \frac{0.638 \times 1.0}{1.40} = 0.62 < 1.00 \text{ (Safe)}$$

Hence the section of the beam column at the field splice is safe against Combined Normal Stresses.



Splice Plates:

1- Flange Splices:

Try 1 = 300 x 12 for outer face of flange.

Try 2 =^S 120 x 14 for inner face of flange.

The area and inertia of splice plates must replace that of the section.

$$\text{Area of splice plates} = 30 \times 1.20 + 2 \times 12 \times 1.40 = 69.60 \text{ cm}^2$$

$$\begin{aligned} \text{Inertia about (x-x) of splice plates} &= 2 \times 30 \times 1.20 \times (17.6)^2 \\ &+ 4 \times 12 \times 1.4 \times (14.1)^2 = 35662.72 \text{ cm}^4 \end{aligned}$$

$$\text{Area of flanges replaced} = 30 \times 2.2 = 66 \text{ cm}^2 < 69.6 \text{ cm}^2 \text{ (O.k)}$$

$$\begin{aligned} \text{Inertia about (x-x) of flanges} &= 2 \times 30 \times 2.2 \times (15.9)^2 \\ &= 33370.9 \text{ cm}^4 < 35662.72 \text{ cm}^4 \text{ (O.k)} \end{aligned}$$

2- Web Splices:

Try 2 =^S 240 x 10 for web splices.

$$\text{Area of splice plates} = 24 \times 2 \times 1.0 = 48.0 \text{ cm}^2$$

$$\begin{aligned} \text{Inertia about (x-x) of splice plates} &= 2 \times (24)^3 \times 1.0 / 12 \\ &= 2304 \text{ cm}^4 \end{aligned}$$

$$\text{Area of web replaced} = 29.60 \times 1.30 = 38.48 \text{ cm}^2 < 48.0 \text{ cm}^2 \text{ Safe}$$

$$\begin{aligned} \text{Inertia of web replaced} &= (29.6)^3 \times 1.30 / 12 = 2809.55 \text{ cm}^4 \\ &> 2304 \text{ cm}^4 \text{ unsafe} \end{aligned}$$

Increase thickness of web splice plates to 12 mm.

N.B. This procedure has been utilized to overcome the effect of the normal force.

CHAPTER 8



CHAPTER 8

CRANE GIRDERS AND BRACKET CONNECTIONS

8.1. Introduction:

In industrial buildings such as factories, workshops, etc. crane track girders are provided to carry overhead cranes. This allows to move heavy items from one point to another during assembly, fabrication, plant maintenance. Generally these cranes provide the heavy lifting facility in any direction over the whole area of the building and are operated either by hand or electrically.

The crane is usually supported on four wheels running on special rails as illustrated in Fig. 8.1. Each of these rails is supported on a crane track girder whose design is a part of the steelwork designer goal.

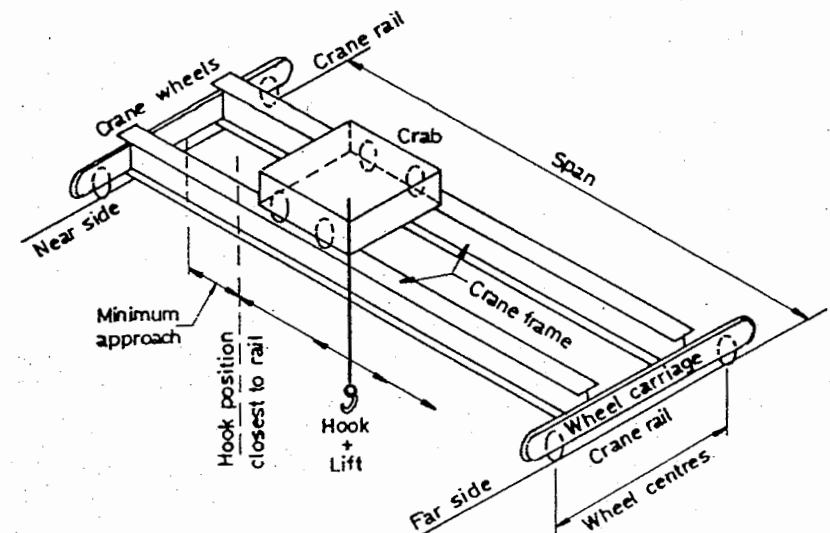


Fig. 8.1. Crane Girder Structural Components

In order to design a suitable steel crane track girder, the crane wheel loads and dimensions should be obtained from the manufacturers handbook. However the following data are required to choose the appropriate section:

(a) Span, crane capacity, weight of crab, weight of crane and hook load.

(b) Minimum hook approach closed to rails, minimum approach (i.e. end clearance) and wheel centers.

Once these data are obtained, the different applied loads are combined and hence the crane wheel loads computed.

Alternatively the crane wheel loads may be given directly by the crane manufacturer.

8.2. Crane Wheel Loads (Refer to Fig. 8.2):

(a) Vertical Static Wheel Loads:

The crane track girders (crane girders) are subjected to vertical loads from the weight of the crab and the hook load which may occupy any position on the crane frame up to the minimum hook approach as shown in fig. 8.1. Hence the vertical load on the near side pair of wheels can be computed, adding half the weight of the crane frame which is divided equally between the wheels.

(b) Dynamic Effect (Impact):

The maximum static wheel loads according to E.C.P. 2001 shall be increased by the following to include the dynamic effect (impact):

- For electric overhead cranes 25%.
- For hand operated cranes 10%.

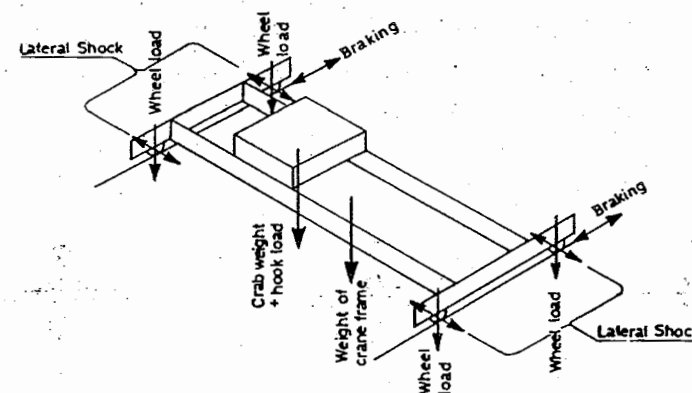


Fig. 8.2. Crane Loads

(c) Horizontal Loads Acting Transverse to the Rails:

In addition to the vertical loads transferred from the wheel loads to the rails, horizontal loads can develop. The first of these is called the lateral shock (or the surge) and acts normal to the rails. This lateral shock force covers the acceleration and braking of the crab when moving along the crane frame, together with the effect of the non-vertical lifting.

The value of this load is prescribed by 10% of the maximum reaction of the wheel.

(d) The Braking Loads:

The second horizontal load (longitudinal) is the braking load of the whole crane, and acts along the crane track girder at the level of the top flange.

According to the prescriptions of the E.C.P. 2001, the braking or tractive force shall be taken equal to $1/7$ of all wheel loads in the most unfavorable position and shall be

assumed acting on all the affected parts of the supporting structure (i.e. the braking force shall be equally distributed on the total number of vertical columns comprising the steel structure).

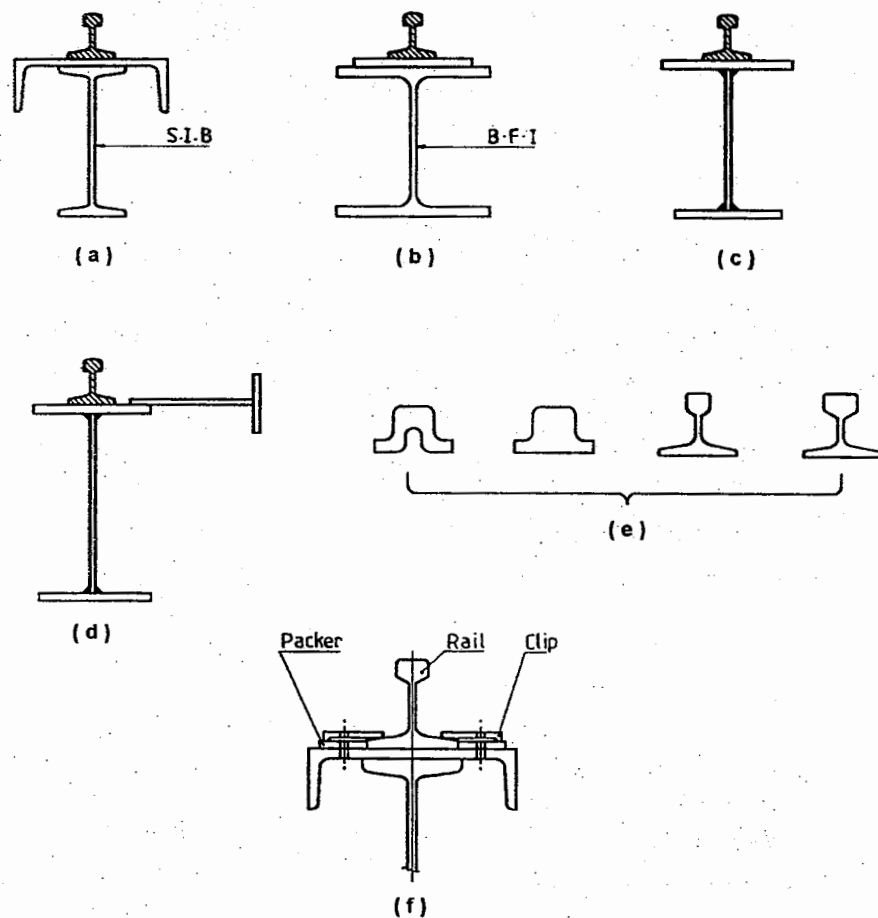


Fig. 8.3

8.3. Different Types of Crane Girders:

The crane girders are subjected to vertical as well as horizontal loads as has been previously outlined in Section 8.2. Hence for light crane capacities (say up to 20 tons capacity) the section may consist either of a S.I.B and a channel as shown in Fig. 8.3.a or a section of large flange (B.F.I) with or without an additional top plate as illustrated in Fig. 8.3.b. A built-up-section may be utilized as shown in Fig. 8.3.c. For heavy cranes a built-up-section with a surge girder acting as a walkway is illustrated in Fig. 8.3.d provided that the local buckling is prevented. Box sections are also used for heavy cranes.

Regarding the rail sections, different types are shown in Fig. 8.3.e. These are not taken into account in computing the crane girder strength. The size of the rails depends on the crane capacity. A connection arrangement of a rail is illustrated in Fig. 8.3.f.

Refer to Example 8.1.

8.4. Analysis of Welded and Bolted Minor Axis Brackets :

Fig. 8.4.a and b illustrate the common configurations of minor axis welded and bolted brackets respectively.

The design of the different steel structural components is a straight forward matter if the transmission of the applied loads from one element to the other is well determined. Fig. 8.4.c, illustrates the path of forces where the lateral shock load (surge) is transmitted to the beam column via a lateral plate support. On the other hand the vertical crane loads as well as the braking force are transmitted to the beam-column via the bracket gusset plates and the corresponding fasteners.

Welded connections are generally used for shop attachment while the bolted brackets for field attachment. This type of brackets don't present any difficulty. Where the loads are transmitted directly to the column flanges which provide the minor axis bending strength of the column.

Fig. 8.4.d, illustrates the stress distribution on the diagonal cross-section (s_1-s_1) of the gusset plate only. Regarding the buckling of the gusset plate, the ratio (a/t) should be based on the distance from the bolts to the free edge, while if welded, the corresponding (a) distance should only be based from the column near edge. The buckling condition of the triangular stiffened seat bracket previously outlined in Section 4.2.9 can be utilized. The following modification may be applied.

The average stress on the top loaded edge of the gusset plate, which would cause buckling of this gusset plate is given by the following critical buckling stress F_{cr} :-

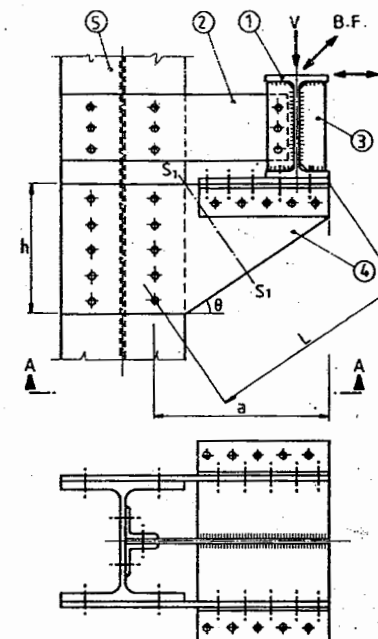
$$F_{cr} = K \frac{\pi^2 E}{12 (1 - \mu^2)} \left(\frac{t}{a}\right) \quad \dots \dots \dots .8.1$$

where: $\pi = 3.14$; $E = 2100 \text{ t/cm}^2$; $\mu = 0.3$

K = the buckling coefficient.

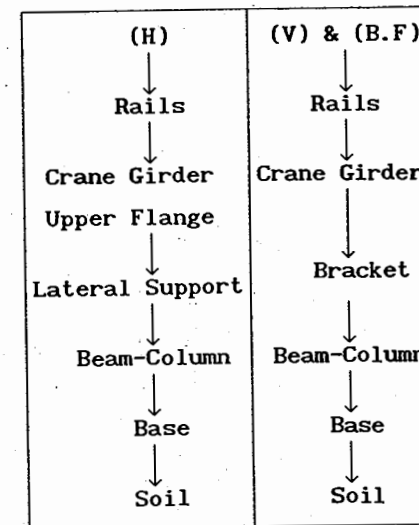
The buckling coefficient " K " can be computed using the laboratory report No. 319.1 of Lehigh University entitled "Buckling and Ultimate loads for plate girder web plates under edge loading" where the following relation has been recommended:

$$K = 3.2 - 3.0 \left(\frac{a}{h}\right) + 1.1 \left(\frac{a}{h}\right)^2 \quad \dots \dots \dots .8.2$$

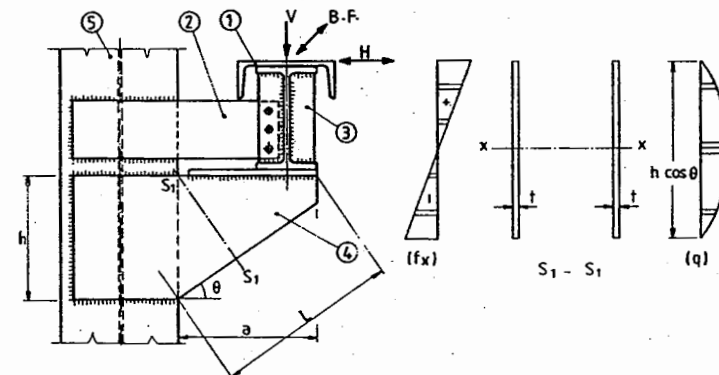


(a) Bolted Bracket

- (1) Crane girder cross section
- (2) Lateral support.
- (3) Support stiffener.
- (4) Bracket gusset plate.
- (5) Beam-column.



(c) Path of Forces-Flow Chart



(b) Welded Bracket
 ○ V = Vertical Crane Loads
 ○ B.F = Braking Force.

(d) Stress Distribution
 ○ H = Lateral Shock (Surge)

Fig. 8.4 Minor Axis Bracket Connections.



Using a factor of safety (F.S = 1.714), hence the induced average stress on the loaded edge is:

$$f_{av} = \frac{V}{a.t} \times 1.714 \quad \dots \dots \dots 8.3$$

Equating (8.1) and (8.3) with the corresponding values of E, Π and μ we get the following buckling condition:

$$t \cong 0.1 \sqrt[3]{\frac{V.a}{k}} \quad \dots \dots \dots 8.4$$

It is to be noticed that the analysis given above is valid only if the loading can be assumed to be distributed over the top edge of the gusset plate. The applied load (V) must be closer as possible to the column flange. When the applied force moves excessively toward the edge, the above analysis becomes qualitative. Hence the approximate procedure (Method B) of Section 8.5 can be utilized to determine the safety of the gusset plate against buckling.

Figs 8.4.a, b show suitable top flanges which prevent the lateral displacement of the bracket tips (i.e. the lateral torsional buckling phenomena). For this situation the corresponding effective lateral unsupported length is (L). In the absence of any top flange, the effective unsupported length to be adopted is 2 L.

Regarding the lateral support and the bracket connections the conventional fatigue calculations must be utilized as will be given in Section 8.10.

Refer to Example 8.2.



8.5. Analysis of Welded Major Axis Column Brackets:

Fig. 8.5, illustrates a welded bracket composed of a Tee-section where the flange is generally butt welded to the column flange while the Tee-stem is fillet welded. Another configuration consists to use an upper flange and a gusset plate attached together by horizontal fillet weld lines.

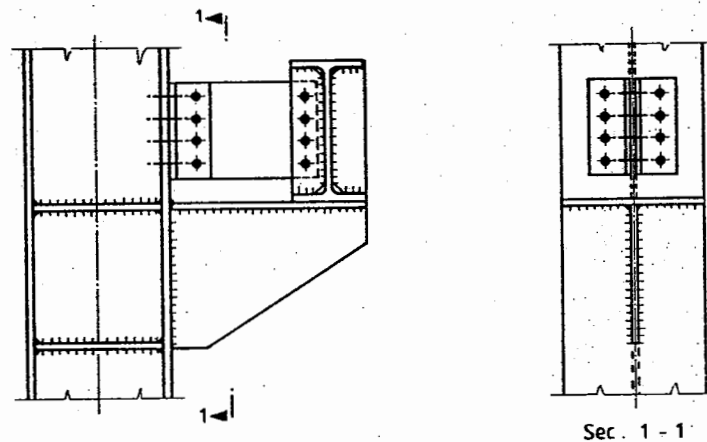


Fig. 8.5

Regarding the analysis, the welded bracket shown in Fig. 8.5 can be visualized as a triangular bracket whose exact analysis has been previously outlined in section 4.2.9. Regarding the buckling of the gusset plate Equation 8.4 can be qualitatively utilized.

However two different approximate analysis can be adopted, both are based on the beam stress theory. However the procedure is as given below:

(a) Approximate Beam Analysis for Stresses «Method A» :

In this method it is assumed that the vertical wheel loads (V) is transmitted to the column flange as follows:

In order to ensure a direct transmission of the applied wheel load (V) it is of common practice to use vertical stiffeners just below the crane girder fitted to the top flange and fillet welded to the gusset plate.

(b) Approximate Beam Analysis «Method B»:

In this method, the combined stress at the free edge on the inclined critical section $S_1 - S_1$ is computed according to the general stress equation. Referring to Fig. 8.7 the maximum induced compressive stresses are as follows:

$$f_{\text{Max}} = f_N + f_b$$

$$f_{\text{Max}} = \frac{V \cos \theta}{t \cdot a \sin \theta} + \frac{6 V \cos \theta \cdot e \sin \theta}{t \cdot (a \sin \theta)^2} + \frac{6 V \sin \theta \cdot (a/2 + e) \cos \theta}{(a \sin \theta) t^2}$$

$$f_{\text{Max}} = \frac{V}{t \cdot a} \cot \theta + \frac{6 V \cdot e}{t \cdot a^2} \cot \theta + \frac{6 V (a/2 + e)}{a \cdot t^2} \cos \theta$$

$$f_{\text{Max}} = \frac{V}{t \cdot a} \cot \theta \left(1 + \frac{6e}{a} + \frac{6 (a/2 + e)}{t} \sin \theta \right)$$

8.6

These maximum compressive stresses must not exceed the prescribed allowable limits.

Regarding the shear stresses, the vertical gusset plate section under the wheel load (V) is of maximum shear stresses and is parabolically distributed.

On the other hand the gusset plate buckling is prevented if either the stability condition of equation 8.4 is utilized or a strip of width $(a \sin \theta / 4)$ is assumed to act as a column with pin ends.

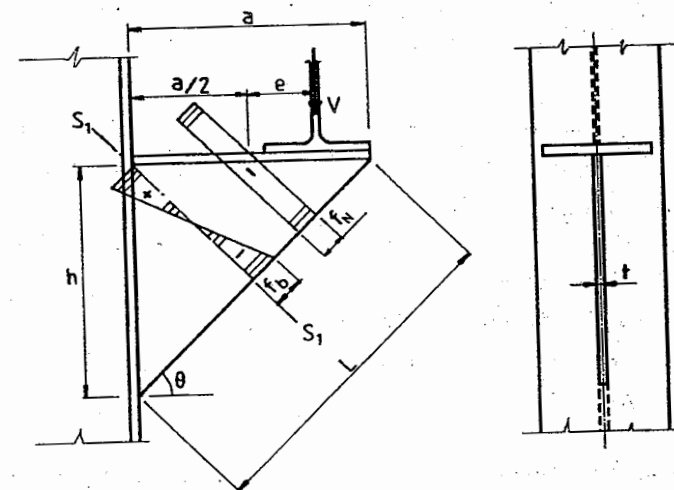


Fig. 8.7

(c) The exact Analysis:

The exact analysis of triangular brackets has been previously outlined in Section 4.2.9. It consists to use either the yielding or the buckling conditions whichever is the most critical as follows:

Yielding:

$$f_{\text{Max}} = \frac{V}{m \cdot a \cdot t} \leq 0.583 f_y \quad \dots \dots \dots 8.7$$

$$\text{where: } m = 1.39 - 2.2 \left(\frac{a}{h} \right) + 1.27 \left(\frac{a}{h} \right)^2 - 0.25 \left(\frac{a}{h} \right)^3 \quad \dots \dots \dots 8.8$$

a, h = dimensions of the triangular bracket as shown in

Fig. 8.7.

Buckling Condition:

It is suggested to use either Equation 8.4 or to assume a strip of width $(a \sin \theta / 4)$ acting as a compression member.

General Remarks Regarding the Welded Brackets:

Fig. 8.8, illustrates the three common types of welded major axis column brackets. For type (1) shown in Fig. 8.8.a it is suggested to compute the maximum induced compressive stresses using Method (A) where the tension force is directly transmitted to the column flange via the bracket top plate.

Type (2) shown in Fig. 8.8.b is the simplest type where the beam theory is directly applied along the vertical section. The induced stresses are similar to those previously outlined using Method (B).

Type (3) shown in Fig. 8.8.c is similar to type (2). The beam theory is to be applied along the vertical and the inclined sections.

It is generally recommended to use fitted vertical stiffeners fillet welded to the bracket web to prevent the local buckling just below the crane girder section.

8.6. Analysis of Bolted Major Axis Column Brackets:

Fig. 8.9, illustrates the three common types of bolted major axis column brackets. As these brackets are subjected to fatigue loadings as will be given in Section 8.10 high strength bolts of the friction type are generally used.

Regarding the analysis it is recommended to apply Method (B) previously given in Section 8.5 for the flushed bolted end plate shown in Fig. 8.9.a. It is necessary to check the induced compressive stress along section $S_1 - S_1$. If an overstress did occur then either the bracket height or the gusset plate thickness had to be increased. However, the induced bending stresses are conservative. The conservatism arises because no

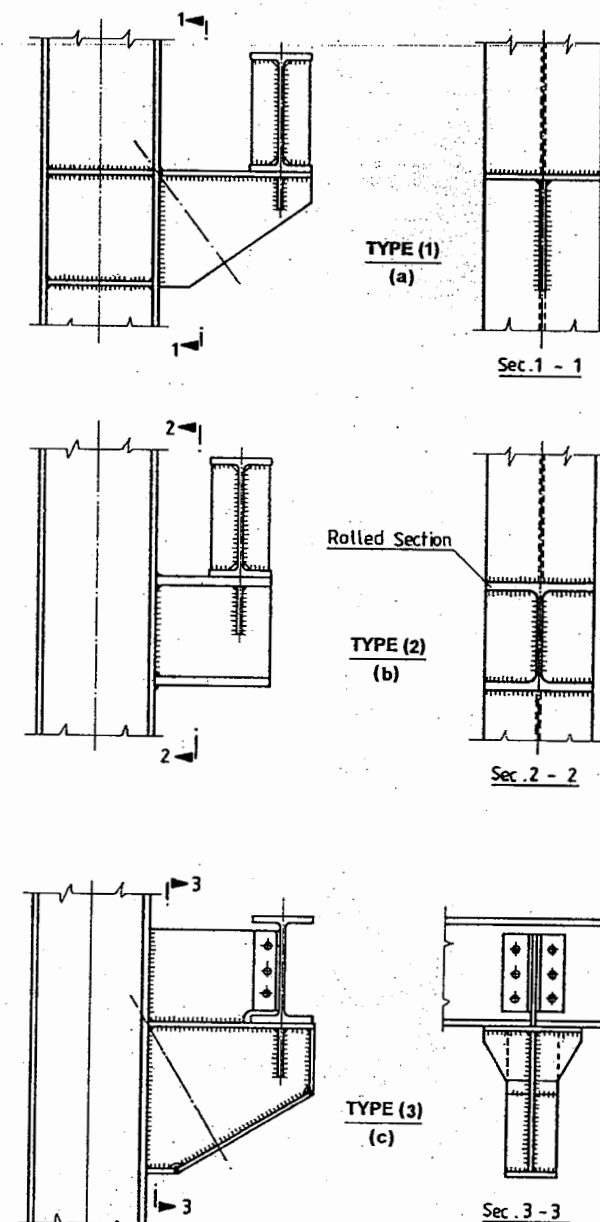


Fig. 8.8. Welded Brackets

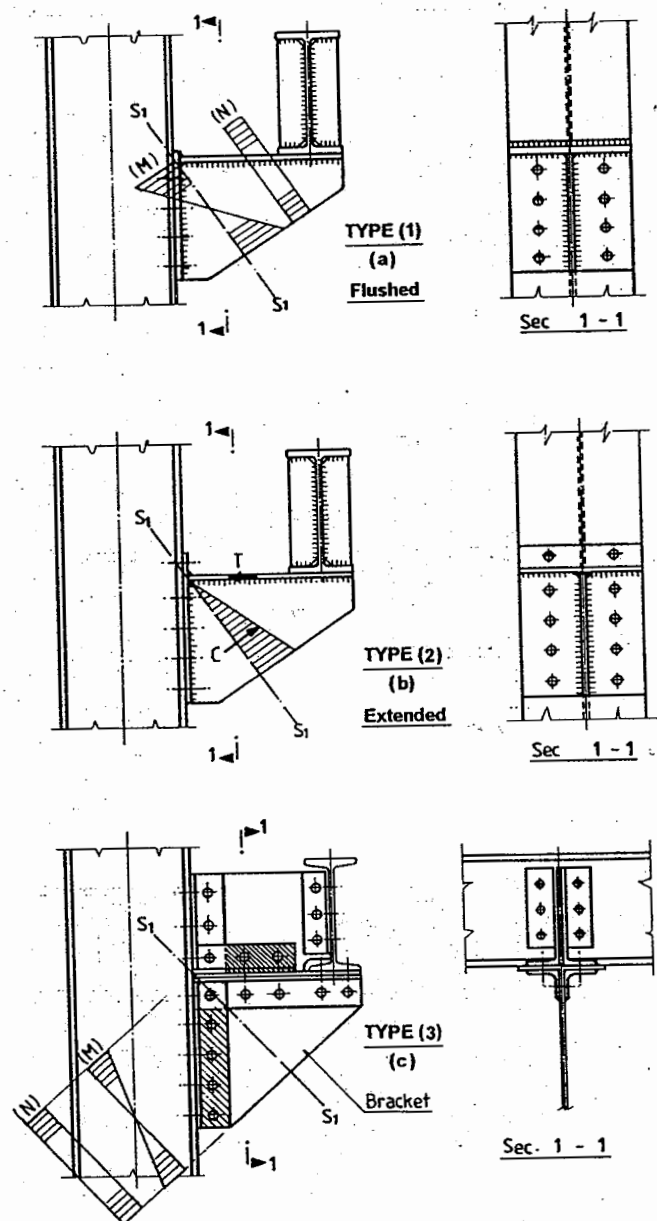


Fig. 8.9. Bolted Brackets

account is taken of the contribution of the top flange and the end plate to bending resistance. —However, because of the influence of shear lag near the discontinuity at the corner and the coincidence of the critical section with this discontinuity, the contribution of the top plate is in fact insignificant. In addition, the induced shear stresses on the critical section S_1-S_1 from the tangential component of the eccentric load should be checked. A shear check on the vertical section adjacent to the point of application of the vertical crane load (V) is also required.

If the end plate is extended as shown in Fig. 8.9.b, the reasonable analysis should be Method (B) previously given in Section 8.5. The bracket top flange is capable to resist the induced tensile force while the gusset plate will be capable to resist the induced compression force.

Fig. 8.9.c, illustrates an ancient traditional ordinary bolted bracket. However high strength bolts of the friction type may be utilized. Regarding the analysis it should be preferable to apply Method (B). The induced stresses are to be computed considering only the gusset plate as the resisting steel section.

8.7. Vertical Supports for Heavy Cranes:

For heavy cranes, separate vertical columns with built-up or rolled sections are to be utilized to transmit directly the crane loads to the soil. This configuration would be an appropriate arrangement for cranes capacity exceeding 20 tons where the column brackets discussed in the previous sections would probably show overstresses regarding the different steel components.

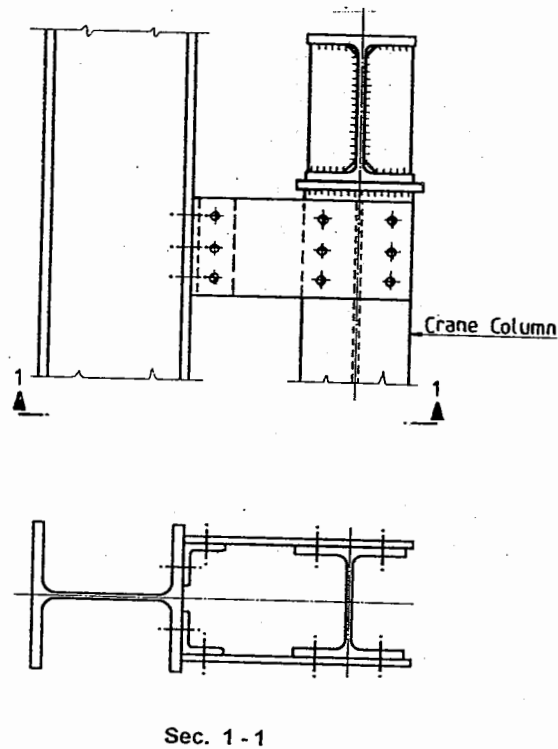


Fig. 8.10. Crane Columns for Heavy Cranes

The utilized additional crane column is to be either laced or battened to the nearest vertical column of the main structural system. Fig. 8.10, illustrates a crane battened column where the batten plates or the lacing bars are to be designed in accordance to the prescriptions of Chapter 3 (Volume 1). These batten plates and lacing bars are required to provide sufficient stiffness to ensure that the crane column and the vertical column of the main structural system are acting compositely. The fasteners of the batten plates and lacing bars



connections are either high strength bolts of the friction type or intermittent fillet weld lines.

8.8. Detailing of Vertical Support Crane Girder Connections:

Usually the crane girders are simply supported beams, hence the most important aspect of the design is the assurance of the end rotations which will occur. Even with a small deflection as prescribed by the E.C.P. 2001 limitation of $(L/800)$, beam end rotations of 0.2° will occur repeatedly.

Fig. 8.11.a, illustrates a crane column vertical support configuration where the end-bearing stiffeners are positioned directly over the column flanges. The column cap and the bolts attaching this cap to the crane girder flange must be capable to resist the induced uplift which corresponds to the end beam rotations. To ensure this transmission a thin cap is to be utilized, in addition the bolts must be positioned as far as possible from the column and beam web. This will lead to the adequate required flexibility.

Fig. 8.11.b shows another alternative where the end-bearing stiffeners are transmitting the vertical crane load to the column web via additional vertical web column stiffeners, a simple additional pack plate may also be utilized.

Fig. 8.11.c, illustrates two extended end plates fillet welded to the crane girders with a slot into a recessed keep plate.

8.9. Beam Lateral Supports:

Crane girder lateral supports are needed to transmit the lateral shock (surge) applied at the crane rail level to the columns of the main structural systems. The most efficient lateral support arrangement is that one which is positioned close to the crane rail level.

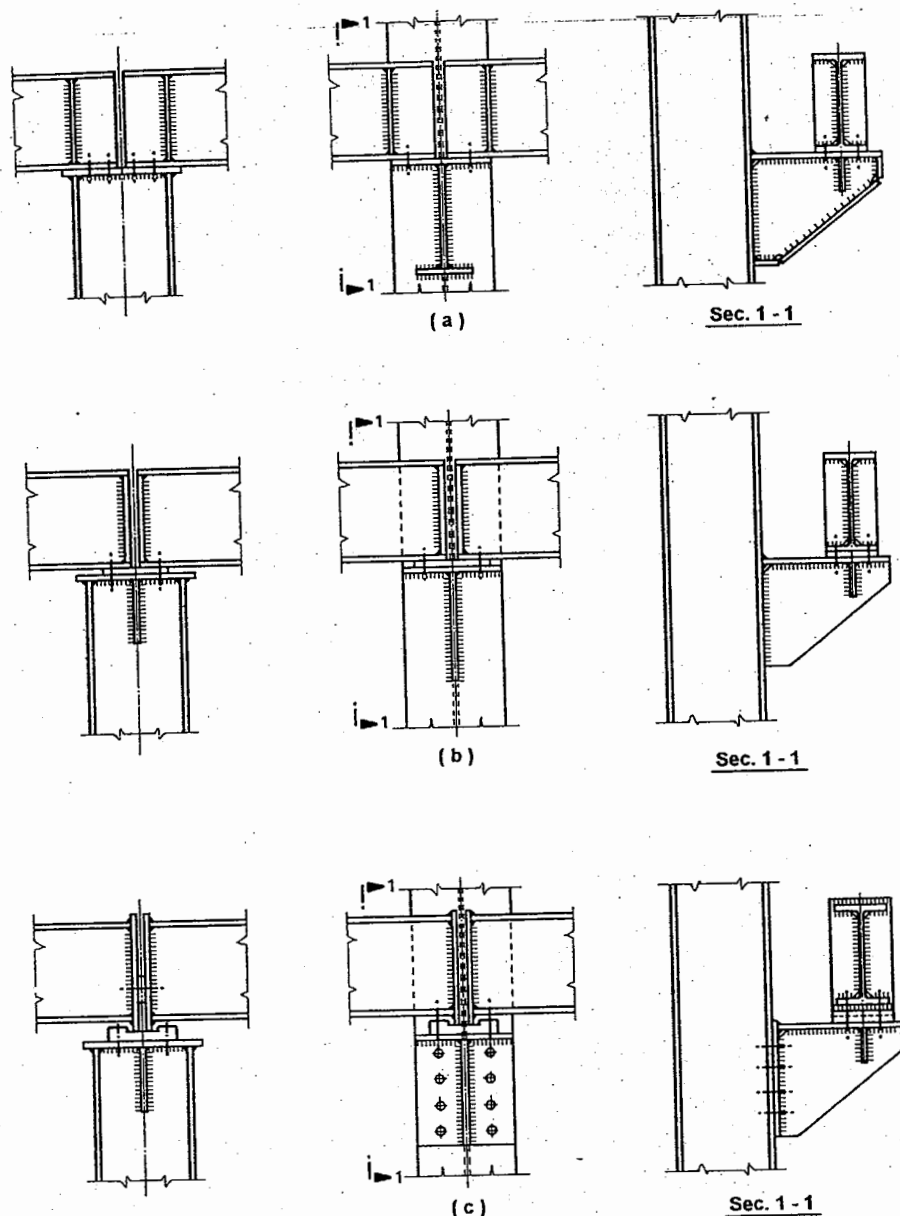


Fig. 8.11. Detailing of Vertical Crane Support Connections

Fig. 8.12.a, illustrates the most effective restraint lateral configuration where the induced moments due to the eccentricity are minimum. While if configuration (B) shown in Fig. 8.12.b is utilized, the induced moments are important and must be taken into account for the design of the lateral support steel elements. As the lateral support is connected to a vertical crane stiffener, it is essential to weld this stiffener to the top flange of the crane girder to minimize as much as possible the stress concentrations.

Similarly Fig. 8.13 shows the details of a crane column lateral support.

It is to be noticed that as with the vertical supports previously discussed in Section 8.8, the lateral supports must provide accommodations to repeated end beam rotations. Some designers specify slotted holes to ensure the required flexibility but unfortunately these are unlikely to permit movement in practice. However for heavy duty installations Fig. 8.14 shows three configurations. For case (a) link members are attached to the upper crane girder flange by pins to transmit the lateral shock (surge) to the adjacent columns, this should ensure adequate flexibility. While case (b) comprises an upper steel horizontal plate attached to the upper crane flange and to additional secondary beams. This steel plate should transmit the lateral shock (surge) and may be utilized as a walkway. Configuration (c) is similar to (b) with the only difference that secondary beams have been replaced by transverse steel plates forming a Tee section with the surge horizontal plate.

8.10. Fatigue Stress Computations:

The main problem which arise for cranes and their supporting structural elements is the designer misunderstanding of the crane load history.

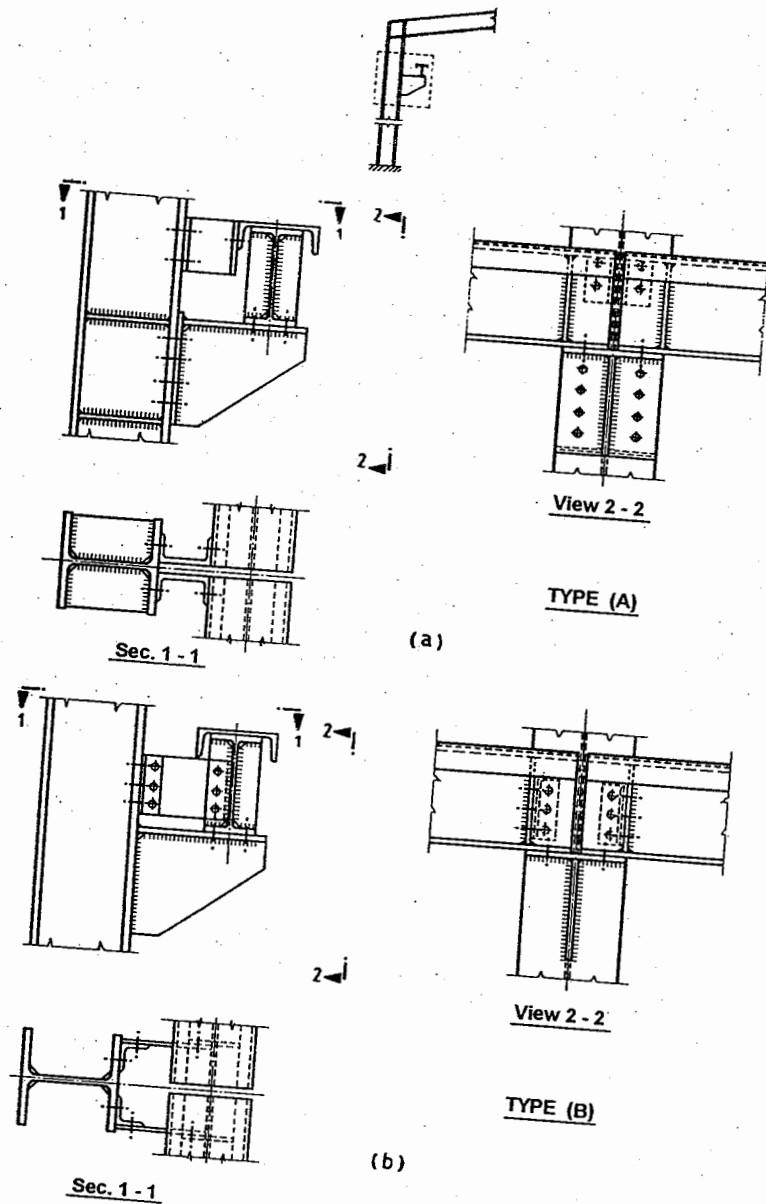


Fig. 8.12. Lateral Supports in Bracket Connections (Light Cranes)

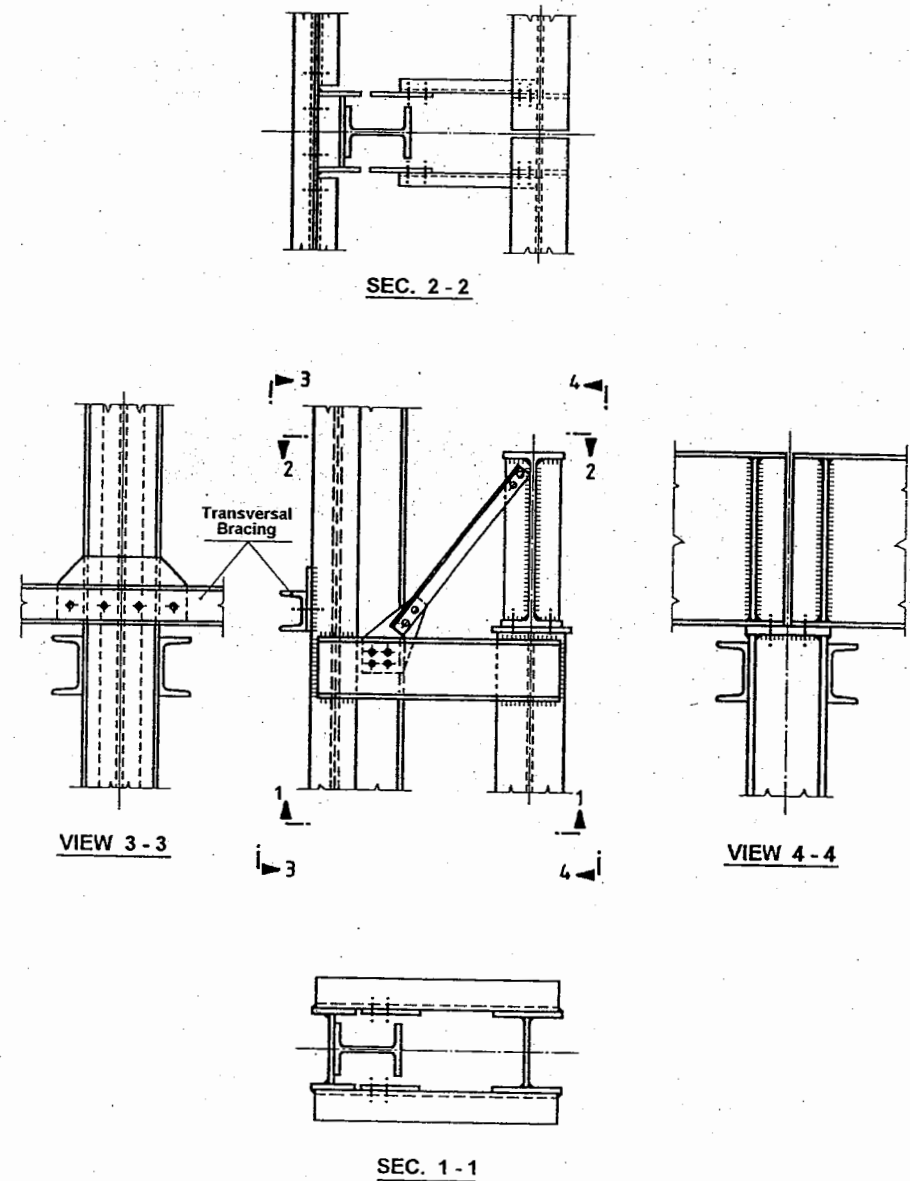


Fig. 8.13. Lateral Support in Bracket Connection (Heavy Cranes)

The classification of the steel components comprising those regarding steel bracket is listed in Table (8.2) as follows:

Note : The arrow indicates the location and direction of the stresses for which the stress range is calculated.

Table 8.2

Group 1: Non-Welded Details

Description	Illustration	Class
1.1. Base metal with rolled or cleaned surfaces; flame cut edges with a surface roughness less than 25 μ m		A
1.2. Base metal with sheared or flame cut edges with a surface roughness less than 50 μ m		B
2.1. Base metal at gross section of high strength bolted slip resistant (friction) connections, except axially loaded joints which induce out of plane bending in connected material.		B
2.2. Base metal at net section of fully tensioned high strength bolted bearing type connections		B'
2.3. Base metal at net section of other mechanically fastened joints (ordinary bolts & rivets).		D
3. Base metal at net section of eye-bar head or pin plate.		E

Group 2: Welded Structural Elements

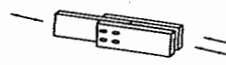
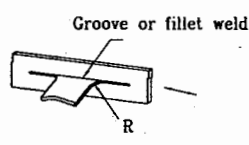
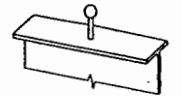
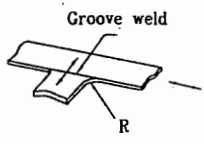
Description	Illustration	Class
4.1. Base metal in members without attachments, built up plates or shapes connected by continuous full penetration groove welds or by continuous fillet welds carried out from both sides without start stop positions parallel to the direction of applied stress.	(a)	B
4.2. Same as (4.1.) with welds having stop - start positions.	(c)	B'
4.3. Base metal in members without attachments, built-up plates or shapes connected by continuous full penetration groove welds with backing bars not removed, or by partial penetration groove welds parallel to the direction of applied stress.	(d)	B'
5. Base metal at continuous manual longitudinal fillet or full penetration groove welds carried out from one side only. A good fit between flange and web plates is essential and a weld preparation at the web edge such that the root face is adequate for the achievement of regular root penetration.		C
6. Base metal at zones of intermittent longitudinal welds with gap ratio $g/h \leq 2.5$		D
7. Base metal at zones containing copes in longitudinally welded T-joints.		D
8. Base metal at toe of welds on girder webs or flanges adjacent to welded transverse stiffness.		C

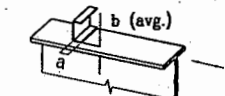
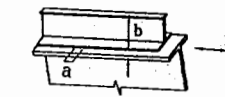


Description	Illustration	Class
9.1. Base metal and weld metal at full penetration groove welded splices (weld made from both sides) of parts of similar cross sections ground flush, with grinding in the direction of applied stress and weld soundness established by radiographic or ultrasonic inspection.		B
9.2. Same as (9.1.) but with reinforcement not removed and less than 0.10 of weld width.		C
9.3. Same as (9.2.) with reinforcement more than 0.10 of weld width.		D
10.1. Base metal and weld metal at full penetration groove welded splices (weld made from both sides) at transitions in width or thickness, with welds ground to provide slopes no steeper than 1 to 2.5 with grinding in the direction of applied stress, and with weld soundness established by radiographic or ultrasonic inspection.		B'
10.2. Same as (10.1.) but with reinforcement not removed and less than 0.10 of weld width.		C
10.3. Same as (10.2.) with slopes more than 1 to 2.5		D
10.4. Same as (10.1.) to (10.3.) but with welds made from one side only.		E'

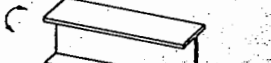
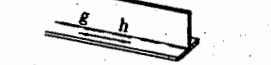
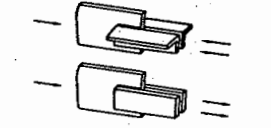
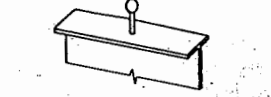


Description	Illustration	Class
11.1. Base metal and weld metal at transverse full penetration groove welded splices on a backing bar. The end of the fillet weld of the backing strip is more than 10-mm from the edges of the stressed plate		D
11.2. Same as (11.1) with the fillet weld less than 10-mm from the edges of the stressed plate.		E
12.1. Base metal at ends of partial length welded cover plates narrower than the flange having square or tapered ends, with or without welds across the ends or wider than the flange with welds at the ends. Flange thickness ≤ 20 -mm		E
Flange thickness > 20 -mm		E'
12.2. Base metal at ends of partial length welded cover plates wider than the flange without end welds.		E'
13. Base metal at axially loaded members with fillet welded connections. $b < 25$ -mm		E
$b > 25$ -mm		E'
14. Base metal at members connected with transverse fillet welds.		C
15.1. Base metal at full penetration weld in cruciform joints made of a special quality weld.		D
15.2. Same as (15.1) with partial penetration or fillet welds of normal quality.		E'

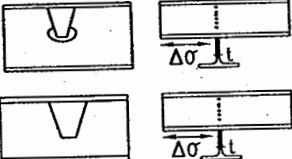
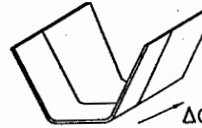
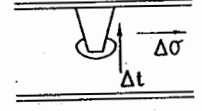
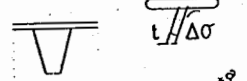
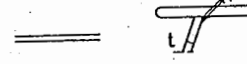
Description	Illustration	Class
16. Base metal AT PLUG OR SLOT WELDS.		E
17. Base metal and attachment at fillet welds or partial penetration groove welds with main material subjected to longitudinal loading and weld termination ground smooth $R > 50\text{-mm}$		D
$R \leq 50\text{-mm}$		E
18. Base metal at stud-type shear connector attached by fillet weld or automatic end weld.		C
19.1. Base metal at details attached by full penetration groove welds subject to longitudinal loading with weld termination ground smooth. Weld soundness established by radiographic or ultrasonic inspection $R > 610\text{-mm}$		B
$610\text{-mm} > R > 150\text{-mm}$		C
$150\text{-mm} > R > 50\text{-mm}$		D
$R < 50\text{-mm}$		E
19.2. Same as (19.1.) with transverse loading, equal thickness, and reinforcement removed. $R > 610\text{-mm}$		B
$610\text{-mm} > R > 150\text{-mm}$		C
$150\text{-mm} > R > 50\text{-mm}$		D
$R < 50\text{-mm}$		E

Description	Illustration	Class
19.3. Same as (19.2.) but reinforcement not removed $R > 610\text{-mm}$		C
$610\text{-mm} > R > 50\text{-mm}$		C
$150\text{-mm} > R > 50\text{-mm}$		D
$R > 50\text{-mm}$		E
19.4. Same as (19.2.) but with unequal thickness $R > 50\text{-mm}$		D
$R < 50\text{-mm}$		E
19.5. Same as (19.4.) but with reinforcement not removed and for all R		E
20. Base metal at detail attached by full penetration groove welds subject to longitudinal loading $50\text{-mm} < a < 12b$ or 100-mm		D
$a > 12b$ or 100-mm ($b < 25\text{-mm}$)		E
$a > 12b$ or 100-mm ($b > 25\text{-mm}$)		E'
21. Base metal at detail attached by fillet welds or partial penetration groove welds subject to longitudinal loading $a < 50\text{-mm}$	 	C
$50\text{-mm} < a < 12b$ or 100-mm		D
$a > 12b$ or 100-mm ($b < 25\text{-mm}$)		E
$a > 12b$ or 100-mm ($b > 25\text{-mm}$)		E'

Group 3: Fasteners (Welds and bolts)

Description	Illustration	Class
22.1. Weld metal of full penetration groove welds parallel to the direction of applied stress (weld from both sides)		B
22.2. Same as (22.1.) but with weld from one side only.		C
22.3. Weld metal of partial penetration transverse groove weld based on the effective throat area of the weld.		F
23.1. Weld metal of continuous manual or automatic longitudinal fillet welds transmitting continuous shear flow and fillet welded lap joints.		D
23.2. Weld metal of intermittent longitudinal fillet welds transmitting a continuous shear flow.		E
23.3. Weld metal at fillet welded lap joints.		E'
24. Transversally loaded fillet welds.		E'
25. Shear on plug or slot welds.		F
26. Shear stress on nominal area of stud-type shear connectors. (Failure in the weld or heat affected zone.)		F
27.1. High strength bolts in single or double shear (fitted bolt of bearing type).		C
27.2. Rivets and ordinary bolts in shear.		D
28. Bolts and threaded rods in tension (on net area)		F

Group 4: Orthotropic Deck Bridges

Description	Illustration	Class
29.1. Base metal at continuous longitudinal rib with or without additional cutout in cross girder. (Bending stress range in the rib) $t \leq 12\text{mm}$		C
29.2. Same as (29.1.) $t > 12\text{mm}$		D
30. Base metal at separate longitudinal ribs on each side of the cross girder. (Bending stress range in the rib)		E'
31. Base metal at rib joints made of full penetration weld with backing plate. (Bending stress range in the rib)		D
32.1. Base metal at rib joints made of full penetration weld without backing plate. All welds ground flush to plate surface in the direction of stress. Slope of thickness transition < 1:4. (Bending stress range in the rib)		B'
32.2. Same as (32.1.) with weld reinforcement ≤ 0.2		C
33. Base metal at connection of continuous longitudinal rib to cross girder. (Equivalent stress range in the cross girder web).		E'
34.1. Weld metal at full penetration weld connecting deck plate to rib section.		D
34.2. Weld metal at fillet weld connecting deck plate to rib section.		E'



Group 5 High Strength Bolts of the friction type :

5.1 High Strength Bolts subjected to shear fatigue loading :-

For high strength bolts (10.9) and (8.8) of the friction type (i.e. tightened to the connected plates by a determined torque) and subjected to shear fatigue loading the allowable friction resistance (P_s) remains unchanged as for static loading. P_s is to be determined using Table 2.6 of Chapter (2).

5.2 High Strength Bolts subjected to external tension fatigue loading :-

When subjected to tensile fatigue loading, the allowable stress range for High Strength Bolts of the friction type shall not exceed the following :-

Table 8.3 High Strength Bolts (Tension)

Number of Cycles (N)	Allowable Stress Range F_{sr} (t/cm^2)	
	8.8	10.9
$N \leq 20,000$	2.9	3.6
$20,000 < N \leq 500,000$	2.6	3.2
$500,000 < N$	2.0	2.5

(c) Determine the stress range (***) which corresponds to the connection category (i.e. class) using Table (4.2) of the E.C.P. 2001 or the Table 8.4 as follows :-

*** Stress range = the algebraic difference between two extreme values of nominal stresses due to fatigue loads. This may be determined through standard elastic analysis of the extremes of a stress history record taken from strain gauges placed at appropriate locations.



8.4. Allowable Fatigue Stress Range F_{sr} for Number of Constant Stress Cycles.

Detail Class	(***) F_{sr} (t/cm^2)			
	1×10^5 Cycles	5×10^5 Cycles	2×10^6 Cycles	$> 2 \times 10^6$ Cycles
A	4.30	2.52	1.68	1.68
B	3.42	2.00	1.26	1.12
B'	2.77	1.62	1.02	0.85
C	2.48	1.45	0.91	0.70
D	1.92	1.12	0.71	0.49
E	1.53	0.89	0.56	0.32
E'	1.11	0.69	0.41	0.16
F	0.72	0.65	0.40	0.25

Refer to:

Example 8.1 (Design of crane girder).

Example 8.2 (Minor column Axis-Welded Bracket).

Example 8.3 (Minor column Axis-Bolted Bracket).

Example 8.4 (Major Axis column-Welded Bracket).

Example 8.5 (Major Axis column-Bolted Bracket).

Example 8.6 (Major Axis column-Welded Bracket).

Example 8.7 (Major Axis column Bolted Bracket).

Design of Crane Track GirderExample (8.1):

Design a crane track girder knowing the following:

- 1- Spacing between frames = 5.0 ms.
- 2- Reaction of crane girder on either of the crane track girders is two concentrated loads 10 tons each, spaced 2.0 ms apart.

Design the crane girder.

