ABOUT CONNECTIONS IN STEEL FRAME BUILDINGS

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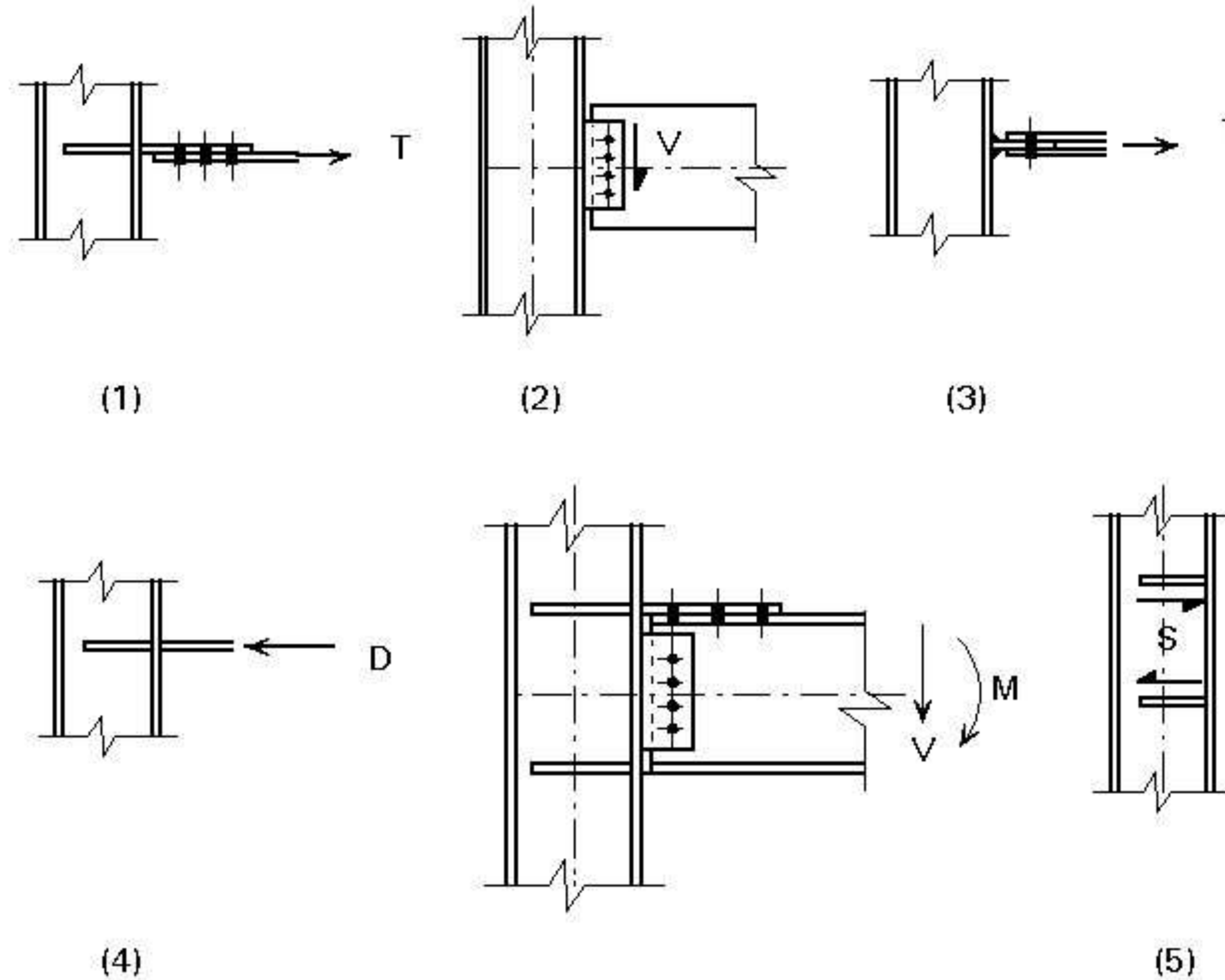
INTRODUCTION

❖ In current construction practice, steel members are joined by either bolting or welding. When fabricating steel for erection, most connections have the connecting material attached to one member in the fabrication shop and the other members attached in the field during erection.

❖This helps simplify shipping and makes erection faster.

❖Welding that may be required on a connection is preferably performed in the more-easily controlled environment of the fabrication shop. If a connection is bolted on one side and welded on the other, the welded side will usually be the shop connection and the bolted connection will be the field connection.



(1) Axial force
(2) Shear force
(3) Introduction of tensile force
(4) Introduction of compression force
(5) Shear panel

FIG. 1 BASIC FORCE TRANSFERS IN CONNECTION

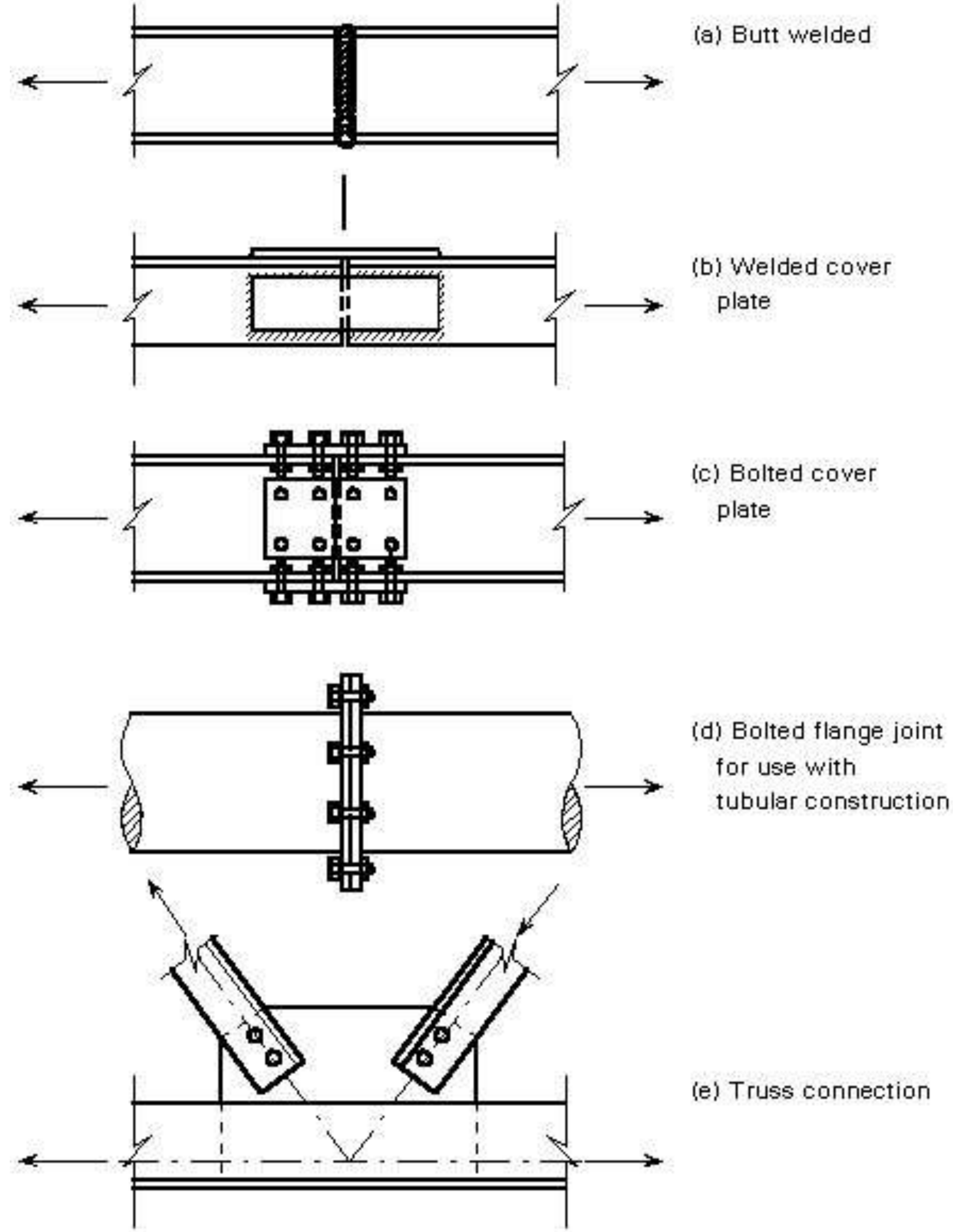


FIG. 2 EXAMPLE OF TRANSFER OF TENSILE OR COMPRESSIVE AXIAL MEMBER FORCE

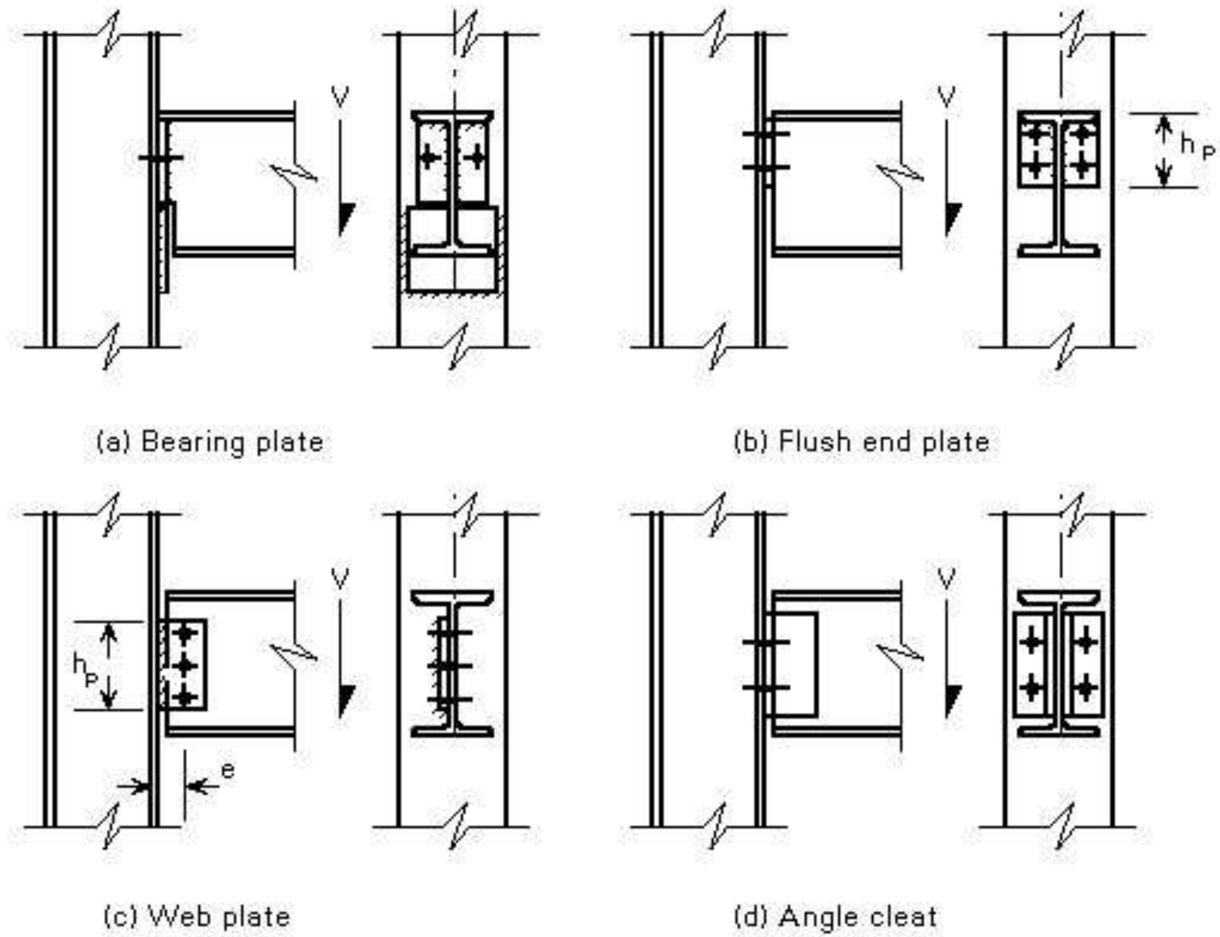


FIG. 3 EXAMPLE OF TRANSFER OF SHEAR FORCE

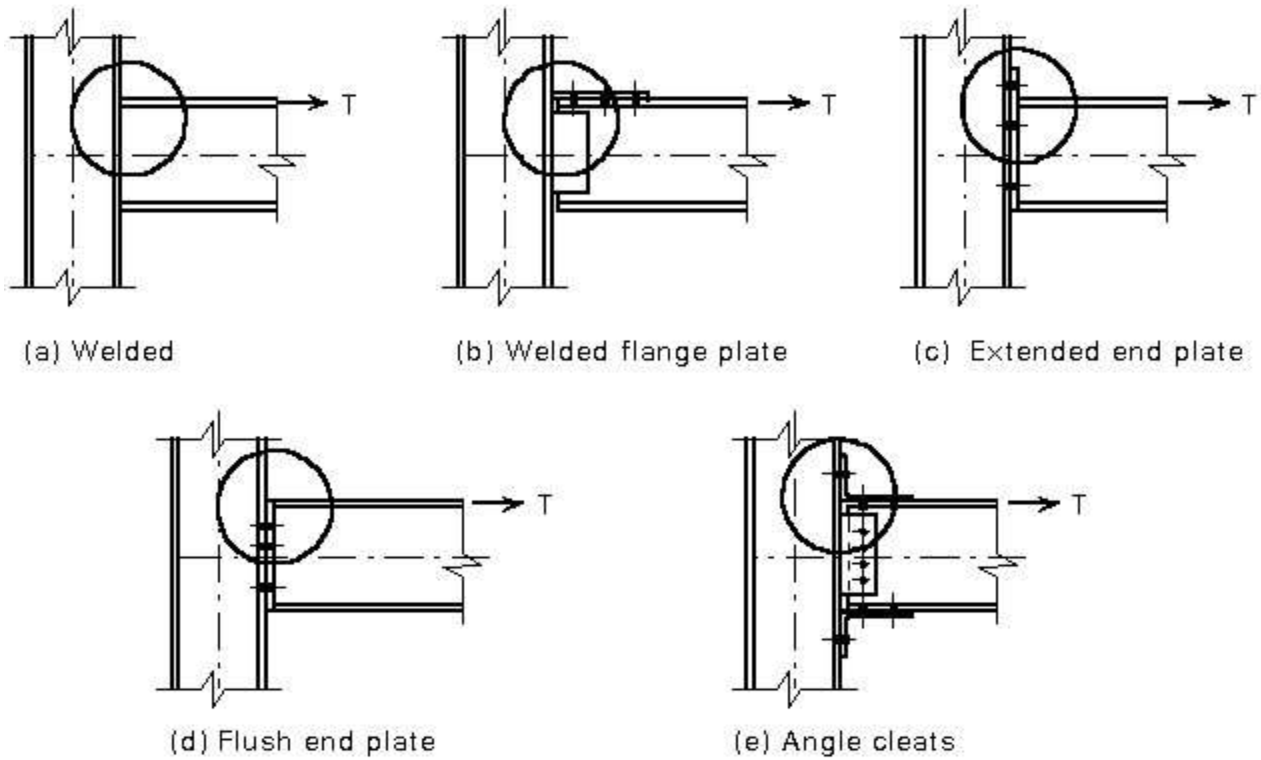


FIG. 4 EXAMPLE OF TRANSFER OF LOCAL TENSILE FORCE

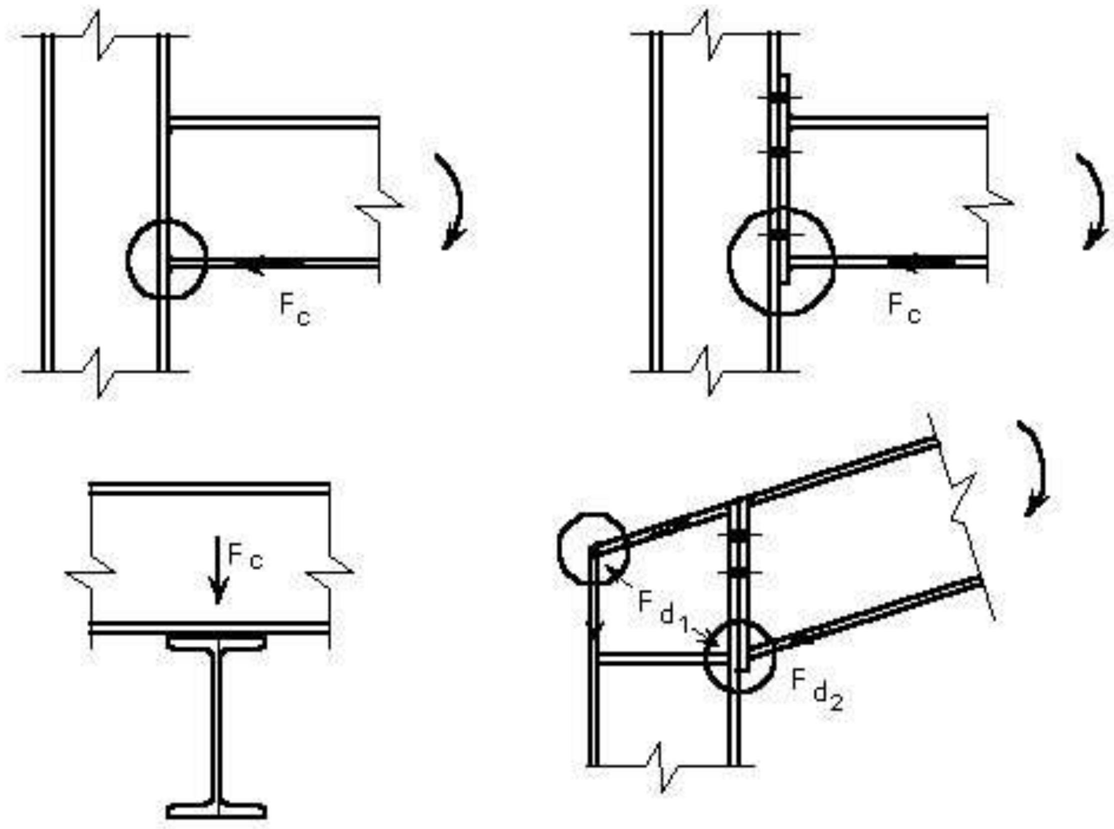


FIG. 5 EXAMPLE OF TRANSFER OF LOCAL COMPRESSIVE FORCE

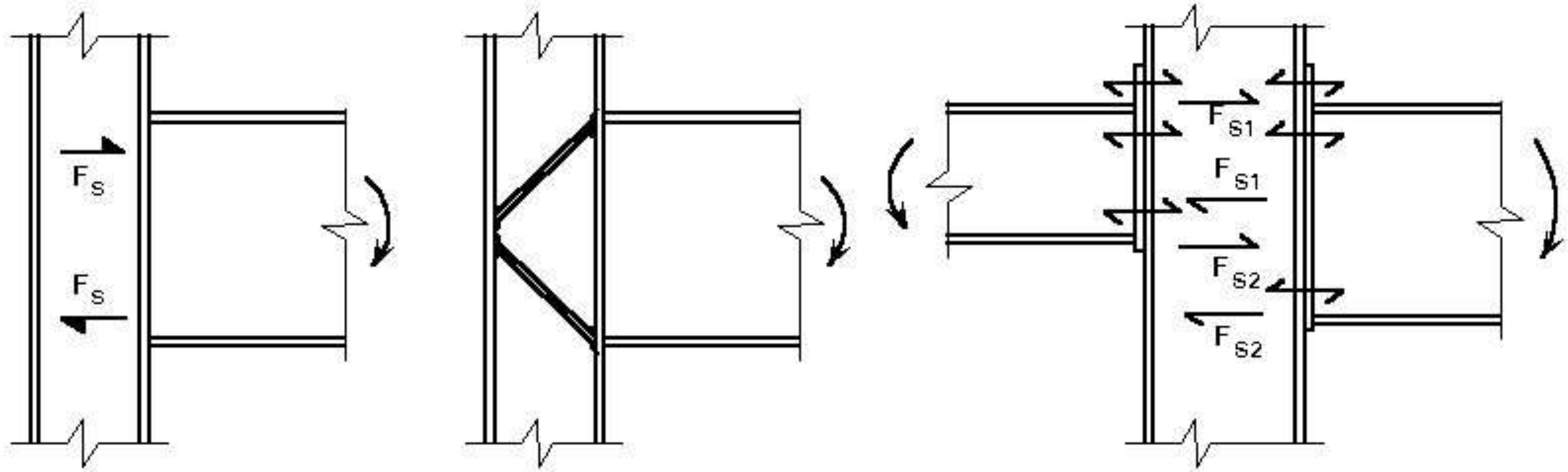


FIG. 6 EXAMPLE OF TRANSFER OF SHEAR FORCE THROUGH A SHEAR PANEL

•The principal structural requirement of a connection is that it be capable of safely transferring load from the supported members to the supporting member.

•The above requirement implies that three properties of the connection needs to be considered: strength, stiffness and deformation capacity.

•Beam-to-column connections can be classified by their stiffness as nominally pinned, semi-rigid or rigid. For their capability to transfer moments, they can be classified as nominally pinned, partial-strength and full-strength connections.

•The analysis of connections implies the assumption of a realistic internal distribution of forces that are in equilibrium with the external forces, where each element is capable of transferring the assumed force and the deformations are within the deformation capacity of the elements.

•In the analysis of connections, a number of basic load transfers can generally be identified.

