Module -1

Connections

- **Module – I…………**
- Types of connections - Design of welded and bolted connections, design of bolted connections using high strength friction grip bolts.

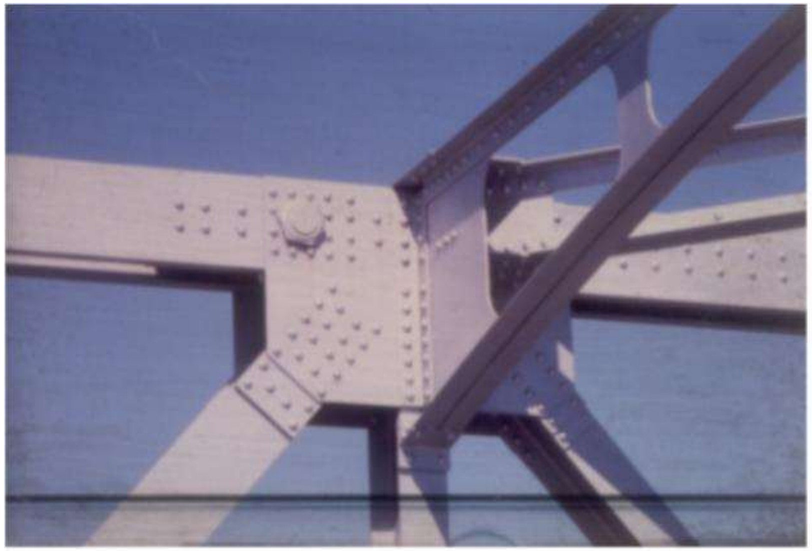
Introduction

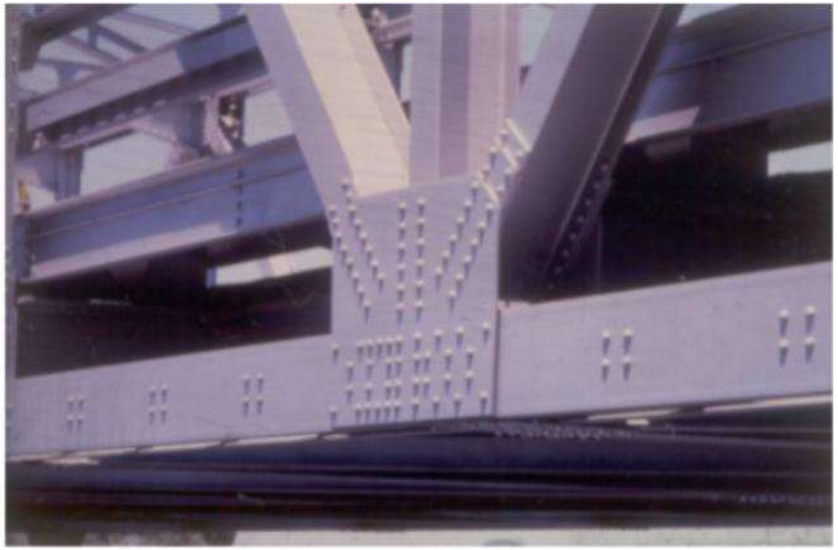
- Necessity for Connections
  - Limited Length of Members
    - Rolling & Transportation Constraints
  - Larger Size of Structures
- Importance of Connection
  - Weakest link
  - Connection failure to be avoided before member failure
    - The full strength of members is to be utilised
    - Connection failure is usually not ductile

Introduction…

- Reliability or Safety of a design depends on
  - Variability of loads
  - Variability of the member strength
  - Variability of Connection Strength
- Larger Uncertainty of Connections is Due to
  - Complexity of Connection Geometry
    - Highly Indeterminate
    - Stress concentration
    - Non-Linearity due to slip, & local yielding
  - Geometric Imperfections
  - Residual Stresses & Strains

COMPLEXITY OF CONNECTIONS





Introduction…

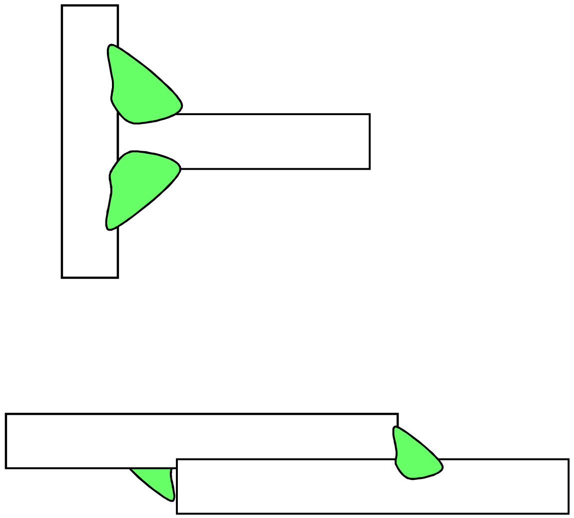
- **RESIDUAL STRESSES & STRAINS**
  - **Differential cooling after hot rolling, gas cutting & welding**
  - **Premature yielding under loading**
  - **Lack of fit in bolted fabrication (Distortions)**

TYPES OF CONNECTIONS

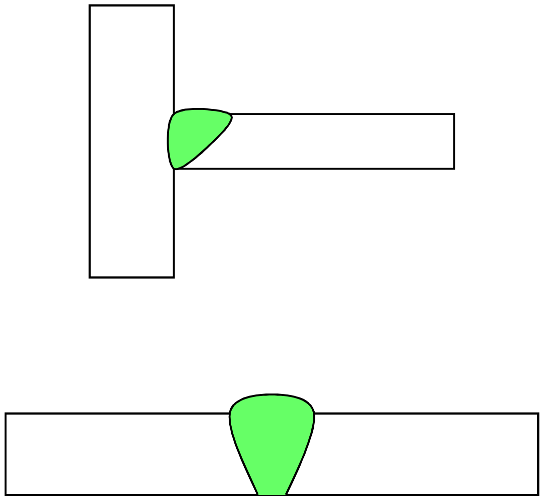
- WELDED CONNECTIONS
  - Fillet welding
  - Butt welding
- BOLTED CONNECTIONS
  - Bearing type - Carbon steel / High strength
  - Friction type - HSFG
- RIVETED CONNECTIONS
  - Mild steel
  - High strength steel

TYPES OF CONNECTIONS

WELDED CONNECTIONS



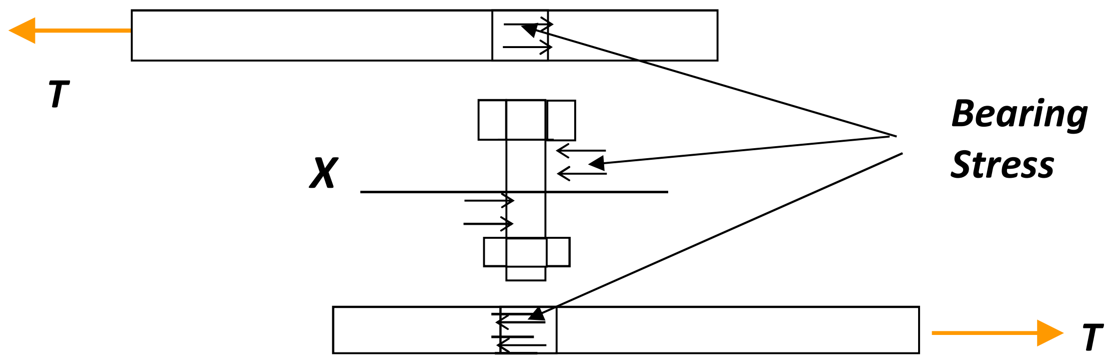
Fillet Welds



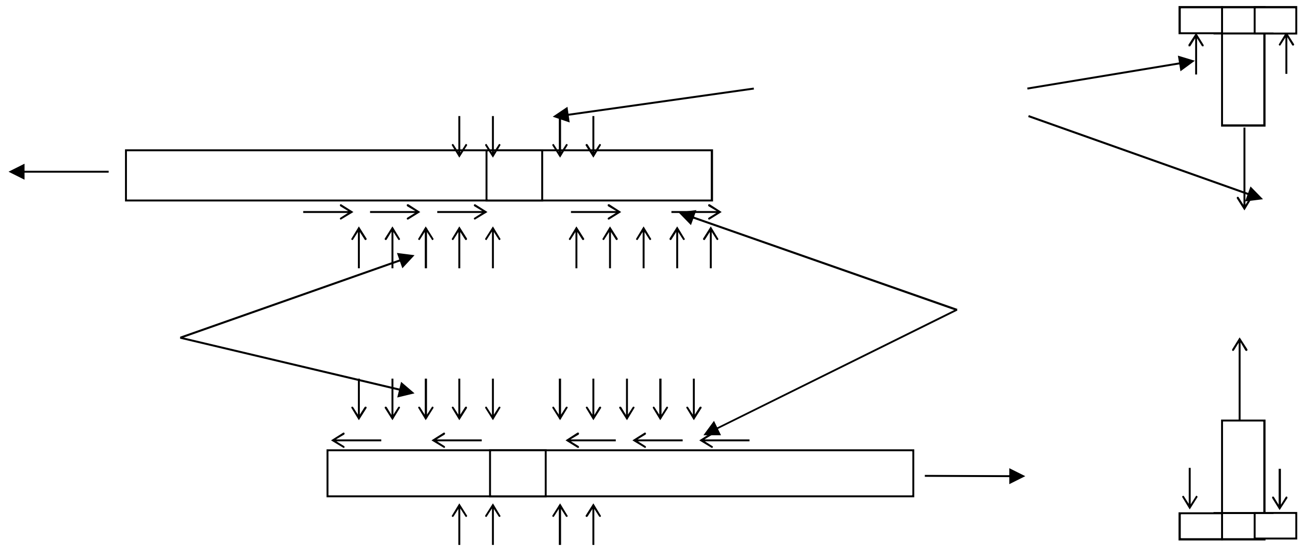
Butt welds

TYPES OF CONNECTIONS

BOLTED CONNECTIONS - Bearing Type



BOLTED CONNECTIONS- Friction Type



Merits & Demerits

- Welded Connections
  - Transfer of forces between elements more direct
  - Requires little additional elements like gussets
  - Shorter length of joints
  - No reduction in member strength due to bolt holes etc.
  - Rigid connections easy to achieve
  - Requires skilled manpower
  - Requires special equipment
  - not easy to achieve at difficult locations
  - less ductile
  - prone to defects & fatigue cracks under cyclic loading

Merits & Demerits

- Bolted Connections -
  - Bearing Type
    - Easy to install even at difficult locations
    - Economical
    - Does not require highly skilled manpower
    - Slip causes flexible joint
    - Joint size larger
  - Friction Type
    - Rigidity of connection
    - Better fatigue performance
    - Expensive due to material & installation labour
    - Requires skilled manpower
    - Requires better inspection

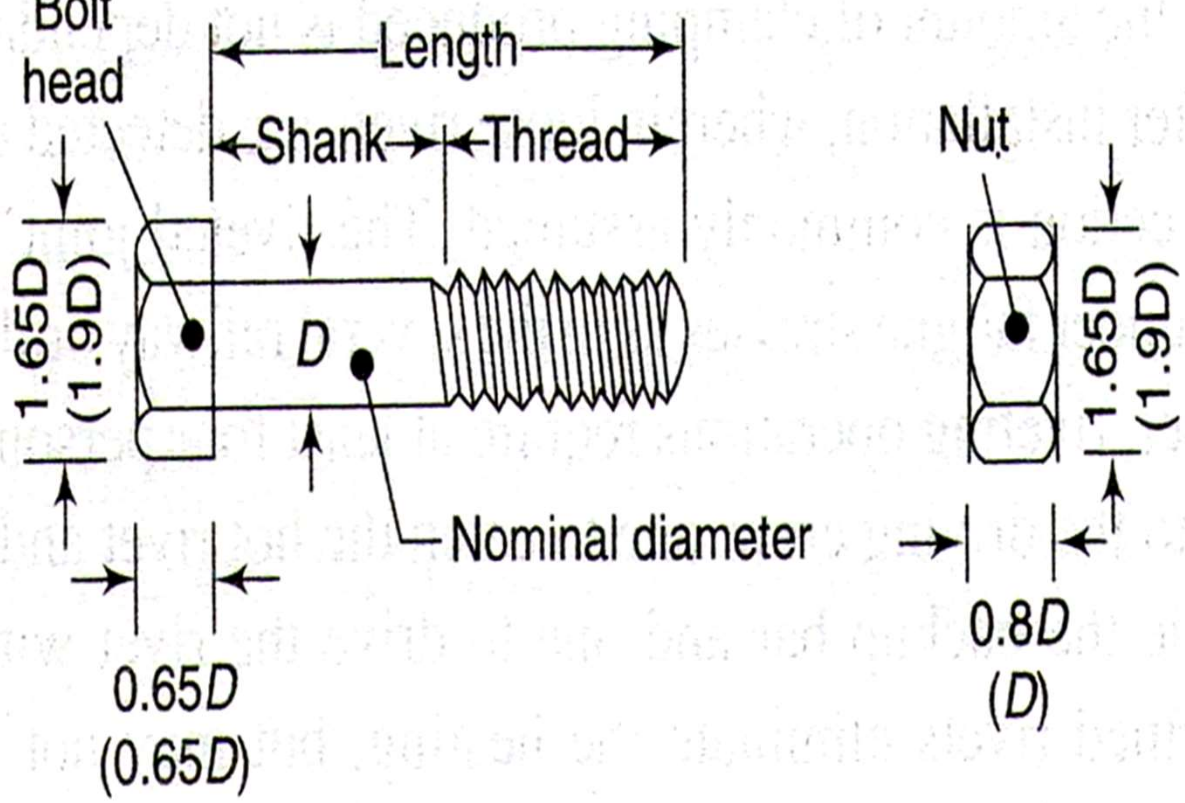
BOLTED CONNECTIONS

Bolted Connection

- A bolt - as a metal pin with a head at one end a shank threaded at the other end to receive a nut.
- Steel washers are usually provided under the bolt as well as under the nut to serve two purposes:
  - to distribute the clamping pressure on the bolted member, and
  - to prevent the threaded portion of the bolt from bearing on the connecting pieces.

- Bolts can be used for making end connections in tension and compression members.
- Used to hold down column bases in position, and as separators for purlins and beams in foundations, etc.