Solution(1) Loads:A. Dead Loads:

Assume the own weight of the crane track girder to be 200 kg/m'.

$$\therefore M_{x,D.L.} = 0.20 \times \frac{(5)^2}{8} = 0.625 \text{ mt.}$$

$$\therefore Q_{y,D.L.} = 0.20 \times \frac{5}{2} = 0.50 \text{ ton.}$$

B. Live Loads:

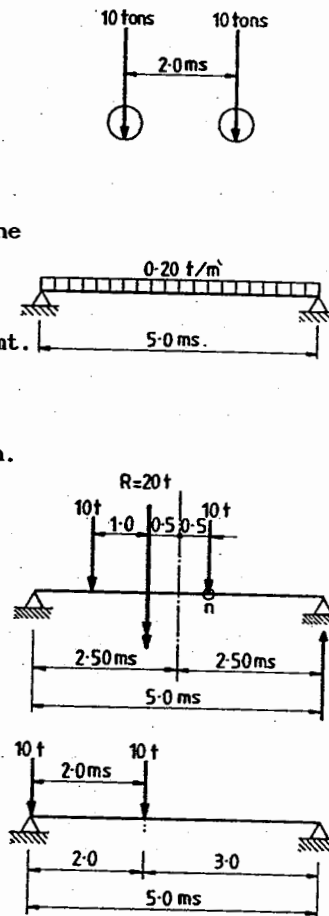
$$M_{x(n)} = \frac{20 \times 2}{5} \times 2 = 16 \text{ mt.}$$

$$M_y = 0.10 \times 16 = 1.60 \text{ mt}$$

(lateral shock)

$$Q_y = 10 + 10 \times \frac{3}{5} = 16 \text{ tons.}$$

$$Q_x = 0.1 \times 16 = 1.60 \text{ ton.}$$

C. Total Straining Actions:

$$M_x (\text{L.L.} + \text{impact}) = 0.625 + 16 \times 1.25 = 20.625 \text{ mt.}$$

$$Q_y (\text{L.L.} + \text{impact}) = 16 \times 1.25 + 0.50 = 20.50 \text{ tons.}$$

$$Q_x = 1.60 \text{ ton.} \quad ; \quad M_y = 1.60 \text{ mt.}$$

D. Braking Force (B.F.) :-

$$\begin{aligned} \text{B.F.} &= \left(\sum P_{L.L.} / 7 \right) / \text{number of columns: (8 columns)} \\ &= (10 + 10) / (7 \times 8) = 0.357 \text{ tons.} \end{aligned}$$

(2) Design as Rolled Section (1st Alternative):

$$f_b = \frac{M_x}{Z} + \frac{M_x}{Z_x / 2} < 1.40 \times 1.20 \text{ t/cm}^2$$

Choose B.F.I. Section ($Z_x \approx 3 Z_y$).

$$\therefore \frac{20.625 \times 100}{Z_x} + \frac{1.6 \times 100}{\frac{1}{2} \times \frac{1}{3} Z_x} \leq 1.4 \times 1.2$$

$$\therefore Z_{x, \text{req.}} = 1799 \text{ cm}^3. \quad \text{Try B.F.I. \# (32) } (Z_x = 2020 \text{ cm}^3)$$

A. Compactness of section regarding L.T.B. :-

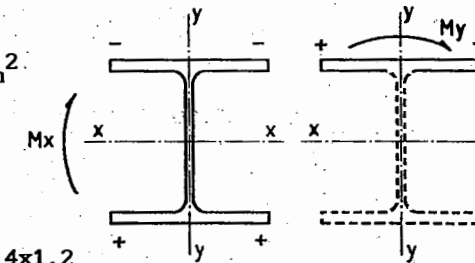
$$L_u = 20 b / \sqrt{F_y} = 20 \times 30 / \sqrt{2.4} = 387.3 \text{ cms} > 500 \text{ cms unsafe}$$

$$L_u = \frac{1380 \cdot A_f}{d \cdot F_y} \cdot C_b = \frac{1380 \times 30 \times 2.2}{32 \times 2.4} \times 1.0 = 1185.9 \text{ cms} > 500$$

O.K.

No need to compute the L.T.B. stresses as $1185.9 > 500 \text{ cms}$

The section is noncompact regarding L.T.B.; $F_{bx} = F_{by} = 0.58 F_y$.



Compactness of section regarding Local buckling :-

All rolled sections are fulfilling the width/thickness ratios regarding compact sections. In order to avoid repetition the steel elements are compact regarding local buckling. Here the compactness is governed by the lateral torsional buckling.

B. Check on Stresses:

$$f_{bt} \text{ (Case I)} = \frac{20.625 \times 100}{2020} = 1.02 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2$$

$$f_{bt} \text{ (Case II)} = \frac{M_x}{Z_x} + \frac{M_y}{Z_y / 2} = \frac{20.625 \times 100}{2020} + \frac{160 \times 2}{661} = 1.50 \text{ t/cm}^2$$

$$< 1.4 \times 1.2 \text{ t/cm}^2 \text{ O.K.}$$

$$q_y = \frac{Q_y}{A_w} = \frac{20.50}{1.3 \times 32} = 0.49 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2 \text{ (O.K.)}$$

$$q_x = \frac{3}{2} \times \frac{Q_x}{A_f} = \frac{3}{2} \times \frac{1.60}{30 \times 2.20} = 0.006 \text{ t/cm}^2 < 0.84 \times 1.2 \text{ t/cm}^2$$

$$\text{(O.K.)}$$

C. Check on Deflection:

$$M_x \text{ (L.L + impact)} = 16 \times 1.25 = 20 \text{ mts.}$$

$$\therefore W_{eq.} = \frac{8 \times 20}{(5)^2} = 6.40 \text{ t/m} = 0.064 \text{ t/cm}$$

$$\therefore \Delta_{act.} = \frac{5 W_{eq.} L^4}{384 E I_x} = \frac{5 \times 0.064 \times (500)^4}{384 \times 2100 \times 32250} = 0.77 \text{ cm.}$$

$$\therefore \Delta_{max.} = \frac{500}{800} = 0.625 \text{ cm} < \Delta_{act.} \text{ Unsafe}$$



$$\frac{\Delta_1}{\Delta_2} = \frac{I_2}{I_1} \therefore \frac{0.625}{0.77} = \frac{32250}{I_x}$$

$$\therefore I_{x \text{ req.}} = 39732 \text{ cm}^4$$

$\therefore$ Choose B.F.I. # (36)

D. Web crippling stresses:

The local buckling of the crane girder web is to be checked at concentrated load locations (i.e. at wheel loads and reactions). If necessary, vertical stiffeners are to be introduced to prevent the crippling. The induced crippling stresses are to be computed in accordance with Clause 3.6.6 of the E.C.P. (2001) where the crippling stresses must not exceed $0.75 F_y$.

(a) At Wheel Load:

Wheel load ≈ 10 tons.

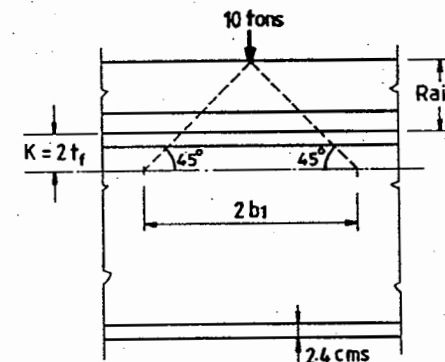
$$b_1 = \text{rail height} + k$$

$$b_1 = 7.5 + 2 \times 2.4$$

$$b_1 = 12.3 \text{ cms.}$$

$$f_{crp} = \frac{10}{2 \times 12.3 \times 1.4}$$

$$= 0.29 \text{ t/cm}^2 < 0.75 F_y \text{ o.k.}$$

(b) At Reaction Location:

$R = 20.5$ tons.

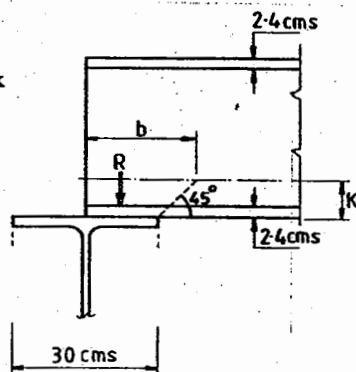
$$b = 15 + k = 15 + 2 \times 2.4$$

$$= 19.8 \text{ cms.}$$

$$f_{crp} = \frac{20.5}{19.8 \times 1.4} = 0.74 \text{ t/cm}^2$$

$$t_w < 0.75 F_y \quad \text{o.k.}$$

Generally the web crane girder local buckling is satisfying the crippling stress conditions for rolled sections, and hence no need to do the above computations.



2nd Alternative

Design as a Built up Section:

$$\left[\frac{M_x}{Z_x} \right] + \left[\frac{M_y}{Z_y} \right] < 1.40 \times 1.20$$

60 % 40 % 60 %

$$\therefore \frac{20.625 \times 100}{Z_x} = 0.60 \times 1.4 \times 1.2$$

$$\therefore Z_{x_{total}} = 2046 \text{ cm}^3$$

$$\therefore Z_x \text{ (S.I.B.)} = 0.70 Z_{x_{total}} = 1432 \text{ cm}^3$$

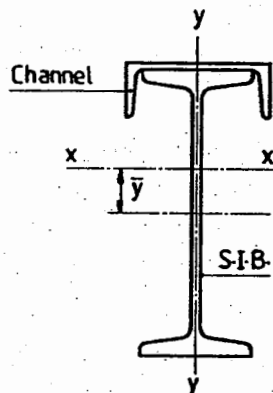
70 %

$$\therefore \text{Choose S.I.B. No. (42}^{1/2}\text{)}. [Z_y = 176 \text{ cm}^3 ; Z_x = 1740 \text{ cm}^3]$$

$$\frac{1.60 \times 100}{Z_y} = 0.40 \times 1.40 \times 1.20$$

40 %

$$\therefore Z_{y_{total}} = 238 \text{ cm}^3 = \frac{1}{2} Z_y \text{ (S.I.B.)} + Z_x \text{ (I)}$$



$$\therefore Z_x \text{ (I)} = 238 - 0.50 \times 176$$

$$= 150 \text{ cm}^3$$

Choose channel No. (20)

Properties of the Section:

$$A_t = 132 + 32.2 = 164.20 \text{ cm}^2$$

$$\bar{y} = \frac{32.20 (42.5 \times 0.50 + 0.85 - 2.01)}{(132 + 32.2)}$$

$$= 3.94 \text{ cms.}$$

$$\therefore I_x = 36970 + 132 \times (3.94)^2 + 148$$

$$+ 32.2 \times (16.15)^2 = 47565.60 \text{ cm}^4$$

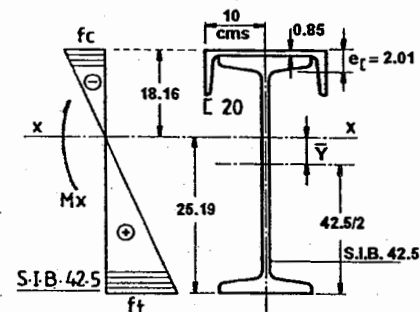
$$I_y = I_{x_t} + \frac{1}{2} I_y \text{ (S.I.B.)}$$

$$= 1910 + \frac{1}{2} \times 1440 = 2630 \text{ cm}^4$$

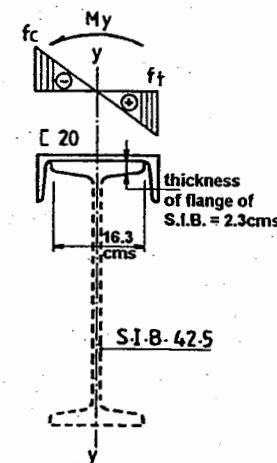
$$\therefore Z_{x_{tension}} = \frac{I_x}{25.19} = 1888.27 \text{ cm}^3$$

$$\therefore Z_{x_{comp.}} = \frac{I_x}{18.16} = 2619.25 \text{ cm}^3$$

$$\therefore Z_y = \frac{I_y}{10} = 263 \text{ cm}^3$$



Due to M_x



Due to M_y

Compactness of cross section :-

Regarding the Lateral Torsional Buckling use Equ. 5.37 by replacing $b = 4 r_y$.



$$r_y = \sqrt{\frac{I_y}{A_c + A_{\text{upper flange of S.I.B}}}} = \sqrt{\frac{2630}{32.2 + 16.3 \times 2.3}} = 6.14 \text{ cms}$$

$$L_u = \frac{20 \times 4 \times 6.14}{\sqrt{F_y}} = 317.1 \text{ cms} < 500 \text{ cms} \quad \text{unsafe}$$

The section is noncompact $F_{bx} = F_{by} = 0.58 F_y$.

Check on Stresses:

$$f_{bt} = \frac{20.625 \times 100}{1888.27} = 1.09 \text{ t/cm}^2 < 1.40 \text{ t/cm}^2$$

$$f_{bc} = \frac{20.625 \times 100}{2619.25} + \frac{1.60 \times 100}{263} = 1.395 \text{ t/cm}^2 < 1.68 \text{ t/cm}^2$$

$$q_y = \frac{20.50}{1.53 \times 42.5} = 0.31 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2$$

(Safe)

Lateral Torsional Buckling:

$$F_{L.T.B.} = \frac{800 \times (16.3 \times 2.3 + 32.2)}{500 \times 42.5} \times 1.0 = 2.758 \text{ t/cm}^2 \quad \text{(Safe)}$$

Check on Deflection:

$$W_{eq.} = 0.064 \text{ t/cm'}$$

$$\therefore \Delta_{act.} = \frac{5 \times 0.064 \times (500)^4}{384 \times 2100 \times 47565.60} = 0.5214 \text{ cm} < 0.56 \text{ cm} \quad \text{Safe}$$

Web crippling stresses:

Proceed as 1st Alternative, the crippling is safe.



Example (8.2):

Design the required weld lengths as well as the gusset plates of the bracket shown in the figure below to resist a vertical load $V = 20.0$ tons and a lateral shock $H = 1.60$ ton. The breaking force B.F. = 0.30 ton.

Solution

Preliminary Dimensioning:

The dimensions of the bracket can be assumed as follows:

$$h \cong 1.5 H ; \quad b \cong 2 B ; \quad d_1 \cong H/2 ; \quad t = t_{fc}$$

where:

H, B = height and flange width of crane girder cross section.

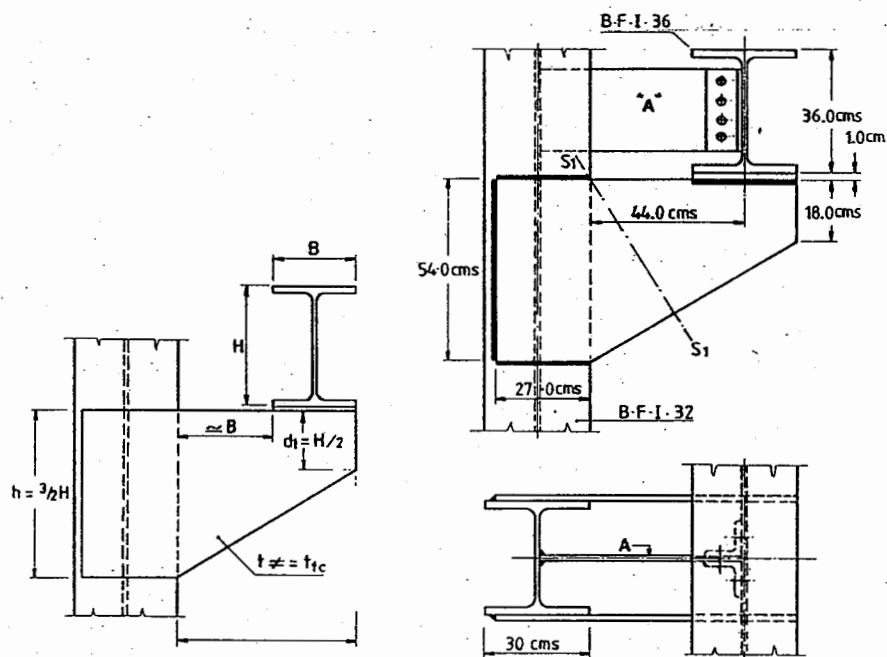
t, t_{fc} = gusset plate and column flange thicknesses.

The Fatigue Effect (Refer to Chapter 4 of the E.C.P. 2001)

$$\text{No. of cycles} = 2 \times 10^6 \quad \text{(Table 8.1)}$$

Detail Class and Stress Range (Tables 8.2 and 8.4):

Bracket Component	Item no.	Class	Stress Range (F_{sr})
Section ($S_1 - S_1$)	14	C	0.91 t/cm^2
Fillet weld	23.3	E'	0.41 t/cm^2



(1) Check of Stresses:

(A) Check of Stresses Along the Inclined Section ($S_1 - S_1$) -

Approximate Method (A):

The straining actions due to dead loads being of minor effect $V_{D.L.} = 0.5$ tons compared to fatigue loads including impact ($V_{L.L. + I} = 20.5$ tons) for simplicity the total vertical force is taken equal to 21 tons. The max. allowable stress for every component of the bracket due to the total loads will be the value of the allowable stress range according to the corresponding class.

Straining Actions Along Section $S_1 - S_1$:

Assume that the lateral shock "H" is transferred directly to the upper plate "A".

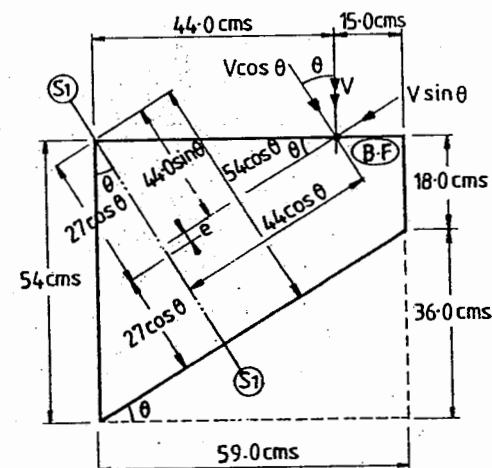
$$e = 27 \cos \theta - 44 \sin \theta = 27 \times 0.853 - 44 \times 0.521 = 0.107 \text{ cm}$$

$$M_x = (V \cos \theta \times 44 \cos \theta - V \sin \theta \times e)$$

$$B.F. = 0.357 \text{ tons}$$

$$\cos \theta = 0.853$$

$$\sin \theta = 0.521$$



$$M_x = 21 (44 \times 0.853^2 - 0.521 \times 0.107) = 671.14 \text{ t.cms}$$

$$M_y = 0.357 \times 44 \cos \theta = 13.4 \text{ t.cms} \quad (\text{Braking force})$$

$$N = V \sin \theta = 21 \times 0.521 = 10.94 \text{ tons}$$

$$Q = V \cos \theta = 21 \times 0.853 = 17.9 \text{ tons}$$

$$M_t = 0.357 \times 0.107 = 0.0357 \text{ t.cm} \quad (\text{neglected}) \quad (\text{Braking force})$$

Shearing force along vertical section below the vertical force

$$V = 21 \text{ tons.}$$

$$\text{Height of the corresponding section} = 18 + 36 \times \frac{15}{59} = 27.15 \text{ cms}$$

Properties One Gusset Plates :-

(Section $S_1 - S_1$):



$$t = 2.2 \text{ cms}$$

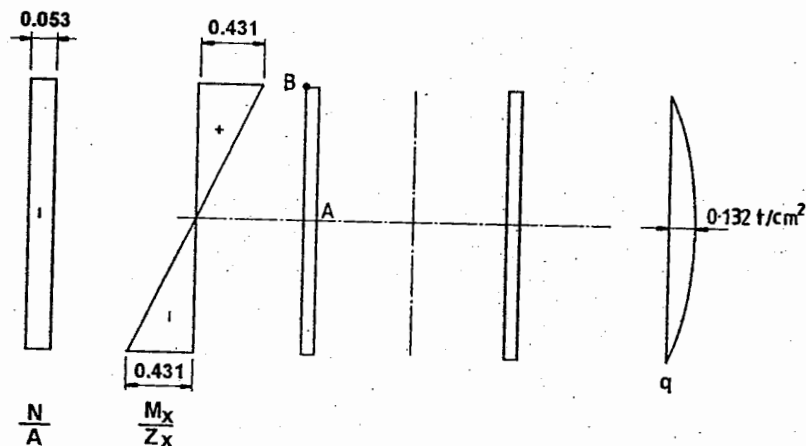
$$I_{xx} = 2 \times 2.2 \times \frac{46.09^3}{12} = 35832 \text{ cm}^4$$

$$I_{yy} \approx 2 \times 46.06 \times 2.2 \times 17.1^2 \approx 59261 \text{ cm}^4$$

$$I_p = I_x + I_y = 95093 \text{ cm}^4$$

$$A = 2 \times 2.2 \times 46.06 = 202.66 \text{ cm}^2$$

Check of Stresses:



stresses are in t/cm^2

Point (B):

$$f_B = \frac{N}{A} + \frac{M_x}{Z_x} + \frac{M_y}{Z_y} = \frac{10.94}{202.66} + \frac{671.14 \times 23.03}{35832} + \frac{13.40 \times 18.2}{59261}$$

$$f_B = 0.053 + 0.431 + 0.00411 = 0.488 \text{ t/cm}^2 < 0.91 \text{ t/cm}^2 \text{ class (C)}$$

Effect of Bracking Force

Point (A):

$$q_{A \downarrow (Q)} = \frac{3}{2} \times \frac{17.9}{202.66} = 0.132 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2 \text{ (Sec. } S_1 - S_1) \text{ o.k.}$$

$$q_{\downarrow (v)} = \frac{3}{2} \times \frac{21}{2 \times 27.15 \times 2.2} = 0.263 \text{ t/cm}^2 \text{ (vertical section) o.k.}$$

(B) Exact Stress Analysis (Refer to Section 4.11):

Yielding:

$$\frac{a}{h} = \frac{59}{54} = 1.09$$

Use Equation (a) - Section 4.11.

$$Z = 1.39 - 2.2 \times 1.09 + 1.27 (1.09)^2 - 0.25 (1.09)^3 = 0.167$$

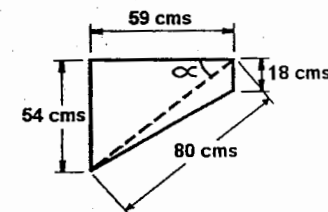
$$t = \frac{V}{F_c \cdot a \cdot z} = \frac{21}{0.91 \times 59 \times 0.167} = 2.34 \text{ cm}$$

Allowable Stress Range

Hence choose $t = 2.4 \text{ cms}$.

Buckling Condition:

o The Bracket can be approximated to the triangular shape.



$$\text{Hence: } L = \sqrt{59^2 + 54^2} \approx 80 \text{ cms ; } a = 44 + 15 = 59 \text{ cms}$$

$$h = 54 \text{ cms ; } \cos \alpha = \frac{59}{80} = 0.7375 ; \sin \alpha = 0.675$$

Use Equation 8.2 to compute the buckling coefficient (k):

$$k = 3.2 - 3.0 \left(\frac{59}{54} \right) + 1.1 \left(\frac{59}{54} \right)^2 = 1.236$$

Substitute in Equation 8.4 to get the corresponding thickness (t):



$$t = 0.1 \sqrt[3]{\frac{21 \times 59}{1.236}} \approx 1.0 \text{ cm}$$

- o If a strip of width $(59 \sin \theta)/4 = 9.95 \text{ cms}$ is assumed to act as a column,

$$i_{\min} = \sqrt{\frac{9.95 \times t^3}{9.95 \times t \times 12}} = \sqrt{\frac{t^2}{12}} = t / \sqrt{12}$$

$$L/i = \frac{80 \sqrt{12}}{t} = \frac{277.13}{t}$$

Using the Euler buckling equation with $(t=2.2\text{cms})$ we get:

$$F_c = 1.4 - 0.000065 \left(\frac{277.13}{t} \right)^2 = 0.368 \text{ t/cm}^2 < f_B = 0.488 \text{ t/cm}^2$$

Unsafe

Increase $t = 2.4 \text{ cms}$

$$f_B = \frac{0.488 \times 2.2}{2.4} = 0.447 \text{ t/cm}^2$$

$$F_c = 1.4 - 0.000065 \left(\frac{277.13}{2.4} \right)^2 = 0.81 \text{ t/cm}^2 > f_B = 0.447 \text{ t/cm}^2$$

For this exact stress analysis refer to Section 4.11.

(2) Fillet Welds Connecting Bracket to Column (Class E'):

Assume the size of weld = 1.00 cm.

$$A_w = 54 \times 1.0 + 2 \times 27 \times 1.0 = 108 \text{ cm}^2$$

$$\bar{X} = \frac{\bar{b}^2}{2b + h} = \frac{(27)^2}{108} = 6.75 \text{ cms}$$

$$I_x = \frac{54 \times 1.0^3}{12} + 2 \times 27 \times 1.0 \times (27)^2 = 52488 \text{ cm}^4$$

$$I_y = 54 \times 1 \times 6.75^2 + \frac{2 \times 27^3}{12} + 2 \times 27 \times 1 \times 6.75^2 = 8201.25 \text{ cm}^4$$

$$I_p = I_x + I_y = 60689.25 \text{ cm}^4$$

Straining Actions :

Effect of braking force :

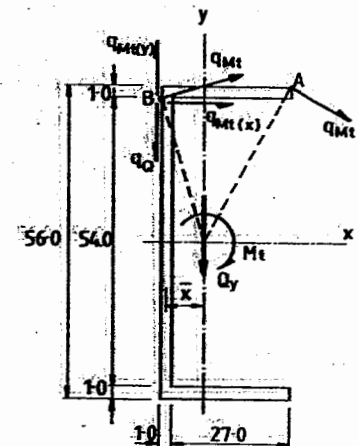
$$N = 0.357 \text{ tons}$$

$$M_x = 0.357 \times (36 + 1.0 + 27) = 22.85 \text{ t.cm.}$$

$$M_y = 0.357 \times (44 + 20.25) = 22.93 \text{ t.cm.}$$

$$M_t = 21 \times (44 + 20.25) = 1349.25 \text{ t.cm.}$$

$$Q_y = 21.0 \text{ tons.}$$



At Point (A):

$$F_{sr} = 0.41 \text{ t/cm}^2$$

$$f_{bt} = \frac{N}{A} + \frac{M_x \cdot y}{I_x} + \frac{M_y \cdot x}{I_y}$$

$$= \frac{0.357}{2 \times 108} + \frac{22.85 \times 27.0}{2 \times 52488} + \frac{22.93 \times 20.25}{2 \times 8201.25} = 0.035 \text{ t/cm}^2$$

B.F. can be neglected

$$q_{Mt} = \frac{1349.25 \times 33.75}{2 \times 60689.25} = 0.375 \text{ t/cm}^2$$

$$f_{\text{princ.}} = \frac{0.035}{2} + \sqrt{\left(\frac{0.035}{2} \right)^2 + (0.375)^2} = 0.38 \text{ t/cm}^2$$

< 0.43 t/cm² Safe



At Point (B):

B.F.

$$f_{bt} = \frac{0.357}{2 \times 108} + \frac{22.85 \times 27.0}{2 \times 52488} + \frac{22.93 \times 6.75}{2 \times 8201.25} = 0.008 \text{ t/cm}^2$$

$$q_{\downarrow} = \frac{21.0}{2 \times 54.0 \times 1.0} = 0.1944 \text{ t/cm}^2$$

$$q_{Mt(x)} = \frac{1349.25 \times 27}{2 \times 60689.25} = 0.3 \text{ t/cm}^2$$

$$q_{Mt(y)} = \frac{1357.25 \times 6.75}{2 \times 60689.25} = 0.075 \text{ t/cm}^2$$

$$\therefore q_{total} = \sqrt{(0.3)^2 + (0.1944 - 0.075)^2} = 0.322 \text{ t/cm}^2$$

$$\therefore f_{princ.} = \frac{0.008}{2} + \sqrt{\left(\frac{0.008}{2}\right)^2 + (0.322)^2} \approx 0.322 \text{ t/cm}^2$$

B.F. B.F.

$$< 0.41 \text{ t/cm}^2$$

Class (E) (Safe)

$$\therefore \text{Use size of weld} = s_w = \frac{0.322}{0.41} \times 1.0 = 0.79 \approx 1.00 \text{ cm}$$

$$\therefore \text{take } s_w = 1.00 \text{ cm.}$$

It quit clear that the effect of the bracking force is minor for both the gusset plate and the fillet weld the error is less than 2%. Hence no need to take its effect into consideration.

Example (8.3):

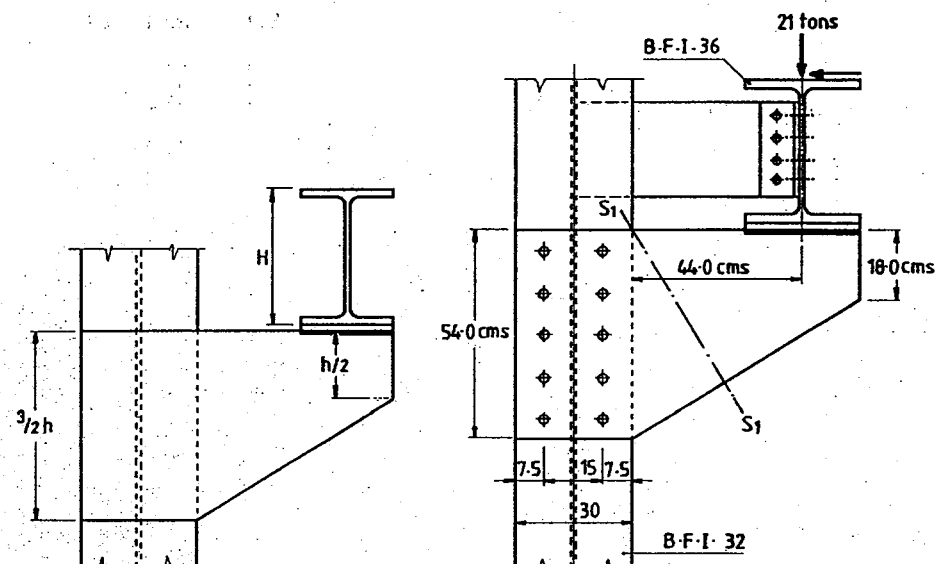
Design the bracket shown in the figure below to resist a total vertical load $V = 21.0$ tons and $H = 1.60$ ton. The breaking force = 0.357 tons. Design the bracket considering the fatigue effect of the applied dynamic loads.

Solution(1) Preliminary Dimensioning:

The dimensions of the bracket can be assumed as shown below. The thickness of the bracket is taken equal to the column flange thickness. Check Section $S_1 - S_1$ as in Example (8.2).

(2) Bolts Connecting Bracket to Column of Grade (10.9)- Friction

Use H.S.B.'s: Try two rows of 5 bolts each at 108 mm vertical pitch and 150 mm horizontal pitch as shown in the figure.



Straining Actions:

$$M_x = 0.357 \times (36 + 1 + 27) = 22.85 \text{ t.cm.} \quad \text{B.F. (Neglect)}$$

$$M_v = 0.357 \times (44 + 15) = 21.06 \text{ t.cm.} \quad \text{B.F. (Neglect)}$$

$$M_t = 21 \times (44 \times 15) = 1239 \text{ t.cm}$$

$$Q_y = 21.0 \text{ tons} \quad ; \quad N = 0.357 \text{ ton B.F. (Neglect)}$$

Properties of Area:

$$I_x = 2 \times 54^3 \times 30 / 12 = 787320 \text{ cm}^4.$$

$$I_v = 2 \times 30^3 \times 54 / 12 = 243000 \text{ cm}^4$$

$$\therefore T_1 \text{ (due to } M_x) = \frac{22.85 \times 21.60}{787320} \times 15 \times 10.80 = 0.1 \text{ tons}$$

B.F. (Neglect)

$$\therefore T_2 \text{ (due to } M_y) = \frac{21.06 \times 7.50}{243000} \times 15 \times 10.80 = 0.107 \text{ tons.}$$

B.F. (Neglect)

$$\therefore T_{\text{ext}, M} = T_1 + T_2 = 0.207 \text{ tons.}]$$

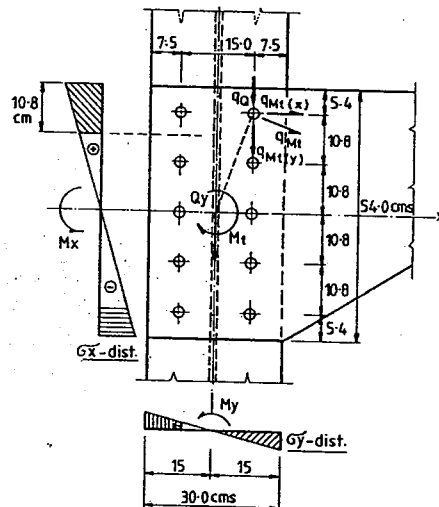
$$\therefore T_{\text{ext.N}} = 0.357/20 = 0.015 \text{ tons}$$

$$\therefore T_{\text{total}} = 0.207 + 0.015 = 0.22 \text{ tons}$$

B.F. (Neglect)

$$q_{0\downarrow} = 21/20 = 1.05 \text{ ton.}$$

$$q_{Mt\downarrow} = \frac{1239 \times 7.50}{2 \times 4 \times (7.5^2 + 13.15^2 + 22.87^2)}$$



= 1.544 ton.

$$Q_{Mt}^{\downarrow} = \frac{1239 \times 21.60}{2 \times 4 \times (7.5^2 + 13.15^2 + 22.87^2)}$$

= 4.45 tons

$$\therefore Q_{\text{total}} = \sqrt{(4.45+1.05)^2 + (1.544)^2}$$
$$= 5.713 \text{ tons.}$$

$$\therefore Q_b = 5.713 \text{ tons.}$$

For H.S.B. of the friction type behaving in Shear the fatigue phenomena has no effect. The high strength bolts are to be checked as in static loads.

∴ Choose H.S.B. M 20 Type (10.9), $P_s = 5.877$ tons

$$Q_h = 5.713 < 5.877 \text{ tons} \quad (\text{O.K.})$$

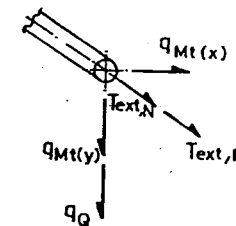
For H.S.B. (tension), $F_{sr} = 2.50 \text{ t/cm}^2$ Table 8.3

$$\therefore \Delta T = \frac{1}{10} T_{\text{ext}} = 0.1 \times 0.22 = 0.022 \text{ ton. B.F. (Neglect)}$$

$$\therefore \Delta f_b = \frac{\Delta T}{A_{\text{bolt}}} = \frac{0.022}{2.45} = 0.00 \text{ t/cm}^2 < 2.5 \text{ t/cm}^2 \text{ (Table 8.3)}$$

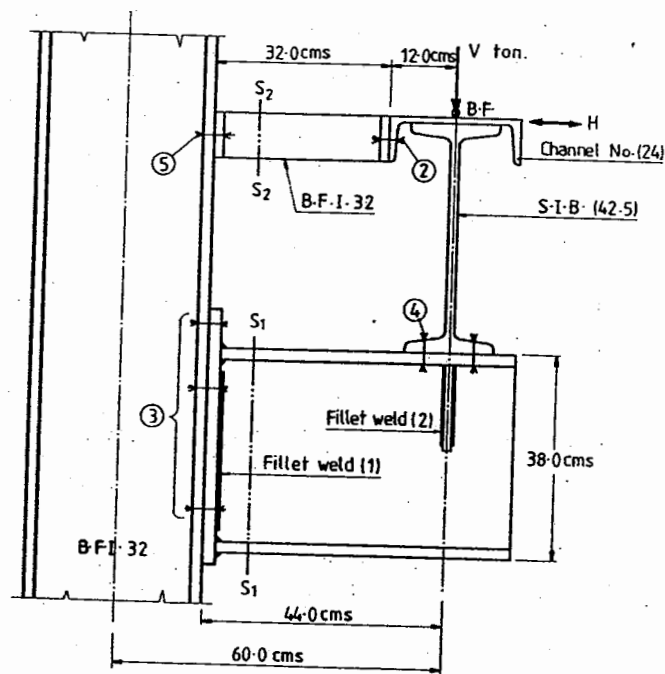
B.F. (Neglect)

∴ very safe



**Example (8.4):**

Design Bracket "1" shown in the figure below considering the fatigue effect. Neglect the effect of the braking force "B.F."



$$V_{D.L.} = 1.0 \text{ ton}$$

$$H_{D.L.} = 0.0$$

$$V_{total} = 21 \text{ tons}$$

$$H = 1.60 \text{ ton (transmitted directly to beam-column).}$$

$$B.F. = 0.30 \text{ ton (Neglect its effect on straining actions).}$$

The values of forces for dead load alone are negligible compared to the total forces. We can assume that the straining



actions due to dead load alone vanish. Thus the maximum allowable stress for each part of the bracket will be the value of the allowable stress range for fatigue effect.

N.B. For such type, the fillet welds joining the bracket to the end-plate are critical with respect to fatigue stress range which means that the design of the bracket must start with these welds.

Fatigue Effect: (E.C.P. 2001 - Chapter 4):

$$\text{No. of cycles} = 2 \times 10^6$$

Detail Classes and Stress Range (Table 8.2 ,8.3 and 8.4):

Bracket Component	Item No.	Class	Stress Range $F_{sr} \text{ t/cm}^2$
Fillet weld (1)	23.3	E'	0.41
Sec. $S_1 - S_1$	14	C	0.91
H.S.B. - friction.	-	-	-
H.S.B. (tension) - 10.9	table 8.3	-	2.5
End-Plate	14	C	0.91
Stiffeners	-	-	-
Fillet weld(2)	23.3	E'	0.41
Sec. $S_2 - S_2$	1	A	1.66

Preliminary Design of B.F.I. Section:

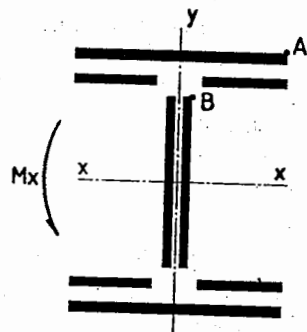
Section $S_1 - S_1$: The critical stress range is that of the fillet welds (1) which equals 0.41 t/cm^2 . Thus, in choosing the B.F.I. section of the bracket, the weld inertia which resists the predominant moment ($M_x = 21 \times 44 = 924 \text{ t.cm}$) is the main factor that governs that choice.

The fillet weld thickness is chosen to be the maximum allowable according to specifications. Thus, weld sizes around the flanges and web are equal to the flange and web thicknesses, respectively. The weld inertia is a ratio of the B.F.I. section inertia.

$$Z_{x(\text{weld shown in figure below})} \cong (1.57 \rightarrow 1.60) Z_{x \text{ B.F.I}}$$

$$M_x = 21 \times 44 = 924 \text{ t.cm}$$

The most critical point will be point (B) which is affected by both normal and shear stresses. Thus, we shall reduce the maximum allowable stress by 35% to take into account shear stresses at (B).



$$\frac{M_x}{Z_{xw}} \cong 0.41 \times 0.65 \rightarrow Z_{x \text{ req. (weld)}} = 3467 \text{ cm}^3$$

65 % to take effect of shear at pt. (B)

$$\therefore Z_{x \text{ B.F.I}} \cong \frac{3467}{1.57} = 2208.3 \text{ cm}^3 \text{ use B.F.I 38 } (Z_x = 2680 \text{ cm}^3)$$

Relation between welds and I-section

Use B.F.I. # (38)

(1) Section ($S_1 - S_1$) (Class A):

$$M_x = 21 \times 44 = 924 \text{ cm.t}$$

$$Q_y = 21.0 \text{ tons}$$

Point (A):

$$f_b = \frac{M_x}{Z_x} = \frac{924}{2680} = 0.344 \text{ t/cm}^2$$

$$< 0.91 \text{ t/cm}^2$$

$\therefore$ Safe (Class A)

Point (B):

$$f_b = \frac{M_x}{Z_x} \times \left(\frac{19-2.4}{19} \right)$$

$$= \frac{924}{2680} \times \frac{(19-2.4)}{19} = 0.301 \text{ t/cm}^2$$

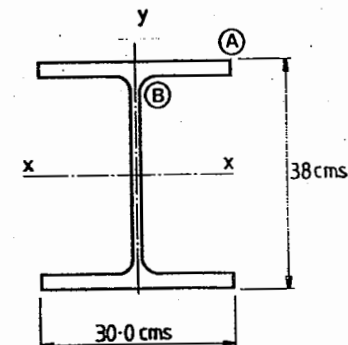
$$q_{Q\downarrow} = \frac{Q}{A_{\text{web}}}$$

$$= \frac{21}{(38 - 2 \times 2.4) \times 1.40} = 0.451 \text{ t/cm}^2$$

$$\therefore f_{\text{eq.}} = \sqrt{(0.301)^2 + 3(0.451)^2} = 0.837 \text{ t/cm}^2$$

$$< 0.91 \text{ t/cm}^2$$

$\therefore$ Safe (Class A)





(2) Weld "1" (Class E'):

To ensure safety of the fillet welds use weld size equal to the flange thickness around the flanges and web thickness around the web

Approximate Analysis

$$T = M / d = 924 / (38 - 2.4) = 26 \text{ tons}$$

$$L_w \text{ (around flange)} = 30 + 2 \times 11.9 = 53.8 \text{ cms}$$

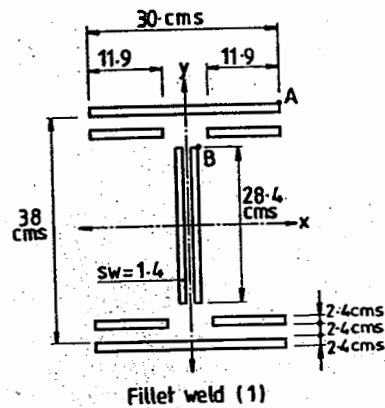
$$q = 26 / (53.8 \times 2.4) = 0.2 \text{ t/cm}^2$$

$$< 0.41 \text{ t/cm}^2$$

∴ Safe (Class E')

Exact Analysis

$$\begin{aligned} \therefore I_x &= 2 \times \left(1.4 \times \frac{28.4^3}{12} \right) \\ &+ 4 \times (11.9 \times 2.4) \\ &\times \left(38 - \frac{3}{2} \times 2.4 \right)^2 \\ &+ 2 \times (30 \times 2.4) \\ &\times \left(38 + \frac{2.40}{2} \right)^2 \\ &= 91195 \text{ cm}^4 \end{aligned}$$

Point (A):

$$f_b = \frac{924 \times (19 + 2.4)}{91195} = 0.217 \text{ t/cm}^2$$

$$< 0.41 \text{ t/cm}^2$$

∴ Safe (Class E')

Point (B):

$$f_b = \frac{924 \times 14.20}{91195} = 0.143 \text{ t/cm}^2$$

$$q_{Q\downarrow} = \frac{21}{2 \times 28.40 \times 1.40} = 0.264 \text{ t/cm}^2$$

$$\therefore f_{\text{princip.}} = \frac{f_b}{2} + \sqrt{\left(\frac{f_b}{2}\right)^2 + (q)^2}$$

$$= 0.33 \text{ t/cm}^2 < 0.41 \text{ t/cm}^2$$

∴ Safe (Class E')

(3) High Strength Bolts (see Table 8.3):

Preliminary Design: ($M_x \gg M_y$)

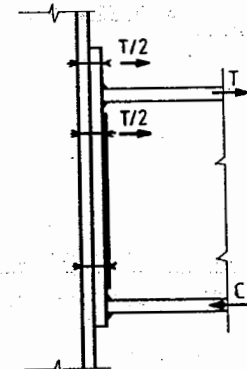
$$T = \frac{M_x}{d} = \frac{924}{(38 - 2.40)} = 26.0 \text{ tons}$$

(Using four bolts per row)

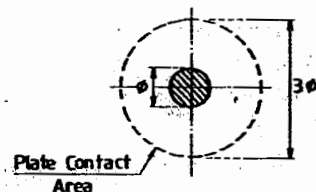
$$\therefore T_{\text{bolt}} = \frac{26}{8} = 3.2 \text{ tons}$$

Use H.S.B. of the (10.9) type

$$M16 \quad T_i = 9.89 \text{ tons}$$



To evaluate the stress range on the bolt shaft, the value of increase in the bolt pretension due to the external tension on the bolt must be obtained.



$$\text{i.e. } T_{\text{ext},b} = \Delta T + \Delta C$$

$$\therefore \frac{\Delta T}{\Delta C} = \frac{A_b}{A_p} = \left(\frac{\pi \phi^2}{4} \right) / \left(\frac{\pi (3\phi)^2}{4} \right) = \frac{1}{9}$$

$$\text{i.e. } \Delta T \approx \frac{1}{10} T_{\text{ext},b}$$

Approximate design :-

A simple design consists to use 2 rows around the tension flange of 4 bolts each [M16 (10.9) $A_b = 1.57 \text{ cm}^2$]

$$T_{\text{bolt}} = 26 / 8 = 3.25 \text{ tons}$$

$$\Delta T = 0.1 \times 3.25 = 0.325 \text{ tons}$$

$$\Delta f_{\text{bolt}} = 0.325 / 1.57 = 0.2 \text{ t/cm}^2 < 2.5 \text{ t/cm}^2$$

(Fatigue - Table 8.3)

Exact Analysis

(A) If We Use Four Bolts Per Row:

The most stressed bolt shall be bolt "1", due to the moments

M_x

* Due to Moment " M_x ":

$$f_1 = \frac{924 \times (47/2)}{30 \times (47)^3 / 12} = 0.083 \text{ t/cm}^2$$

$$f_2 = 0.083 \times \frac{15.1}{23.50} = 0.054 \text{ t/cm}^2$$

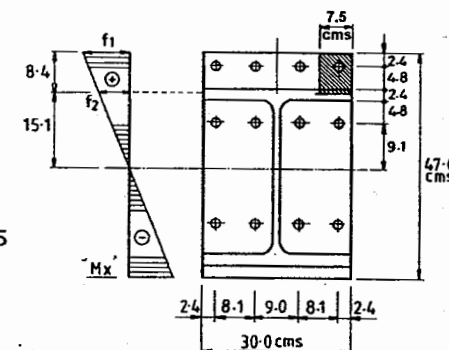
$$\therefore T = \frac{0.083 + 0.054}{2} \times 8.40 \times 7.5 = 4.31 \text{ tons}$$

$$\Delta T = 0.10 \times 4.3 = 0.431 \text{ ton}$$

$$\Delta f_{\text{bolt}} = \frac{\Delta T}{A_b} = \frac{0.431}{1.57} = 0.275 \text{ t/cm}^2$$

$$< 2.5 \text{ t/cm}^2$$

(Fatigue - Table 8.3)



(B) If We Use Two Bolts Per Row:

$$\therefore T = 4.31 \times \frac{15}{7.50} = 8.62 \text{ tons}$$

(due to M_x)

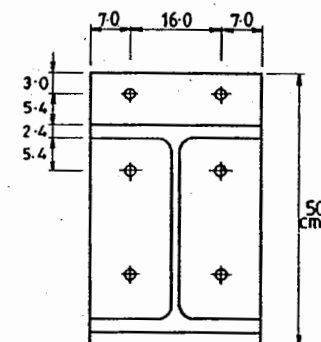
$$\therefore T_{\text{ext},b} = 8.62 \text{ tons}$$

$$\therefore T_{i \text{ req.}} = \frac{8.62}{0.80} = 10.77 \text{ tons.}$$

$\therefore$ Use bolts M20 (10.9)

Where ($T_i = 15.43 \text{ tons}$).

$$\therefore \Delta T = 0.10 \times 8.62 = 0.862 \text{ tons.}$$





$$\therefore \Delta F_{\text{bolt}} = \frac{\Delta T}{A} = \frac{0.862}{2.45}$$

$$= 0.351 \text{ t/cm}^2$$

$$< 2.5 \text{ t/cm}^2 \quad (\text{Fatigue - Table 8.3})$$

(4) End-Plate (Class C):

We shall get the tension corresponding to the right upper two bolts (1 & 2) due to M_x and M_y .

$$T_{\text{flange}} = \frac{M_x}{d} = \frac{924}{(38 - 2.4)} = 26.0 \text{ tons}$$

$$\therefore M_e = \frac{T \times 12}{8} = 39 \text{ t.cm.}$$

$$\therefore T_{ep} = \sqrt{\frac{6M}{b f_{\text{all.}}}} = \sqrt{\frac{6 \times 39}{30 \times 0.91}} = 2.91 \text{ cms.}$$

Where $F_{sr} = 0.91 \text{ t/cm}^2$ (Class C)

$$\therefore \text{Take } T_{ep} = 3.0 \text{ cms.}$$

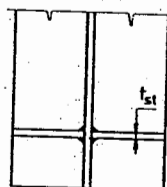
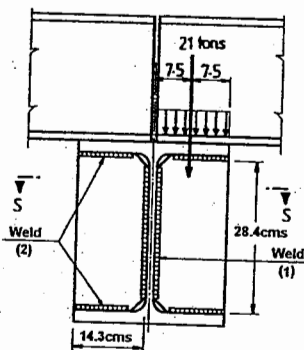
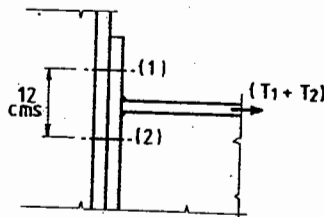
(5) Stiffeners - No Fatigue effect :-

$$F_{\text{one stiff.}} = V = 21.0 \text{ tons}$$

$$\text{Take } b_{st.} \leq \frac{30 - 1.40}{2} \leq 14.3 \text{ cms}$$

$$\therefore A_{\text{stiff}} = b_{st.} \times t_{st.} = \frac{21.0}{1.4} = 15 \text{ cm}^2$$

$$\therefore t_{st.} = \frac{15}{14.3} = 1.04 \text{ cm.}$$



Sec. S - S



$$\therefore \text{Take } t_{st.} = 1.1 \text{ cm.}$$

$$\frac{b_{st.}}{t_{st.}} = \frac{14.30}{1.1} = 13 < 16 \text{ Safe} \quad (b_{st.}/t_{st.} \leq 25/\sqrt{F_y})$$

(6) Welds of Stiffeners (Fillet Weld 2) - Class (E'):

$$F_{sr} = 0.41 \text{ t/cm}^2$$

$$\text{height of lines (1)} = 38 - 4 \times 2.4 = 28.4 \text{ cms}$$

$$\text{length of lines (2)} = 15 - 1.4/2 - 2.4 = 11.9 \text{ cms}$$

$$\text{Lines (1) - class E' (23.3)} \quad F_{sr} = 0.41 \text{ t/cm}^2$$

The vertical lines are assumed to resist the vertical force $V = 21$ tons.

$$s = \frac{21}{2 \times 28.4 \times 0.41} = 0.9 \text{ cms} \quad \text{take } s = 1.0 \text{ cm.}$$

$$\text{Lines (2) - class E' (23.3)} \quad F_{sr} = 0.41 \text{ t/cm}^2$$

These weld are resisting the force "F" due to the eccentricity of $V = 21$ ton where $M = 21 \times 7.5 = 157.5 \text{ t.cm}$

$$F = \frac{21.0 \times 7.5}{(38.0 - 2 \times 2.4)} = 4.74 \text{ tons}$$

$$s = \frac{4.74}{2 \times 11.9 \times 0.41} = 0.48 \text{ cms} \quad \text{take } s = 0.5 \text{ cms.}$$

(7) Bolts Group (4): $F_{sr} = 2.5 \text{ t/cm}^2$ (H.S.B. 10.9 - Table 8.3)

Class (F):

$$M_x = B.F. \times (0.95 + 42.50) \\ = 13.035 \text{ t.cm}$$

Use High Strength Bolts:

$$f = \frac{13.035 \times 15}{16.3 \times 30^3 / 12}$$

$$= 0.0053 \text{ t/cm}^2$$

$$\therefore T_{\text{bolt}} = \frac{1}{2} \frac{(0.0053)}{2} \times 16.3 \times 15 \\ = 0.326 \text{ ton}$$

$$\therefore \Delta T = 0.10 \times 0.326 = 0.0326 \text{ ton}$$

$$\therefore \Delta f_{\text{bolt}} = \frac{\Delta T}{A_b} < 2.5 \text{ t/cm}^2$$

$$\therefore A_s = \frac{0.0326}{2.5} = 0.013 \text{ cm}^2$$

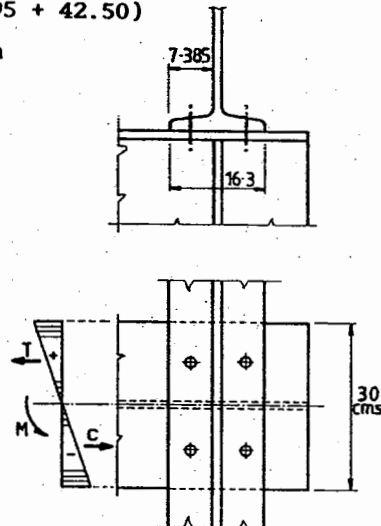
$\therefore$ Use M12 type (10.9) - $A_s = 0.84 \text{ cm}^2$ - $P_s = 1.83 \text{ tons}$

Check on Shear (No fatigue calculations for friction type) :-

$$Q = \frac{0.30}{4} = 0.075 \text{ tons} < P_s = 1.83 \text{ tons}$$

(8) Bolts Group (2) and (5) $F_{sr} = 2.5 \text{ t/cm}^2$ - Table 8.3 :-

For construction allowance use H.S.B's M12 type (10.9) on both sides.



$$M = H \times \frac{8.50}{2} = 1.60 \times \frac{8.50}{2} = 6.80 \text{ t.cm}$$

$$N = H = \pm 1.60 \text{ ton}$$

$$f_1 = \frac{M}{Z} = \frac{6.80 \times (8.50/2)}{30 \times (8.5)^3 / 12} \\ = 0.0188 \text{ t/cm}^2$$

$$\therefore T_{\text{ext},M} = \frac{1}{2} \left(\frac{0.0188}{2} \right) \times 30$$

$$\times \frac{8.50}{2} = 0.60 \text{ ton.}$$

$$\therefore T_{\text{ext},N} = \frac{N}{\text{no.}} = \frac{1.60}{4} = 0.40 \text{ ton}$$

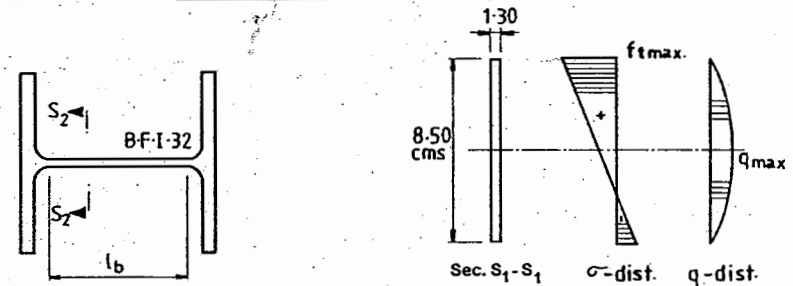
$$T_{\text{ext},M} + T_{\text{ext},N} = 0.60 + 0.40 = 1.0 \text{ ton}$$

$$\Delta T = \frac{1}{10} T_{\text{ext},(M+N)} = 0.10 \text{ ton}$$

$$\Delta f_{\text{bolt}} = \frac{\Delta T}{A_s} = \frac{0.10}{0.84} = 0.12 \text{ t/cm}^2 < 2.5 \text{ t/cm}^2 \text{ (H.S.B. 10.9 - Table 8.3)}$$

- N.B. Consequently Group of bolts (5) is safe.