BEAM-TO-BEAM CONNECTIONS

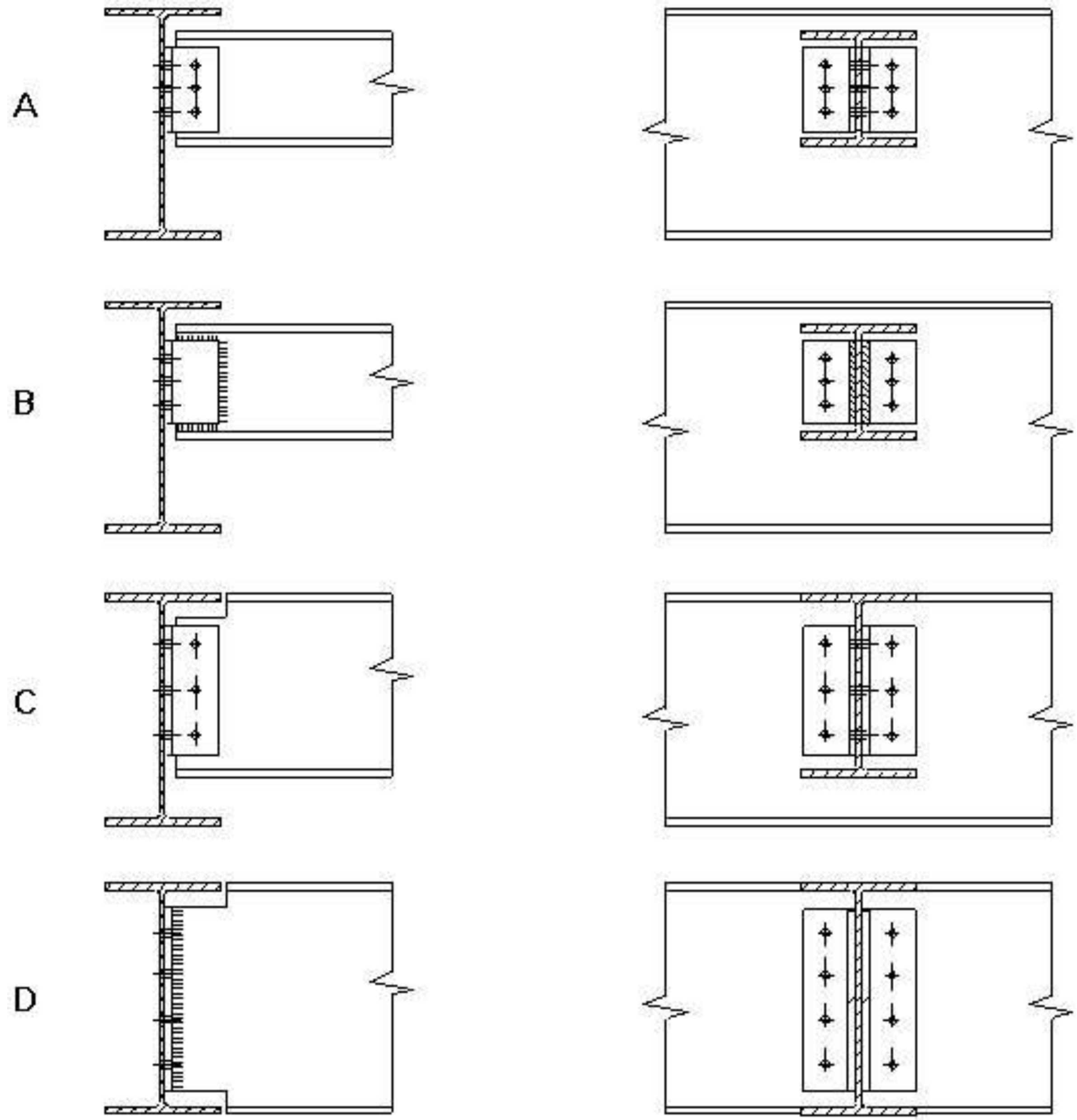


FIG. 7a BEAM TO BEAM CONNECTIONS

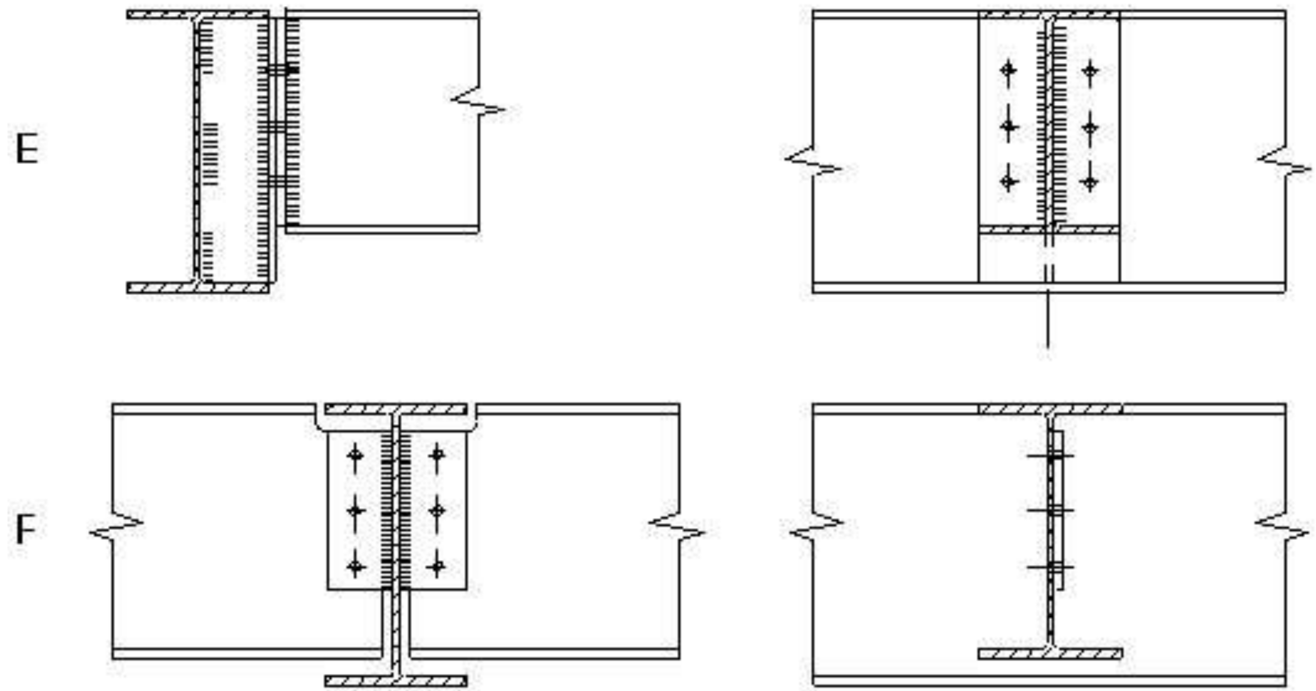


FIG. 7b BEAM TO BEAM CONNECTIONS

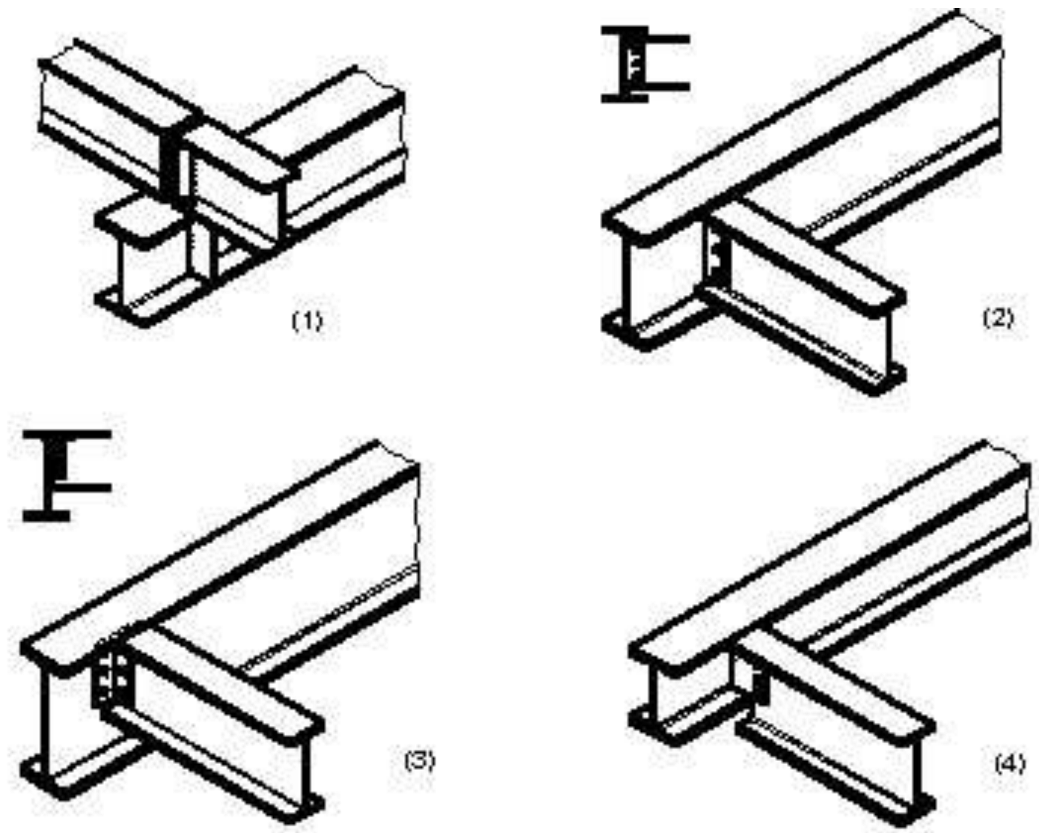


FIG. 7c BEAM TO BEAM CONNECTIONS

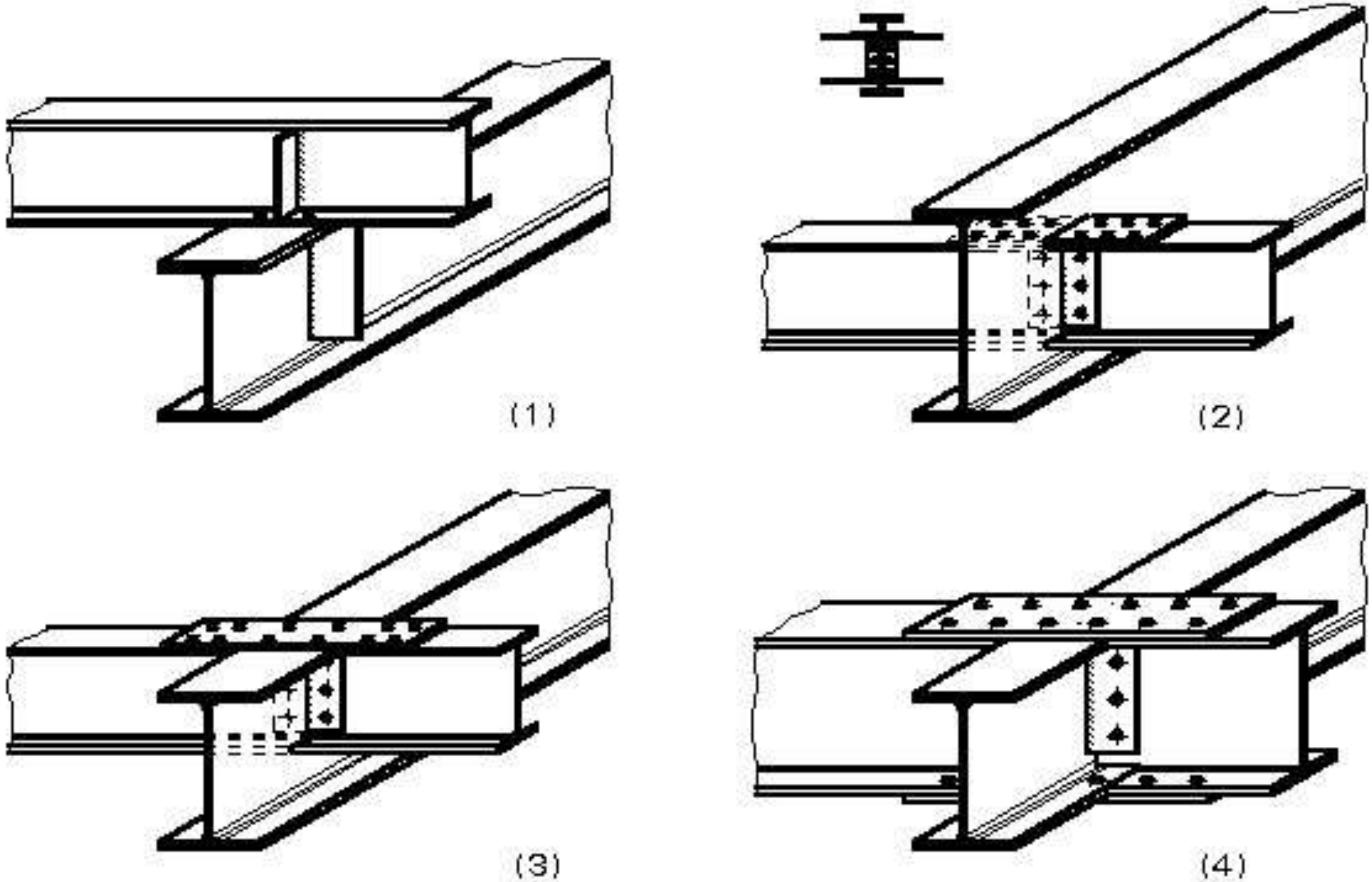


FIG. 7d BEAM TO BEAM CONNECTIONS

BEAM-TO-COLUMN CONNECTIONS

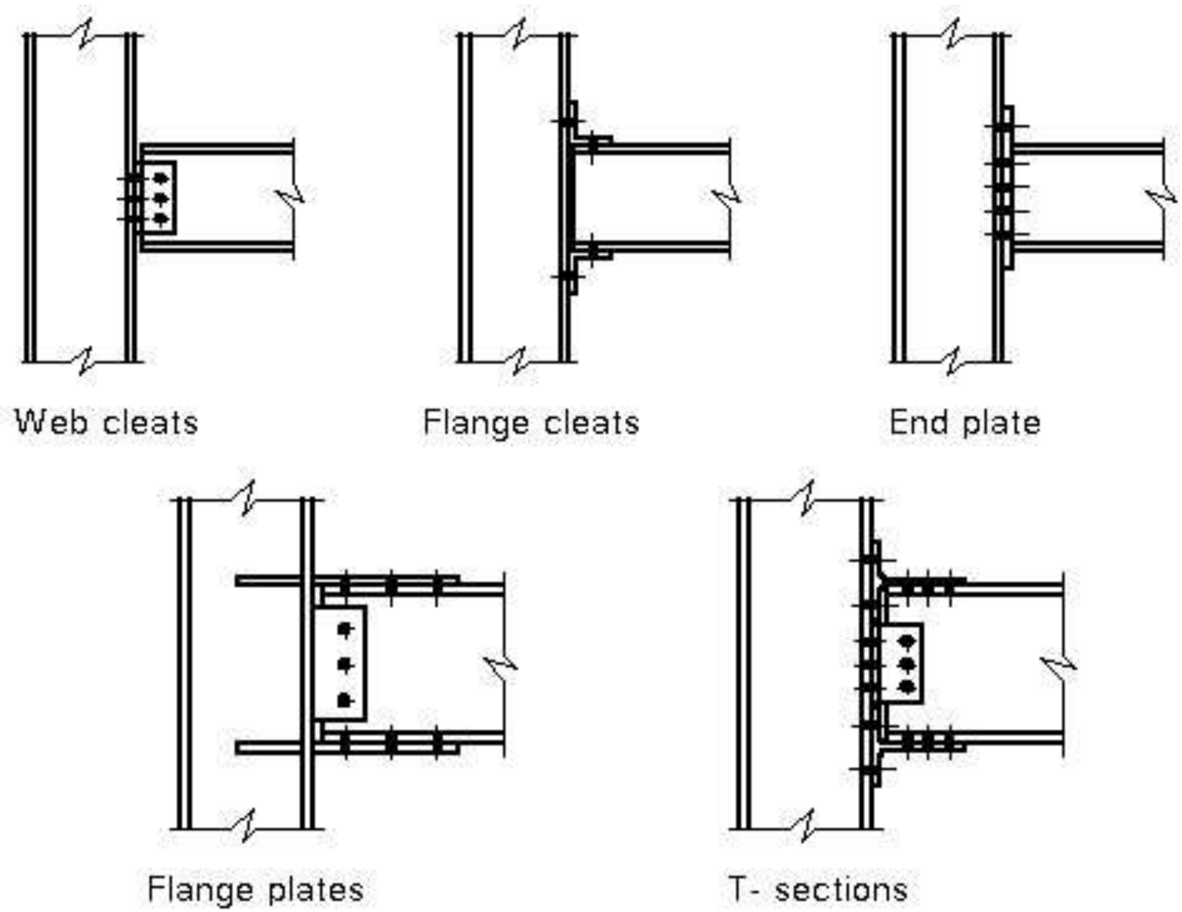


FIG. 8a BEAM TO COLUMN CONNECTIONS

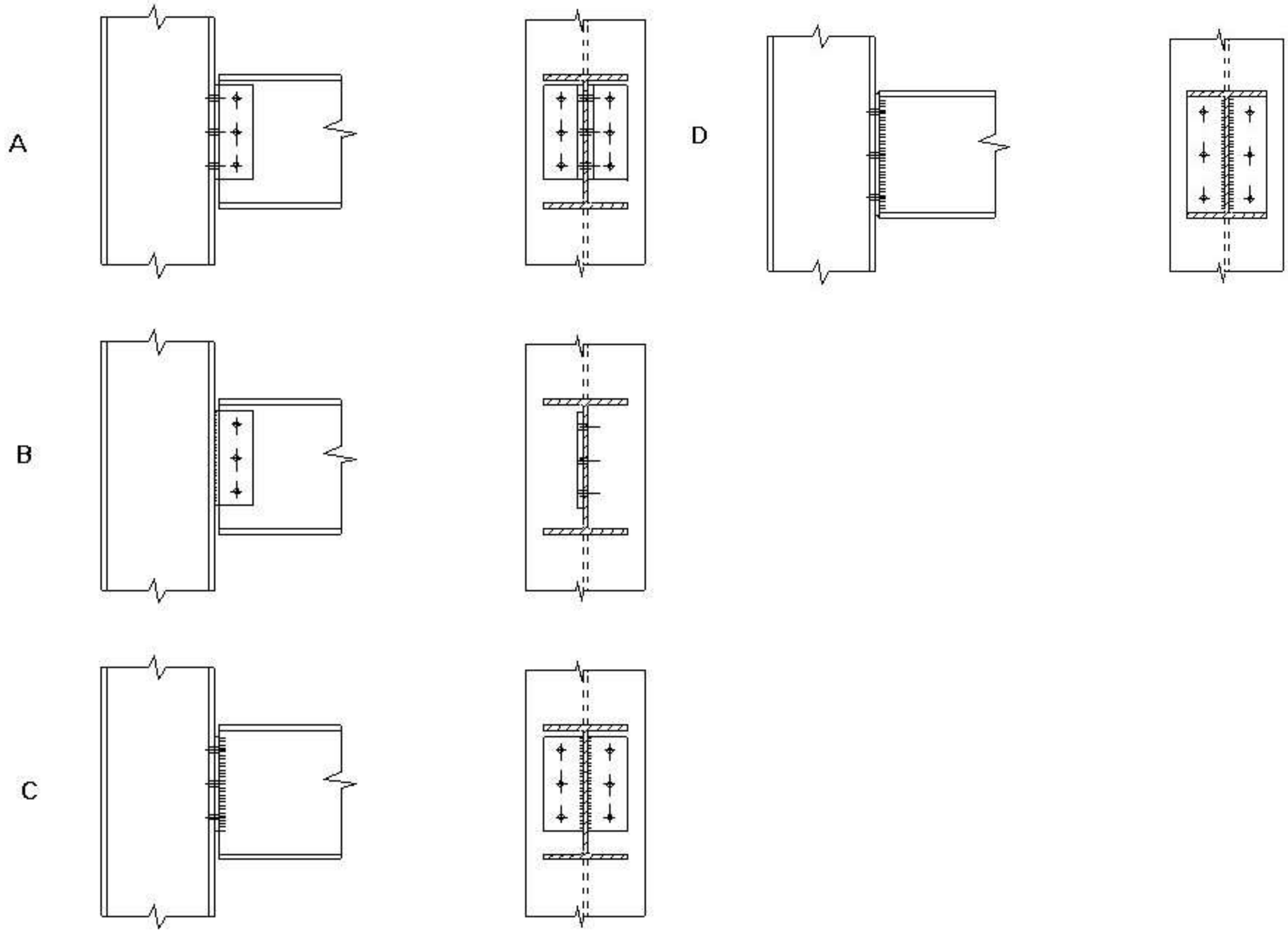


FIG. 8b BEAM TO COLUMN CONNECTIONS

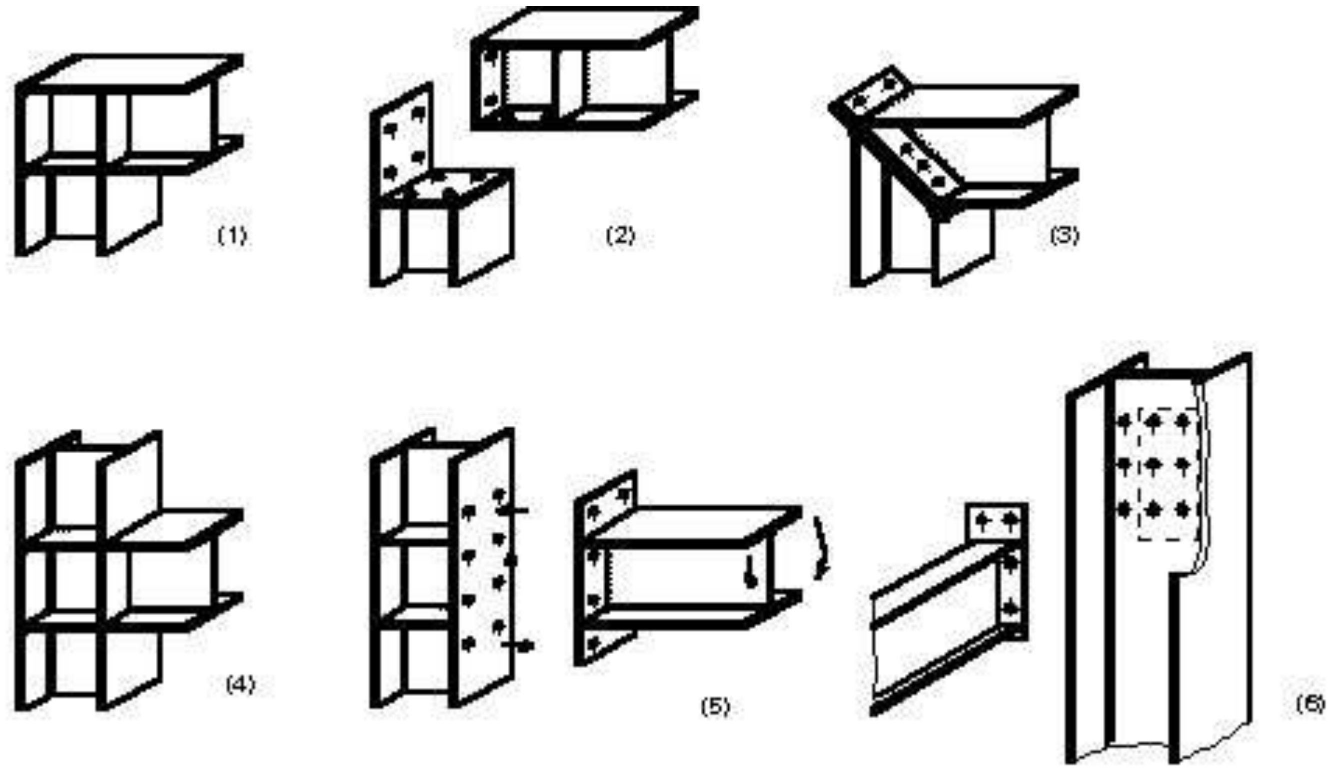


FIG. 8c BEAM TO COLUMN CONNECTIONS

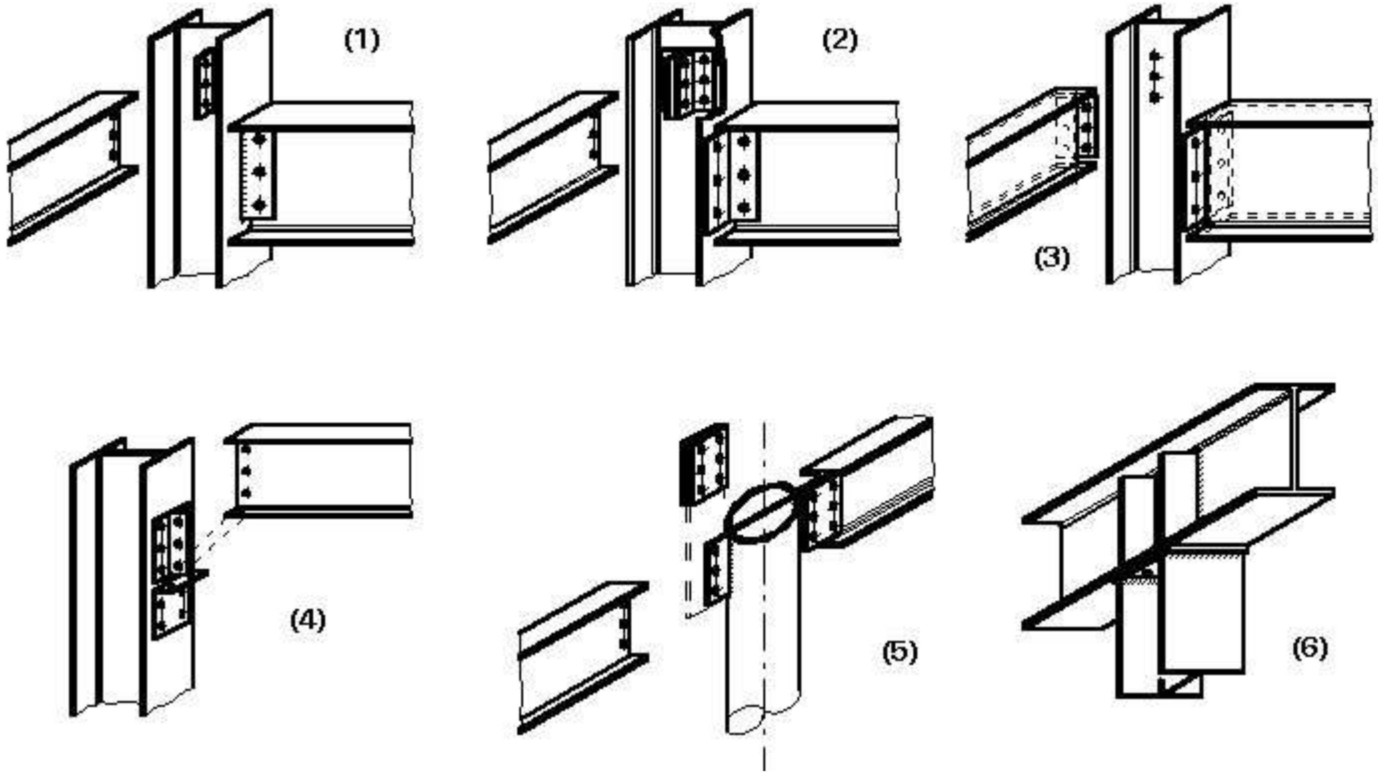


FIG. 8d BEAM TO COLUMN CONNECTIONS

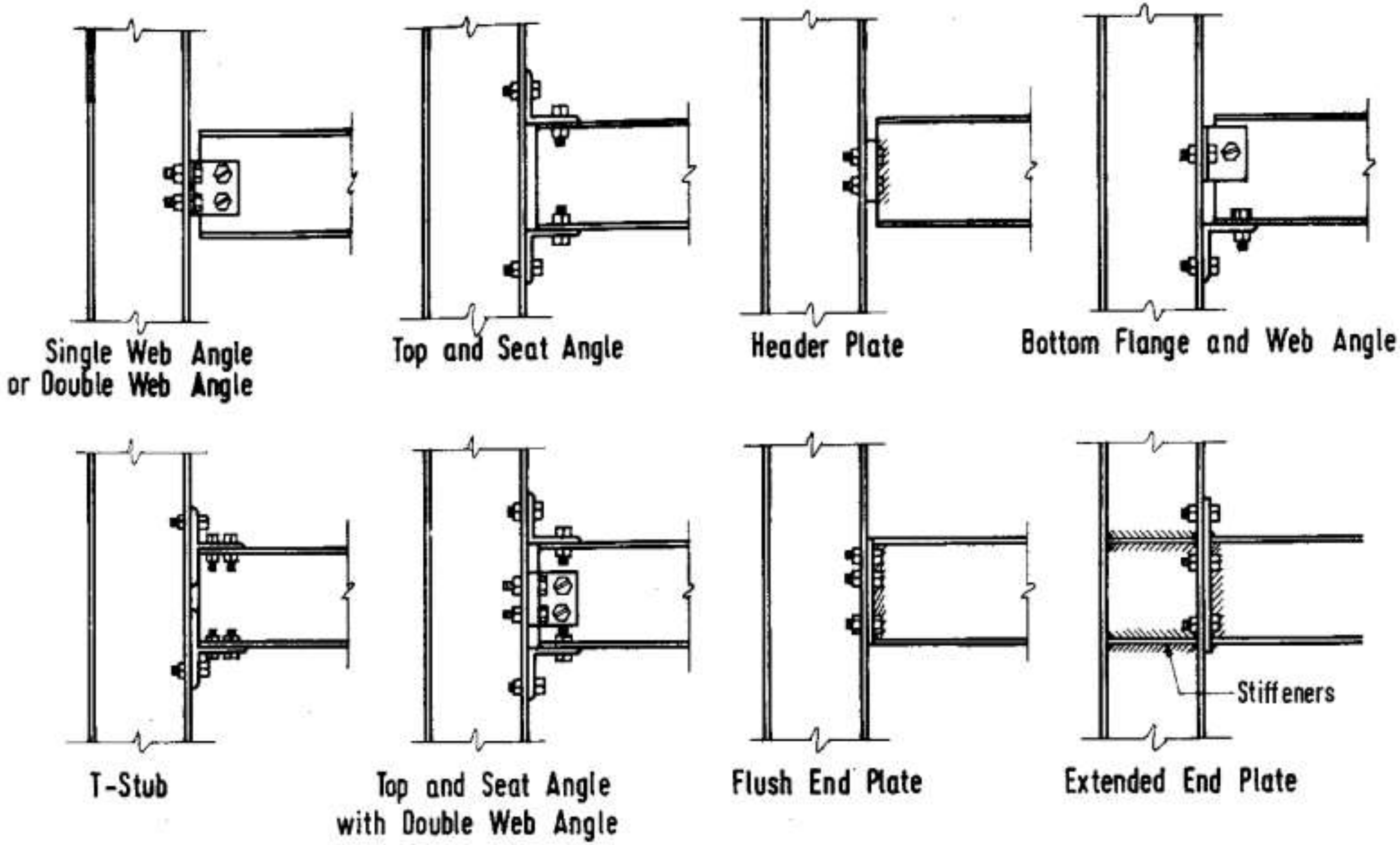


FIG. 8e BEAM TO COLUMN CONNECTIONS

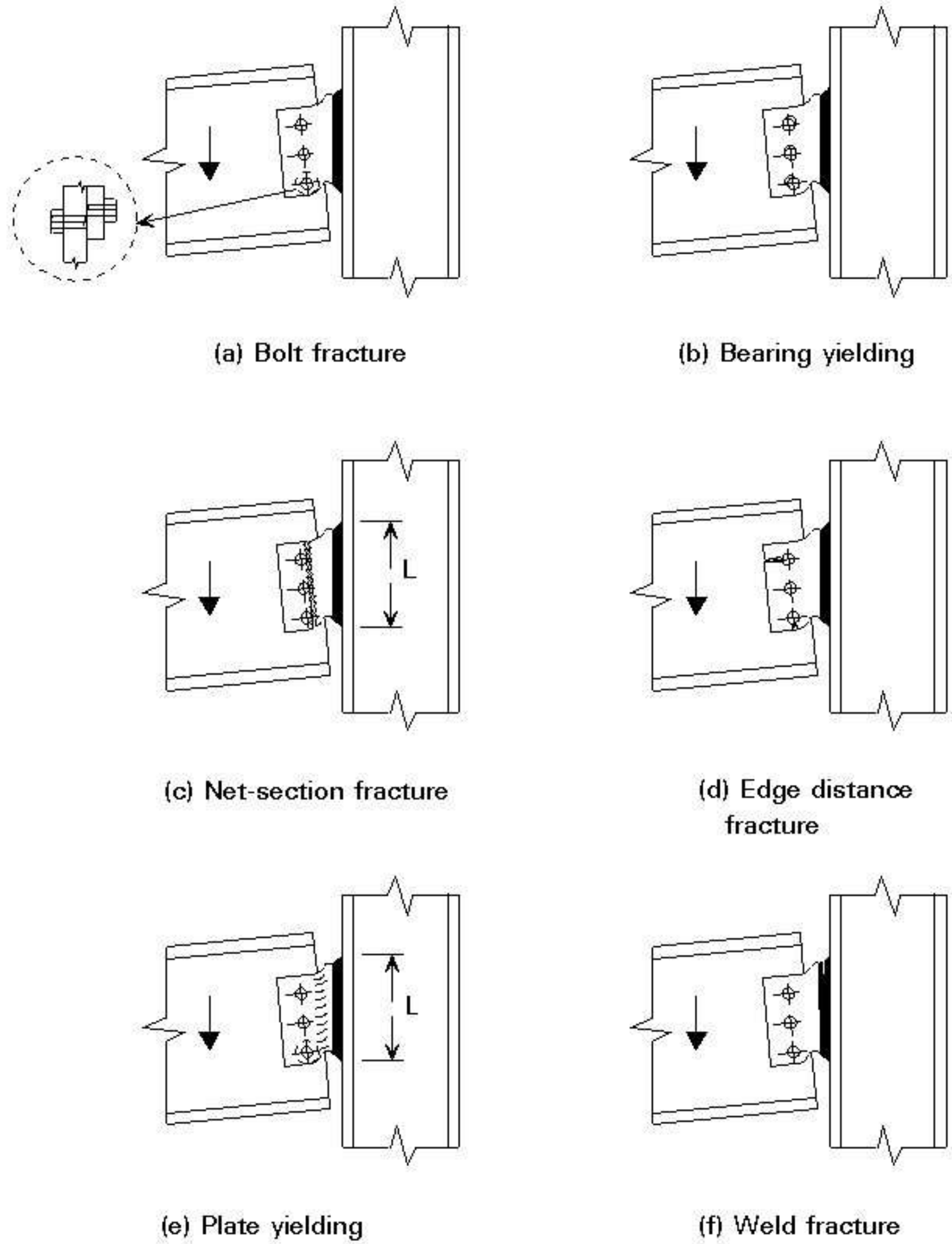


FIG. 9 MODE OF FAILURE FOR FIN PLATES

COLUMN SPLICES

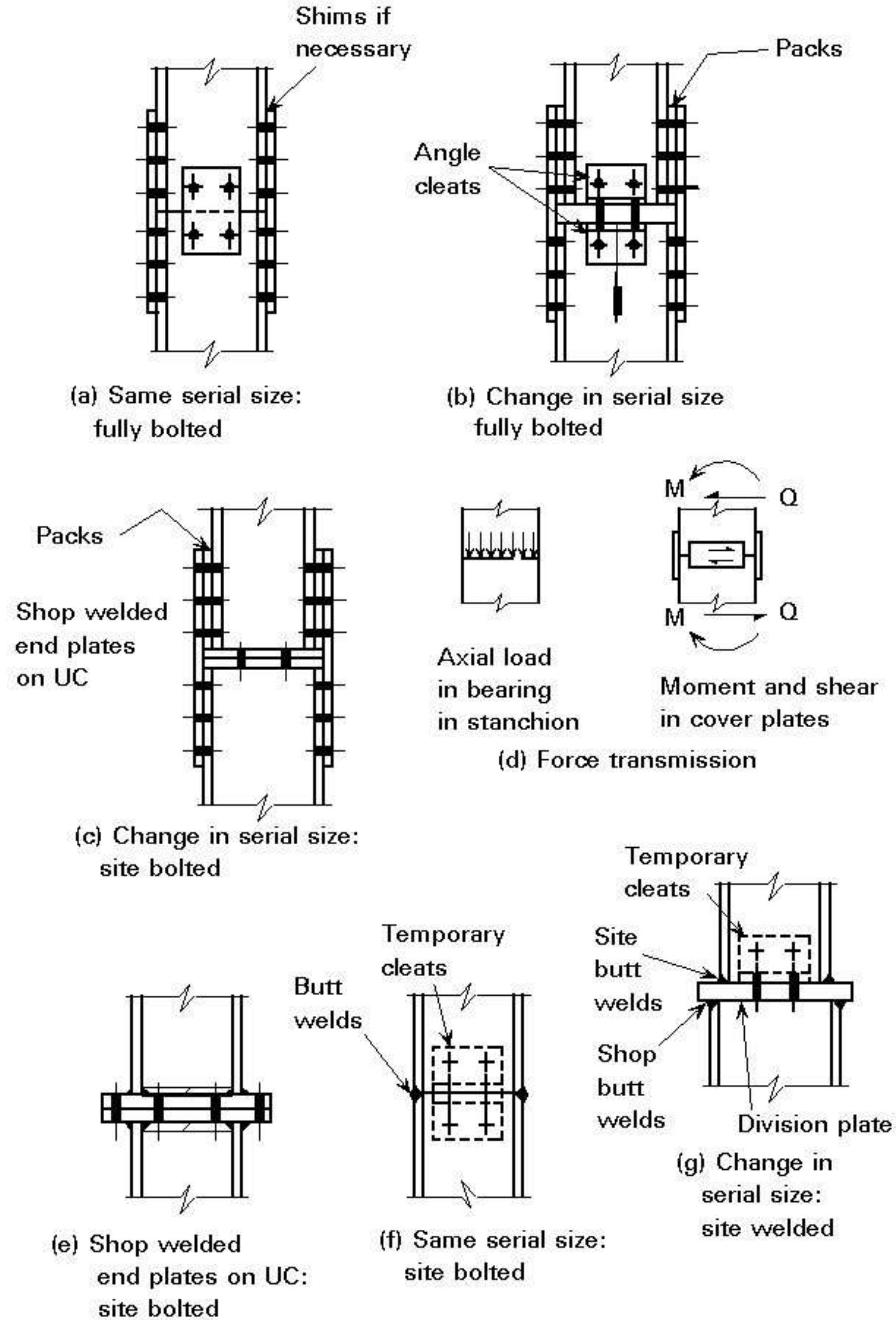


FIG. 10a COLUMN SPLICES

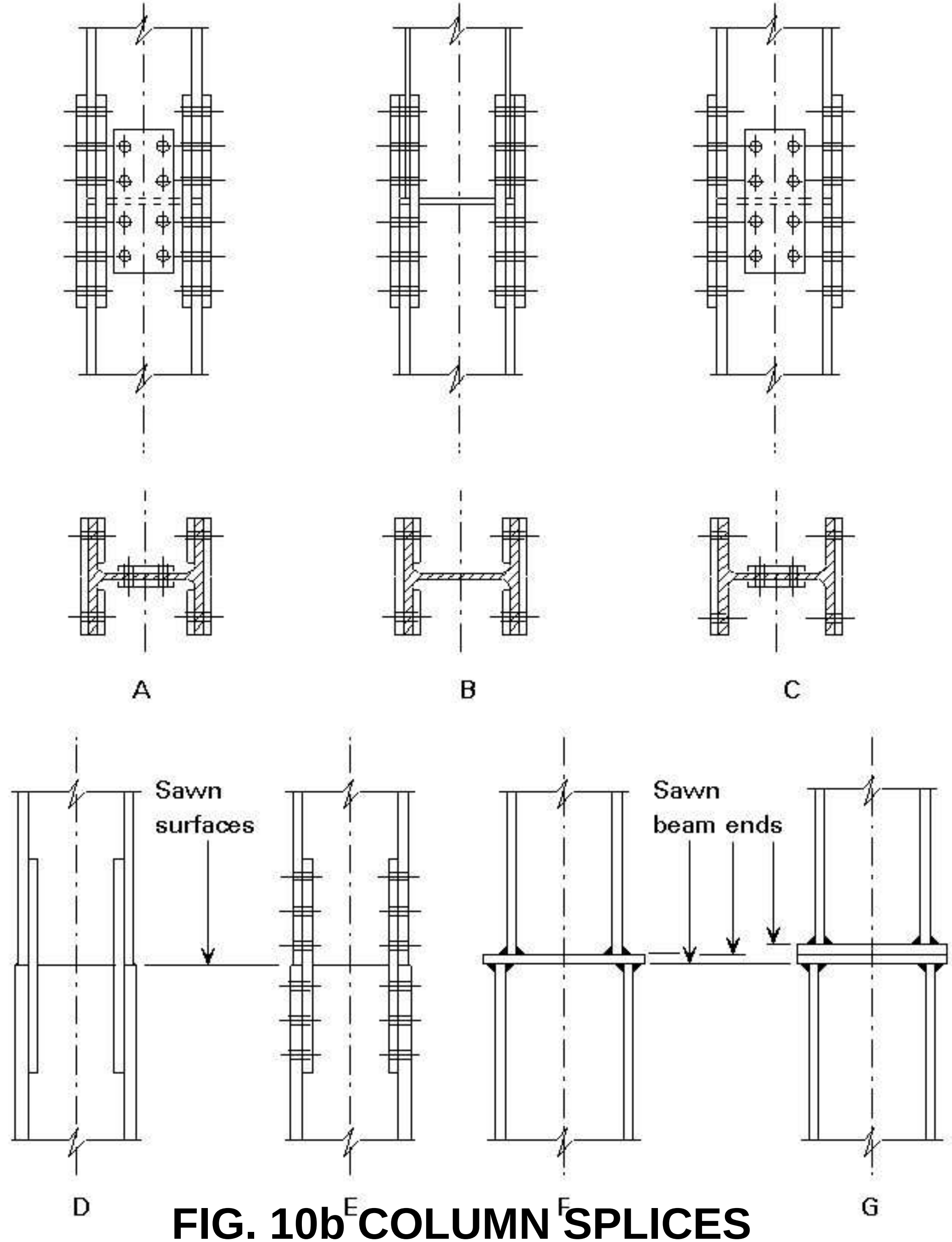


FIG. 10b COLUMN SPLICES

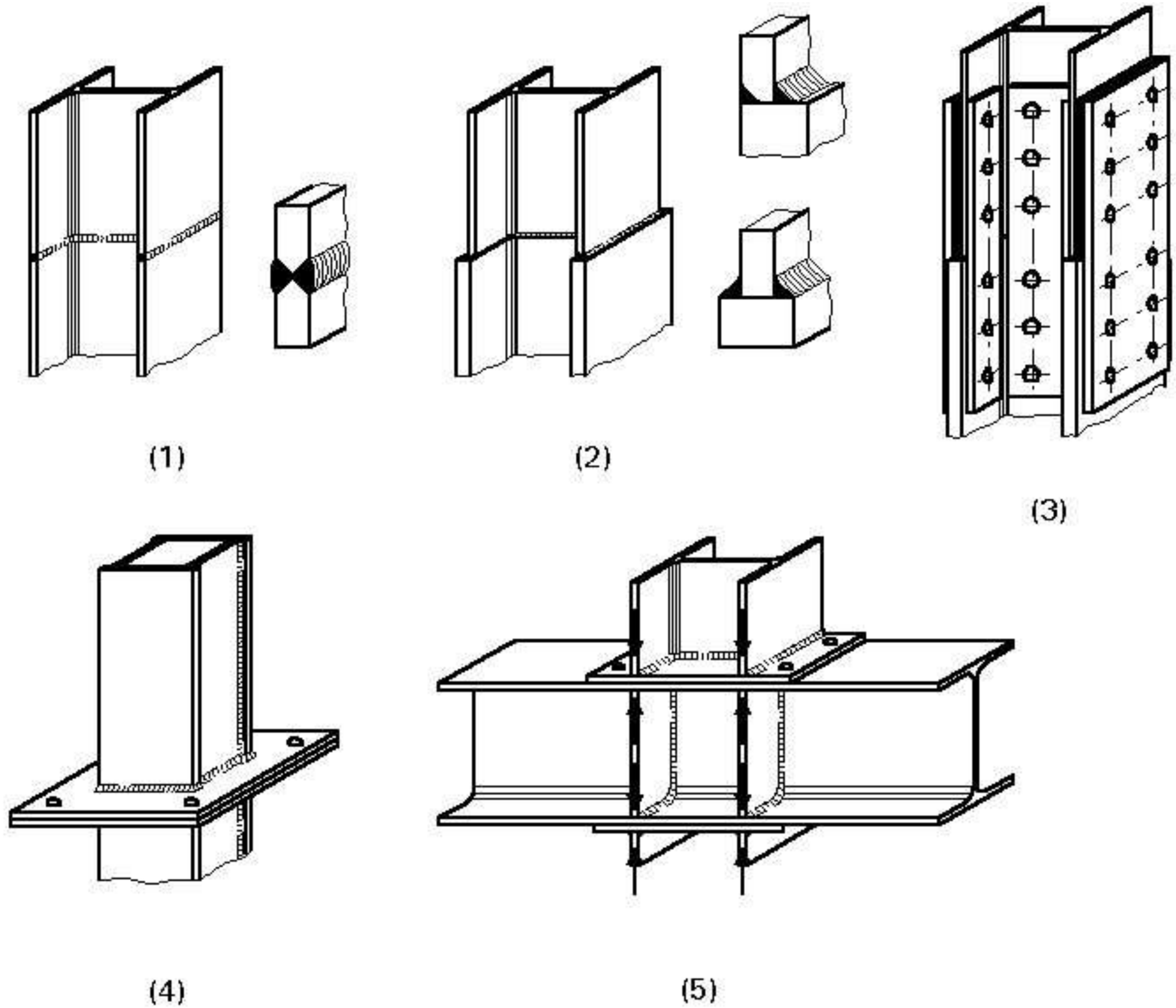


FIG. 10c COLUMN SPLICES

BRACING CONNECTIONS

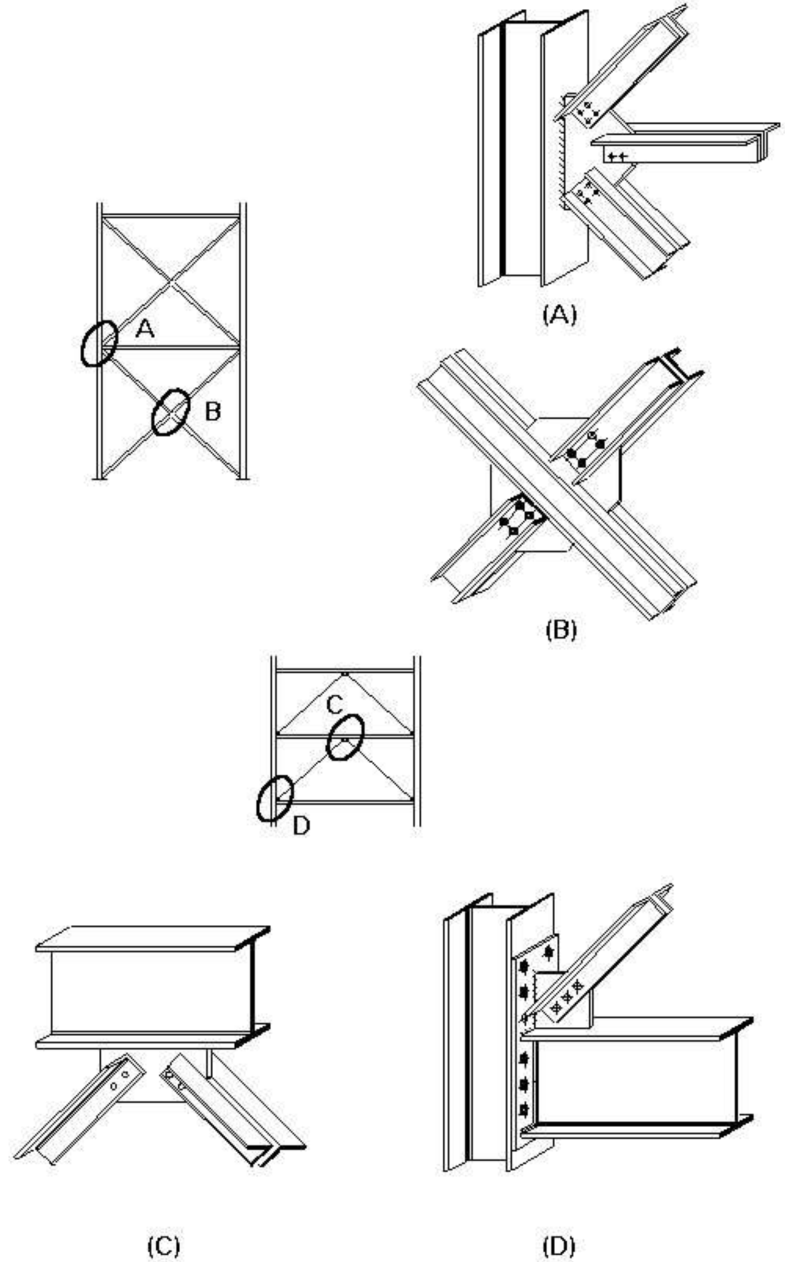