- In order to assure proper functioning of the connection, the parts to be connected must be tightly clamped between the bolt head and nut. If the connection is subjected to vibrations, the nuts must be locked in position



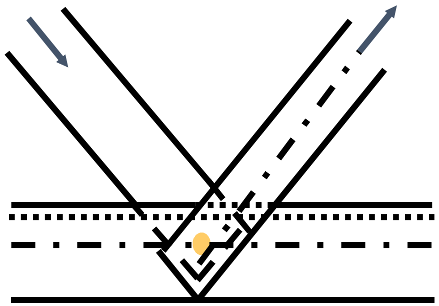
Hexagonal head black bolt and nut. Figures in brackets are for hig
strength bolts and nuts.

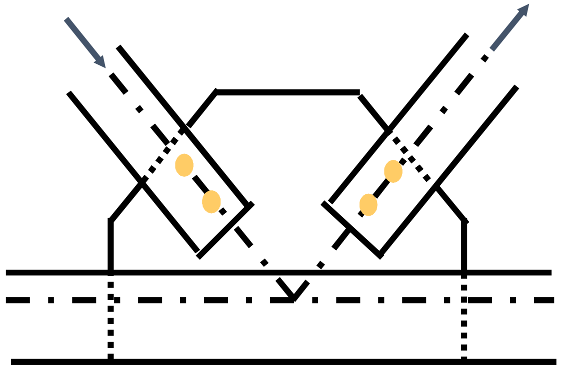
- **Bolted connections**
- **advantages-**
  - The erection of the structure can be speeded up.
  - Less skilled persons are required is cheaper than that of riveted construction because of reduced labour and equipment costs and the smaller number of bolts required resisting the same load.
  - Use of simple tools
  - Noiseless and quick fabrication
  - No special equipment/process needed for installation
  - Fast progress of work
  - Accommodates minor discrepancies in dimensions
- **Disadvantages-**
  - Cost of material is high, about double that of rivets.
  - The tensile strength of the bolt is reduced because of area reduction at the root of the thread and also due to stress concentration.
  - Normally, these are of a loose fit excepting turned bolts and hence their strength is reduced.
  - When subjected to vibrations or shocks, bolts may get loose.

TYPES OF CONNECTIONS

Classification based on type of resultant force transferred

(a)

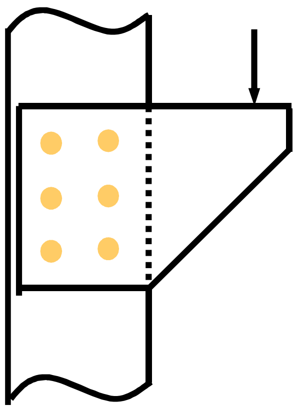


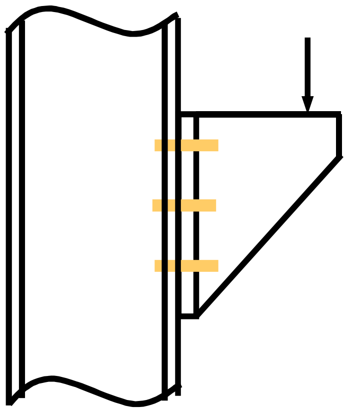


(a)

Concentric Connections

(a)

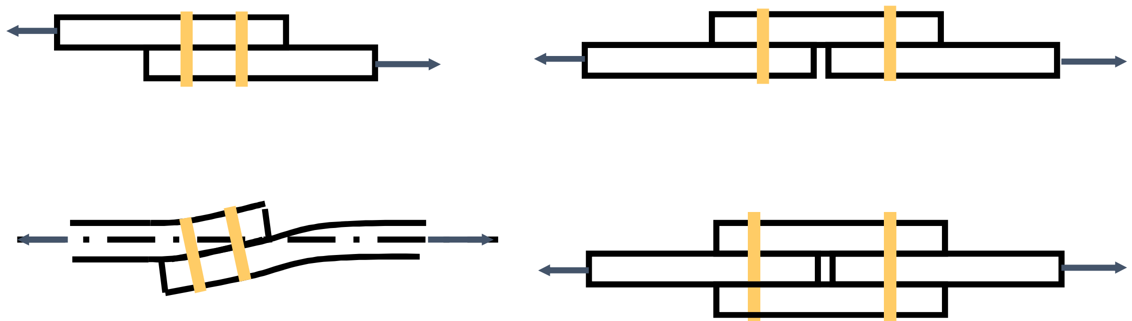




(b)

Moment Connections

Classification based on type of force in the bolts



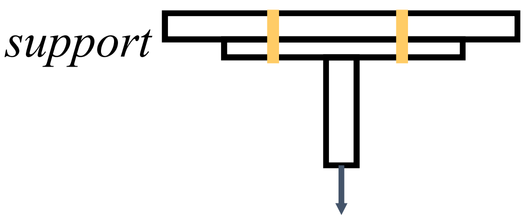
Single shear

Double shear

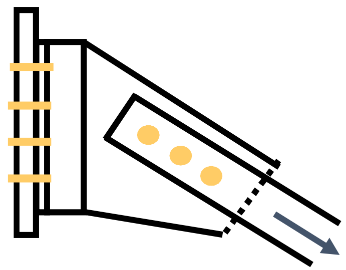
a) Lap Connection ***b) Butt Connection***

Shear Connections

(a)



(b)



Tension Connection and Tension plus Shear Connection

BOLTS AND BOLTING

Bolt Types

Black Bolts:

- usually Gr.4.6,
- made snug tight,
- ductile and cheap,
- only static loads

Turned & Fitted;

- Gr.4.6 to 8.8,
- Close tolerance drilled holes,
- 0.2% proof stress

HSFG Bolts:

- Gr.8.8 to 10.9,
- less ductile,
- excellent under dynamic/fatigue loads

Black Bolts

- Unfinished bolts are also called ordinary, common, rough or black bolts. These are for light structures subjected to static loads and for secondary members such purlins, bracings, roof trusses etc.

- IN STEEL CONSTRUCTION, GENERALLY, BOLTS OF PROPERTY CLASS 4.6 ARE USED.
- CLASS 4.6, THE NUMBER 4 INDICATES 1/100TH OF THE NOMINAL ULTIMATE TENSILE STRENGTH IN N/MM^2 AND THE NUMBER 6 INDICATES THE RATIO OF YIELD STRESS TO ULTIMATE STRESS, EXPRESSED AS A PERCENTAGE.
- ULTIMATE TENSILE STRENGTH OF A CLASS 4.6 BOLT IS 400 N/MM^2 AND YIELD STRENGTH IS 0.6 TIMES 400, WHICH IS 240

- Bolts diameter
  - 5-36 mm
  - M16,M20, M24,M30 commonly used
  - Grade-4.6

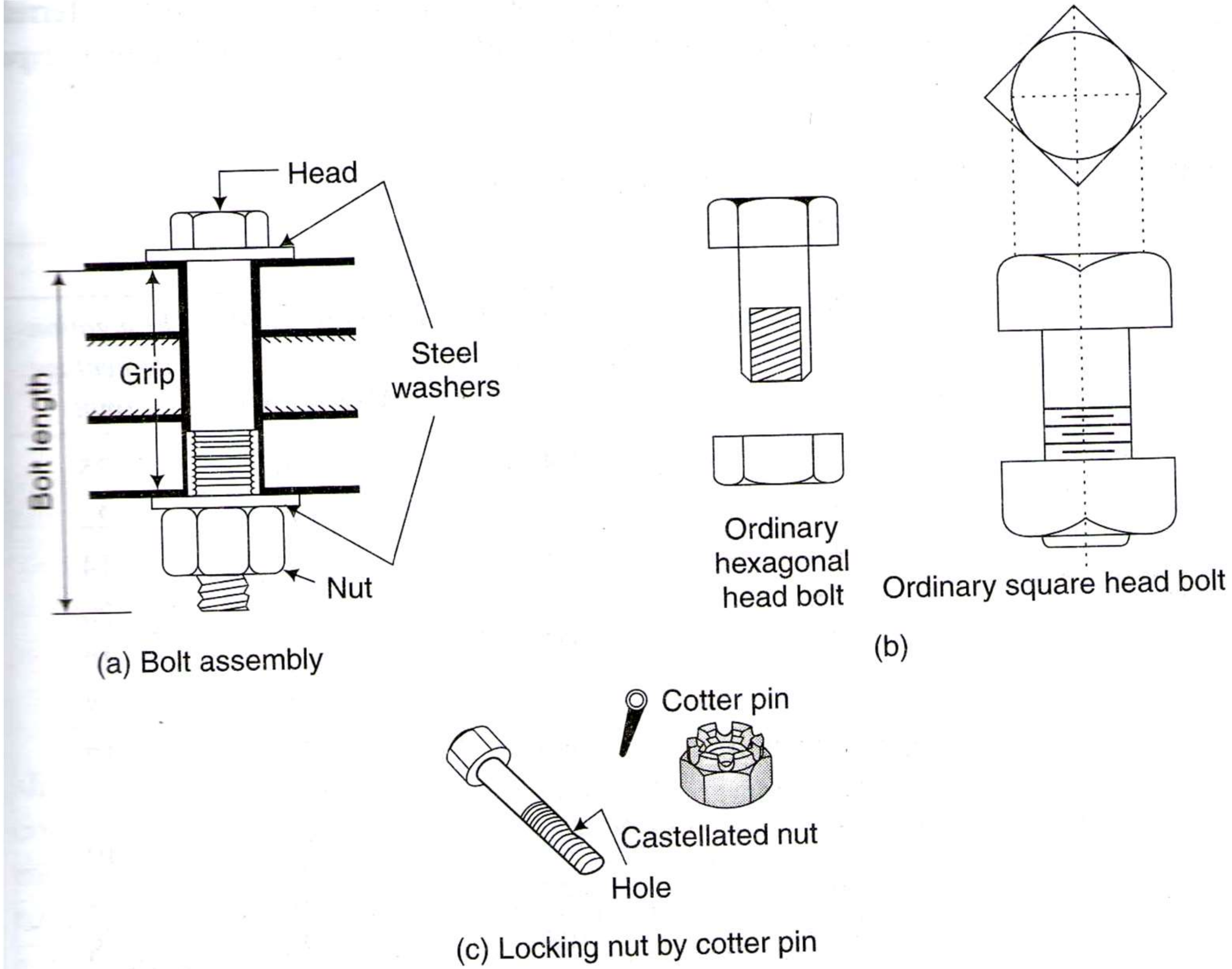


Fig. 4.3 *Ordinary Bolts*

Table 5.1 Tensile properties of fasteners used in steel construction

Specification	Grade/ classification	Properties: Yield stress, MPa (Min)	Ultimate tensile stress, MPa (Min)	Elongation percentage (Min)
IS 1367 (Part 3) (ISO 898) Specifications of fasteners-threaded steel for technical supply conditions	3.6	180	330	25
	4.6	240	400	22
	4.8	320	420	14
	5.6	300	500	20
	5.8	400	520	10
	6.8	480	600	8
	8.8 ($d < 16$ mm)	640	800	12
	9.8	720	900	10
	10.9	940	1040	9
	12.9	1100	1220	8
IS 7557 Specification for steel wire (up to 20 mm) for the manufacture of cold forged rivets	Annealed condition	160	330–410	30
	As-drawn condition	190	410–490	20

High strength bolts

- High strength bolts are made from bars of medium carbon heat-treated steel and from alloy steel.
- Tightened until they have very high tensile stresses, two or more times that of ordinary bolts, so that the connected parts are clamped tightly together between the bolts and nut heads; this permits loads to be transferred primarily by friction and not by shear. The surfaces in contact must fee free of mill scale, rust, paint, grease, etc., which would prevent solid contact between the surfaces and lower the slip factor.
- Due to this friction, the slip in the joint, which is there in joints with ordinary bolts, is eliminated.
- Friction is developed by applying a load normal to the joint by tightening these bolts to proof load. That is why these bolts are also known as friction-type bolts.

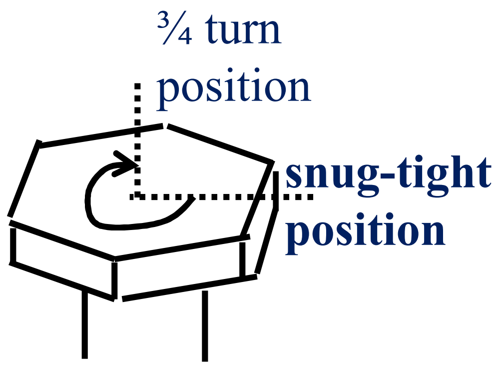
- Care must be taken that bolts are tightened up to the required tension, otherwise slip may occur at service loads and the joint will act as an ordinary bolted joint.
- The direct shank tension can be achieved by either part-turning method or torque control method or by using load-indicating washers
- If the bolts are tightened by the part-turning method, also called turn of nut method, the nut is made snug and is tightened half turn more by hand wrenches, then the washers are not required
- In the torque control method a power operated or hand-torque wrench is used to deliver a specified torque to the nut.
- In the load indicator washer & bolt method, the washers have projections which squash down as the bolt is tightened.