(9) Sec. ($S_2 - S_2$) Class (A):



$$N = \pm 1.60 \text{ ton.}; \quad M = 6.80 \text{ t.cm}$$

$$\begin{aligned} f_{\text{act.}} &= \frac{N}{A} + \frac{M}{Z_x} \\ &= \frac{1.60}{8.5 \times 1.30} + \frac{6 \times 6.80}{1.3 \times (8.5)^2} \\ &= 0.58 \text{ t/cm}^2 < 1.4 \text{ t/cm}^2 \text{ (Static)} \\ &< 1.68 \text{ t/cm}^2 \text{ (Stress Range)} \end{aligned}$$

$$\therefore l_b = 32 - 4 \times 2.2 = 23.20 \text{ cms.}$$

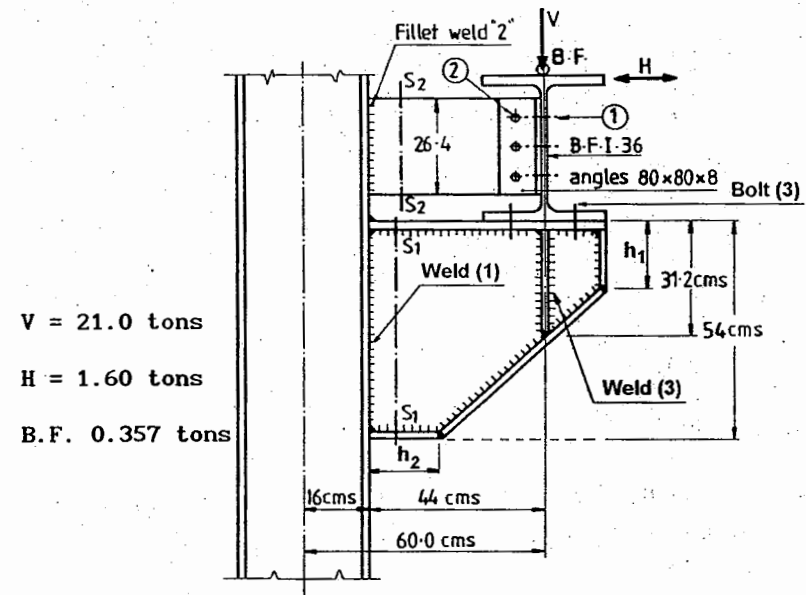
$$\begin{aligned} i_{\text{min.}} = i_y &= \sqrt{\frac{I_y}{A}} = \sqrt{\frac{8.5 \times (1.3)^3}{12 \times (8.5 \times 1.3)}} \\ &= 0.38 \text{ cm.} \end{aligned}$$

$$\therefore \frac{l_b}{i_{\text{min.}}} = \frac{23.20}{0.380} = 61.82$$

$$\begin{aligned} f_{\text{pb}} &= 1.40 - 0.000065 \left[\frac{l_b}{i_{\text{min.}}} \right]^2 \\ &= 1.152 \text{ t/cm}^2 > f_{\text{act.}} \therefore \text{Safe} \end{aligned}$$

Example (8.5):

Design Bracket "2" shown in the figure below taking the fatigue effect into consideration.

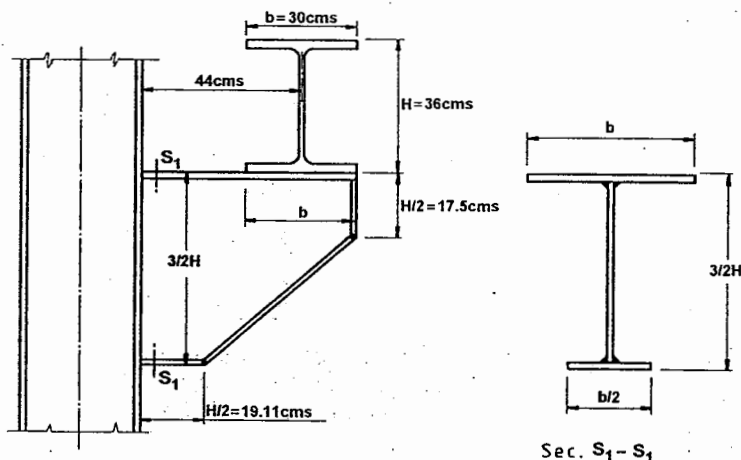


Item	Sec. S ₁ -S ₁	Sec. S ₂ -S ₂	Weld (1)	Weld (2)	O.B. (2)	H.S.B (3)	Stiff-ener	Weld (3)
Class	C (14)	C (14)	E' 23.3	E' 23.3	D 27.2	Table 8.3	--	E' 23.3
F _{sr} t/cm ²	0.91	0.91	0.41	0.41	0.71	2.5	--	0.41

The same stress ranges for bracket "1" of Ex. 8.4 are used here except for that of section S₁-S₁ (steel plate with transverse fillet weld) which will behave according to category (C) Item 14 with F_{sr} = 0.91 t/cm² while in Ex. 8.4 the connection was made with bolts and hence the corresponding Class was (A).

Preliminary Dimensioning:

First of all, the dimensions are determined according to the previous experience as follows:



The following check of stresses will be done by the approximate method (A) previously outlined in Section 8.5.

(1) Vertical Section ($S_1 - S_1$):

($F_{sr} = 0.91 \text{ t/cm}^2$) Class (C):

$$\bar{y} = \frac{15 \times 2 \times 53 + 50 \times 1 \times 27 + 60 \times 1}{(30 \times 2 + 50 \times 1 + 15 \times 2)}$$

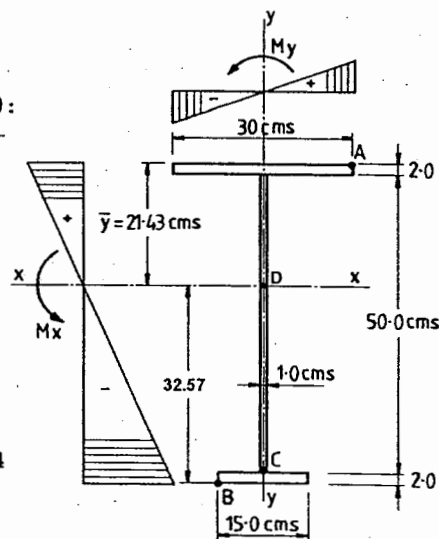
$$= 21.43 \text{ cms}$$

$$I_x = 30 \times 2 \times (20.43)^2 + 1 \times \frac{(50)^3}{12}$$

$$+1 \times 50 \times (5.57)^2 + 15 \times 2 \times$$

$$(53-21.43)^2 = 66911 \text{ cm}^4$$

$$I_y = 2 \times \frac{30^3}{12} = 4500 \text{ cm}^4$$



Sec. S₁ - S₁

$$J = \frac{1}{3} \sum a t^3 = \frac{1}{3} (30 \times 2^3 + 15 \times 2^3 + 50 \times 1^3) = 136.67 \text{ cm}^4$$

Straining Actions:

$$M_x = 21 \times 44 = 924 \text{ cm.t}$$

$$Q_y = 21.0 \text{ tons}$$

$$M_y = 0.357 \times 44 = 15.7 \text{ cm.t (resisted by upper flange only)}$$

$M_t = 0.357 \times (36 + 21.43) = 20.5 \text{ cm.t}$ (Its effect is neglected where the approximation of I_y is calculated by taking the upper falnge only)

Check on Stresses:

$$f = \frac{M_{x,y}}{I_x} + \frac{M_{y,x}}{I_y} \dots$$

$$\alpha_{Mt} = \frac{M_t \cdot t}{J} \quad - - - - -$$

$$q_Q \text{ (web only)} = \frac{Q.S_x}{I_{x.b}} - -$$

point A	point B	point C	point D
+0.348 t/cm ²	-0.449 t/cm ²	-0.422 t/cm ²	-- t/cm ²
$\xleftrightarrow{\quad}$ 0.3 t	$\xleftrightarrow{\quad}$ 0.3 t	0.15 $\uparrow \downarrow$	0.15 $\uparrow \downarrow$
—	—	0.0095 $\downarrow$	0.444 $\downarrow$

$$\text{Point (A): } f_{\text{princ.}} = \frac{0.348}{2} + \sqrt{\left[\frac{0.339}{2}\right]^2 + (0.3)^2} = 0.52 \text{ t/cm}^2 < 0.91$$

$$\text{Point (B): } f_{\text{princ.}} = \frac{0.449}{2} + \sqrt{\left[\frac{0.449}{2}\right]^2 + (0.3)^2} = 0.59 \text{ t/cm}^2 < 0.91$$

$$\text{Point (C): } f_{\text{princ.}} = \frac{0.422}{2} + \sqrt{\left[\frac{0.422}{2}\right]^2 + (0.0095 + 0.15)^2}$$

$$= 0.471 \text{ t/cm}^2 < 0.91$$

$$\therefore s_2 = \frac{5.79}{2 \times (15 - 0.5 - 2) \times 0.41} = 0.56 \text{ cm} \rightarrow s_2 = 0.6 \text{ cms}$$

Weld (3): Class E' (23.3) $\therefore F_{sr} = 0.41 \text{ t/cm}^2$

$$\therefore s_3 = \frac{5.79}{2(7.50 - 0.5 - 2) \times 0.41} = 1.41 \text{ cm} \rightarrow s_3 = 1.5 \text{ cms}$$

The stiffener thickness is to be increased to 1.5 cms also

(6) Welds Between Bracket and Column [Class (E')]:

The weld section is as shown:

$$A = 30 \times 2 + 13.5 \times 2 \times 2 + 2 \times 46$$

$$+ 12 \times 2 + 15 \times 2 = 260 \text{ cm}^2$$

$$\bar{y} = \frac{1}{260} (2 \times 13.5 \times 2 \times 3 + 2 \times 46 \times 1 \times 27$$

$$+ 12 \times 2 \times 51 + 15 \times 2 \times 55 -$$

$$30 \times 2 \times 1) = 21.0 \text{ cms}$$

$$I_x = 30 \times 2 \times 22^2 + 27 \times 2 \times 18^2 + 2 \times \frac{46^3}{12}$$

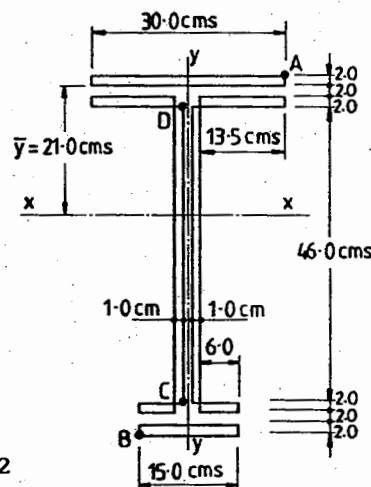
$$+ 2 \times 46 \times 6^2 + 12 \times 2 \times 30^2 + 15 \times 2 \times 34^2$$

$$= 122350 \text{ cm}^4$$

$$I_y = 2 \times 2 \times \frac{30^3}{12} + 2 \times 1 \times 46 \times 1.5^2 + 2 \times 2 \times \frac{15^3}{12}$$

$$- 1 \times 2 \times \frac{3^3}{12} = 10329 \text{ cm}^4$$

$$I_p = I_x + I_y = 132679 \text{ cm}^4$$



Straining Actions:

$$M_x = 21 \times 44 = 924 \text{ cm.t}$$

$$M_y = 0.357 \times 44 = 15.7 \text{ cm.t}$$

$$M_t = 0.357 \times (36 + 21.43) = 20.5 \text{ cm.t}$$

$$Q_y = 21.0 \text{ tons} \downarrow$$

Check on Stresses:

$$f = \frac{M_x y}{I_x} + \frac{M_y x}{I_y}$$

$$q_{Mt} = \frac{M_t \cdot r}{I_p}$$

$$q_Q \text{ (web only)} = \frac{Q}{A_{web}}$$

point A	point B	point C	point D
+ 0.19 t/cm ²	- 0.27 t/cm ²	- 0.22 t/cm ²	+ 0.13 t/cm ²
0.0035t	0.0055t	0.0044t	0.0026t
—	—	0.228t _↓	0.228t _↓

$$f_{\text{princ. max.}} = \frac{f_t}{2} + \sqrt{\left(\frac{f_t}{2}\right)^2 + (q)^2} < 0.35 \text{ t/cm}^2$$

$$f_{p.(A)} = 0.19 \text{ t/cm}^2 < 0.41 \text{ t/cm}^2$$

$$f_{p.(B)} = 0.27 \text{ t/cm}^2 < 0.41 \text{ t/cm}^2$$

$$f_{p.(C)} = 0.36 \text{ t/cm}^2 < 0.41 \text{ t/cm}^2$$

$$f_{p.(D)} = 0.30 \text{ t/cm}^2 < 0.41 \text{ t/cm}^2$$

(7) Bolts "3", $F_{sr} = 2.5 \text{ t/cm}^2$ - Table 8.3 :-

Use High Strength Bolts (10.9) :-

$$M_x = 0.357 \times 36 = 12.85 \text{ cm.t (B.F. effect)}$$

$$f_t = \frac{12.85 \times 6}{30 \times (30)^2} = 0.00285 \text{ t/cm}^2$$

$$T_{\text{bolt}} = \frac{0.00285}{2} \times 30 \times \frac{15}{2} = 0.32 \text{ tons}$$

$$\therefore \Delta T = 0.032 \text{ tons}$$

$$A_s = 0.84 \text{ cm}^2 \text{ (M 12)}$$

$$\Delta f_{\text{bolt}} = \frac{\Delta T}{A_s} < 2.5 \text{ t/cm}^2$$

$$\therefore [A_s]_{\text{req}} = 0.0128 \text{ cm}^2 \text{ choose M 12 (} A_s = 0.84 \text{ cm}^2 \text{)}$$

where: $T_i = 5.3 \text{ tons}$ $P_s = 1.83 \text{ tons}$ (Case II)

$$\therefore Q_{\text{bolt}} = \frac{0.357}{4.0} = 0.089 \text{ tons} < 1.83 \text{ (Safe) (static)}$$

(8) Bolts "1": $F_{sr} = 2.5 \text{ t/cm}^2$ (10.9) - Table 8.3 :-

Use High Strength Bolts:

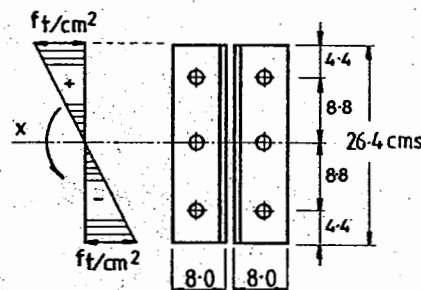
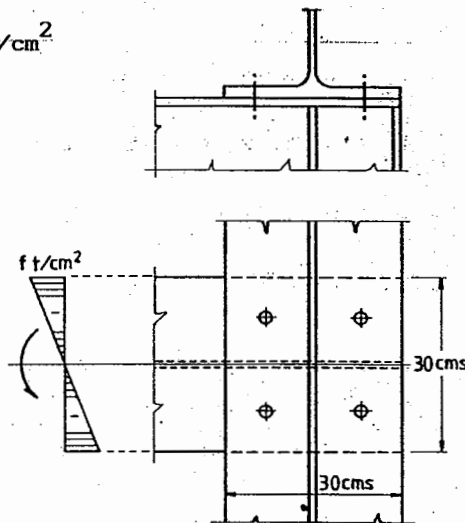
$$N = H = 1.60 \text{ ton}$$

$$M_x = 1.60 \times \frac{36}{2} = 28.80 \text{ t.cm}$$

From the construction condition

$$d \leq (8.8/3 = 2.9 \text{ cms})$$

$$f = \frac{28.80 \times (26.4/2)}{16 \times (26.4)^3/12} = 0.0155 \text{ t/cm}^2$$



$$T_{\text{ext},M} = \frac{0.0155}{2} \times 13.2 \times 8 = 0.82 \text{ ton}$$

$$T_{\text{ext},N} = \frac{1.60}{6} = 0.266 \text{ ton}$$

$$\text{Total } T_{\text{ext.}} = 0.82 + 0.266 = 1.086 \text{ ton}$$

$$\therefore \Delta T = 0.1 \times 1.086 = 0.1086 \text{ ton}$$

$$\therefore \Delta f_{\text{bolt}} = \frac{\Delta T}{A_s} < 2.5 \text{ t/cm}^2$$

$$\therefore A_b < \frac{0.1086}{2.5} = 0.043 \text{ cm}^2$$

$\therefore$ Use M12 H.S.B. Type (10.9) where $A_s = 0.84 \text{ cm}^2$.

(9) Bolts "2": [Category "D", item 27.2] $F_{sr} = 0.71 \text{ t/cm}^2$

$\therefore$ we can use Non-pretensioned bolts for this group.

Straining Actions:

$$\rightarrow Q_{Mt} = \frac{1.6 \times 18 \times 8.8}{2 \times 8.8^2} = 1.64 \text{ ton}$$

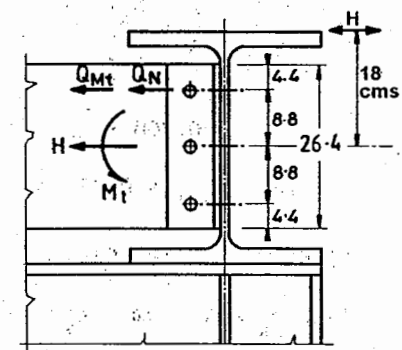
$$\rightarrow Q_N = \frac{1.60}{3} = 0.53 \text{ ton}$$

$$\rightarrow Q_{\text{total}} = 1.64 + 0.53 = 2.17 \text{ tons}$$

$$\therefore q_{\text{bolt}} = \frac{2.170}{\frac{2 \times \pi (1.6)^2}{4}} = 0.54 \text{ t/cm}^2 < 0.71 \text{ t/cm}^2 (F_{sr})$$

Safe

$\therefore$ Use Non-pretensioned bolts M16



(10) Section ($S_2 - S_2$) $F_{sr} = 0.91 \text{ t/cm}^2$, Class (C) - Item (14)

$$N = \pm 1.60 \text{ ton}$$

$$M = 1.60 \times (36/2) = 28.80 \text{ cm.t.}$$

$$l_b = 44 - 3.6 - 0.80 - 0.70 = 38.90 \text{ cms}$$

$$\frac{l_b}{t} \leq 44 \quad \therefore t \geq \left(\frac{38.90}{44} \right) = 0.88 \text{ cm}$$

$\therefore$ Take $t = 1.0 \text{ cm}$.

$$i_{\min.} = i_y = \sqrt{\frac{I_y}{A}} = \sqrt{\frac{26.40 \times (1)^3 / 12}{26.4 \times 1.0}} = 0.288 \text{ cms}$$

$$\therefore \frac{l_b}{i_{\min}} = 134.75$$

$$\therefore F_c = \frac{7500}{(134.75)^2} = 0.413 \times 1.2 \text{ t/cm}^2$$

$$\therefore f_{\text{act}} = \frac{N}{A} + \frac{M_x \cdot y}{I_x} = \frac{1.60}{26.4 \times 1.0} + \frac{28.8 \times (26.4/2)}{1 \times (26.4)^3 / 12}$$

$$= 0.308 < 0.91 \text{ t/cm}^2 \quad \therefore \text{ (Safe)}$$

$$\text{Also } f_{\text{act}} < 0.413 \times 1.2 \text{ t/cm}^2 \text{ (safe)}$$

(11) Fillet Welds, Class (E'), $F_{sr} = 0.41 \text{ t/cm}^2$

$$\text{Take } s_w = 1.0 \text{ cm} \quad A_w = 1 \times 2 \times 26.4 = 52.80 \text{ cm}^2$$

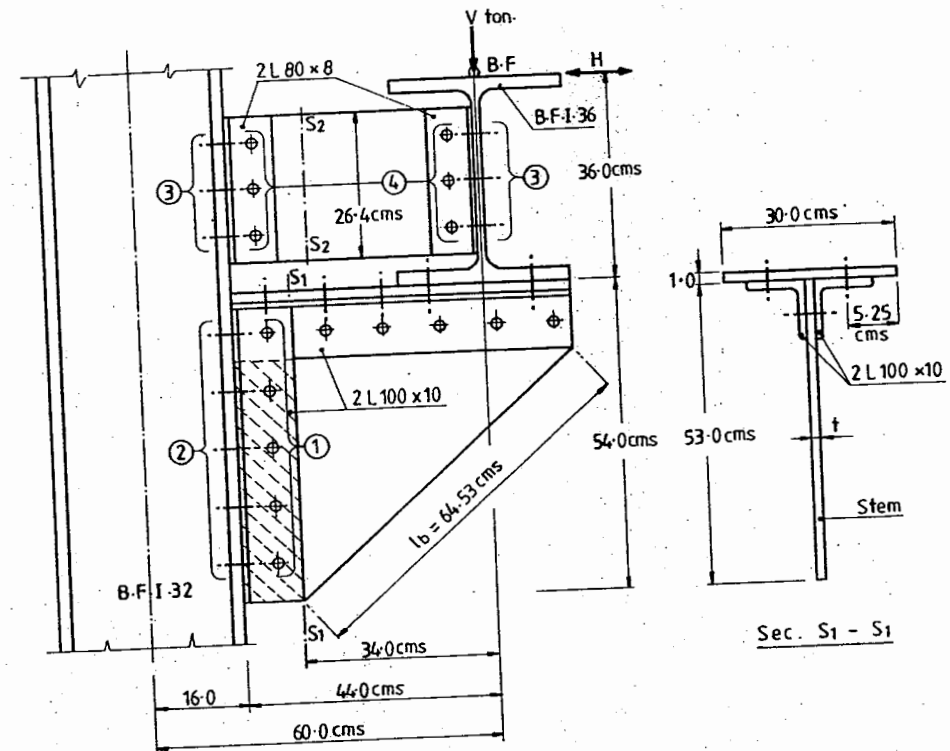
$$I_x = 2 \times 1 \times (26.4)^3 / 12 = 3066.6 \text{ cm}^4$$

$$\therefore f_{\text{max.}} = \frac{1.60}{52.80} + \frac{28.80 \times (26.4/2)}{3066.66} = 0.154 \text{ t/cm}^2 < F_{sr} \text{ Safe}$$

Example (8.6):

Design Bracket "3" shown in the Fig. below.

$$V = 21.0 \text{ tons} ; H = 1.60 \text{ tons} ; B.F. = 0.357 \text{ tons}$$



Item	Sec. $S_1 - S_1$	Sec. $S_2 - S_2$	H.S.B 10.9 (1)	H.S.B 10.9 (2) - (3)	O.B. (4)
Class	B	B	--	--	D
$F_{sr} \text{ t/cm}^2$	1.26	1.26	--	2.5	0.71
ITEM	2.1	2.1	--	Table 8:3	27.2

Preliminary Dimensioning:

Dimensions are chosen such that:

- * Bracket total height = $1\frac{1}{2}$ the height of C.T.G.
- * Breadth of top plate = 30 cms = breadth of C.T.G.
- * Stem thickness is chosen to satisfy the plate buckling conditions.

Estimation of Gusset Plate Thickness:

As has been computed in the previous examples, the exact analysis of stresses has shown to be the most critical regarding the gusset plate thickness. Hence it is recommended to start the design using the exact analysis.

(1) Analysis of Bracket Cross Section Class B - $F_{sr} = 1.26 \text{ t/cm}^2$

(A) Exact Analysis: Item 2.1

Buckling Condition:

$$a = 59 \text{ cms}, \quad h = 53 \text{ cms},$$

$$a/h = 1.113$$

$$k = 3.2 - 3.0(1.113) + 1.1(1.113)^2$$

$$= 1.21$$

$$t = 0.1 \sqrt[3]{\frac{21 \times 59}{1.21}} = 1.075 \text{ cms}$$

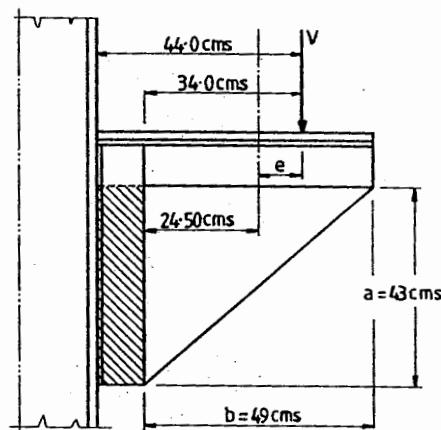
$$\text{take } t = 1.1 \text{ cms}$$

Yielding:

$$Z = 1.39 - 2.2 (1.113)$$

$$+ 1.27 (1.113)^2 - 0.25$$

$$\times (1.113)^3 = 0.17$$

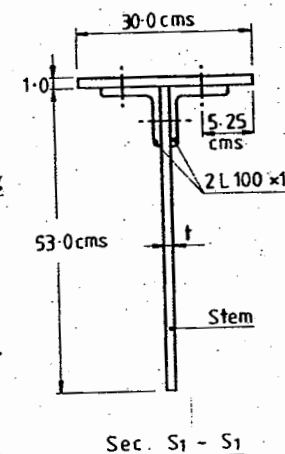


$$t = \frac{V}{F_{sr} \cdot a \cdot Z} = \frac{21}{1.26 \times 59 \times 0.17}$$

$$= 1.66 \text{ cms take } t = 1.7 \text{ cms}$$

(B) Approximate Analysis Considering Eccentricity:

The section comprising the gusset plate and the flange angles are assumed to resist the induced straining actions. However this assumption will lead to a lower bound solution.



Section ($S_1 - S_1$): Category "B" Item (2.1)

$$F_{sr} = 1.26 \text{ t/cm}^2$$

Properties of Area:

$$\bar{y} = \frac{30 \times 1 \times 0.50 + 2 \times 19.2 \times (1 + 2.82) + 53 \times 2 \times 27.50}{(1 \times 30 + 2 \times 19.2 + 2 \times 53.0)}$$

$$= 17.64 \text{ cms.}$$

$$I_x = 1 \times 30 \times 17.14^2 + 2 \left[177 + 19.20 \times (13.82)^2 \right] + 2 \times \frac{53^3}{12} + 2 \times 53 \times (9.86)^2$$

$$= 51620 \text{ cm}^4$$

$$I_y = 1 \times \frac{30^3}{12} + 2 \times \left[177 + 19.2(1.0 + 2.82)^2 \right] = 3164 \text{ cm}^4$$

For the calculation of I_y only the upper flange (30x1.0) and the 2 angles 100x100x10 have been taken into consideration.

$$S_{x(c)} = 30 \times 1 \times (17.66 - 0.5) = 514.8 \text{ cms}$$



$$J = \frac{1}{3} \sum at^3 = \frac{1}{3} \left[4 \times 10 \times 1^3 + 53 \times 2.0^3 + 30 \times 1^3 \right]$$

$$= 164.663 \text{ cm}^4$$

Straining Actions:

$$M_x = 21 \times 34 = 714 \text{ cm.t}$$

$$M_y = 0.357 \times 34 = 12.138 \text{ cm.t}$$

$$M_t = 0.357 \times (36 + 17.64) = 19.15 \text{ cm.t}$$

$$Q_y = 21 \text{ tons}$$

Point (A):

$$f_A = \frac{M_x}{I_x} \cdot y + \frac{M_y}{I_y} \cdot x$$

$$= \frac{714}{51620} \times 17.64 + \frac{12.138}{3164} \times 15 = 0.292 \text{ t/cm}^2$$

$$q_{Mt} = \frac{M_t \cdot t}{J} = \frac{19.15 \times 1.0}{164.66} = 0.116 \text{ t/cm}^2$$

$$\therefore f_{\text{princ. (A)}} = \frac{0.292}{2} + \sqrt{\left(\frac{0.292}{2}\right)^2 + (0.116)^2}$$

$$= 0.33 \text{ t/cm}^2 < 1.26 \text{ t/cm}^2$$

Point (B):

O.K.

$$f_B = \frac{M_x}{I_x} \cdot y = \frac{714}{51620} \times 36.36$$

$$= 0.503 \text{ t/cm}^2$$



$$q_{Mt} = \frac{19.15 \times 2.0}{164.66} = 0.23 \text{ t/cm}^2$$

$$\therefore f_{\text{princ (B)}} = \frac{0.503}{2} + \sqrt{\left(\frac{0.503}{2}\right)^2 + (0.23)^2}$$

$$= 0.59 \text{ t/cm}^2 \text{ (comp.)} < F_{sr} (1.26 \text{ t/cm}^2)$$

For point (C) proceed for the normal stresses and the shear torsional stresses as before only add the shear stresses due to $Q_y = 21$ tons.

$$q_{C|y} = \frac{21 \times 514.8}{51620 \times 4} = 0.0523 \text{ t/cm}^2$$

The principal stresses for point (C) are less than 1.26 t/cm^2

If the vertical section $S_1 - S_1$ is taken just at the attachment with the column flange. Hence the straining action are:

$$M_x = 21 \times 44 = 924 \text{ t.cms}$$

$$M_y = 0.357 \times 44 = 15.7 \text{ t.cms}$$

$$Q_y = 21 \text{ tons}; M_t = 0.357 \times (36 + 17.64) = 19.15 \text{ t.cms}$$

Point (A):

$$f_A = \frac{924 \times 17.64}{51620} + \frac{15.7 \times 15}{3164} = 0.388 \text{ t/cm}^2 \text{ (o.k.)}$$

$$q_{Mt} = \frac{19.15 \times 1}{164.66} = 0.116 \text{ t/cm}^2$$

$$f_{\text{princ. (A)}} = \frac{0.388}{2} + \sqrt{\left(\frac{0.388}{2}\right)^2 + (0.116)^2} = 0.42 \text{ t/cm}^2$$

$$< 1.26 \text{ (o.k.)}$$



The bolts are double shear bolts.

$$F_R / \text{Surface} = 14.04 / 2$$

$$= 7.02 \text{ tons}$$

Use H.S.B.'s of diameter 22mm (10.9) type - $P_s = 7.26 \text{ tons}$
(O.K.)

(3) Bolt's Group (2) - $F_{sr} = 2.5 \text{ t/cm}^2$ (H.S.B. 10.9) Table 8.3 :

$$I_x = \frac{20 \times 51^3}{12} = 221085 \text{ cm}^4$$

$$I_y = \left[51 \times \frac{10^3}{12} + 51 \times 10 \times 7.25^2 \right] \times 2$$

$$= 62114 \text{ cm}^4$$

Straining Actions:

$$M_x = 21 \times 44 = 924 \text{ t.cm}$$

$$M_y = 0.357 \times 44 = 15.7 \text{ t.cm}$$

$$M_t = 0.357 \times (25.5 + 3 + 36)$$

$$= 23.0 \text{ t.cm}$$

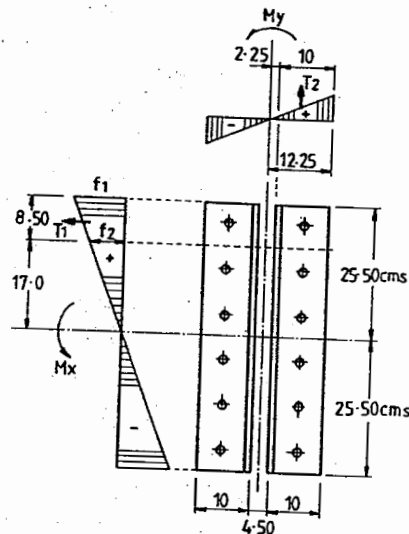
$$Q_y = 21.0 \text{ tons}$$

Normal Stresses:

* Due to M_x :

$$f_1 = \frac{924 \times 25.50}{221085} = 0.1066 \text{ t/cm}^2$$

$$f_2 = 0.071 \text{ t/cm}^2$$



* Due to M_y :

$$f_1 = \frac{15.7 \times 12.25}{62114} = 0.003 \text{ t/cm}^2$$

$$\therefore T_1 (\text{due to } M_x) = \frac{0.1066 + 0.071}{2} \times 8.5 \times 10 = 7.55 \text{ tons}$$

$$\therefore T_2 (\text{due to } M_y) = 0.003 \times 8.5 \times 10 = 0.255$$

$$\therefore T_{\text{ext.}} = 7.55 + 0.255 = 7.805 \text{ tons}$$

$$\therefore T_i > \frac{7.805}{0.80} > 9.75 \text{ tons}$$

Use M20 Where $T_i = 15.43 \text{ tons}$

$$\therefore \Delta T = 0.1 \times 7.805 = 0.7805 \text{ ton}$$

$$\therefore \Delta F = \frac{\Delta T}{A_s} = \frac{0.7805}{2.45} = 0.32 \text{ t/cm}^2 < 2.5 \text{ t/cm}^2 (F_{sr})$$

$\therefore$ Safe

Shear Stresses - No Fatigue Calculations :-

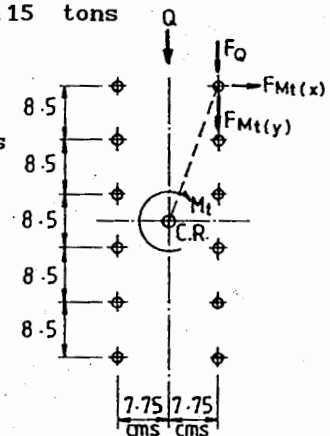
$$F_{Mt(x)} = \frac{23.0 \times 21.25}{4 [22.6^2 + 14.9^2 + 8.8^2]} = 0.15 \text{ tons}$$

$$F_{Mt(y)} = \frac{23.0 \times 7.75}{\sum r^2} = 0.0546 \text{ tons}$$

$$F_Q = \frac{21}{12} = 1.75 \text{ tons}$$

$$F_R = \sqrt{(1.75 + 0.0546)^2 + (0.15)^2}$$

$$= 1.81 \text{ tons}$$





$$\therefore F_R < 2 * P_s$$

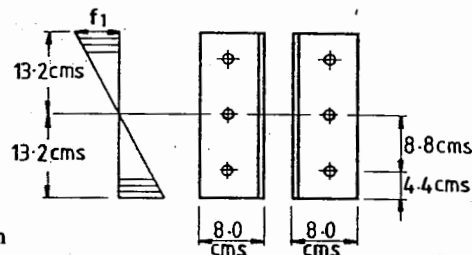
for M20 (10.9) H.S.B's as group (1) (Safe)

(4) Group (3) of Bolts:

$$F_{sr} = 2.5 \text{ t/cm}^2$$

$$N = \pm 1.60 \text{ ton}$$

$$M = 1.60 \times \frac{36}{2} = 28.80 \text{ t.cm}$$



* Use High Strength Bolts:

$$f_1 = \frac{M.y}{I} = \frac{28.80 \times 13.20}{16 \times (26.4)^3 / 12} = 0.016 \text{ t/cm}^2$$

$$T_{ext,M} = \frac{0.016}{2} \times 13.2 \times 8.8 = 0.93 \text{ ton}$$

$$T_{ext,N} = \frac{N}{\text{no. of bolts}} = \frac{1.60}{6} = 0.267 \text{ ton}$$

$$T_{ext,M} + T_{ext,N} < 0.8 T_i$$

$$\therefore T_i > 1.35 \text{ ton}$$

From the construction condition $d \leq 2.93 \text{ cm}$

$\therefore$ Use H.S.B's M 12

$$\therefore \Delta T = \frac{0.93 + 0.267}{10} = 0.1197 \text{ ton}$$

$$\therefore \Delta f = \frac{\Delta T}{A_s} = \frac{0.1197}{0.84} = 0.142 \text{ t/cm}^2 < 2.5 \text{ t/cm}^2 \text{ (Safe)}$$



(5) Group (4) of Bolts Class (D) - 27.2 - ($F_{sr} = 0.71 \text{ t/cm}^2$)

Ordinary bolts can be used for this group where:

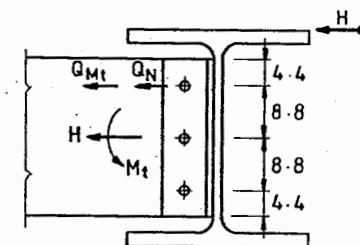
$$N = 1.60 \text{ ton}$$

$$M_t = 28.80 \text{ t.cm}$$

$$\therefore Q_{Mt} = \frac{28.80 \times 8.80}{2 \times (8.80)^2} = 1.64 \text{ ton}$$

$$\therefore Q_N = \frac{1.60}{3} = 0.53 \text{ ton}$$

$$\therefore Q_{total} = 2.17 \text{ tons}$$



Use (3) Non-pretensioned M16

$$\therefore q_{bolt} = \frac{2.170}{2 \times \pi (1.6)^2 / 4} = 0.54 \text{ t/cm}^2 < 0.71 \text{ t/cm}^2 (F_{sr})$$

$\therefore$ Safe

Example (8.7):

Redesign the bracket connection of the previous examples using a bolted end plate connection of the following types:

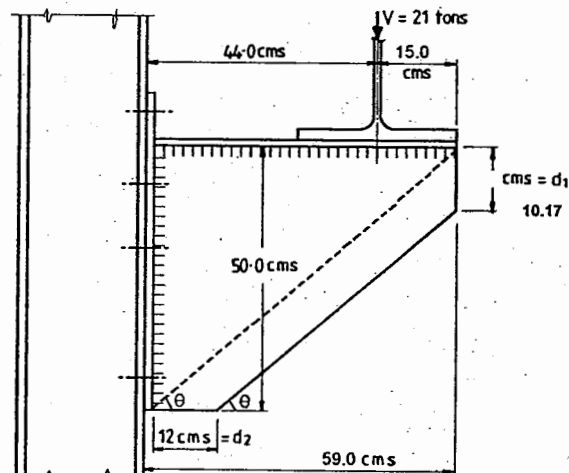
- (a) an extended end plate.
- (b) a flushed end plate.

Choose a convex shape for the gusset plate with $d_2 = 12 \text{ cms}$ and $d_1 = 10.17 \text{ cms}$ from inclination (θ).

It is required only to design the vertical support (i.e. the top flange and the gusset plate) where the other steel structural elements remain unchanged.

Solution

(a) Extended End Plate

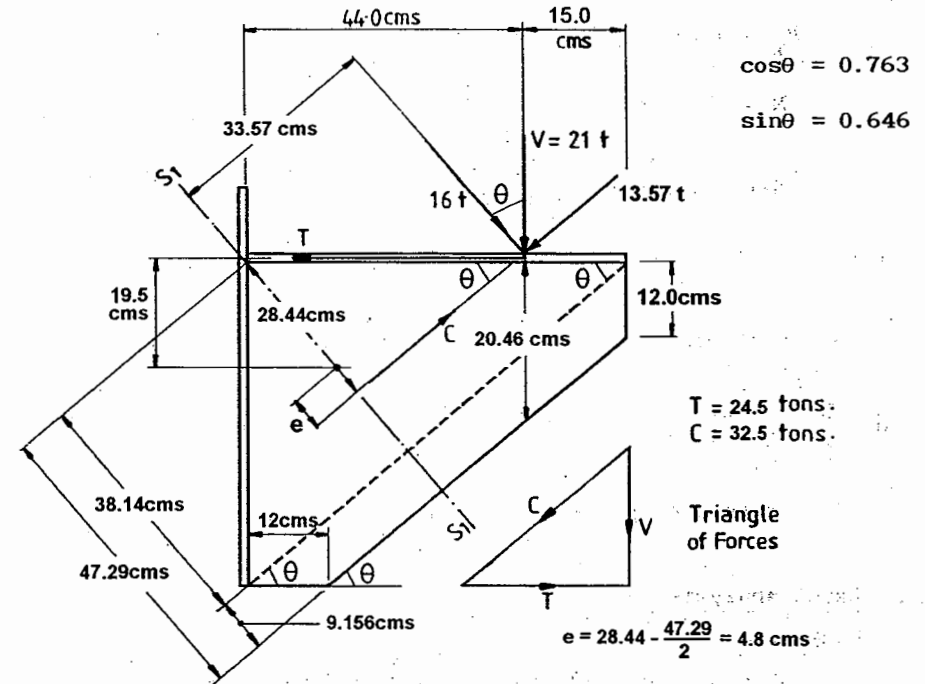


(1) Approximate Analysis Method (A)-Inclined Section ($S_1 - S_1$):

$$\text{Class (C) - 14 - (F}_{sr} = 0.91 \text{ t/cm}^2)$$

For the extended end plate configuration both the top flange bracket as well as the gusset plate will contribute to

resist the induced straining actions M_x and N . Hence, the approximate Method (A) can be applied.



From triangle of forces get:

$$T = 24.8 \text{ tons} , C = 32.5 \text{ tons}$$

Top flange bracket connection breadth = 30 cms.

Hence the thickness (t) of the upper flange can be determined using:

$$F_{sr} = 0.91 \text{ t/cm}^2$$

$$t = \frac{24.8}{30 \times 0.91} = 0.91 \text{ cms} \quad \text{take } t = 1.0 \text{ cm}$$



The straining actions in the gusset plate along section S_1-S_1 neglecting the effect of the braking force are:

$$M_x = 32.5 \times 4.8 = 156 \text{ t.cms}$$

$$N = 32.5 \text{ tons}$$

$$f_{\text{Max}} = \frac{32.5}{47.29 t} + \frac{160 \times 6}{t \times 47.29^2} \quad \text{put } F_{sr} = 0.91 \text{ t/cm}^2$$

Hence: $t = 1.22 \text{ cms}$; take $t = 1.3 \text{ cm}$.

Shear stresses under the vertical load $V = 21 \text{ tons}$.

$$q = \frac{3}{2} \times \frac{21}{1.3 \times 20.46} = 1.18 \text{ t/cm}^2 > 0.84 \text{ t/cm}^2 \text{ Unsafe (static)}$$

Take t (gusset plate) = 1.9 cms

(2) Exact Analysis

Buckling:

$$a = 59 \text{ cms} ; h = 50 \text{ cms} ; a/h = 59/50 = 1.18$$

$$k = 3.2 - 3.0 (1.18) + 1.1 (1.18)^2 = 1.19$$

$$t = 0.1 \sqrt{\frac{21 \times 59}{1.19}} \cong 1.07 \text{ cm.}$$

Yielding:

$$Z = 1.39 - 2.2 (1.19) + 1.27 (1.19)^2 - 0.25 (1.19)^3 = 0.152$$

$$t = \frac{21}{0.91 \times 59 \times 0.152} = 2.57 \text{ cms.}$$

Hence choose $t = 2.6 \text{ cms}$



(b) Flushed End Plate

Vertical Support of Bracket (Inclined Section $S_1 - S_1$):

Apply method (B) for the analysis where only the gusset plate is resisting the applied straining actions.

$$M_x = 16 \times 33.57 + 13.57 \times 4.8 = 602.25 \text{ t.cms}$$

$$N = 13.57 \text{ tons}$$

$$f_{\text{Max}} = \frac{13.57}{47.29 t} + \frac{602.25 \times 6}{t \times 47.29^2} \quad \text{put } F_{sr} = 0.91 \text{ t/cm}^2$$

$$t = 2.09 \text{ cms. Take } t = 2.1 \text{ cms}$$

$$q = \frac{3}{2} \times \frac{21}{47.29 \times 2.1} = 0.217 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2 \text{ (o.k.)}$$

Shear stresses under vertical load $V = 21.0 \text{ tons}$.

$$q = \frac{3}{2} \times \frac{21}{2.1 \times 20.46} = 0.733 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2$$

The buckling and yielding conditions will be the same as for the extended end plate.

Hence choose $t = 2.1 \text{ cms}$.

CHAPTER 9



CHAPTER 9

BASE CONNECTIONS AND SUPPORTS

9.1. Introduction:

Steel base connections and supports transmit the external applied loads to the concrete foundation and then to the soil.

This chapter deals with the conventional elastic analysis of the different types of steel base connections and supports. Simplifying assumptions are necessary in order to render the analysis and design procedures practical. Nevertheless, the approximate methods considered herein are, in most of the cases, conservative.

9.2. Main Types of Base Connections:

The main different types of base connections may be classified as follows:

- (a) The Hinged base connections shown in Fig.9.1.a. transmit both the axial and the shear reactions to the foundations via the different steel components comprising the connection.
- b) The Fixed base connections shown in Fig. 9.1.b. transmit axial, shear, and moment reactions. Depending on the values of the moments and the axial compressive reactions, the base can be either entirely or partially subjected to compressive normal stresses.
- (c) The Pocket base connections illustrated in Fig. 9.1.c. can be considered either hinged or fixed. Columns are directly embedded in the concrete footings.
- (d) The Grillage base connections shown in Fig. 9.1.d. are used either hinged or fixed. These types are mainly used for heavy axial reactions.

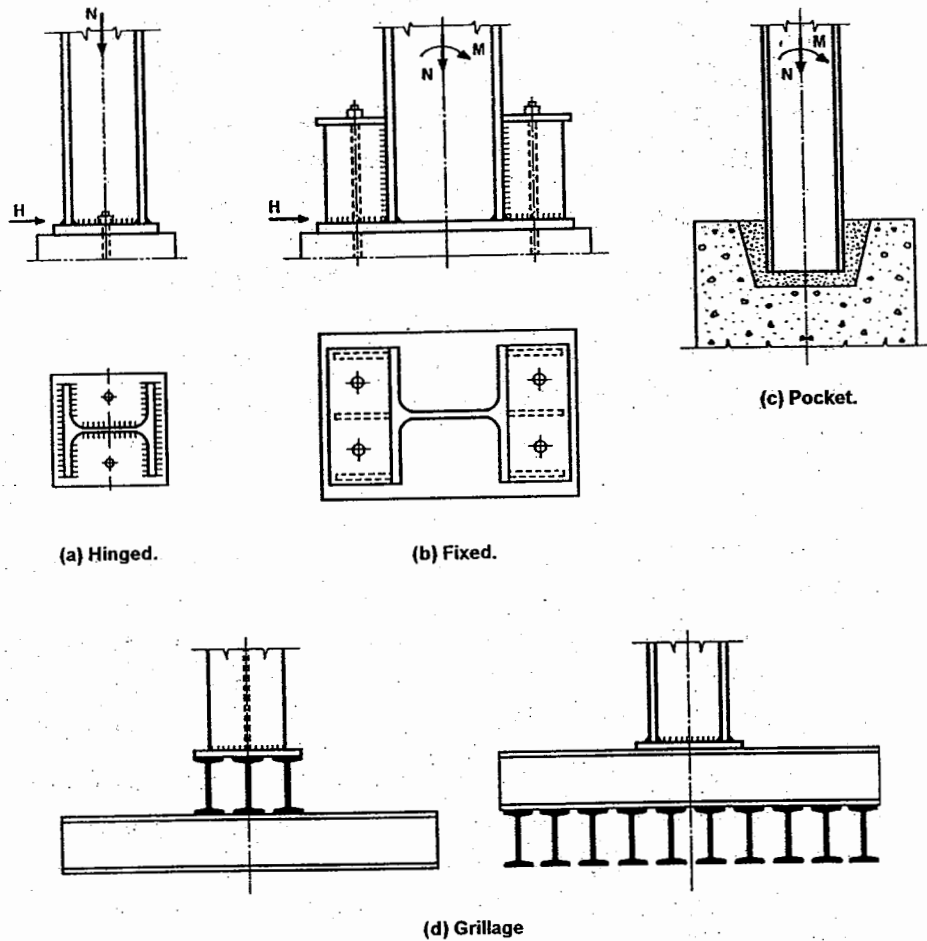


Fig. 9.1 Basic Types of Conventional Base Connections.

Figs. 9.2.a., b, illustrate the main different types of hinged and fixed base connections respectively. In the following sections each type will be analysed. The analysis and design of the basic conventional types. are based on realistic assumptions.

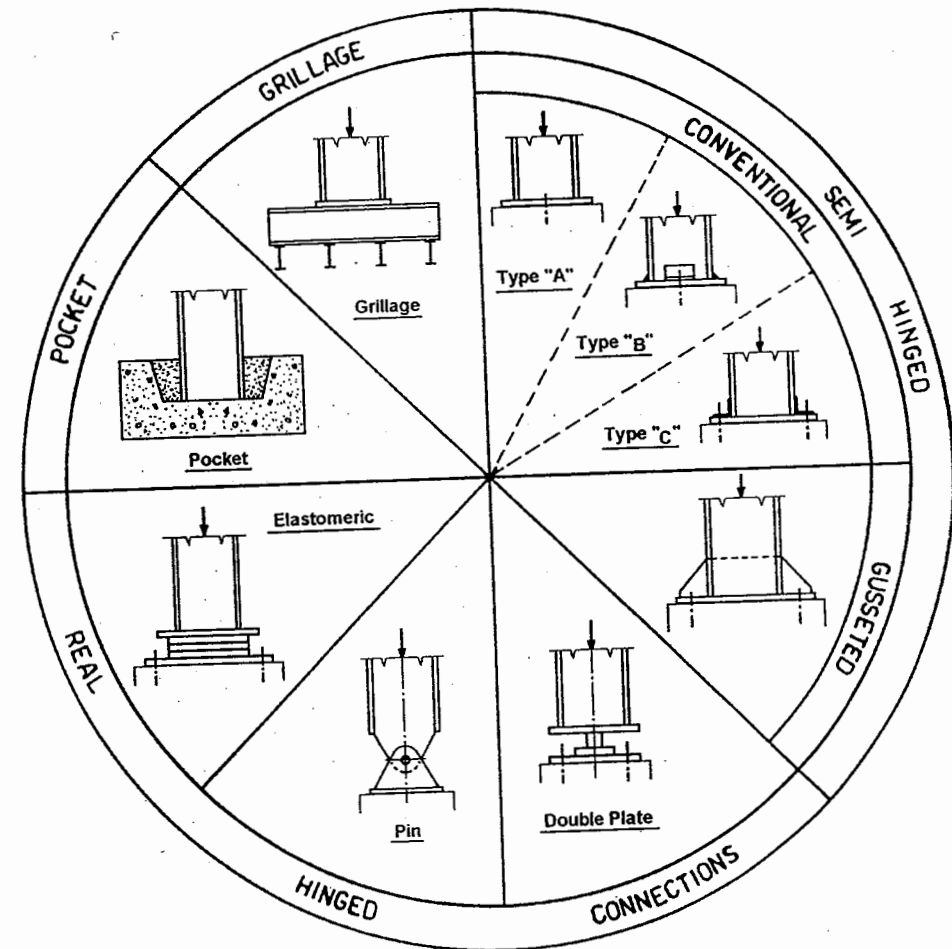


Fig. 9.2 Types of Hinged Base Connections.

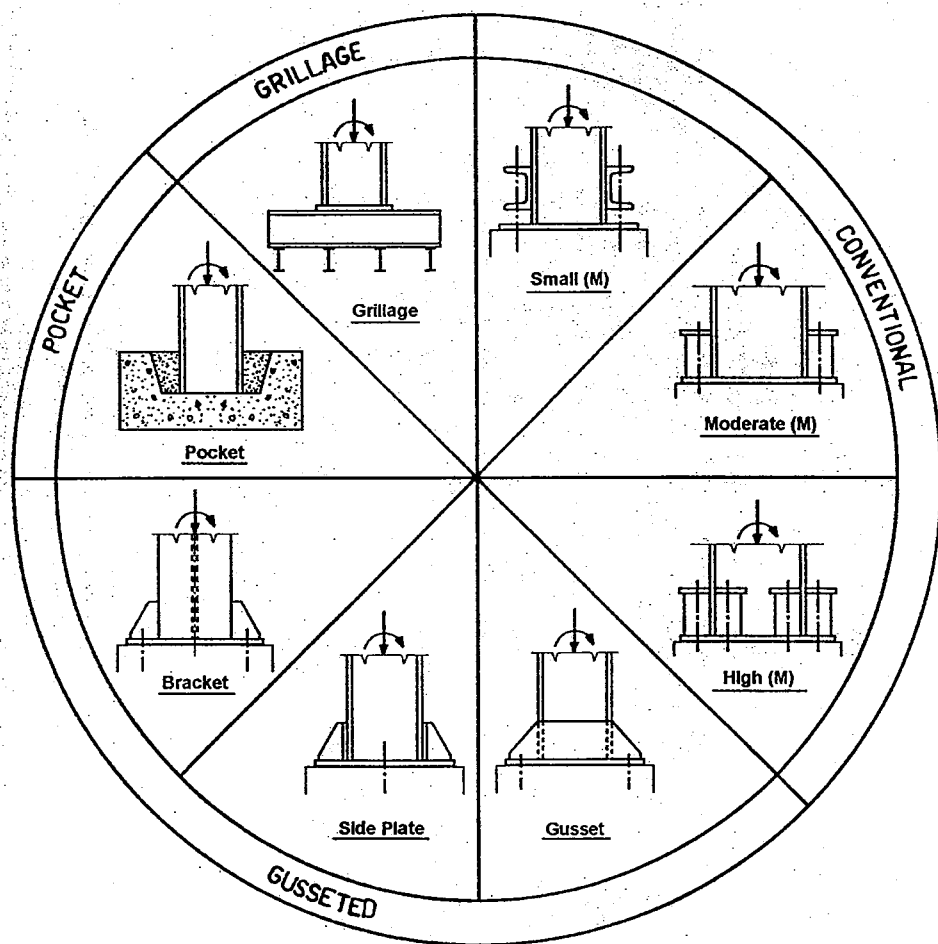


Fig. 9.3 Types of Fixed Base Connections.



9.3. Semi-Hinged Base Connections:

Base connections which are not purely hinged are termed as semi-hinged. The rotation is provided by the amount of the flexibility provided by the base plate. Generally, the transmitted moment is small and can be neglected.

9.3.1. Conventional Hinged Base Connection:

9.3.1.1. Different Types:

Fig. 9.3, shows three types of conventional hinged base connections. Type (A) is the simplest one for arc welded fabrication and is specially used for small vertical reactions. The base plate is first punched for anchor bolts then shop welded to the column.

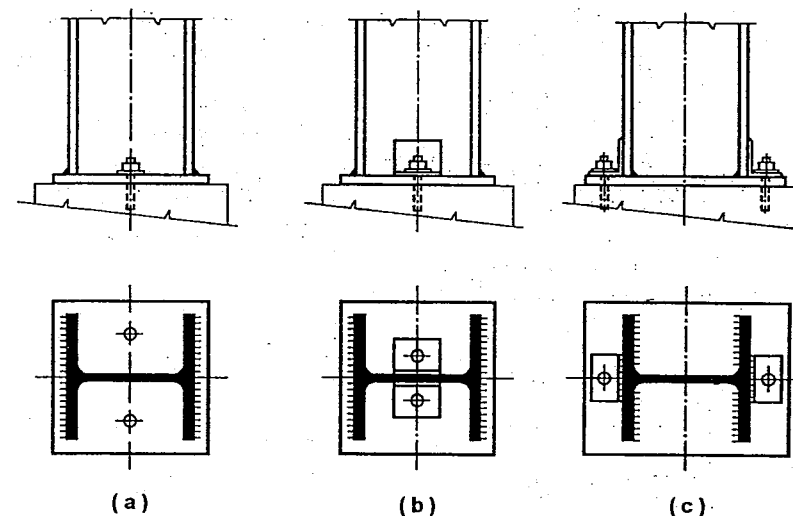


Fig. 9.4 Types of Conventional Hinged Base Connections.

On the other hand, types (B) and (C) are utilized when the column and the base plate are erected separately. The angles are welded to the base plate after erection.



For all the conventional types, if the end of the column is milled, which implies complete contact between column and base plate, there must be just sufficient welding to transmit horizontal shear and to hold all parts securely in place. If the column is not milled (i.e. incomplete contact), the connecting weld must be thick enough to safely withstand the compressive vertical and the horizontal reactions.

9.3.1.2. Method of Analysis:

In the following, the simplest conventional hinged base connection "Type A" is analyzed. Fig. 9.5. shows a free body diagram indicating all the relevant applied loads; the axial compressive load "N" and the horizontal shear "H".

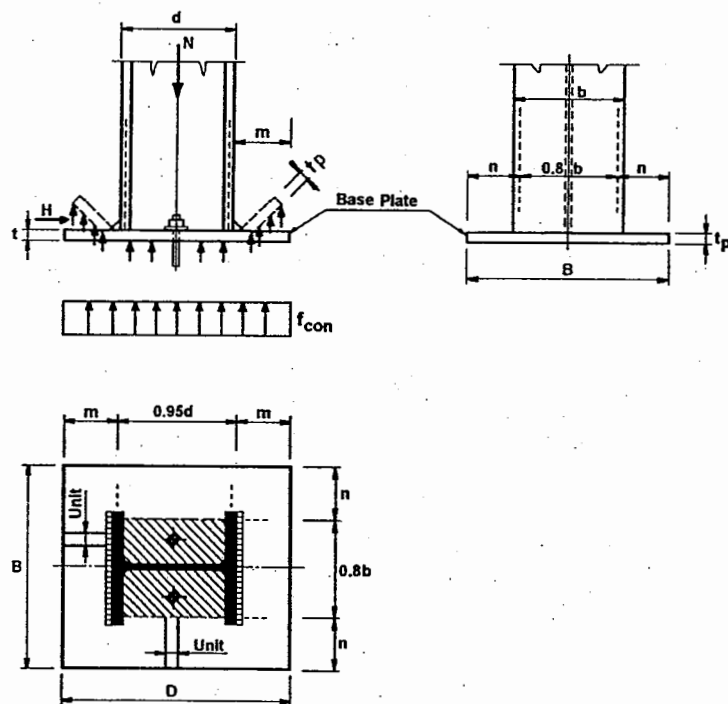


Fig. 9.5. Conventional Hinged Base Connection.