FIG. 11a BRACING CONNECTIONS

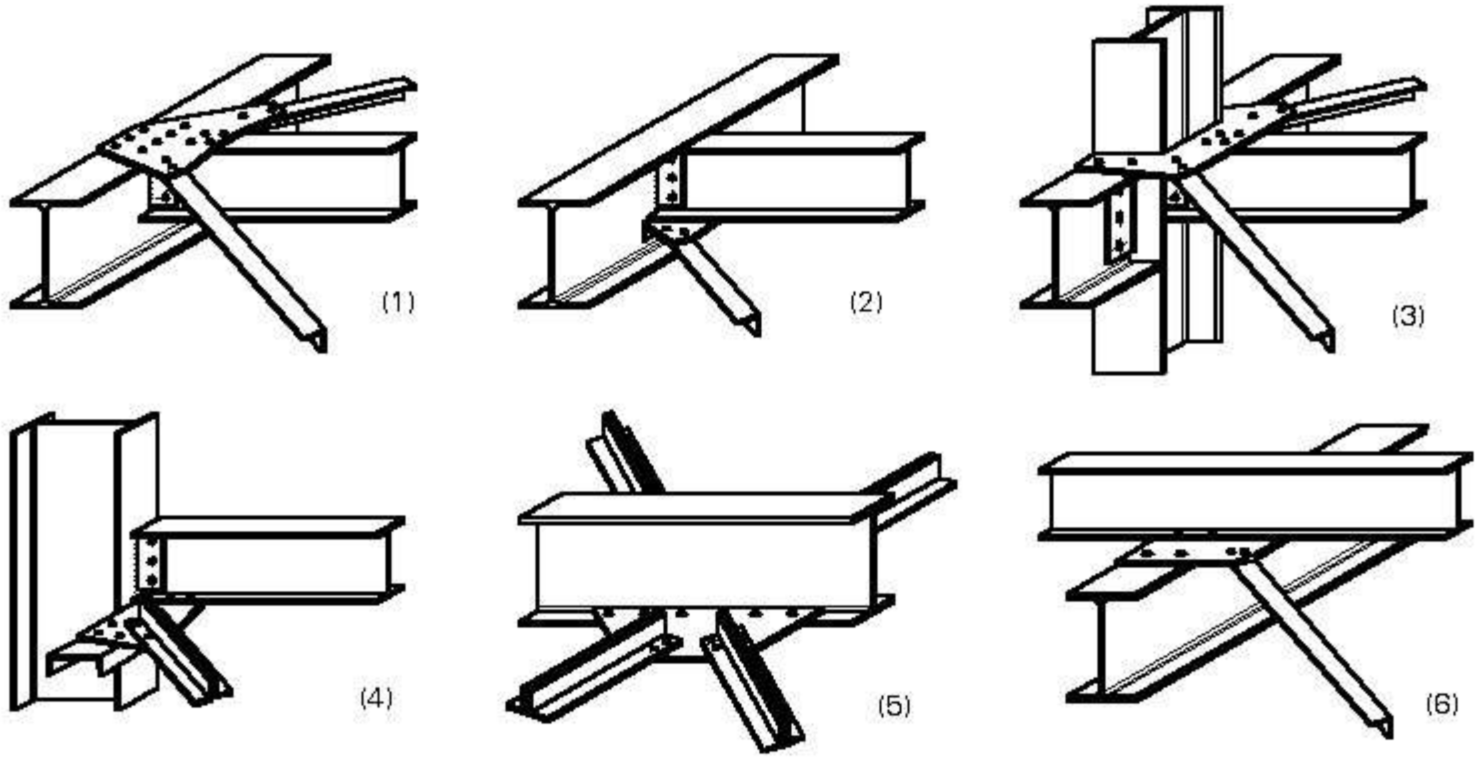


FIG. 11b HORIZONTAL BRACING CONNECTIONS

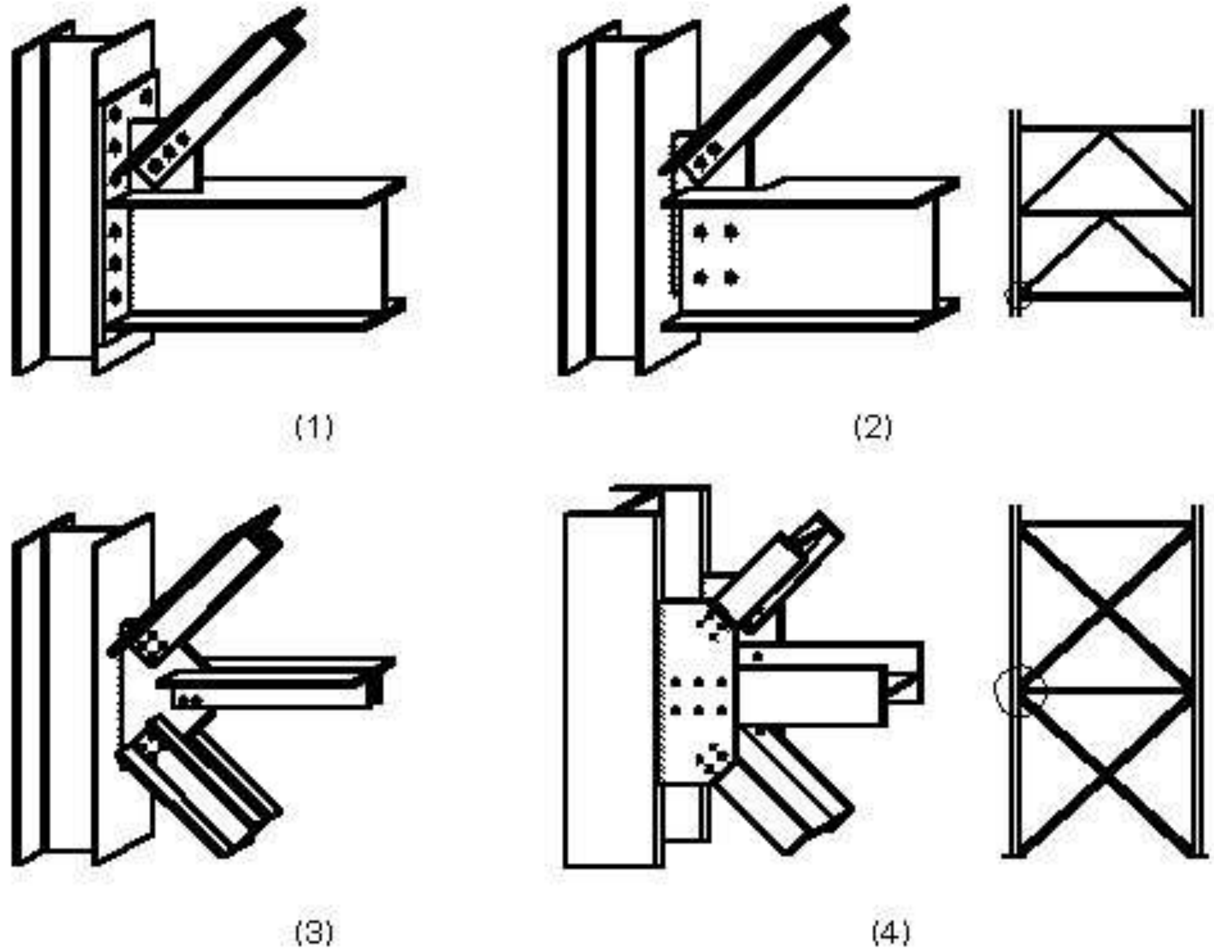


FIG. 11c VERTICAL BRACING CONNECTIONS

COLUMN BASES

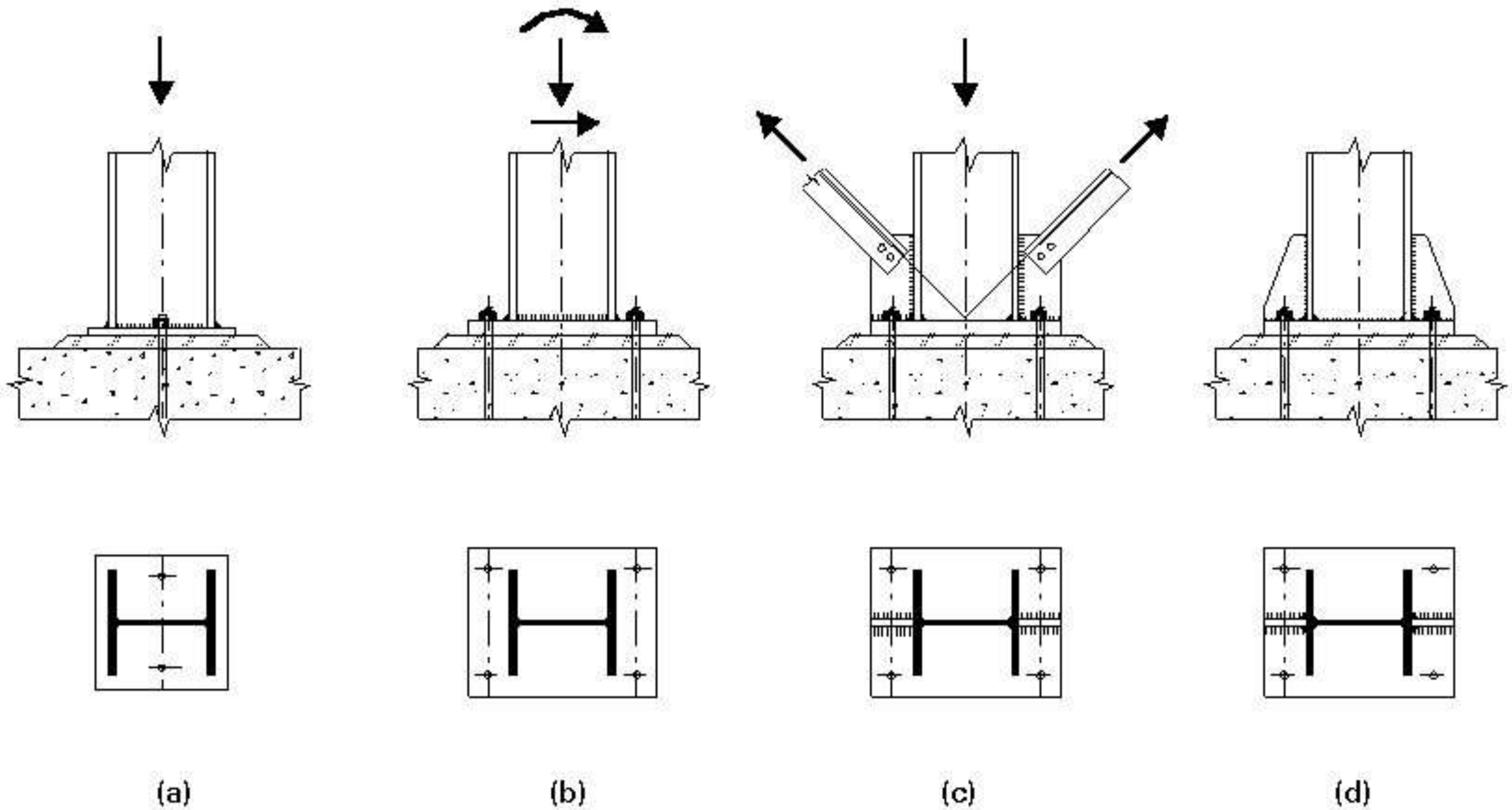


FIG. 12a COLUMN BASES

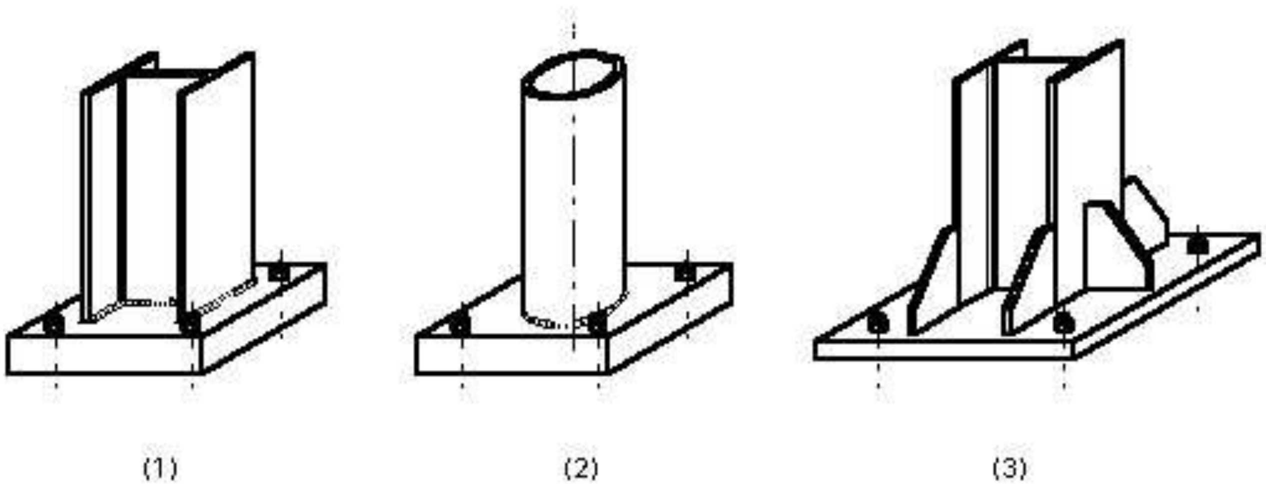


FIG. 12b COLUMN BASES

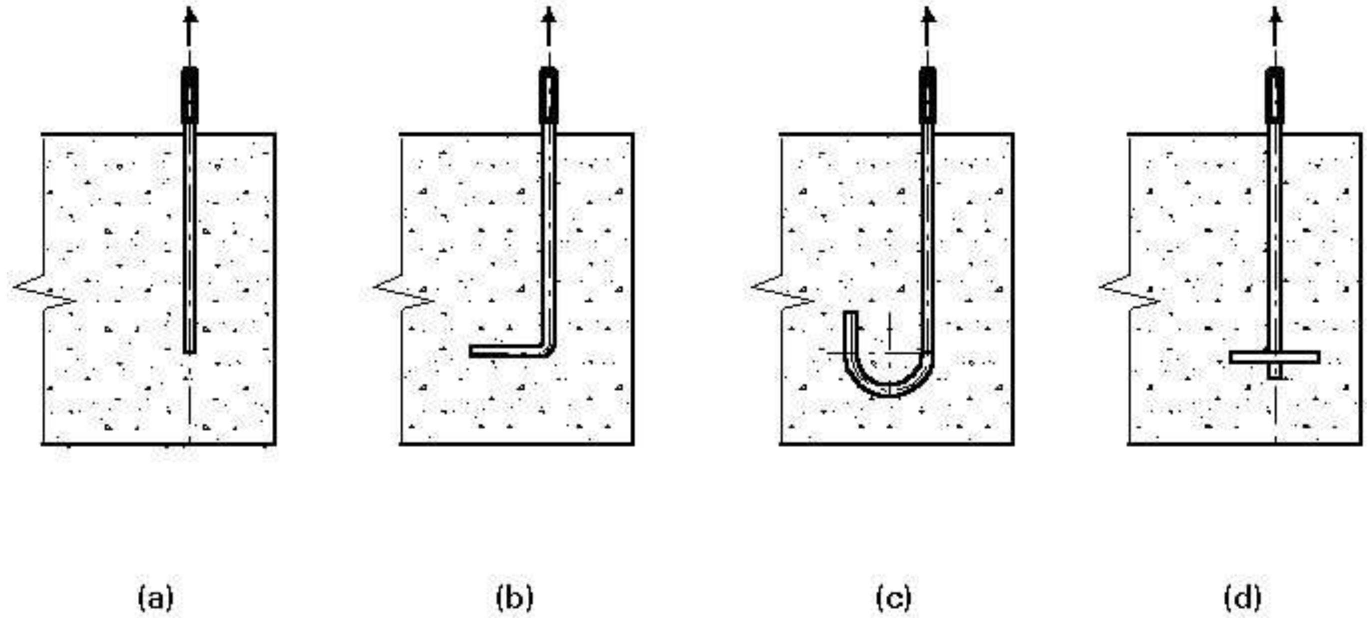


FIG. 12c ANCHORAGES OF HOLDING DOWN BOLTS

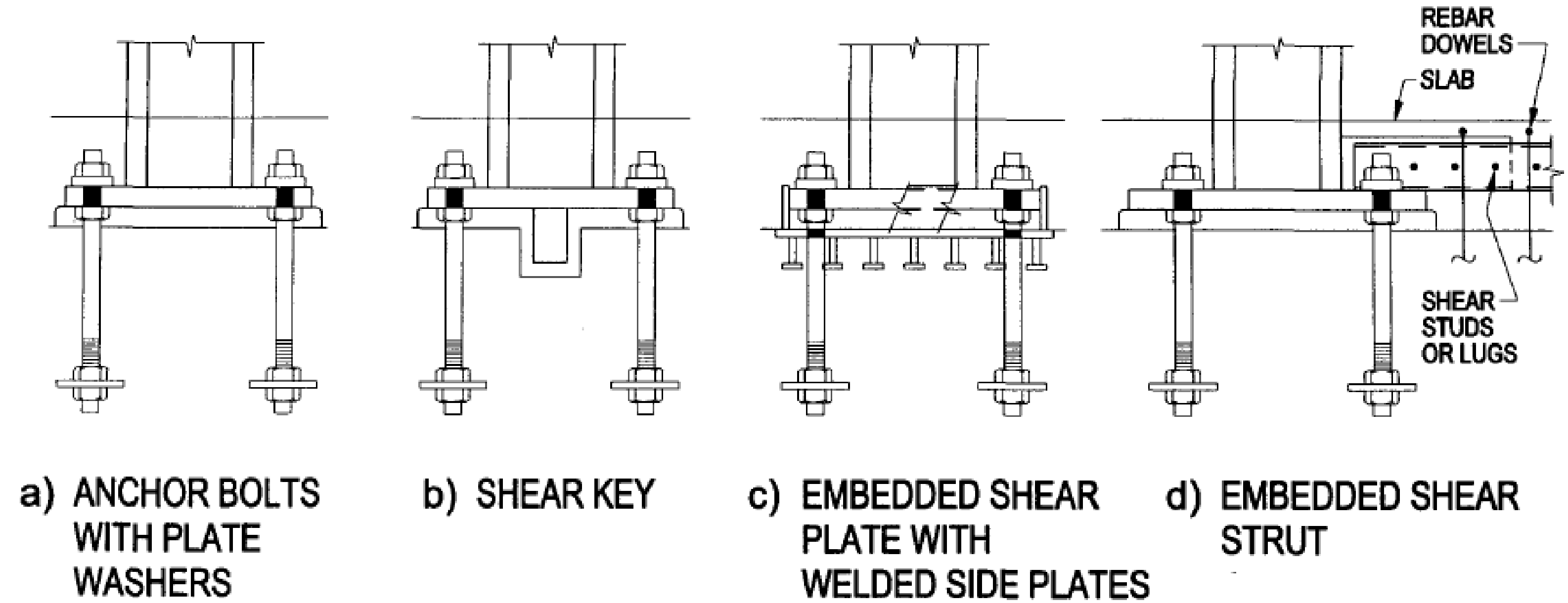
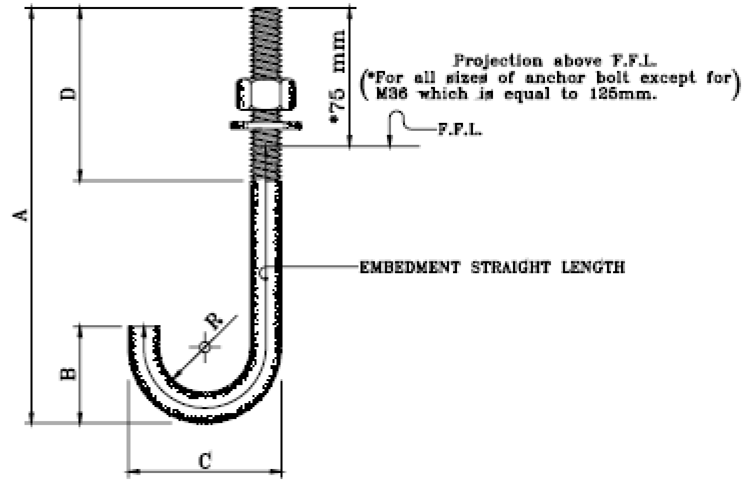


FIG. 12d ANCHORAGES OF HOLDING DOWN BOLTS



DIMENSIONAL PROPERTIES

BOLT NOMINAL DIAMETER	BOLT THREAD PITCH	WEIGHT (Kg)	A (mm)	B (mm)	C (mm)	D (mm)	RADIUS "R" (mm)	TOTAL STRAIGHT LENGTH (mm)	EMBEDMENT STRAIGHT LENGTH (mm)
M16	2.00	0.80	400	90	80	100	24	511	436
M20	2.50	1.56	500	110	100	125	30	636	561
M24	3.00	2.73	600	140	128	125	40	775	700
M30	3.50	6.15	900	170	160	150	50	1114	1039
M36	4.00	10.04	1000	210	192	200	60	1263	1138

ALLOWABLE LOADS

BOLT NOMINAL DIAMETER	TENSION (kN)	SHEAR (kN)	PULL-OUT STRENGTH (kN)
M16	26.54	13.67	30.21
M20	41.47	21.36	40.55
M24	59.72	30.76	50.65
M30	93.31	48.07	75.15
M36	134.36	69.22	82.3