- HSFG BOLTS
  - Diameter- M16, M20, M24, M30 NORMALLY USED
  - (Range- 16 to 36 mm)
  - Grade- 8.8, 9.8, 10.9

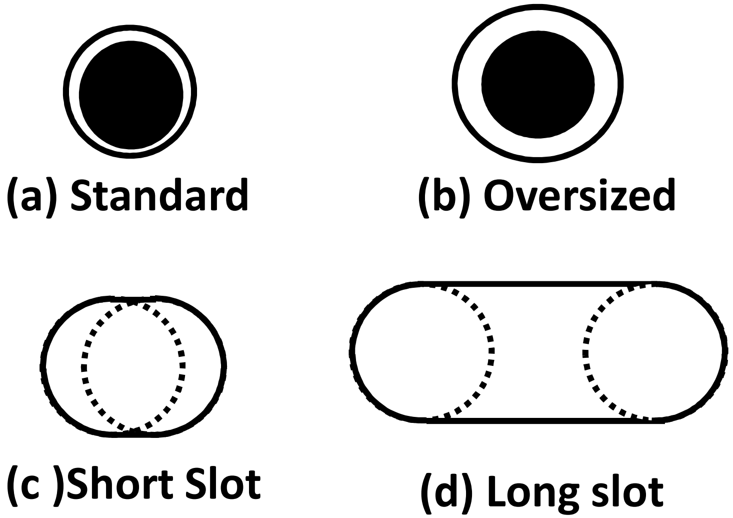
  - Parallel Shank –
    - ❑No slip at Service load, but slip permitted at ultimate load
  - Wasted Shank –
    - ❑No slip at Service or Ultimate load

TIGHTENING OF HSFG BOLTS

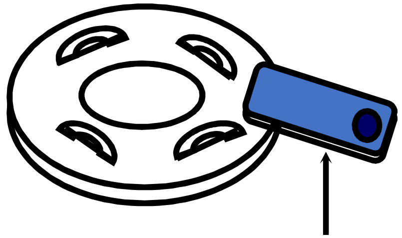
1) Turn-of-nut Tightening
2) Calibrated Wrench Tightening
3) Alternate Design Bolt Installation
4) Direct Tension Indicator Method



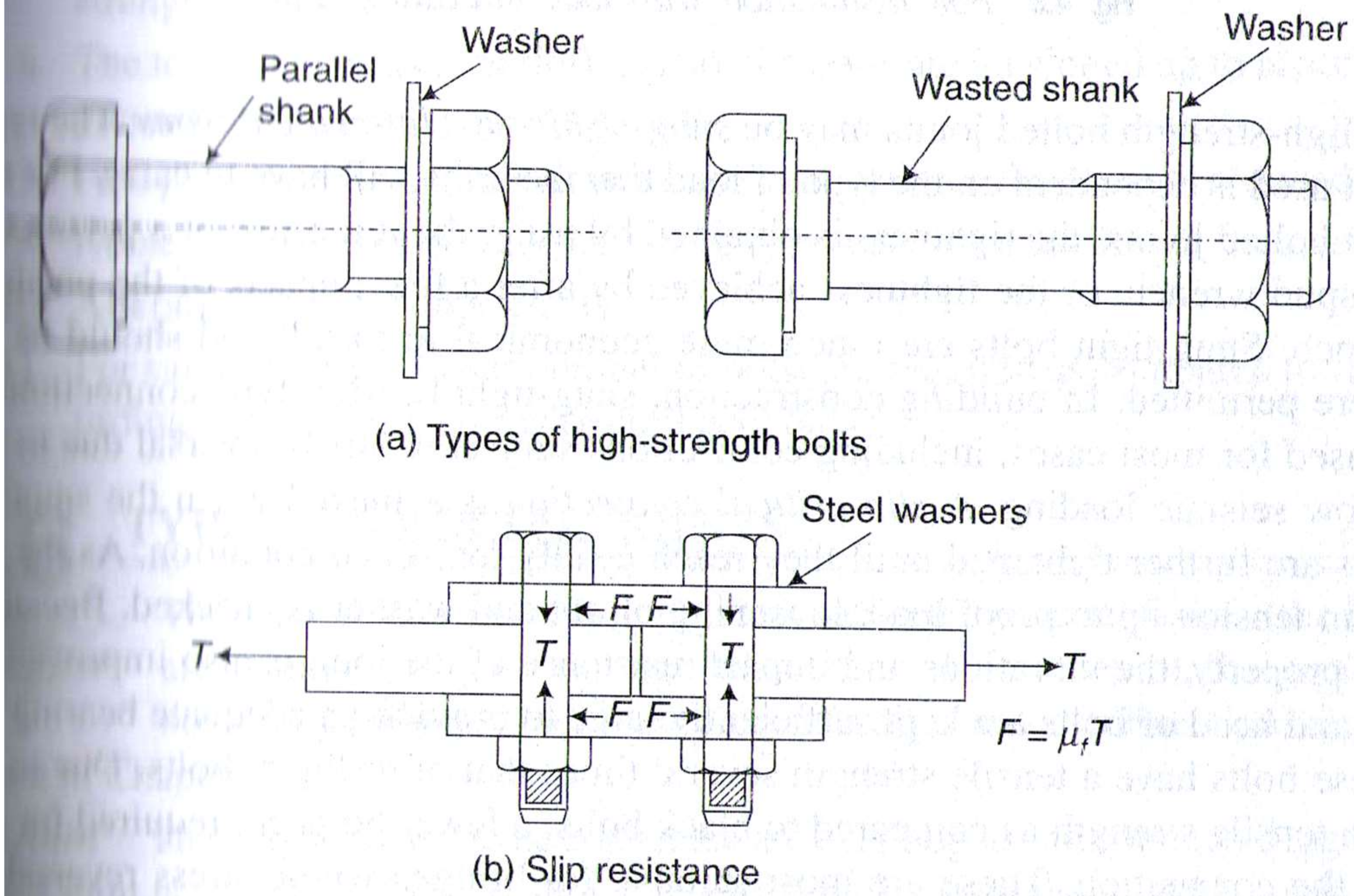
Tightening of HSFG bolts



Hole types for HSFG bolts



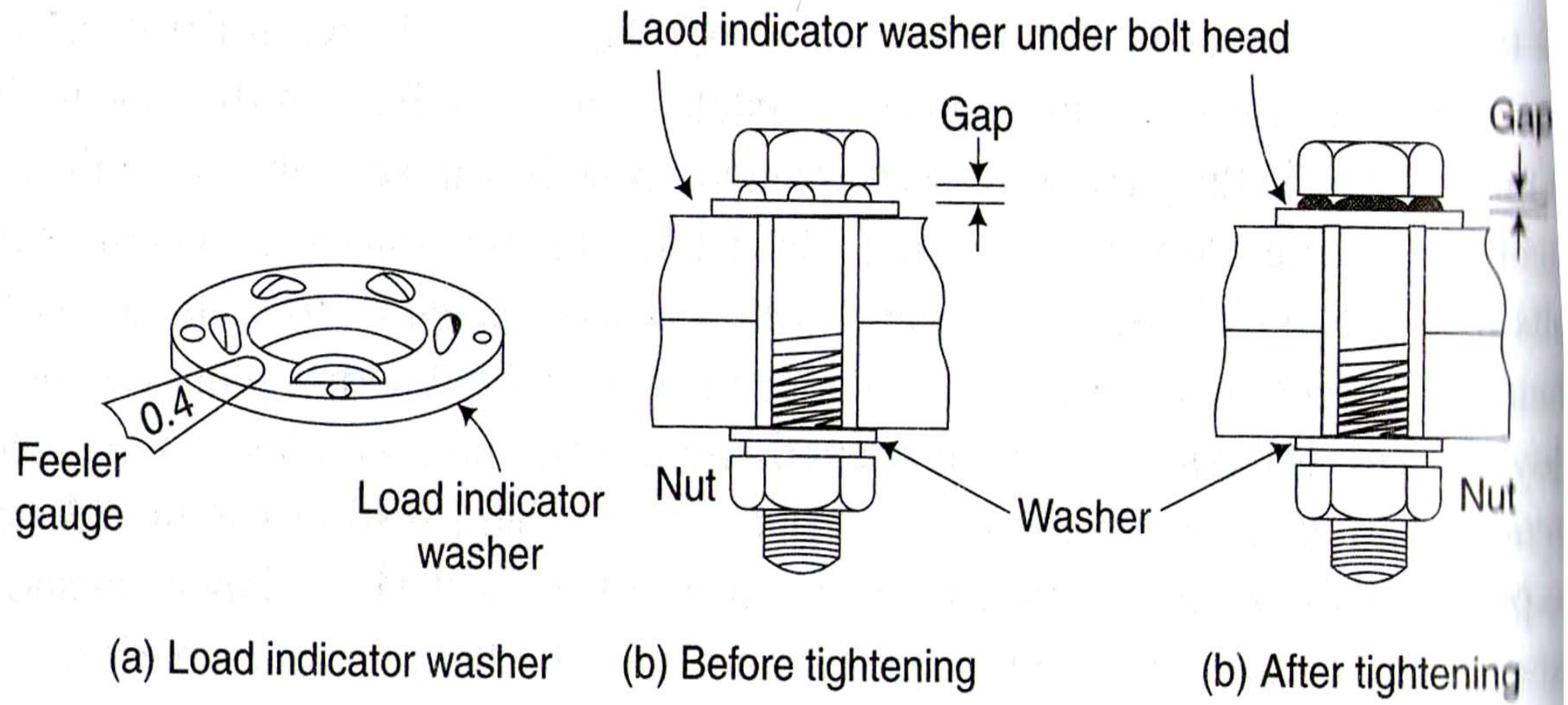
Feeler gauge



(a) Types of high-strength bolts

(b) Slip resistance

High-Strength Bolted Connection



Bolt installation with load indicating washers

Table 5.2 Dimensions of hexagon head black bolts (grade 4.6) as per IS 1364 (Part 1)

Bolt size (d), mm	Head diagonal (e), mm	Head thickness (k), mm	Thread* length (b), mm	Pitch of thread, mm	Washer (IS 5370) Outer diameter, mm	Washer (IS 5370) Inner diameter, mm	Washer (IS 5370) Thickness, mm
(12)	20.88	8	20	1.75	24	14	3
16	26.17	10	23	2.0	30	18	3
20	32.95	13	26	2.5	37	22	3
(22)	35.03	14	28	2.5	39	24	3
24	39.55	15	30	3.0	44	26	4
(27)	45.20	17	33	3.0	50	30	4
30	50.85	19	35	3.5	56	33	4
36	60.79	23	40	4.0	66	39	5

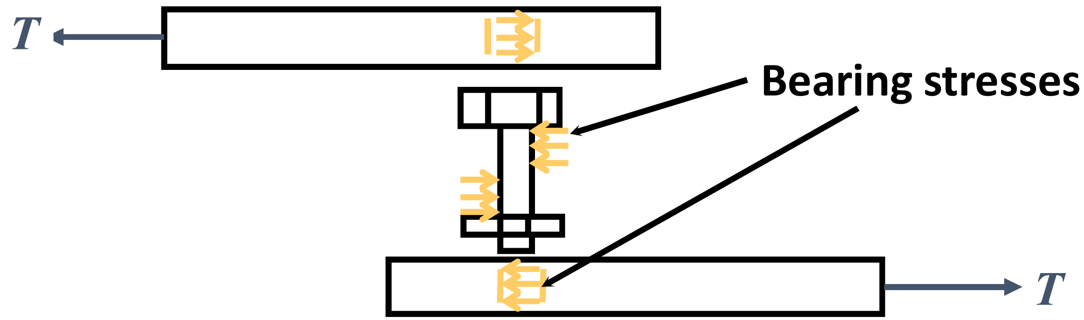
*For length $l \leq 125$ mm. For $125 < l \leq 200$, b is 6 mm more and for $l > 200$, b is 19 mm more.
Sizes in brackets not preferred.

- When shear acts on thread
- Net Area of bolt = $\pi/4 \cdot (d-0.9382\ p)^2$
- p – pitch of thread in mm
- d – bolt diamater in mm

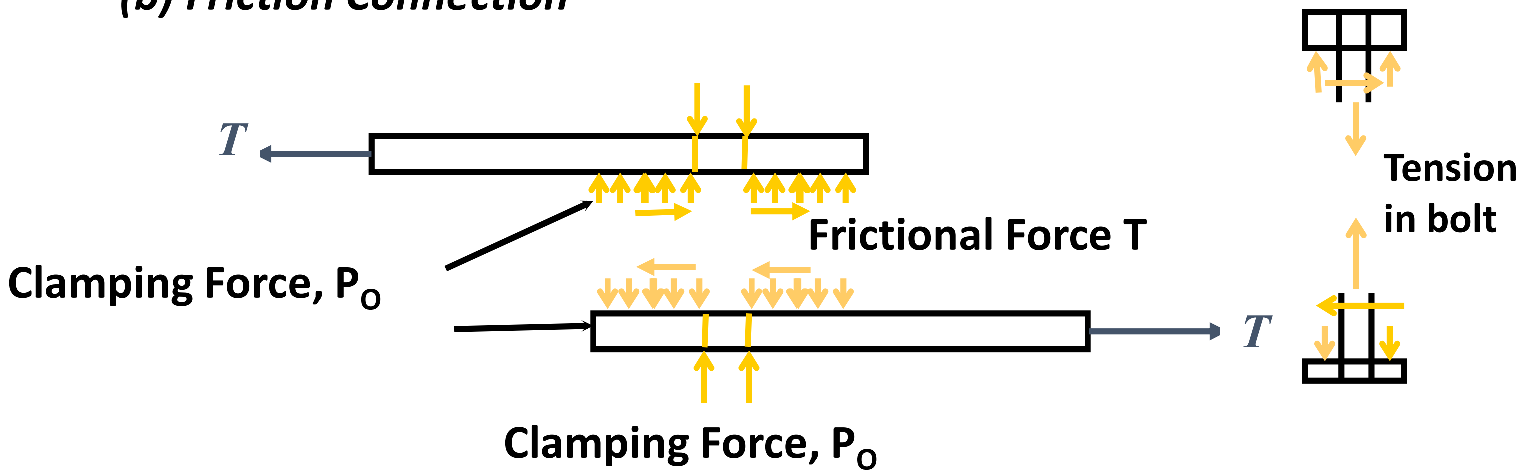
- Approximately An = $0.78 \cdot \pi/4 \cdot (d)^2$