(a) Dimensioning of the Base Plate:

In the present method of analysis, it is assumed that the column axial force "N" will be uniformly transmitted to the base plate through the indicated shaded area ($0.95 d \times 0.8 b$). The applied stress is hence equal to $N / (0.95 d \times 0.8 b)$. Furthermore, the reactive stress applied on the concrete foundation is assumed to be uniformly distributed over the base plate area ($B \times D$). Therefore the actual compressive stress applied on the concrete footing, f_{con} , can be computed by the following relation:

$$f_{con} = \frac{N}{(B \times D)} \quad 9.1$$

Where B and D are the breadth and length of the base plate respectively. The induced stress f_{con} must not violate the limits prescribed by the R.C. Code of Practice of 1999.

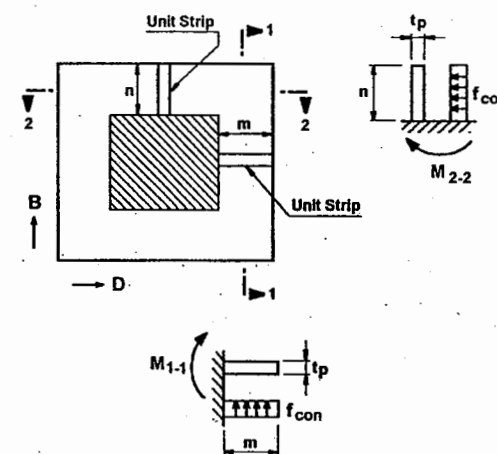


Fig. 9.6 Base Plate.

In order to determine the required plate thickness, further simplifications are necessary. The overhanged portions of the plate having the lengths m in the D -direction and n in the



For all the conventional types, if the end of the column is milled, which implies complete contact between column and base plate, there must be just sufficient welding to transmit horizontal shear and to hold all parts securely in place. If the column is not milled (i.e. incomplete contact), the connecting weld must be thick enough to safely withstand the compressive vertical and the horizontal reactions.

9.3.1.2. Method of Analysis:

In the following, the simplest conventional hinged base connection "Type A" is analyzed. Fig. 9.5. shows a free body diagram indicating all the relevant applied loads; the axial compressive load "N" and the horizontal shear "H".

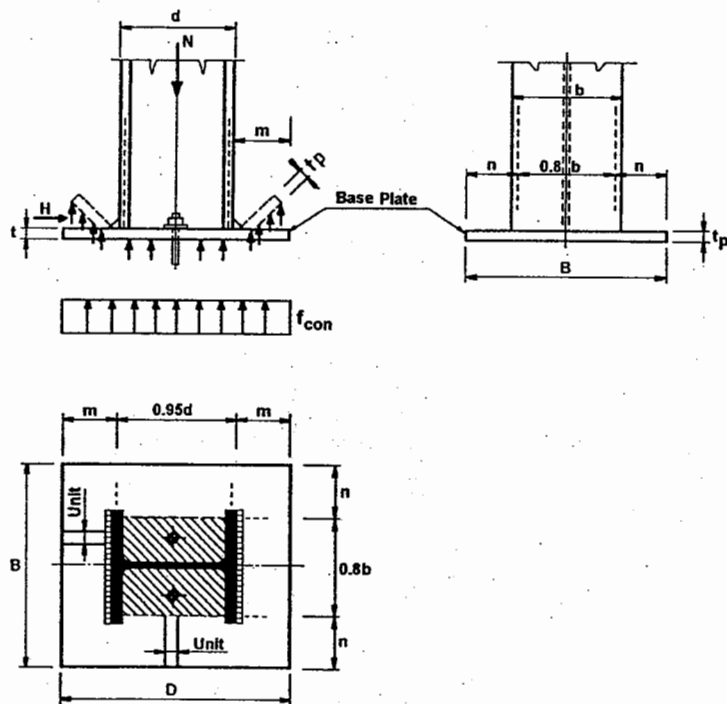


Fig. 9.5. Conventional Hinged Base Connection.



(a) Dimensioning of the Base Plate:

In the present method of analysis, it is assumed that the column axial force "N" will be uniformly transmitted to the base plate through the indicated shaded area (0.95 d X 0.8 b). The applied stress is hence equal to $N / (0.95 d \times 0.8 b)$. Furthermore, the reactive stress applied on the concrete foundation is assumed to be uniformly distributed over the base plate area (B X D). Therefore the actual compressive stress applied on the concrete footing, f_{con} , can be computed by the following relation:

$$f_{con} = \frac{N}{(B \times D)} \quad 9.1$$

Where B and D are the breadth and length of the base plate respectively. The induced stress f_{con} must not violate the limits prescribed by the R.C. Code of Practice of 1999.

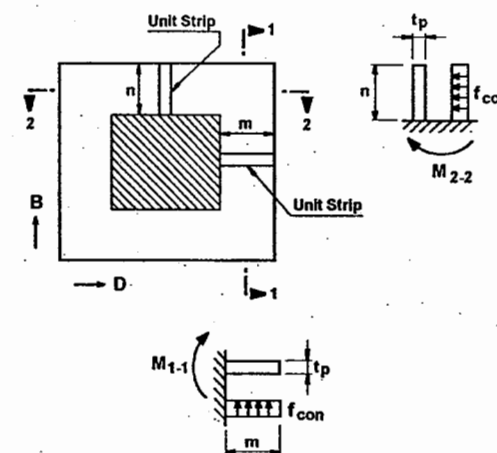


Fig. 9.6 Base Plate.

In order to determine the required plate thickness, further simplifications are necessary. The overhanged portions of the plate having the lengths m in the D-direction and n in the

B-direction are divided into unit strips. The problem becomes a one dimensional cantilever beam subjected to a uniformly distributed load of intensity f_{con} . The induced moments per unit length are:

$$\left. \begin{aligned} M_{1-1} &= f_{con} \cdot \frac{n^2}{2} \\ M_{2-2} &= f_{con} \cdot \frac{m^2}{2} \end{aligned} \right\} \quad 9.2$$

Where $m = (D - 0.95 d) / 2$; $n = (B - 0.8 b) / 2$.

The optimum solution is to choose m equal to n so that the induced moments M_{1-1} and M_{2-2} are equal in both directions. The thickness of the base plate, t_p , can be determined as follows :-

$$f_b = \frac{M \cdot Y}{I} ; I = \frac{1 \cdot t^3}{12} ; Y = \frac{t}{2} ; M = f_{con} \cdot \frac{m^2}{2}$$

$$\text{Hence: } t_p = m \sqrt{\frac{3 f_{con}}{F_b}} \quad 9.3$$

Where F_b is the allowable bending stress of the steel material for solid rectangular sections equals $(0.72 F_y)$ as given by clause 2.6.5.2. E.C.P 2001.

(b) Design of Anchor Bolts:

The main function of the anchor bolts is to transmit the shear reaction and the uplift force to the concrete footing. The design parameters are defined in the following:

d : diameter of the bolt.

L : embeded length in concrete footing.

n : number of bolts.

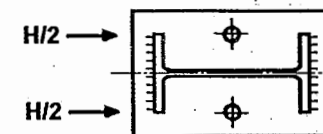


Fig. 9.7

The horizontal shear H is transmitted from the column to the base plate via the two side fillet welds, then, to the footings by anchor bolts. The diameter (d) can be computed from the following shear stress relation:

$$q = \frac{H}{2 (\pi d^2 / 4)} \quad 9.4$$

Where q = the allowable shear stress in bolt shank taken according to E.C.P. of steel 2001 (clause 6.4.1.).

It is to be noted that the horizontal reaction H may be transmitted partially or totally to the foundation by the friction force " H_f " between the base plate and the concrete according to the following:

$$H_f = \mu \cdot N \quad 9.5$$

where: μ = the coefficient of friction between steel and concrete.

When the column transmits only an axial compressive force the length of the bolt is taken equal to a minimum value of twenty times the diameter. In cases of uplift forces, the anchor bolts must have sufficient embeded length L inside the concrete footing in order to transmit the uplift force safely to the concrete by bond stress, according to the following criteria of plain reinforcing bars.

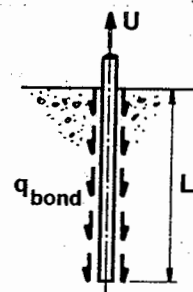


Fig. 9.8

$$L = \frac{U}{n_b \pi d q_b} \quad 9.6$$

Where U = Resultant uplift force.

(case of D.L. + W.L.).

q_b = Allowable bond stress between steel and concrete.

9.3.1.3. Results of Finite Element Analysis and Load Transmission Scheme:

In 1991 a research work (25) was conducted in the Steel Division of the structural Engineering Department namely "Stress analysis of base connections". A three dimensional finite elements program was prepared and utilized. All steel structural elements comprising the base connection as well as the reinforced concrete footing were included in the mathematical model. The three conventional hinged base connections (A), (B) and (C) shown in Fig. 9.2 as well as the gusseted type hinged base connection were analyzed.

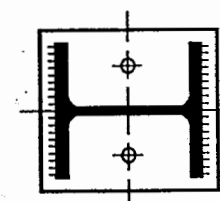
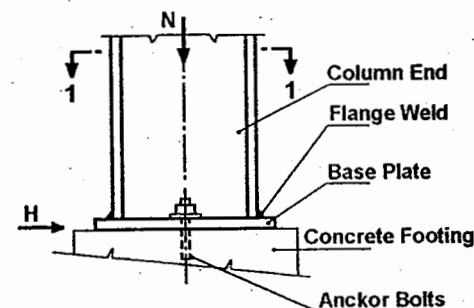
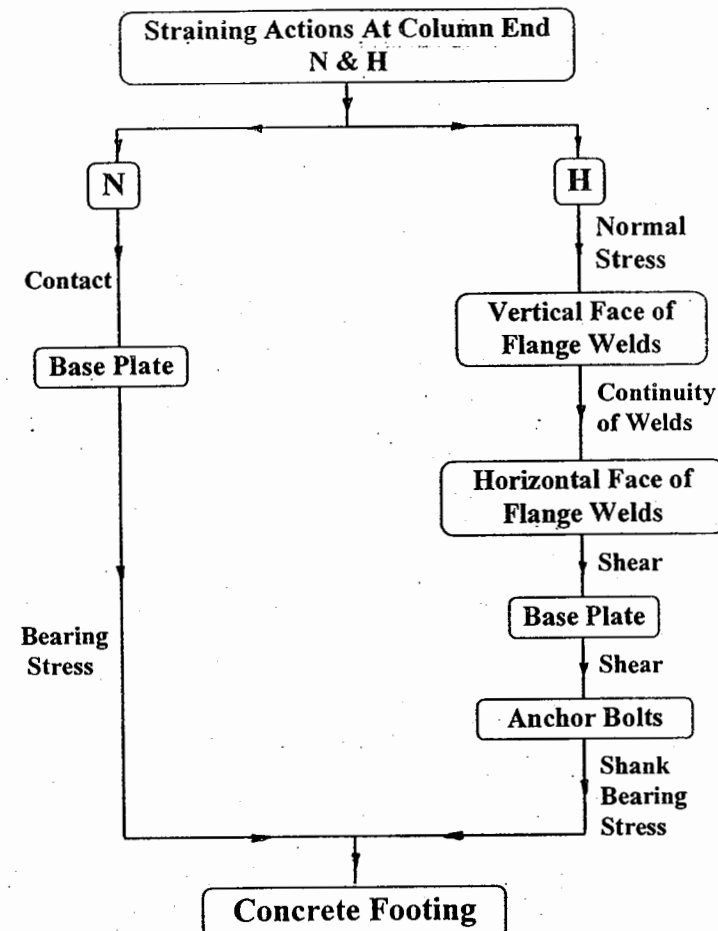


Fig. 9.9

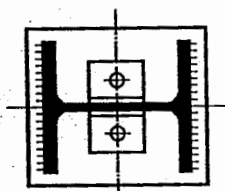
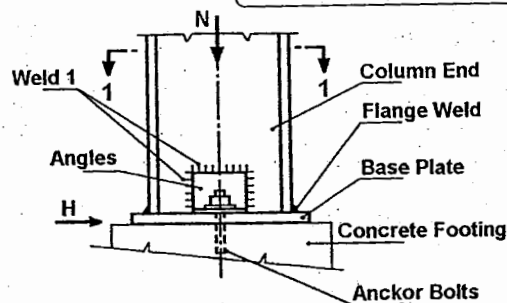
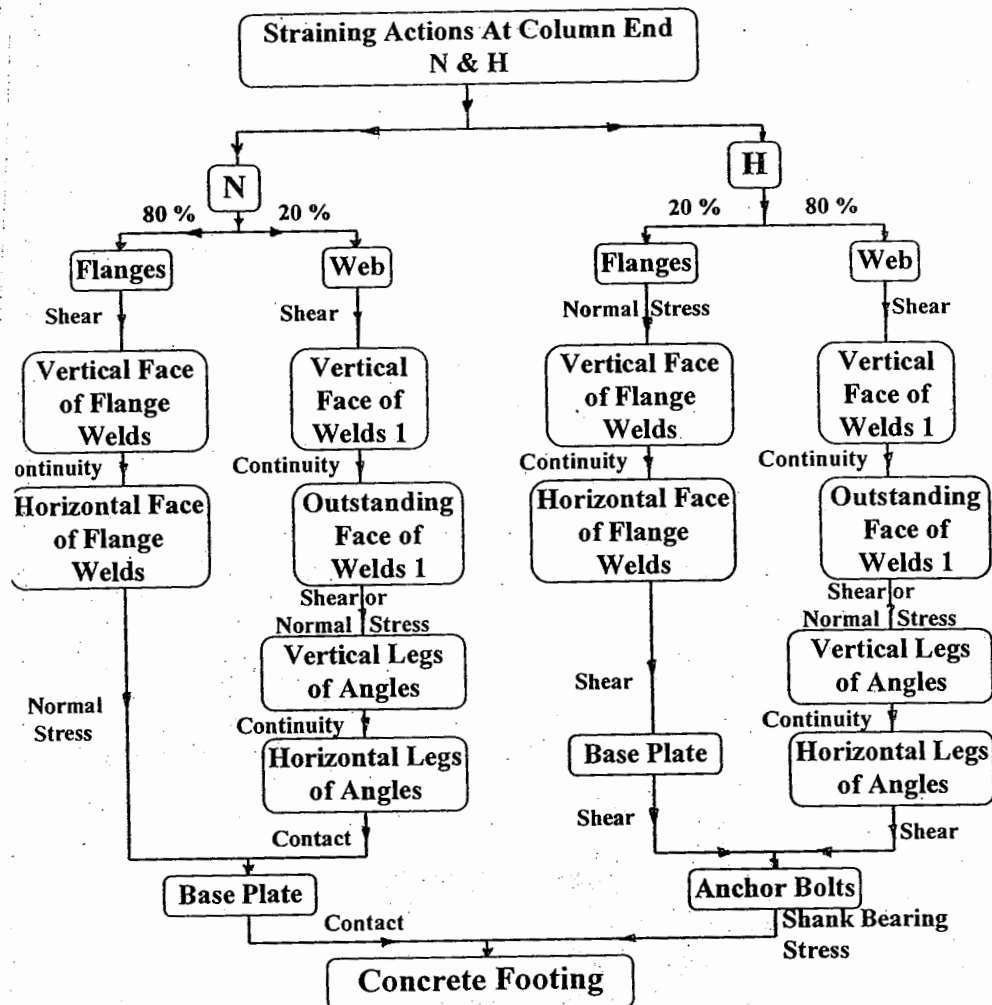


Fig. 9.10

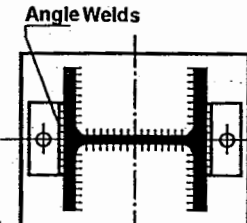
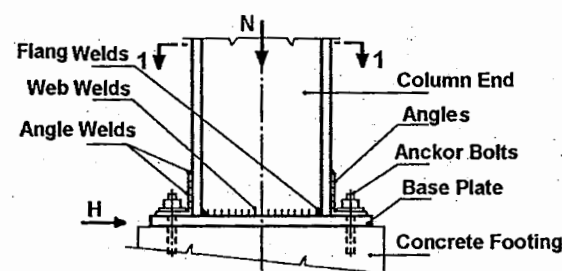
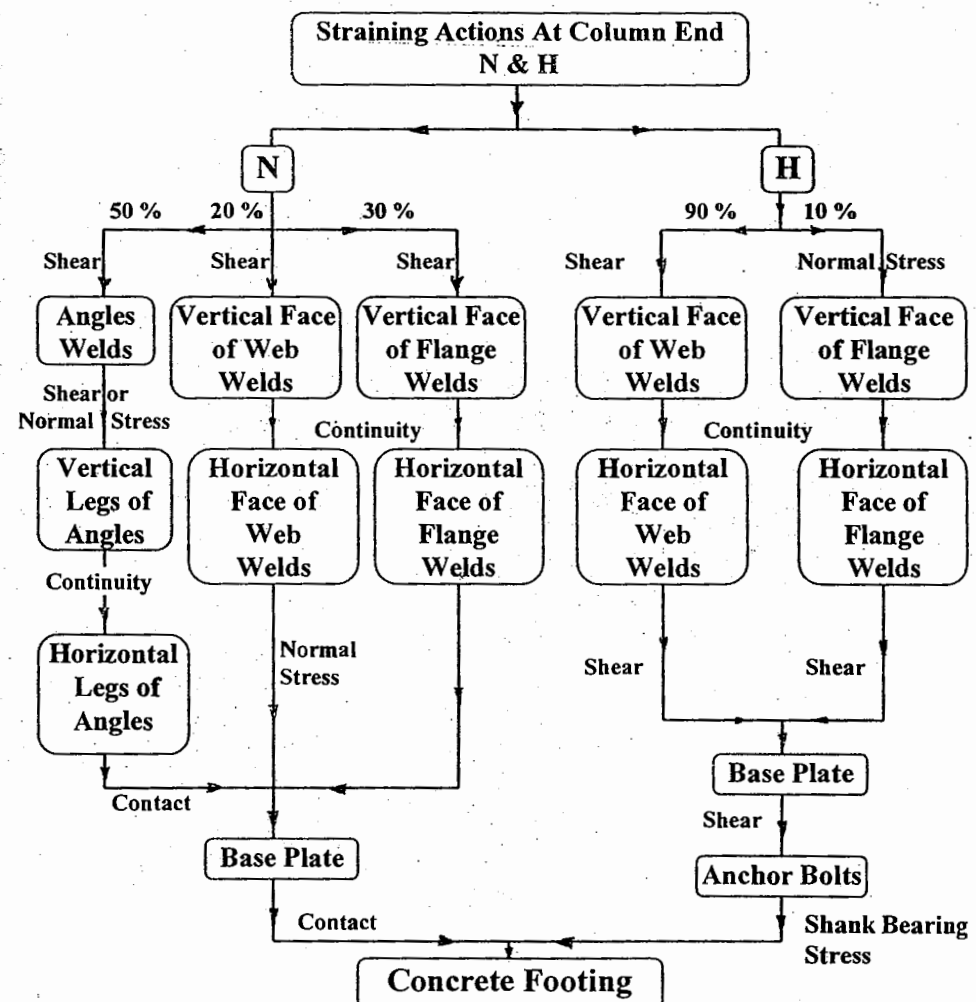


Fig. 9.11

The stress distribution on the reinforced concrete footing, in the vertical stiffeners, in the steel base plate as well as the percentage of share of the applied forces regarding each steel component were deduced.

Figs. 9.9, 9.10 and 9.11 show the transmission of forces via the various steel components as well as the percentage of share of these forces computed from the finite element analysis.

For more detailed analyses of the conventional hinged base connections, the reader may refer to Examples 9.1, 9.2 and 9.3.

9.3.2. Gusseted-Hinged Base Connection:

9.3.2.1. Configuration:

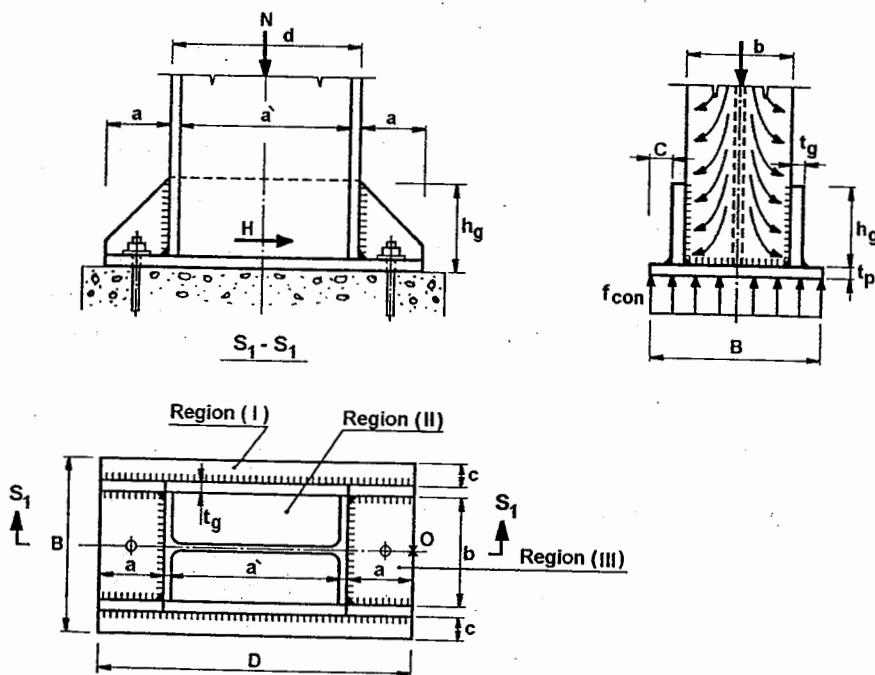


Fig. 9.12 Gusseted Base Connection (Hinged Type).

Fig. 9.12 shows a typical gusseted base connection. This type may be regarded as semi-hinged or fixed. The later will be considered in a subsequent sections.

In the present analysis the created moment due to connection rigidity is small, hence the base connection can be considered as a hinged base connection. As shown in Fig. 9.12, the column flanges are welded to the base plate by two horizontal side fillet welds. Two supporting gusset plates are attached to the flanges tips by four vertical lines of weld. The load is transmitted from the gusset plate to the base plate by contact bearing stress if the lower edges of the gussets are milled. Otherwise, horizontal lines of fillet welds are to be provided.

9.3.2.2. Load Transmission Scheme:

Fig. 9.13, illustrates the flow chart of the axial load transmission from the beam column to the concrete foundation.

9.3.2.3. Analysis and Dimensioning:

(a) Base Plate:

The determination of the required dimensions of the base plate will follow the same procedure previously mentioned in the analysis of the conventional type. The breadth B and the length D of the base plate projected area can be calculated using Equation 9.1.

Regarding the calculation of the plate thickness t_p , the problem is somewhat difficult compared to the conventional type. As shown in Fig. 9.13, the plate may be divided into three regions having each different boundary conditions.

Table 9.1. Coefficients α_1 , α_2 and α_3 for Analysis of Bending of Rectangular Plates Supported on Four and Three Sides

Plates Supported on Four Sides			Plates Supported on Three Sides	
Ratio of Sides (a' / b')	Coefficient		Ratio of Sides (a / b)	Coefficient α_3
	α_1	α_2		
1.0	0.048	0.048	0.5	0.060
1.1	0.055	0.049	0.6	0.074
1.2	0.063	0.050	0.7	0.088
1.3	0.069	0.050	0.8	0.097
1.4	0.075	0.050	0.9	0.107
1.5	0.081	0.050	1.0	0.122
1.6	0.086	0.049	1.2	0.120
1.7	0.091	0.048	1.4	0.126
1.8	0.094	0.048	2.0	0.132
1.9	0.098	0.047	Over 2.0	0.133
2.0	0.100	0.046		
Over 2.0	0.125	0.037		

$$M_a = \alpha_1 f_{con} b'^2$$

$$M_b = \alpha_2 f_{con} a'^2$$

$$M_3 = \alpha_3 f_{con} b^2$$

If (a/b) < 0.5

$$M_3 = 0.5 f_{con} a^2$$

b = length of free edge.

Where M_a and M_b are the induced moments per unit length about the directions a' and b' respectively.

a' = long side of plate.

b' = short side of plate.

α_1 , α_2 = coefficients depending on the plate aspect ratio (a' / b'), and are as given in Table 9.1.

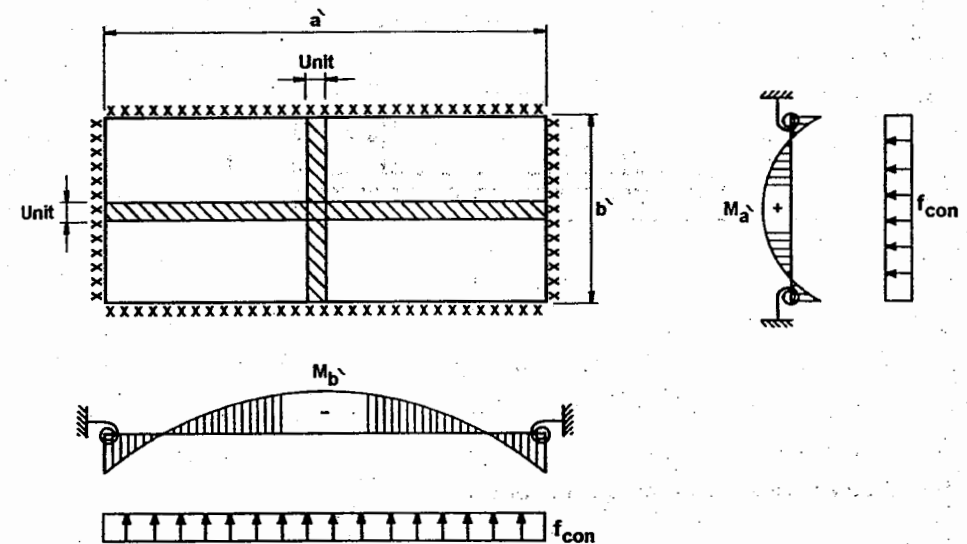


Fig. 9.15 Base Plate Region (2) Supported on Four Sides.

If the ratio (a' / b') exceeds 2, the moment can be determined from the simply supported beam formula ($Wl^2/8$) with $W \equiv f_{con}$ and $l \equiv b'$.

For (a' / b') less than 2, the coefficient α_2 has an average value equal to 0.05. Since the plate edges are elastically

restrained as shown in Fig. 9.15, the moments obtained from Equation 9.8.a ought to be reduced by about 25%. The maximum of M_a and M_b should be taken in the design of the base plate. Hence for region (2), the design moment to be considered in the analysis is:

$$M_2 = \max. (M_a, M_b) \quad 9.8.b$$

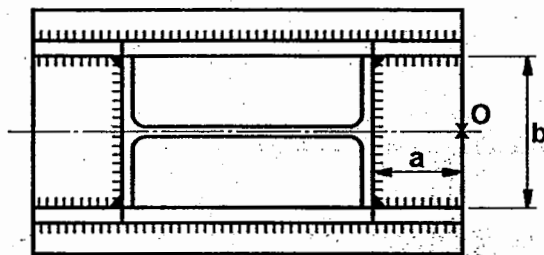


Fig. 9.16

The third region of the base plate is supported on three sides only. The critical point at which the moment should be calculated is the point (o) at the middle of the free edge. The induced moment is as given below:

$$M_3 = \alpha_3 \cdot f_{con} \cdot b^2 \quad 9.9$$

Where α_3 = coefficient given in Table 9.1 depending on the ratio (a/b).

b = length of free edge.

If (a/b) is less than 0.5, this third plate region may be analyzed as a cantilevered plate.

The thickness of the plate is to be determined using the maximum of M_1 , M_2 , and M_3 . Hence:

$$t_p = \sqrt{\frac{6 \cdot M_{\max}}{F_b}} \quad 9.10$$

$$M_{\max} = \text{Max. of } (M_1, M_2, M_3).$$

F_b being the allowable tensile stress of rectangular steel sections and equals $0.72 F_y$.

The optimum design of the base plate may be attained by changing the dimensions a and c in order to obtain equal moments M_1 , M_2 and M_3 .

(b) Gusset Plates:

The significant design parameters of the supported gusset plates are the thickness t_g and the height h_g as shown in Fig. 9.17. The stress analysis of such problem is a complicated process. The three dimensional finite element model previously outlined in Section 9.3.1.3 has proved that the following assumptions are adequate:

- (i) The normal force transmitted by the column's web can be neglected. Fig. 9.17.a shows that the web transmits 20% of the total normal force irrespective of the percentage of contact between stiffeners and base plate.
- (ii) The problem can be considered as a two dimensional plane stress problem if the induced torsional moments are neglected. The exact solution for a typical configuration is given in Fig. 9.17.b.

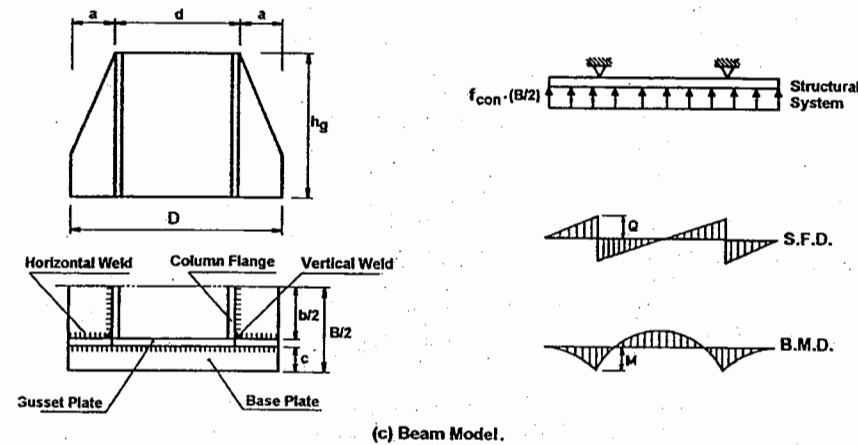
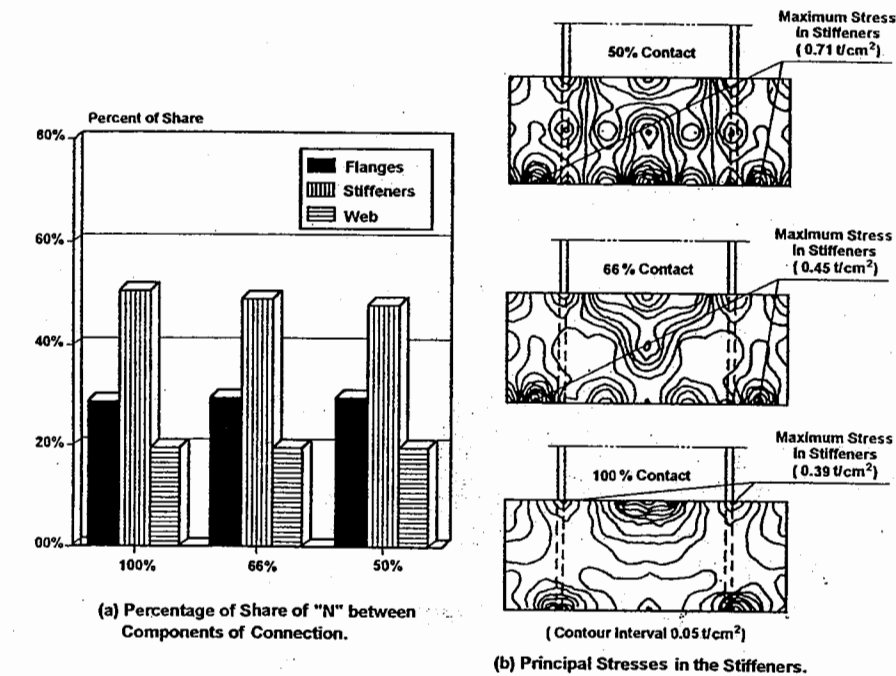


Fig. 9.17 Modeling of the Gusset Plate.

(iii) The gusset plate may be modeled as a beam with two overhangs supported at the locations of the vertical welds attaching the column's flanges and the gusset plate (see Fig. 9.17.c).

The effective upward distributed load for one stiffener is equal to $f_{\text{con}} \times B / 2$ where $f_{\text{con}} = N/B.D$. Considering the critical Section S-S. The shear and moment are as given below:

$$\left. \begin{aligned} Q_{\text{S-S}} &= \frac{N}{2.D} \cdot a \\ M_{\text{S-S}} &= \frac{N.a}{2.D} \cdot \frac{a}{2} = \frac{N.a^2}{4.D} \end{aligned} \right\} 9.11.a.$$

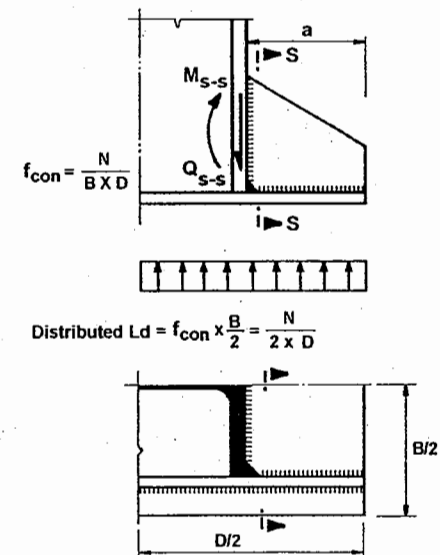


Fig. 9.18

The corresponding maximum shear and bending stresses are:

$$\left. \begin{aligned} q &= \frac{3 Q_{s-s}}{2 t_g h_g} \\ f_b &= \frac{6 M_{s-s}}{t_g h_g^2} \end{aligned} \right\} \quad 9.11.b$$

The values of q and f must not exceed the allowable values material as prescribed in the E.C.P.2001 by Clauses 2.6.5 and 2.6.3 respectively

(C) Anchor Bolts:

The design of the anchor bolts is similar to the conventional hinged base connection.

Refer to Example 9.4.

9.4. Ideal-Hinged Base Connections:

The previous hinged base connections will induce a certain amount of moments due to the rigidity of the steel components. In order to minimize such moments, more real hinged base connections ought to be employed. In the following types maximum possible rotation is provided.

9.4.1. Double-Plate Hinged Base Connection:

Fig. 9.19 shows a typical ideal-hinged connection with top and bottom base plates. The vertical force "N" is transmitted to the top plate via the diagonal stiffeners which are attached to the column web by fillet welds.

A thick plate "G" of cast iron is welded to the top plate and rests on the bottom one. This plate concentrates the distributed vertical load along the vertical axis and hence prevents the creation of moments. Two keeper side plates are welded to the bottom base plate to prevent any possible horizontal displacements.

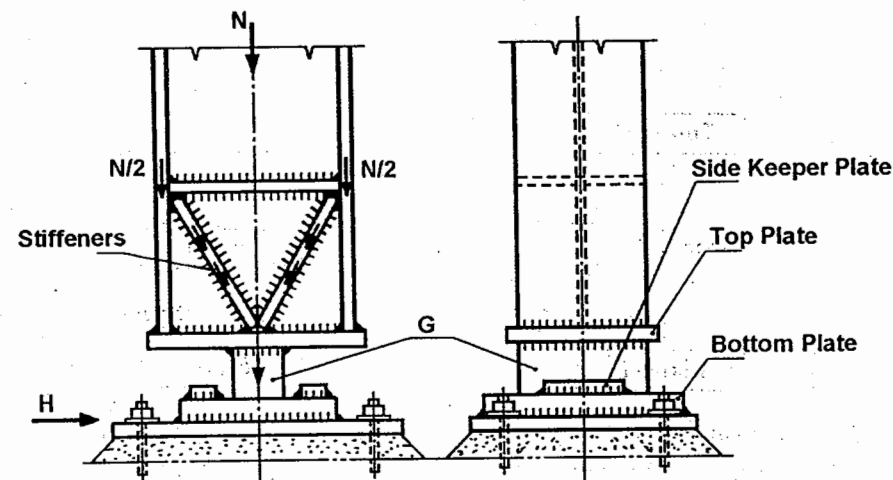


Fig. 9.19 Double-Plate Hinged Base Connection

The design of such base configuration is identical to the previous conventional types. However, stiffeners which are subjected to compression forces must satisfy the safety against normal stresses and local buckling.

Fig. 9.20 depicts the load transmission scheme for this type.

Refer to Example 9.5.

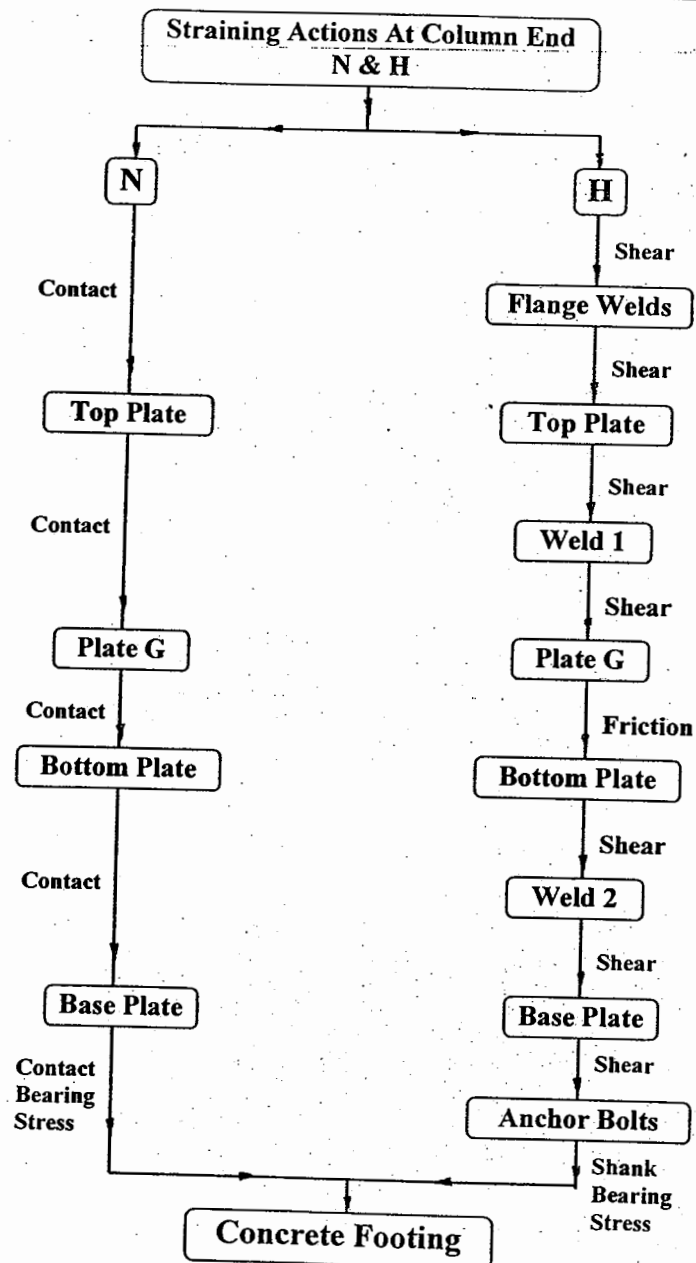


Fig. 9.20

9.4.2. Pin-Hinged Base Connection:

The pin-hinged connection is mainly used for cases where an uplift axial force exists. A typical configuration is illustrated in Fig. 9.21.

The pin is composed of a steel cylindrical body and a head. The cylindrical body is introduced inside the holes of the hatched plates with the minimum possible clearance ($\Delta d < 1 \text{ mm}$). The pin is maintained by a threaded keeper rod. Regarding the design, the pin is subjected to shear and bending stresses which govern the choice of the diameter d . The clear height "h" and the thickness of the plates in contact with the pin are determined so that the induced bearing stresses are in accordance with the allowable values.

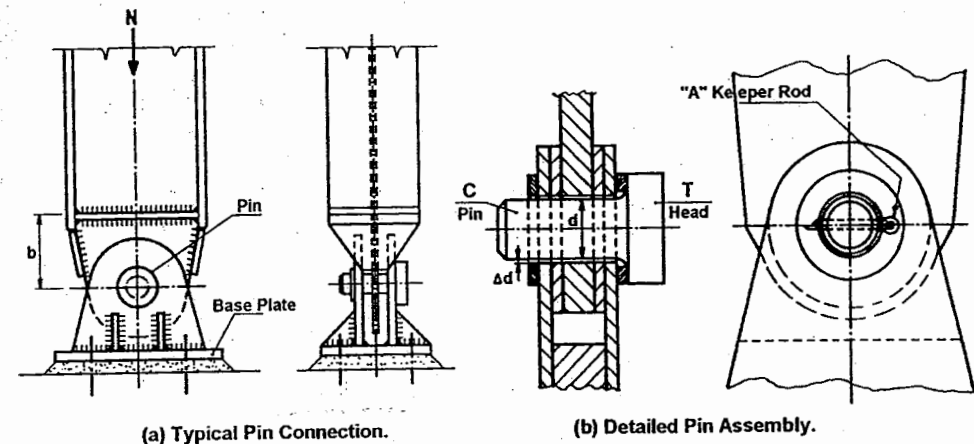


Fig. 9.21 Pin-Hinged Base Connection.

The Chart in Fig. 9.22 shows the load transmission scheme.

Refer to Examples 9.6.a and 9.6.b.

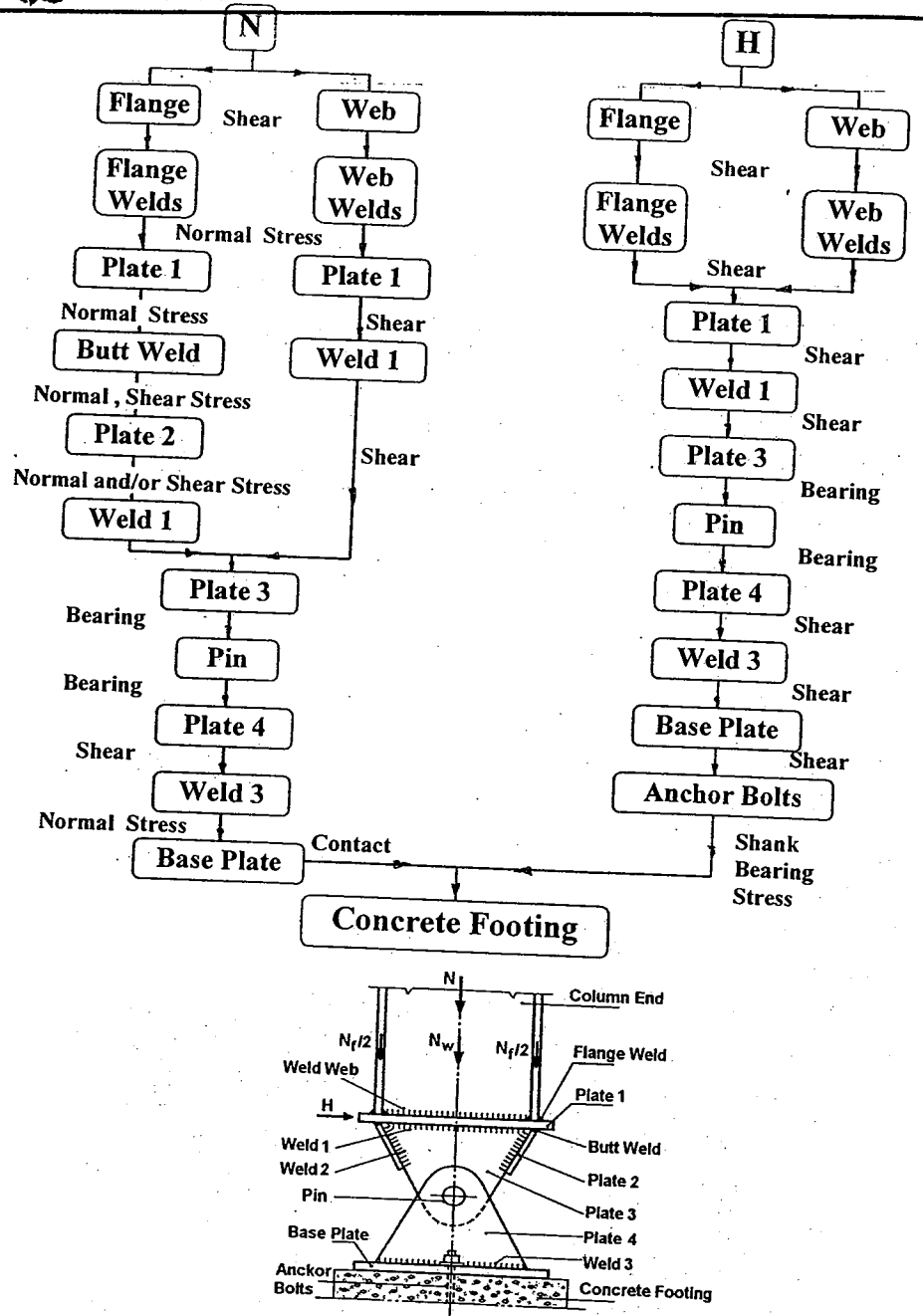


Fig. 9.22

9.4.3. Elastomeric Bearings:

An elastomeric base connection would resemble a sandwich construction. As shown in Fig. 9.23, the top and bottom base plates contain in between neoprene or rubber material which can safely transmit loads inducing stresses up to 100 kg/cm^2 .

Bearings are either plain or laminated as shown in Fig. 9.23. The plain type consists of single elastomer layer while the laminated type consists of elastomer layers restrained at their interfaces by bonded laminates such as steel. The distortion scheme of neoprene plates is shown in Fig. 9.23.c. This type allow easier thermal deformation and easier rotation of the lower end of the columns.

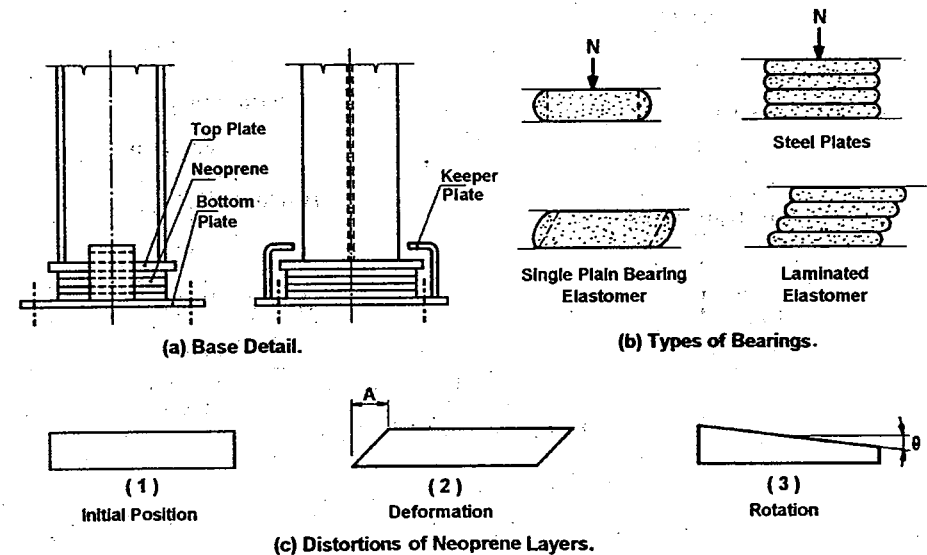


Fig. 9.23 Elastomeric Hinged Base Connection.

The Chart in Fig. 9.24 shows the load transmission scheme.

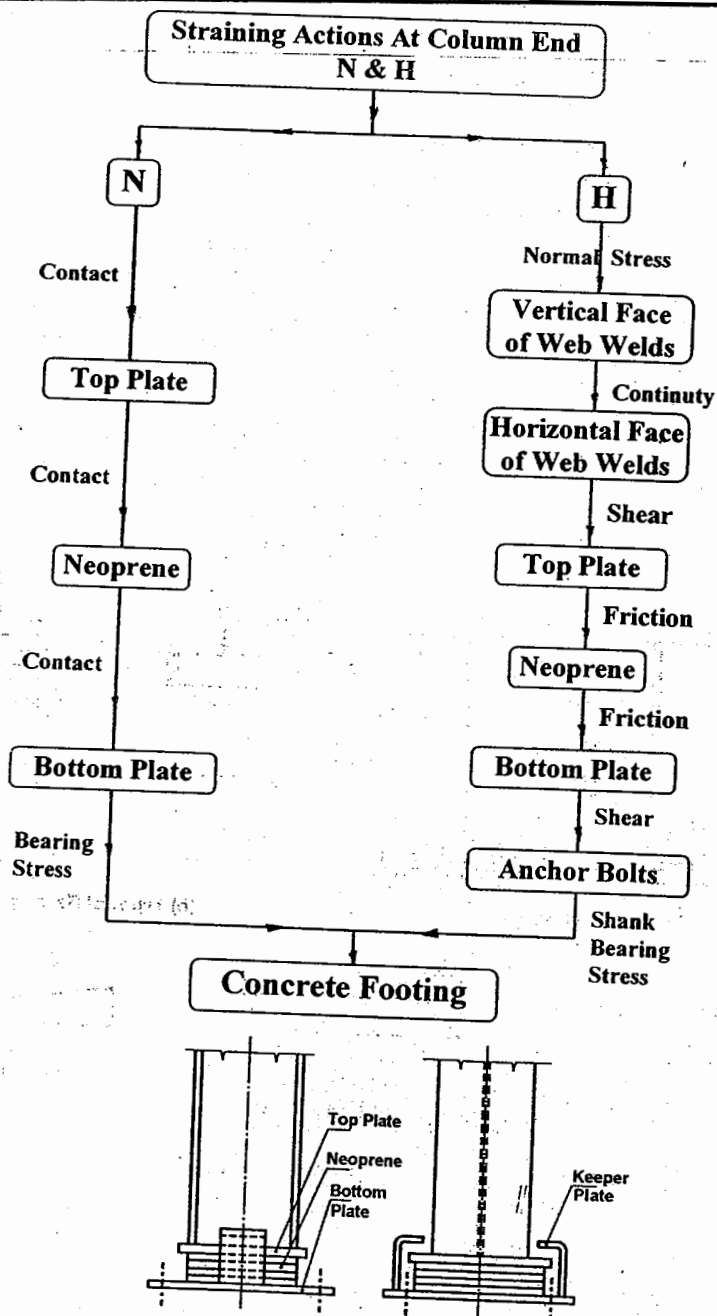


Fig. 9.24



9.5. Hinged-Base Connection of Columns with Hollow Circular Sections:

Fig. 9.25 shows a hinged base connection of a circular hollow column section. The thickness of the base plate is determined in accordance to Equation 9.3, while the diameter and length of anchor bolts are governed by the uplift and the horizontal reactions. If the end of the column is milled, the force "N" will be transmitted by contact bearing to the base plate. otherwise the circular outside fillet welds will transmit the vertical and horizontal reactions to the base plate

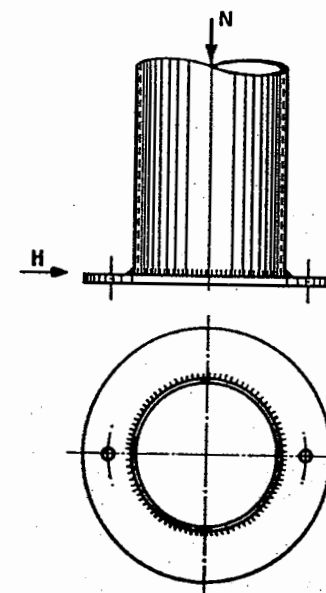


Fig. 9.25

9.6. Fixed Base Connections:

Beam columns are frequently required to carry end moments "M" induced either by eccentricity of the axial load "N" or by base fixity. Fig. 9.26, illustrates a typical fixed base

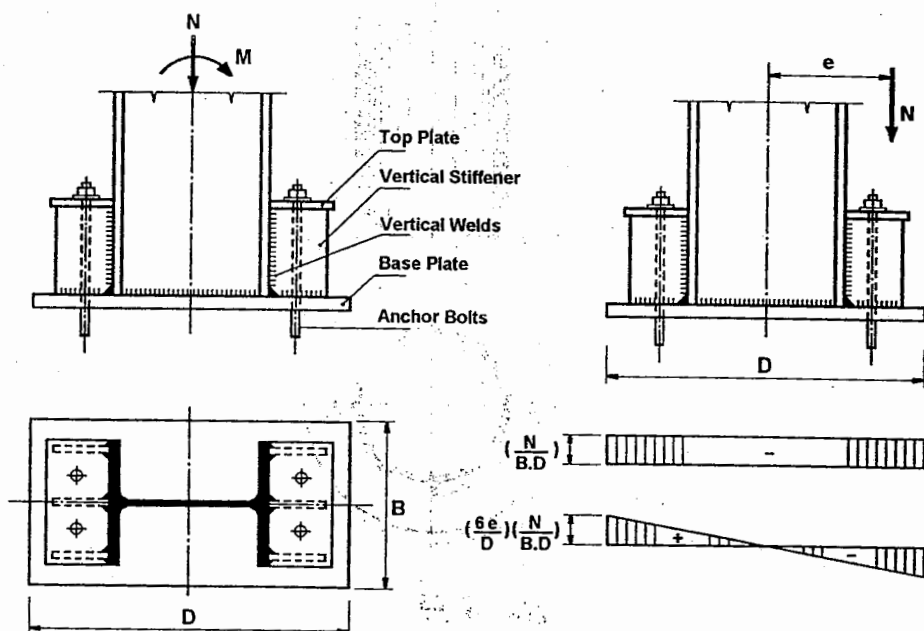
connection subjected to an axial compressive force "N" and an end moment "M = N.e", where the eccentricity (e) is measured from the column neutral axis.

The maximum normal stress (f_{con}) on the concrete footing is given by:

$$f_{con} = -\frac{N}{BD} \pm \frac{(N.e)(D/2)}{(BD^3/12)} = -\frac{N}{BD} \pm \frac{6.(N.e)}{BD^2}$$

$$f_{con} = -\frac{6N}{BD^2} \left(\frac{D}{6} \pm e \right) \quad 9.12$$

Where: $e = M / N$



(a) Constructional Details of base Connection (b) Stress Distribution "f" due to "N" and "M" respectively

Fig. 9.26. A Typical Fixed Base Connection Subjected to "N" and "M"

Table 9.2 gives three possible cases for the stress distribution. In the first two cases where $e \leq D/6$, the base plate is entirely subjected to compression stress f_{con} as calculated from Equation 9.12. In the third case where $e > D/6$ Equation 9.12 is no longer valid since the tension side of the concrete footing becomes inactive.

In the following sections, each type will be considered separately and approximate methods of analysis will be developed for some conventional configurations.

Table 9.2. Stress Distribution Patterns Over Base Plate

Configuration	Eccentricity "e"	Stress Distribution "f _{con} "
Base Entirely Subjected to Compression	$e < D/6$	
	$e = D/6$	
Base Partially Subjected to Compression	$e > D/6$	

T = resultant tensile force.
C = resultant compression force.