BEAM-TO-CONCRETE WALL CONNECTIONS

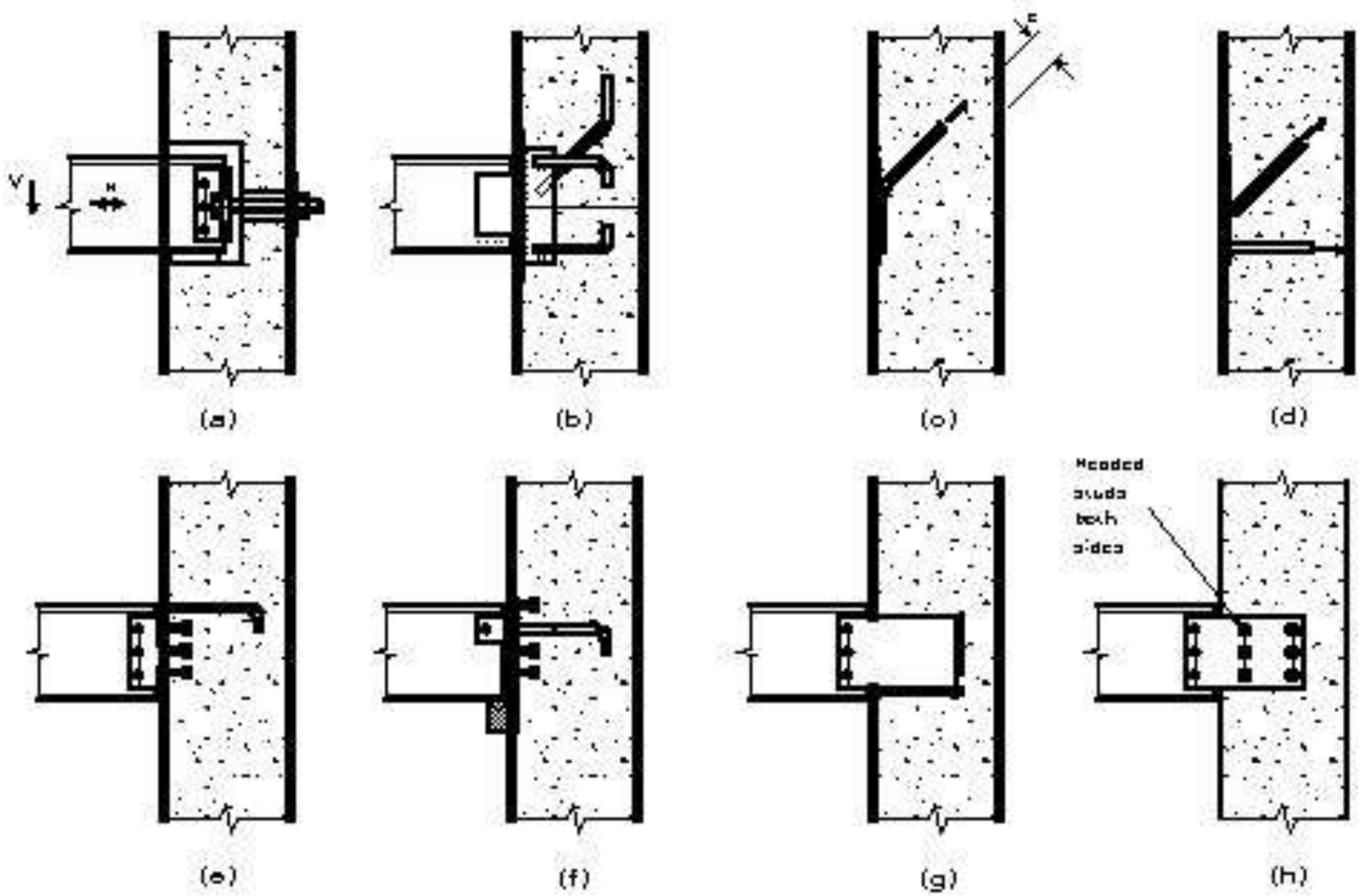


FIG. 13 BEAM TO CONCRETE WALL CONNECTIONS

Moment Connections

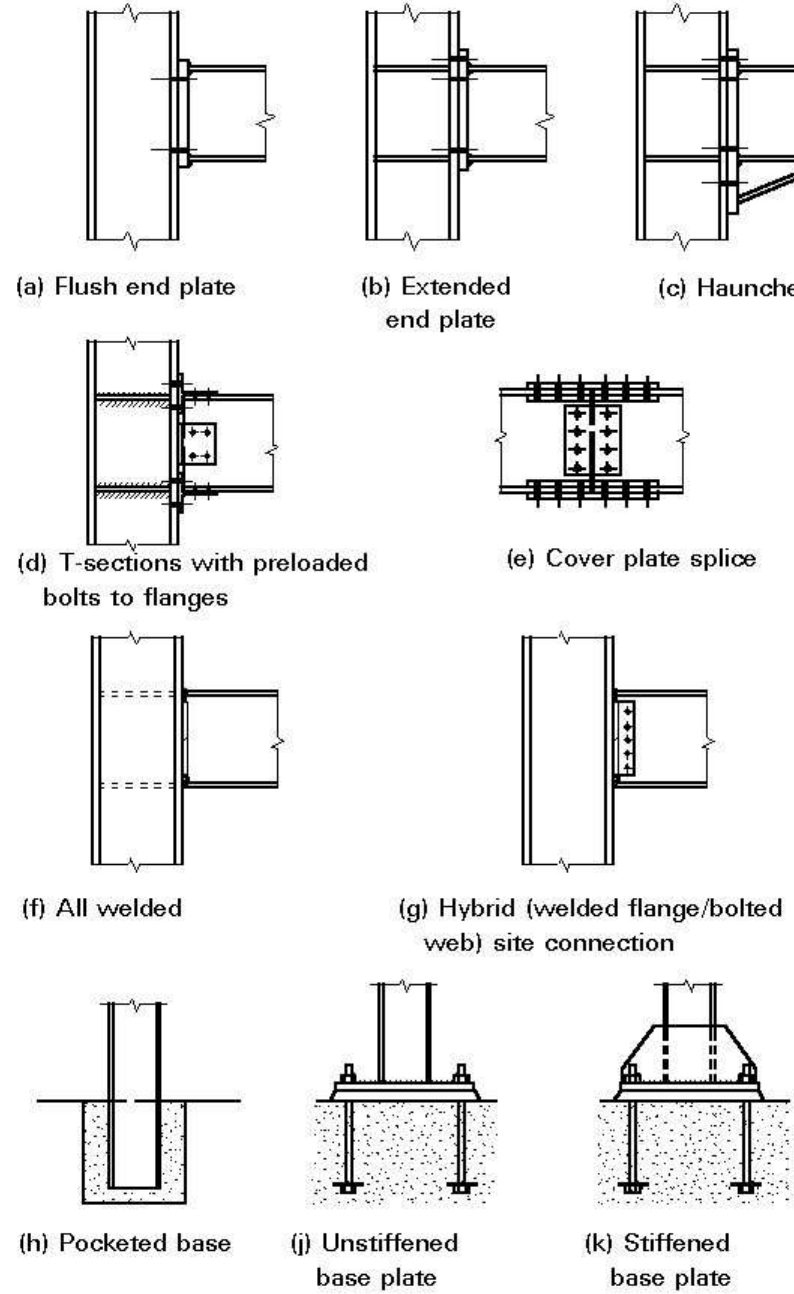
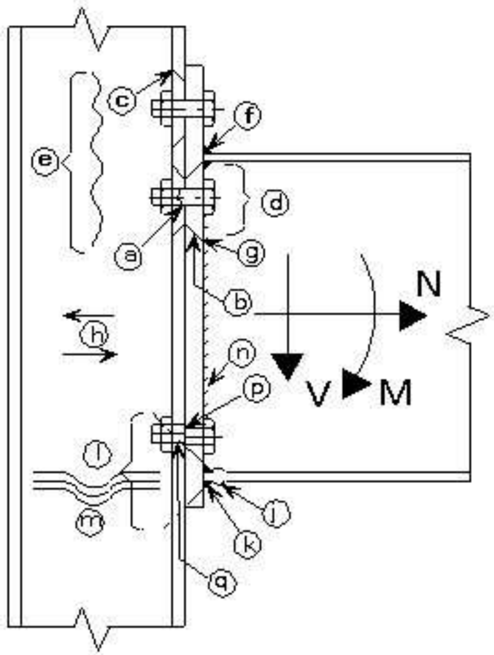
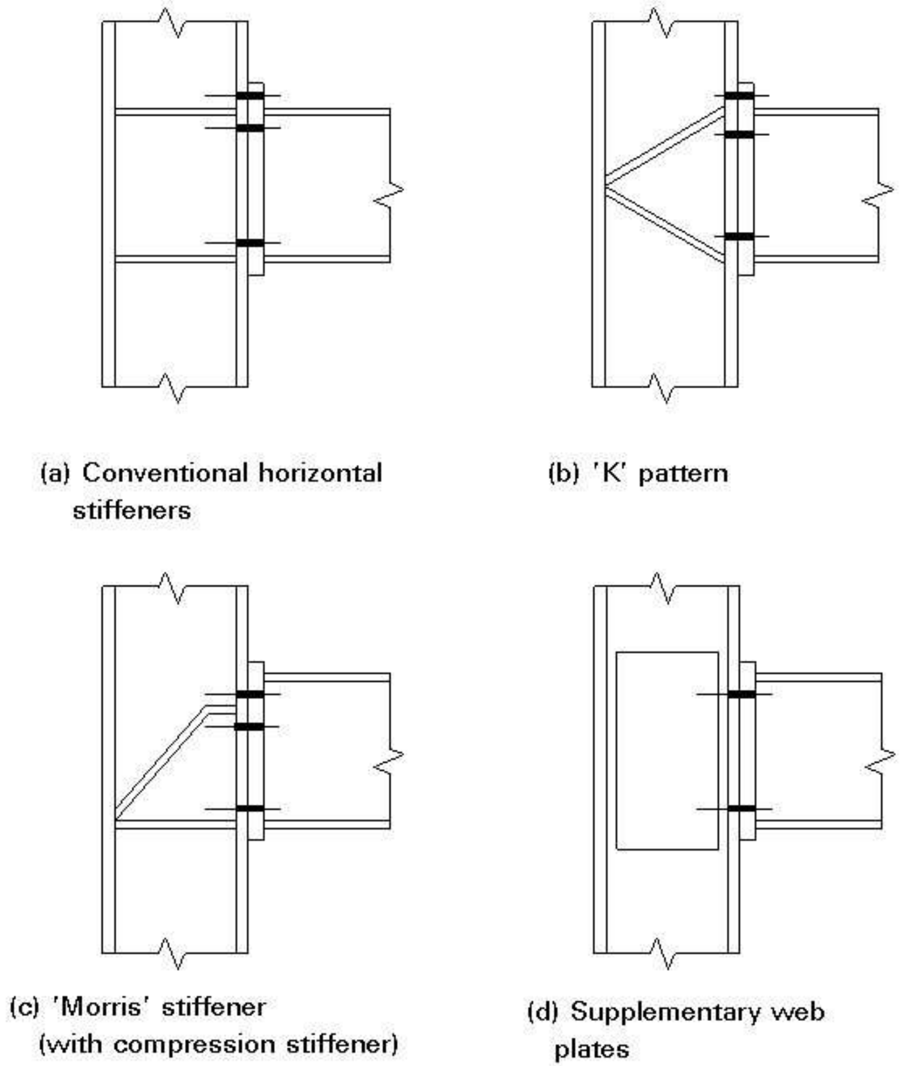


FIG. 14 TYPICAL MOMENT CONNECTIONS



Zone	Ref	Checklist item
Tension	a b c d e f g	Bolt tension End plate bending Column flange bending Beam web tension Column web tension Flange to end plate weld Web to end plate weld
Horizontal shear	h	Column web shear
Compression	j k l m	Beam flange compression Beam flange weld Column web bearing Column web buckling
Vertical shear	n p q	Web to end plate weld Bolt shear Bolt bearing

FIG. 15 CRITICAL COMPONENETS IN MOMENT CONNECTIONS



(a) Conventional horizontal stiffeners

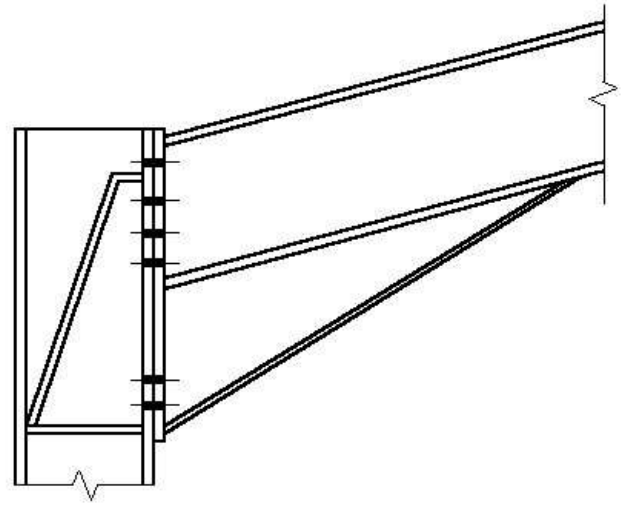
(b) 'K' pattern

(c) 'Morris' stiffener (with compression stiffener)

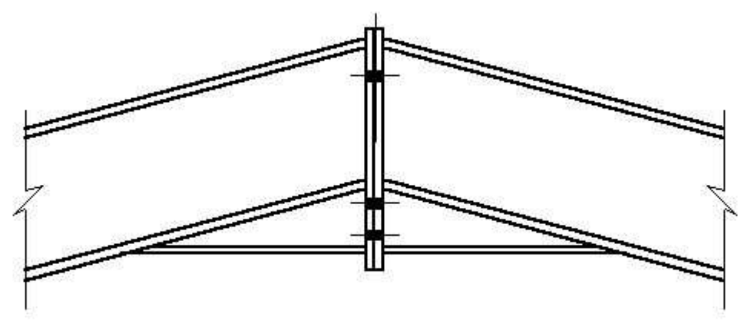
(d) Supplementary web plates

FIG. 16 STIFFENING / STRENGTHENING POSSIBILITIES

PITCHED-ROOF PORTAL FRAME

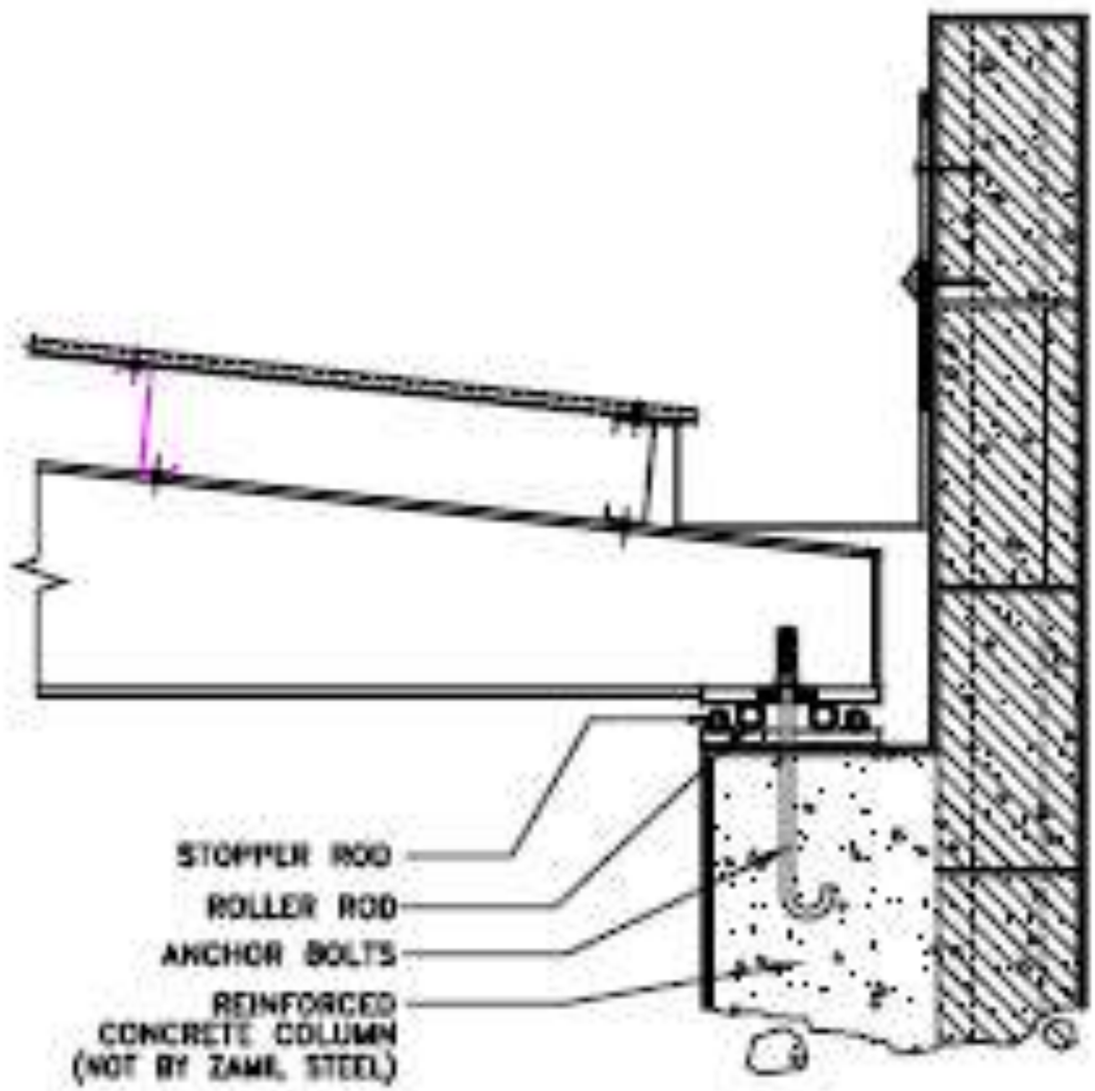


(a) Eaves connection

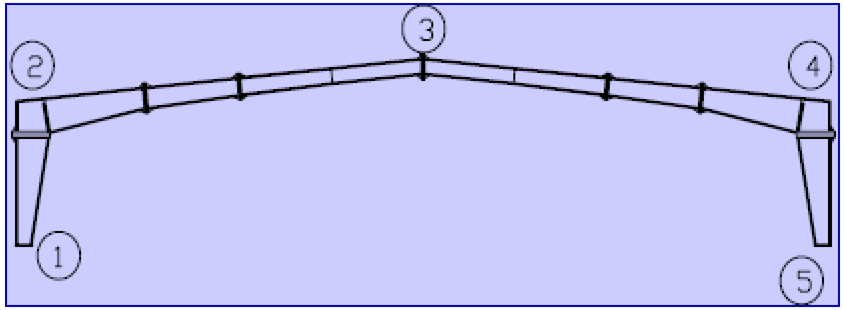


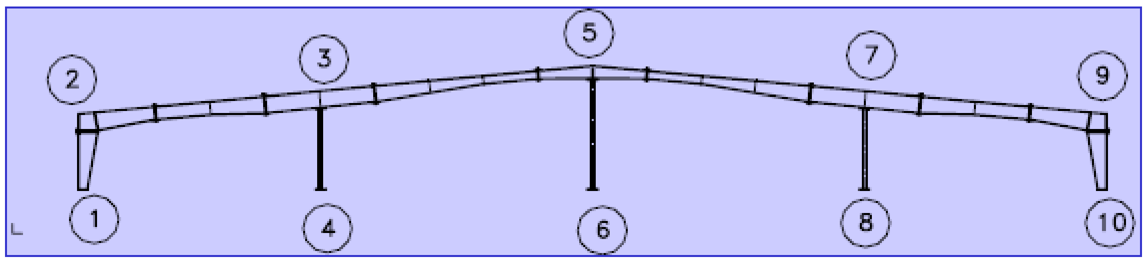
(b) Apex connection

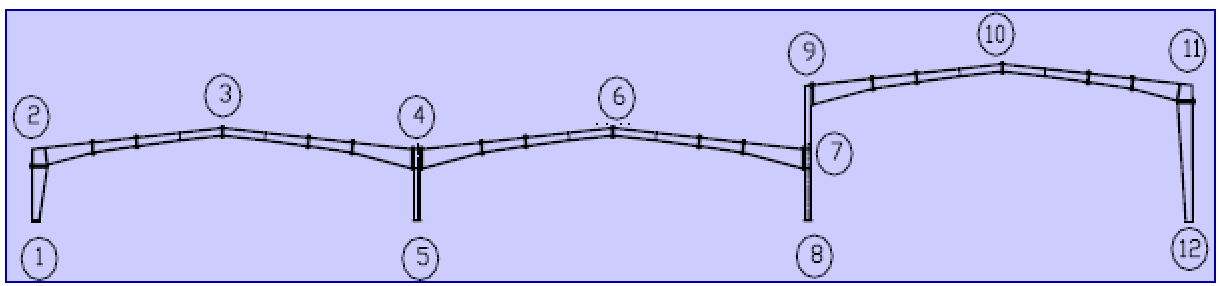
FIG. 17 PORTAL FRAME CONNECTIONS



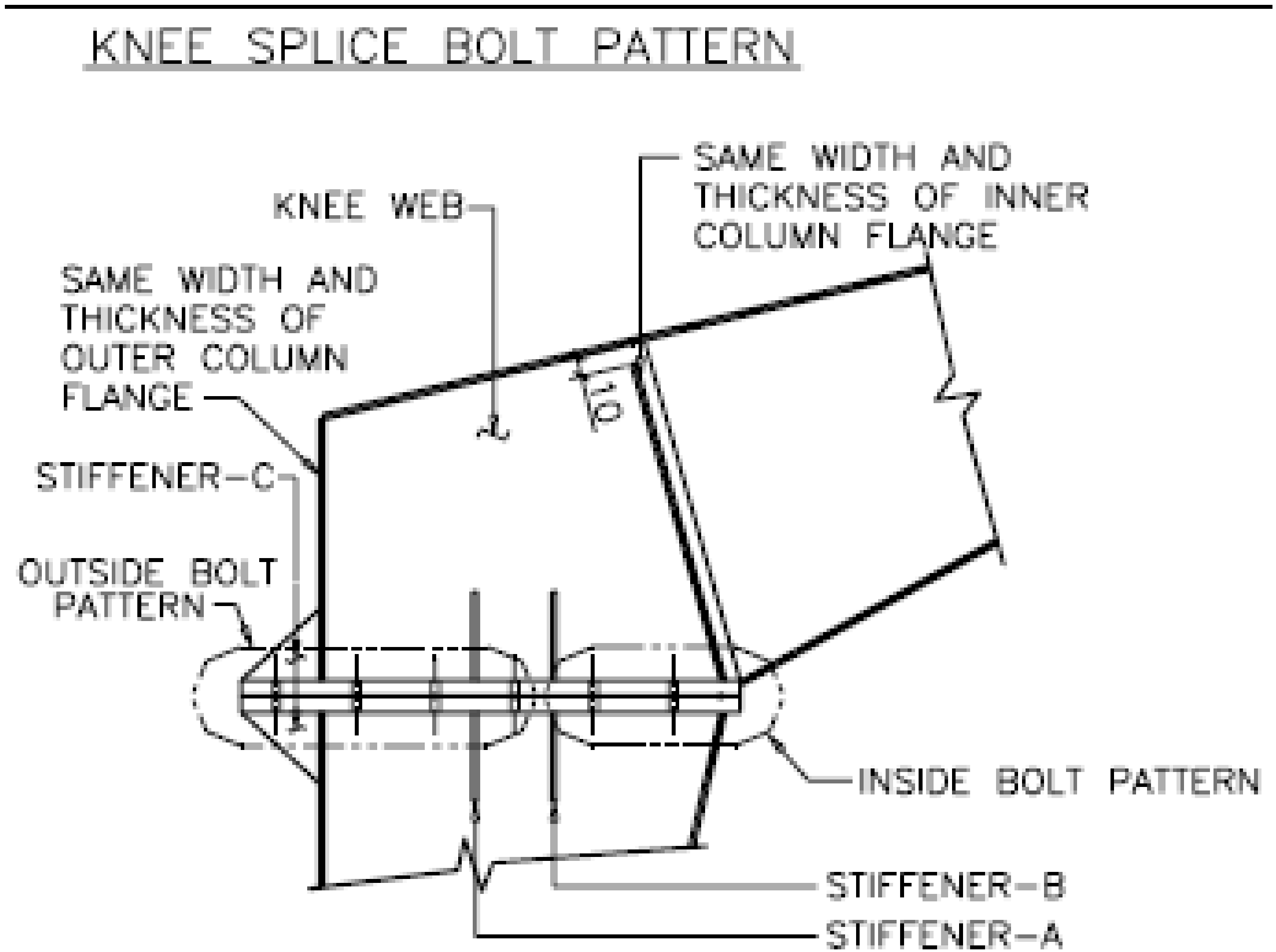
TYPICAL ROLLER ARRANGEMENT



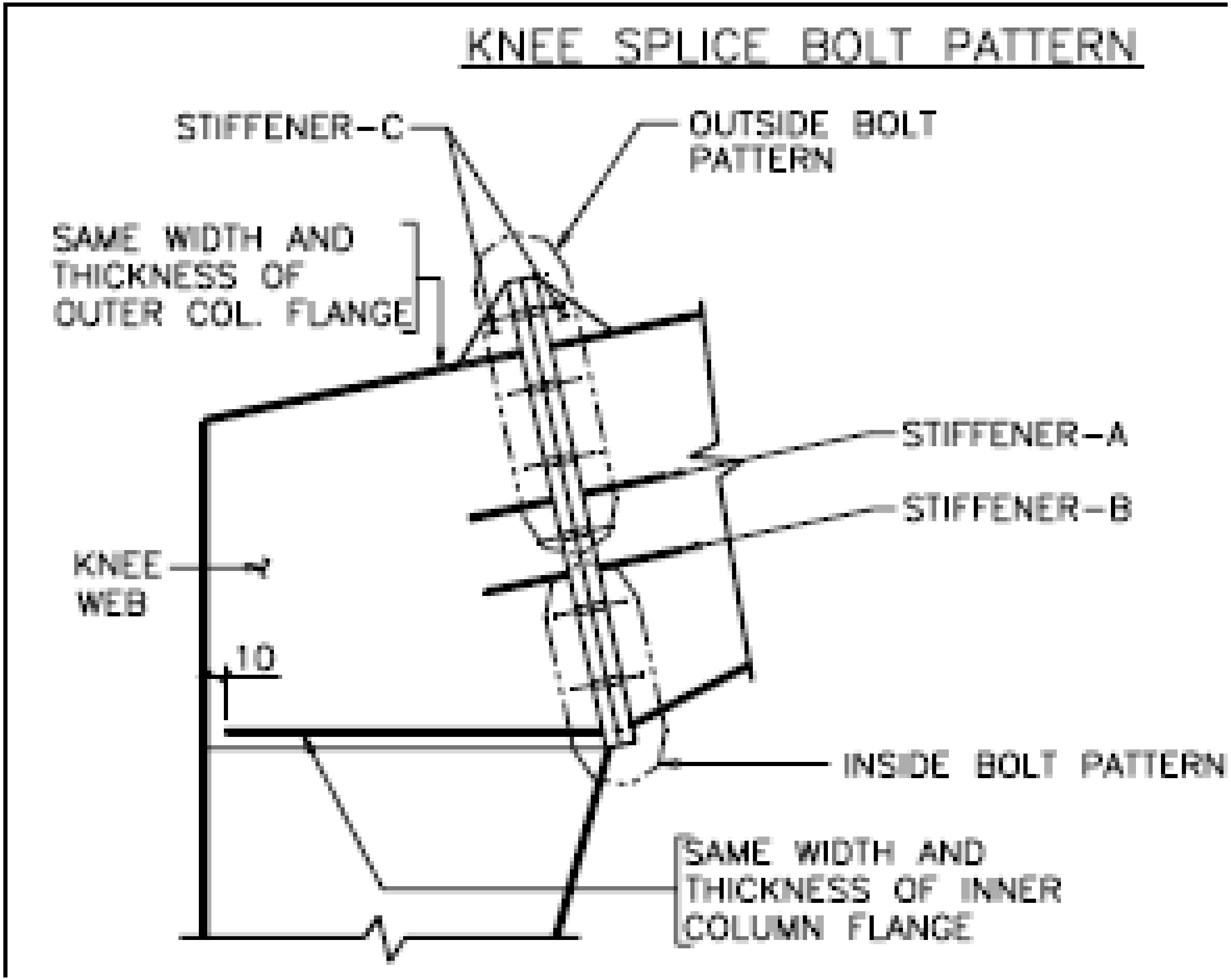




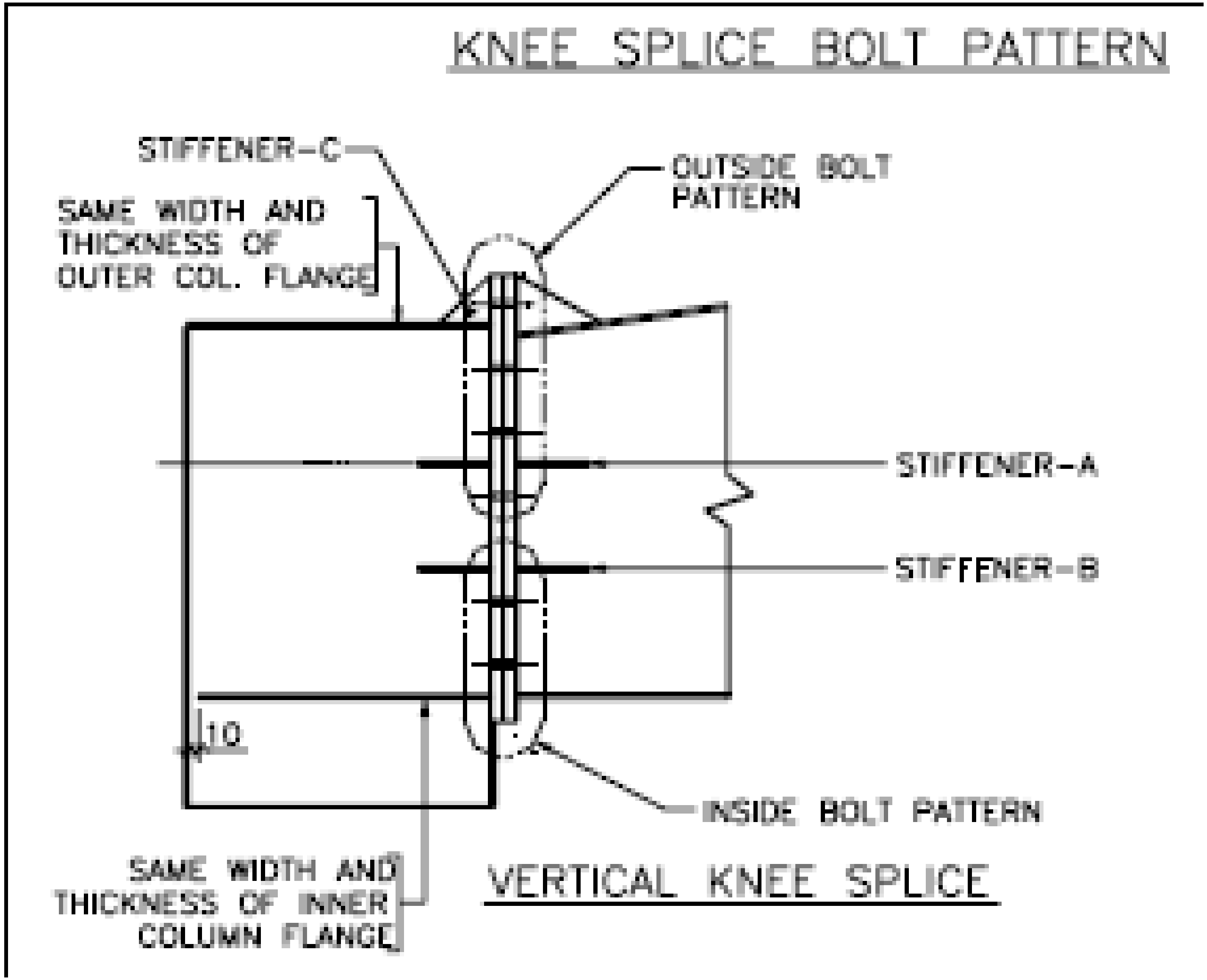
CABLE FRAMES



Horizontal Knee Connection Details

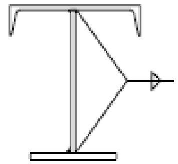


Vertical Knee Connection Details

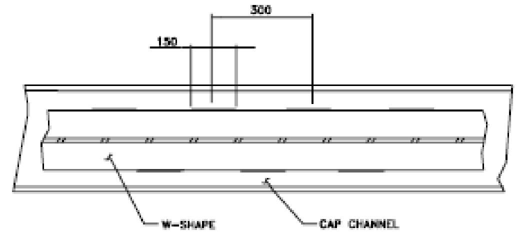


Vertical Knee Connection (straight column)