FORCE TRANSFER MECHANISM

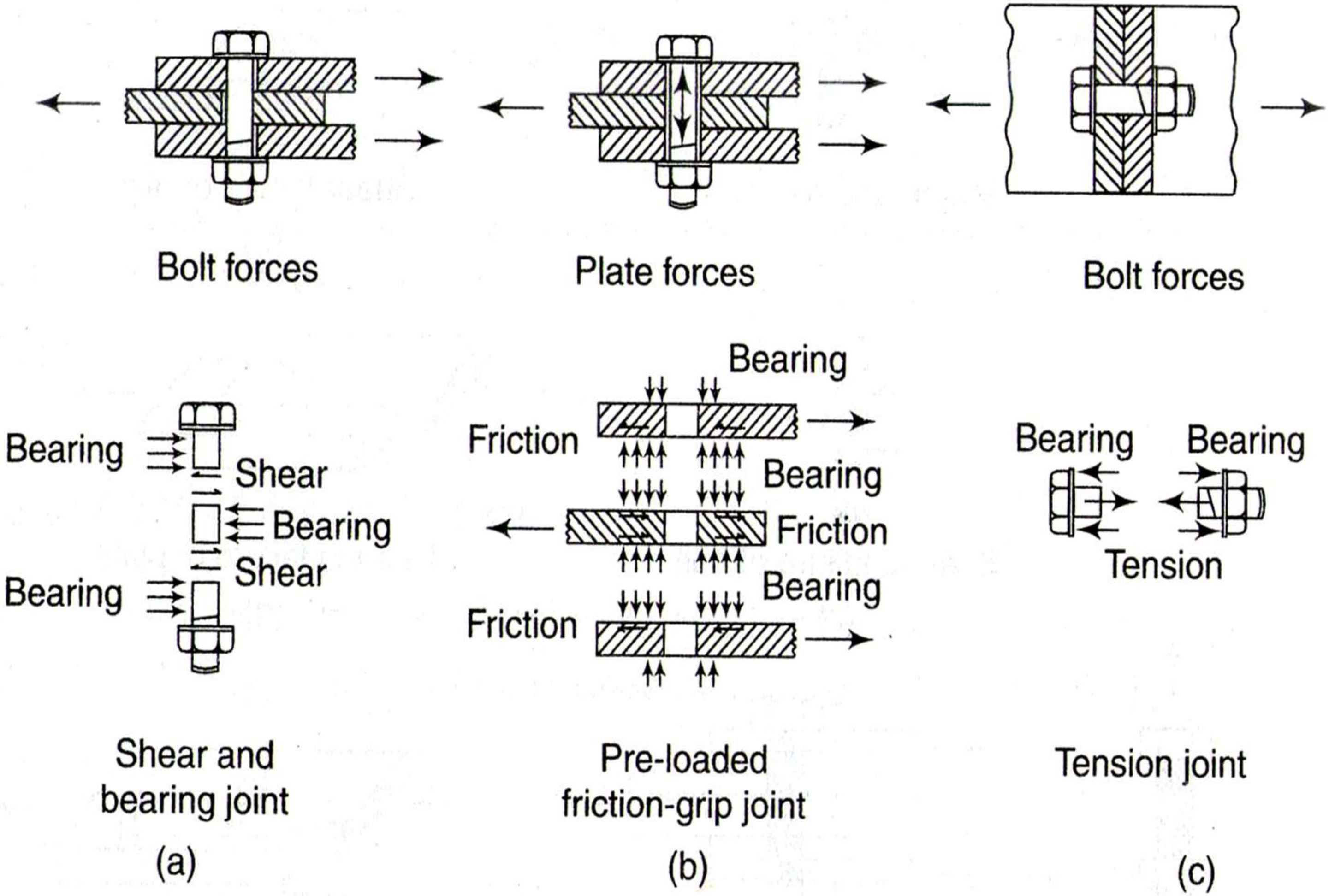
(a) Bearing Connection



(b) Friction Connection



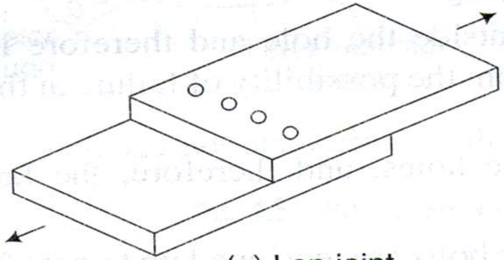
Bolt Shear Transfer – Free Body Diagram



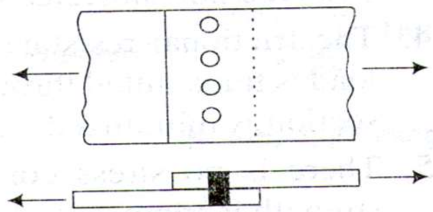
Force transmission through bolts (Trahair et al. 2001)

Advantages of HSFG Bolts

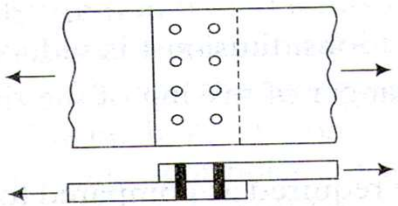
- HSFG bolts do not allow any slip between the elements connected, especially in close tolerance holes thus providing rigid connections.
- Due to the clamping action, load is transmitted by friction only and the bolts are not subjected to shear and bearing
- Due to the smaller number of bolts, the gusset plate sizes are reduced
- Deformation is minimized
- Since HSFG bolts under working loads do not rely on resistance from bearing, holes larger than usual can be provided to ease erection and take care of lack of fit. Thus the holes may be standard, extra large, or short/long slotted
- Noiseless fabrication, since the bolts are tightened with wrenches
- The possibility of failure at the net section under the working loads is eliminated.
- *Alterations can be done easily*



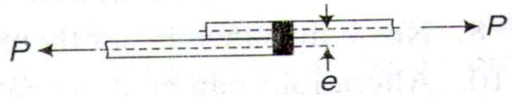
(a) Lap joint



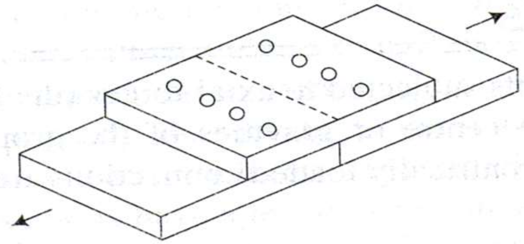
(b) Single bolted lap joint



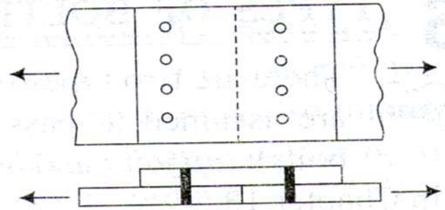
(c) Double bolted lap joint



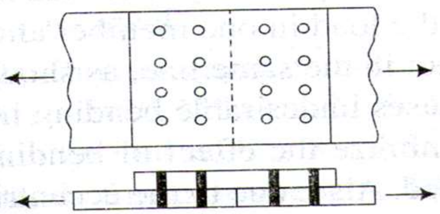
(d) Eccentricity in lap joint



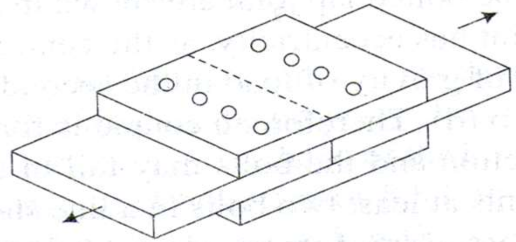
(e) Single-cover butt joint



(d) Single-cover single bolted butt joint

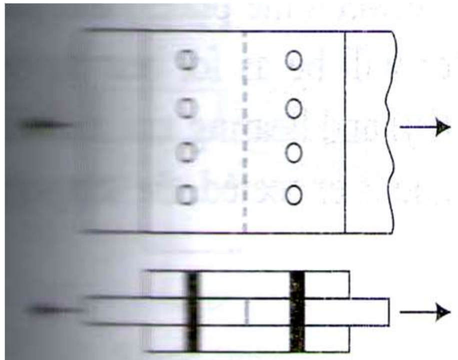


(g) Single-cover double bolted butt joint

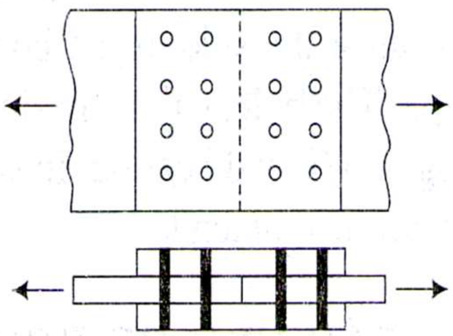


(h) Double-cover single bolted butt joint

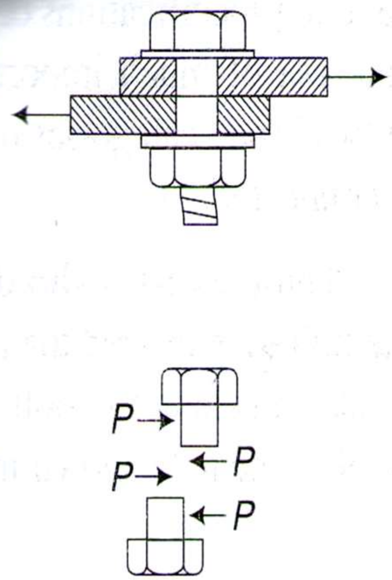
Table Cont



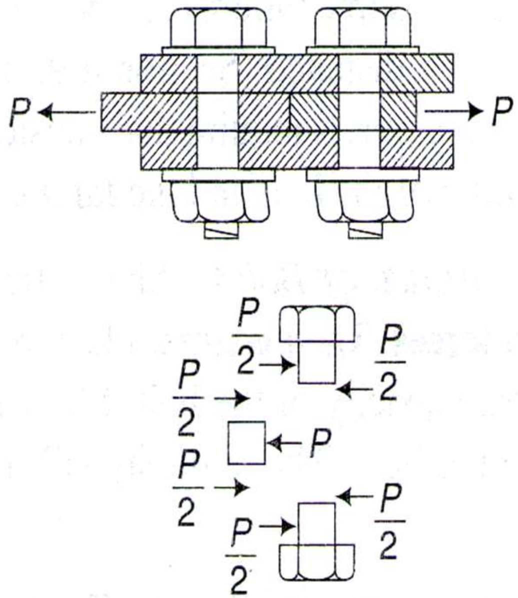
(i) Double-cover single bolted butt joint



(j) Double-cover double bolted butt joint



(k) Lap joint, bolt in single shear



(l) Butt joint, bolt in double shear

Types of bolted joints

Failure of Bolt

- Two broad categories of failure:
  - failure of the bolt and the failure of the parts being connected

- Shear Failure of Bolts: Shear stresses are generated when the plates slip due to applied forces. The maximum factored shear force in the bolt may exceed the nominal shear capacity of the bolt. Shear failure of the bolt takes place at the bolt shear plane.

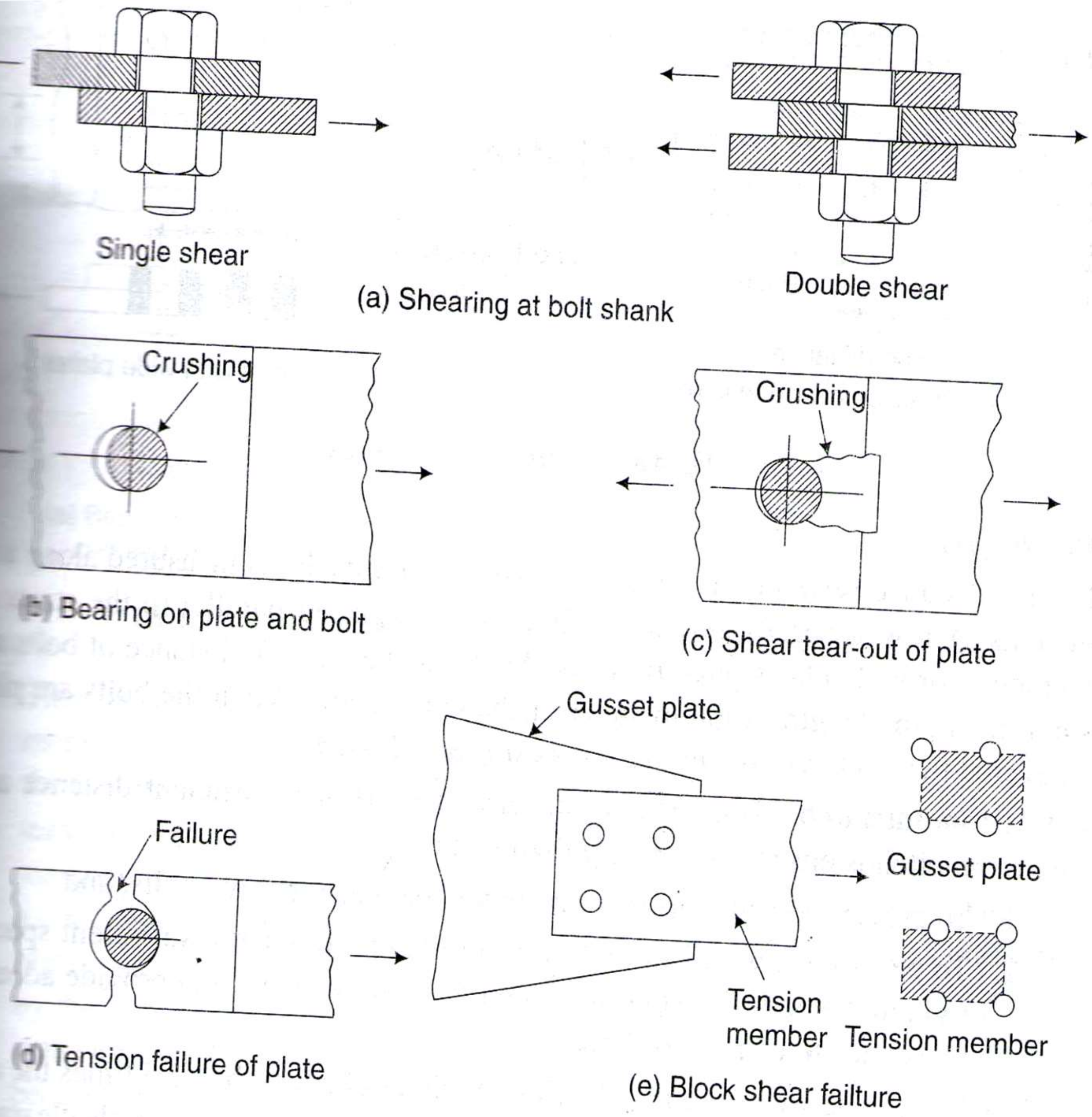
- Bearing Failure of Bolts- The bolt is crushed around half circumference. The plate may be strong in bearing and the heaviest stressed plate may press the bolt shank Bearing failure of bolts generally does not occur in practice except when plates are made of high strength steel.

- Bearing Failure of Plates - When an ordinary bolt is subjected to shear forces, the slip takes place and bolt comes in contact with the plates. The plate may get crushed, if the plate material is weaker than the bolt material.

- Tension Failure of Bolts- Bolts subjected to tension may fail at the stress area, if any of the connecting plates is sufficiently flexible additional prying forces induced in the bolts must also be considered.