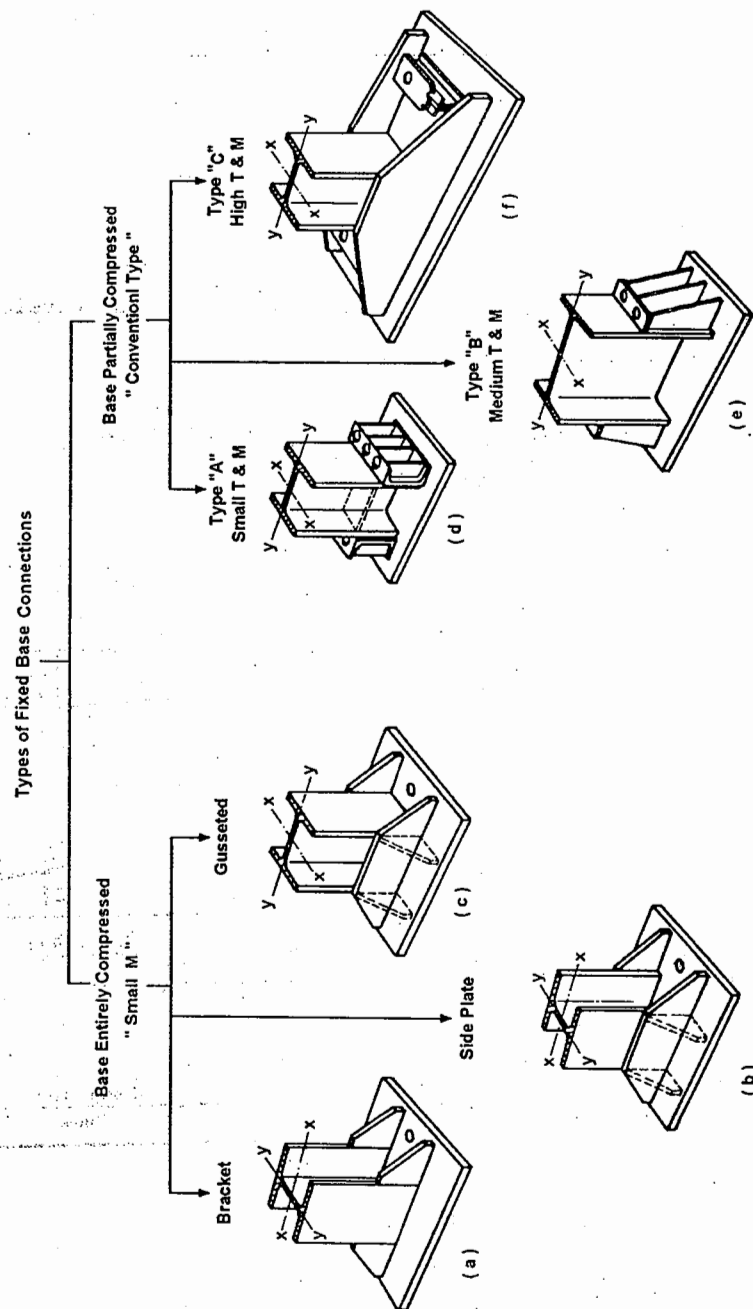


Fig. 9.27 Different Types of Fixed Base Connections .



Fig. 9.27, describes the most commonly used types of fixed base connections. They are classified according to the stress distribution pattern over the base plate.

9.6.1. Base is Entirely Subjected to Compression " $e = (M/N) \leq (D/6)$ ":

9.6.1.1. Different Configurations:

From the previous discussion, it can be concluded that as long as the end moment M is small, it would be expected that compressive stresses are always present between the column and the concrete foundation. As given in Fig. 9.27, the main types of fixed base connections that are entirely subjected to compressive stresses are classified as follows:

(a) **The Bracket Type**: In this type as shown in Fig. 9.27.a brackets are attached to the outer edges of the column flanges by small braces of butt welds. This can be obtained by just beveling the edges of the brackets. No beveling is required for column flanges. Fillet welds joining the column and the brackets to the base plate can transmit the induced moment.

(b) **Side-Plate Type**: This type is shown in Fig. 9.27.b and is utilized for columns of nominal flange thickness. The moment " M " and the axial force " N " are transmitted from the flanges to the side plates via vertical lines of fillet weld. These straining actions are then transmitted to the base plate via the lower horizontal lines of fillet weld.

(c) **Gusseted Type**: This type is illustrated in Fig. 9.27. and is also used for nominal flange thicknesses. Gussets are attached to the outer edges of the flanges which stiffen the overhanged portions of the base plate.

9.6.1.2. Method of Analysis:

Connection Type for which the base plate is entirely subjected to compression can be analyzed by the same previously developed methods of the hinged base connections. The dimensions B and D of the base plate are determined such that the maximum applied stress f_{con} calculated by Equation 9.12 does not exceed the allowable prescribed values. Regarding the calculation of the minimum required thickness of the base plate, the problem is somewhat difficult than that of the hinged types. The distribution of the stress f_{con} over a unit strip of the plate is no longer uniform so some averaging values may be assumed to compute the plate thickness.

For more detailed analysis, the reader may refer to the following examples:

Example 9.6 : Fixed-base connection of the bracket type.

Example 9.7 : Fixed - base connection of the side plate type.

Examples 9.8 and 9.9 : Fixed-base connection of the gusseted type.

9.6.2. Base is Partially Subjected to Compression "e = (M/N ≥ (D/6))":

This configuration is termed as the conventional fixed base connection. Tension will be induced over a part of the base and then transmitted to the footing via anchor bolts by bond stress. As has been previously outlined in Figs. 9.27. (d), (e) and (f), the main types may be classified as type "A", type "B", and type "C".

9.6.2.1. Different Configurations:

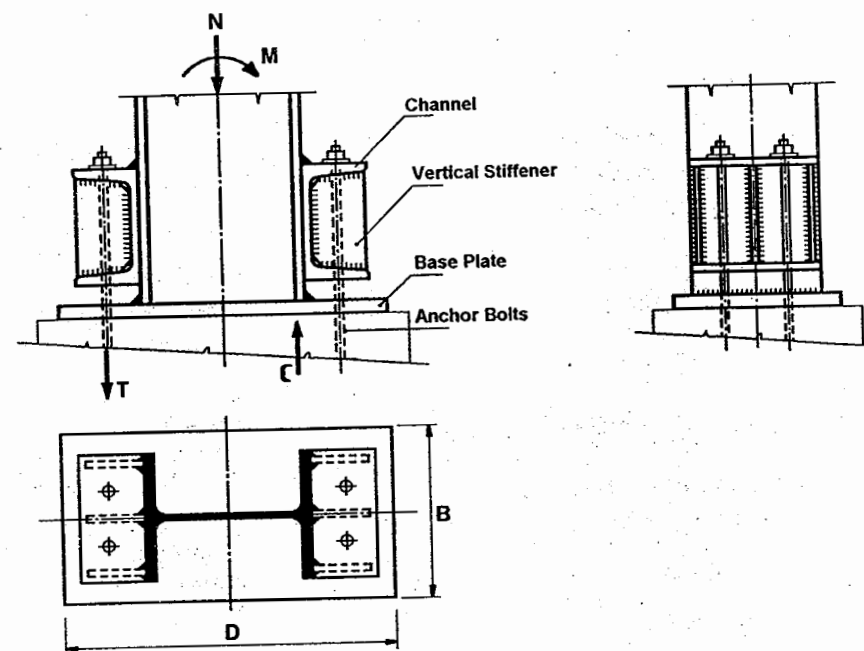


Fig. 9.28. A Detailed Construction of A Typical Fixed Base Connection Type "A".

Type "A": This type is mainly used for small values of moment M and tensile force T. Fig. 9.28., illustrates a typical connection. As shown, two channels are attached to the column flanges by horizontal lines of fillet weld. The channels at both sides are not in contact with the base plate. Thus, the resultant compressive force "C" acts on the center line of the beam-column compression flange. Vertical stiffeners are held in the interior of the channels by vertical and horizontal fillet welds. They transmit the tensile force "T" to the anchor bolts via the top flange of the channel. Fig. 9.29. shows the load transmission scheme.



Left Side

Right Side

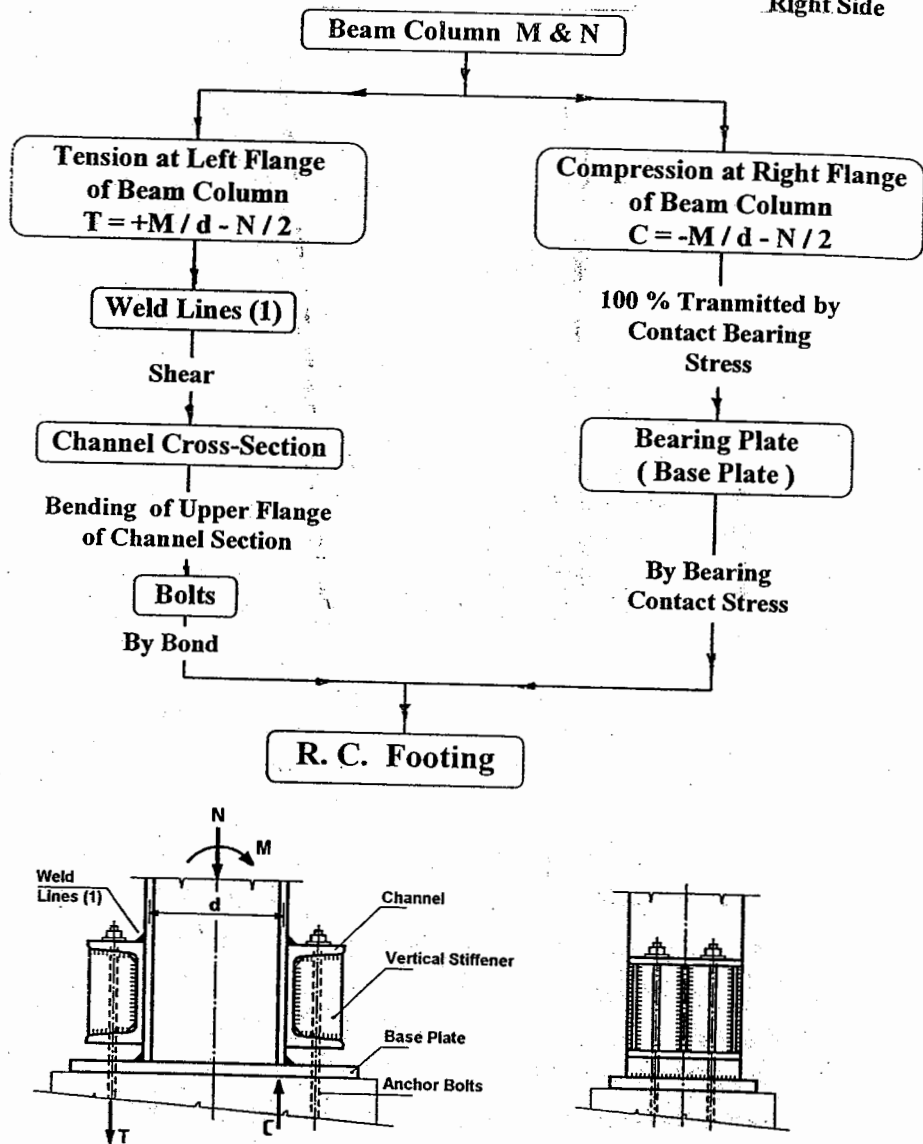


Fig. 9.29. Fixed-Base Connection Type "A"
Load Transmission Scheme



Type "B": This type is used for medium values of MandT. The various steel components of the connection are shown in Fig. 9.30. Vertical stiffeners are attached to the column flanges by vertical lines of fillet weld. The compression force is transmitted from the flanges to the stiffeners via these vertical welds and then to the base plate by direct contact. Tension is transmitted to the top plate via the stiffeners through the vertical weld lines and then to the bolts by tension and then to the footing by bond.

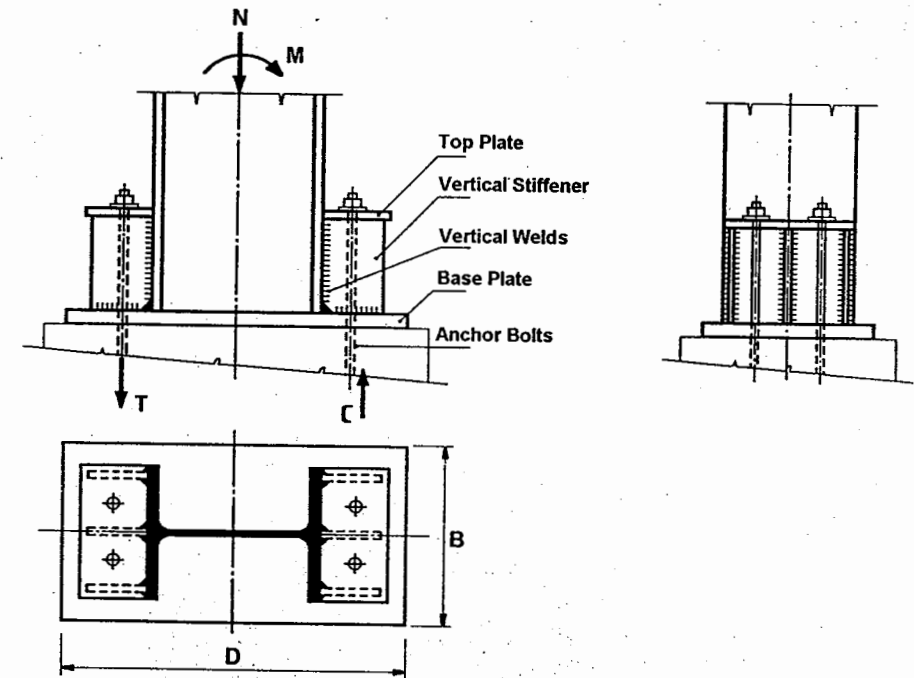


Fig. 9.30. Typical Detailed Construction of
Type "B" Fixed Base Connection

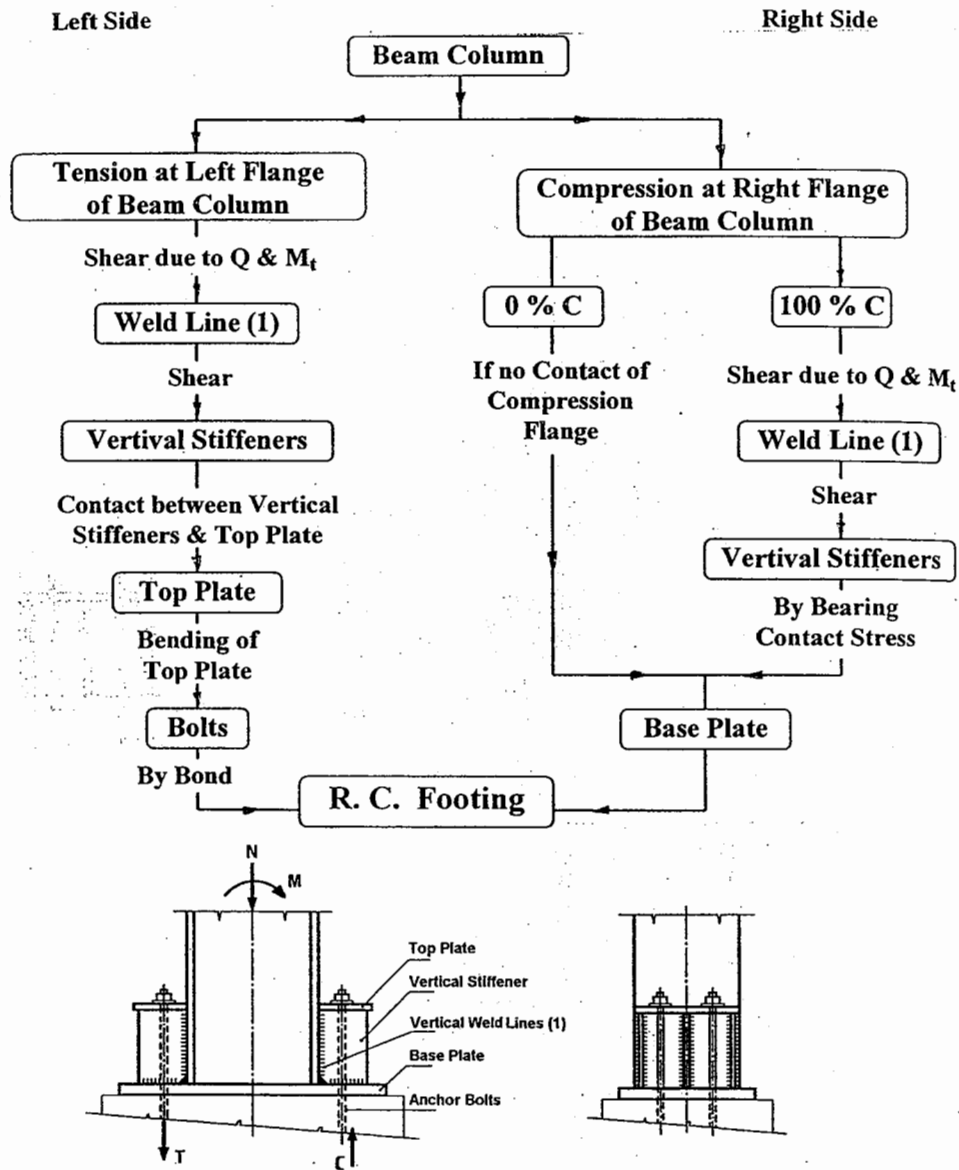


Fig. 9.31. Fixed-Base Connection Type "B"
Load Transmission Scheme



The transmission of forces from the beam-column to the foundation is shown in Fig. 9.31.

Type "C": This type is commonly used for high values of " M " and " T ". Referring to Fig. 9.32., it can be observed that the gusset plates have been extended outer far in order to induce the greatest moment of resistance.

This particular detail uses a pair of channels with a top plate for the hold-down bolts to transfer the tensile force back, and, in turn, back to the beam column.

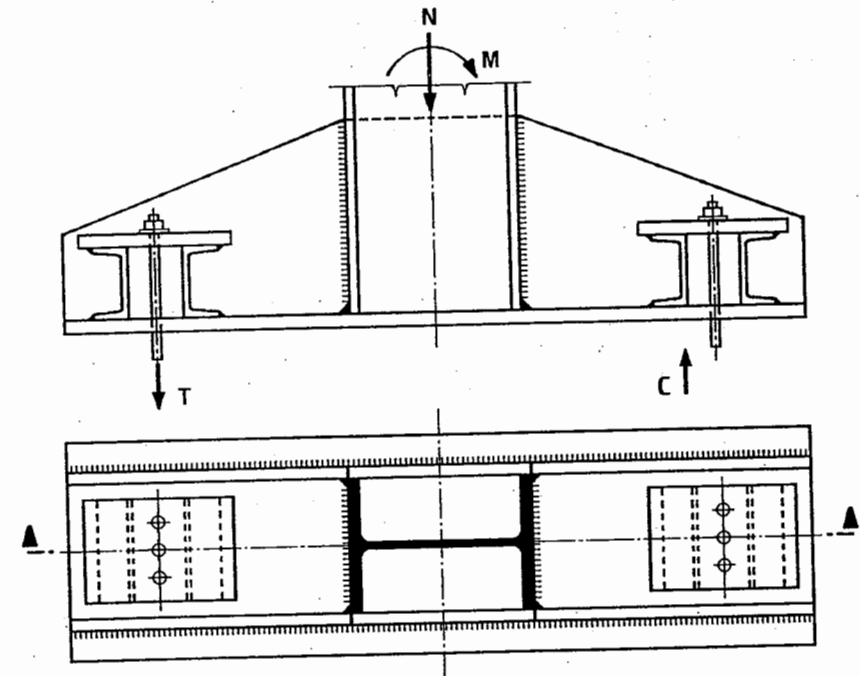


Fig. 9.32. A Detailed Construction of the Conventional
Type "C" Fixed-Base Connection



9.6.2.2. Method of Analysis:

A Working Load Approach:

In the following an analytical method for analyzing a typical partially compressed fixed base connection is presented. The eccentricity (e) is great enough to cause tension on one side of the base; $e > D / 6$.

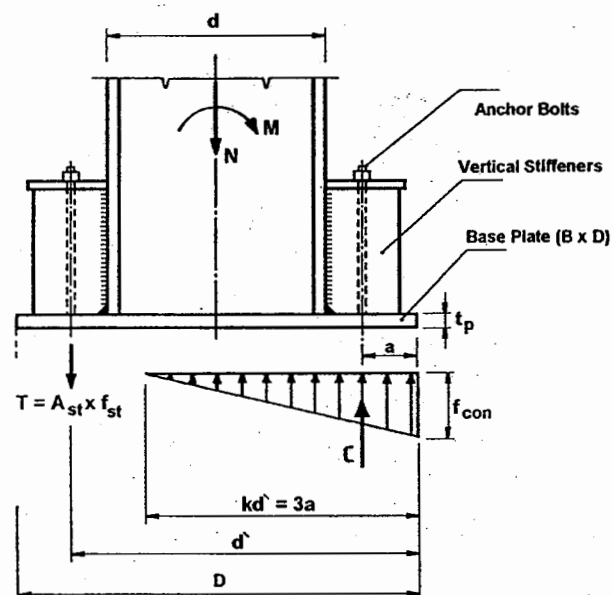


Fig. 9.33. Free Body Diagram of Resultant Loads Applied to A Fixed Base Connection

Referring to Fig. 9.33, the following design parameters are defined:

d' : Location of the resultant tensile force measured from the right edge of the base plate



K_d : Effective length of contact bearing between base plate and concrete foundation; k is a multiplier factor.

A_{st} : Effective steel area of bolts subjected to tension.

T & C : Resultant tensile and compressive forces respectively.

f_{st} : Induced tensile stress in bolts.

f_{con} : Maximum induced compressive stress on concrete footing.

Basic Assumptions:

- 1- Both steel and concrete behaves as a linear elastic material in which Hook's law applies (i.e. stress = young's modulus X strain).
- 2- The theory of bending is applicable, where plane sections remain plane after deformation.

Equilibrium Equations of Normal Force and Moment:

- Equilibrium of normal forces gives:

$$N = C - T \quad \dots \dots \dots 9.13.a.$$

Where $C = \frac{1}{2} f_{con} \cdot K_d B \quad \dots \dots \dots 9.13.b.$

and $T = A_{st} \cdot f_{st} \quad \dots \dots \dots 9.13.c.$



- Moment balance about the right edge of base plate gives:

$$M = N \cdot \frac{D}{2} + (T - \frac{K}{3} C) \cdot d' \quad \dots 9.14.$$

Stress-Strain Relations:

From the assumption of linear strain distribution, the following relations are obtained:

Strain Ratio:

$$\frac{\epsilon_{st}}{\epsilon_{con}} = \frac{(1-k)}{k}$$

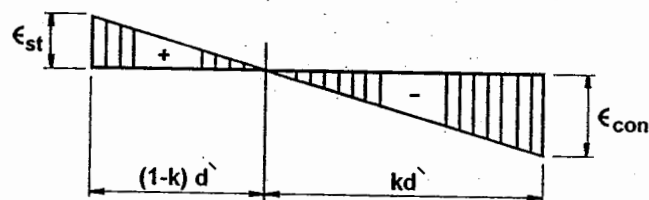


Fig. 9.34. Strain Distribution.

Stress Ratio:

$$\frac{f_{st}}{f_{con}} = \frac{E_{st} \cdot \epsilon_{st}}{E_{con} \cdot \epsilon_{con}}$$

Therefore: $f_{st} = f_{con} \cdot \frac{(1-k)}{k} \cdot n \quad \dots 9.15.$

Where: ϵ_{st} & ϵ_{con} : Maximum strains produced in steel and concrete respectively.

E_{st} & E_{con} : Module of elasticity of steel and concrete respectively.

n : Modular ratio ($= E_{st} / E_{con}$).



By assuming trial reasonable values for the parameters B , D , d' and A_{st} , Equations 9.13, 9.14, and 9.15, can be solved for the unknowns k , f_{con} , and f_{st} . The tensile force acting on the bolts can then be determined and the required fillet weld lines of stiffeners can be proportioned. The thickness of the base plate is calculated from the resulting stress distribution f_{con} .

Refer to Example 9.10.

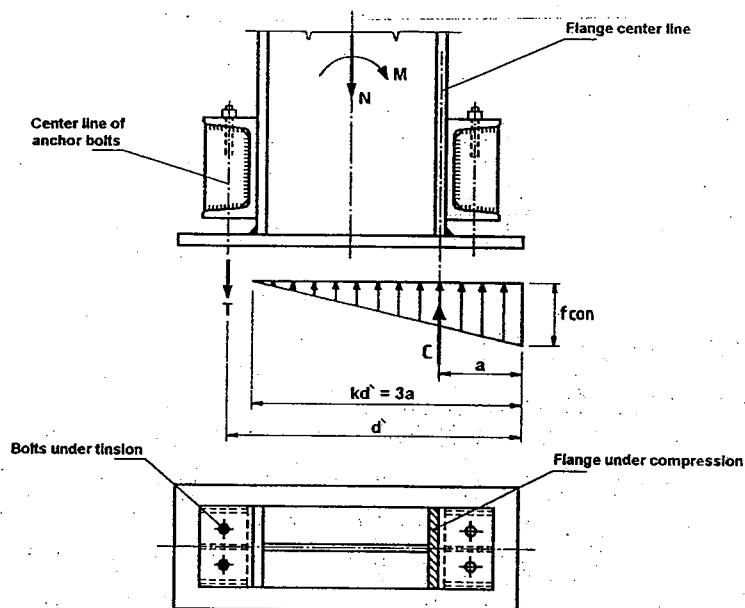
9.6.2.3. Simple Methods of Analysis:

(a) Type "A": In this type, the supporting channels are not in contact with the base plate as illustrated in Fig. 9.35.a. The anchor bolts will transfer the tensile forces by bond to the foundation, while the compressive forces will be transmitted to the base plate mainly through the column flange as indicated in Fig. 9.35.a. The end of the beam column is assumed to be milled. The effective bearing length " Kd " can be determined by considering that the centroid of the compressive stress triangle is coinciding with the column flange center line.

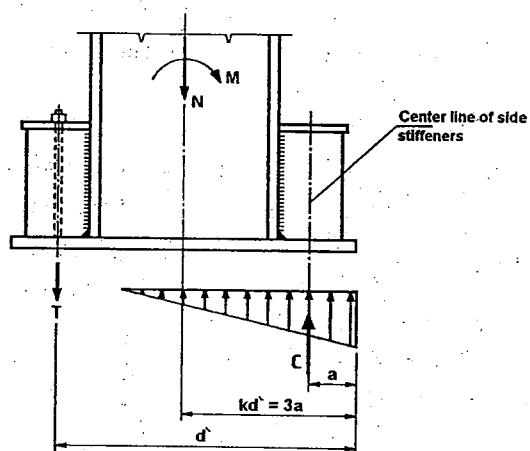
Refer to Example 9.11.

(b) Type "B": Here the bearing length is determined assuming that the centroid of the stress triangle coincides with the center line of the side stiffeners as shown in Fig. 35.b. This assumption is based on the fact that the end of the beam column is not milled. Thus the compressive force of the flange is transmitted to the stiffeners via the weld lines and then to the base plate by contact. In reality this assumption is somewhat imaginary. It is preferable to consider more realistic assumptions as given in the following sections.

Refer to Example 9.12.



(a) Conventional Type "A".



(b) Conventional Type "B".

Fig. 9.35. Induced Forces

9.6.2.4. Finite Element Analysis:

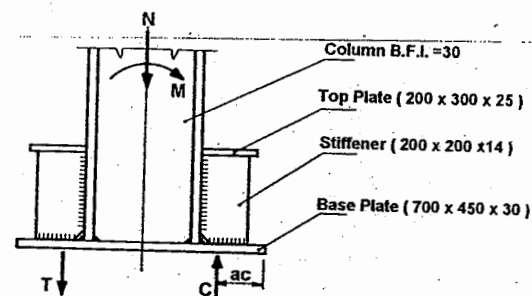
A research work (25) has been conducted in the Structural Department of Cairo University where the hinged and fixed base connections have been analyzed. The technique of Finite Elements has been utilized using a quadrilateral element with six degrees of freedom. The obtained results have shown that the compressive stress distribution on concrete footing is parabolic for a base plate thickness of common practice.

As a case of study, Fig. 9.36.a. shows a typical configuration of a fixed base connection. The shapes of the stress distribution on the concrete footing for different thicknesses of base plate are shown in Fig. 9.36.b. As the thickness increases the plate becomes more rigid and the stress distribution over the concrete becomes uniform.

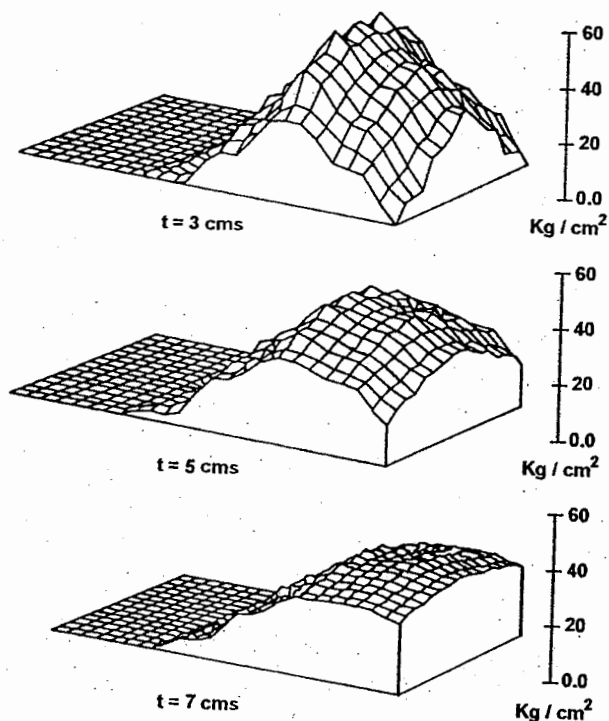
The normal base plate thickness used in the practical design ranges from 2.0 to 4.0 cm. For this range the base plate may not be assumed as rigid, and it should be expected that the actual stress on the concrete base is somewhat greater than the calculated one from the conventional method. Base plates with thicknesses of around 1.0 cm are flexible while base plates with thicknesses around 6.0 cm may be assumed as rigid.

Regarding the position of the compression force, the conventional method assumes it to be either under the column compression flange (case of flange column which is only in contact) or at the middle of the stiffener width (case of partial contact). Table 9.3 gives the exact position as obtained from the finite element analysis.

The distance (a) is measured from the right edge of the base plate as shown in Fig. 9.36.a. It is to be noted that this distance (a) equals 10 cms as computed by the conventional analysis.



(a) Typical Configuration.
(N = 40 tons, e = 25cms)



(b) Stress Distribution on Concrete for
Different Base Plate Thickness

Fig. 9.36. Finite Element Analysis of
Fixed Base Connection.

Table 9.3

% of Contact		a_c (cm)
Flange	100%	14.56
Stiffeners	100%	
Flange	0%	14.2
Stiffeners	100%	
Flange	66%	14.6
Stiffeners	66%	
Flange	50%	14.74
Stiffeners	50%	

Referring to Table 9.3, it can be concluded that the percentage of contact does not affect the position of the compression force. This may be explained by the fact that the distances between the nodes of the elements are relatively small. Thus the base plate redistributes the acting loads over the same area almost similarly.

Considering the finite elements method as an accurate analysis, it can also be concluded that the dimensioning of the fixed base connection using the conventional method (Type B) is sometimes unsafe to withstand the induced stresses. Hence it is recommended to proceed the dimensioning using the refined approximate analysis of Section 9.6.2.4.1.

9.6.2.4.1. Refined Approximate Analysis:

As a more refined approximation, the compression force should be assumed at the centroid of an area comprising the column compression flange and the stiffeners in the compression side. This assumption gives a value of 14.16 cm for the position of the compression force, which is very close to values obtained from the finite elements analysis.

It should be noted that for the case where the compression flange is in contact only, while the stiffeners are separated from the base plate, the compression resultant lies under the compression flange for both the conventional solution and the finite element analysis.

Refer to Examples 9.13, 9.14, and 9.15.

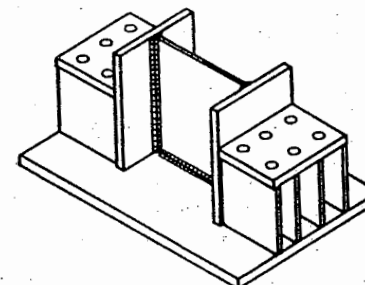
9.6.2.4.2 Effect of Bolts Position:

Case for Which the Number of Bolts Exceeds the Breadth Column Flange:

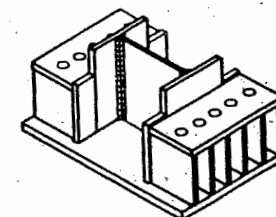
Three details were chosen to study the effect of bolts position on the analysis of the fixed base connections. These details are shown in Fig. 9.37. The dimensions of each of the three details were computed according to the conventional solution. The main characteristics of the three details are as follows:

- 1- In detail 1, six bolts are used on two rows outside the column flanges.
- 2- In detail 2, five bolts are used on one row outside the column flanges. The column flanges were extended by using edge butt welded bracket plates, in order to allow a convenient breadth to add two additional bolts.

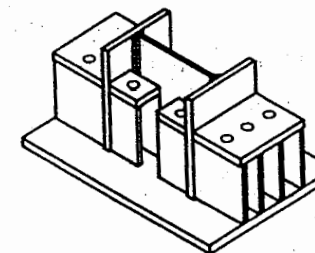
- 3- In Detail 3, five bolts are used on two rows. Three bolts on a row outside the column flanges and two bolts on a row inside the column flanges.



Detail (1)



Detail (2)



Detail (3)

Fig. 9.37 .. Different Bolt Positions of a Typical Fixed Base Connection

A finite elements analysis has been proceeded regarding the three details. The following results have been obtained.

Detail (1):

The first row at the neighborhood of the column flange was carrying 75% of the total tensile force while the second one was carrying 25%. Hence due to this unequal distribution of forces, the bolts of the first row will be overstressed when using the conventional method of analysis.

Detail (2):

The tension force was mainly carried by the three central bolts. The central bolt, the two intermediate, and the two exterior ones were carrying 28%, 44%, and 28% of the tensile force respectively. Also due to this inequality the central bolt becomes unsafe if designed according to the conventional method.

Detail (3):

The tension force was equally distributed on the five bolts. Each bolt carries about 20% of the tensile force.

From the above discussion, it can be concluded that detail (3) is the best detail, the stress distribution on bolts being uniform. On the other hand, one can also use detail (2) where the error can be admitted. Detail (1) is forbidden since the dimensions of bolts are unsafe.

Refer to Examples 9.16 and 9.17.

9.6.3. Fixed Base Connections with Hollow Columns:

9.6.3.1. Columns with Circular Hollow Sections:

Fig. 9.38 illustrates three configurations of base connections for hollow circular columns. Type (A) shown in Fig. 9.38.a is utilized for small fixed base moments. These moments are transmitted to the corresponding bolts and to the base plate via circular lines of fillet welds. The computation of the induced compressive stresses and the base plate thickness will follow exactly the procedure previously outlined in Section 9.6.1. For moderate moments, Type (B) illustrated in Fig. 9.38.b is recommended, where the applied moments are transmitted to the base plate via vertical stiffeners directly fillet welded to the columns. However for relatively high moments, Type (c)



shown in Fig. 9.38.c is to be utilized where a circular top plate, vertical stiffeners and tension bolts are transmitting the applied straining actions to the base. The design of the different steel elements will follow the conventional fixed base connection previously outlined in Section 9.6.2.2.

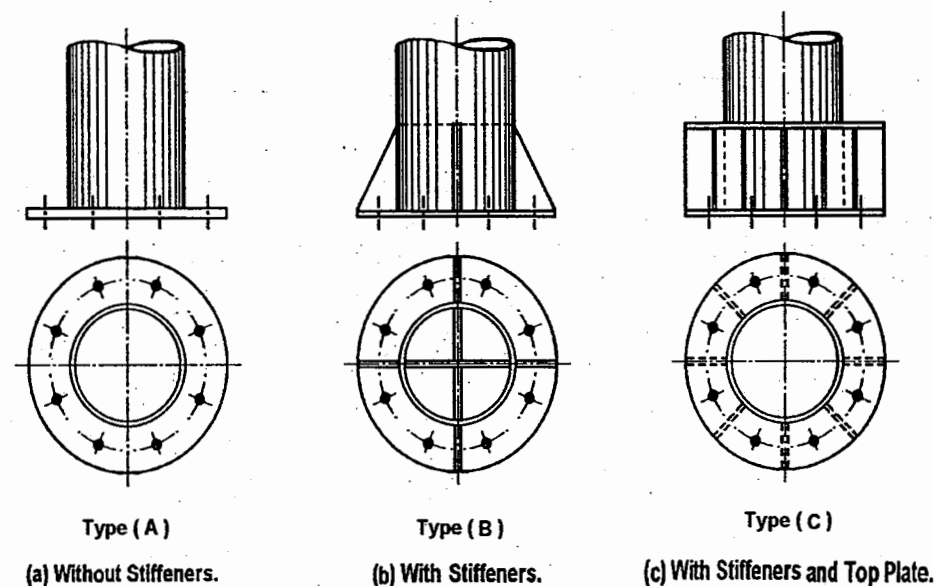
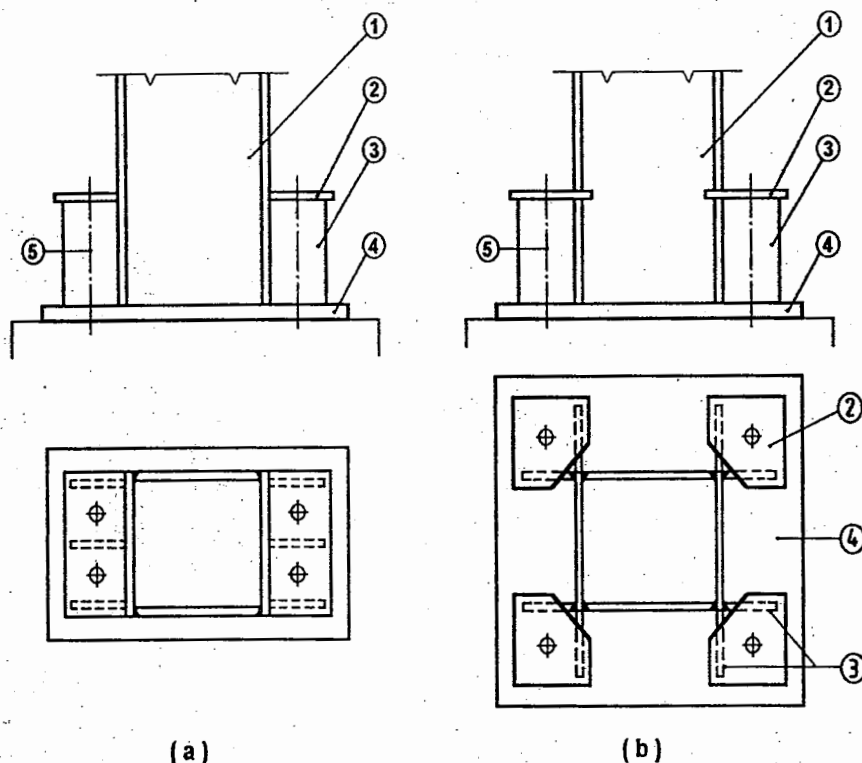


Fig. 9.38. Fixed Base Connection with Hollow Circular Columns

9.6.3.2. Columns with Square Hollow Sections:

For box columns, the traditional solution shown in Fig. 9.39.a use vertical stiffeners, a top plate and bolts. The other alternative is illustrated in Fig. 9.39.b where the stiffeners are extended so that the top plate has symmetric support about the bolt center line.



(1) Box Column ; (2) Top Plate ;
(3) Vertical Stiffeners
(4) Base Plate ; (5) Bolt Centre-Line.

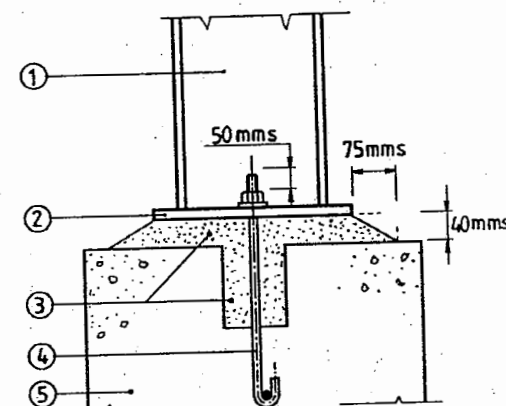
Fig. 9.39. Base Connections of Box Columns

Depending on the magnitude of the applied moment the vertical stiffeners are either welded directly to the base plate or stopped short at the lower end of the main column cross-section.

9.7. General Remarks Regarding the Holding-Down Bolts:

9.7.1. Pinned Bases Under Axial Loads:

As has been previously outlined in Section 9.3.1.2 if the horizontal shear reactions are small compared to the axial loads (say less than 15%), the friction between the base plate and the reinforced concrete footing should be able to accommodate this modest horizontal reactions. Only nominal holding-down bolts are to be provided to resist any construction loading. The best location of these bolts should be close to the column centre-line.



1- Vertical Column.
2- Base Plate.
3- Mortar.
4- Bolt.
5- R.C. Footing.

Fig. 9.40. Holding-Down Bolt Detail of a Hinged Base Connection

The use of holding-down pockets as shown in Fig. 9.40 is recommended to rend the process of erection easy. However, it is essential that proper attention be paid to the following:

(a) The holding-down pocket is to be cleaned before the steel erection. The existance of any foreign materiel can affect seriously the group which can became inefficient to support the bolt.

(b) Good access to the pocket must be satisfied to ensure that it can be easily filled with compacted fine concrete.

(c) The concrete mixture should be cement, sharp sand, fine aggregate and water in proportions $1:1\frac{1}{4}:2:0.5$. This composition would give workable mortar which can be compacted into the bolt pocket.

(d) A nominal depth of bedding mortar of 40 mm is to be provided for small base connections and a greater figure should be used for larger bases. It will be possible to pack the reduced gap if the top of the concrete is 2 cm high. On the other hand this will allow extra bedding if the concrete footing is low.

(e) The bolts may be cranked if their diameter do not exceed 30 mm. A pocket 150-200 mm deep is to be provided at the top of the bolt as shown in Fig. 9.40.

(f) The bolts should be detailed with an extra extension of 50 mm than required to overcome any decrease that should appear in the concrete footings. This will allow to use the bolts without any extension studs.

(g) For erection purposes, excessive clearance between the bolts and the holes in the base plate is allowed.

9.7.2. Holding-Down Bolt Details for Fixed Base Connections:

For the computation of the induced bolt tension forces it is necessary to investigate the case of loading which gives the maximum value of moment with the minimum normal force (i.e. the case of dead loads with the corresponding critical case of wind loading).



The anchor bolts are generally arranged symmetrically with respect to the column flanges. The diameter of bolts is ranging 20 mm to 76 mm since thicker anchor bolts are very difficult to fabricate.

The bolt tension forces must be transmitted safely to the concrete footings. This can be achieved using one of the following configurations:

(a) J Bolt System:

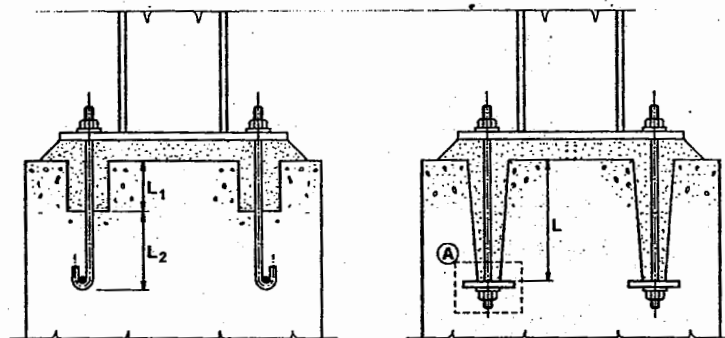
Fig. 9.41.a, illustrates a J bolt configuration which is usually used for bolt diameter less than 24 mm.

The design of the holding down bolts to resist static uplift forces does not present any particular difficulty. The anchorage length (L_2) may be calculated from conventional criteria of plain reinforcing bars using equation 9.6. If only a good access is provided to the bolt pocket to ensure that it can be properly filled with compacted fine concrete the length (L_1) can share by bond the transmission of a portion of the tension uplift bolt forces. Otherwise the length (L_1) is to be excluded.

A common doubtful practice is to wire the J bolts to reinforcement as shown in Fig. 9.41.a. During concreting these reinforcements are disturbed and probably may not work.

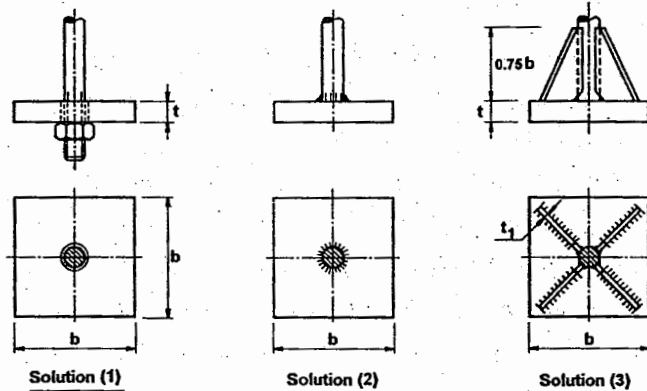
(b) Washer Plate System:

For excessive anchorage length (L_2) which generally corresponds to bolt diameters above 24 mm, the J bolts are replaced by the washer plate system shown in Fig. 9.41.b.



(a) J Bolt System.

(b) Washer Plate System.

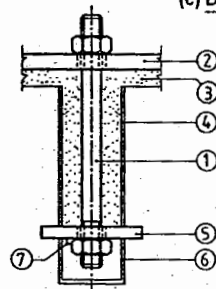


Solution (1)

Solution (2)

Solution (3)

(c) Detail "A"



(d) Cap System.

- (1) Bolt.
- (2) Base plate.
- (3) Grout (Mortar).
- (4) Bolt
- (5) Washer Plate.
- (6) Nut cap.
- (7) Bar to prevent nut rotation.

Fig. 41.



With this system no account is taken of any bond with the grout because of its possible shrinkage. All the tension force is hence resisted by the flexure of the washer plate which may be designed as a simple cantilever upturned base plate. This means that the required safe washer plate thickness can be computed as follows:

$$t = \sqrt{\frac{6 M}{b \cdot F_b}}$$

$$\text{where: } M = \frac{T}{b \times b} \times b \times \left(\frac{b}{2}\right)^2 = \frac{T \cdot b}{4}$$

F_b = allowable bending stress.

b = breadth of the washer plate.

(Refer to equations 9.1, 9.2 and 9.3)

The washer plates may be attached to the bolts either by direct circular fillet welds or by using a lower nut as shown in Fig. 9.41.c. Alternatively if the washer plate thickness is relatively thick, four vertical stiffeners may be welded to both the anchor bolts and the washer plate as shown by solution (3) of Fig. 9.41.c. The design of this latter solution will follow section 9.3.2 regarding the gusseted hinged base connection.

(c) The Cap System:

For larger bolts where washer plates are used it is important to ensure that the bolts are free for adjustment after concreting. This can be achieved by the use of a nut cap as shown in Fig. 9.41.d.



9.8. Pocket Base Connections:

Fig. 9.42.a shows a typical pocket base connection in which the beam column is grouted into a pocket inside the concrete foundation. Depending upon the transmitted straining actions, the connection is regarded either as a hinged or a fixed base connection. Here the fixed type will be considered.

The axial load "N" is resisted by direct bearing and bond between steel and concrete as shown in Fig. 9.42.b. On the other hand, the end moment "M" is resisted by horizontal pressure forces acting on column flanges from the concrete foundation (Fig. 9.42.c).

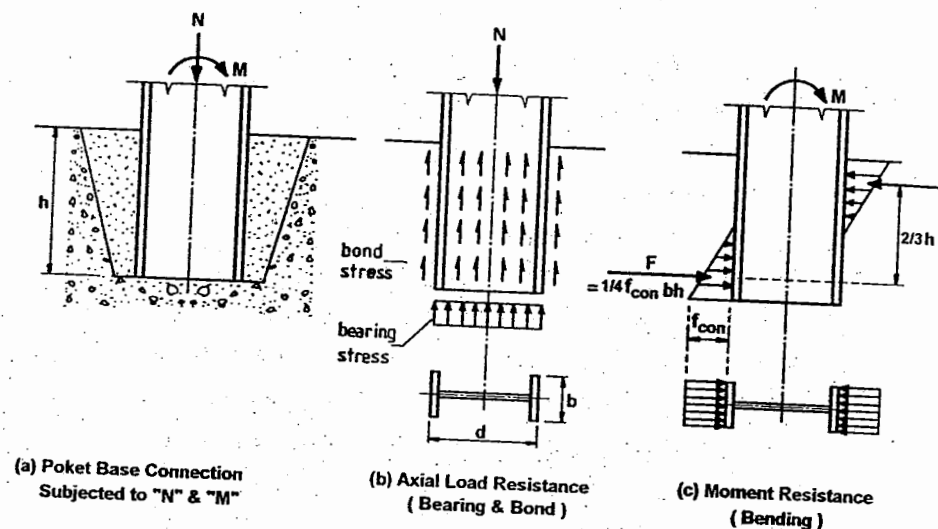


Fig. 9.42. Pocket Base Connection

The required embedded depth of the column can be determined by considering moment equilibrium as shown in Fig. 9.42.c. The minimum depth is then given by:



$$h_{\min} = \sqrt{\frac{6 M}{b F_{\text{con},b}}}$$

9.16

$$h \geq h_{\min}$$

where $F_{\text{con},b}$ is the allowable compressive bending stress of concrete and b is the breadth of the column flanges.

The design value of the embedded depth, h , must not be less than $h_{\min}$ as given by Equation 9.16. This depth must also be checked for bond stress by considering the balance of the vertical forces as shown in Fig. 9.42.b.

Refer to Example 9.18.

9.9. Grillage Base Connection:

Grillage foundation is used in cases of heavily loaded columns. It is made of two or more sets of beams with the successive sets disposed at right angles to each other. The whole system is encased in concrete as shown in Fig. 9.43.

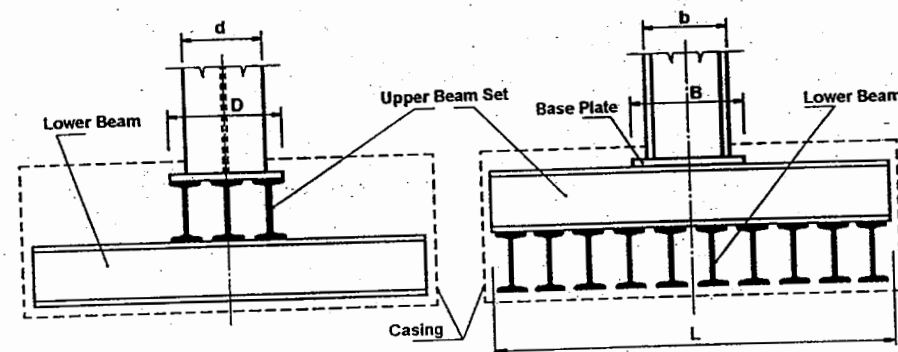


Fig. 9.43. Grillage Base Connection



In the following the analysis of the grillage hinged base connection transmitting only an axial load N is considered. As given in Fig. 9.44.a, the upper beam set is treated as a double cantilever beam. It is assumed that the base plate distributes the vertical load N uniformly over its width D . The load is then transmitted uniformly to the lower set over the length L as indicated in Fig. 9.44.a. The determination of a proper value of the length " L " depends on the safe transmission of the load " N " to the soil.

For a square grillage:

$$L = \sqrt{(N + G) / P} \quad 9.17$$

where : G = Total weight of grillage base.

P = Safe bearing pressure on the soil.

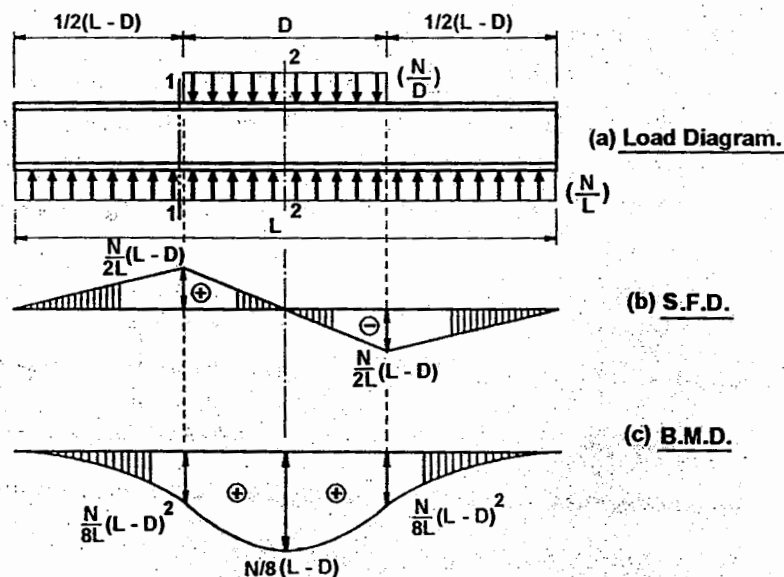


Fig. 9.44. Shear and Moment Distribution on the Upper Beam Set



The required safe cross sections of beams are determined by considering the combined stresses due to both shear and moment at the most critical sections (1-1) and (2-2) as given in Fig. 9.44.b, c. A complete analysis would also consider web buckling and crushing stresses. For more details the reader may refer to Example 9.19.

9.10. Main Types of Support Connections:

In the following, some of the commonly used support connections will be briefly presented.

9.10.1. Hinged Supports for Heavy Vertical Reactions:

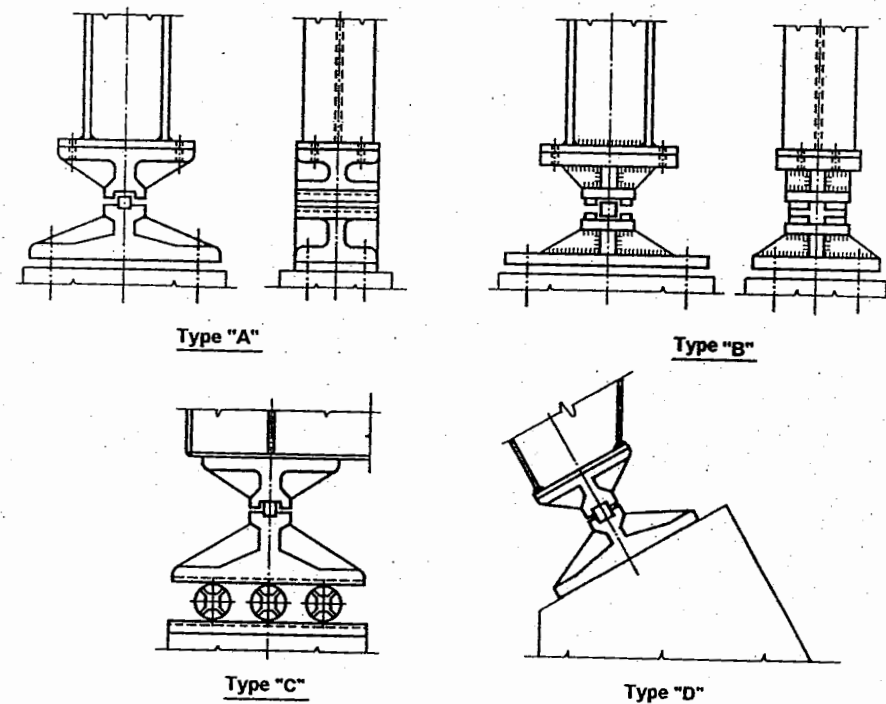


Fig. 9.45



Four different configurations are shown in Fig. 9.45. Configurations (A) and (B) are used for the redistribution of the vertical normal forces transmitted by the support to the foundation over a much larger area.

Type "C" where rollers are provided, allows horizontal displacement of the supported structure. In type "D" the support is inclined in order to transmit a significant horizontal reaction.

Generally these types of hinged supports are used for girders of bridges.