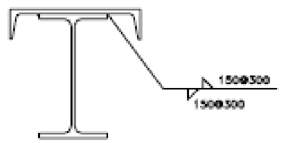
Crane Beam



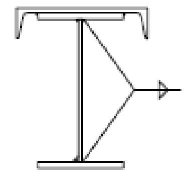
BUILT-UP SECTION
W/OUT TOP FLANGE



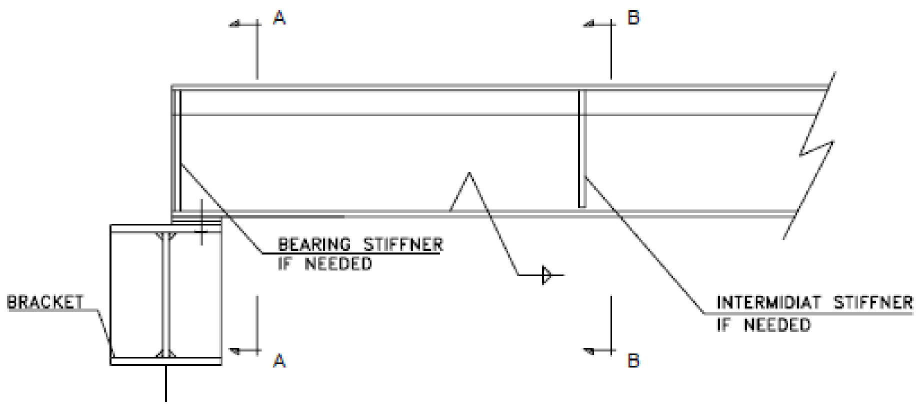
CRANE BEAM CAP CHANNEL

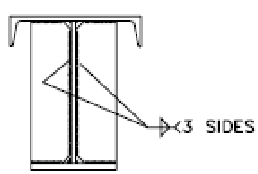


HOT ROLLED SECTION

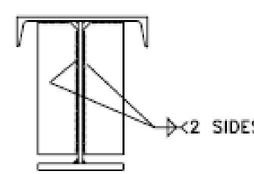


BUILT-UP SECTION
W/ TOP FLANGE

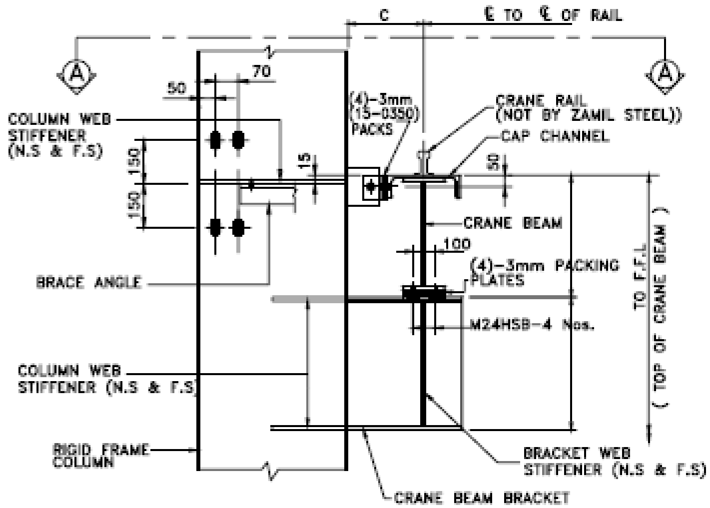




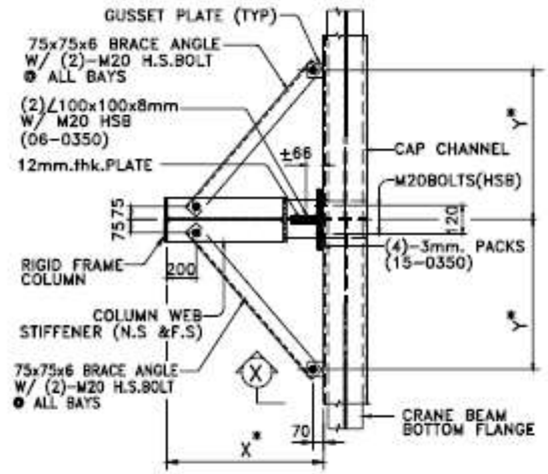
SEC. A-A



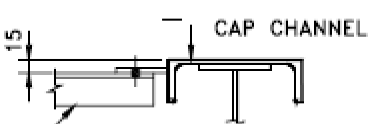
SEC. B-B



CRANE BEAM/BRACKET ARRANGEMENT

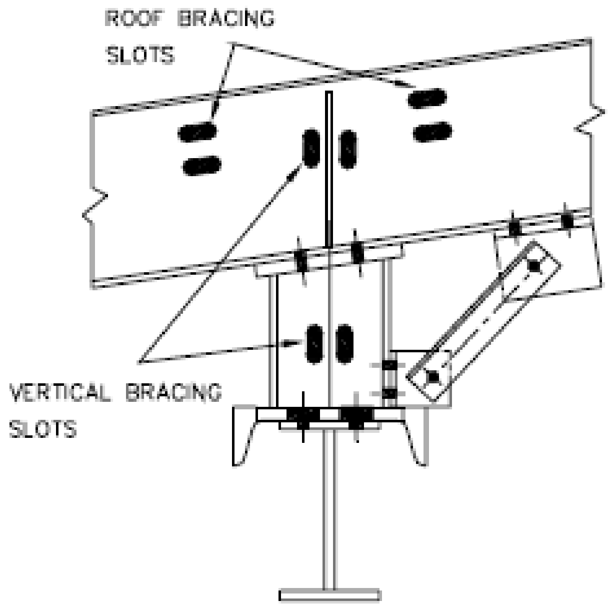


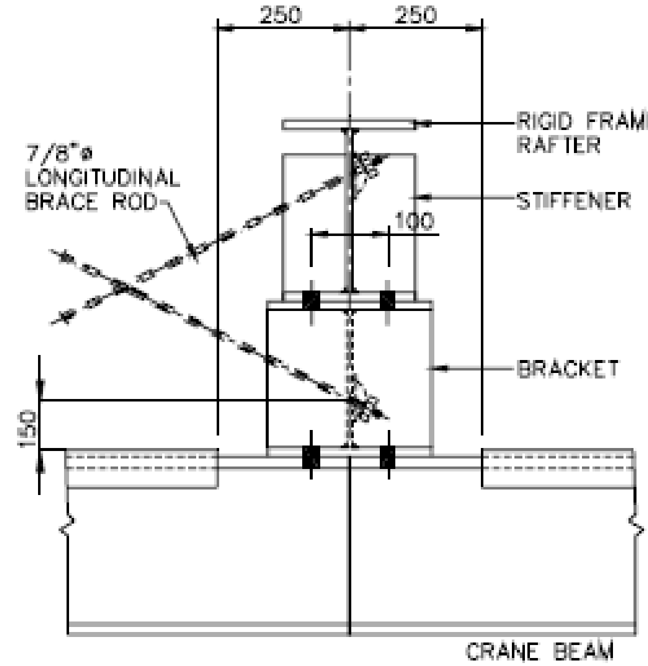
SECTION "A-A"



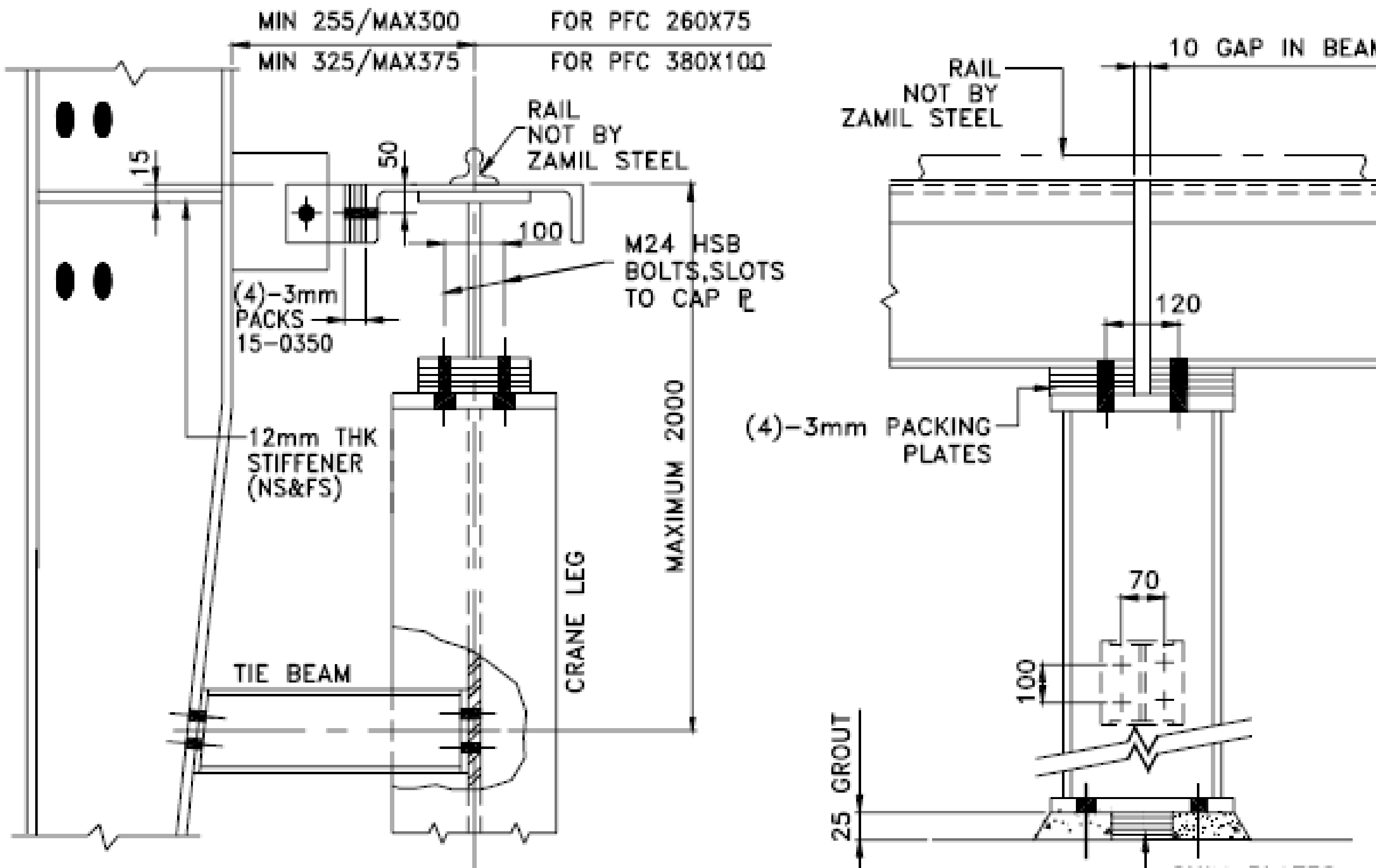
SECTION - X

TOP RUNNING CRANE BRACING DETAILS

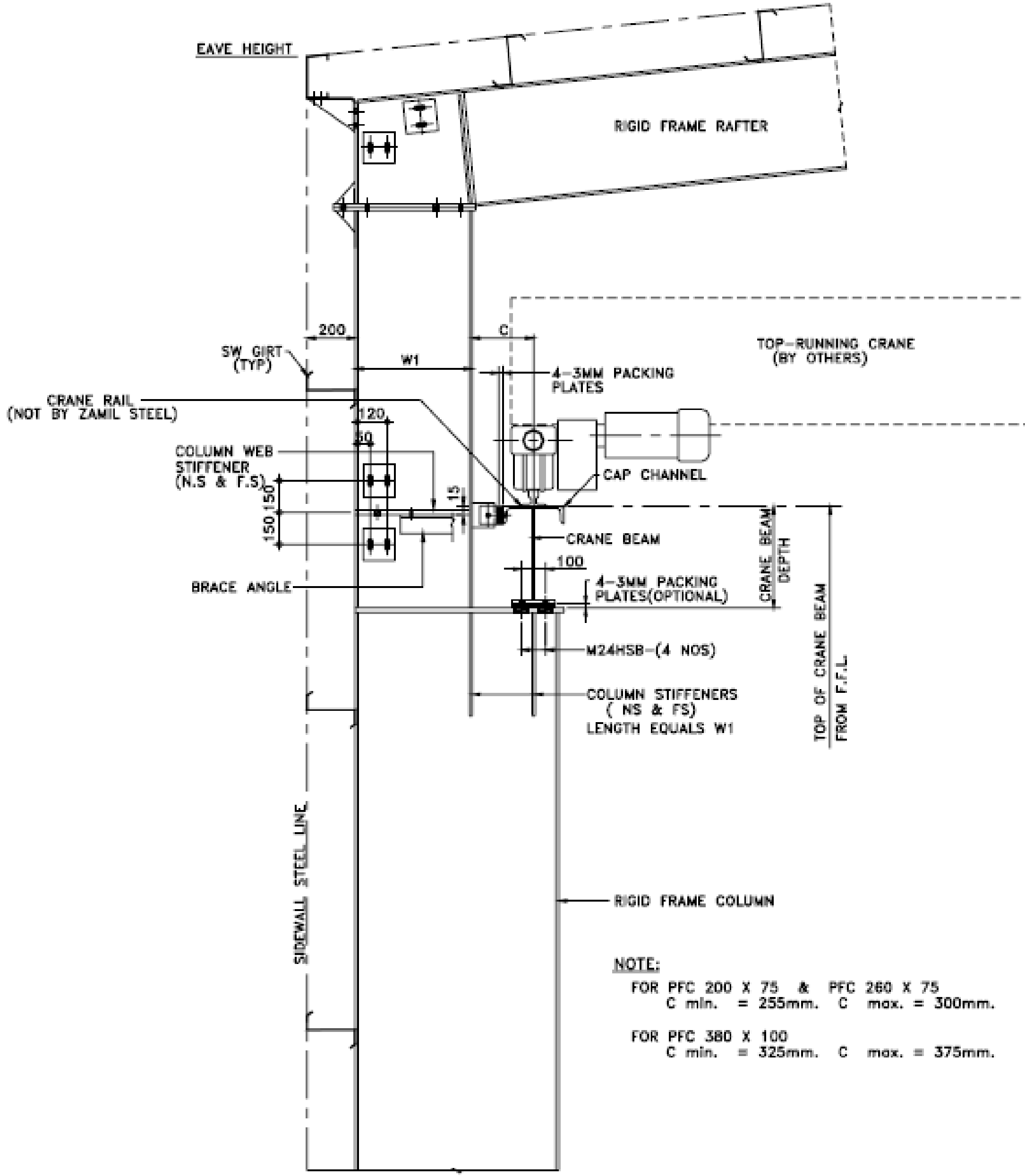




UNDERHUNG CRANE BRACING



INDEPENDENT CRANE COLUMN



TYPICAL STEPPED COLUMN / CRANE BEAM DETAILS

IS 800 -2007
CODAL PROVISIONS

SECTION 10 CONNECTIONS - Contents

GENERAL OBJECTIVES AND CONTENTS

To enable the designer to complete the design without the need to refer several other codes for simple values

- strengths of bearing and friction grip bolts
- strengths of welds for various fusion angles
- guidelines for design of splices, connections etc
- Analysis of bolt/weld groups kept simple
- guidelines for design of semi-rigid connections

10.2 Fasteners spacing and edge distance

10.2.1 *Minimum Spacing* - **2.5** times the nominal diameter

10.2.2 *Maximum Spacing* - shall not exceed **32*t* or 300 mm**, whichever is less, where *t* is thickness of the thinner plate

10.2.2.2 pitch shall not exceed **16*t* or 200 mm**, in tension members and **12*t* or 200 mm**, whichever is less, in compression members

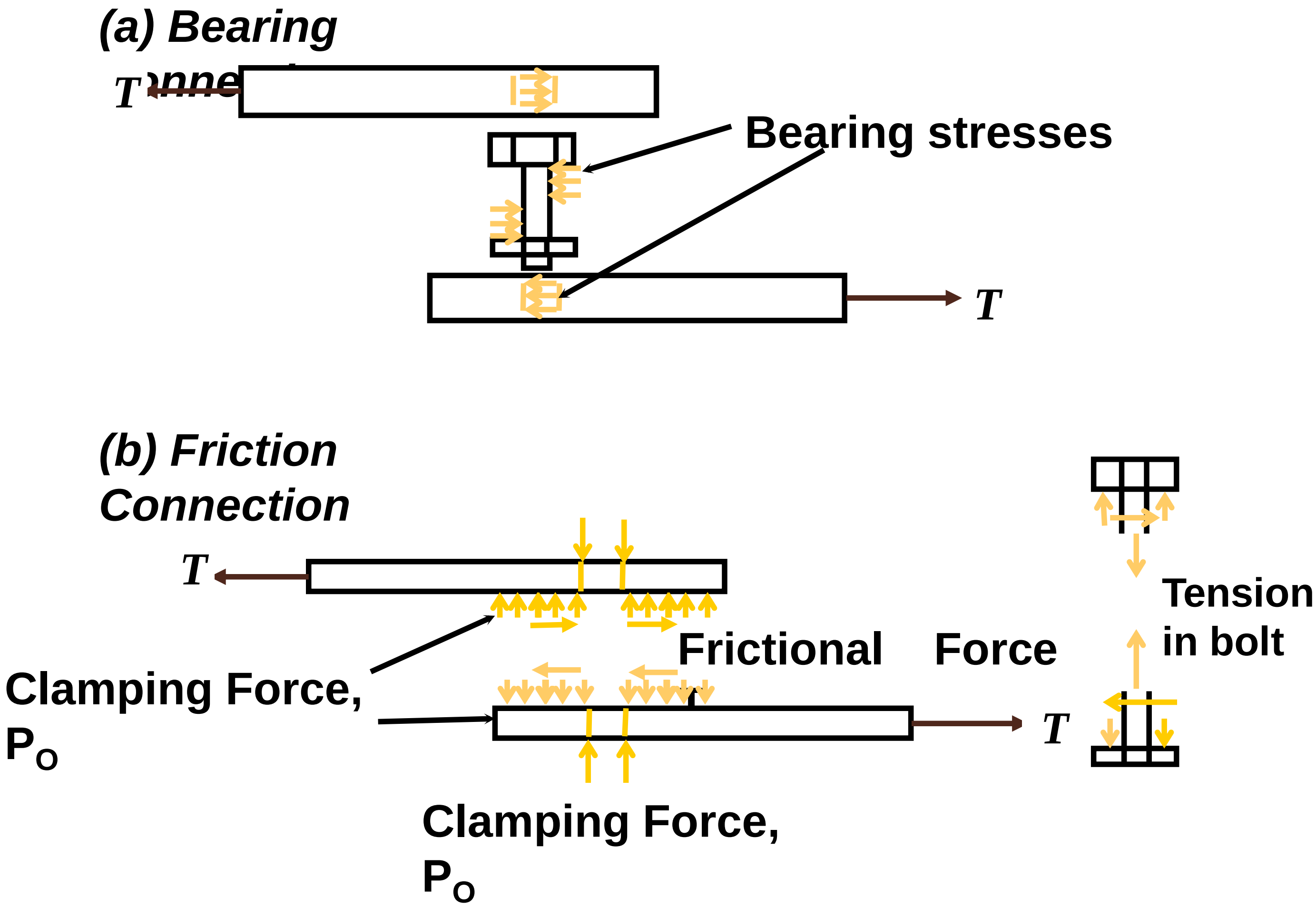
10.2.3 E*dge and End Distances* minimum edge shall be not less than that given in Table 10.1. maximum edge distance should not exceed **12 $t\varepsilon$**, where $\varepsilon = (250/f_y)^{1/2}$

10.2.4 *Tacking Fasteners* spacing in line not exceeding **32t or 300 mm** If exposed to the weather, **16 t or 200** mm

max. spacing in tension members **1000 mm**

max. spacing in compression members **600 mm**

FORCE TRANSFER MECHANISM

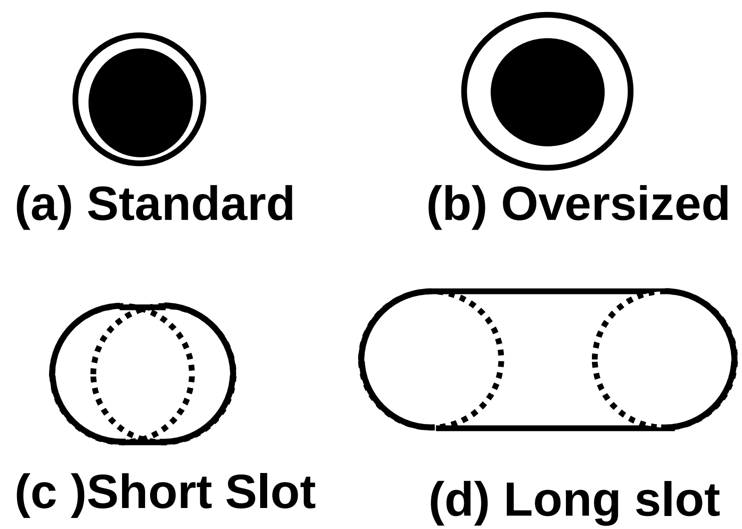


Bolt Shear Transfer – Free Body Diagram

Bolts and Bolting

Bolt Grade: Grade 4.6 :- f_u = 40 kgf/mm^2 and f_y = 0.6*40 = 24 kgf/mm^2

Bolt Types: Black, Turned & Fitted, High Strength Friction Grip



Hole types for HSFG bolts

Table 19 Clearances for Fastener Holes

(*Clause* 10.2.1)

Sl No.	Nominal Size of Fastener, *d* mm	Size of the Hole = Nominal Diameter of the Fastener + Clearances mm			
		Standard Clearance in Diameter and Width of Slot	Over Size Clearance in Diameter	Clearance in the Length of the Slot	
				Short Slot	Long Slot
(1)	(2)	(3)	(4)	(5)	(6)
i)	12 – 14	1.0	3.0	4.0	2.5 *d*
ii)	16 – 22	2.0	4.0	6.0	2.5 *d*
iii)	24	2.0	6.0	8.0	2.5 *d*
iv)	Larger than 24	3.0	8.0	10.0	2.5 *d*

IS 800 : 2007

10.2.3 *Maximum Spacing*

10.2.3.1 The distance between the centres of any two adjacent fasteners shall not exceed 32*t* or 300 mm, whichever is less, where *t* is the thickness of the thinner plate.

10.3 Bearing Type Bolts

$$V_{sb} = \frac{f_u}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)/\gamma_{mb}$$

10.3.1.1 Reduction factor in shear for Long Joints

$$\beta_{lj} = 1.075 - (l_j/200d)$$

$$\text{but} \quad 0.75 \le \beta_{lj} \le 1.0$$

10.3.1.2 Reduction factor in shear for Large Grip Lengths

$$\beta_{lg} = 8\,d/(3\,d + l_g)$$

10.3.2.3 Reduction factor for Packing Plates

$$\beta_{pk} = (1 - 0.0125\,t_{pk})$$

10.3 Bearing Type Bolts

10.3.3 Bearing Capacity of bolt on any ply

$$V_{sb} = (2.5\, d\, t\, f_u)/\, \gamma_{mb}$$

10.3.4 Tension Capacity

$$T_b = (0.90\, f_{ub}\, A_n)/\, \gamma_{mb} \quad < (f_{yb}\, A_{sb}\, (\gamma_{m1} / \gamma_{m0}))/\, \gamma_{mb}$$

10.3.5 Bolt subjected to combined shear and tension

$$\left(\frac{V}{V_{sd}}\right)^2 + \left(\frac{T_e}{T_{nd}}\right)^2 \leq 1.0$$

10.4 Friction Grip Type Bolting

10.4.3 Slip resistance

$$V_{sf} = (\mu_f \; n_e \, K_h \, F_o) / \gamma_{mf}$$

Where,

μ_f = coefficient of friction (slip factor) as in Table 10.2 ($\mu_f \leq 0.55$)

n_e = number of effective interfaces offering frictional resistance to slip

K_h = 1.0 for fasteners in clearance holes

= 0.85 for fasteners in oversized and short slotted holes

= 0.7 for fasteners in long slotted holes loaded parallel to the slot.

γ_{mf} = 1.10 (if slip resistance is designed at service load)

γ_{mf} = 1.25 (if slip resistance is designed at ultimate load)

F_o = minimum bolt tension (proof load) at installation (0.8 $A_{sb} f_o$)

A_{sb} = shank area of the bolt

f_o = proof stress (= 0.70 f_{ub})

Note: V_{ns} may be evaluated at a service load or ultimate load using appropriate partial safety factors, depending upon whether slip resistance is required at service load or ultimate load.

TABLE 10.2 TYPICAL AVERAGE VALUES FOR COEFFICIENT OF FRICTION *(μ_f)*

Treatment of surface	*Coefficient of friction (μ_f)*
Clean mill scale	0.33
Sand blasted surface	0.48
Red lead painted surface	0.1

10.4 Friction Grip Type Bolting

10.4.2 Bearing capacity

$$V_{bf} = (2.2\, d\, t\, f_{up}) / \gamma_{mf} \leq (3\, d\, t\, f_{yp}) / \gamma_{mf}$$

10.4.3 Tension capacity

$$T_f = (0.9\, f_u\, A) / \gamma_{mf}$$

10.4.4 Combined Shear and Tension

$$\left(\frac{V}{V_{sdf}}\right)^2 + \left(\frac{T_e}{T_{ndf}}\right)^2 \leq 1.0$$

Reduction factor in shear for Long Joints will apply here

10.4 Friction Grip Type Bolting

10.4.5 Prying Force

$$Q = \frac{l_v}{2 l_e}\left[T_e - \frac{\beta \gamma f_o b_e t^4}{27 l_e l_v^2}\right]$$

$$l_e = 1.1\, t \sqrt{\frac{\beta f_o}{f_y}}$$

β = 2 for non-pretensioned and 1 for pretensioned
γ = 1.5 for LSM
b_e = effective width of flange per pair of bolts

(Conti….)

10.5 Welds and Welding

10.5.1 End returns- not less than twice the size of the weld

10.5.2 Lap joint - not less than four times the thickness of the thinner part

10.5.3 Size of weld

10.5.4 Effective throat thickness - shall generally not exceed 0.7*t* , *K* times the fillet size

10.5.5 Effective length or Area of weld

10.5.6 Intermittent welds - effective length of not less than four times the weld size, with a minimum of 40 mm,

10.5.7 weld types and quality – Confirm to IS:814

10.5.8 Design stresses in welds $f_{wd} = f_u/(\sqrt{3})\ \gamma_{mw}$

(Conti….)

10.5 Welds and Welding

10.5.8 Fillet weld applied to the edge of a plate or section

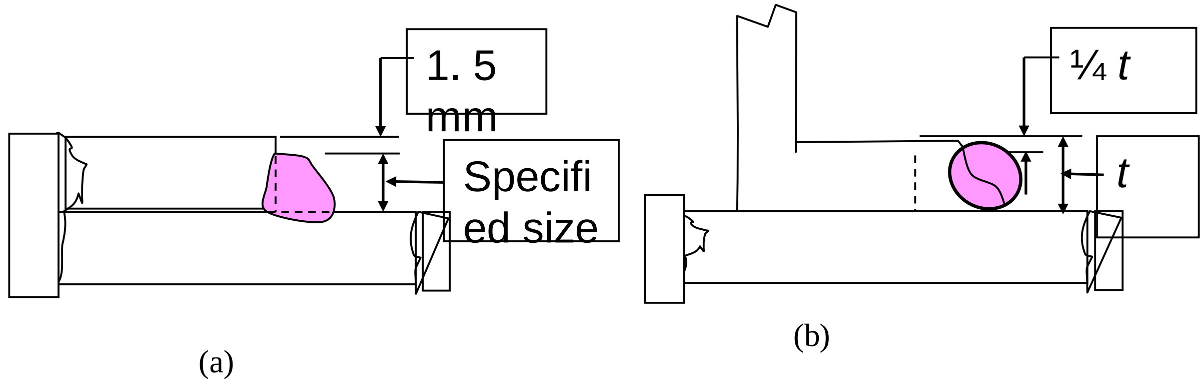
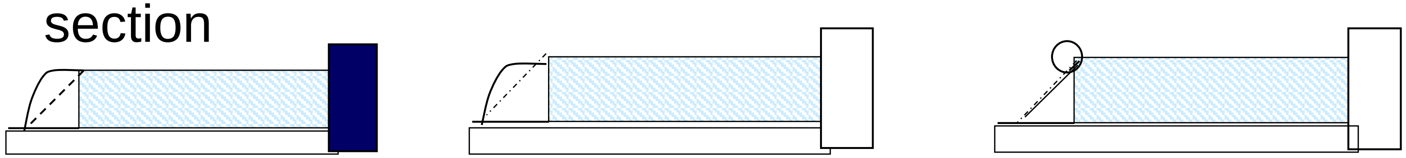


Fig. 10.1 fillet welds on square edge of plate or round toe of rolled section



a) desirable

b) acceptable because of full throat thickness

c) not acceptable because of reduced throat thickness

Fig10.2 Full size fillet weld applied to the edge of a plate or section

10.5.9 Stresses due to Individual forces

$$f_a \text{ or } q = \frac{P}{t_t\, l_w}$$

10.5.10 Combination of stresses

10.5.10.1 Fillet welds

$$f_e = \sqrt{f_a^2 + 3q^2} \le \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$$

Combined bearing, bending and shear

$$f_e = \sqrt{f_b^2 + f_{br}^2 + f_b f_{br} + 3q^2}$$

(Conti….)

10.6 Design of Connections

- **Connections and adjacent regions of the members shall be designed such that :**

a) the design action effects distributed to various elements shall be in equilibrium with the design action effects on the connection,

b) the required deformations in the elements of the connections are within their deformations capacities,

c) all elements in the connections and the adjacent areas of members shall be capable of resisting the design action effects acting on them,

d) the connection elements shall remain stable under the design action effects and deformations

10.7 Minimum Design Action on Connection

Connections carrying design action effects, shall be designed to transmit the greater of.

a) The design action in the member; and

b) The minimum design action effects expressed either as the value or the factor times the member design capacity for the minimum size of member required by the strength limit state, specified as follows:

i) Connections in Rigid Construction – a bending moment of at least 0.5 times the member design moment capacity

ii) Connections to Beam in Simple Construction – a shear force of at least 0.15 times the member design shear capacity or 40 kN. Whichever is lesser.

iii) Connections at the ends of Tensile or Compression Member – a force of at least 0.3 times the member design capacity

iv) Splices in Members Subjected to Axial Tension – a force of at least 0.3 times the member design capacity in tension.

v) Splices in Members Subjected to Axial Compression – for ends prepared for full contact – adequate fasteners to keep line and transmit $0.15P_d$

for ends not prepared for full contact – adequate fasteners to keep line and transmit $0.3P_d$ and a moment of Pd L/1000 where L= dist. bet. lat supports

vi) Splices in Flexural Members – a bending moment of 0.3 times the member design capacity in bending unless designed to transmit shear only

vii) Splices in Members Subject to Combined Actions – a splice in a member subject to a combination of design axial tension or design axial compression and design bending moment shall satisfy requirements in (iv), (v) and (vi) above

Other details

10.8 Intersections

At a joint, the member centroidal axes shall meet at a point, otherwise the members shall be designed for the bending moment arising due to eccentricity

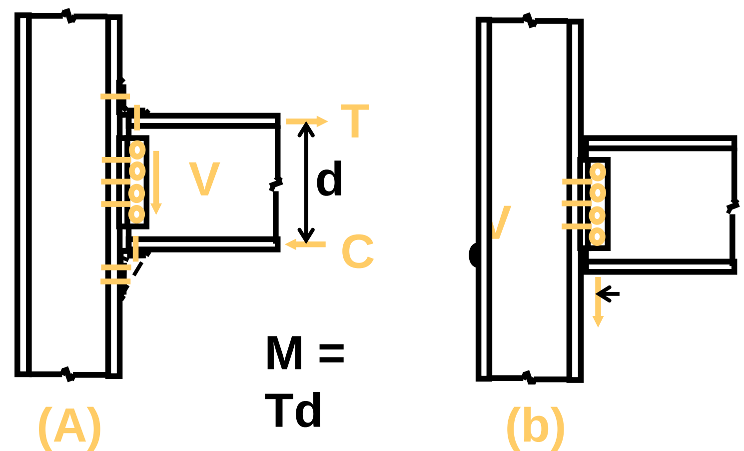
10.9 Choice of fasteners – Use HSFG, weld or fitted bolts to avoid slip in serviceability. When ordinary bolts are subjected to impact or vibration use locking devices

10.10 Connection Components – (Cleats, gusset plates, brackets and the like) shall have their capacities assessed using the provisions of Sections 5,6,7,8 and 9 as applicable.