- Tension or Tearing Failure of Plates -occurs when the bolts are stronger than the plates. Tension on both the gross area and net effective area must be considered

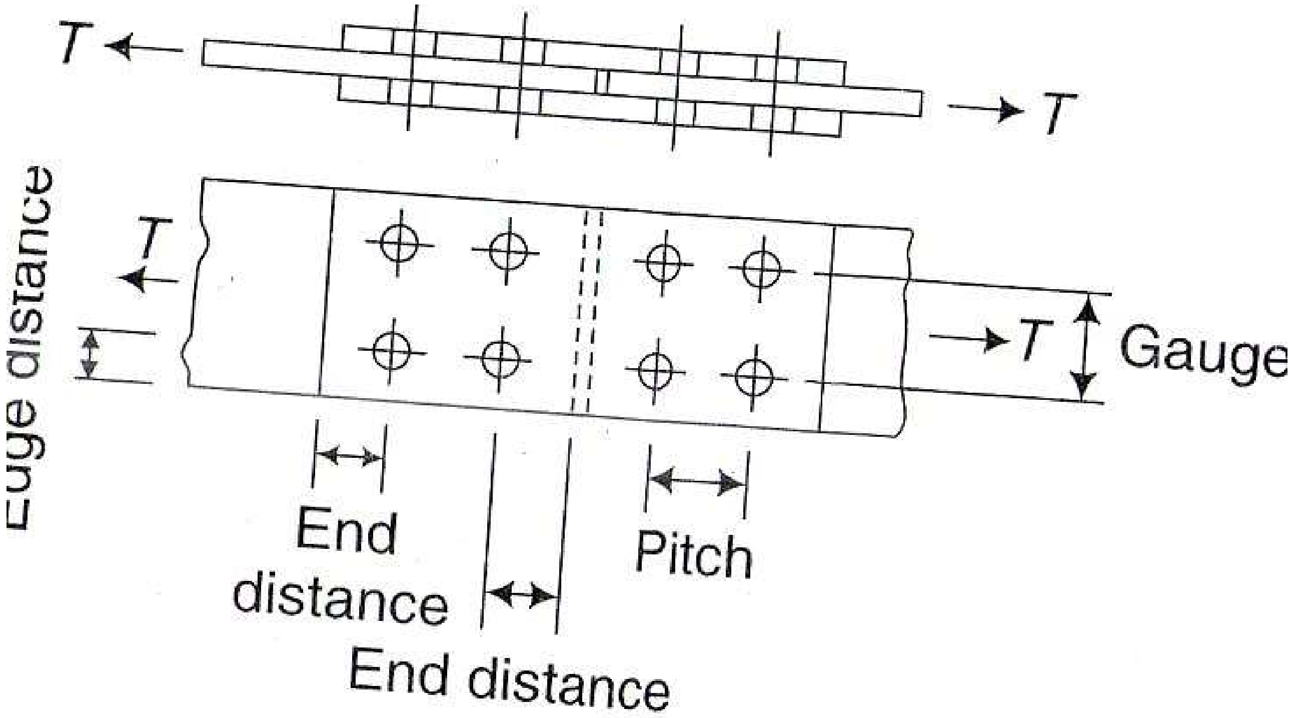
- Block Shear Failure Bolts may have been placed at a lesser end-distance than required causing the plates to shear out which, however, can be checked by observing the specifications for end-distance
- May occur when a block of material within the bolted area breaks away from the remainder area. The possibility of this increases when high strength bolts are used.



Failure modes of bolted joints

Specifications for Bolted Joints

- Diameter of Bolts
- Spacing of bolt holes
  - Pitch, Edge distance
- *Minimum Spacing Cls.10.2.2*
  - *To prevent bearing failure of members btwn 2 bolts*
  - *To permit efficient installation of bolts*
- The distance between centre of fasteners shall not be less than 2.5 times the nominal diameter of the fastener.
- Cls.10.2.3 *Maximum Spacing*
  - *To reduce the length of connection & gusset plate*
  - *To have uniform stress in bolts*
- The distance between the centres of any two adjacent fasteners shall not exceed 32t or 300 mm, whichever is less, where *t* is the thickness of the thinner plate.


T
T
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Edge distance
Gauge
End
distance
End distance
Pitch

- The distance between the centres of two adjacent fasteners (pitch) in a line lying in the direction of stress, shall not exceed 16t or 200 mm, whichever is less, in tension members and 12t or 200 mm, whichever is less, in compression members; where *t* is the thickness of the thinner plate.

- 10.2.4 *Edge and End Distances*
  - *To prevent the failure of plate in tension*
  - *To prevent the bulging out of plate opposite of hole*
- Edge distance is the distance at right angles to the direction of stress from the centre of a hole to the adjacent edge.
- End distance is the distance in the direction of stress from the centre of a hole to the end of the element.
- In slotted holes, the edge and end distances should be measured from the edge or end of the material to the centre of its end radius or the centre line of the slot, whichever is smaller. In oversize holes, the edge and end distances should be taken as the distance from the relevant edge/end plus half the diameter of the standard clearance hole corresponding to the fastener, less the nominal diameter of the oversize hole.
- The minimum edge and end distances from the centre of any hole to the nearest edge of a plate shall not be less than 1.7 times the hole diameter in case of sheared or hand-flame cut edges; and 1.5 times the hole diameter in case of rolled, machine-flame cut, sawn and planed edges.

- Tacking Bolts
- Tacking or *stitch bolts* are used to make the sections act as whole, and to prevent buckling in compression members, when two or more sections are in contact.
- As per cls. 10.2.5

Design Equations as per code

- V_{db} is the design strength of the bolt taken as the smaller of the value as governed by
  - shear, V_{dsb} and bearing, V_{dpb}
  - Tearing of plate

Efficiency of Joint

$$\frac{\text{Strength of Bolted Joint / Pitch Length}}{\text{Strength of solid plate / pitch length}} \times 100$$

FAILURE OF CONNECTIONS

Ordinary Bolts

Shearing of Bolts	**Cls.10.3.3**
- **Long Joint Cls.10.3.3.1**	
- **Large grip Joint**	**Cls.10.3.3.2**
- **Packing Plates**	**Cls.10.3.3.3**
Bearing on Bolts	**Cls. 10.3.4**
Tension Capacity of Bolt	**Cls.10.3.5**

Friction Type Bolt

Shear force	**Cls.10.4.3**
Tension Resistance	**Cls.10.4.5**
Prying Force	**Cls.10.4.7**

- Tension Capacity of Plate
- Yielding of Gross section Cls. 6.2
- Design strength of rupture of critical section
  - Plates Cls.6.3.1
- Design Strength due to Block shear Cls.6.4

$$A_n = \left[b - nd_h + \sum_i \frac{p_{si}^2}{4g_i} \right] t$$

where

b, t = width and thickness of the plate, respectively,

d_h = diameter of the bolt hole (2 mm in addition to the diameter of the hole, in case the directly punched holes),

g = gauge length between the bolt holes, as shown in Fig. 5,

p_s = staggered pitch length between line of bolt holes, as shown in Fig. 5,

n = number of bolt holes in the critical section, and

i = subscript for summation of all the inclined legs.

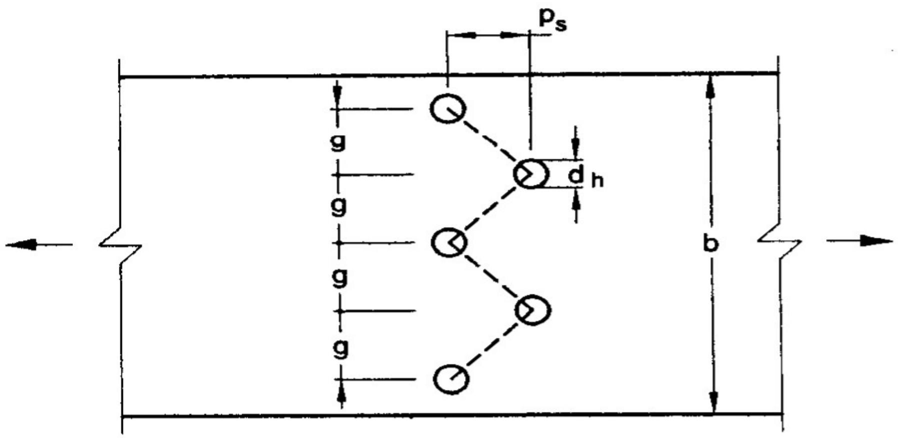
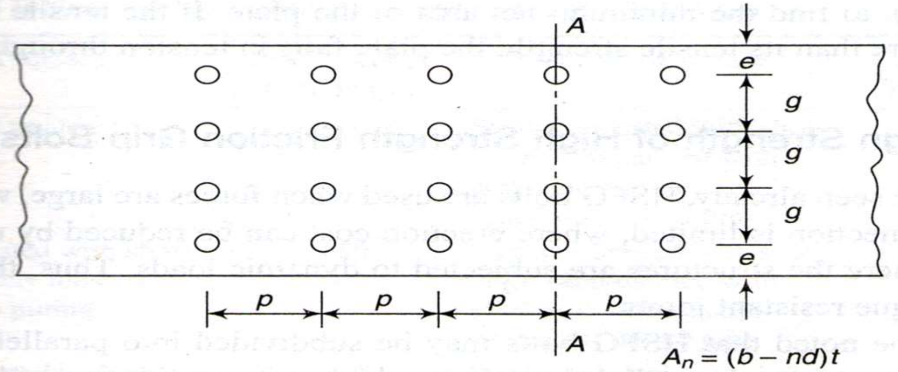
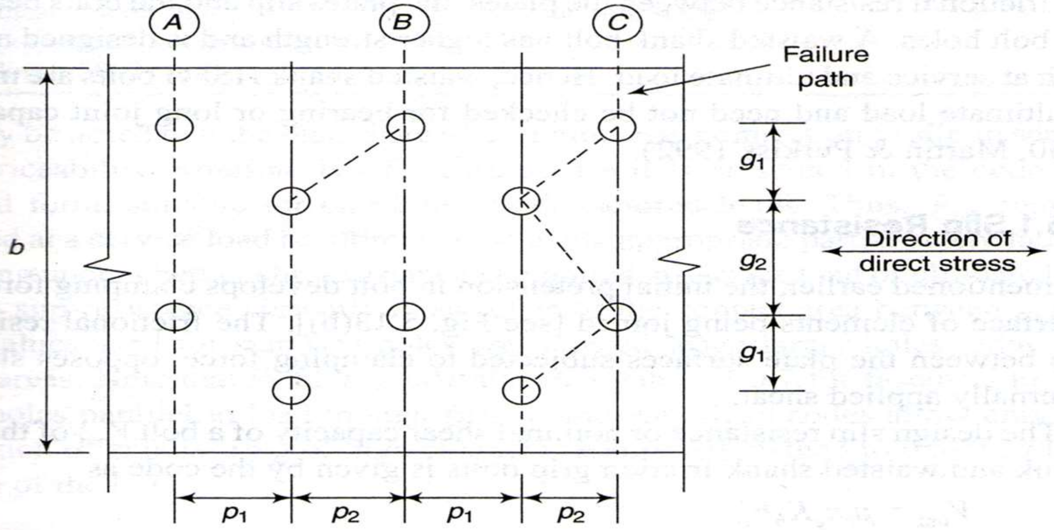


FIG. 5 PLATES WITH BOLTS HOLES IN TENSION



Chain of holes in rows

(a)



Staggered holes

(b)

On line *A*:

$A_n = t[b - 2d_h]$

On line *B*:

$A_n = t[b - 3d_h + 0.25p_2^2/g_1]$

On line *C*:

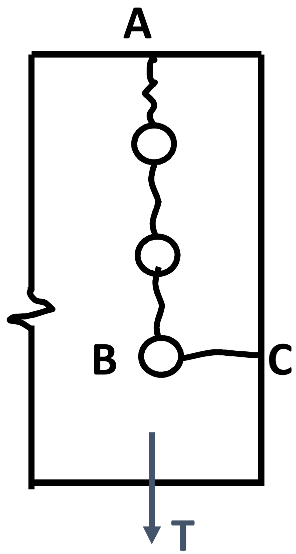
$A_n = t[b - 4d_h + 0.5p_2^2/g_1 + 0.25p_2^2/g_2]$