9.10.2. Hinged Supports of Columns on Beams:

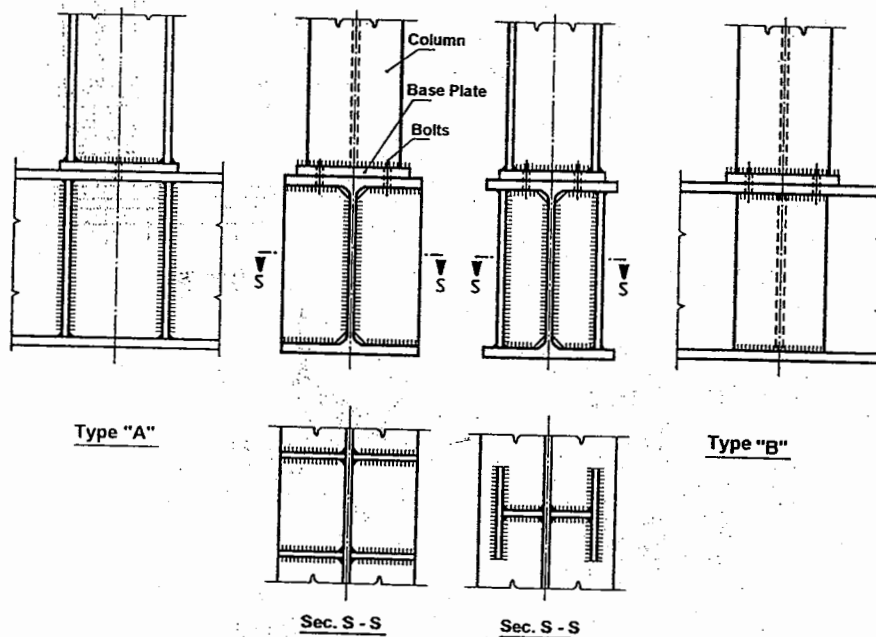


Fig. 9.46. Hinged Supports of Columns on Beams



Fig. 9.46 shows two various types; "A" and "B" in both types an intermediate column rests on the upper flange of a beam girder by a base plate welded to the column end. Vertical stiffeners are attached to the girder's web to prevent crippling. They are designed on the vertical load transmitted from the column to the girder. Types "A" and "B" have two different configurations of vertical stiffeners as shown in Fig. 9.46.

9.10.3. Hinged Support Allowing for Thermal Deformations:

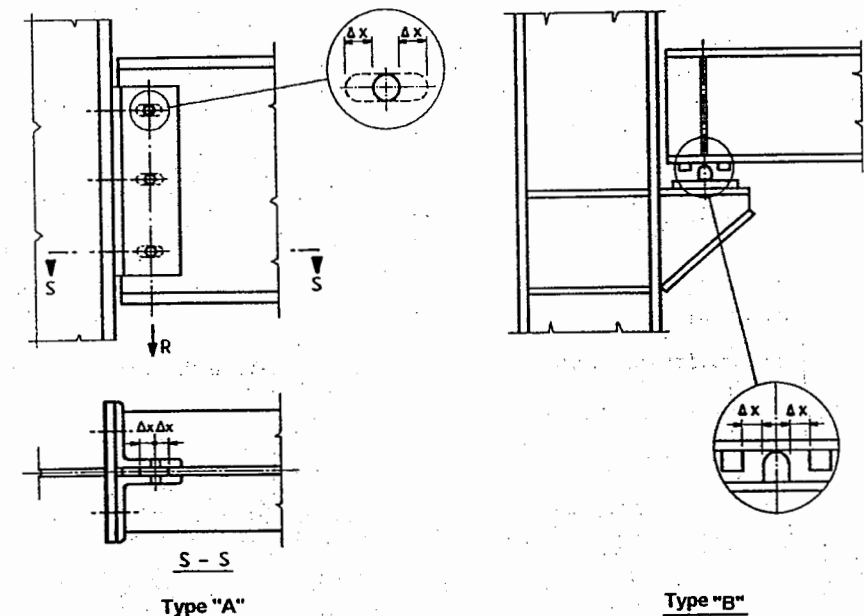


Fig. 9.47. Beam-Column Connection Allowing Horizontal Displacement

Two types are shown in Fig. 9.47. For type "A", the oval holes permit displacement of the beam along x-x direction a



distance Δx from both sides. The vertical reaction R is transmitted by shear.

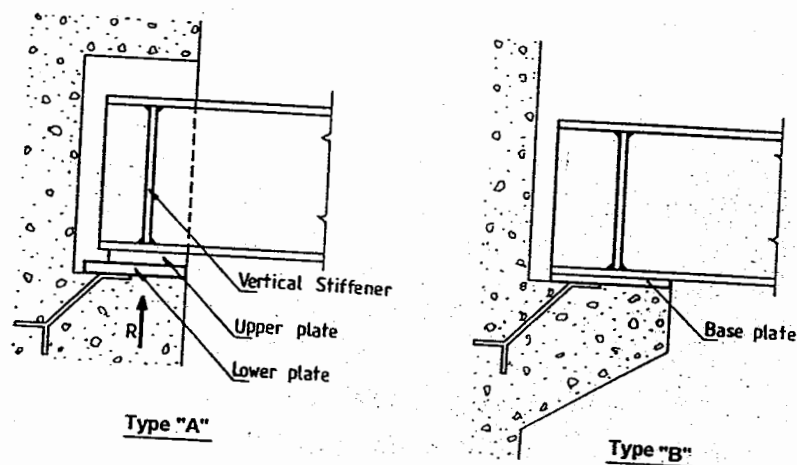
Type "B" is used for heavy loads. The support rests on a steel bracket. It should be noticed that uplift reactions can not be resisted by such a configuration.

9.10.4. Supports of Beams on Masonry Walls or Concrete Columns:

Types of these supports are classified as hinged types and fixed types.

9.10.4.1. Hinged Types:

As shown in Fig. 9.48, Type "A" has two base plates. The upper plate is welded to the beam girder while the lower one rests on the masonry and is welded to anchor bolts embedded in the walls. A vertical stiffener is attached to the beam web in order to safely transmit the vertical reaction R without crippling. The upper base plate ought to have a compact section to provide the largest possible rotation of the support.



Simple Support with Two Base Plates

Support on Bracket

Fig. 9.48. Hinged Supports of Beams on Columns or Walls



In type "B", only one base plate is used. Rotations of such configurations are of minor importance.

9.10.4.2. Fixed Types:

Fig. 9.49, shows two different types for a fixation of a beam in a reinforced concrete column or wall.

For Type "A", the moment is replaced by the two vertical forces (F) transmitted by bearing stresses to the concrete column via a top and a bottom plate.

For Type "B", the moment is replaced by a top tensile force transmitted to the concrete wall by anchorage steel bolts. On the other hand, the compressive force will be transmitted by bearing to the concrete wall.

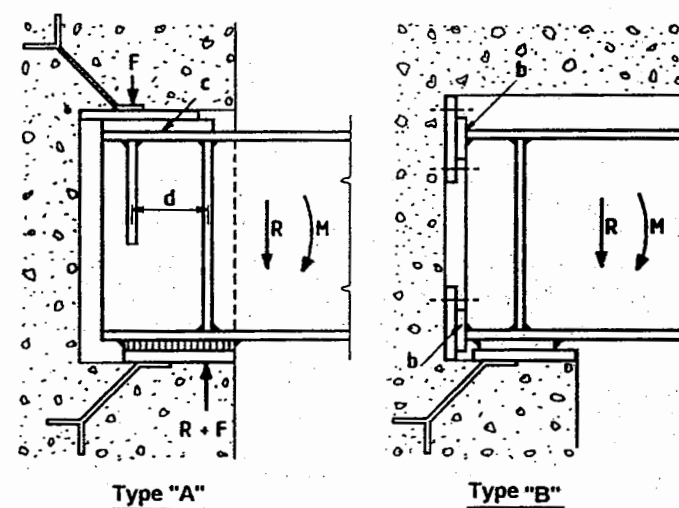


Fig. 9.49. Fixed Supports of Beams on Columns

**Example (9.1):**

Design the hinged base connection of a frame knowing the following:

$$R_{(D.L+L.L)} = 30 \text{ tons} \quad H_{(D.L+L.L)} = 3.0 \text{ ton}$$

The cross section of the column is a B.F.I.B. No. (30).

Solution

Refer to Section 9.3.1.2

(1) Base Plate:

Take $m = n = 5.0$ cms.

$$D = 0.95 \times 30 + 2 \times 5 \\ = 38.50 \text{ cms}$$

$$B = 0.80 \times 30 + 2 \times 5 \\ = 34.0 \text{ cms.}$$

$$f_{con} = \frac{30 \times 1000}{34 \times 38.50} \\ = 22.9 \text{ kg/cm}^2 \\ < 50 \text{ kg/cm}^2 \text{ Safe}$$

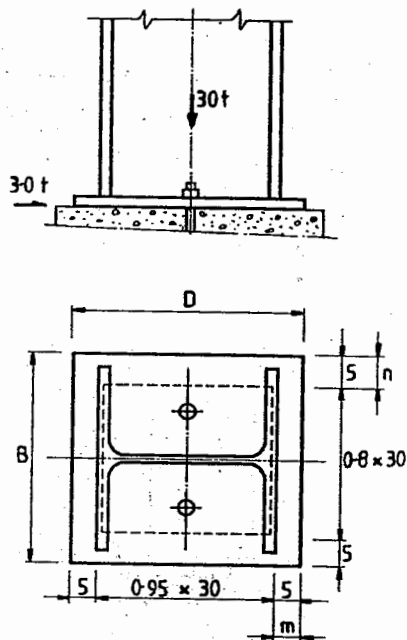
$$M/\text{unit width} = \frac{22.9}{1000} \times \frac{(5)^2}{2} \\ = 0.28625 \text{ t.cm}$$

$$Z = \frac{I}{Y} = \frac{t^2}{6}$$

$$f = \frac{M}{Z} = \frac{M \times 6}{t^2}, \quad F_b = 0.72 \times 2.4 = 1.728 \text{ t/cm}^2 \\ (\text{solid rectangular section}) - \text{Table 5.6}$$

$$t = \sqrt{\frac{6M}{F_b}} = \sqrt{\frac{6 \times 0.28625}{1.728}} = 0.99 \text{ cm.}$$

∴ Base plate is (38.5 x 34 x 1.0)

**(2) Welds:****A. Assuming Complete Contact Between Column and Base Plate:**

In this case the total amount of vertical load N goes directly from the column section to the foundation by contact via the base plate. While the horizontal force H is transmitted from the column to the base plate by welds, then to the foundation by shear on bolts.

$$\text{Allowable stress of weld, } f_{all} = 0.5 f_t = 0.70 \text{ t/cm}^2.$$

$$A_{weld(1-1)} = A_{weld(2-2)} \\ = s \times b \times 2 \\ = 60 s.$$

Section (1-1): Assuming $s = 0.40$ cm

(minimum weld dimension from E.C.P)

$$A_{weld} = 60 \times 0.40 = 24.0 \text{ cm}^2.$$

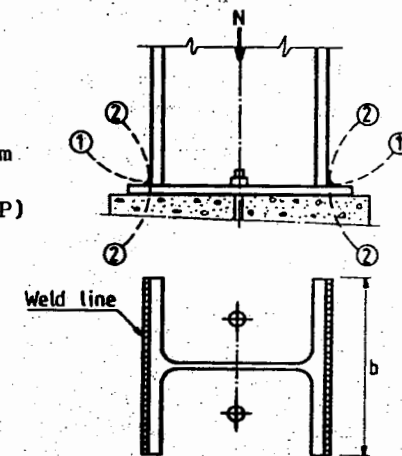
$$H = 3.0 \text{ ton}$$

$$q_{//} = \frac{3}{24} = 0.125 \text{ t/cm}^2 \\ < 0.72 \text{ t/cm}^2 \text{ Safe}$$

Section (2-2):

$$f_{\perp} = \frac{3}{2 \times 30 \times 0.40} \\ = 0.125 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2$$

Safe





B. If we assume no Contact Between the Column & the Base Plate:-

Assuming all of the force N will be transmitted to the base plate by fillet welds (end of column is not milled). Hence the welds will carry a horizontal force (H) as well as the vertical force N . This means that:

$$R = \sqrt{H^2 + N^2}$$

$$\therefore R = \sqrt{(3)^2 + (30)^2} = 30.15 \text{ tons}$$

Section (1-1): $s = 1.00 \text{ cm}$

$$q_{//} = \frac{3}{2 \times 30 \times 1.0} = 0.05 \text{ t/cm}^2$$

$$f_{\perp} = \frac{30}{2 \times 30 \times 1.0} = 0.50 \text{ t/cm}^2$$

$$\therefore f_{eq} = \sqrt{(0.5)^2 + 3(0.05)^2} = 0.502 \text{ t/cm}^2 < 0.72 \times 1.10 \text{ t/cm}^2$$

$$\therefore \text{Use } s = 1.0 \times \frac{0.502}{0.72} = 0.697 \text{ cm} \quad \text{Take } s = 0.7 \text{ cms}$$

Section (2-2):

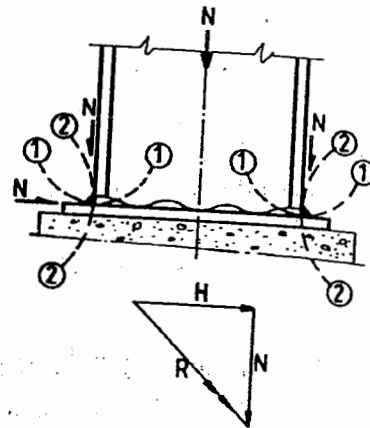
$$q_{\perp} = \frac{30}{2 \times 30 \times 1.0} = 0.50 \text{ t/cm}^2$$

$$f_{\perp} = \frac{3}{2 \times 30 \times 1.0} = 0.05 \text{ t/cm}^2$$

$$\therefore f_{eq} = \sqrt{(0.05)^2 + 3(0.5)^2} = 0.86 \text{ t/cm}^2 > 1.1 \times 0.72 \text{ t/cm}^2$$

$\therefore$ unsafe

$$\therefore \text{Use weld of size } s = \frac{1 \times 0.86}{0.792} = 1.08 \text{ cm. use } s = 1.1 \text{ cms}$$



The base plate thickness is to be increased to 1.1 cms

Bolts:

Bolts will transmit the total horizontal force (H) by direct shear to the foundation.

$$q_{\text{bolt}} = \frac{3}{2 \times A_{\text{bolt}}} \quad (2 \text{ bolts})$$

$$q_{\text{all}} = 0.25 \times 4.0 = 1.0 \text{ t/cm}^2 \text{ (single shear)}$$

$$\therefore A_{\text{bolt}} = \frac{3}{2 \times 1.0} = 1.5 \text{ cm}^2$$

$\therefore$ Use bolts ϕ (16 mms)

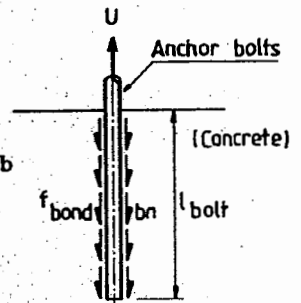
As the bolt doesn't carry any tension force, thus its length is taken as a minimum of 20 times the diameter (i.e. $L_{\text{bolt}} = 20 \times 1.6 = 32 \text{ cms}$)

Note: If there is a case of loading for which ($N = U$) is upward $\uparrow (R_{D.L+W.L} \uparrow)$, then: This force will be transmitted from the column to the base plate by the welds, then the base plate will transmit the force to the bolts which will transmit it to the foundation by bond.

$$\therefore q_{\text{bolt}} = \frac{H}{2 A_{\text{bolt}}}, \quad f_{t \text{ bolt}} = \frac{U}{2 A_{\text{bolt}}}$$

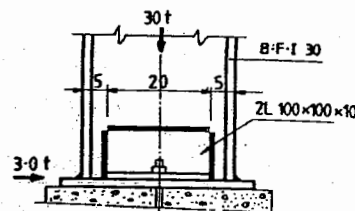
$$\therefore f_{eq} = \sqrt{(f_{t \text{ bolt}})^2 + 3(q_{\text{bolt}})^2} \leq 1.1 F_{tb}$$

$$\therefore L_{\text{bolt}} = \frac{T_{\text{bolt}}}{\pi d \times f_{\text{(all. bond)}}}$$



Example (9.2):

Design a hinged base connection of the conventional type (Type B) to carry a vertical force ($N = 30$ tons) and a horizontal reaction ($H = 3$ tons). No contact is to be considered



Solution

Refer to Section 9.3.1.2. and Fig. 9.10

(1) Concrete:

$$F_{\text{concrete}} = 60 \text{ kg/cm}^2$$

$$A_{\text{min base plate}} = \frac{30 \times 1000}{60} = 500 \text{ cm}^2$$

$$\text{Take } m = n = 5.0 \text{ cms}$$

$$\therefore D = 0.95 \times 30 + 10 = 38.50 \text{ cms}$$

$$\therefore B = 0.80 \times 30 + 10 = 34.0 \text{ cms}$$

$$f_{\text{conc}} = \frac{30 \times 1000}{34 \times 38.50} = 22.90 \text{ kg/cm}^2$$

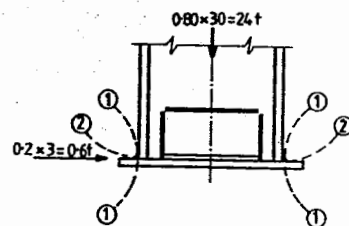
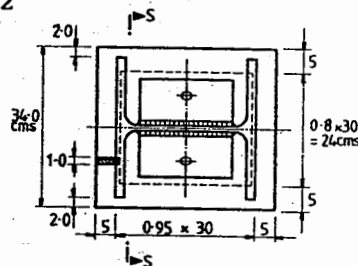
$$< 60 \text{ kg/cm}^2 \text{ Safe}$$

(2) Base Plate:

$$M/\text{unit width} = \frac{22.9}{1000} \times 1.0 \times \frac{(5)^2}{2}$$

$$= 0.286 \text{ t.cm}$$

$$\therefore t = \sqrt{\frac{6M}{bF_b}} = \sqrt{\frac{6 \times 0.286}{1.0 \times 0.72 \times 2.4}} = 0.99 = 1.0 \text{ cm}$$



(3) Flange Welds (Refer to Fig. 9.9.b):

Flange welds carry about 80% of the normal force "N" and 20% of the shearing force "H".

Assume size of weld = 1.0 cm.

Sec. (1-1):

$$q_{\perp} = \frac{0.8 \times 30}{30 \times 1.0 \times 2} = 0.40 \text{ t/cm}^2$$

$$f_{\perp} = \frac{0.2 \times 3.0}{30 \times 1.0 \times 2} = 0.01 \text{ t/cm}^2$$

$$\therefore f_{\text{eq}} = \sqrt{(0.01)^2 + 3(0.4)^2} = 0.70 \text{ t/cm}^2 < 1.1 \times 0.72 \text{ t/cm}^2 \text{ Safe}$$

Sec. (2-2):

$$f_{\perp} = \frac{0.8 \times 30}{30 \times 1.0 \times 2} = 0.40 \text{ t/cm}^2$$

$$q_{//} = \frac{0.2 \times 3.0}{30 \times 1.0 \times 2} = 0.01 \text{ t/cm}^2$$

$$\therefore f_{\text{eq}} = \sqrt{(0.4)^2 + 3(0.01)^2} = 0.40 \text{ t/cm}^2 < 0.72 \times 1.1 \text{ t/cm}^2$$

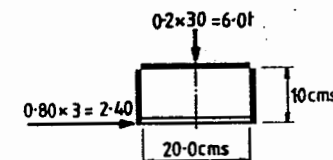
Safe

(4) Welds Between Web and Angles (Refer to Fig. 9.9.b):

Approximate Analysis

These welds carry the remaining 20% of the normal force and 80% of the shearing force.

Assume that the horizontal weld carries the horizontal force and the vertical welds carry the vertical forces.





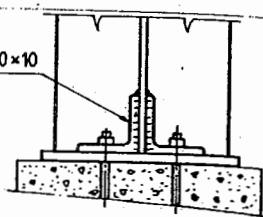
$$s_{\text{vertical weld}} = \frac{4.2}{0.72 \times 2 \times 10}$$

$$= 0.30 \text{ cm}$$

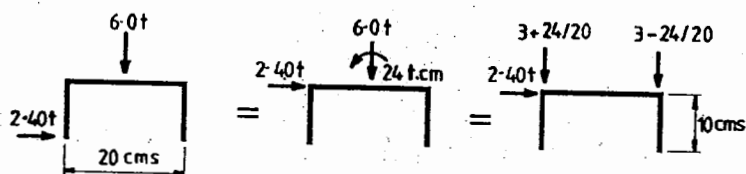
$$\therefore s_{\text{horizontal weld}} = \frac{2.40}{0.72 \times 20 \times 2}$$

$$\approx 0.10 \text{ cm}$$

2L 100×100×10



Sec. (S - S)

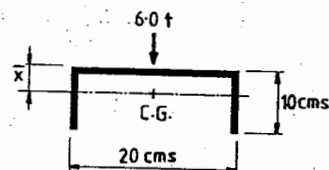


$\therefore$ Take the size of welds $s_w = 4 \text{ mm}$ (min. size)

Exact Analysis

$s = 0.4 \text{ cm}$. From table 3.2 get :-

$$\bar{X} = \frac{(10)^2}{2 \times 10 + 20} = 2.50 \text{ cms.}$$



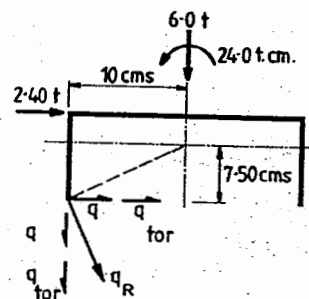
$$\therefore I_p = \left\{ \frac{8 \times 10^3}{12} + \frac{6 \times 10 \times (20)^2}{12} + \frac{(20)^3}{12} - \frac{(10)^4}{(2 \times 10 + 20)} \right\} s$$

$$= 3083 \times 0.40 = 1233.3 \text{ cm}^4.$$

$$M_p = 2.4 \times (10 - 2.50) = 18 \text{ t.cm.}$$

$$q_{\text{torsion}} = \frac{18 \times 10}{1233.3} = 0.15 \text{ t/cm}^2$$

$$q_{\text{torsion}} = \frac{18 \times 7.50}{1233.3} = 0.11 \text{ t/cm}^2$$



$$q_{\downarrow} = \frac{6}{40 \times 0.40} = 0.375 \text{ t/cm}^2$$

$$q_{\rightarrow} = \frac{2.40}{40 \times 0.40} = 0.15 \text{ t/cm}^2$$

$$q_R = \sqrt{(0.15 + 0.375)^2 + (0.15 + 0.11)^2} = 0.58 \text{ t/cm}^2$$

$< 0.72 \text{ t/cm}^2$ Safe

(5) Anchor Bolts:

The anchor bolts carry the horizontal shearing force only.

$$F/\text{bolt} = \frac{3}{2} = 1.50 \text{ ton}$$

$$q_b = 0.25 \times 4.0 = 1.0 \text{ t/cm}^2$$

$$A_{\text{bolt}} = \frac{1.5}{1.0} = 1.5 \text{ cm}^2$$

$\therefore$ Use 2 anchor bolts $d = 16 \text{ mm}$ with a length 32 cms.

Example (9.3):

Design a hinged base connection of the conventional type (Type C) to carry a vertical force ($N = 50$ tons) and a horizontal reaction ($H = 3.0$ ton)

Solution

Refer to Section 9.3.1.3 and Fig. 9.11

(1) Concrete:

$$f_{\text{con.}} = \frac{50 \times 10^3}{34 \times 52} = 28.28 \text{ kg/cm}^2$$

$$< 60 \text{ kg/cm}^2$$

(2) Base Plate:

$$m = 1 + 10 - 2 = 9.0 \text{ cms}$$

$$\frac{M}{(s_1 - s_1)} = \frac{28.28}{1000} \times 1.0 \times \frac{(9)^2}{2} = 1.145 \text{ t.cm}$$

$$\therefore t = \sqrt{\frac{6M}{bf_b}} = \sqrt{\frac{6 \times 1.145}{1.0 \times 0.72 \times 2.4}} = 1.99 \text{ take } t = 2.0 \text{ cms}$$

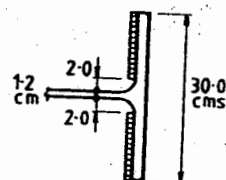
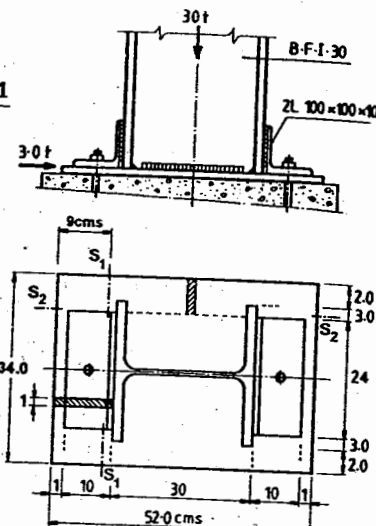
= 1.0 (angle) + 1.0 (base plate)

$$n = \frac{1}{2} (30 - 0.8 \times 30) + 2 = 5.0 \text{ cms}$$

$$M_{s_2 - s_2} = 0.354 \text{ t.cm} \quad ; \quad t = 1.1 \text{ cms}$$

(3) Flange Welds:

Flange welds carry 30% of the normal force and 10% of the shearing force.



$$\therefore \text{Length of weld} = 2 (30 - 1.20 - 2 - 2) = 49.60 \text{ cms.}$$

Assume size of weld = 7 mms

for Sec. (1-1):

$$q_{\perp} = \frac{50 \times 0.3}{49.60 \times 0.7} = 0.43 \text{ t/cm}^2$$

$$f_{\perp} = \frac{3 \times 0.1}{49.60 \times 0.7} = 0.0086 \text{ t/cm}^2$$

$$\therefore f_{eq} = \sqrt{(0.0086)^2 + 3(0.43)^2}$$

$$= 0.744 \text{ t/cm}^2 < 0.792 \text{ t/cm}^2$$

∴ Use size of weld = 7 mms.

for Sec. (2-2):

$$f_{\perp} = \frac{50 \times 0.3}{49.60 \times 0.7} = 0.43 \text{ t/cm}^2$$

$$q_{\perp} = \frac{3 \times 0.1}{49.60 \times 0.7} = 0.0086 \text{ t/cm}^2$$

$$\therefore f_{eq} = \sqrt{(0.43)^2 + 3(0.0086)^2} = 0.43 \text{ t/cm}^2 < 0.792 \text{ t/cm}^2 \text{ Safe}$$

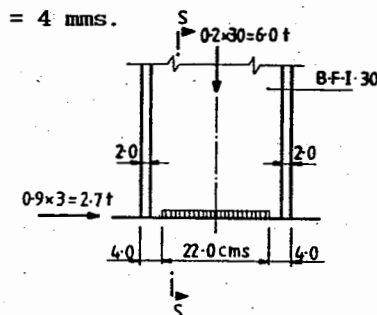
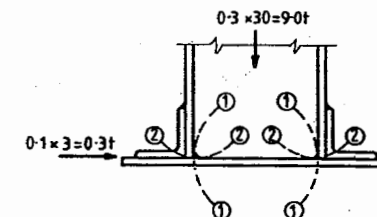
(4) Web Welds:

Web welds carry 20% of the normal force and 90% of the shearing force. Assume size of weld = 4 mm. S

* Vertical Face (V) :-

$$q_{\perp} = \frac{50 \times 0.2}{0.4 \times 22 \times 2} = 0.568 \text{ t/cm}^2$$

$$q_{\text{с}} = \frac{3 \times 0.9}{0.4 \times 22 \times 2} = 0.153 \text{ т/см}^2$$





$$q_R = \sqrt{(0.568)^2 + (0.153)^2}$$

$$= 0.588 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2$$

∴ Safe

* Horizontal Face (H) :-

$$q_{//} = \frac{3 \times 0.9}{0.40 \times 22 \times 2.0} = 0.153 \text{ t/cm}^2$$

$$f_{\perp} = \frac{.50 \times 0.2}{0.4 \times 22 \times 2.0} = 0.568 \text{ t/cm}^2$$

$$\therefore f_{eq} = \sqrt{(0.568)^2 + 3(0.153)^2}$$

$$= 0.626 \text{ t/cm}^2 < 0.792 \text{ t/cm}^2$$

∴ Safe

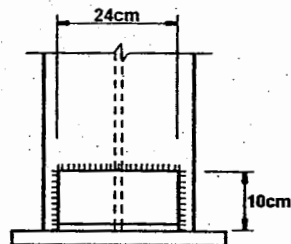
(5) Angle Welds:

Angles carry the remaining 50% of the normal force. The total length of weld lines = $2 \times 24 + 4 \times 10 = 88 \text{ cms}$.

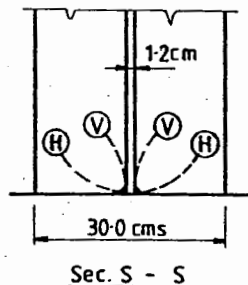
$$\therefore s = \frac{0.50 \times 50}{88 \times 0.72} = 0.394 \text{ cm}$$

∴ Take $s = 4 \text{ mms}$

Then the horizontal legs of the angles will transmit by contact this 50 % of normal force to the base plate.



(6) Anchor Bolts: as Example 9.2

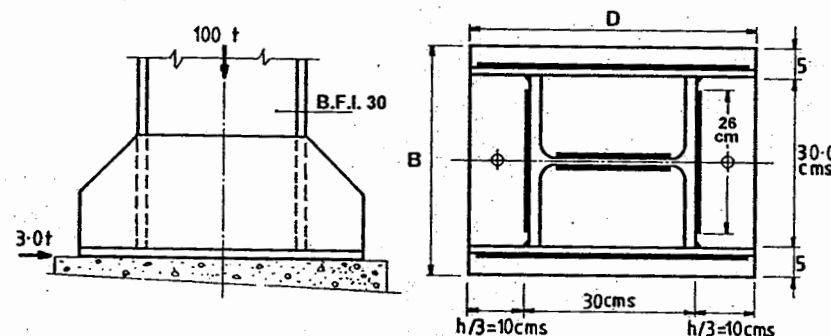


Example (9.4):

Design a hinged base connection of the conventional type (Type D) to carry a vertical force ($N = 100 \text{ tons}$) and a horizontal reaction ($H = 3.0 \text{ tons}$).

Solution

Refer to Section 9.3.2.1



(1) Concrete:

$$f_{con} = \frac{100 \times 1000}{50 \times 40} = 50 \text{ kg/cm}^2 < 60 \text{ kg/cm}^2 \text{ Safe}$$

(2) Flange Welds (Refer to Fig. 9.12):

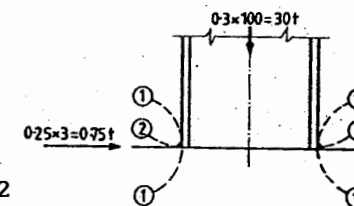
Flange welds carry 30% of the normal force and 25% of the shearing force.

* Sec. (1-1):

Assume size of weld = 10 mms.

$$q_{\perp} = \frac{0.30 \times 100}{2 \times 26} = 0.576 \text{ t/cm}^2$$

$$f_{\perp} = \frac{0.25 \times 3.0}{2 \times 26} = 0.014 \text{ t/cm}^2$$





$$f_{eq.} = \sqrt{(0.014)^2 + 3(0.576)^2}$$

$$= 0.99 \text{ t/cm}^2 > 0.792 \text{ t/cm}^2 \text{ Unsafe} \quad \text{Use } s = \frac{1 \times 0.99}{0.792} = 1.25 \text{ cms}$$

* Sec. (2-2):

$$q_{\perp} = 0.443 \text{ t/cm}^2 \quad q_{\parallel} = 0.011 \text{ t/cm}^2 \quad (s = 13 \text{ mms})$$

$$\therefore f_{eq.} = \sqrt{(0.443)^2 + 3(0.011)^2} = 0.44 \text{ t/cm}^2 < 0.792 \text{ t/cm}^2$$

(3) Web Welds:

Web welds will carry 20% of the normal force and 75% of the shearing force.

Assume the size of welds = 7 mms.

* Vertical Face:

$$q_{\perp} = \frac{0.2 \times 100}{2 \times 22 \times 0.70} = 0.65 \text{ t/cm}^2$$

$$q_{\parallel} = \frac{0.75 \times 3}{2 \times 22 \times 0.70} = 0.07 \text{ t/cm}^2$$

$$q_r = \sqrt{(0.65)^2 + (0.07)^2} = 0.65 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ Safe}$$

* Horizontal Face:

$$q_{\perp} = 0.65 \text{ t/cm}^2, \quad q_{\parallel} = 0.07 \text{ t/cm}^2$$

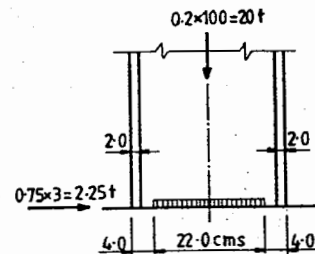
$$f_{eq.} = \sqrt{(0.65)^2 + 3(0.07)^2} = 0.66 \text{ t/cm}^2 < 1.1 \times 0.72 \text{ t/cm}^2$$

(4) Stiffeners:

Stiffeners carry the uplift stress due to the normal force.

Assume thickness of stiffeners = 1.0 cm.

Refer to Eq. 9.11.a.



$$Q = \frac{N \cdot a}{2 \cdot D} = \frac{100 \times 10}{2 \times 50} = 10 \text{ tons.}$$

$$M_x = \frac{100 \times 10^2}{4 \times 50} = 50 \text{ t.cm.}$$

Take $h_{st} = 20 \text{ cms}$; $t_{st} = 1.0 \text{ cms}$

$$f_b = \frac{6 \times 50}{t \cdot h^2} = \frac{6 \times 50}{1 \cdot (20)^2} = 0.75 \text{ t/cm}^2$$

$$q = \frac{3 \times 10}{2 \times 1 \times 20} = 0.75 \text{ t/cm}^2$$

$$f_{eq.} = \sqrt{(0.75)^2 + 3(0.75)^2}$$

$$= 1.50 \text{ t/cm}^2 < 1.1 \times (0.72 \times 2.4) \text{ t/cm}^2$$

(Rectangular solid section)

(5) Stiffeners Welds (50% of N):

Each of these four weld lines transmits a vertical force F

$$F = w (10 + 15) = 25w \text{ tons}$$

$$w = 0.5 \times 100 / 2 \times 50 = 0.5 \text{ t/cm}$$

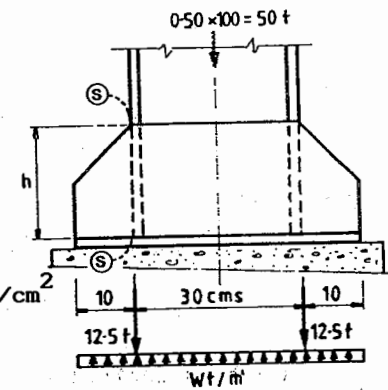
$$F = 25 \times 0.5 = 12.5 \text{ tons}$$

$$s_w = \frac{12.50}{20 \times 0.72} = 0.88 \text{ cm} < t_{st.} \text{ (Safe)} \quad s = 10 \text{ mms.}$$

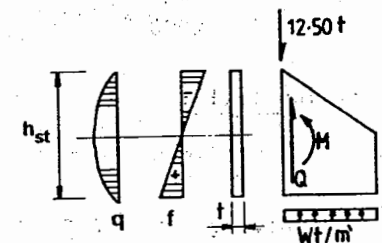
Take stiffeners 20 x 1.0 cms

(6) Horizontal Welds for Stiffeners:

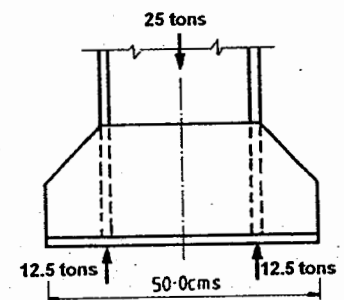
$$s = \frac{25}{50 \times 0.72} = 0.694 \text{ cms}$$



Stresses on Stiffener



Sec. in Stiffener s-s





(7) Base Plate: (Moments are Calculated for unit width)

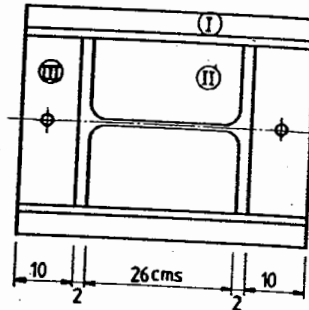
Region (I):

$$M = \frac{50}{1000} \times \frac{(4)^2}{2} = 0.40 \text{ t.cm}$$

Region (II):

$$a' = 26 \text{ cms} ; b' = \frac{30 - 1.2}{2} = 14.4 \text{ cms}$$

$$a' / b' = 26 / 14.4 = 1.8$$



Hence use Table 9.1 to get $\alpha_1 = 0.094$, $\alpha_2 = 0.048$

$$M a' = \frac{50}{1000} \times 0.048 \times (26)^2 = 1.62 \text{ t.cm}$$

$$M b' = \frac{50}{1000} \times 0.094 \times (14.4)^2 = 0.97 \text{ t.cm}$$

Region (III):

$$a = 10 \text{ cms} ; b = 30 \text{ cms} ; a/b = 10/30 = 0.33$$

Having $a/b < 0.5$, then this region is to be treated as a cantilever portion.

$$M_3 = \frac{50}{1000} \times \frac{(10)^2}{2} = 2.5 \text{ t.cm}$$

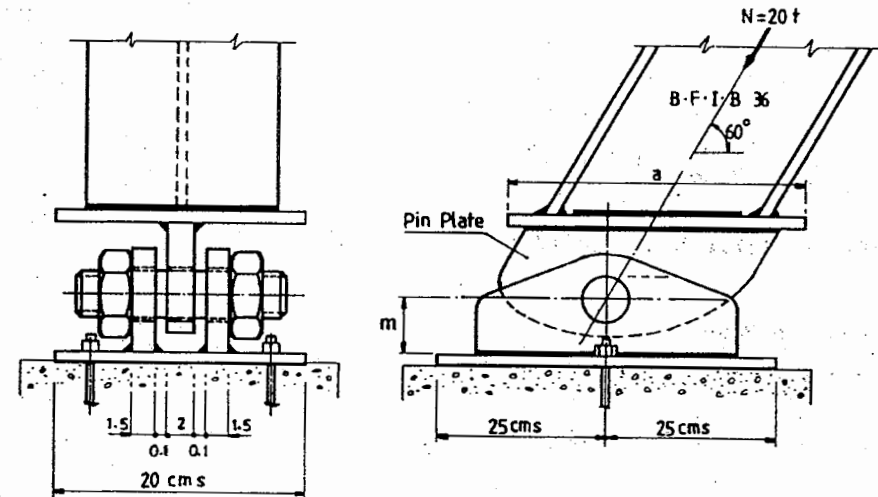
$$M_{\text{Max}} = 2.5 \text{ t} \quad t_p = \sqrt{\frac{6 \times 2.5}{0.72 \times 2.4}} = 2.94 \text{ cms} \text{ take } t_p = 3.0 \text{ cms}$$



Example (9.5):

Design the pin connection of an arched structure. The details of the pin are as shown in the figure.

Solution



$$a = \frac{36}{\sin 60^\circ} + 2 \times 4 = 49.57 \approx 50$$

Pin Plate:

Assume the thickness of pin plate $t = 2.0 \text{ cms}$

Assume the diameter of pin $d = 5.0 \text{ cms}$

Shear on Pin:

$$q = \frac{20}{2 \times \frac{\pi \times (5)^2}{4}} = 0.51 \text{ t/cm}^2 < 0.25 \times 4 \quad \text{O.K.}$$

$$\therefore f_b = \frac{20}{2 \times 5.0} = 2.0 \text{ t/cm}^2 < 0.6 F_u$$

Take edge distance $= 1.75 d = 8.75 \text{ cms}$

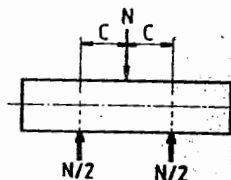
Bending on Pin:

Consider forces Concentrated at the center of each pin plate.

$$Z_x = \frac{\pi d^3}{32} = \frac{3.14 \times (5)^3}{32} = 12.26 \text{ cm}^3$$

$$M = \frac{20}{2} \times 1.85 = 18.5 \text{ t.cm}$$

$$f_b = \frac{18.5}{12.26} = 1.5 \text{ t/cm}^2 < 0.72 \times 2.4 \text{ t/cm}^2 \text{ O.K.}$$



Refer to table 5.6 - Solid circular section.

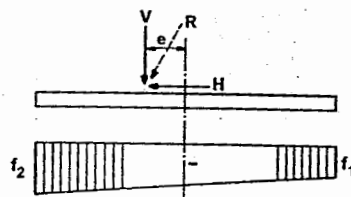
Stress on Base:

$$m = 8.75 + 2.5 = 11.25 \text{ cms take } m = 11.5 \text{ cms}$$

$$e = 9 \cot (60^\circ) = 6.64 \text{ cms}$$

$$V = 20 \sin (60^\circ) = 17.32 \text{ tons}$$

$$H = 20 \cos (60^\circ) = 10.0 \text{ tons}$$



$$f_1 = \frac{-17.32}{50 \times 20} \pm \frac{17.32 \times 6.64}{20 \times (50)^2/6} = -0.0173 \pm 0.0138 \text{ t/cm}^2$$

$$\therefore f_1 = -3.5 \text{ kg/cm}^2 < F_{\text{con}}$$

$$\therefore f_2 = -31.1 \text{ kg/cm}^2 < F_{\text{con}}$$

Height of Pin Plate:

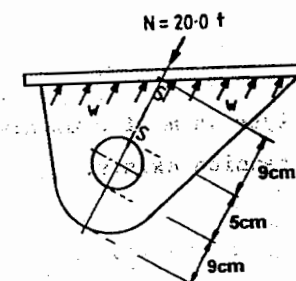
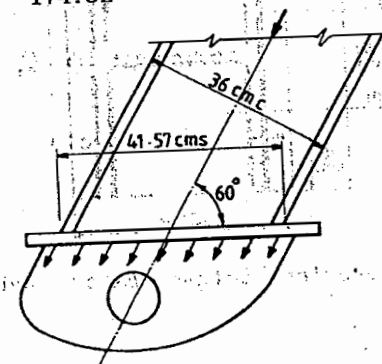
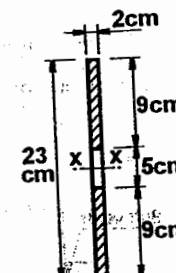
$$W = \frac{20}{41.57} = 0.481 \text{ t/cm}$$

$$M_{s-s} = 0.481 \times \frac{41.57}{2} \times \frac{1}{2} \times \frac{41.57}{2} = 103.89 \text{ t.cm}$$

$$I_x = \frac{2(23^3 - 5^3)}{12} = 2007 \text{ cm}^4$$

$$Z_x = \frac{2007}{11.5} = 174.52 \text{ cm}^3$$

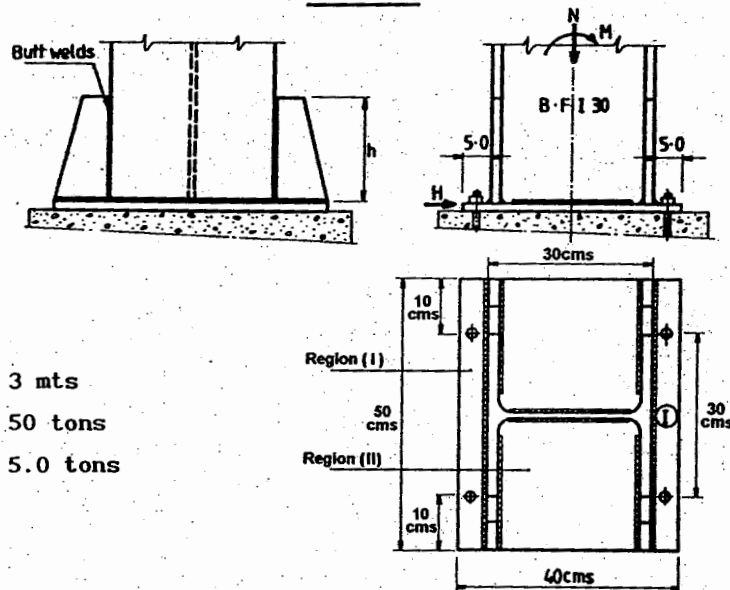
$$f_b = \frac{103.89}{174.52} = 0.595 \text{ t/cm}^2 < 1.4 \text{ O.K.}$$



Example (9.6):

Design a fixed base connection of the bracket type (Type "1") to support a vertical force $N = 50$ tons, a bending moment $M = 3$ mt and a horizontal force $H = 5$ tons.

Solution



$M = 3$ mts
 $N = 50$ tons
 $H = 5.0$ tons

This type is most commonly used for column bases under which no tension exists.

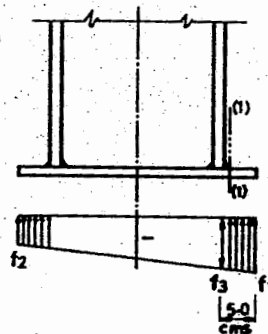
i.e. $e < \frac{B}{6}$ or the resultant is inside the core.

$$e = \frac{M}{N} = \frac{300}{50} = 6.0 \text{ cms} < \frac{40}{6} < 6.66 \text{ cm}$$

(1) Stresses on Concrete:

$$f_1 = -\frac{N}{A} \pm \frac{M.y}{I}$$

$$\therefore f_1 = -\frac{50}{40 \times 50} \pm \frac{300 \times 20}{50 \times 40^3 / 12}$$



$$f_1 = -0.0475 \text{ t/cm}^2 ; f_2 = -0.0025 \text{ t/cm}^2$$

$$f_1 = -47.5 \text{ kg/cm}^2 ; f_2 = -2.5 \text{ kg/cm}^2 \text{ (o.k. } < 60 \text{ kg/cm}^2 \text{)}$$

(2) Base Plate:

Region (I): (Cantilever action)

The cantilever length being 5.0 cms.

$$\text{Thus, } f_3 = \frac{50}{40 \times 50} + \frac{300 \times 15}{50 \times (40)^3 / 12} = 0.041875 \text{ t/cm}^2$$

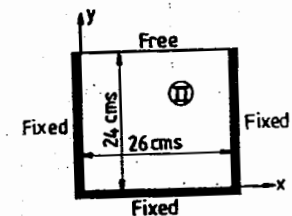
$$\text{and } f_1 = 0.0475 \text{ t/cm}^2$$

The moment here is calculated for the whole breadth ($B = 50$ cms)

$$M_{1-1} = 0.041875 \times 5 \times 50 \times 2.50 + \frac{1}{2} (0.0475 - 0.041875) \times 5 \times 50 \times \frac{2}{3} \times 5 = 28.52 \text{ t.cm}$$

$$t_1 = \sqrt{\frac{6M}{b F_b}} = (F_b = 0.72 F_y)$$

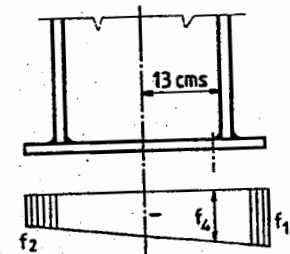
$$= \sqrt{\frac{6 \times 28.52}{50 \times 0.72 \times 2.4}} = 1.4 \text{ cm} \quad (1)$$



Region (II): (plate supported on three sides)

We shall consider that this region is affected by a uniformly distributed load with a value equal to the maximum stress under that region. (i.e. f_4)

$$\therefore f_4 = \left[\frac{50}{40 \times 50} + \frac{300 \times 13}{50 \times 40^3 / 12} \right] \times 1000 = 39.63 \text{ kgm/cm}^2$$



$$f \approx 0.04 \text{ ton/cm}^2$$

$$b = 26 \text{ cms} ; a = 24.4 \text{ cms} ; a/b = 24.4/26 = 0.94$$

Use Table 9.1 where $\alpha_3 = 0.11$

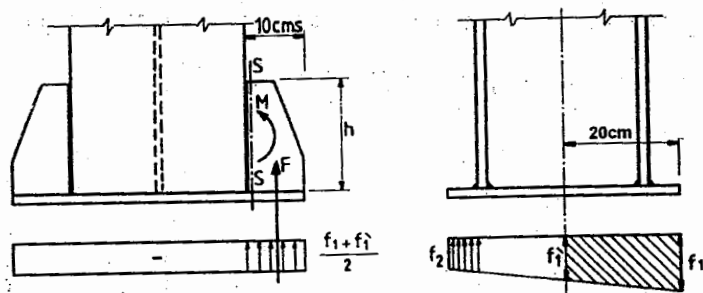
$$M_3 = 0.11 \times (26)^2 \times 0.04 = 2.97 \text{ t.cm}$$

$$t_{\text{req.}} = \sqrt{\frac{6 \times 2.97}{1 \times 0.72 \times 2.4}} = 3.21 \text{ cms} \quad (2)$$

From (1) & (2) thickness of the base plate = 3.6 cms.

(3) Height of Bracket:

The stiffener will withstand the shaded region of stresses.



$$f_1' = \frac{f_1 + f_2}{2} = \frac{0.0475 + 0.0025}{2} = 0.025 \text{ t/cm}^2$$

$$F = \frac{f_1' + f_1}{2} \times 20 \times 10 = 7.25 \text{ tons} ; M = F \times \frac{10}{2} = 36.24 \text{ t.cm}$$

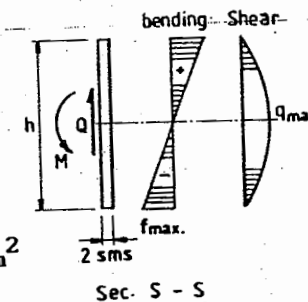
$$\text{Sec. } (S_1 - S_1): Q = 7.25 \text{ tons} ; M = 36.25 \text{ t.cm}$$

$$t_{\text{st}} = t_{\text{fc}} = 2.0 \text{ cms} ; h_{\text{st}} = 10 \text{ cms}$$

$$f_b = \frac{36.25 \times (h/2)}{2 \times (h^3/12)} = 1.08 \text{ t/cm}^2$$

$$q_{\text{max}} = \frac{3}{2} \times \frac{Q}{A} = \frac{3}{2} \times \frac{7.25}{2 \times h} = 0.543 \text{ t/cm}^2$$

$$f_{\text{eq.}} = \sqrt{(1.08)^2 + 3(0.543)^2} = 1.43 \text{ t/cm}^2 < 1.1 \times 0.72 \times 2.4 \text{ t/cm}^2$$

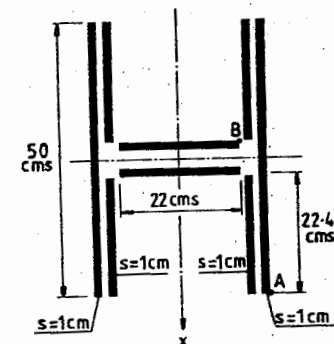


(4) Welds to the Base Plate:

Assume the size of welds is 1 cm around the flanges and the web.

$$A_w = (22 \times 2 \times 1) + 4(22.4 \times 1.0) + 2 \times 50 \times 1 = 233.60 \text{ cm}^2$$

$$I_{xw} = 4 \times 22.4 \times 1 \times 13.5^2 + 2 \times 50 \times 1 \times 15.5^2 + 2 \times 1 \times \frac{22^3}{12} = 42129 \text{ cm}^4$$



Assuming complete separation between column and base plate. This assumption is very conservative.

$$f_A = \frac{N}{A_w} + \frac{M y_{\text{max}}}{I_{xw}} = \frac{50}{233.60} + \frac{300 \times 16}{42129} = 0.33 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2$$

$$f_B = \frac{50}{233.60} + \frac{300 \times 11}{42129} = 0.292 \text{ t/cm}^2$$

$$q_B = \frac{H}{A_w} = \frac{5}{2 \times 22} = 0.1136 \text{ t/cm}^2$$

$$f_{\text{princ.}} = \frac{f_B}{2} + \sqrt{\left(\frac{f_B}{2}\right)^2 + (q)^2} = 0.33 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2$$

Safe

We can reduce the weld size as follows:

$$\left. \begin{aligned} A_w &= 233.6 \text{ s cm}^2 \\ I_w &= 42129 \text{ s cm}^4 \end{aligned} \right\} \text{ for weld size "s"}$$

Point (B):

$$f_B = \frac{0.292}{s}, \quad q_B = \frac{0.1136}{s}$$

$$f_{\text{princ.}} = \frac{0.33}{s} < 0.72 \text{ t/cm}^2 \therefore s_{\text{req.}} = 0.417 \text{ cm}$$

Point (A):

$$f_A = \frac{0.33}{s} > 0.70 \text{ t/cm}^2 \therefore s_{\text{req.}} = 0.417 \text{ cm}$$

$\therefore$ take the size of fillet welds = 5 mms

(4) Bolts :-

As there no tension on the base plate hence the bolts will transmit only the shearing force $H = 5.0$ tons to the concrete footing.

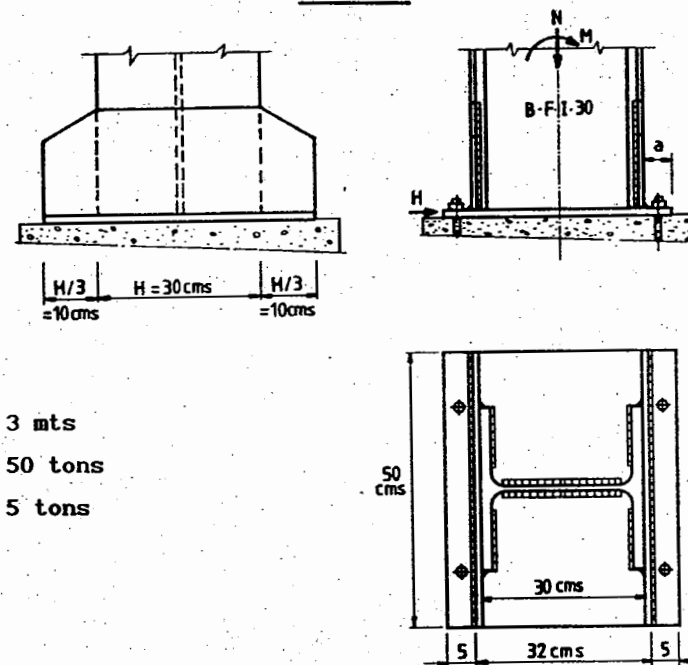
$$A_b = \frac{5}{2 \times 0.25 \times 4} = 2.5 \text{ cm}^2$$

Use 2 bolts of diameter 20 mms.

Example (9.7):

Design a fixed base connection of the side plate type "Type 3" to carry a vertical force $N = 50$ tons, a bending moment $M = 3$ mt and a horizontal force $H = 5$ tons.

Solution



$M = 3 \text{ mts}$

$N = 50 \text{ tons}$

$H = 5 \text{ tons}$

To determine the cantilever distance (a), we have to estimate the diameter of the bolts used first.

The bolts carry only the horizontal reaction (H).

* If we use only two bolts:

$$\therefore A_{(\text{one bolt})} = \frac{H}{2 \times 0.980} = \frac{5.0}{2 \times 0.98} = 2.55 \text{ cm}^2$$

$$\therefore 2.55 = \frac{\pi d^2}{4} \rightarrow d = 1.80 \text{ cm}$$

✓ Weld Size

$$\therefore a > 1 + 3d \quad a > 6.40 \text{ cms}$$

* If we Use Four Bolts:

$$\therefore A_{(\text{one bolt})} = \frac{H}{4 \times 0.98} = \frac{5.0}{4 \times 0.98} = 1.275 \text{ cm}^2$$

$$\therefore d = 1.27 \text{ cm} \quad \therefore \text{use } d = 14 \text{ mms}$$

$$a > (1 + 3 \times 1.40) < 5.20 \text{ cms}$$

$$\therefore \text{Use four bolts } d \text{ 14 mms ; } a = 5.0 \text{ cms.}$$

(1) Stresses on Concrete:

$$f_1 = \frac{N}{BL} + \frac{M \times (B/2)}{D \times B^3/12}$$

$$\therefore f_1 = 1000 \times \left[\frac{50}{50 \times 42} + \frac{300 \times 21}{50 \times (42)^3/12} \right]$$

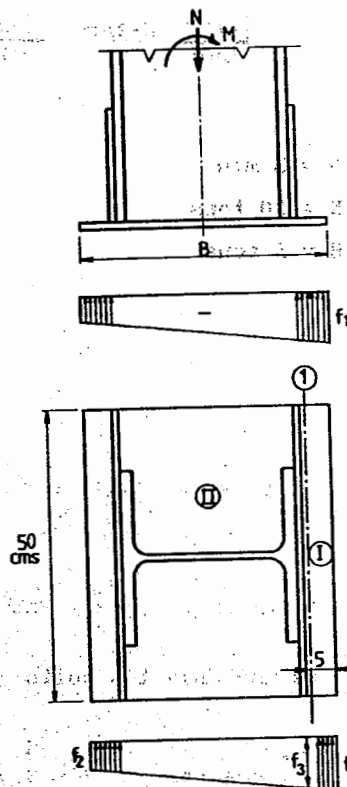
$$\approx -44 \text{ kg/cm}^2 < 60 \text{ kg/cm}^2$$

$$f_2 \approx -3.8 \text{ kg/cm}^2$$

(2) Base Plate:

The base plate is divided into two regions as shown.