GENERAL ISSUES IN CONNECTION DESIGN

Assumptions in traditional analysis

- Connection elements are assumed to be rigid compared to the connectors
- Connector behavior is assumed to be linearly elastic
- Distribution of forces arrived at by assuming idealized load paths
- Provide stiffness according to the assumed behavior
- ensure adequate ductility and Rotation capacity
- provide adequate margin of safety



Standard Connections (a) moment connection (b) simple connection

10.11 Analysis of a Bolt/Weld Group

10.11.1 Bolt/Weld Group Subject to In-plane Loading

The design force in a bolt/weld shall be determined by

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

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

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

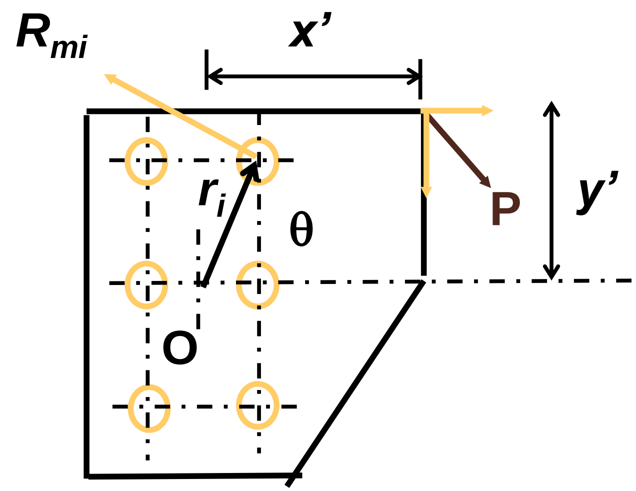
COMBINED SHEAR AND MOMENT IN PLANE

- **Bolt shear due to P_x and P_y**
  $R_{xi} = P_x/n$ and $R_{yi} = P_y/n$

- **$M = P_x\, y' + P_y\, x'$**

- **$R_{mi} = k\, r_i$**
  $M_i = k\, r_i^2$
  $MR = \Sigma\, k\, r_i^2 = k\, \Sigma\, r_i^2$

- **Bolt shear due to M**
  $R_{mi} = M\, r_i / \Sigma\, r_i^2$



Bolt group eccentrically loaded in shear

Combined shear

$$R_i = \sqrt{\left[\left(R_{xi} + R_{mi}\cos\theta_i\right)^2 + \left(R_{yi} + R_{mi}\sin\theta_i\right)^2\right]}$$

$$R_i = \sqrt{\left\{\left[\frac{P_x}{n} + \frac{My_i}{\sum(x_i^2 + y_i^2)}\right]^2 + \left[\frac{P_y}{n} + \frac{Mx_i}{\sum(x_i^2 + y_i^2)}\right]^2\right\}}$$

10.11 Analysis of a Bolt/Weld Group

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

The design force shall be determined by

a) The design force resulting from shear or axial force shall be considered to be equally shared by all bolts or over the length weld

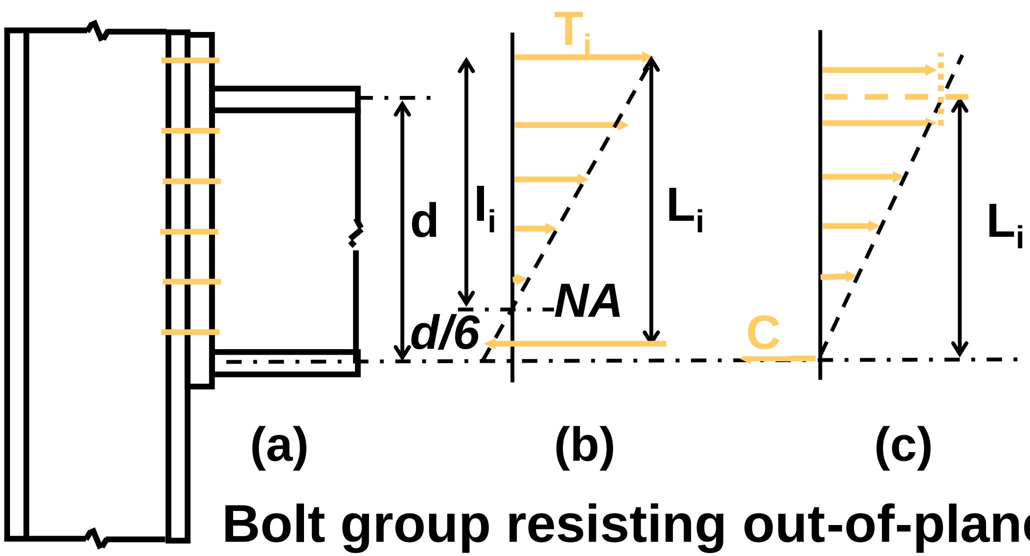
b) The design force resulting from a bending moment shall be considered to vary linearly with the distance from the centroidal axes for the calculation of centroid and second moment:

i) In bearing type of bolt group, plates in the compression side of the NA and only bolts in the tension side may be considered.

ii) In the friction grip bolt group only the bolts shall be considered

iii) The fillet weld group shall be considered in isolation from the connected element; of the weld length.

COMBINED SHEAR AND MOMENT OUT-OF-PLANE



Bolt group resisting out-of-plane moment

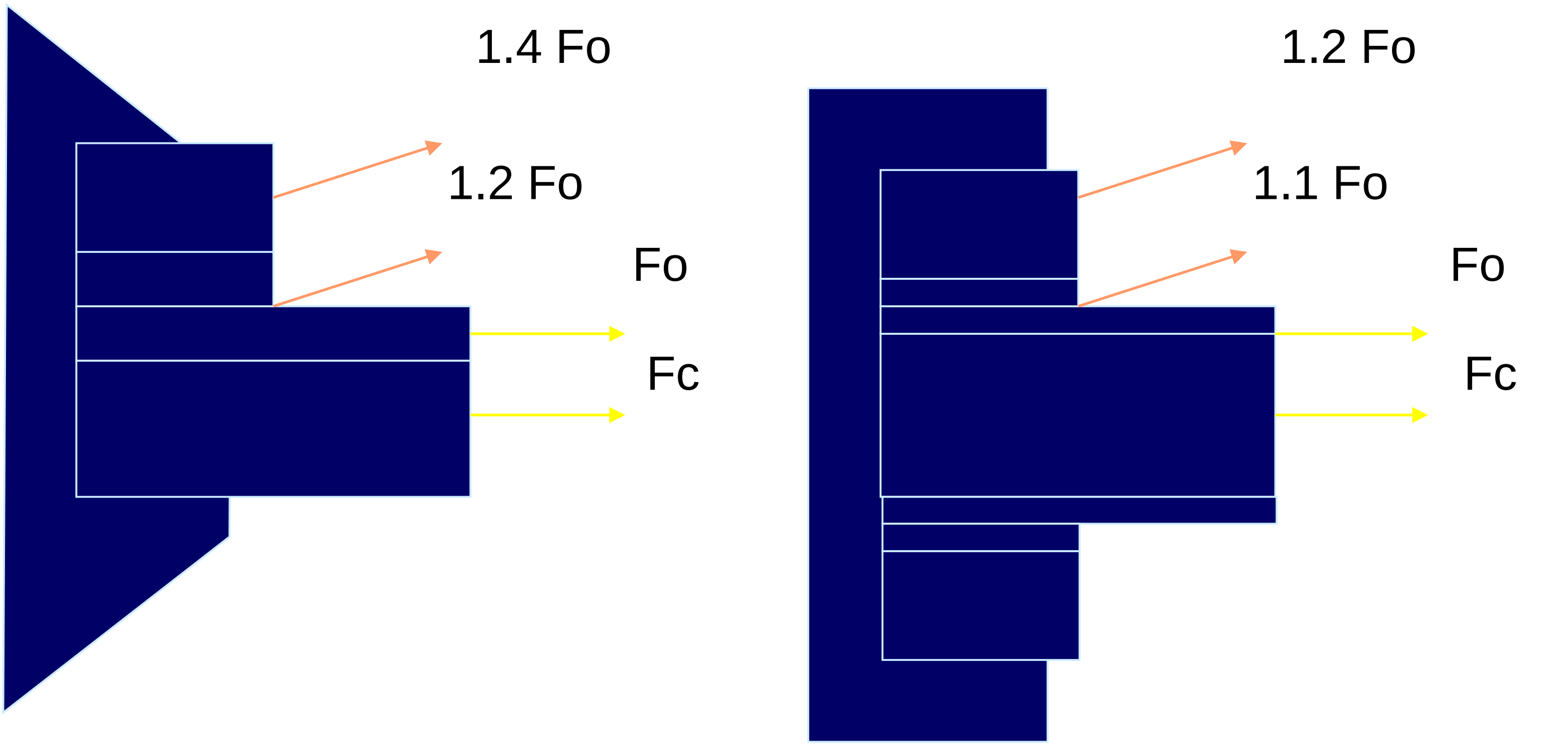
$T_i = kl_i$ where k = constant

$M = \Sigma T_i L_i = k \Sigma l_i L_i$

$T_i = Ml_i / \Sigma l_i L_i$

Shear assumed to be shared equally and bolts checked for combined tension+(prying)+shear

10.12 Lug Angles



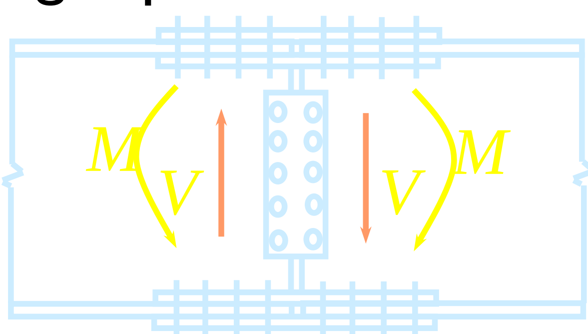
G.2 Beam Splices

G.2.1 For rolled section, assumed that flange splice carries the moment and web splice carries shear

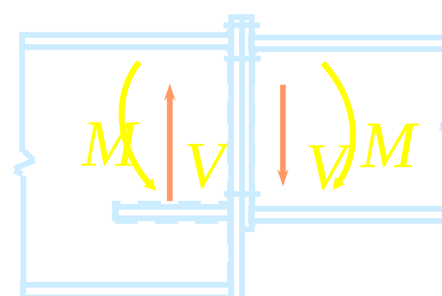
For a deep girder, the moment may be divided. The web connection should then be designed to resist its share of moment and shear.

Even if web splice is designed to carry only shear force, the bolt group on either side should be designed for moment due to eccentricity.

Flange splice area = 1.05 flange area



Inner Splice Plates (optional)



Stiffener Plate (optional)

G.3 Column Splice

- Where the ends of compression members are machined for bearing over the whole area, they shall be spliced to hold the line
- Else splices shall be designed to transmit all the force
- splices shall be proportioned and arranged so that centroidal axis of the splice coincides with member

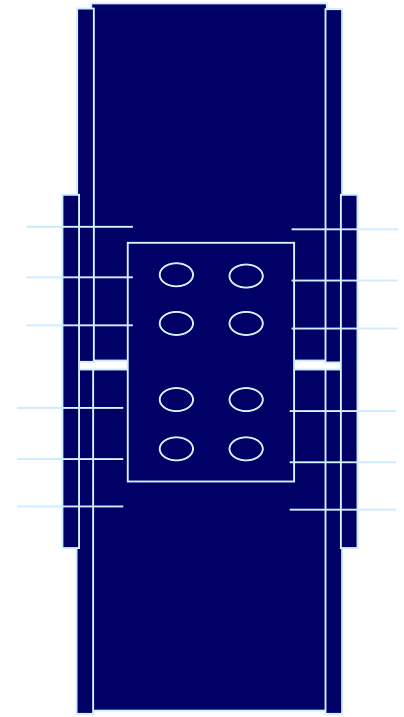
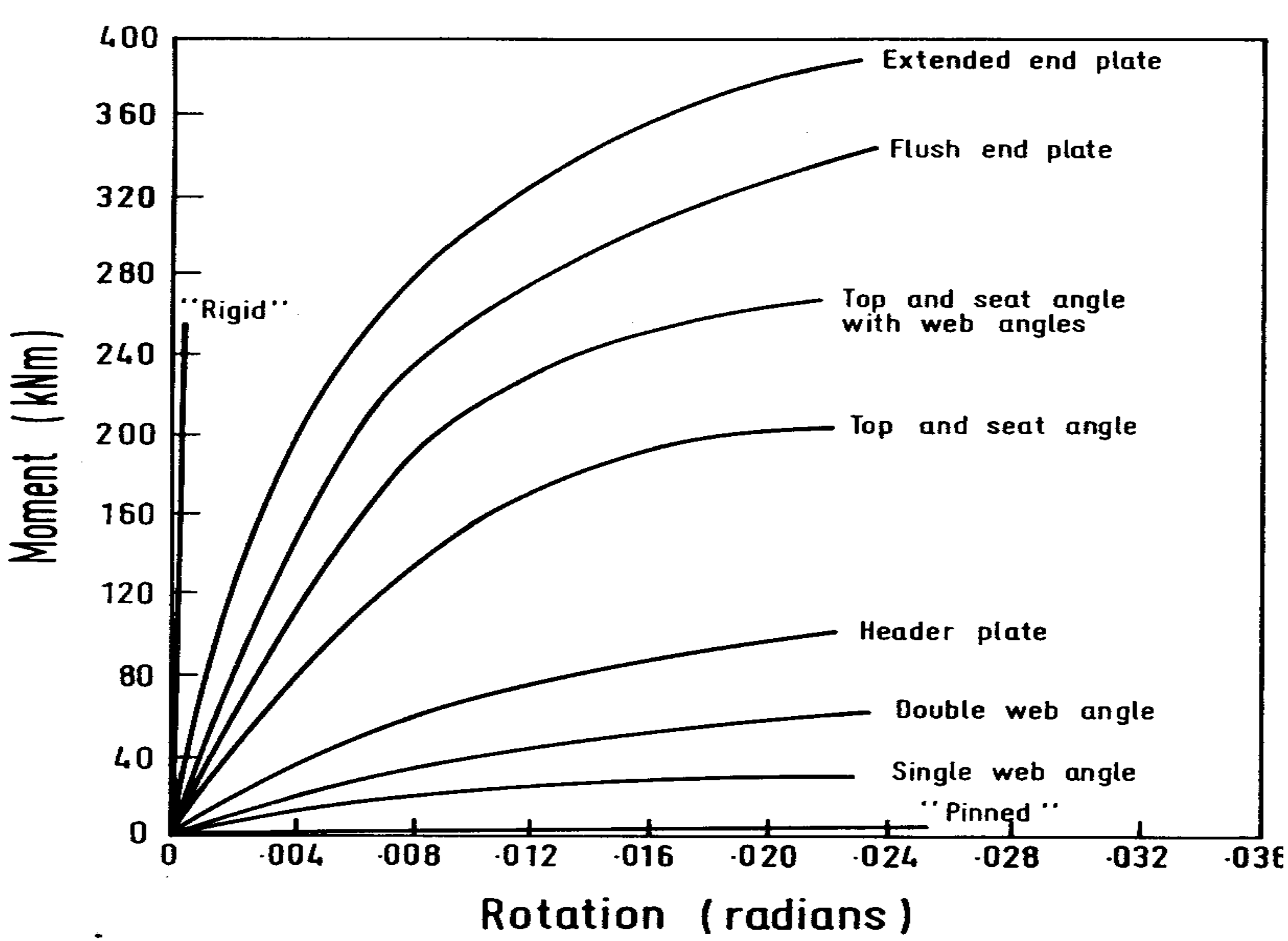
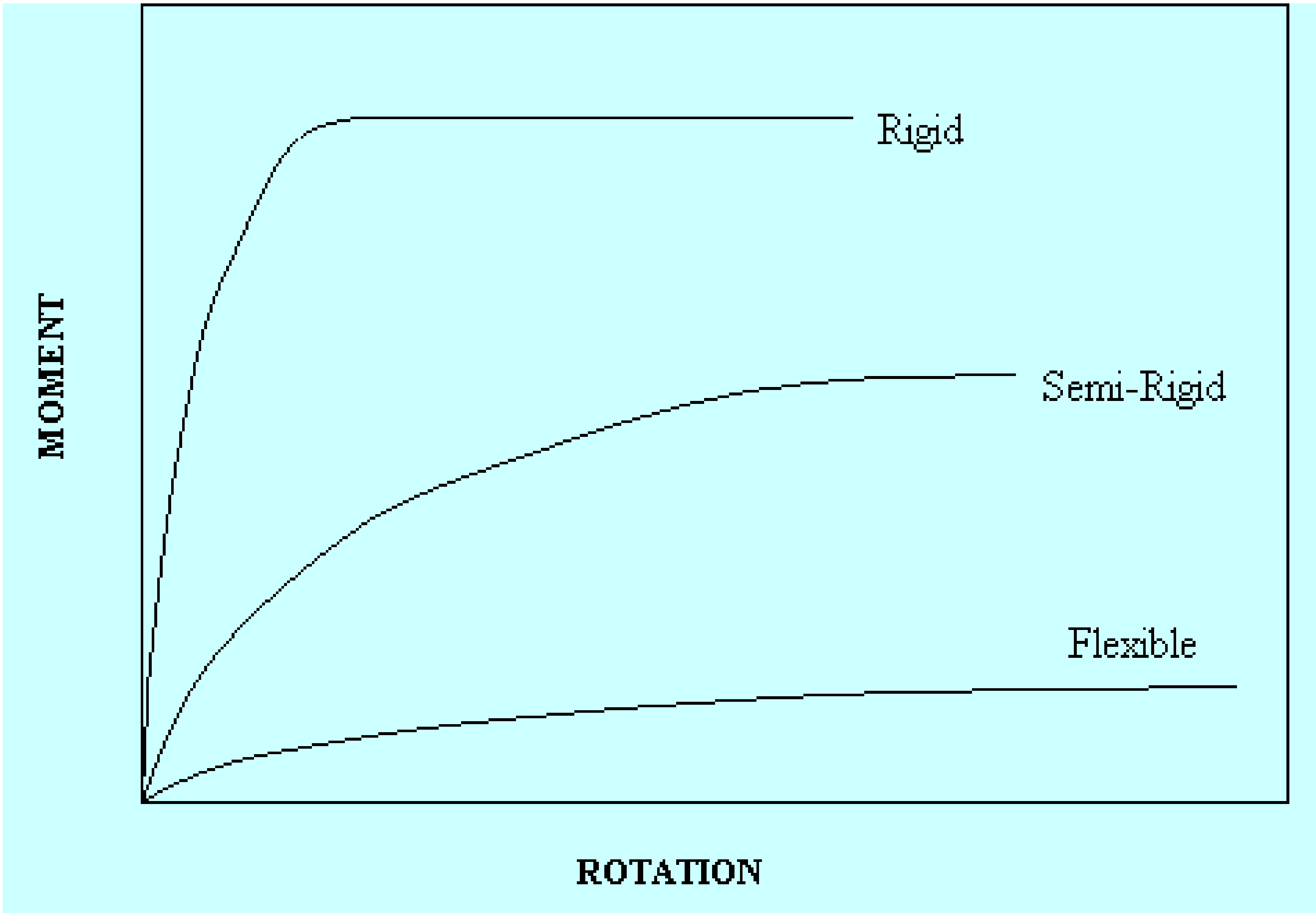


FIG. G.2 COLUMN SPLICE
(TYPICAL)

CONNECTION BEHAVIOUR



10.6.2.3 Moment- rotation relationship



10.6.2.4 Classifications of connections according to Bjorhovde (1990)

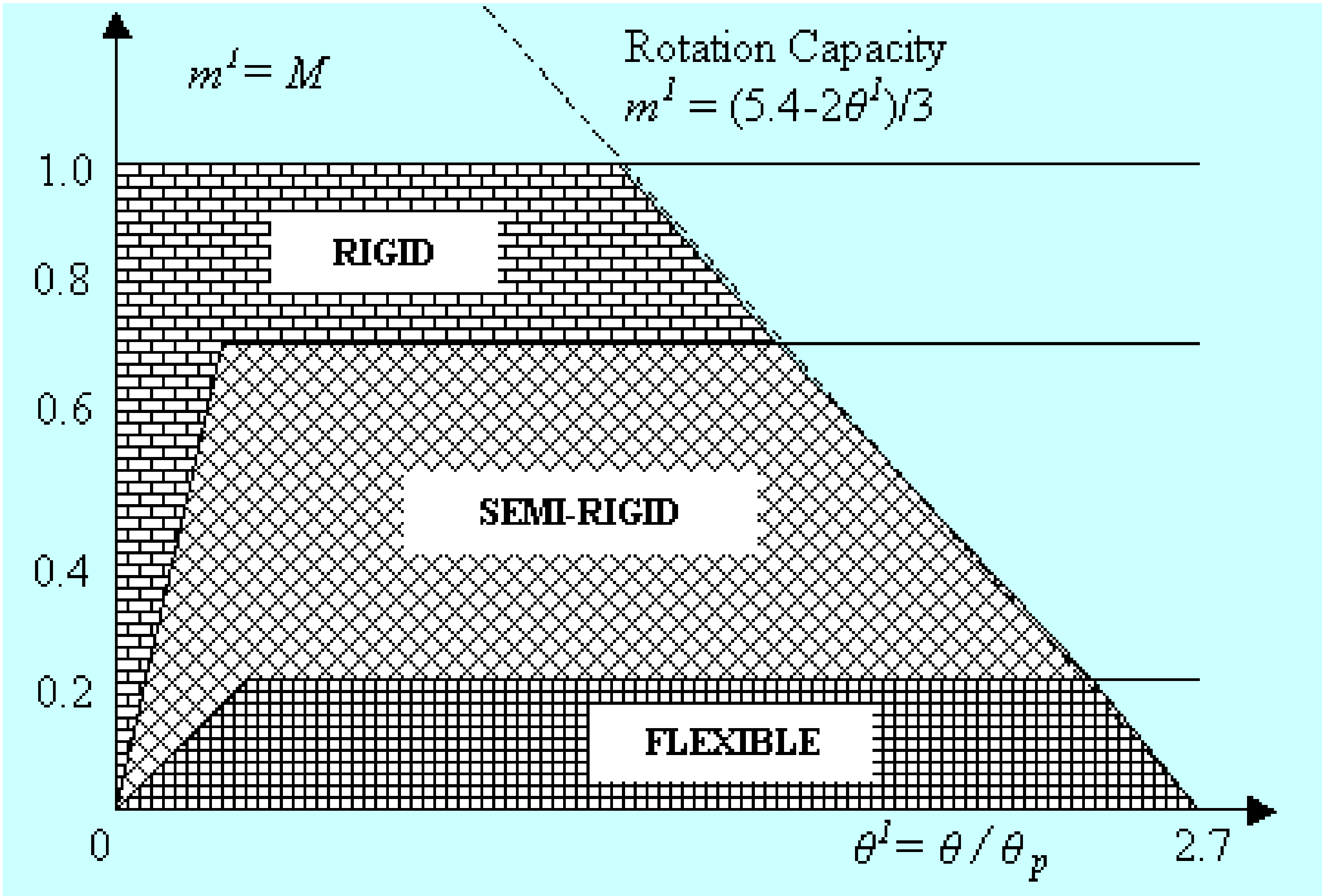


TABLE G.1 CONNECTION CLASSIFICATION LIMITS

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

SEMI-RIGID CONNECTION MODEL

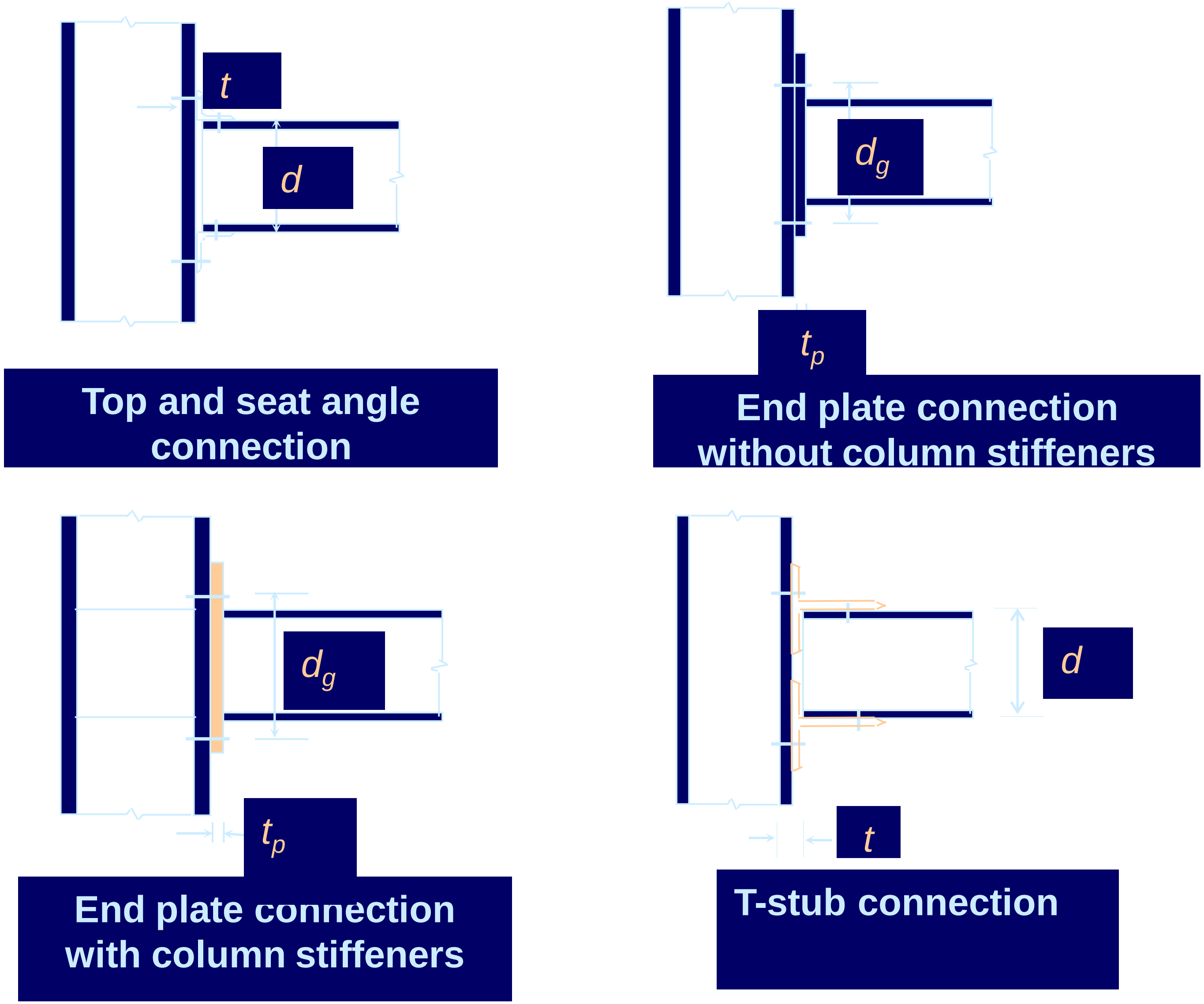
- Frye-Morris polynomial model

$\theta r = C_1(Km)^1 + C_2\,(Km)^3 + C_3\,(Km)^5$

K is a standardization parameter Table G 2

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

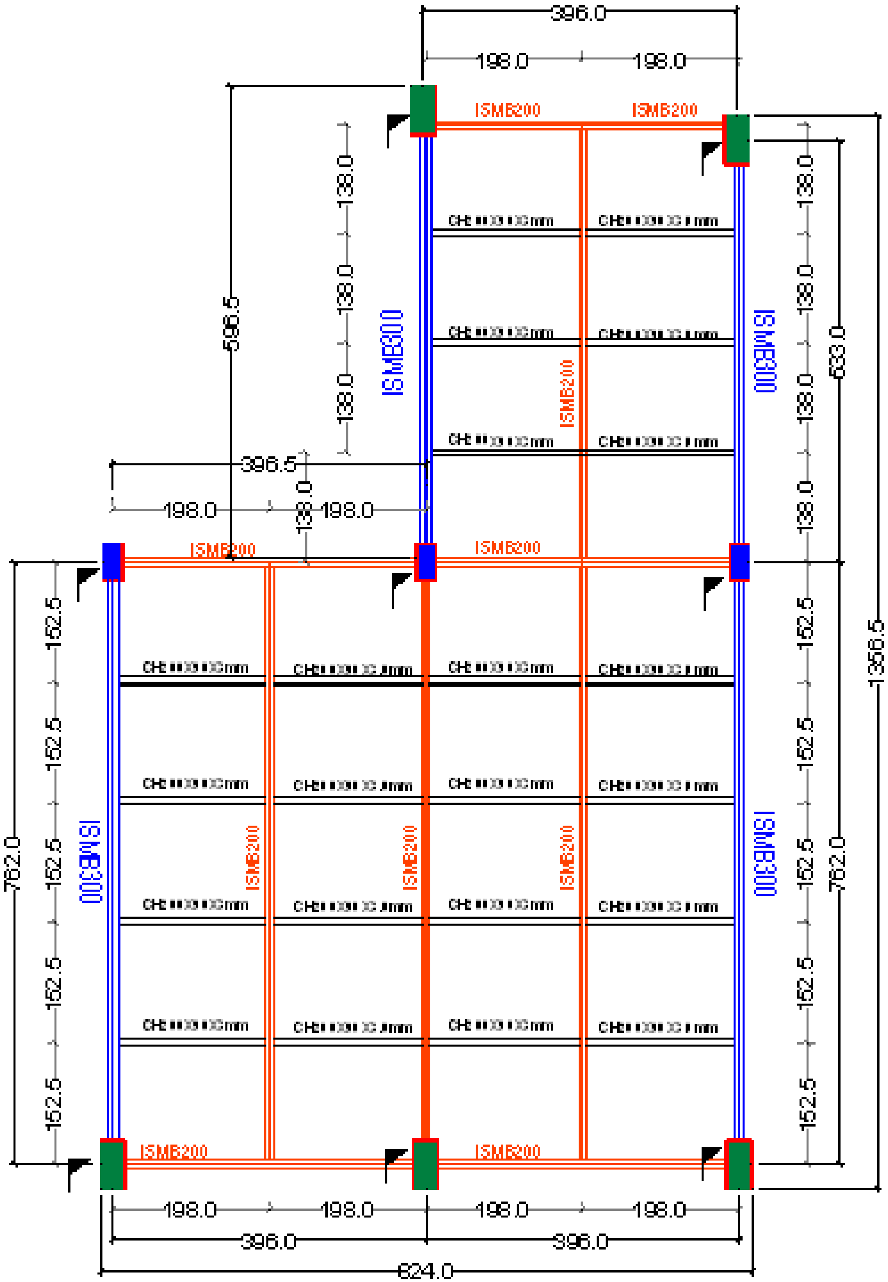
Fig G.7 Size parameter for various connection type



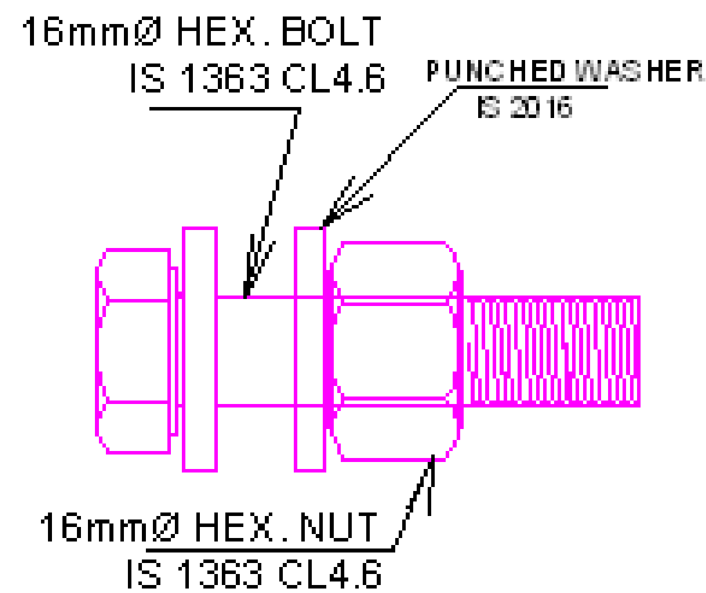
SUMMARY

- **Design of Connections**
- **Minimum Design Action on Connection**
- **Other Details**
- **Analysis of a Bolt/Weld Group**
- **Beam and Column Splices**
- **Lug Angles**
- **Connection classification**
- **Semi-rigid Connection Models**