where d_h is the hole diameter

Tension capacity of plates

BLOCK SHEAR FAILURE

Capacity=Shear Capacity of AB + Tension Capacity of BC



Block Shear

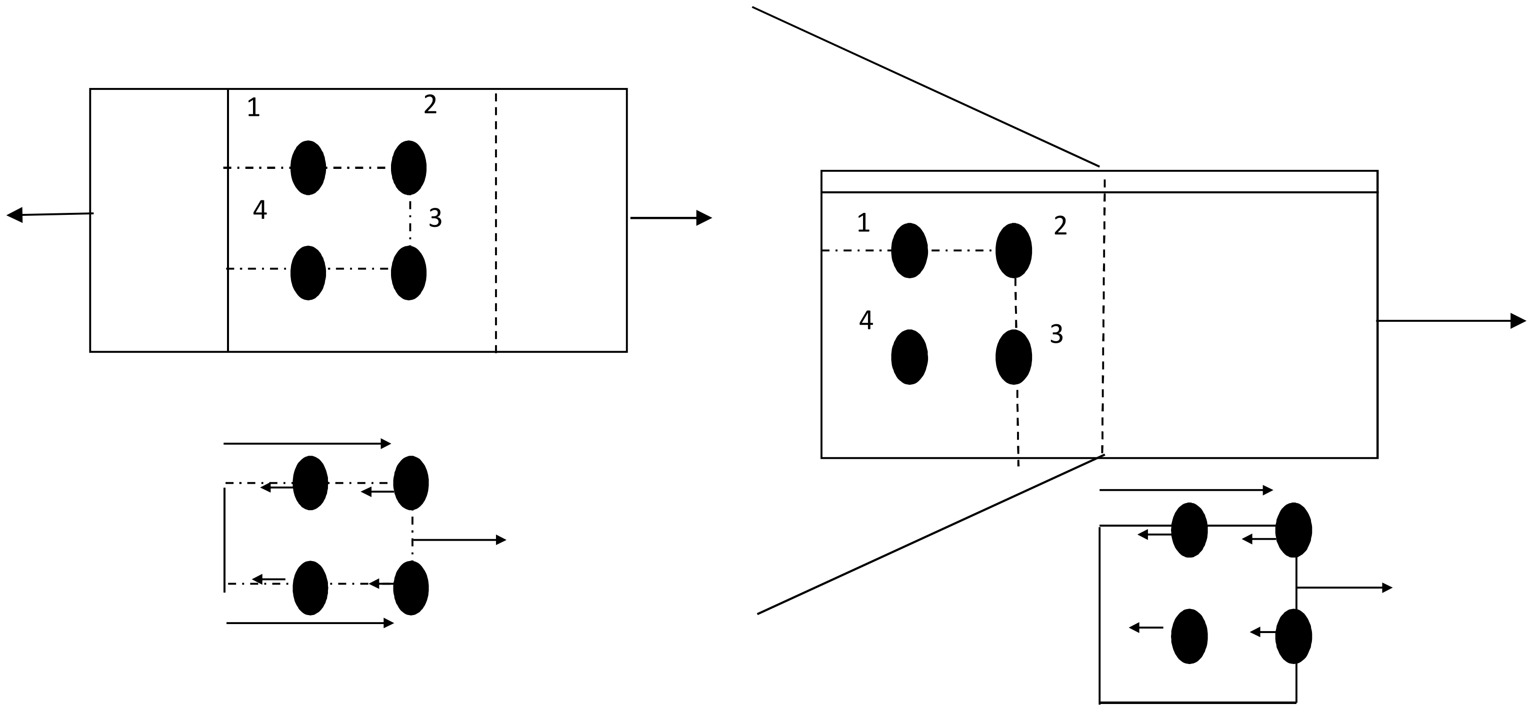


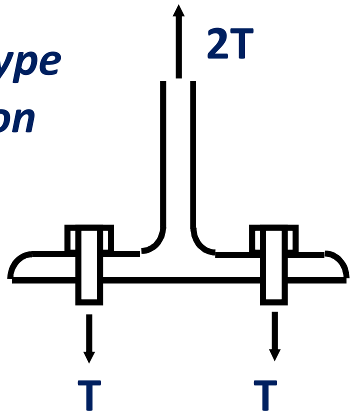
FIG 6.3 BLOCK SHEAR FAILURE

Prying Effect

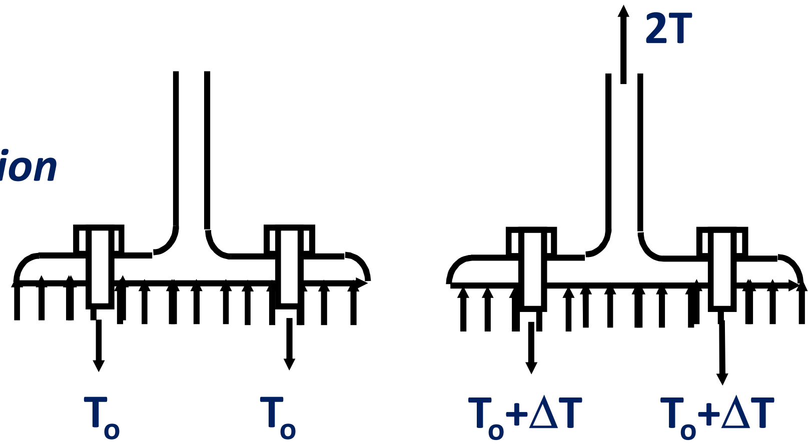
- In moment resisting, beam to column connections the bolt has to transfer load by direct tension
- When plates are less stiffer, then there is a tendency for plate to get lifted up
- To counteract the bending, a force is developed at end of plate Q . This reaction is in direction of applied force. This force is Prying force
- Prying forces are mainly due to flexibility of connected plates
- When Q is significant it should be added to tension in bolt
- If (T + Q) > Strength of bolt, then thickness of plate has to be increased.

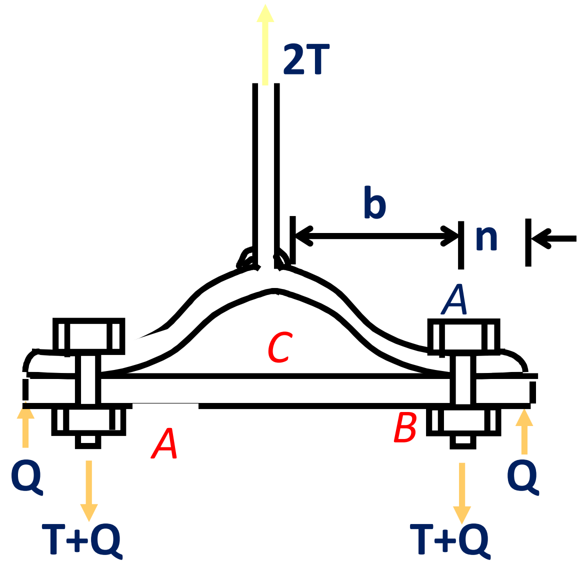
BOLTS UNDER TENSION AND PRYING EFFECT

Bearing type connection



(b) HSFG Connection





(d) Prying Effect

- Equation for Prying force in Is code is approximately
- $Q = Te. Lv/(2.Le)$ (neglecting small term in brackets)

- Thickness of T- flange is determined so that it does not yield.

- $Ma = \mathbf{Q . Le} = (Te. Lv/ 2. Le) . Le = \mathbf{Te. Lv/2}$
- $Mc = Te . Lv – Q . Le = Te . Lv - Te . Lv/2 = Te . Lv/2$

- $Ma = Mc = Te. Lv/2 = Mp$

- Equation for Prying force in Is code - Thickness t is the thickness of End Plate . (Thickness of T- flange plate)

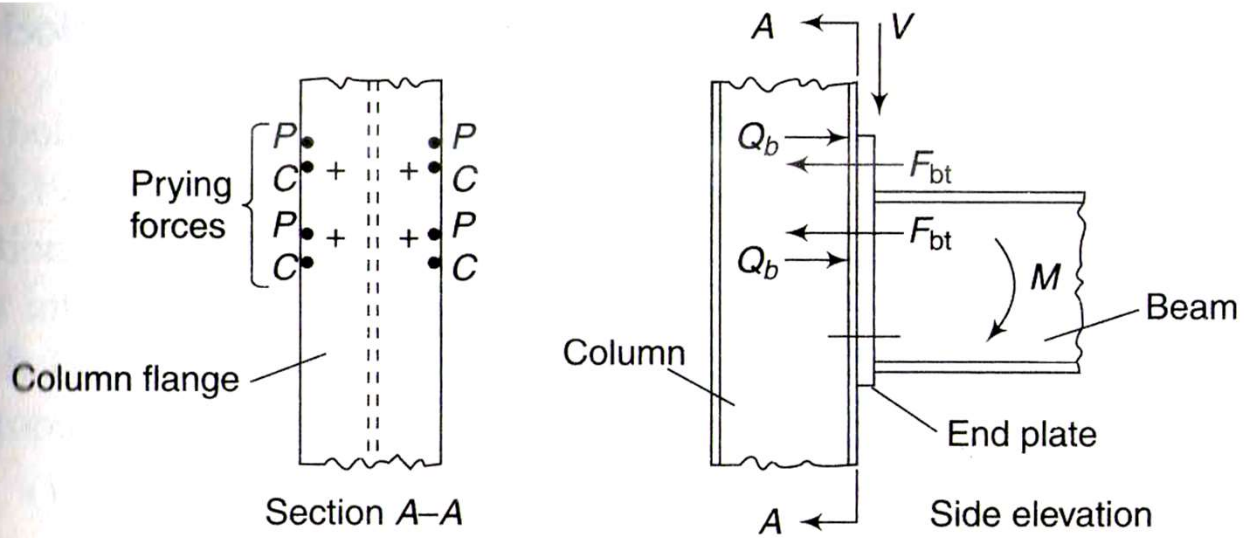
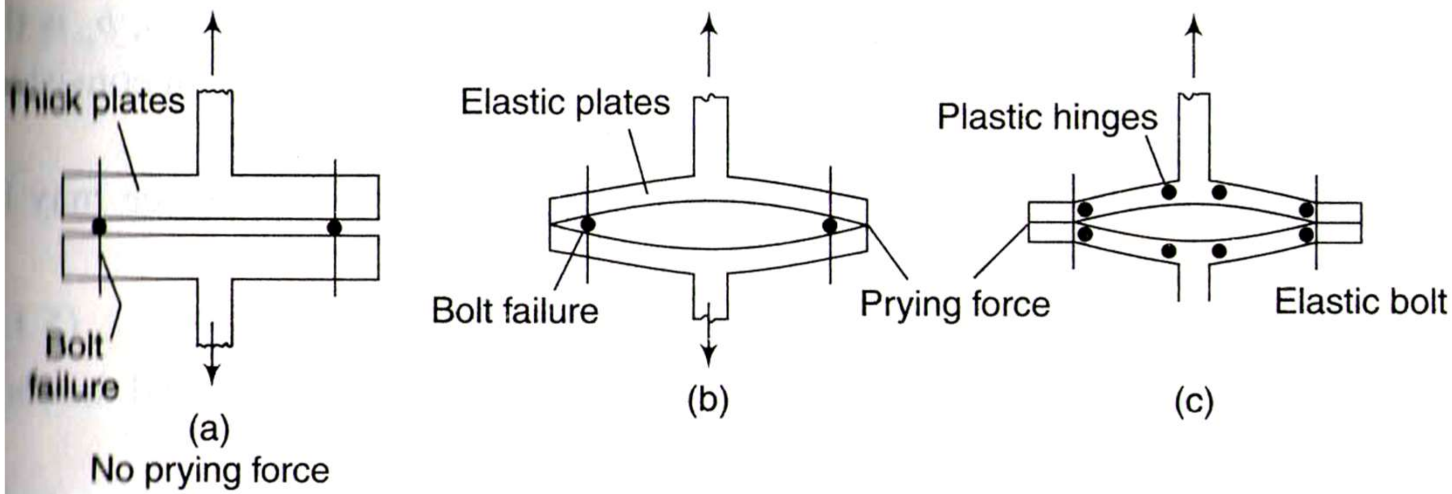
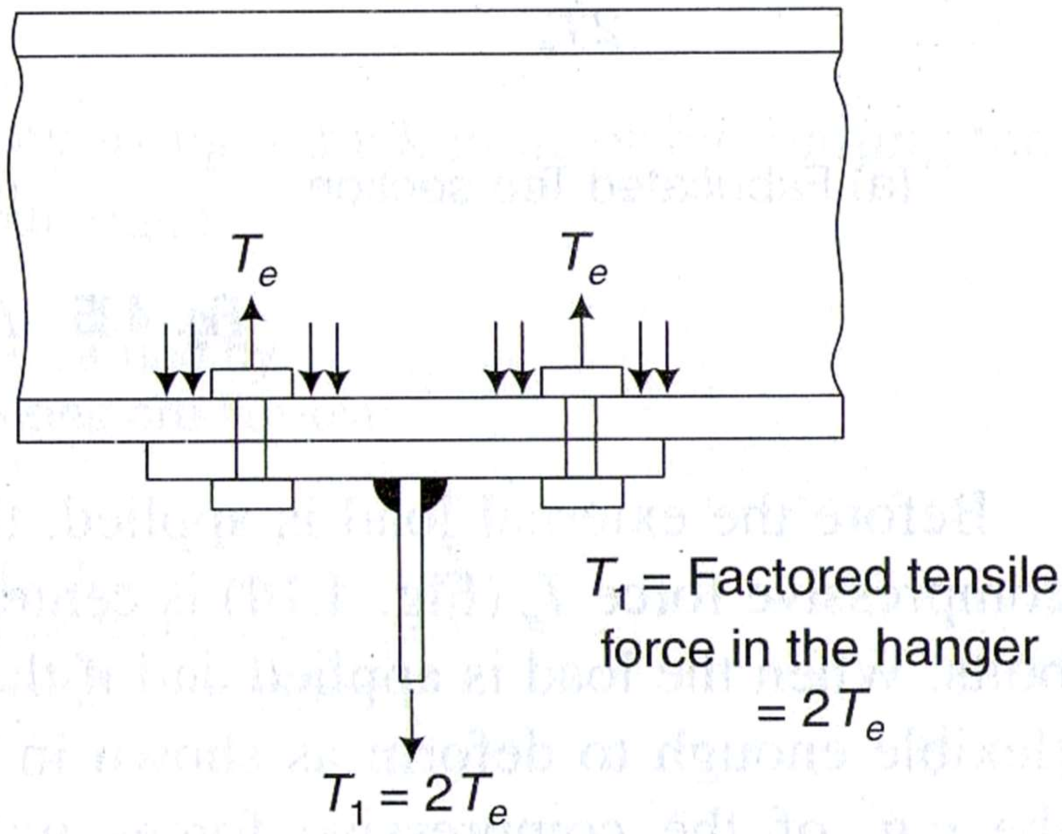