The edges of the both regions are considered to be fixed.



Region (I): (Cantilever)

$$f_1 = 0.044 \text{ t/cm}^2$$

$$f_3 = \frac{50}{50 \times 42} + \frac{300 \times 16}{50 \times (42)^3/12} \approx 0.04 \text{ t/cm}^2$$

$$\therefore M_{1-1} = (0.04 \times 5) \times 2.50 + \frac{(0.044 - 0.04)}{2} \times 5 \times \frac{5 \times 2}{3}$$

$$M_1 = 0.53 \text{ t.cm}$$

$$\therefore t = \sqrt{\frac{6M}{b f_b}} = \sqrt{\frac{6 \times 0.53}{0.72 \times 2.4}} = 1.35 \text{ cms}$$

Region (II): (supported on three sides)

Assume that this region is uniformly loaded by the value of the maximum stress on it which equals;

$$f_3 = \frac{50}{50 \times 42} + \frac{300 \times 13}{50 \times (42)^3/12} = 0.0364 \text{ t/cm}^2$$

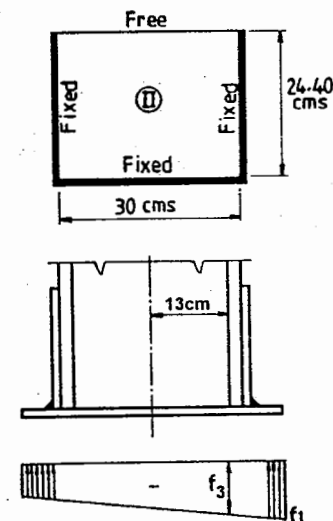
$$b = 30 \text{ cms} , \quad a = 24.40 \text{ cms}$$

$$a/b = 24.4/30 = 0.81 ;$$

$$\alpha_3 = 0.098$$

$$\therefore M_3 = 0.0364 \times (30)^2 \times 0.098 = 3.21 \text{ t.cm}$$

$$\therefore t = \sqrt{\frac{6M}{b f_b}} = \sqrt{\frac{6 \times 3.21}{1 \times 0.72 \times 2.4}} = 3.33 \text{ cms}$$



From (1) & (2) use base plate thickness of 3.4 cms.

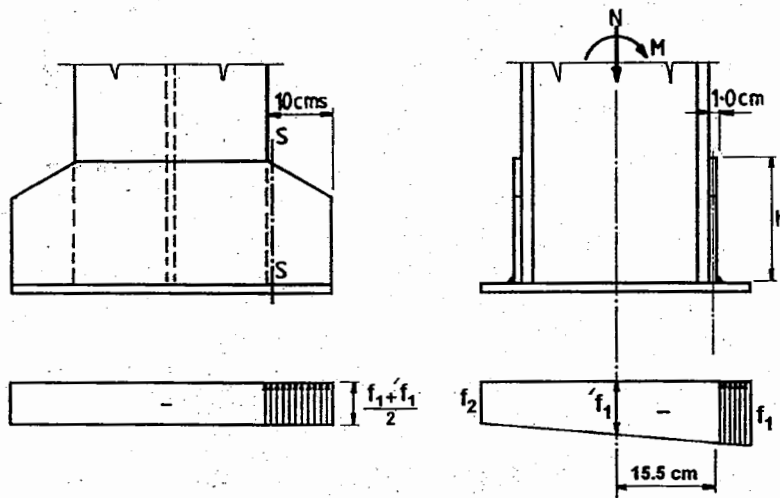
N.B. This thickness can be decreased by adding two additional stiffeners to decrease the dimension "b" (see Example 9.9).

(3) Height of Vertical Stiffeners:

$$f_1' = \frac{f_1 + f_2}{2} = 0.0239 \text{ t/cm}^2$$

$$f_1 = \frac{f_1 + f_1'}{2} = \frac{44 + 23.9}{2} \approx 34 \text{ kg/cm}^2$$

$$F = 0.034 \times 10 \times 21 = 7.14 \text{ tons}$$



Sec. (S -S):

$$M = 7.14 \times 5 = 35.70 \text{ t.cm}$$

$$Q = 7.14 \text{ tons}$$

$$\text{Assume } h_{st} = 20 \text{ cms ; } t_{st} = 1.0 \text{ cms}$$

$$F_b = \frac{6 \times 35.7}{t h^2} = \frac{6 \times 35.7}{1 (20)^2} = 0.72 \times 2.4 \text{ t/cm}^2$$

$$h = 11.13 \text{ cms take } h = 12 \text{ cms}$$

$$q = \frac{3 \times 7.14}{2 \times 1 \times 12} = 0.89 \text{ t/cm}^2 > 0.84 \text{ t/cm}^2 \text{ unsafe}$$

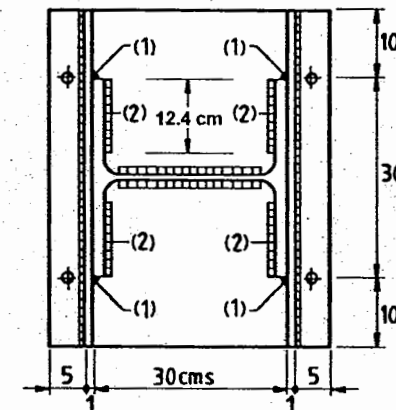
$$\text{Hence use } h_{st} = 14 \text{ cms ; } t_{st} = 1.0 \text{ cms}$$

(4) Fillet Welds:

The following assumptions are to be Considered:

(a) The straining actions (M, N and H) are transmitted from the beam column to the side plates and to the base plate via the weld lines (1) and (2), as shown below. This means that the existing contact has been neglected.

(b) The moment (M) and the normal force (N) are shared by welds (1) and (2) according to their corresponding areas.



Hence:

$$\text{Area of vertical faces of weld (2)} = 4 \times 12.4 \times 1.0 = 49.6 \text{ cm}^2$$

Where $s = 1.0 \text{ cm}$

$$\text{Area of vertical faces of weld (1)} = 4 \times 14 \times 1 = 56 \text{ cm}^2$$

$$\frac{\text{Force resisted by welds (1)}}{\text{Force resisted by welds (2)}} = \frac{56}{49.6} = 1.129 \approx 1.13$$

$$\text{Total compressive force} = \frac{-50}{2} - \frac{3 \times 100}{28} = 35.7 \text{ tons}$$

$$\text{Hence } F(\text{weld (1)}) = \frac{35.7}{1 + 1.13} \times 1.13 = 18.94 \text{ tons}$$

$$F(\text{weld (2)}) = 35.7 - 18.94 = 16.76 \text{ tons}$$

Check of Stresses:

$$\text{Weld (1) } q = \frac{18.94}{2 \times 14 \times 1} = 0.676 \text{ t/cm}^2 \text{ (o.k.)}$$

$$\text{Weld (2) } q = \frac{16.76}{2 \times 12.4 \times 1} = 0.676 \text{ t/cm}^2 \text{ (o.k.)}$$

Check of Welds Connecting the Side Plate and the Beam-Column Flanges to the Base Plate:

Assumptions:

(a) The straining actions (M and N) are transmitted entirely to the base plate by the welds (2) and (3) shown in the figure below.

(b) The horizontal shear is transmitted by welds (4).

$$s_2 = 1.0 \text{ cm}, s_3 = 0.8 \text{ cm}$$

Weld lines (2) and (3) :-

$$\begin{aligned} A_{w(2,3)} &= 4 \times 1 \times 12.40 + 2 \times 0.8 \times 50 \\ &= 129.6 \text{ cm}^2 \end{aligned}$$

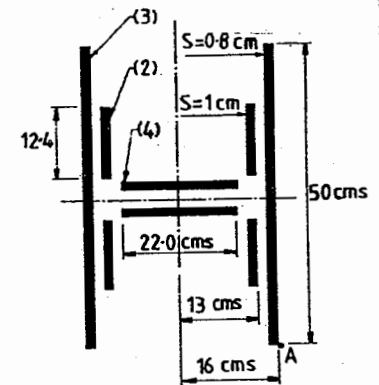
$$\begin{aligned} I_{x_{w(2,3)}} &= 4 \times 1 \times 12.40 \times 12.5^2 \\ &\quad + 2 \times 0.8 \times 50 \times 16.4^2 \\ &= 29266 \text{ cm}^4 \end{aligned}$$

$$\begin{aligned} f_{\perp} &= \frac{N}{A_w} + \frac{M y_{\max}}{I_{x_w}} \\ &= \frac{50}{129.6} + \frac{300 \times 16.80}{29266} = 0.56 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ (o.k.)} \end{aligned}$$

Weld lines (4) :-

$$q = \frac{5}{2 \times 22 \times s} = 0.72 \text{ t/cm}^2$$

$$\therefore s = 0.158 \text{ cm}, \text{ take } s = 0.4 \text{ cm (minimum)}$$



Example (9.8):

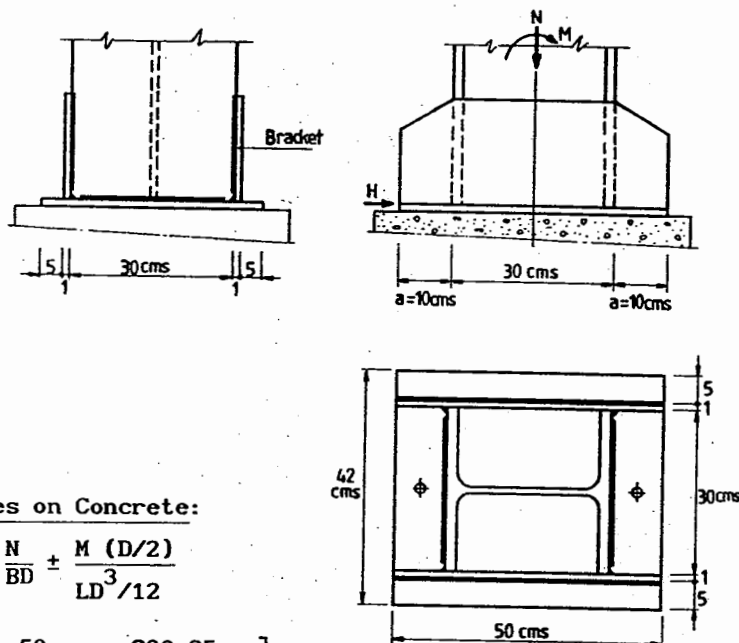
Design a fixed base connection of the conventional type (Type "2") to carry a vertical force ($N = 50 \text{ t}$), a bending moment ($M = 3 \text{ mts}$) and a horizontal force ($H = 5 \text{ t}$).

Solution

$$M = 3.0 \text{ mts} \quad N = 50 \text{ tons} \quad H = 5.0 \text{ tons}$$

Preliminary Design:

$$a = \frac{H}{3} = \frac{30}{3} = 10 \text{ cms} \quad \text{Cantilevers} = 5.0 \text{ cms}$$



(1) Stresses on Concrete:

$$f_{1,2} = - \frac{N}{BD} \pm \frac{M (D/2)}{LD^3/12}$$

$$f_{1,2} = \left[\frac{-50}{50 \times 42} \pm \frac{300 \times 25}{42 \times (50)^3 / 12} \right] \times 1000$$

$$f_1 = -41 \text{ kg/cm}^2 < 60 \text{ kg/cm}^2 \text{ Safe}$$

$$f_2 = -6.7 \text{ kg/cm}^2 \approx 7 \text{ kg/cm}^2$$

(2) Base Plate:

For the estimation of the base plate thickness, the base plate is divided into three regions where the support conditions are varying regarding these regions.

For the design of the different regions it has been assumed that the edges are completely fixed.

Region (1): (cantilever)

In this region, the base plate acts as a cantilever with a span equals to 5.0 cms.

The stress distribution under this region is trapezoidal with the shown shape.

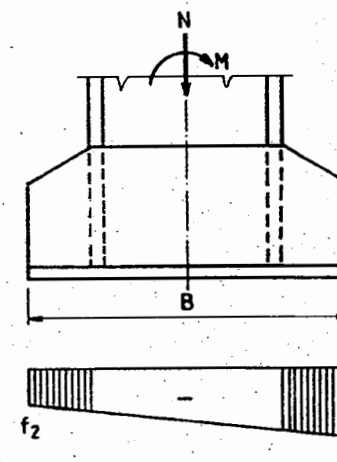
$$f_1 = \frac{50}{50 \times 42} \pm \frac{300 \times 25}{42 \times (50)^3 / 12}$$

$$\therefore f_1 = 0.041 \text{ t/cm}^2$$

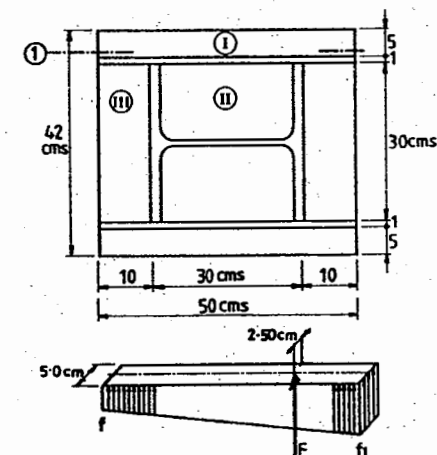
$$f_2 = 0.007 \text{ t/cm}^2$$

$$f_{av} = \frac{f_1 + f_2}{2} = 24 \text{ Kgs./cm}^2$$

$$M_{1-1} = \frac{24}{1000} \times 1 \times \frac{5^2}{2} = 0.3 \text{ t.cm.}$$



Stresses on Concrete



Region (2): (Supported on Four Sides)

The maximum stress on this region is f_3 .

$$f_3 = \frac{50}{50 \times 42} + \frac{300 \times 13}{42 \times (50)^3 / 12}$$

$$= 0.033 \text{ t/cm}^2$$

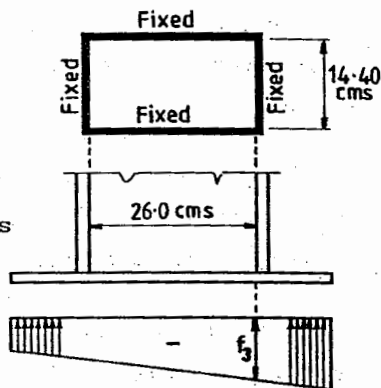
$$a' = 26 \text{ cms}, \quad b' = 14.40 \text{ cms}$$

$$\frac{a'}{b} = 1.80; \quad \alpha_1 = 0.094;$$

$$\alpha_2 = 0.048$$

$$M_{a'} = 0.033 \times (14.4)^2 \times 0.094 = 0.64 \text{ t.cm}$$

$$M_{b'} = 0.033 \times (26)^2 \times 0.048 = 1.07 \text{ t.cm}$$

Region (III): (Supported on Three Sides)

The maximum compressive stress under this region is equal to,

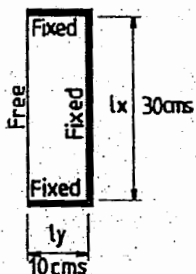
$$f_{\max} = 41 \text{ kg/cm}^2 = 0.041 \text{ t/cm}^2$$

$$b = 30 \text{ cms}, \quad a = 10 \text{ cms}$$

$$\therefore \frac{a}{b} = \frac{10}{30} = 0.33 < 0.5$$

Hence treat this region as a cantilever beam:

$$\therefore M_{\max} = \frac{0.041 \times (10)^2}{2} = 2.05 \text{ t.cm}$$



$$\therefore t = \sqrt{\frac{6M}{bF_b}} = \sqrt{\frac{6 \times 2.05}{1 \times 0.72 \times 2.4}}$$

$$= 2.66 \text{ cms} \quad (3)$$

From (1), (2) and (3)

The base plate thickness will be 2.7 cms.

(3) Bracket Height:

$$f_1 = 0.041 \text{ t/cm}^2$$

$$f_2 = \frac{50}{50 \times 42} + \frac{300 \times 15}{42 \times (50)^3 / 12}$$

$$= 0.034 \text{ t/cm}^2$$

$$\therefore F_1 = 0.034 \times 10 \times 21$$

$$= 7.14 \text{ tons (Rectangle)}$$

$$\therefore F_2 = \frac{1}{2} (0.041 - 0.034) \times 10 \times 21$$

$$= 0.735 \text{ ton (Triangle)}$$

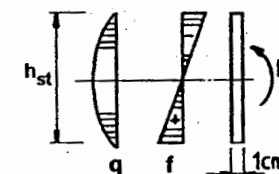
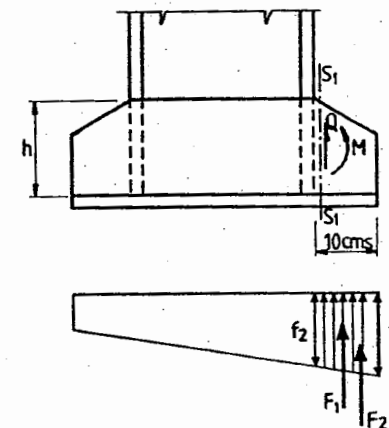
Sec. ($S_1 - S_1$):

$$\therefore M_{1-1} = 7.14 \times 5 + 0.735 \times \frac{20}{3} = 40.53 \text{ t.cm}$$

Bending: (take $t = 1.0 \text{ cm}$)

$$f = \frac{6M}{t h^2} = 0.72 \times 2.4 \text{ t/cm}^2$$

$$\therefore h = 11.86 \text{ cms take } h = 12 \text{ cms}$$



Sec. $S_1 - S_1$



Shear:

$$q = \frac{3Q}{2A} \rightarrow = \frac{3}{2} \cdot \frac{(7.14 + 0.735)}{1 \times h} = 0.984 \text{ t/cm}^2 \text{ unsafe}$$

Take $h = 14 \text{ cms}$

∴ Take the height of bracket to be 14cms and thickness 1.2cms.

(4) Fillet Welds:

Welds (1) and (2) vertical face :-

The straining actions in the column are partially transmitted to the brackets by fillet welds (1), where as the remaining part is directly transmitted to the base plate by fillet welds (2).

Assume equal weld size for all fillet welds:

$$N = N_{(1)} + N_{(2)}$$

$$M = M_{(1)} + M_{(2)}$$

Where: $M_{(1)} = F_{(1)} \times h_{BFI}$

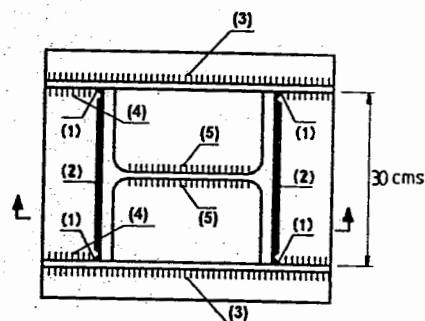
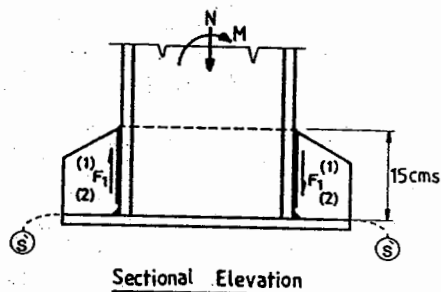
$$M_{(2)} = F_{(2)} \times h_{BFI}$$

If the weld size is "s"

$$\begin{aligned} \therefore N_{(1)} + F_{(1)} &= 14 \times 2 \times s \times f_{\text{all.weld}} \\ &= 14 \times 2 \times 0.72 \text{ s} \end{aligned}$$

$$\therefore N_{(2)} + F_{(2)} = 28 \times s \times f_{\text{all.weld}}$$

The areas of welds (1) and (2) are equal.



$$\text{i.e. } F_{(1)} = F_{(2)} = \frac{M}{2 h_{BFI}} = \frac{300}{2 \times 30} = 5 \text{ tons (for each flange)}$$

$$\& \quad N_{(1)} = N_{(2)} = \frac{N}{2} = 25 \text{ tons}$$

$$\therefore 14 \times 2 \times 0.72 \text{ s} = \left(\frac{25}{2} + 5\right) \quad \therefore \text{s} = 0.86 \text{ cm}$$

∴ Use weld size 9 mms

* Fillet Welds (2), (3) and (4) Connected to the Base Plate horizontal face (s - s) :-

Assume weld size = 1.0 cm

$$\begin{aligned} \therefore A_w &= 1 \times (50 \times 2) + 1 \times (9 \times 4) \\ &+ 1 \times (28 \times 2) = 192 \text{ cm}^2 \end{aligned}$$

$$\begin{aligned} \therefore I_{x_w} &= 2 \left(1 \times \frac{50^3}{12}\right) + 2(28 \times 1 \times 15.5^2) \\ &+ 4 \times \left(1 \times \frac{9^3}{12} + 1 \times 9 \times 20.5^2\right) \\ &= 49660 \text{ cm}^4 \end{aligned}$$

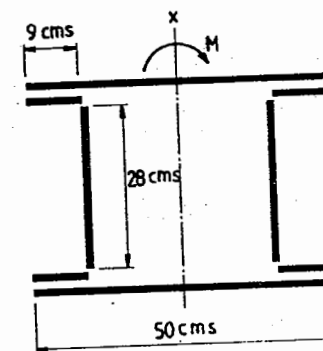
$$f_{\perp(A)} = \frac{N}{A_w} + \frac{M y_{\max}}{I_{x_w}}$$

$$f_{\perp(A)} = \frac{50}{192} + \frac{300 \times 25}{49660} = 0.41 \text{ t/cm}^2$$

$$q_{(A)} = \frac{H}{A} = \frac{5.0}{50 \times 2 + 9 \times 4} = 0.04 \text{ t/cm}^2$$

$$q_{\text{princ.}(A)} = \frac{0.41}{2} + \sqrt{\left(\frac{0.41}{2}\right)^2 + (0.04)^2} = 0.413 \text{ t/cm}^2$$

< 0.72 t/cm² Safe

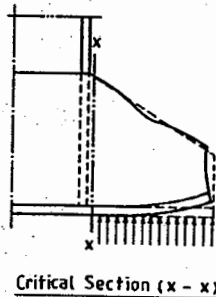
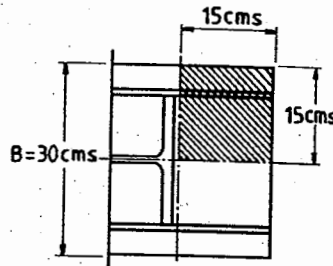
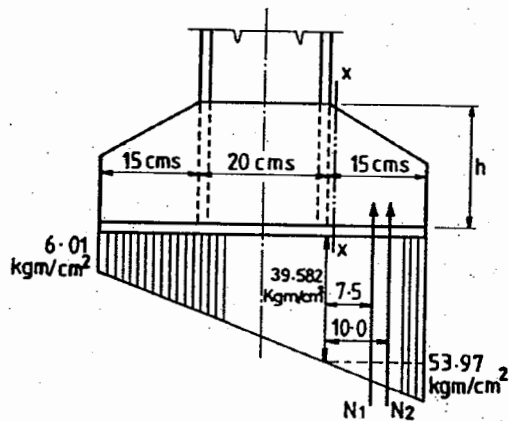


$$\therefore M_{III} = \frac{53.97 \times (7.5)^2}{1000 \times 2} = 1.52 \text{ t.cm}$$

$$\therefore t = \sqrt{\frac{1.52 \times 6}{1.40}} = 2.55 \text{ cms} \quad \therefore \text{take } t_z = 26 \text{ mm}$$

(3) Design of Gusset Plate:

The pressure under the base plate and the load on one gusset are as shown in figures.



$$N_1 = \frac{39.582 \times 15 \times 15}{1000} = 8.91 \text{ tons}$$

$$N_2 = \frac{1}{2} \frac{(53.97 - 39.582) \times 15 \times 15}{1000} = 1.62 \text{ ton}$$

At Section (X-X):

$$Q_x = 8.91 + 1.62 = 10.54 \text{ tons}$$

$$M_x = 8.91 \times 7.50 + 1.62 \times 10 = 83.03 \text{ t.cm}$$

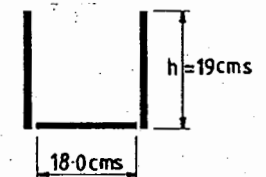
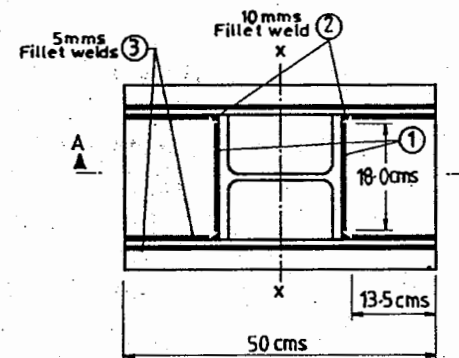
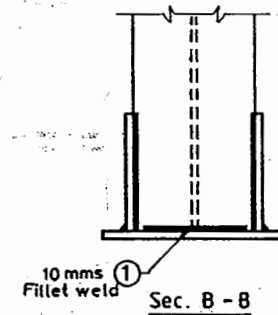
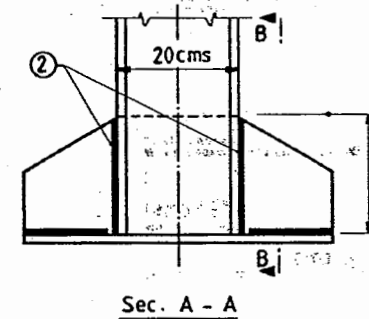
Assuming the thickness of the gusset plate to be 10 mm, we can determine the required height (h) in order to satisfy the allowable compressive stress $f_c = 1.40 \text{ t/cm}^2$.

$$\therefore h = \sqrt{\frac{83.03 \times 6}{1.0 \times 0.72 \times 2.4}} = 16.97 \text{ cms} \quad \text{take } h = 17.0 \text{ cms}$$

(4) Design of Weld Between Gusset and Beam-Column:

Fillet welds (1) and (2) will transmit (F):

$$F = \frac{3 \times 100}{20} + \frac{45}{2} = 37.5 \text{ tons}$$



Assume $s = 1.0 \text{ cm}$

$$q = \frac{37.5}{(18 + 2 \times 17) \times 1.0} = 0.72 \text{ t/cm}^2 \quad (\text{o.k.})$$

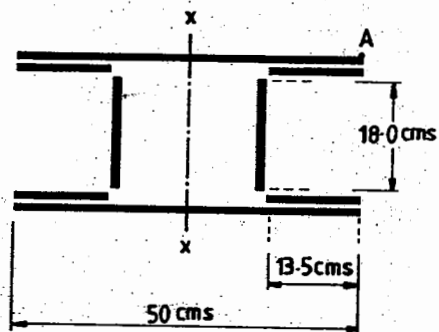


(5) Design of Fillet Welds Connecting Gussets and Beam-Column to the Base Plate:

$$A_w = 1.0 (2 \times 50 + 4 \times 13.5 + 2 \times 18) = 190 \text{ cm}^2$$

$$I_{xx_w} = 1.0 \left\{ \frac{2 \times (50)^3}{12} + 4 \left[\frac{(13.5)^3}{12} + 13.5 \times (18.25)^2 \right] + 2 \times 18 \times (10.5)^2 \right\}$$

$$= 42848.4 \text{ cm}^4$$



$$f_{\perp} = \frac{45}{190} + \frac{3 \times 100 \times 25}{42848.4} = 0.507 \text{ t/cm}^2 < 0.7 \text{ t/cm}^2 \text{ (o.k.)}$$

$$q_{//} = \frac{3}{2 \times 50 + 4 \times 13.5} = 0.02 \text{ t/cm}^2 \text{ (neglected)}$$

This horizontal reaction can be assumed transmitted to the base plate via welds attaching the web to the base plate.

(6) Anchor bolts :-

$$A_b = \frac{3}{2 \times 0.25 \times 4} = 1.5 \text{ cm}^2$$

choose $d = 18 \text{ mm}$.



Example (9.10):

Design the fixed-base connection shown in figure.

Where: $N = 18.0 \text{ tons}$, $M = 22 \text{ mts}$. The cross section of the column is a B.F.I.B. No. (55).

Solution

Check of Type of Stresses on the Base:

$$\frac{M}{N} = \frac{2200}{18} = 122 \text{ cms} > D/6$$

∴ There will be compression over only a part of the base and tension over the other part.

Exact Solution:

Assumption: All tension is carried by bolts only.

$$\sum y = 0.0$$

$$\therefore N = C - T$$

$$= 1/2 f_c k d' B - T \quad (1)$$

$$\sum M = 0 \text{ about (o)}$$

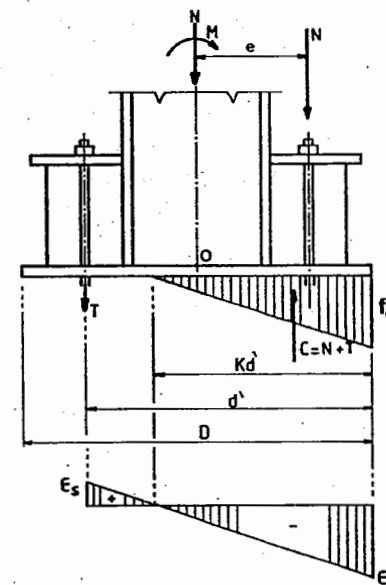
$$N \cdot e = T(d' - D/2) + (N + T)$$

$$(D/2 - 1/3 k d')$$

$$= T(d' - 1/3 k d') + N$$

$$(D/2 - 1/3 k d')$$

$$\therefore T = N \times \frac{(e + 1/3 k d' - D/2)}{(d' - 1/3 k d')} \quad (2)$$





* From Strain Relation-Ship:

$$\frac{\epsilon_s}{d'(1-k)} = \frac{\epsilon_c}{kd'}$$

$$\epsilon_s = \frac{f_s}{E_s}, \quad \epsilon_c = \frac{f_c}{E_c}, \quad \frac{E_s}{E_c} = n$$

$$\therefore \frac{f_s}{(1-k)} = n \frac{f_c}{k}$$

$$\therefore f_c = \frac{f_s k}{n(1-k)}, \quad f_s = \frac{T}{A_s}$$

$$\therefore f_c = \frac{T}{nA_s} \cdot \frac{k}{(1-k)} \quad (3)$$

Substituting from (2) & (3) into (1) we get;

$$N = \frac{1}{2} \frac{k^2 d' B}{A_s n} \cdot \frac{1}{(1-k)} \cdot \frac{(e + 1/3 kd' - D/2)}{d' (1 - 1/3 k)} N - N \frac{(e + 1/3 kd' - D/2)}{d' (1 - 1/3 k)}$$

Multiplying by $(1-k)(1-1/3k)$ we get;

$$(1-k)(1-1/3k) = \frac{1}{2} \frac{B}{nA_s} k^2 (1/3kd' + e - D/2) - \frac{1}{d'} (1-k)(e - D/2 + 1/3kd')$$

$$(1-4/3k+1/3k^2) = \frac{1}{6} \frac{Bd'}{nA_s} k^3 + \frac{1}{2} \frac{B}{nA_s} (e - D/2) k^2 - \frac{1}{d'} \left\{ (e - D/2) + (1/3kd' - e + D/2)k - 1/3k^2 d' \right\}$$

$$\therefore 1-4/3k = \frac{1}{6} \frac{Bd'}{nA_s} k^3 + \frac{1}{2} \frac{B}{nA_s} (e - D/2) k^2 - \frac{(e - D/2)}{d'} - \frac{1}{d'} (1/3kd' - e + D/2)k$$

$$\frac{1}{6} \frac{Bd'}{nA_s} k^3 + \frac{1}{2} \frac{B}{nA_s} (e - D/2) k^2 + \left\{ 1 + \frac{1}{d'} (e - D/2) \right\} k - \frac{(e - D/2)}{d'} + 1 = 0$$



Data:

(from approx. solution)

$$B = 52 \text{ cms} \quad D = 95 \text{ cms}$$

$$e = 122 \text{ cms} \quad d' = 84 \text{ cms}$$

$$A_s = 3 \times 5.6 = 16.80 \text{ cm}^2$$

$$n = 15$$

$$\therefore 2.888 k^3 + 7.6865 k^2 + 1.8869 k - 1.8869 = 0 = A$$

By trial and error:

k	0.20	0.30	0.40	0.38	0.37
A	-1.178	-0.55	0.28	0.09	0

$$\therefore k \approx 0.37$$

From Equation (2)

$$T = 20.74 \text{ tons}$$

From Equation (1)

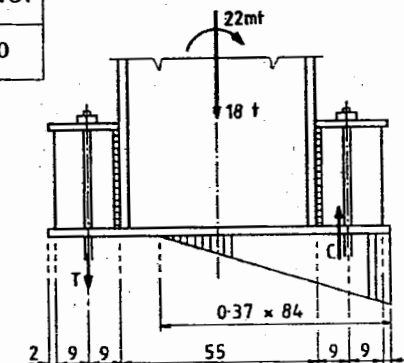
$$C = 38.74 \text{ tons}$$

$$f_c = \frac{2 \times 38.74 \times 1000}{0.37 \times 84 \times 52} = 48 \text{ kg/cm}^2$$

Repeat the previous steps using the new values of T, C and f_c

Note: 1- This method of analysis will require several iterations.

2- There is only a small difference between the exact and the approximate solutions and hence it is simpler to refer for the analysis to Example 9.12.



Fixed Base Plate Connection
(Channel Without Contact)

Example (9.11):

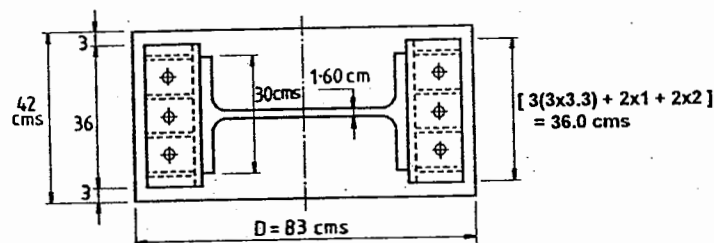
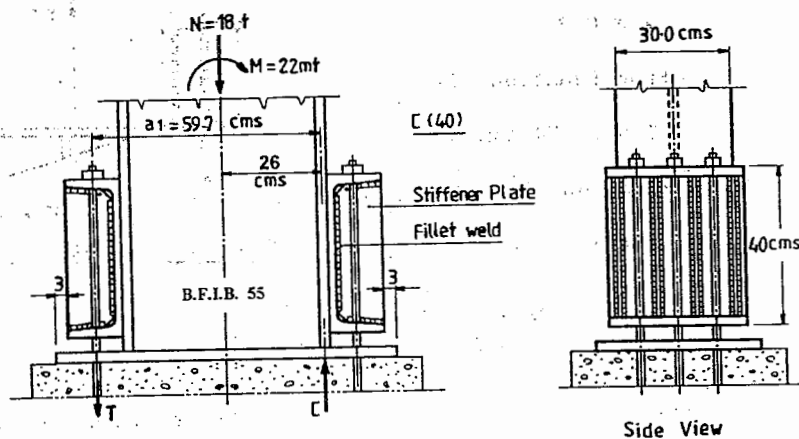
$$e = \frac{M}{N} = \frac{2200}{18} = 122.22 \text{ cms} > \frac{D}{6} \text{ (then tension exists).}$$

Assumptions:

The channels at both sides are not in contact with the base plate. This will lead to the following:

(i) Compression (C) is located under the compression flange of the column.

(ii) Tension (T) is located at the tension bolts.



Bolts:

Assume to use Channel No. 40, $b = 11 \text{ cms}$

$$t_w = 1.40 \text{ cm} \therefore (b - t_w) = 11 - 1.40 = 9.60 \text{ cms}$$

$$a_1 = 55 - \frac{t_{fc}}{2} + t_{wl} + \frac{(b - t_w)}{2}$$

$$\therefore a_1 = 55 - 1.50 + 1.40 + 9.60/2 = 59.70 \text{ cms}$$

$$\therefore T = \frac{2200}{59.70} - \frac{18 \times 26}{59.70} = 29.01 \text{ tons}$$

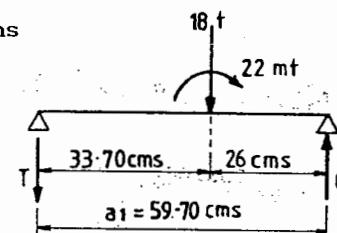
$$A_{\text{bolts}} = \frac{29.01}{0.33 \times 4} = 21.97 \text{ cm}^2$$

$$\text{where : } F_{tb} = 0.33 F_{ub}$$

$$A_{\text{one bolt}} = \frac{21.97}{3} = 7.32 \text{ cm}^2$$

$$\therefore d = \sqrt{\frac{7.32 \times 4}{3.14 \times 0.80}} = 3.41 \text{ cms}$$

↙ reduction for threaded shank



Base Plate:

$$\text{Compression flange reaction } C = \frac{2200}{59.70} + 18 \times \frac{33.70}{59.70} = 47.01 \text{ tons}$$

$$a_2 = \frac{t_f}{2} + b_l + 3$$

$$\therefore a_2 = 1.50 + 11 + 3 = 15.50 \text{ cms}$$

$$\therefore C = f_c \times \frac{3 \times 15.50}{2} \times 36$$

$$\therefore f_c = \frac{47.01 \times 1000 \times 2}{3 \times 15.50 \times 36}$$

$$= 56.16 \text{ kg/cm}^2 \text{ Safe}$$

In order to determine the required safe thickness of the base plate take M_{1-1}

$$M_{1-1} = \frac{1}{1000} \left[39.25 \times \frac{(11 + 3)^2}{2} \times 36 + (56.16 - 39.25) \times \frac{14}{2} \times 36 \times \frac{14 \times 2}{3} \right]$$

$$= 138.474 + 39.772 = 178.25 \text{ ton.cm}$$

$$\therefore t = \sqrt{\frac{178.25 \times 6}{36 \times 0.72 \times 2.4}} = 4.14 \text{ cms}$$

Stiffeners:

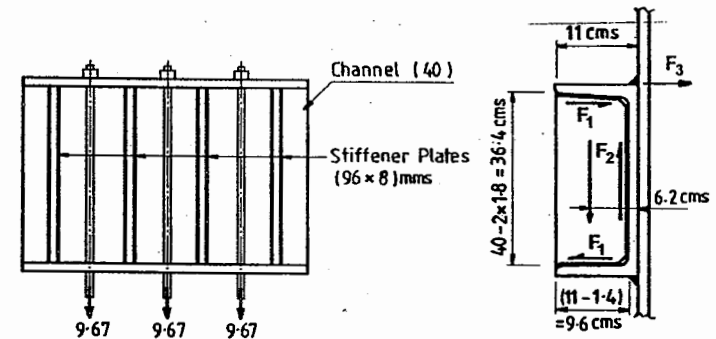
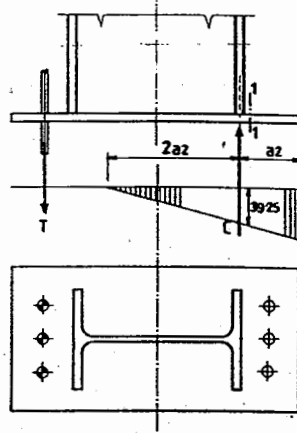
The load on the bolts is supported by the top flange of the channel No. 40, reinforced by four stiffener plates welded between the channel flanges. Each of the two interior plates supports a full bolt load, i.e.

$$F_{\text{intermediate stiffener}} = \frac{29.01}{3} = 9.67 \text{ tons}$$

$$A_{\text{stiffener}} = \frac{9.67}{1.40} = 6.907 \text{ cm}^2$$

Take for stiffeners $b = 9.60 \text{ cms}$, $t = 0.80 \text{ cm}$

$$\therefore b/t = \frac{9.60}{0.80} = 12 < 16 \text{ (safe)}$$



Welding Between Stiffeners and Channel:

As shown in figure, the horizontal weld forces (F_1) and the vertical weld force (F_2) combine to resist the 9.67 tons acting with an eccentricity of 6.20 cms.

From channel web we get:

$$q_{\downarrow} = \frac{9.67}{2(36.40 + 2 \times 9.6)} = 0.087 \text{ t/cm'}$$

From Table (3.2) we get:

$$\therefore \bar{X} \text{ from channel web} = \frac{(9.6)^2}{2 \times 9.60 + 36.40} = 1.657 \text{ cm}$$

$$\therefore I_p = 2 \left[\frac{8 \times (9.6)^3 + 6 \times 9.6 \times (36.4)^2 + (36.4)^3}{12} - \frac{(9.6)^4}{(2 \times 9.6 + 36.4)} \right]$$

$$= 21632 \text{ cm}^4$$

$$q_{\rightarrow}^{[M_t]} = \frac{M \cdot y}{I_p} = \frac{9.67(6.2 - 1.40) \times 18.20}{21632} = 0.039 \text{ t/cm'}$$

$$q_{\downarrow}^{[M_t]} = \frac{M \cdot x}{I_p} = \frac{9.67(6.2 - 1.40) \times (9.6 - 1.657)}{21632} = 0.017 \text{ t/cm'}$$

$$q_t = \sqrt{(0.087 + 0.017)^2 + (0.039)^2} = 0.11 \text{ t/cm}^2$$

Using fillets on both sides of stiffeners the required size (s) will be;

$$s = \frac{0.11}{0.72 \times 1.0} = 0.152 \text{ cms}$$

∴ Take s = 4 mms (minimum size of weld).

As an alternative solution we can assume F_1 carrying the moment and the force F_2 carrying the shear and so we get:

$$F_2 = 9.67 \text{ tons} \quad F_1 = \frac{9.67 \times (6.20 - 1.40)}{36.40} = 1.27 \text{ tons}$$

$$\therefore q_{\left[\begin{smallmatrix} \downarrow \\ Q \end{smallmatrix} \right]} = \frac{9.67}{2 \times 0.40 \times 36.40} = 0.332 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \quad (\text{Safe})$$

$$\therefore q_{\left[\begin{smallmatrix} \rightarrow \\ M_t \end{smallmatrix} \right]} = \frac{1.27}{2 \times 0.40 \times 9.60} = 0.165 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \quad (\text{Safe})$$

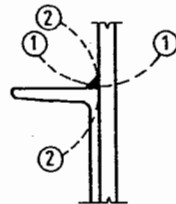
Welding Between Channel and Column Flange:

$$F_3 = \frac{9.67 \times 3 \times 6.20}{40} = 4.496 \text{ tons}$$

$$L_{\text{weld}} = b_{\text{(flange of column)}}$$

Section (1-1):

$$q_{\left[\begin{smallmatrix} \rightarrow \\ M_t \end{smallmatrix} \right]} = \frac{4.496}{30 \times 0.40} = 0.375 < 0.72 \text{ t/cm}^2 \quad (\text{Safe})$$



Section (2-2):

$$f_t = \frac{4.496}{30 \times 0.40} = 0.375 < 0.72 \text{ t/cm}^2 \quad (\text{Safe})$$

The shear (9.67 x 3) will be resisted by vertical welds between web of channel and flange of column.

$$q_{\left[\begin{smallmatrix} \downarrow \\ Q \end{smallmatrix} \right]} = \frac{3 \times 9.67}{2 \times 40 \times 0.40} = 0.91 \text{ t/cm}^2 \quad (\text{Unsafe})$$

$$\therefore \text{Take size of weld } s = \frac{0.91}{0.72} \times 0.40 = 0.51 \text{ cm}$$

$$s \cong 6 \text{ mms}$$

It can be noted that we can consider only the horizontal lines of weld (i.e. no vertical weld lines between channel and column flange). In such a case:

Section (1-1):

$$q_{\left[\begin{smallmatrix} \rightarrow \\ M_t \end{smallmatrix} \right]} = \frac{4.496}{30 \times 0.40} = 0.375 \text{ t/cm}^2$$

$$f_{\perp} = \frac{4.496}{30 \times 0.40} = 0.375 \text{ t/cm}^2$$

$$\therefore f_{\text{eq.}} = \sqrt{(0.375)^2 + 3(0.375)^2} = 0.75 \text{ t/cm}^2 < 1.1 \times 0.72 \text{ t/cm}^2$$

Section (2-2):

$$q_{\left[\begin{smallmatrix} \downarrow \\ Q \end{smallmatrix} \right]} = \frac{4.496}{30 \times 0.40} = 0.375 \text{ t/cm}^2$$

$$f_{\perp} = \frac{4.496}{30 \times 0.40} = 0.375 \text{ t/cm}^2$$

$$\therefore f_{\text{eq.}} = \sqrt{(0.375)^2 + 3(0.375)^2} = 0.75 \text{ t/cm}^2 < 0.72 \times 1.1 \text{ t/cm}^2$$

Safe

Example (9.12):

Design the fixed base connection shown in figure below.

Approximate Solution:

Assumptions:

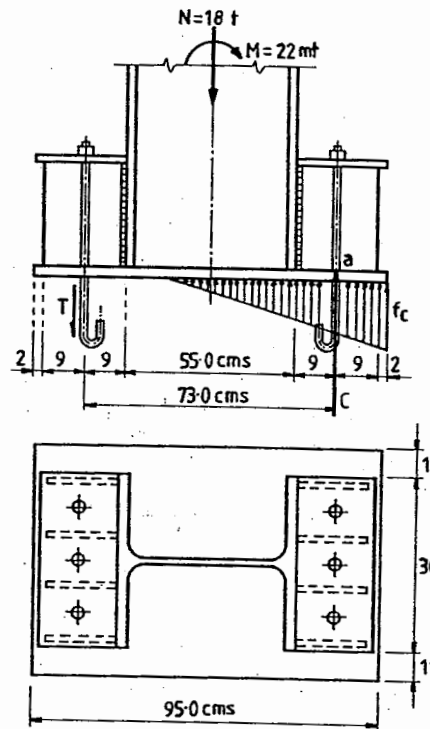
1- The whole tension force is supported by bolts on the tension side.

2- The line of action of the compression force lies on the centerline of the vertical stiffeners.

This means that the vertical stiffeners are assumed to support the entire resultant compression force "C". Hence the contact between the beam-column flange and the base plate is neglected.

These two assumptions will render the structural system statically determinate. This means that we have two unknowns, namely either (T and C) or (f_t and f_c) and two equations of equilibrium ($\sum M = 0$ and $\sum y = 0$).

The second assumption is somewhat imaginary. In fact, the flange of the beam-column will transmit the greatest amount of the resultant compressive force. However, this assumption will considerably simplify the analysis. For more accurate



assumptions, refer to Examples (9.13) and (9.14) where the utilized assumptions are based on the finite elements method.

Pre-dimensioning:

Having:

h = height of column section.

b = width of column flange.

It is of good practice to start the design by assuming the following dimensions:

$$D \cong b + \frac{2}{3} b, \quad b_{st} \cong \frac{1}{3} h$$

$$B = h + 2 \times \frac{1}{3} h + 2 \times (2 \rightarrow 5 \text{ cms})$$

Hence:

$$D \cong 30 + \frac{2}{3} \times 30 = 50 \text{ cms}$$

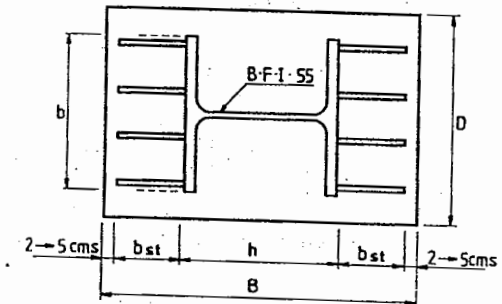
$$\text{Take } D = 52 \text{ cms}$$

$$b_{st} \cong \frac{1}{3} \times 55 = 18 \text{ cms}$$

$$B = 55 + 2 \times 18 + 2 \times 2 = 95 \text{ cms}$$

$$e = \frac{M}{N} = \frac{22 \times 100}{18} = 122 \text{ cms. ; } D/6 = 95/6 = 15.8 \text{ cms.}$$

(There are tension)



1- Bolts:

$$T = - \frac{18}{2} + \frac{22 \times 100}{73} = 21.14 \text{ tons}$$

$$A_{bolts} = \frac{21.14}{0.33 \times 4} = 16 \text{ cm}^2, \text{ use bolts } d = 30 \text{ mms}$$

$$F_{tb} = 0.33 F_{ub}$$

$$A_{bolt(stress area)} \cong 0.8 \times \frac{\pi (3)^2}{4} = 5.6 \text{ cm}^2; \text{ Use 3 bolts}$$



2- Vertical Stiffeners:

$$\sum y = 0 \rightarrow C = T + N = 21.14 + 18.0 = 39.14 \text{ tons}$$

"C" is transmitted from compression flange through welds to the stiffeners at the compression side and then to the base plate by bearing. Tension "T" is transmitted through welds and stiffeners at tension side to the bolts via the top plate and then to the R.C. footing. Assuming that intermediate stiffeners support double the load of the edge stiffeners,

$$T = 21.14 \text{ tons}$$

$$C = -\frac{18}{2} - \frac{2200}{73} = 39.14 \text{ tons} \leftarrow \text{governing}$$

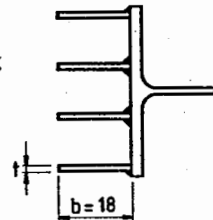
$$\therefore F_{\text{stiff}} = \frac{39.14}{3} = 13.04 \text{ tons}$$

To prevent buckling of stiffeners

$$\frac{b}{t} < 16 \quad \therefore \frac{18}{t} < 16 \quad \therefore t = 1.20 \text{ cm}$$

$$f_t = \frac{13.04}{1.20 \times 18} = 0.60 \text{ t/cm}^2$$

< 1.40 Safe



3- Vertical Welds:

Assume $s = 1.0 \text{ cm}$

Height of stiffeners $\cong \frac{2}{3} b \rightarrow b$

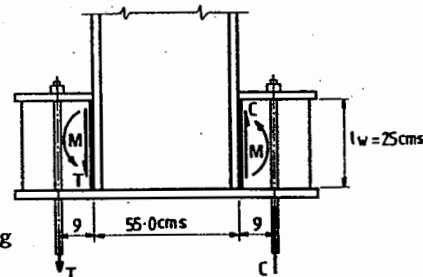
Take $L_w = 25 \text{ cms}$

$$T = 21.14$$

$$C = 39.14 \text{ tons} \leftarrow \text{governing}$$

$$Q = C = 39.14 \text{ tons}$$

$$M = 39.14 \times 9 = 352.26 \text{ cm.t}$$



* Sec. (1-1):

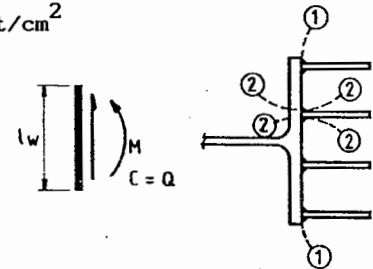
$$q \left[\frac{Q}{\parallel} \right] = \frac{39.14}{8 \times 1.0 \times 25} = 0.196 \text{ t/cm}^2$$

$$f_{\perp} = \frac{M}{Z} = \frac{6 \times 352.26}{8 \times 1.0 \times 25^2} = 0.422 \text{ t/cm}^2$$

$$\therefore f_{\text{eq.}} = \sqrt{(f_t)^2 + 3(q)^2}$$

$$= \sqrt{(0.422)^2 + 3(0.196)^2}$$

$$= 0.54 \text{ t/cm}^2 < 1.1 \times 0.72 \text{ (o.k.)}$$



* Sec. (2-2):

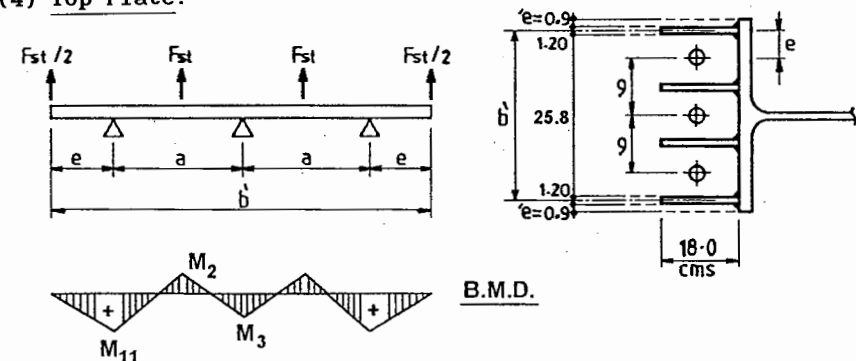
$$q \left[\frac{Q}{\parallel} \right] = 0.196 \text{ t/cm}^2$$

$$q_{(M_t)_{\perp}} = \frac{M}{Z} = 0.422 \text{ t/cm}^2$$

$$\therefore q_{\text{total}} = \sqrt{(0.196)^2 + (0.422)^2} = 0.465 \text{ t/cm}^2 \text{ (o.k.)}$$

Take $s = 1 \times 0.54 / (0.72 \times 1.1) = 0.68 \text{ cms.}$

(4) Top Plate:





$$a = 3d \cong 3 \times 3.0 = 9.0 \text{ cms}$$

$$e = a/2 = 4.5 \text{ cms}$$

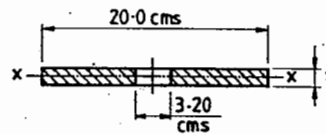
$$e' = \frac{1}{2} (30 - 9 \times 2 - 4.5 \times 2 - 1.2) = 0.9 \text{ cm}$$

 t_{st}

$$M_{11} = \frac{f_{stiff}}{2} \times e = \frac{13.04}{2} \times 4.5 = 29.34 \text{ ton.cm}$$

$$\text{Take } b = 18 + 2 = 20 \text{ cms}$$

$$Z_{xx} = \frac{(20 - 3.2) \times t_{\#}^2}{6}$$



$$\therefore f_{ta} = \frac{M}{Z} = \frac{M}{\frac{bt^2}{6}}$$

$$\therefore t = \sqrt{\frac{6 \times 29.34}{16.8 \times 1.40}} = 2.73 \text{ cms}$$

$$\left. \begin{array}{l} M_3 = 13.21 \text{ t.cm} \\ M_2 = 2.02 \text{ t.cm} \end{array} \right\} \begin{array}{l} \text{which} \\ \text{are less} \\ \text{than } M_{11} \end{array}$$

$$F_b = 1.4 \text{ t/cm}^2 \text{ as the section is not solid rectangular section}$$

$$\text{Take } t = 2.80 \text{ cms}$$

$$(5) \text{ Base Plate: } C = 39.14 \text{ tons}$$

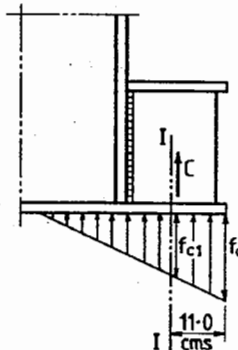
$$C = \frac{1}{2} f_c \times 33 \times b$$

$$\therefore 39.14 \times 1000 = \frac{1}{2} \times f_c \times 3 \times 11 \times 52$$

$$f_c = 45.6 \text{ kg/cm}^2 < 60 \text{ kg/cm}^2 \text{ Safe}$$

$$f_{c1} = \frac{2}{3} f_c = 30.41 \text{ kg/cm}^2$$

Assuming that the critical section is at the bolts' centerline.



$$M_{II} = \frac{1}{1000} \times \left\{ 30.41 \times \frac{(11)^2}{2} \times 52 + \frac{1}{2} (45.6 - 30.41) \times 11 \times 52 \times \frac{2}{3} \times 11 \right\}$$

$$\therefore M_{II} = 127.56 \text{ ton.cm}$$

* If holes of bolts are taken into consideration

$$\therefore b_{net} = 52 - 3 \times 3.20 = 42.40 \text{ cms}$$

$$\therefore t_{\#} = \sqrt{\frac{127.56 \times 6}{42.4 \times 1.4}} = 3.59 \cong 3.60 \text{ cms}$$

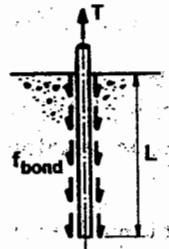
$$F_b = 1.4 \text{ t/cm}^2 \text{ as the section is not solid rectangular section}$$

$$(6) \text{ Length of Bolts: } f_{bond} = 6.0 \text{ kg/cm}^2$$

$$T = n \pi d L_{bolt} \times f_{bond}$$

$$\therefore 21.14 \times 1000 = 3 \times \pi \times 3.0 \times L_{bolt} \times 6.0$$

$$\therefore L = 125 \text{ cms}$$



Generally, the depth of the footing is not that large, so we do not usually use bolts of length more than 50 cms.