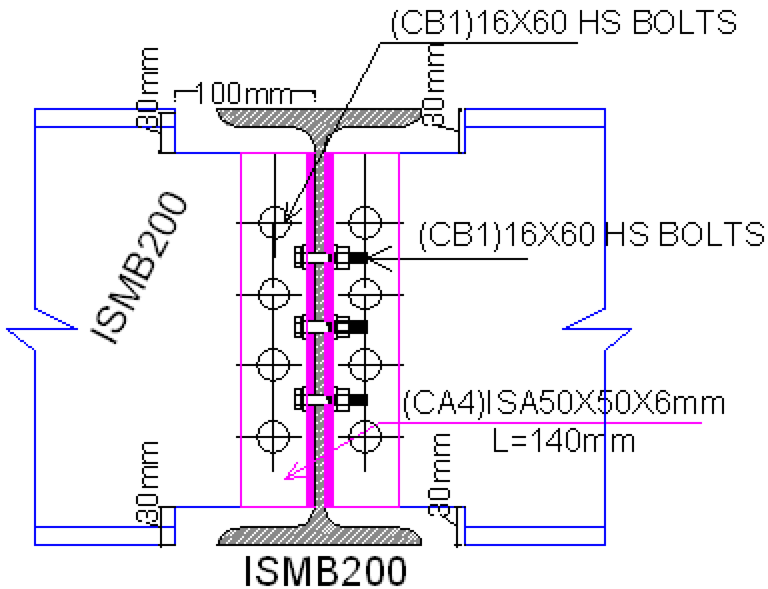
CONSULTANCY UNDERTAKEN



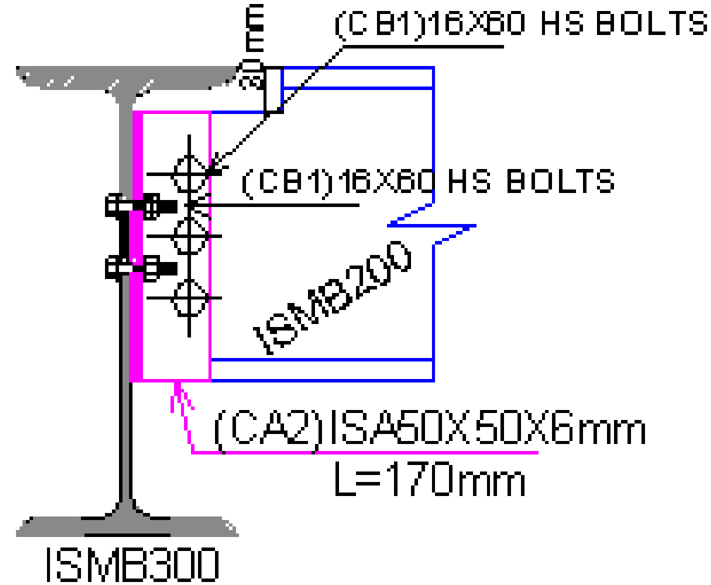
MEZZNINE FLOOR(2) LAYOUT



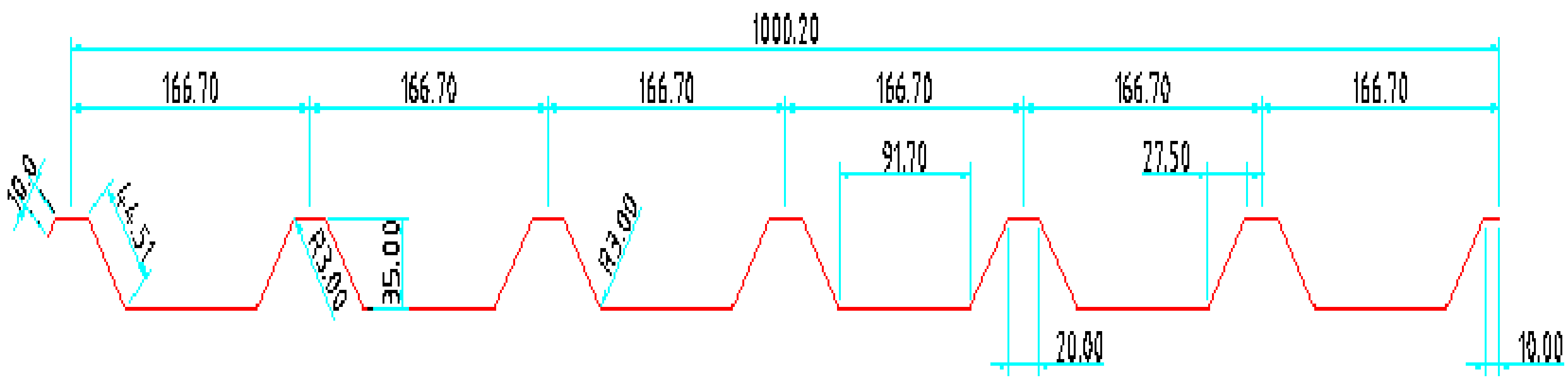
BOLT ASSEMBLY(TYP.)

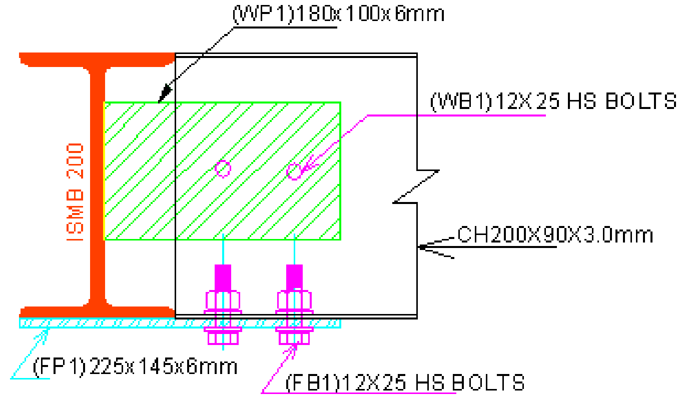


DETAILS OF BEAM TO BEAM CONNECTION
IN ISMB200 AND ISMB200(BC4-36Nos)

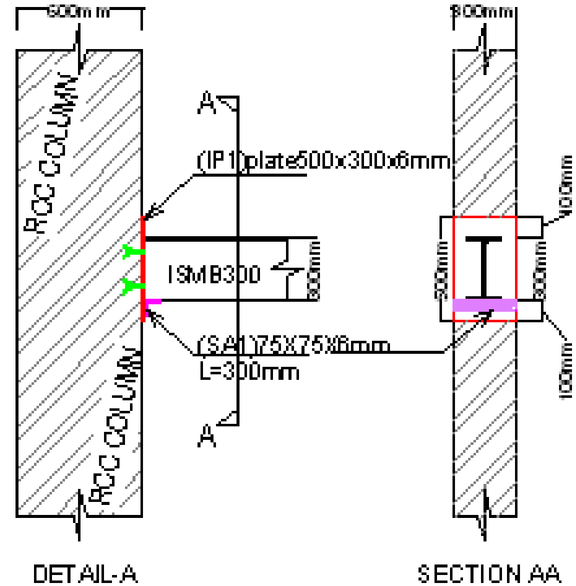


DETAILS OF BEAM TO BEAM CONNECTION
IN ISMB300 AND ISMB200(BC2-15Nos)

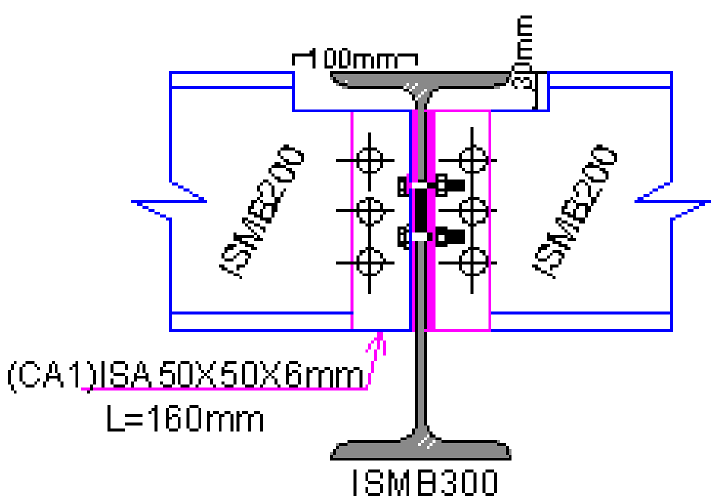




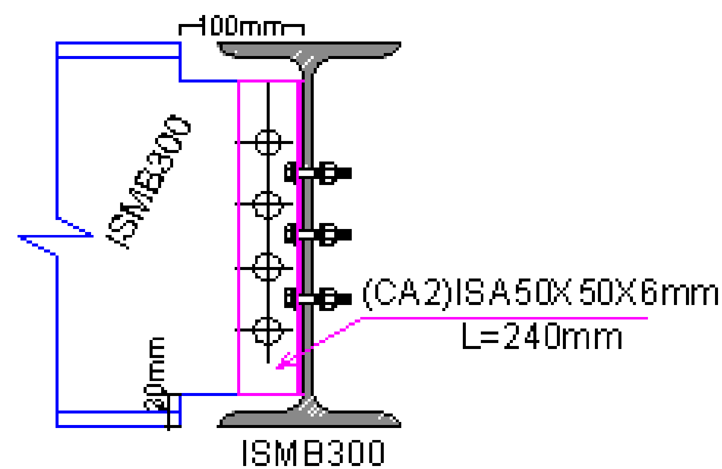
DETAIL CONNECTION OF STEEL SECTION AND
COLD FORM SECTION ISMB200 WITH CH200X90X3.0mm



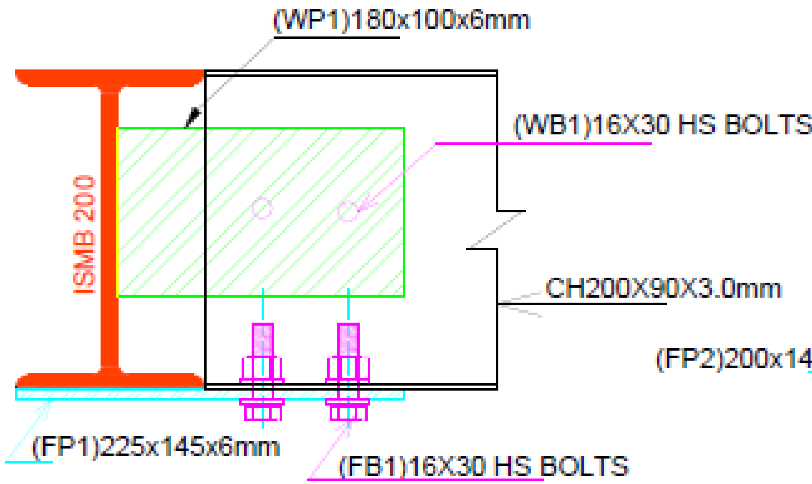
DETAILS OF RCC COLUMN(300X600mm) AND STEEL
BEAM ISMB300



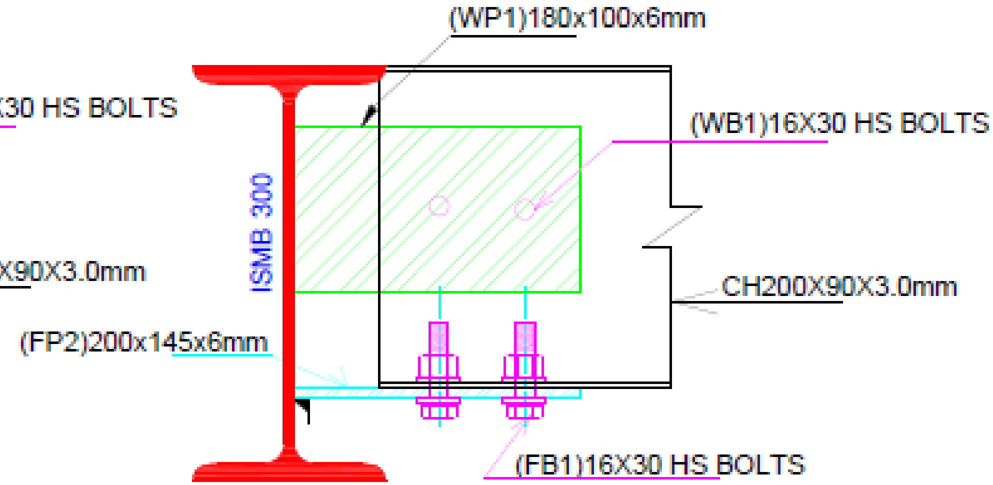
DETAILS OF BEAM TO BEAM CONNECTION
IN ISMB300 AND ISMB200



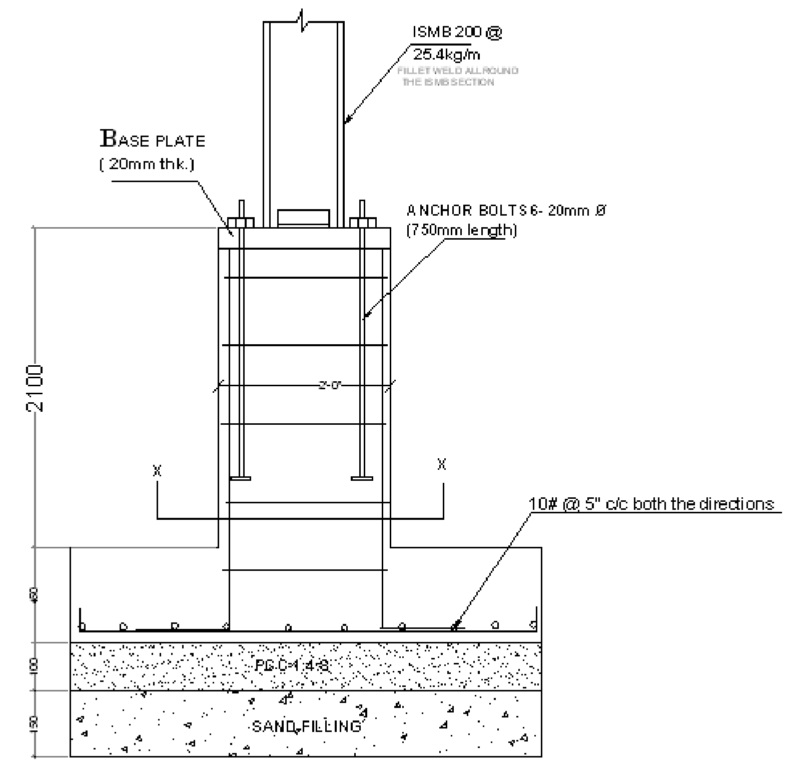
DETAILS OF BEAM TO BEAM CONNECTION
IN ISMB300 AND ISMB300



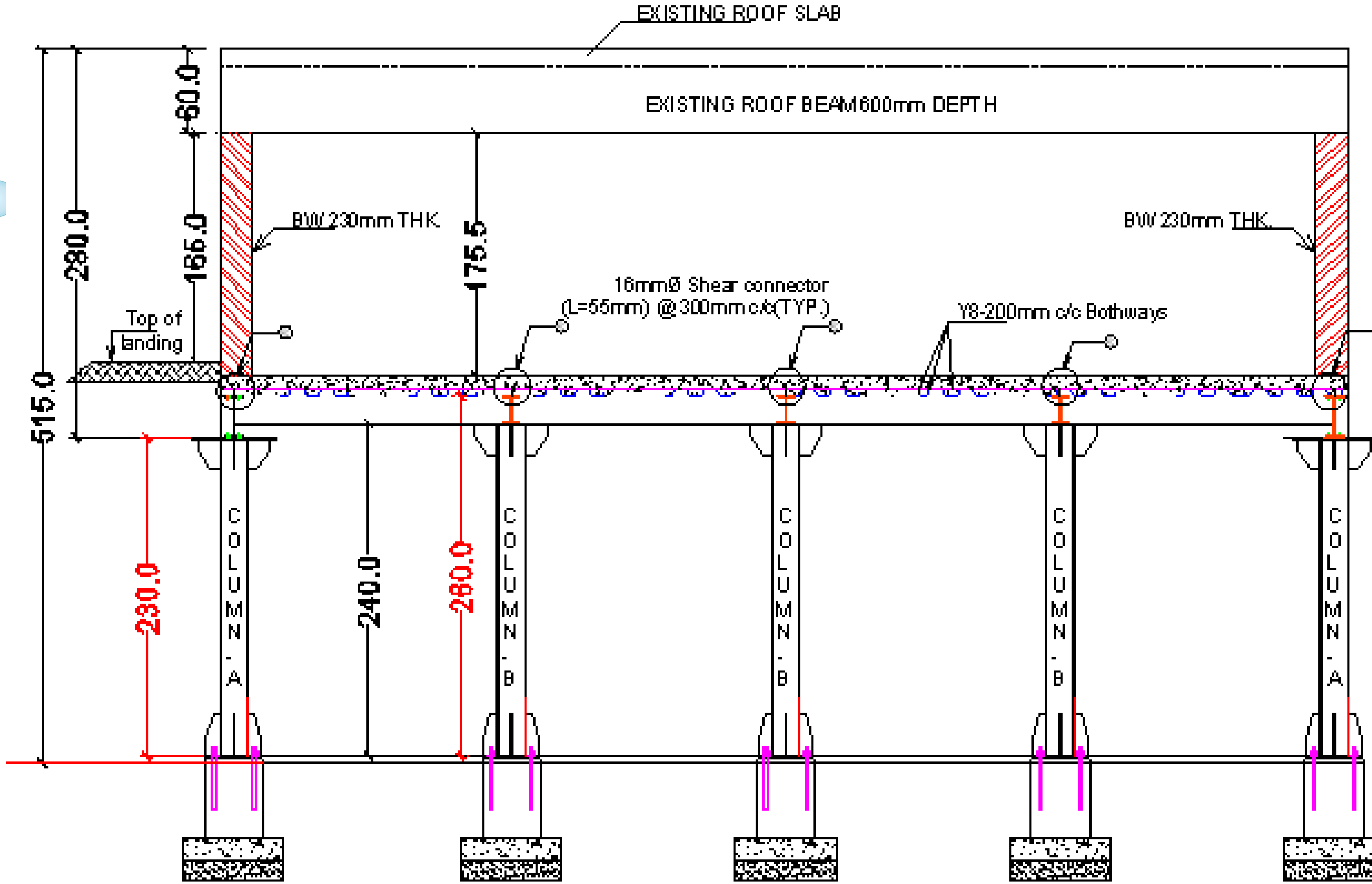
DETAIL CONNECTION OF STEEL SECTION AND COLD FORM SECTION ISMB200 WITH CH200X90X3.0mm(BC6)



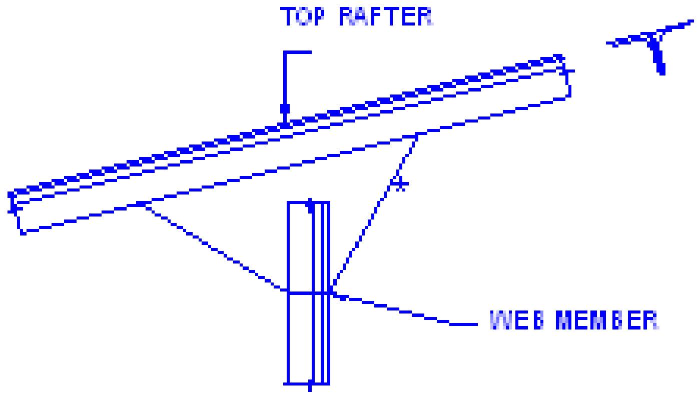
DETAIL CONNECTION OF STEEL SECTION AND COLD FORM SECTION ISMB300 WITH CH200X90X3.0mm(BC7)



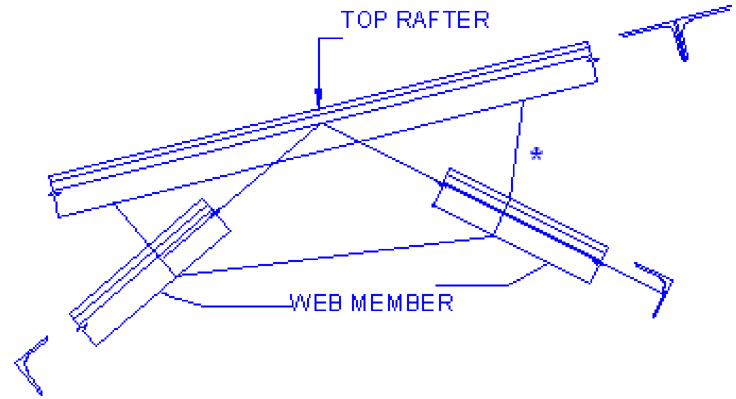
SECTIONAL ELEVATION OF PEDESTAL



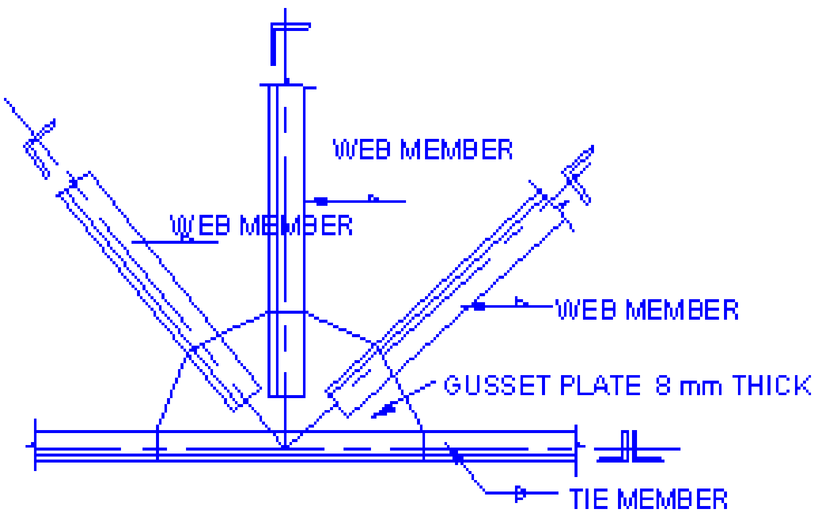
SECTION ELEVATION-PP



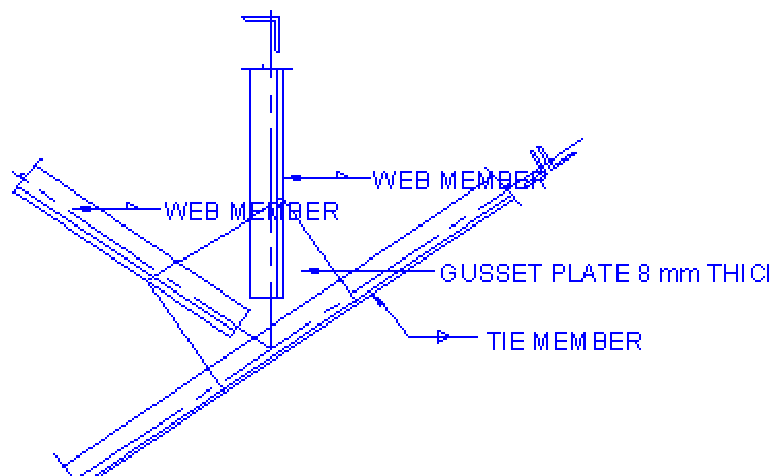
TYPICAL CONNECTION DETAIL - C1



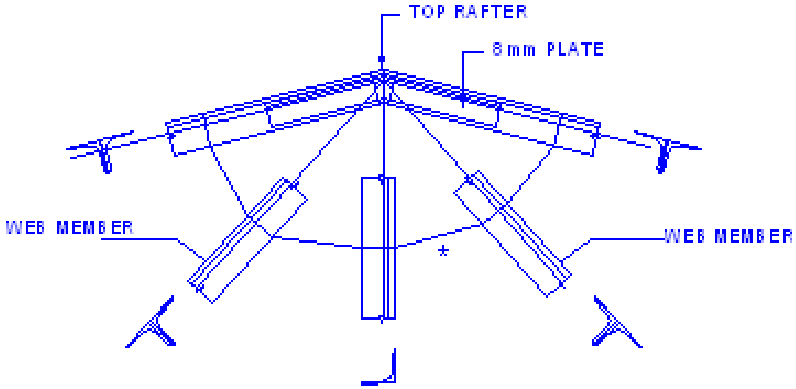
TYPICAL CONNECTION DETAIL - C2



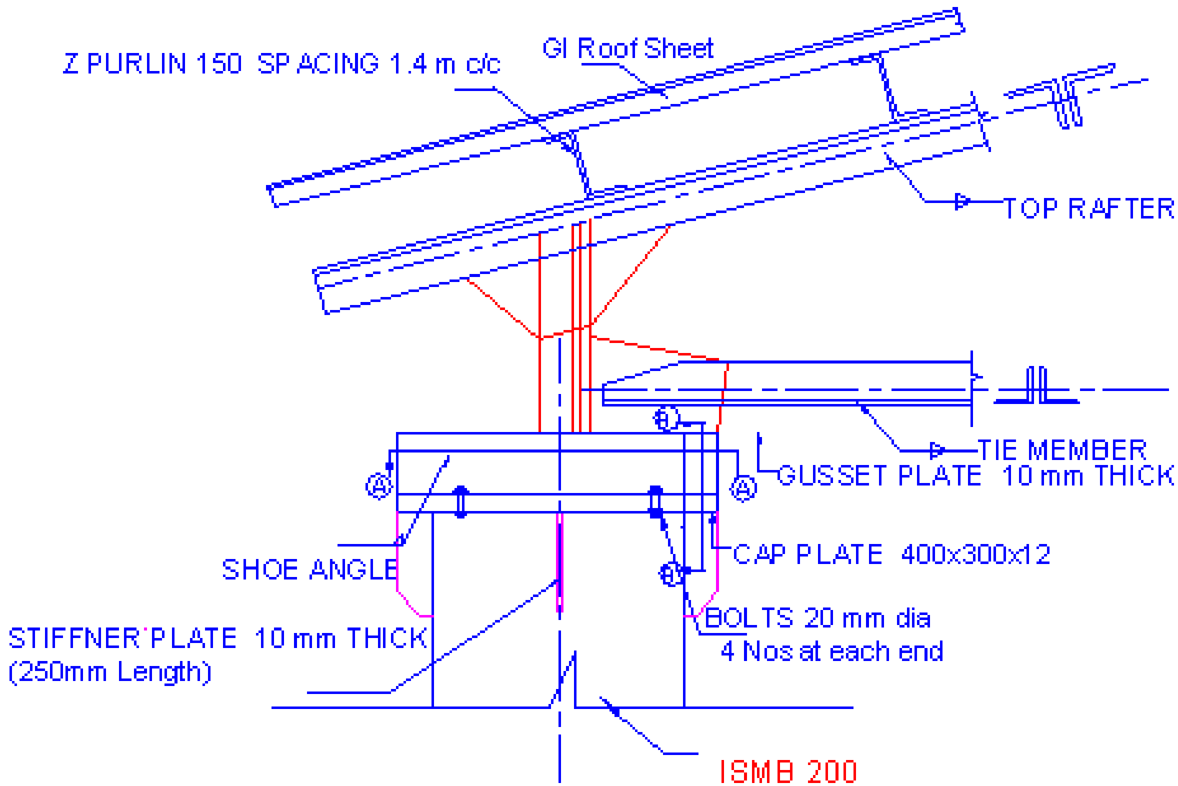
TYPICAL CONNECTION DETAIL - C3



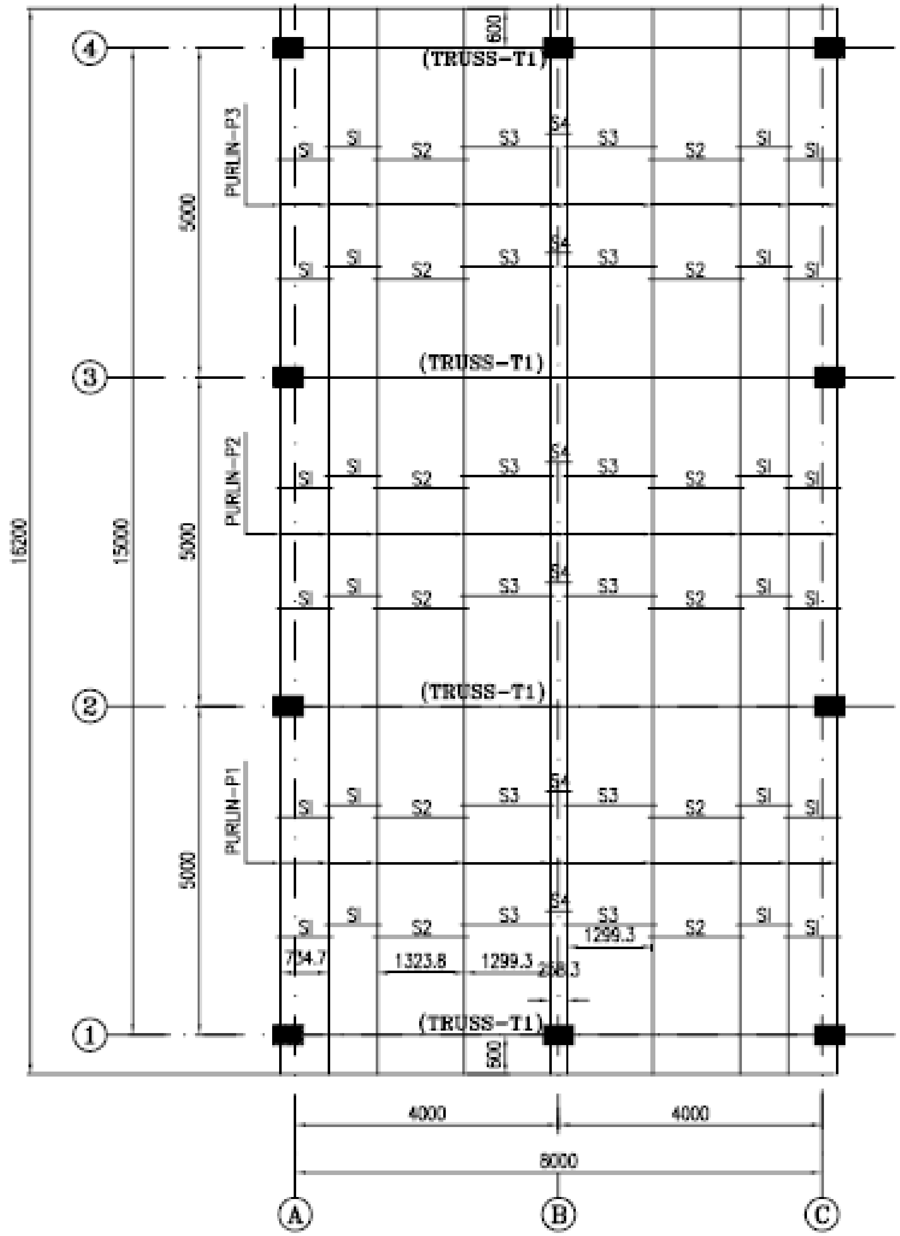
TYPICAL CONNECTION DETAIL - C4



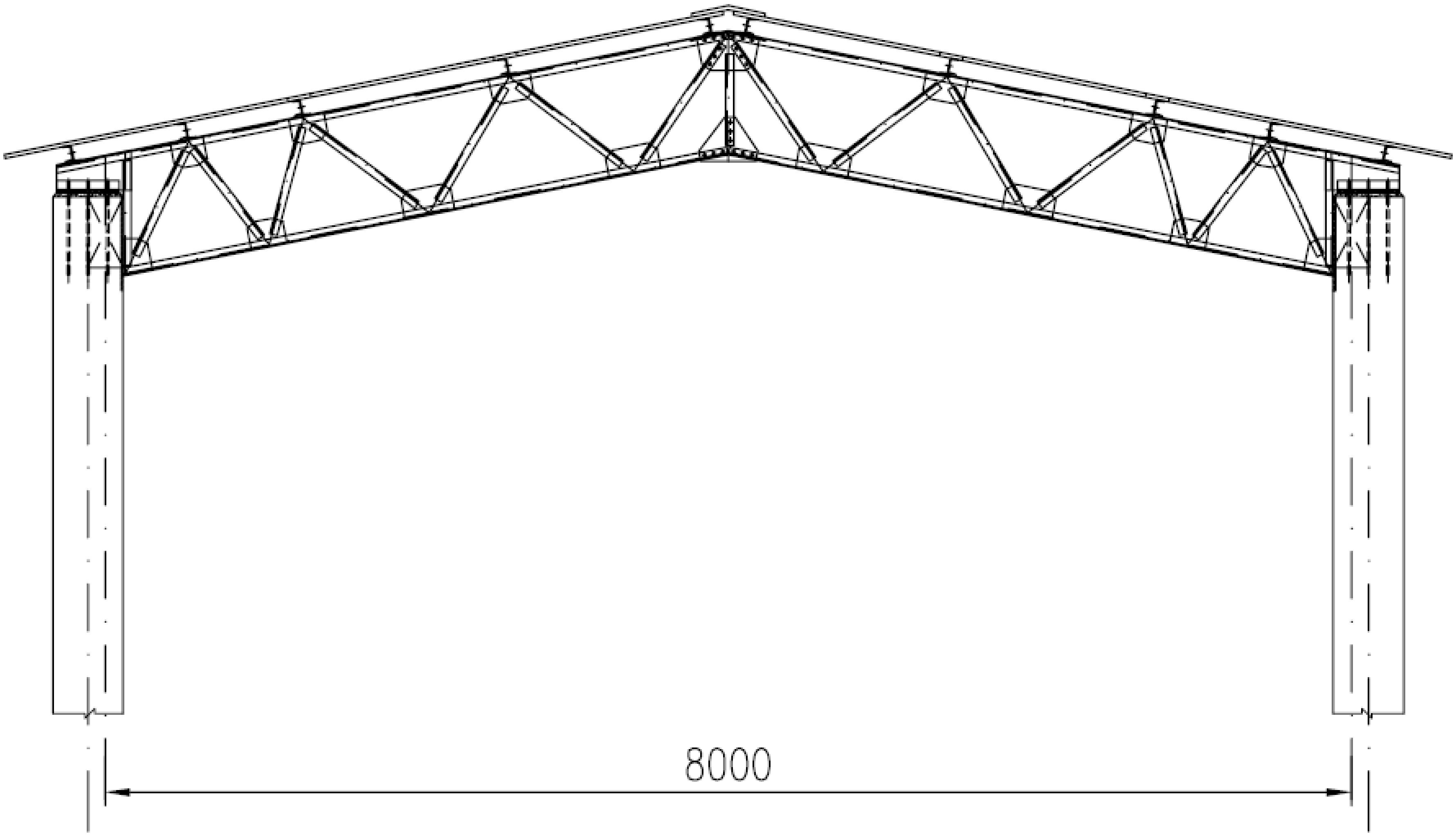
TYPICAL CONNECTION DETAIL - C5



SIDE ELEVATION (ANGLE TRUSS)