Fig. 5.15 Prying forces in a beam-to-column connection



Failure modes due to prying forces

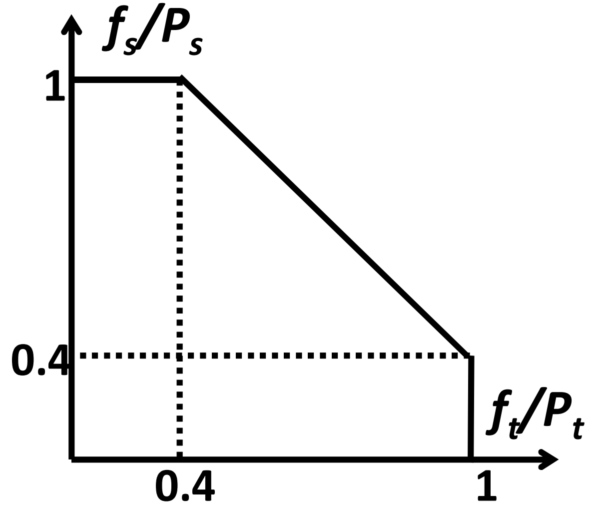


Hanger connection

COMBINED SHEAR AND TENSION

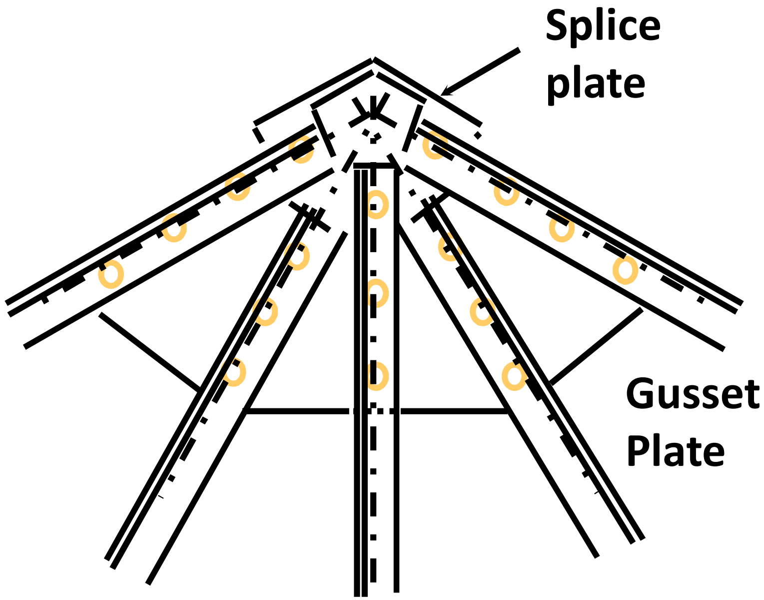
Bearing Bolts Cls. 10.3.6

HSFG Bolts Cls. 10.4.6

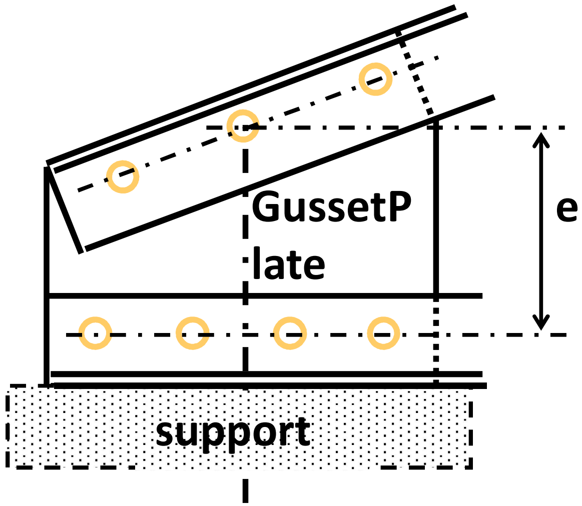


Shear and Tension Interaction Curve

TRUSS CONNECTIONS



(a) Apex Connection



(b) Support connection

Truss Connections

- **Shear Lag**

  - Plate when subjected to tension are subjected to shear deformation near edges . (Tensile stress near to zero at edges)
  - Shear stress produced in material gradually transfer tension at edges to central axis of plate.
  - Transfer of stresses take place in the length of member approximately equal to its width
  - Beyond this length tensile stresses are assumed to be uniformly distributed over the whole section of plate.
  - Transmission of tension at edges to full width by shear stress is Shear lag

- **Shear Lag…**

- In case of I- beam, internal transfer of force from flange to web is by shear
- In case of angles transfer of forces from on leg to other is by shear

- Efficiency of Joint

  = Capacity of Joint / Gross area Yield Strength

WELDED CONNECTIONS I

INTRODUCTION

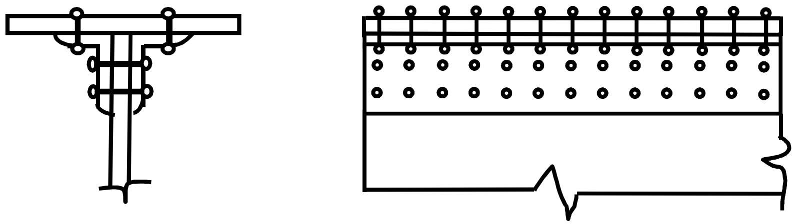
Efficient and direct way of connecting is by welding

Metallurgical bond by heat or pressure or both

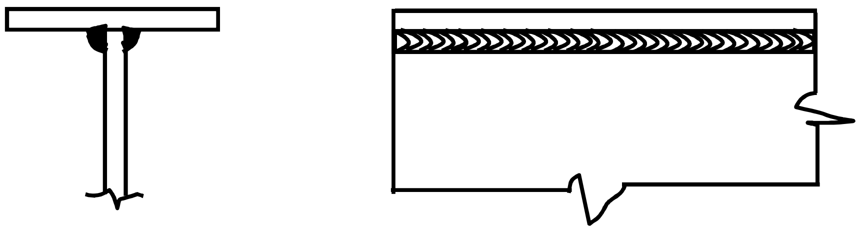
Advantages of welding

- **Direct transfer of stress - minimum weight , efficiency**
- **Less fabrication**
- **Economy - 15% saving in weight in bridges,less labor**
- **Neat appearance**
- **More rigid**

COMPARISON OF APPEARANCE OF RIVETED AND WELDED PLATE GIRDERS



(a) Bolted girder section

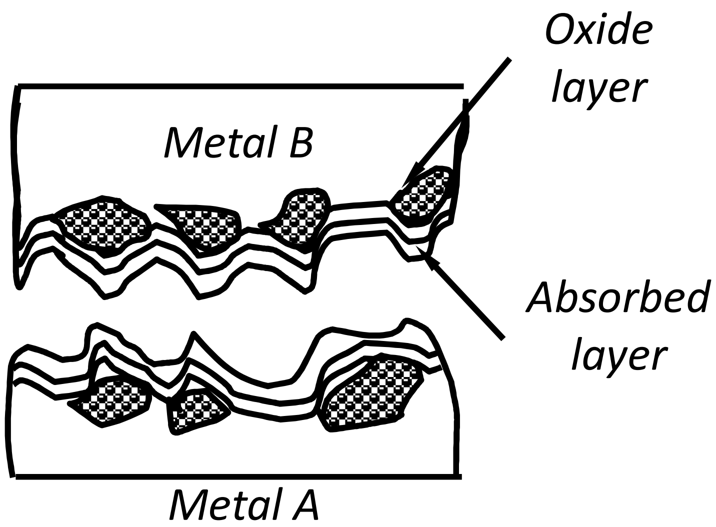


(a) Welded girder section

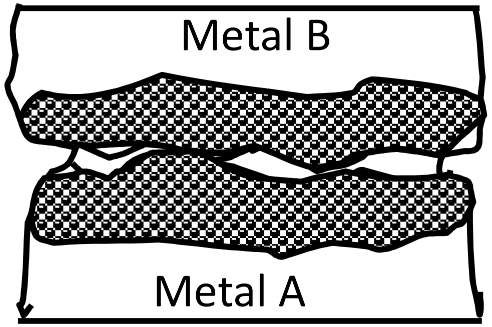
FUNDAMENTALS OF WELDING

- Tendency of atoms to bond and form metallic bonds
- Inter diffusion between the materials
- Diffusion in liquid , solid or mixed state
- Real surfaces are not smooth and clean
- Welding process needs some form of energy-Heat or pressure or both
- Heat alone applied - eg. Gas Tungsten Arc welding, Shielded metal arc welding, Submerged arc welding
- Pressure alone - eg. Cold welding, Roll welding
- Both pressure and heat - eg. Resistance welding , Friction welding

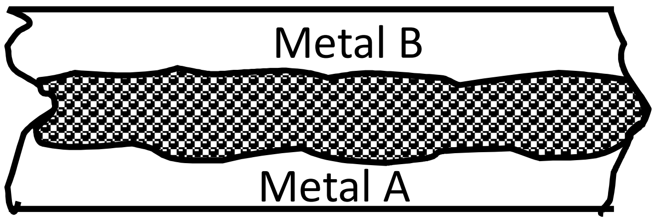
TWO IRREGULAR SURFACES FORMING A WELD



Surface contaminants



Addition of filler material



Near perfect weld

BASIC WELDING PROCESSES

Gas welding - Oxyacetelene welding , simple , slow,
repair and maintenance work

Arc welding - All structural welding

Electric arc by use of electric energy

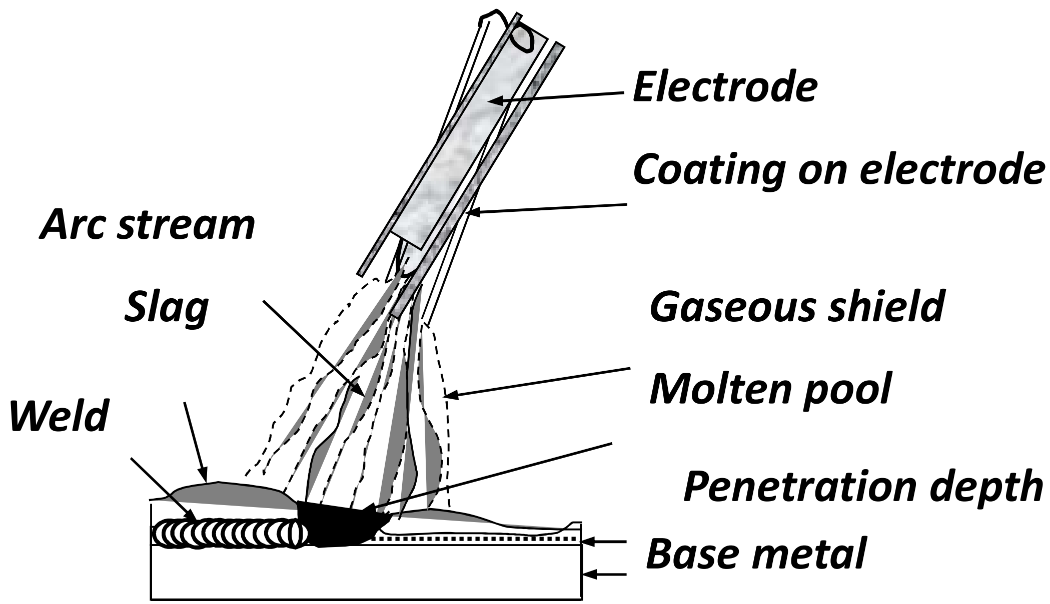
• Shielded Metal Arc Welding

- Coated electrode from gaseous shield to exclude oxygen.
- Also deposits slag in molten metal.

- Coating shields the arc ,coats the molten pool against oxidation, stabilising the arc and provides alloying element weld metal.

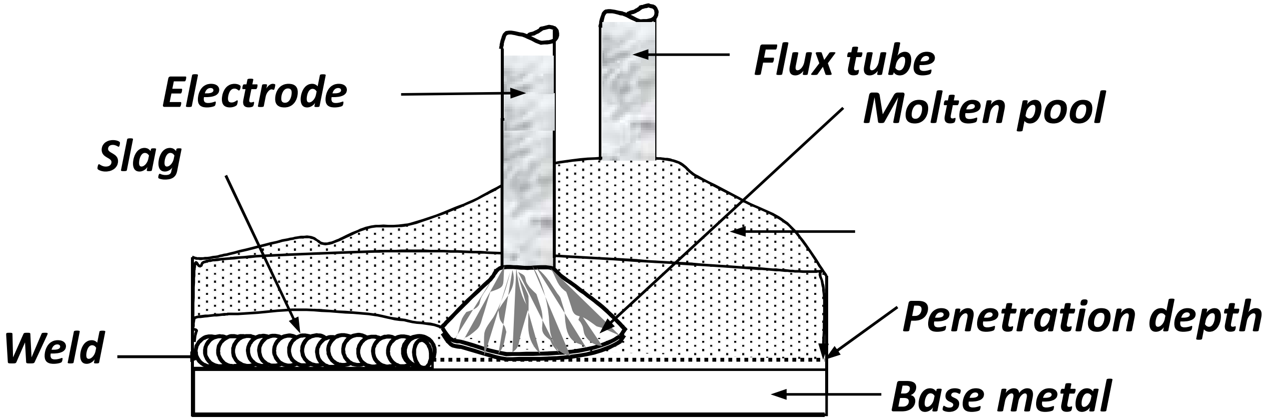
- Type of welding electrode decides strength, ductility and corrosion resistance.

SHIELDED METAL ARC WELDING (SMAW) PROCESS -1



- **Submerged arc welding**

- **Arc is covered by powdered flux**
- **High quality, ductility , impact resistance , corrosion resistance**
- **More heat impart; hence deeper penetration.**



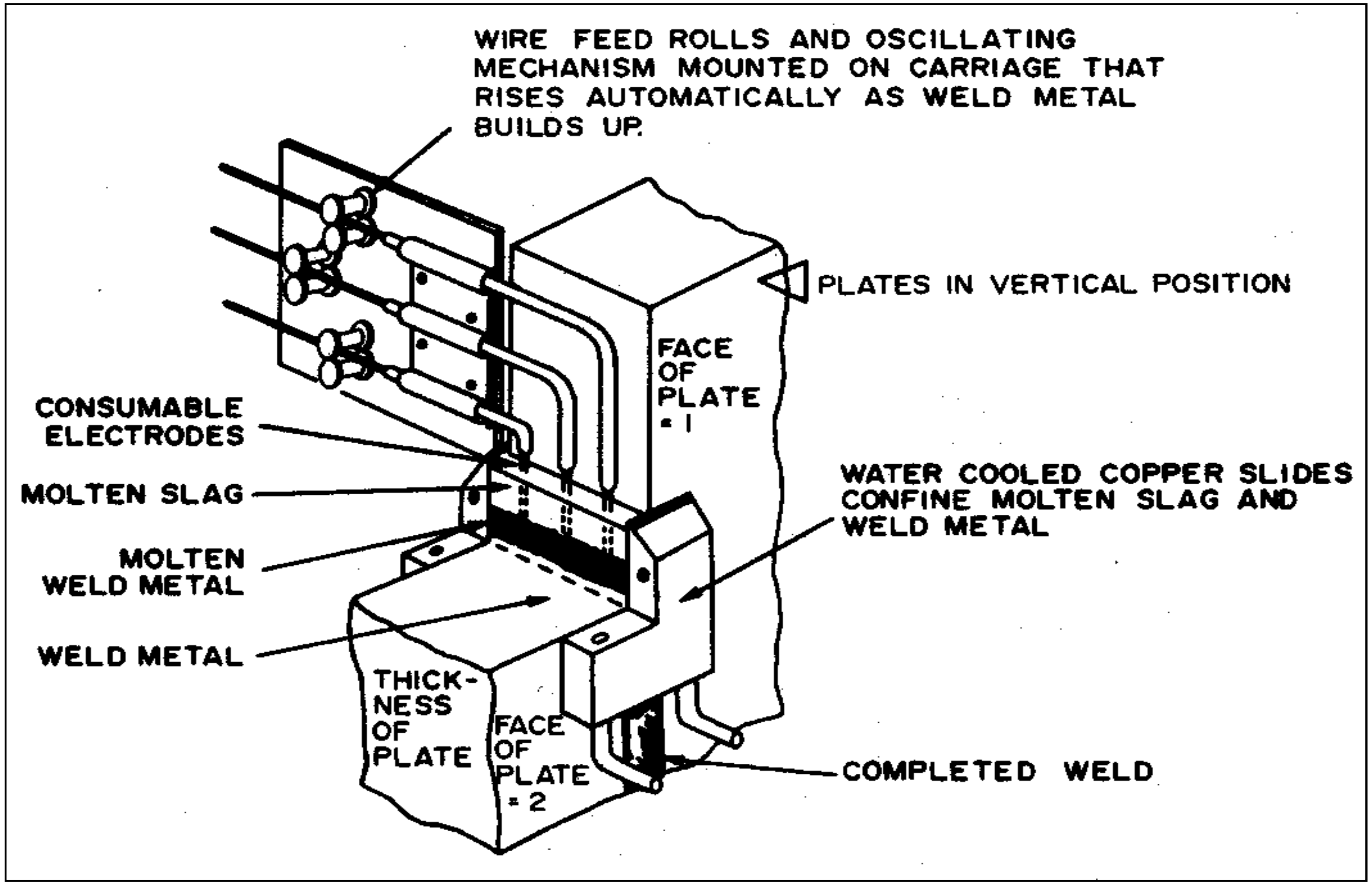
- **Manual Metal Arc Welding**
  - Manually operated; hence requires skill electrode in a steel core wire(3.2 - 6.0 mm diameter)
  - Flux contains manganese and silicon
  - Low capital cost and freedom of movement
  - Advantageous for short welds

- **Metal Active Gas Welding**
  - The arc and weld pool are protected by an inert gas, often carbondioxide.
  - Flux is not necessary
  - Welding is easier
  - Highly suitable for fillet welded joints

Electroslag welding process

- Vertical automatic welding.
- Electrode is immersed in molten slag and the melt is heated to high temperature.
- Weld pool forms at the bottom of the slag pool and forms the weld joining the faces.
- Useful for joining thick sections in vertical position.