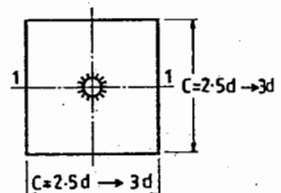
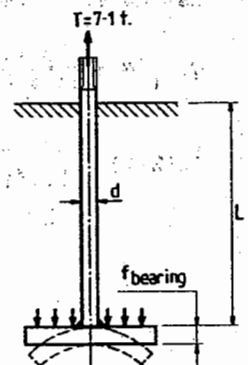
Bearing Washer Plate:

Hence, the suitable choice is to use a bearing washer plate. It is always preferable to consider that this bearing plate is resisting the applied tension force.

$$T = \frac{21.14}{3} = 7.04 \cong 7.1 \text{ tons}$$

Assume the embeded length:

$$L = 30 \text{ cms; } C = 2.5 d = 7.5 \text{ cms}$$



$$T_{\text{bond}} = 3.14 \times 3 \times 30 \times 6 = 1696 \text{ kgs} \approx 1.69 \text{ tons}$$

$$\text{The remaining tension force} = 7.1 - 1.69 = 5.41 \text{ tons}$$

$$f_{\text{bearing}} = \frac{5.41}{7.5 \times 7.5} \approx 0.1 \text{ t/cm}^2$$

$$M_{11} = 0.1 \times 7.5 \times \frac{(3.75)^2}{2} = 5.27 \text{ t.cm}$$

$$t = \sqrt{\frac{6 \times 5.27}{7.5 \times 0.72 \times 2.4}} = 1.56 \text{ cm}$$

Take t (bearing washer plate) = 16 mms.

Check of Stresses regarding the welds attaching the bolt to the washer bearing plate.

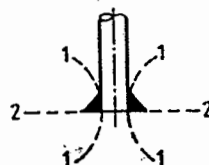
Section (1-1):

Assume $s = 1 \text{ cm}$

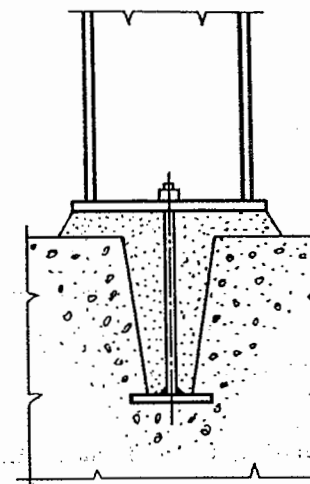
$$q_{1-1} = \frac{5.41}{3.14 \times 3 \times 1} = 0.574 \text{ t/cm}^2 \text{ (o.k.)}$$

$$f_{t(2-2)} = \frac{5.41}{3.14(4^2 - 3^2)/4} = 0.985 \text{ t/cm}^2 > 0.72 \text{ unsafe}$$

$$\text{Take } s = \frac{0.985}{0.72} \times 1.0 = 1.36 \text{ cm } s = 14 \text{ mms}$$



Some designers do not like to transmit part of the tension force by bond. They consider that the portion of the bolt embedded in the pocket filled with compacted fine concrete is not capable of transmitting by bond the corresponding T_{bolt} . Hence, the bearing washer plate is designed to transmit the total induced tension force. Thus:



$$f_{\text{bearing}} = \frac{7.1}{7.5 \times 7.5} = 0.126 \text{ t/cm}^2$$

$$M_{1-1} = 0.126 \times 7.5 \times \frac{3.75^2}{2} = 6.64 \text{ cms}$$

$$t = \sqrt{\frac{6 \times 6.64}{7.5 \times 0.72 \times 2.4}} = 1.75 \text{ cm} ; \quad t = 18 \text{ mms}$$

Example (9.13):

Design a fixed base connection to support a bending moment $M = 22 \text{ m.t}$ and a normal force $N = 18.0 \text{ tons}$.

Make use of the results of the finite elements method to make reasonable assumptions

Procedure

The following example is based on assumptions which were found to be in accordance with the finite elements method previously outlined in Section 9.6.2.4. These assumptions are:

(a) Get the tension force induced in the beam-column flange

$$T_f = \frac{M}{h} - \frac{N}{2}$$

where: h = height of beam-column cross section

(b) Assume the induced tension force in the bolts $T_{\text{bolt}} = 0.675 T_f$. This decrease is a consequent result due to the increase of the arm.

(c) Get a pre-dimensioning of the bolts and the stiffeners on the compression side.

(d) The compression resultant force is assumed to be transmitted to the base plate by both the beam-column flange and the corresponding stiffeners.

(e) Compute the center of gravity of the area of the compression flange and those stiffeners.

(f) This center of gravity is assumed to be coincide with the compression resultant force.

(g) Recalculate again a more exact tension force regarding the bolts; and from equilibrium a new resultant compression force.

(h) Proceed for the analysis of the different steel components as for the previous example.

Solution

Pre-dimensioning:

T_f = flange beam-column tension force

$$= \frac{M}{h} - \frac{N}{2} = \frac{2200}{55} - \frac{18}{2} = 31 \text{ tons}$$

Assume that the induced tension force in the bolts is:

$$T_{\text{bolt}} = 0.675 T_f \quad (\text{finite elements}) \\ = 21 \text{ tons}$$

$$C = 21 + 18 = 39 \text{ tons}$$

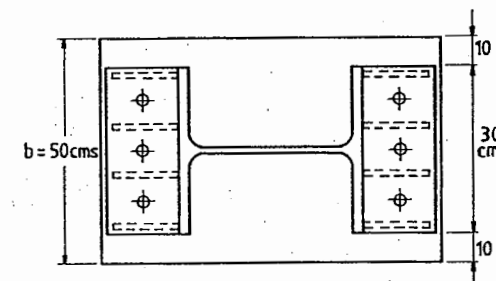
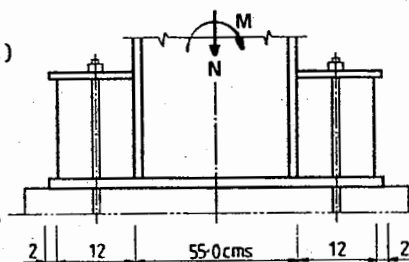
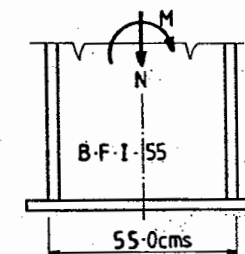
$$\therefore A_b = \frac{21}{0.33 \times 4} = 15.9 \text{ cm}^2$$

$\therefore$ Use bolts $d = 30 \text{ mm}$

$$A_{\text{bolt min}} = 0.80 \times \frac{\pi(3)^2}{4} \\ = 5.60 \text{ cm}^2$$

$$\therefore \text{No. of bolts} = \frac{15.9}{5.60}$$

$$= 3.0 \text{ bolts}$$





∴ No. of stiffeners =

$$\text{No. of bolts} + 1 = 4.0$$

$$F_{st.} = \frac{39}{3} = 13 \text{ tons}$$

(conservative but allowed for pre-dimensioning)

$$A_{st.} = 13/1.40 = 9.30 \text{ cm}^2$$

$$\therefore b_{st.} \times t_{st.} = 9.30 \text{ cm}^2$$

$$b_{st.} = t_{st.} \times 16 \text{ (local buckling cond.)}$$

$$\therefore \text{Take } b_{st.} = 12 \text{ cms and } t_{st.} = 0.80 \text{ cm}$$

Assumptions:

1- All the tension force is supported by the bolts in the tension side.

2- The compression resultant lies at the C.G. of the area comprising the compression flange and the stiffeners taking only half the area of the edge stiffeners.

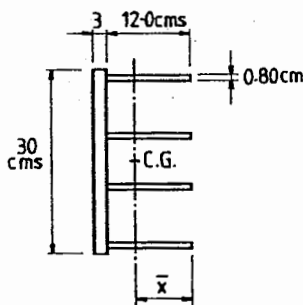
$$\therefore \bar{X} = \frac{28.80 \times 6 + 90 \times 13.50}{(28.80 + 90)} = 11.70 \text{ cms}$$

(1) Anchor Bolts:

$$\therefore T = \frac{1}{61.30} \left[2200 - 18 \times \left(\frac{55}{2} + 0.30 \right) \right]$$

$$= 27.70 \text{ tons}$$

$$f_{ta} = \frac{27.70}{3 \times 5.60} = 1.60 \text{ t/cm}^2 \text{ use Grade 5.6 (} F_{tb} = 0.33 \times 5 = 1.65 \text{ t/cm}^2 \text{)}$$



$$A_f = 3 \times 30 = 90 \text{ cm}^2$$

$$A_{st.} = 3 \times 12 \times 0.80$$

$$= 28.8 \text{ cm}^2$$

∴ Use 3 anchor bolts $d = 30 \text{ mms}$

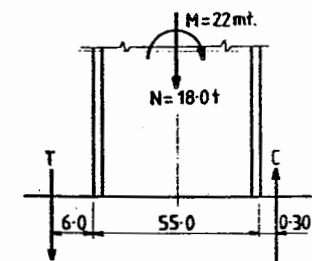
(St. 44)

$$(2) \text{ Stiffeners: } C = 27.7 + 18$$

$$= 45.70 \text{ tons}$$

$$F_{st.} = \frac{27.70}{3} = 9.20 \text{ tons}$$

tension side



Compression force carried by stiffeners at the:

$$\text{Compression side} = \frac{28.80}{(28.8 + 90)} \times 45.70 = 11.0 \text{ tons}$$

$$\therefore F_{st.} = \frac{11}{3} = 3.70 \text{ tons}$$

compression side

∴ stiffeners at the tension side govern the design.

$$\therefore f_{ta} = \frac{9.20}{12 \times 0.80} = 1.0 \text{ t/cm}^2 < 1.40 \text{ t/cm}^2 \text{ (Safe)}$$

(3) Vertical Welds:

$$Q = 9.20 \text{ t}$$

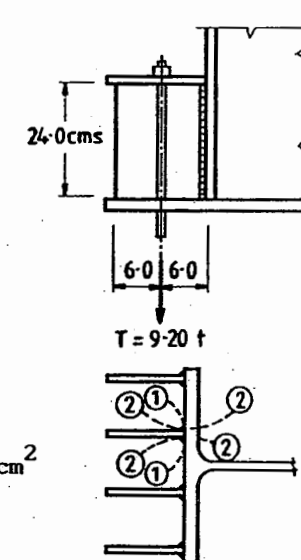
$$M = 9.20 \times 6 = 55.20 \text{ t.cm}$$

Assume the size of weld = 0.60 cm.

Sec. (1-1):

$$q_{\downarrow} = \frac{9.20}{24 \times 2 \times 0.60} = 0.33 \text{ t/cm}^2$$

$$f_{\rightarrow} = \frac{55.20 \times 12}{2 \times 0.60 \times (24)^3 / 12} = 0.48 \text{ t/cm}^2$$





$$\therefore f_{eq} = \sqrt{(0.48)^2 + 3(0.33)^2} = 0.73 \text{ t/cm}^2$$

$$< 0.72 \times 1.1 \text{ t/cm}^2 \quad (\text{Safe})$$

Sec. (2-2):

$$q_{\downarrow} = 0.33 \text{ t/cm}^2$$

$$q_{\rightarrow} = 0.48 \text{ t/cm}^2$$

$$\therefore q_r = \sqrt{0.32^2 + 0.48^2} = 0.58 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \quad \therefore \text{Safe}$$

(4) Top Plate:

$$M_{1-1} = \frac{9.20}{2} \times 4.50 = 20.70 \text{ t.cm}$$

$$b_{st.} = 14 - 3.20 = 10.80 \text{ cms}$$

$$\therefore t_{\neq} = \sqrt{\frac{6 \times 20.70}{10.8 \times 1.40}} = 3.0 \text{ cms}$$

(5) Stresses on Concrete:

$$a_c = 11.70 + 2 = 13.70 \text{ cms}$$

$$1/2 f_c \times 3 a_c \times b = C$$

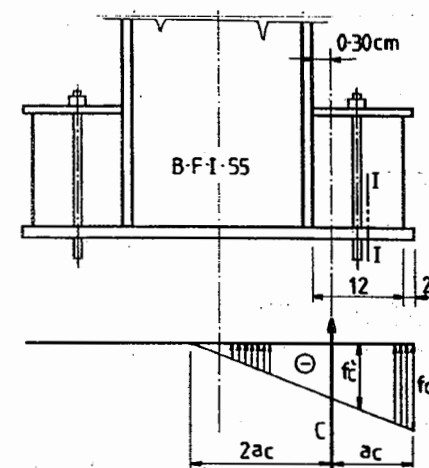
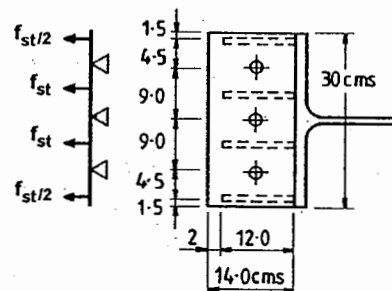
$$\therefore 1/2 \times 60 \times 3 \times 13.70 \times b = 45.70 \times 10^3$$

$$\therefore b_{min.} = 40 \text{ cms}$$

$$b_{act.} = 50 \text{ cms}$$

$$f_{c \text{ act.}} = 44.50 \text{ kg/cm}^2 < 60 \text{ kg/cm}^2$$

Safe



(6) Base Plate:

$$f_{c'} = \frac{3 \times 13.7 - 8.0}{3 \times 13.70} \times 44.50$$

$$\therefore f_{c'} = 35.80 \text{ kg/cm}^2$$

$$M_{1-1} = \frac{1}{1000} \left[35.80 \times \frac{(8)^2}{2} + \frac{1}{2} (44.5 - 35.8) \times 8 \times \frac{2}{3} \times 8 \right]$$

$$= 1.331 \text{ t.cm/cm'}$$

$$\therefore t_{\neq} = \sqrt{\frac{6 \times 1.331}{1 \times 1.40}} = 2.60 \text{ cms}$$

$\therefore$ Take thickness of base plate = 2.60 cms

**Example (9.14):**

Redesign the fixed base connection of Example 9.13 assuming that 70% of the compression resultant is transmitted via the beam-column flange while the remaining 30% is transmitted via the corresponding stiffeners.

Solution**Pre-dimensioning of the Connection:**

Same as Example (9.13).

Assumptions:

1- The whole tension force is supported by the bolts in the tension side.

2- 30% of the compression force resultant is transmitted through the stiffeners while 70% through the flange.

$$a_c = \frac{0.30 C \times 8 + 0.70 C \times 15.50}{C} = 13.25 \text{ cms}$$

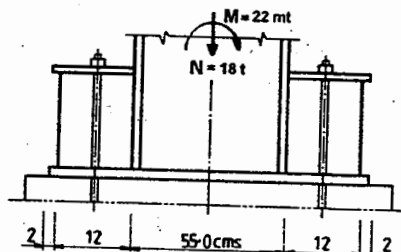
(1) Anchor Bolts:

$$T = \frac{1}{61.75} \left[2200 - 18 \left(\frac{55}{2} + 0.75 \right) \right] = 27.40 \text{ tons}$$

$$C = 27.40 + 18 = 45.40 \text{ tons}$$

Almost the same values as the previous Example (9.13).

∴ Proceed as Example (9.13).

**(2) Stiffeners: C = 45.40 tons**

$$f_{st.} = \frac{27.40}{3}$$

Tension side = 9.10 tons

Compression force supported by stiffeners at the compression side = $0.30 \times 45.40 = 13.62 \text{ tons}$

$$\therefore f_{st.} = \frac{13.62}{3} = 4.54 \text{ tons comp. side}$$

∴ Stiffeners at the tension side are governing.

$$f_{act.} = \frac{9.10}{12 \times 0.80} = 1.0 \text{ t/cm}^2 < 1.40 \text{ t/cm}^2 \text{ Safe}$$

(3) Vertical Welds:

$$Q = 9.10 \text{ tons}$$

$$M = 9.10 \times 6 = 54.60 \text{ cm.t}$$

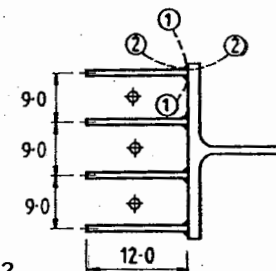
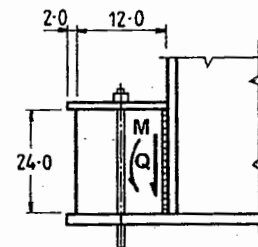
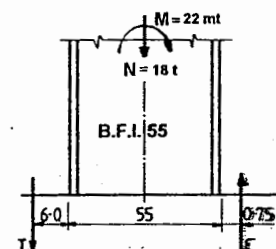
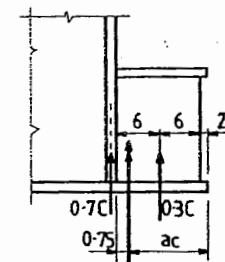
Assume the size of weld = 0.60 cm

Sec. (1-1):

$$q_{\downarrow} = \frac{9.10}{24 \times 2 \times 0.60} = 0.32 \text{ t/cm}^2$$

$$\bar{f} = \frac{54.60 \times 12}{2 \times 0.60 \times (24)^3 / 12} = 0.475 \text{ t/cm}^2$$

$$\therefore f_{eq.} = \sqrt{(0.475)^2 + 3(0.32)^2} = 0.72 \text{ t/cm}^2 < 0.72 \times 1.1 \text{ t/cm}^2 \text{ Safe}$$





Sec. (2-2):

$$q_{//} = 0.32 \text{ t/cm}^2 \quad q_{\perp} = 0.475 \text{ t/cm}^2$$

$$\therefore q_r = \sqrt{0.32^2 + 0.475^2} = 0.574 \text{ t/cm}^2$$

$$< 0.72 \text{ t/cm}^2 \quad \text{Safe}$$

(4) Top Plate:

$$M_{1-1} = \frac{9.1}{2} \times 4.50 = 20.50 \text{ t.cm}$$

$$b_{st.} = 14 - 3.2 = 10.80 \text{ cms}$$

$$\therefore t_{\neq} = \sqrt{\frac{6 \times 20.50}{10.8 \times 1.40}} = 3.0 \text{ cms}$$

(5) Stresses on Concrete:

$$b_{act.} = 50 \text{ cms}$$

$$C = 1/2 f_c \times 3 a_c \times b$$

$$\therefore 45.40 \times 10^3 = 1/2 \times f_c \times 3 \times 13.25 \times 50$$

$$\therefore f_c = 45.70 \text{ kg/cm}^2 \quad \text{Safe}$$

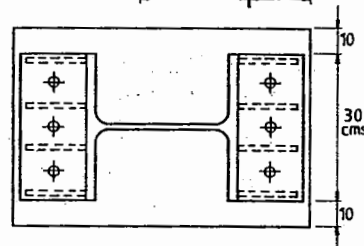
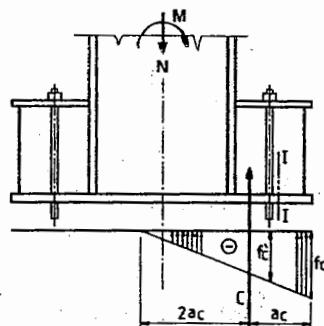
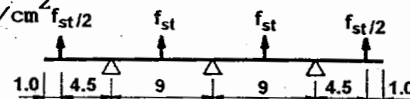
(6) Base Plate:

$$f_{c'} = \frac{3 \times 13.25 - 8}{3 \times 13.25} \times 45.70 = 36.50$$

$$M_{I-I} = \frac{1}{1000} \left[36.5 \times \frac{(8)^2}{2} + 1/2 (45.7 - 36.5) \times (8)^2 \times \frac{2}{3} \right]$$

$$= 1.363 \text{ cm.t/cm'}$$

$$\therefore t_{\neq} = \sqrt{\frac{6 \times 1.363}{1 \times 1.40}} = 2.60 \text{ cms}$$



Example (9.15):

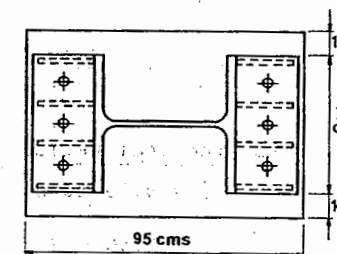
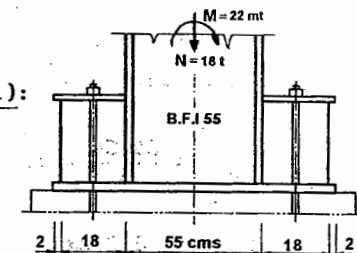
Redesign Example (9.12) using the same dimensioning of the vertical stiffeners and make use of the assumptions of Example (9.14) (i.e. 70% of compression resultant transmitted by the beam-column flange while 30% by the corresponding stiffeners).

Solution

* Approximate Dimensioning (as Ex.9.11):

$$b_{st.} = \frac{h}{3} = \frac{55}{3} = 18.0 \text{ cms}$$

$$\text{Over hanging part} = \frac{b}{3} = \frac{30}{3} = 10 \text{ cms}$$



(1) Anchor Bolts:

Assume that 70% of the compression goes directly from the flange to the base plate by contact. The remaining 30% is transmitted through stiffeners.

$$p = 10.5 \times 0.3 = 3.15 \text{ cms}$$

$$T = \frac{1}{62.65} \left[2200 - 18 \left(\frac{55}{2} + 1.65 \right) \right]$$

$$\therefore T = 25.50 \text{ tons}$$

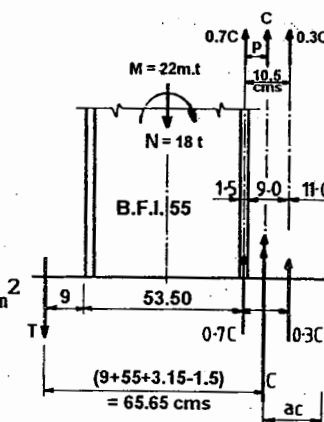
Use bolts d = 30 mms Grade 5.6

$$F_{tb} = 0.33 \times 5 = 1.65$$

$$A_{min. /bolt} = 0.80 \times \frac{\pi(3)^2}{4} = 5.60 \text{ cm}^2$$

$$\text{No. of bolts} = \frac{25.50}{1.65 \times 5.60} = 2.76$$

Use 3 bolts d = 30 mms Grade 5.6



**(2) Vertical Stiffeners:**

$$C = 25.50 + 18 = 43.50 \text{ tons}$$

$$\therefore 0.30 C = 13.05 \text{ tons}$$

$$T = 25.50 \text{ tons}$$

$\therefore$ Stiffeners at the tension side are governing.

$$a_c = \frac{0.3Cx11 + 0.7Cx21.50}{C}$$

$$\therefore a_c = 18.35 \text{ cms}$$

$$\therefore F_{st.} = \frac{25.50}{3} = 8.50 \text{ tons}$$

$$b_{st.} = 18.0 \text{ cms} \quad \frac{b_{st.}}{t_{st.}} < 16 \quad \therefore t_{st.} = 1.20 \text{ cm}$$

$$f_{act.} = \frac{8.50}{18 \times 1.20} = 0.40 \text{ t/cm}^2$$

(3) Vertical Welds:

$$Q \uparrow = 8.50 \text{ tons}$$

$$M = 8.50 \times 9 = 76.50 \text{ t.cm}$$

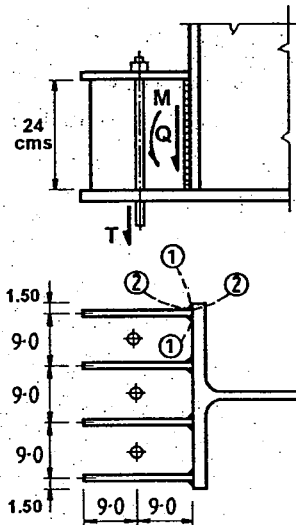
Assume size of weld = 0.70 cm

Sec. (1-1):

$$q_{//} = \frac{8.50}{2 \times 0.70 \times 24} = 0.25 \text{ t/cm}^2$$

$$f_{\perp} = \frac{76.50 \times 12}{2 \times 0.70 \times (24)^3/12} = 0.57 \text{ t/cm}^2$$

$$f_{eq.} = \sqrt{(0.57)^2 + 3(0.25)^2} = 0.71 \text{ t/cm}^2 < 1.1 \times 0.72 \text{ t/cm}^2$$

**Sec. (2-2):**

$$q_{//} = 0.25 \text{ t/cm}^2 \quad q_{\perp} = 0.57 \text{ t/cm}^2$$

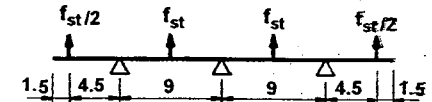
$$q_r = \sqrt{(0.25)^2 + (0.57)^2} = 0.62 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ Safe}$$

(4) Top Plate:

$$M = \frac{8.50}{2} \times 4.50 = 19.125 \text{ t.cm}$$

$$b = 20 - 3.20 = 16.80 \text{ cms}$$

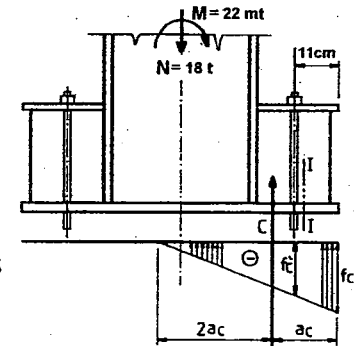
$$t_{\neq} = \sqrt{\frac{6 \times 19.125}{16.80 \times 1.40}} = 2.40 \text{ cms}$$

**(5) Stresses on Concrete:**

$$C = 1/2 f_c \times 3a_c \times b$$

$$43.5 \times 10^3 = 1/2 f_c \times 3 \times 18.35 \times 50$$

$$\therefore f_c = 31.60 \text{ kg/cm}^2 < 60 \text{ kg/cm}^2 \text{ Safe}$$

**(6) Base Plate:**

$$f_{c'} = \frac{3 \times 18.35 - 11}{3 \times 18.35} \times 31.60 = 25.28 \text{ kg/cm}^2$$

$$M_{I-I} = \frac{1}{1000} \left[25.28 \times \frac{(11)^2}{2} + \frac{1}{2} \times (31.6 - 25.28) \times (11)^2 \times \frac{2}{3} \right] = 1.78 \text{ t.cm/cm'}$$

$$\therefore t_{\neq} = \sqrt{\frac{6 \times 1.78}{1 \times 1.40}} = 2.80 \text{ cms}$$

Comments Regarding the Different Assumptions:

From the table below, it can be observed that:

(a) The assumption of Ex. 9.12 that only the vertical stiffeners transmit the total resultant compressive force is somewhat imaginary and leads to a conservative base plate thickness.

	Stiffeners		Bolt		Top Plate	f_c	Base Plate		
	b cms	t cms	n	d cms	t cms	kg/cm ²	t cms	B cms	D cms
Ex 9.12	18	1.2	3	3.0 St 37	2.8	45.6	3.7	50	95
Ex 9.13	12.0	0.8	3	3.0 St 44	3.0	44.5	2.6	50	83
Ex 9.14	12.0	0.8	3	3.0 St 44	3.0	45.7	2.6	50	83
Ex 9.15	18.0	1.2	3	3.0 St 44	2.4	31.6	2.8	50	95

(b) The assumption of Ex. 9.14 that 70% of the resultant compressive force is transmitted through the compression flange and 30% by the vertical stiffeners, seems to be the simplest approximate method. The obtained results are nearly the same as for Ex. 9.13 where the assumptions were based on the finite elements method.

Hence for simplicity and for realistic path of forces it is recommended to consider that:

- 70% of C is transmitted via the compression beam-column flange and that the remaining 30% by the corresponding vertical stiffeners.

For the predimensioning, take $T_{bolts} = 0.675 T_f$ and $C = N + T_{bolts}$, while conservative but it can be modified when proceeding the analysis.

Example (9.16):

Design a fixed base connection to support a bending moment of 38.0 mt and a normal force of 20 tons as shown in the figure below.

Solution

* Dimensioning of Connection:

$$T_f = \frac{3800}{42.50} - \frac{20}{2} = 79.41 \text{ tons}$$

$$T_{bolts} \cong 0.675 \times 79.41 = 53.60 \text{ tons}$$

$$C = 53.60 + 20 = 73.60 \text{ tons}$$

$$A_{bolts} = \frac{53.60}{0.33 \times 1.4} = 40.6 \text{ cm}^2 \text{ (Grade 4.6)}$$

Use bolts d = 36 mms

$$A_b = 0.80 \times \frac{\pi(3.6)^2}{4} = 8.17 \text{ cm}^2_{\min}$$

$$\text{No. of bolts} = \frac{40.6}{8.17} \cong 5.0 \text{ bolts}$$

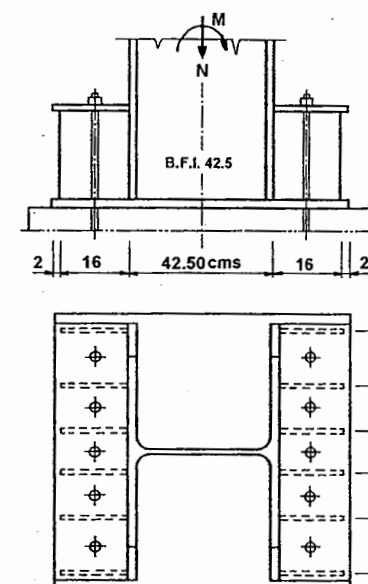
1st Alternative (Extended Plate):

Extend the column flanges and put the five bolts on one row outside the column flange.

$$\text{No. of stiffeners} = 6.0$$

$$F_{st.} = \frac{73.60}{5} = 14.72 \text{ tons}$$

$$\therefore b_{st.} \times t_{st.} = \frac{14.72}{1.40} = 10.50 \text{ cm}^2$$



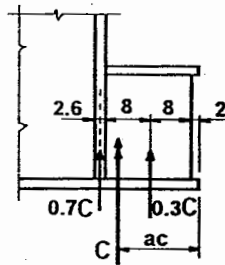


$$b_{st.} = 16 t_{st.} \text{ (Local Buckling)}$$

$$\therefore b_{st.} = 16.0 \text{ cms and } t_{st.} = 1.0 \text{ cm}$$

Assumptions:

- 1- All the tension is supported by the bolts in the tension side. This tension is equally distributed among bolts.
- 2- 30 % of the compression force is transmitted through the stiffeners while 70% by the flange.



$$a_c = \frac{0.3C \times 10 + 0.7C(18 + 1.3)}{C}$$

$$\therefore a_c = 16.51 \text{ cms}$$

$$\sum M_o = 0.0$$

$$\therefore T = \frac{1}{52} \times \left[3800 - 20 \times \left(\frac{42.50}{2} + 1.49 \right) \right] = 64.30 \text{ tons}$$

$$\therefore C = 64.30 + 20 = 84.30 \text{ tons}$$

(1) Anchor Bolts:

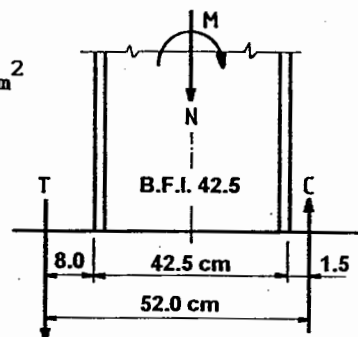
$$f_{act}/\text{bolt} = \frac{64.30}{5 \times 8.14} = 1.58 \text{ t/cm}^2$$

$$\therefore \text{Use 5 bolts } d = 36 \text{ mms St. (44)}$$

(2) Stiffeners:

$$F_{st.} = \frac{64.30}{5.0} = 12.90 \text{ tons}$$

tension side



$$\text{Compression Supported by stiffeners in the compression side} \\ = 0.30 \times 84.30 = 25.30 \text{ tons}$$



$\therefore$ Stiffeners at tension side govern.

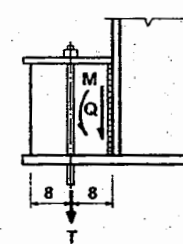
$$\therefore f_{act.} = \frac{12.90}{16 \times 1.0} = 0.81 \text{ t/cm}^2 < 1.40 \text{ t/cm}^2 \text{ Safe}$$

(3) Vertical Welds:

$$Q \uparrow = 12.90 \text{ tons}$$

$$M = 12.90 \times 8 = 103.20 \text{ t.cm}$$

$$\text{Use weld size} = 0.80 \text{ cm}$$

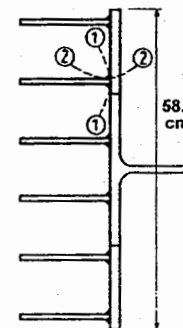
* Sec. (1-1):

$$q_{//} = \frac{12.90}{28 \times 2 \times 0.80} = 0.29 \text{ t/cm}^2$$

$$f_{\perp} = \frac{103.20 \times 14}{2 \times 0.80 \times (28)^3/12} \\ = 0.49 \text{ t/cm}^2$$

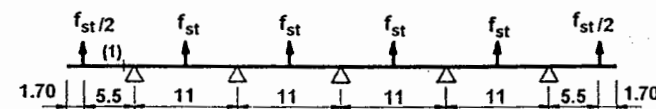
$$f_{eq.} = \sqrt{(0.49)^2 + 3(0.29)^2} = 0.70 \text{ t/cm}^2$$

$$< 0.72 \times 1.1 \text{ t/cm}^2 \text{ Safe}$$

Sec. (2-2):

$$q_{//} = 0.29 \text{ t/cm}^2 \quad q_{\perp} = 0.49 \text{ t/cm}^2$$

$$q_r = \sqrt{(0.29)^2 + (0.49)^2} = 0.57 \text{ t/cm}^2 < 0.72 \text{ t/cm}^2 \text{ Safe}$$

(4) Top Plate:



$$M_{I-I} = \frac{12.9}{2} \times 5.50 = 35.5 \text{ t.cm}$$

$$b_{st.} = 18 - 3.80 = 14.20 \text{ cms}$$

$$t_{\neq} = \sqrt{\frac{6 \times 35.5}{14.20 \times 1.40}} = 3.40 \text{ cms}$$

(5) Concrete Footing:

$$C = 1/2 f_c \times 3a_c \times b$$

$$84.3 \times 10^3 = 1/2 \times 60 \times 3 \times 16.50 \times b_{min.}$$

$$\therefore b_{min.} = 57 \text{ cms take } b = 70 \text{ cms}$$

$$\therefore 84.3 \times 10^3 = 1/2 f_c \times 3 \times 16.50 \times 70$$

$$\therefore f_c = 48.70 \text{ kg/cm}^2 < 60 \text{ kg/cm}^2 \text{ Safe}$$

(6) Base Plate:

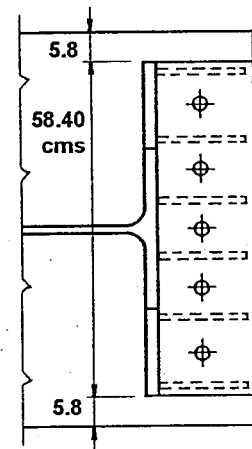
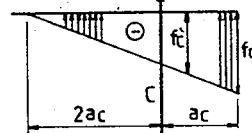
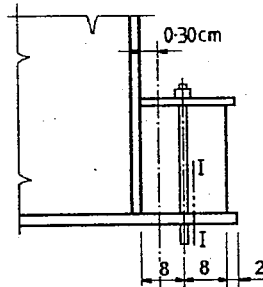
$$f_{c'} = \frac{3 \times 16.5 - 10}{3 \times 16.5} \times 48.70$$

$$= 38.80 \text{ kg/cm}^2$$

$$M_{I-I} = \frac{1}{1000} \left[38.8 \times \frac{(10)^2}{2} + \frac{1}{2} \times (48.70 - 38.8) \times (10)^2 \times \frac{2}{3} \right]$$

$$= 2.27 \text{ t.cm/cm'}$$

$$\therefore t_{\neq} = \sqrt{\frac{6 \times 2.27}{1.0 \times 1.40}} = 3.20 \text{ cms}$$



Example (9.17):

2nd Alternative: (Two Rows of Bolts)

Use one row of bolts outside the column flange and another row inside it.

Assumptions:

- 1- The whole tension force is transmitted to the bolts in the tension side. This tension is equally distributed among the bolts.
- 2- The compression force resultant coincides with the outer face of the column compression flange.

$$a_t = \frac{3 \times 8 - 2 \times 10.60}{5.0} \approx 0.60 \text{ cm}$$

(1) Anchor Bolts:

$$T = \frac{1}{43.10} \left[3800 - 20 \times \frac{42.50}{2} \right]$$

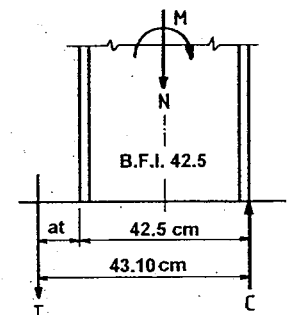
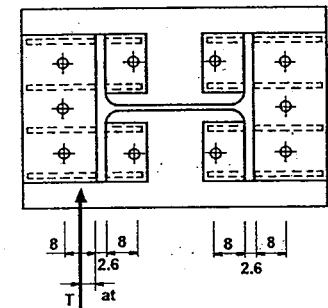
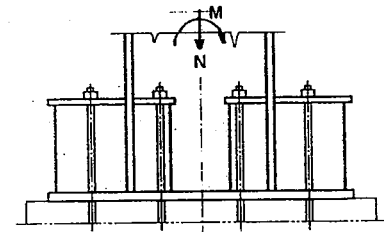
$$= 78.30 \text{ tons}$$

$$A_{bolts} = \frac{78.30}{0.33 \times 5} = 47.45 \text{ cm}^2 \quad (\text{Grade 5.6})$$

$$\text{Use } d = 36 \text{ mms, } A_{min.} = 8.17 \text{ cm}^2$$

$$\text{No. of bolts} = \frac{47.45}{8.17} = 6 \text{ bolts}$$

$\therefore$ Use 4 bolts outside column flange and 2 bolts inside it.





$$a_t = \frac{4 \times 8 - 2 \times 10.6}{6} = 1.80 \text{ cm}$$

$$T = \frac{1}{44.30} \left[3800 - 20 \times \frac{42.50}{2} \right]$$

$$= 76.20 \text{ tons}$$

$$f_{\text{act. bolt}} = \frac{76.20}{6 \times 8.14} = 1.56 \text{ t/cm}^2$$

$$< 1.65 \text{ t/cm}^2 \text{ Safe}$$

Use 6 bolts $d = 36 \text{ mms}$ Grade 5.6

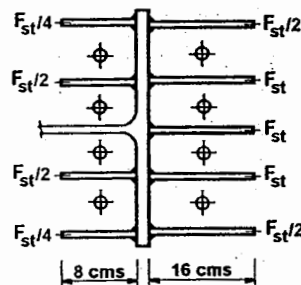
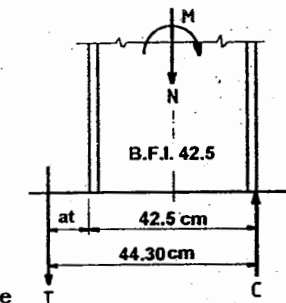
(2) Stiffeners:

$$C = 76.2 + 20 = 96.20 \text{ tons}$$

$$F_{\text{st.}} = \frac{96.20}{5.5} = 17.40 \text{ tons}$$

$$f_{\text{st.}} = \frac{17.40}{16 \times 1.0} = 1.08 \text{ t/cm}^2$$

$$< 1.40 \text{ t/cm}^2 \text{ Safe}$$



The eccentricity of the compression force (C) regarding the stiffeners has been neglected.

(3) Vertical Welds:

$$Q \downarrow = 17.40 \text{ tons}$$

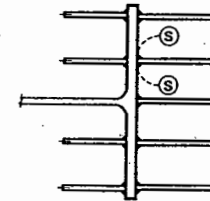
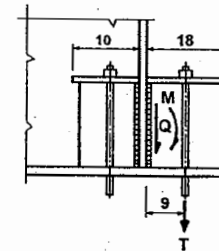
M can be neglected.

Use weld size = 0.80 cm



Sec. (s-s):

$$q_{//} = \frac{17.4}{28 \times 2 \times 0.80} = 0.435 \text{ t/cm}^2 < 0.72 \text{ (o.k.)}$$

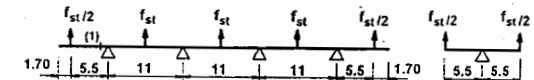


(4) Top Plate:

$$M_{1-1} = \frac{17.4}{2} \times 5.50 \approx 47.85 \text{ t.cm}$$

$$b_{\text{st.}} = 18 - 3.80 = 14.20 \text{ cms}$$

$$t_{\neq} = \sqrt{\frac{6 \times 47.85}{14.2 \times 1.40}} = 3.80 \text{ cms}$$



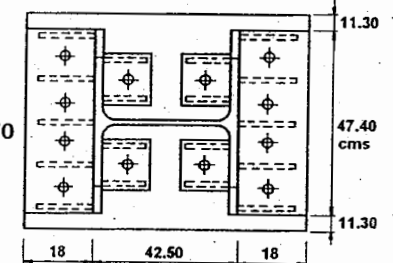
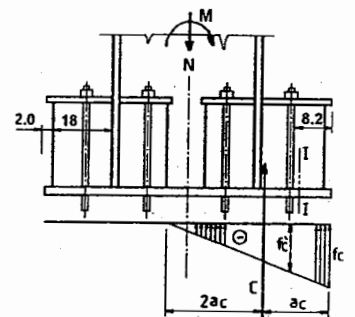
(5) Concrete Footing:

$$C = 1/2 f_c \times 3a_c \times b$$

$$C = 76.20 + 20 = 96.20 \text{ tons}$$

$$96.20 \times 10^3 = 1/2 f_c \times 3 \times 18 \times 70$$

$$f_c = 50.90 \text{ kg/cm}^2$$



(6) Base Plate:

$$f_c = \frac{3 \times 18 - 8.2}{3 \times 18} \times 50.90 = 43.10 \text{ kg/cm}^2$$

$$M_{I-I} = \frac{1}{1000} \left[43.10 \times \frac{(11)^2}{2} + \frac{1}{2} \times (50.9 - 42.42) \times (11)^2 \times \frac{2}{3} \right]$$

$$\approx 3.00 \text{ t.cm}$$

$$\therefore t_x = \sqrt{\frac{6 \times 3.00}{1.0 \times 1.40}} = 3.58 \text{ cms} \rightarrow 3.6 \text{ cms}$$

Example (9.18):

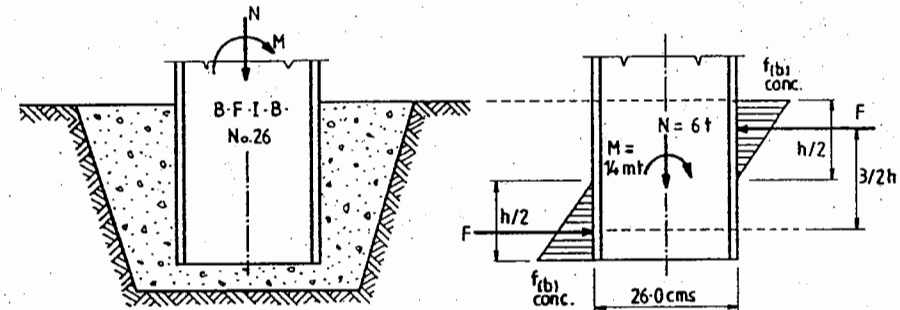
Design the base connection of a beam-column supporting a moment $M = 14 \text{ mts}$ and a normal force $N = 6.0 \text{ tons}$. The beam-column has a B.F.I.B. cross section No. 26.

Use the pocket base concept.

SolutionProperties of B.F.I.B. No. (26):

$$b = 26 \text{ cms} \quad h = 26 \text{ cms} \quad t_f = 1.80 \text{ cm} \quad t_w = 1.10 \text{ cm}$$

$$A = 121.0 \text{ cm}^2 \quad Z_x = 1160 \text{ cm}^3$$

Determination of the embedded height (h):

Refer to section 9.8 using equation 9.16 get

$$h = \sqrt{\frac{6 M}{b f_{con}}}$$

$$\therefore h = \sqrt{\frac{6 \times 14 \times 10^5}{60 \times 26}} = 73.0 \text{ cms}$$

Use $h = 80 \text{ cms}$.

Check of Bearing and Bond Stresses:

Consider that the bond will exist only between the flange in compression and half the web. While the bearing will exist between the whole cross sectional area and the concrete.

$$\text{i.e. Area in bond} = \left[26 + 12.45 + 2 \frac{(26 - 2 \times 1.80)}{2} \right] 80$$

$$= 5864 \text{ cm}^2$$

$$\text{Area in bearing} = 121 \text{ cm}^2$$

$$\text{If we assume: } f_{\text{bond}} = 7 \text{ kg/cm}^2$$

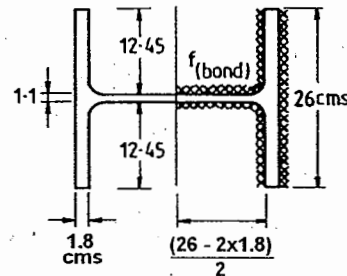
$$f_{(b)} = 60 \text{ kg/cm}^2$$

conc.

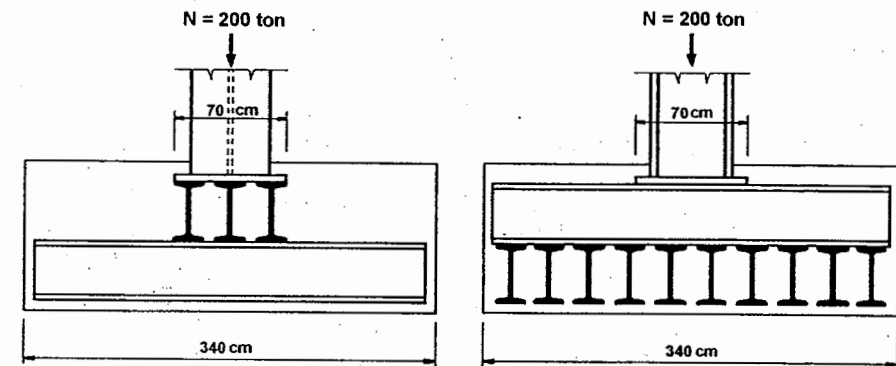
The axial force resisted by bond

$$\text{and bearing} = \frac{5864 \times 7}{1000} + \frac{121 \times 60}{1000}$$

$$= 41.048 + 7.26 = 48.308 \text{ tons} > 6.0 \text{ tons (Safe)}$$

Example (9.19):

Design a square base connection to support a total vertical normal force of 200 tons. The column is a B.f.I.B. section No. (40). Choose the grid base connection concept.

Solution

* Consider that the approximate weight of gird base represents 10% of the applied vertical force.

i.e. Total load = $1.10 \times 200 = 220$ tons.

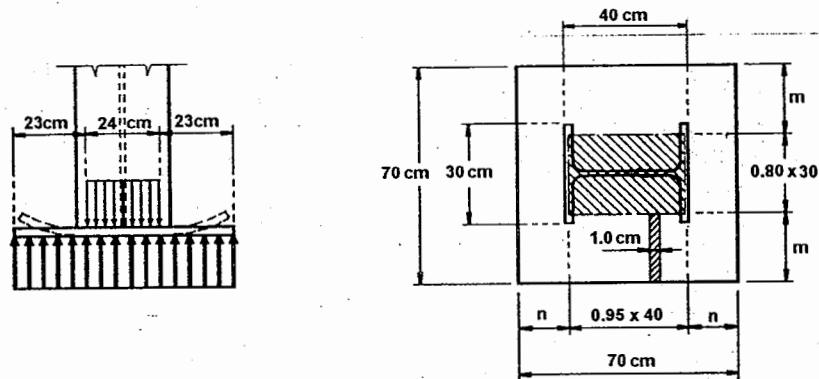
* Given the safe bearing pressure on the ground is 2.0 kg/cm^2 .

(a) Base Plate:

Assume to use a square base plate 70 cms x 70 cms.

The pressure under the base plate is:

$$w_1 = \frac{200}{70 \times 70} = 0.0408 \text{ t/cm}^2$$



$$\therefore M = 0.0408 \times \text{Unit length} \times 23 \times \frac{23}{2} = 10.79 \text{ t.cm}$$

$$\therefore t = \sqrt{\frac{6 \times 10.79}{1.0 \times 1.40}} = 6.80 \text{ cms}$$

$$m = \frac{70 - 0.80 \times 30}{2} = 23 \text{ cms}$$

← (governs)

$$n = \frac{70 - 0.95 \times 40}{2} = 16 \text{ cms}$$

(b) Grid Moments and Shears:

$$\text{Area of concrete base} = \frac{220}{20} = 11 \text{ m}^2$$

$\therefore$ Use a square base 3.40 m x 3.40 m

$$\begin{aligned} \text{The pressure on the lower set of beams} &= W_2 = \frac{200}{3.40 \times 3.40} \\ &= 17.30 \text{ t/m}^2 \end{aligned}$$

For the upper set of beams:

Section (A-A):

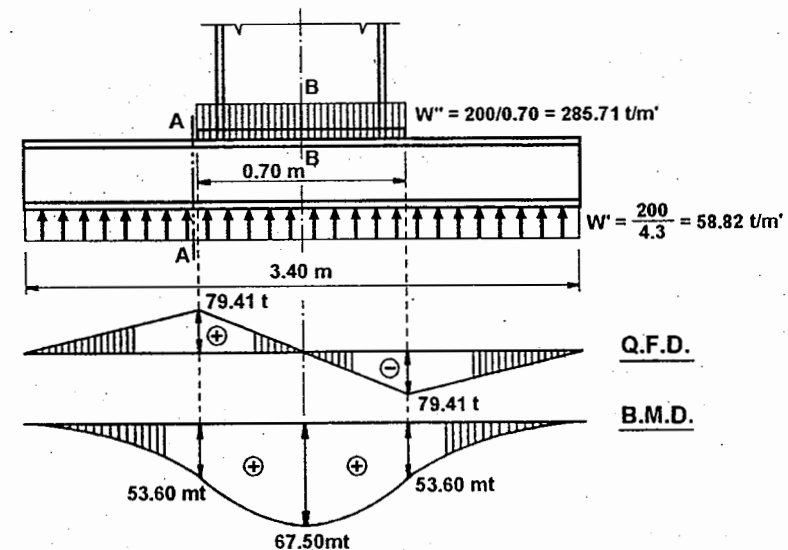
$$\text{Shear} = \left(\frac{200}{3.40} \right) \times \left(\frac{3.40 - 0.70}{2} \right) = 79.41 \text{ tons}$$

$$\text{Moment} = \frac{200}{3.40} \times \frac{1}{2} \left(\frac{3.40 - 0.70}{2} \right)^2 = 53.60 \text{ ton.m}$$

Section (B-B):

$$\text{Shear} = 0.0$$

$$\begin{aligned} \text{Moment} &= \frac{200}{3.40} \times \frac{3.40}{2} \times \frac{3.40}{4} - \frac{200}{0.70} \times \frac{0.70}{2} \times \frac{0.70}{4} \\ &= \frac{200}{8} (3.40 - 0.70) = 67.50 \text{ ton.m} \end{aligned}$$



(c) Upper Set of Beams:

Neglect the effect of concrete acting with the steel beams.

Assume to use three beams, then the section modulus required for each beam is:

$$Z_x = \frac{67.50 \times 100}{3 \times 1.40} = 1607.1 \text{ cm}^3$$

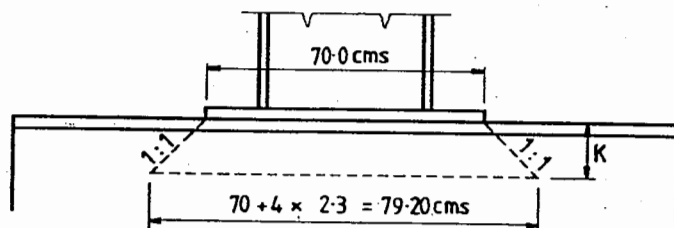
$$\therefore \text{Try three S.I.B. No. } 42\frac{1}{2} \text{ (} Z_x = 1740 \text{ cm}^3 \text{)}$$

$$q = \frac{79.41}{3 \times (42.5 - 2 \times 2.3) \times 1.53} = 0.45 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2$$

Safe

The I cross section is checked against web crushing.

Refer to the following figure:



$$k = 2 t_f = 2 \times 2.3$$

$$k = 4.60 \text{ cms}$$

$$\therefore t_w = 1.53 \text{ cm}$$

$$\text{Crippling stress} = f_{\text{crp}} = \frac{200}{3 \times 79.20 \times 1.53} = 0.55 \text{ t/cm}^2 < 1.80 \text{ Safe}$$

Combined stresses are checked at section (A-A) at the edge the base plate.

$$f_c = \frac{53.6 \times 100}{3 \times 1740} = 1.02 \text{ t/cm}^2$$

$$q = 0.45 \text{ t/cm}^2$$

$$f_{\text{eq.}} = \sqrt{(1.02)^2 + 3(0.45)^2} = 1.27 \text{ t/cm}^2 < 1.10 \times 1.40 \text{ Safe}$$

Be sure that the breadth of the flange of the three S.I.B. do not exceed the breadth of the base plate i.e. :-

$$3b + 2c = 70 \text{ cms}$$

$$c = 70 - \frac{3 \times 16.3}{2} = 10.1 \text{ cms (o.k.)}$$

(d) Lower Set of Beams:

Using ten beams, then

$$Z_x |_{\text{req.}} = \frac{6750}{10 \times 1.40} = 482.1 \text{ cm}^3$$

Use ten S.I.B. No. (28) ($Z_x = 542 \text{ cm}^3$)

$$(b = 11.90 \text{ cms } h = 28 \text{ cms } t_w = 1.01 \text{ cm } t_f = 1.52 \text{ cm})$$

$$\therefore f_c = \frac{6750}{10 \times 542} = 1.24 \text{ t/cm}^2 < 1.40 \text{ t/cm}^2 \text{ Safe}$$

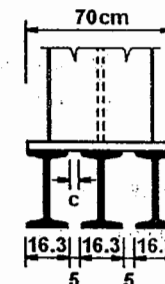
$$\therefore q = \frac{79.41}{10 \times (28 - 2 \times 1.52) \times 1.01} = 0.317 \text{ t/cm}^2 < 0.84 \text{ t/cm}^2 \text{ Safe}$$

$$\therefore f_{\text{eq.}} = \sqrt{(1.24)^2 + 3(0.317)^2} = 1.35 \text{ t/cm}^2 < 1.10 \times 1.40 \text{ Safe}$$

The size chosen for the two sets of beams, will allow adequate spacing between flanges for placing and compaction of concrete :-

$$10 \times 11.9 + 9C = 340 \text{ cms}$$

$$C = 24.55 \text{ cms}$$



REFERENCES

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