TRUSS, PURLIN & SAG ROD LAYOUT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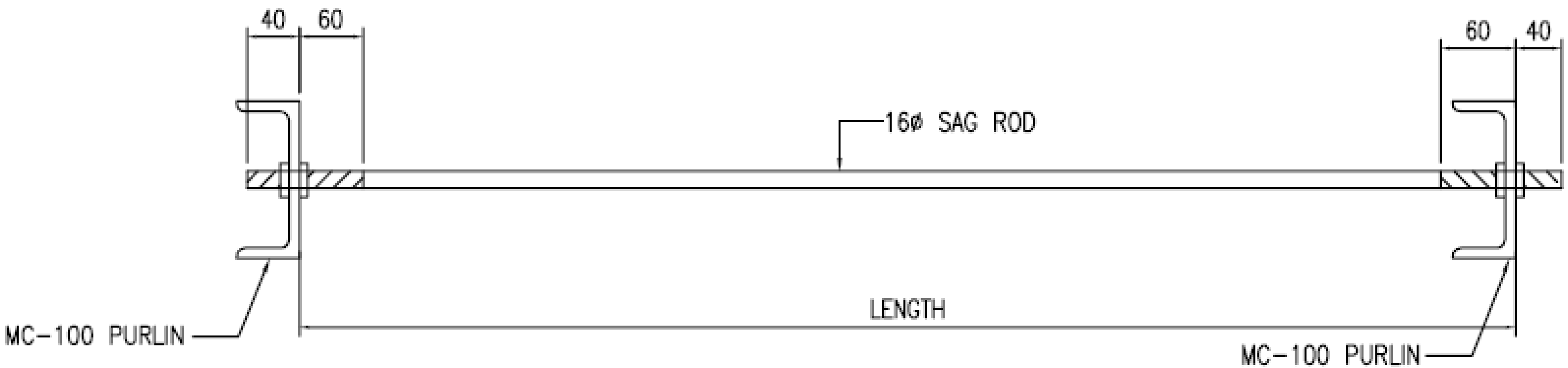
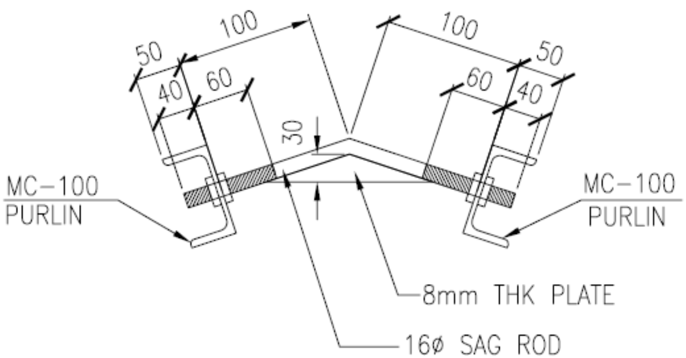


SAG ROD DETAILS

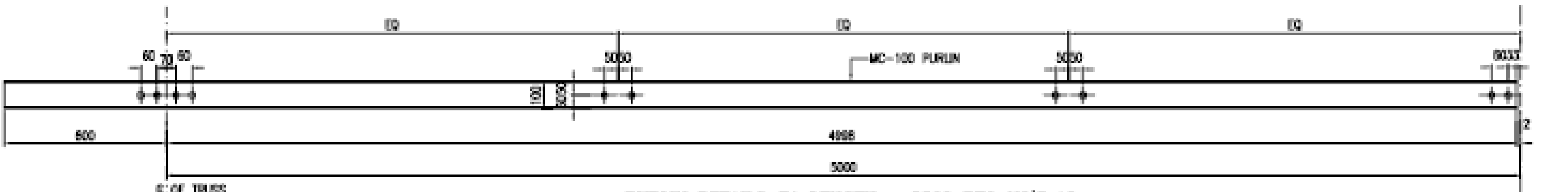
SAG ROD S1 LENGTH =830 REQ NO'S 24

SAG ROD S2 LENGTH =1430 REQ NO'S 12

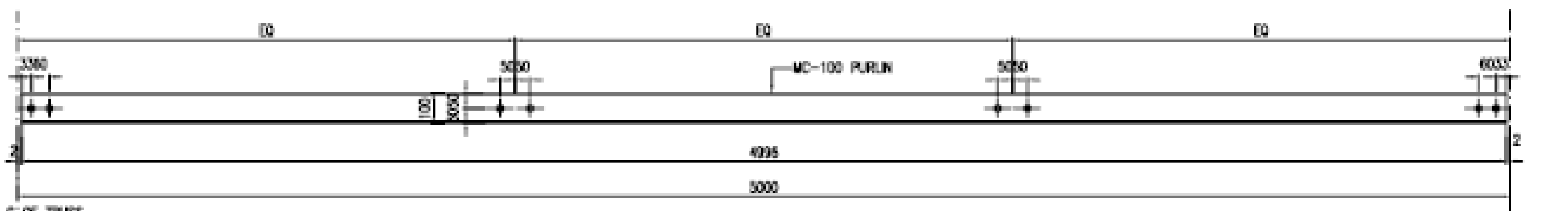
SAG ROD S3 LENGTH =1405 REQ NO'S 12



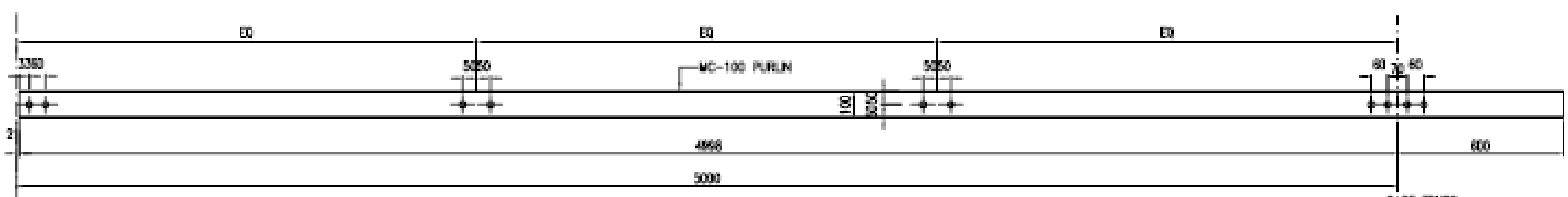
SAG ROD DETAIL S4 (REQ NO'S 6)



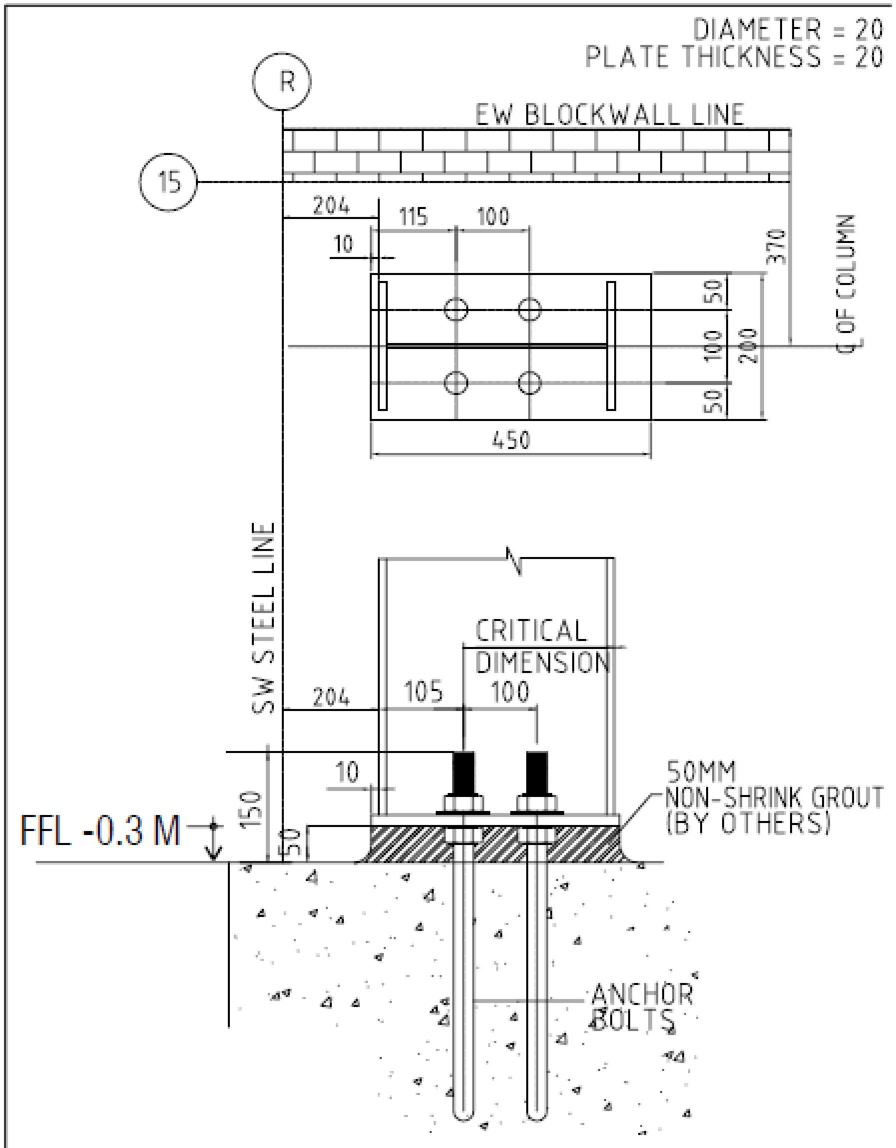
PURLIN DETAILS-P1 LENGTH = 5598 REQ NO'S 10



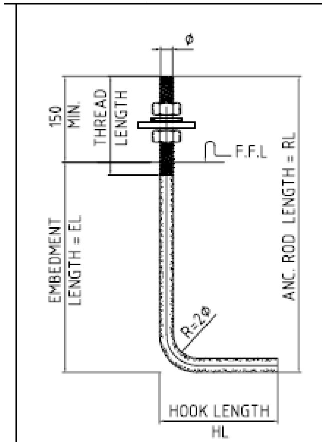
PURLIN DETAILS-P2 LENGTH = 4996 REQ NO'S 10



PURLIN DETAILS-P3 LENGTH = 5598 REQ NO'S 10



DETAIL A



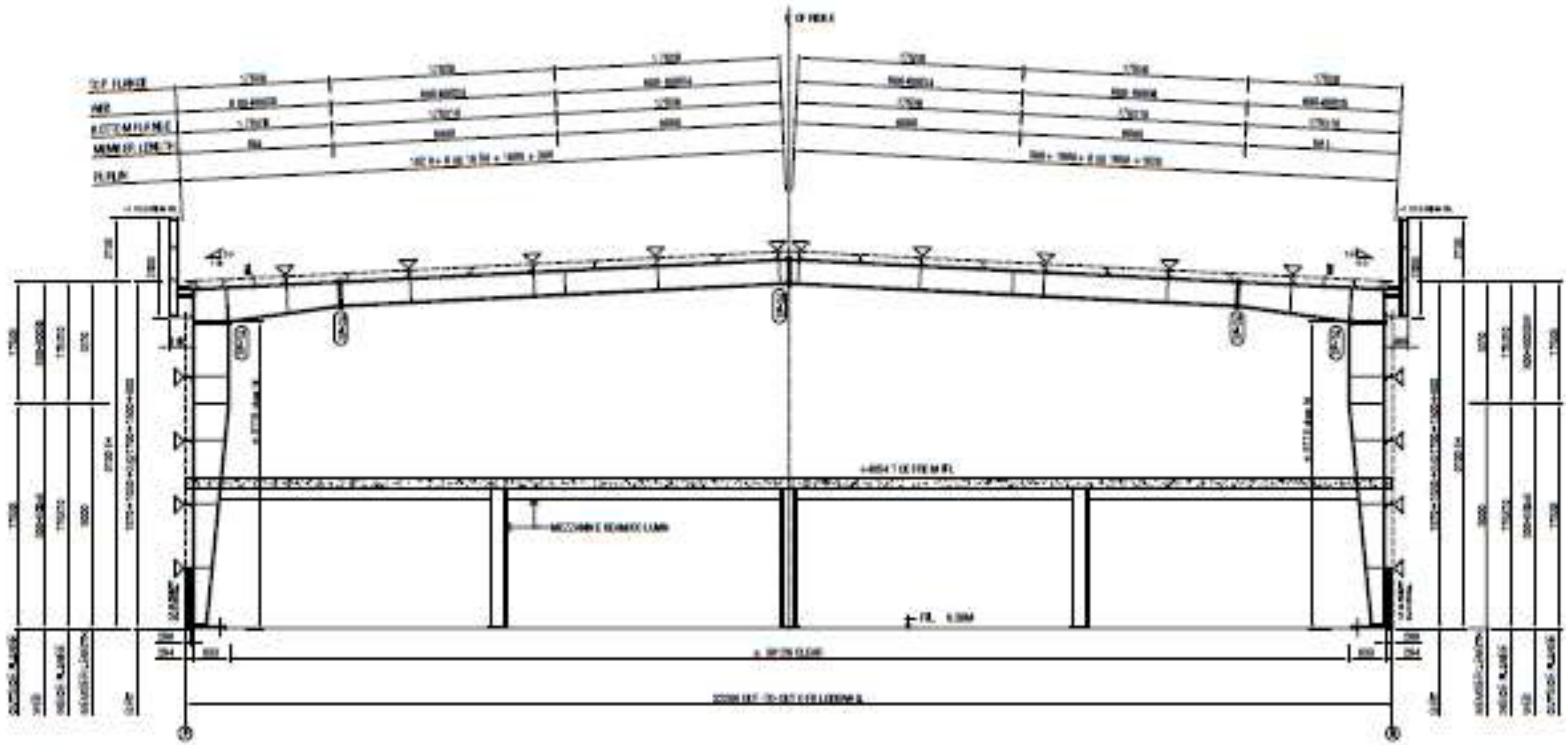
ANCHOR BOLT DETAIL

QTY.	DIA. OF BOLT	DIA. OF HOLE	THREAD LENGTH	HOOK LENGTH	ANC. ROD LENGHT
	ϕ	D	TL	HL	RL
	M16	22	100	120	375
	M20	26	150	175	450
	M24	30	150	205	525
	M30	36	200	265	750
	M36	42	200	295	1050
	>M36	ϕ + 8	200	AS PER DESIGN	

REFERENCE NOTES :-

1. FOUNDATION DESIGN IS NOT THE RESPONSIBILITY OF MABANI STEEL.
2. BRACING REACTIONS ARE TO BE CONSIDERED WITH THE MAIN FRAME REACTIONS.
3. BOTTOM OF BASE PLATES ARE AT THE SAME ELEVATIONS (UNLESS NOTED OTHERWISE)

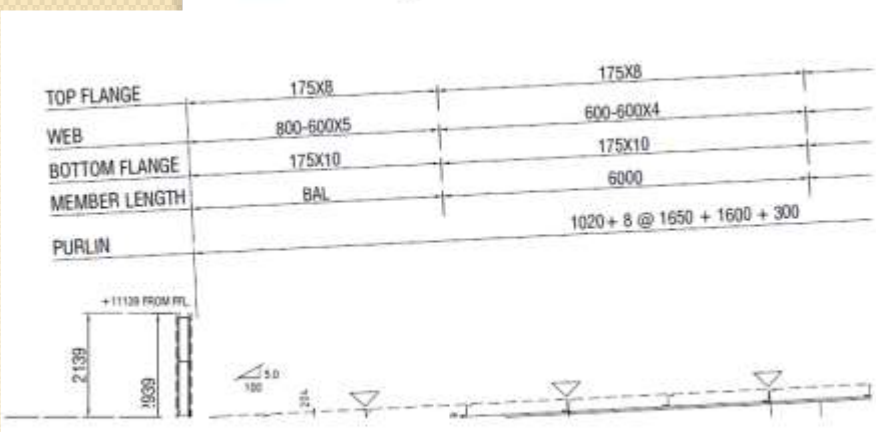
5. 50mm GROUT (by others) IS REQUIRED AT ALL BASE OF COLUMN.
6. 40 x 40 CONCRETE NOTCH IS FOR SINGLE SKIN PANEL ONLY. MAY VARY DEPENDING ON THE ACTUAL WALL PANEL CONDITION.
7. ALL DIMENSIONS ARE IN MILLIMETERS.

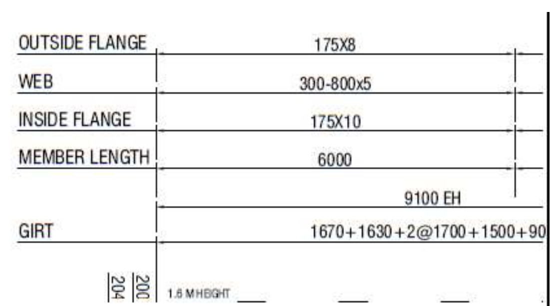


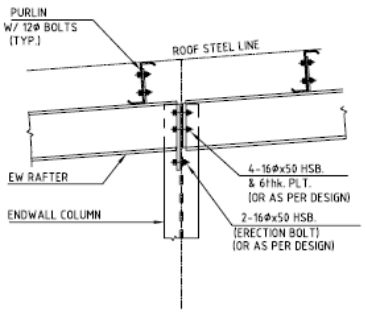
FRAME CROSS SECTION

- DENOTES 50MM THK GROUT (BY OTHERS)

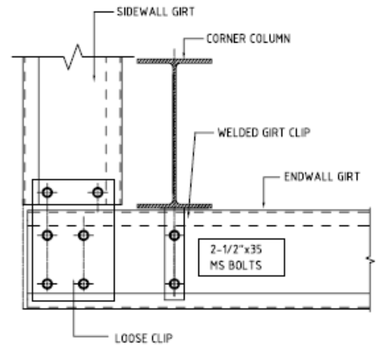
DENOTES DOUBLE FLANGE BRACE LOCATION



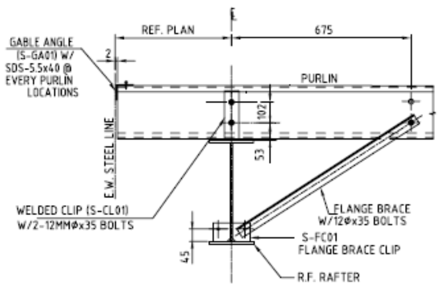




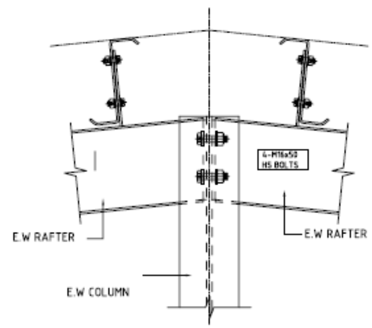
RAFTER SPLICE ALONG SURFACE
WITH COLUMN SUPPORT



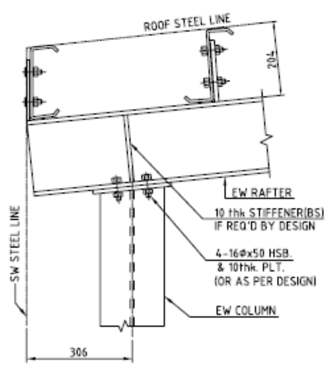
D12 CORNER COLUMN TO WALL GIRT
BY-PASS ENDWALL GIRT CONDITION



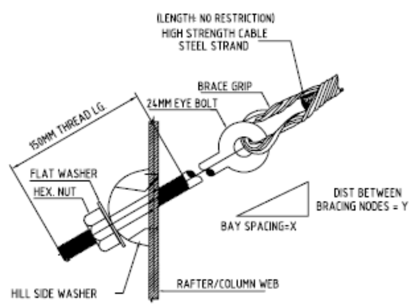
A10 ROOF PURLIN TO EXPANDABLE ENDWALL
RIGID FRAME ENDWALL



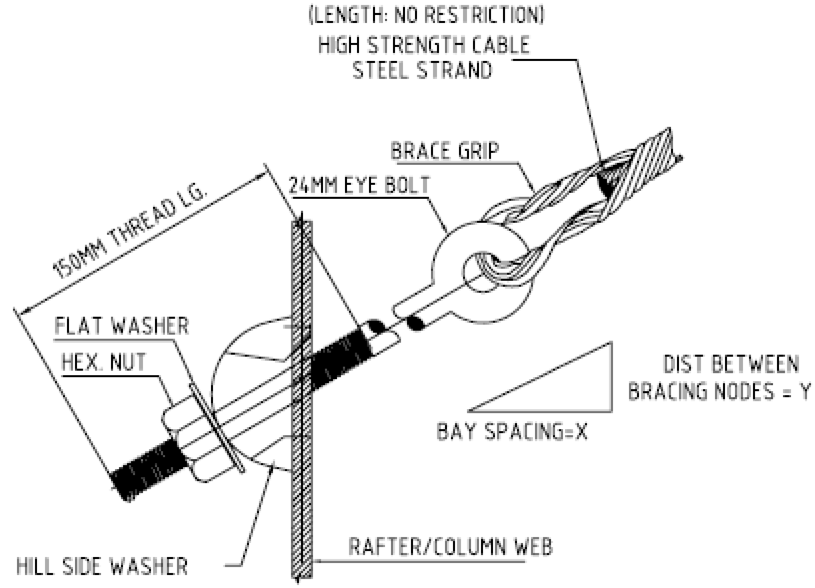
F38 CONNECTION @ RIDGE
AT BEARING FRAME END



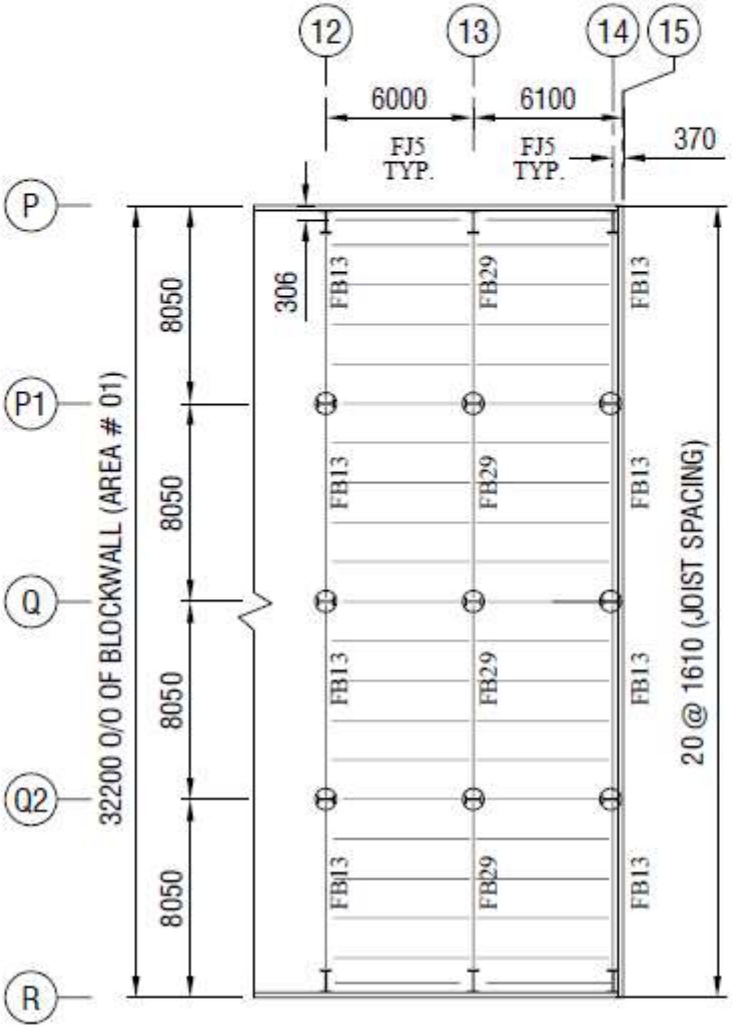
B3 ENDWALL RAFTER CONNECTION
AT ENDWALL RAFTER COLUMN



Q2 CABLE BRACE CONNECTION



Q2 CABLE BRACE CONNECTION

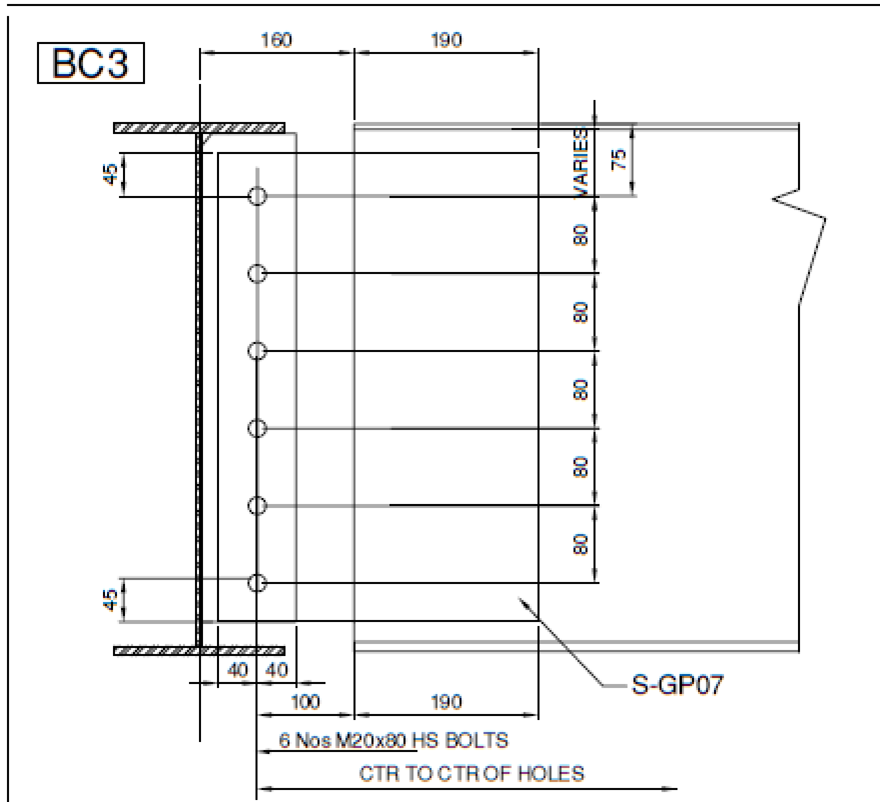


MEZZANINE LAYOUT PLAN

⊖-DENOTES MEZZANINE COLUMN FC6 TYP. U.N.O

MEZZANINE DESIGN SUMMAY (AREA # 01)		
DEAD LOAD	3.00	(KN/M²)
LIVE LOAD	5.00	(KN/M²)
COLLATERAL LOAD	0.00	(KN/M²)
TOTAL LOAD	8.00	(KN/M²)
CONCRETE SLAB THICKNESS	100	MM
TOP OF MEZZANINE FROM FFL	3954	MM (ABOVE FFL)
ROOF PANEL IS MEZZ DECKING PANEL	0.7MM THK (35/167) GALV.	

JOIS		MAXIMUM	CONNECTION	
FJ5				
BEA				
FB				N.
FB1				
FB2				
COLU				SIZE THICK
FC				2



MEZZANINE BEAM TO BEAM CONNECTION

CONCLUDING SUMMARY

- Connections are required when a change of component occurs, at changes in framing directions, and to ensure manageable member sizes.
- Connections must satisfy the requirements of structural behaviour. They should be strong enough to transmit the design loads and at the same time have the intended degree of flexibility or rigidity.
- Connection design has a major influence on the costs of real structures.
- Two types of fasteners are used for connections - welds and bolts.
- Normally welding is applied in the fabrication shop and bolts are used for erection.
- When detailing connections, thought should be given to fabrication practicalities and erection sequence and method.

ADDITIONAL READING

Boston, R.M. and Pask, J.W. 'Structural Fasteners and their Applications', BCSA 1978.

Drawings of bolts of all kinds and photographs of fixings procedures, plus examples of connection design.

Interfaces: Connections between Steel and other Materials, Ove Arup and Partners. Edited by R. G. Ogden, 1994.

Hogan, T.J. and Firkins, A., 'Standardized structural connections', Australian Institute of Steel Construction, 1981, 3rd Ed, 1985.

Presents design models and resistance tables for the main connection types.

Blodgett, O.W., 'Design of welded structures', James F Lincoln Arc Welding Foundation, Cleveland, Ohio, USA, 1972.

Informative and well illustrated reference manual covering all aspects of welded design and construction.

Ballio, G. and Mazzolani, F.M., 'Theory and design of steel structures', Chapman and Hall, London, 1983.

Comprehensive text on theory and design of steel structures. Deals extensively with connections. A detailed treatment of combined loads on fillet welds is of particular interest.

Draft for Development DD ENV 1993-1-1: 1992 Eurocode 3: Design of Steel Structures, Part 1, General Rules and Rules for Buildings.

THANK YOU