ELECTROSLAG WELDING PROCESS-1



- **Stud Welding**

- **Shear connectors in composite construction**

- **Another form of arc welding**

- **Stud main be plain bar with an upset head or threaded**

- **Stud is the electrode**

WELDING PROCEDURE

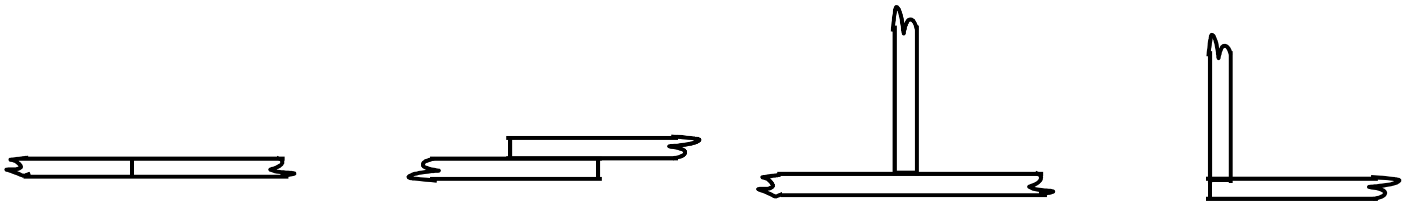
- **Environment**
- **Welding position**
- **Current : controls heat input**
- **Shrinkage**
- **Preheating**

Factors affecting Weld Cracking

- Joint restraint
- Bead shape
- Carbon and alloy content of the base metal
- Cooling rate
- Hydrogen and nitrogen absorption

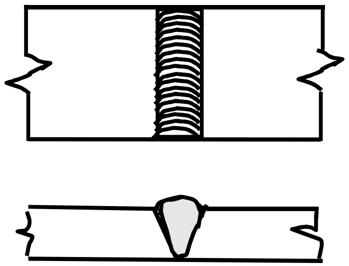
TYPES OF JOINTS OR WELDS

- **Joints:Lap,Tee,Butt and Corner**
- **Welds: Groove, fillet, plug and slot**
- **Welded joint description - Type of joint and weld**
- **Position of welding**

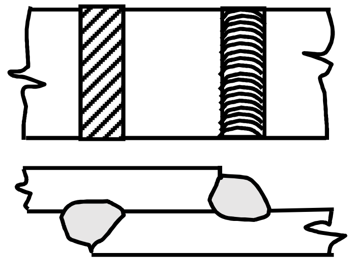


(a) Butt joint *(b) Lap joint* *(c) Tee joint* *(d) Corner joint*

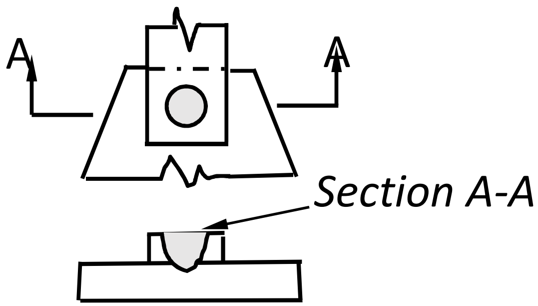
COMMON TYPES OF WELDS



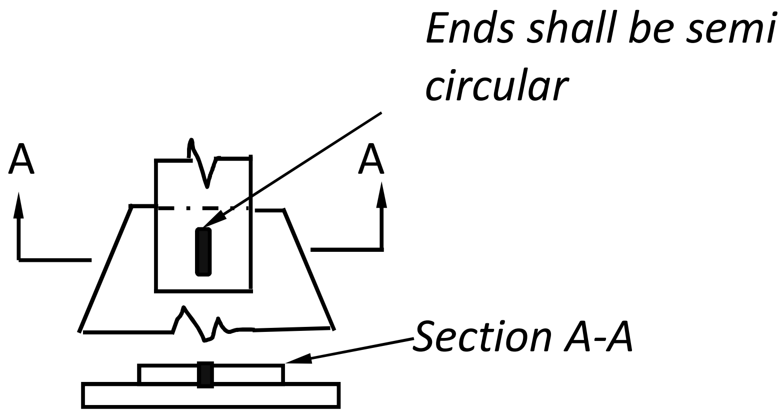
(a) Groove welds



(b) Fillet welds



(d) Plug weld



(c) Slot weld

Groove welds

Selection of a particular type of groove weld depends

- **Size of the plate to be joined**
- **welding by hand or automatic**
- **Type of welding equipment**
- **Accessibility of both sides**
- **Position of weld**

- **Size of butt weld**
  - **Thickness of connected plate for full penetration**
  - **Depth of penetration for partial penetration**

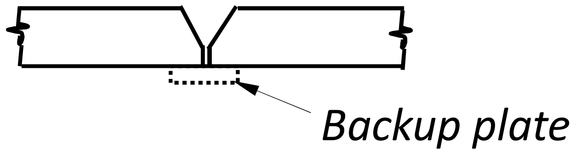
- **Advantages**

  High strength, high resistance to impact and cyclic stress

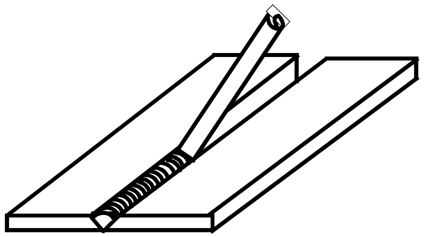
- **Disadvantages**

  High residual stress, edge preparation and proper aligning

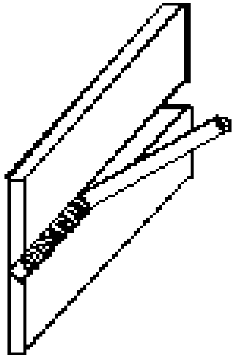
CLASSIFICATION BASED ON POSITION



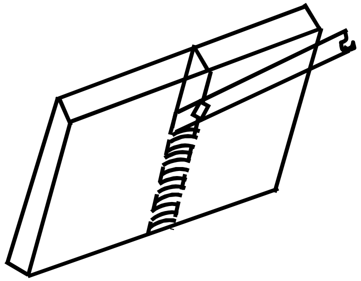
Shaping of surface and backup plate



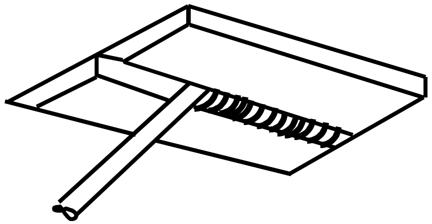
(a) Flat



(b) Horizontal

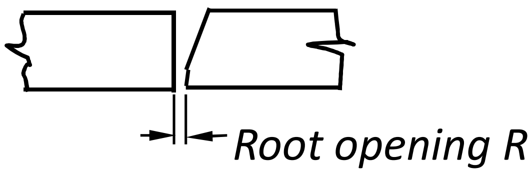


(c) Vertical

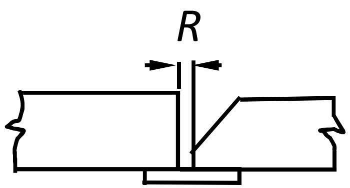


(d) Overhead

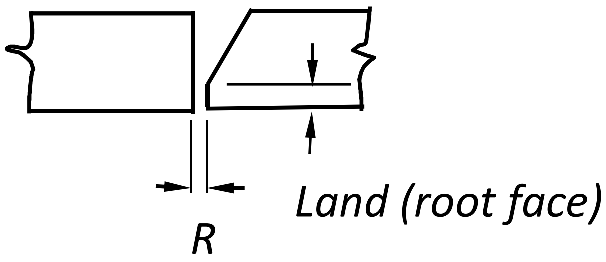
TYPICAL EDGE PREPARATION FOR BUTT WELD



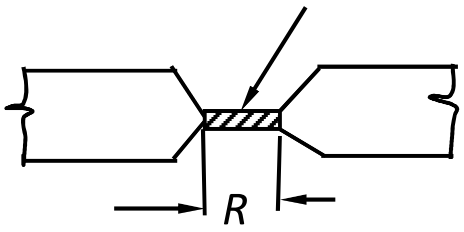
(a) Bevel with feathered edge



(b) Bevel with backup plate

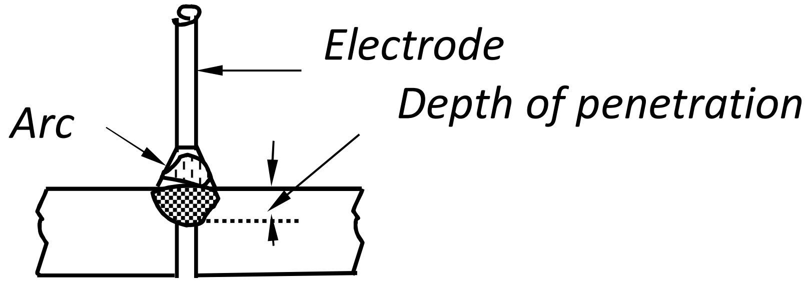


(c) Bevel with a land

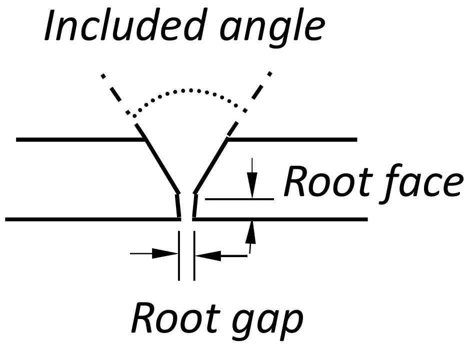


(d) Double bevel with a spacer

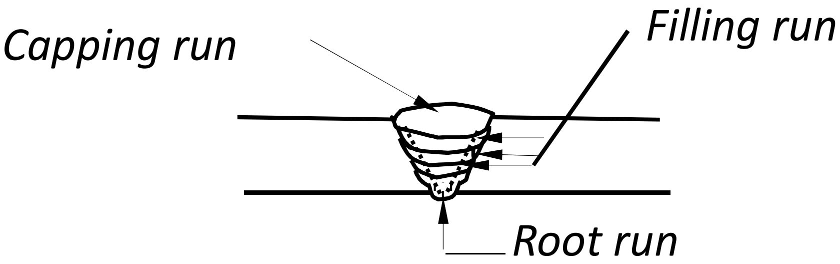
GROOVE WELD DETAILS



(a) Depth of penetration

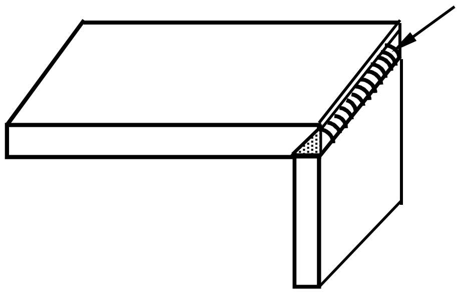


(b) Root gap

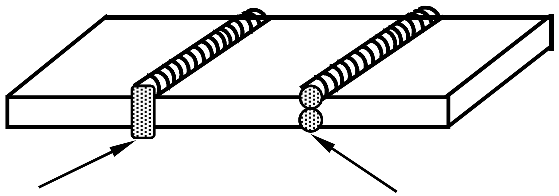


(c) Root run

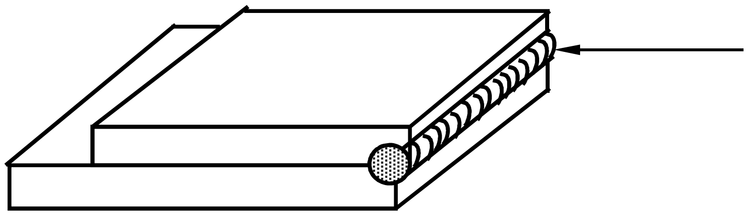
TYPICAL CONNECTIONS WITH GROOVE WELD



(a) Corner joint



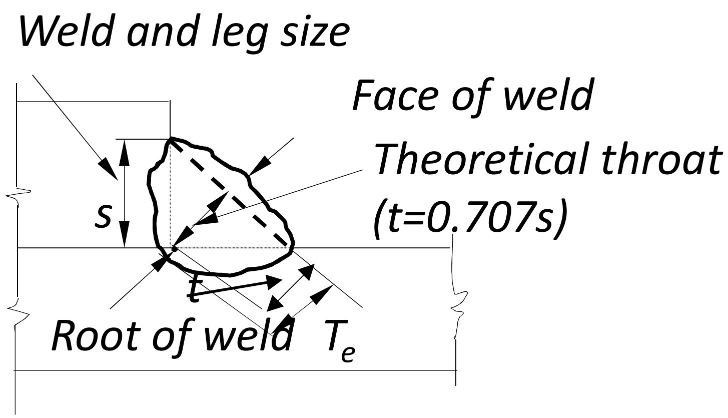
(b) Butt joint



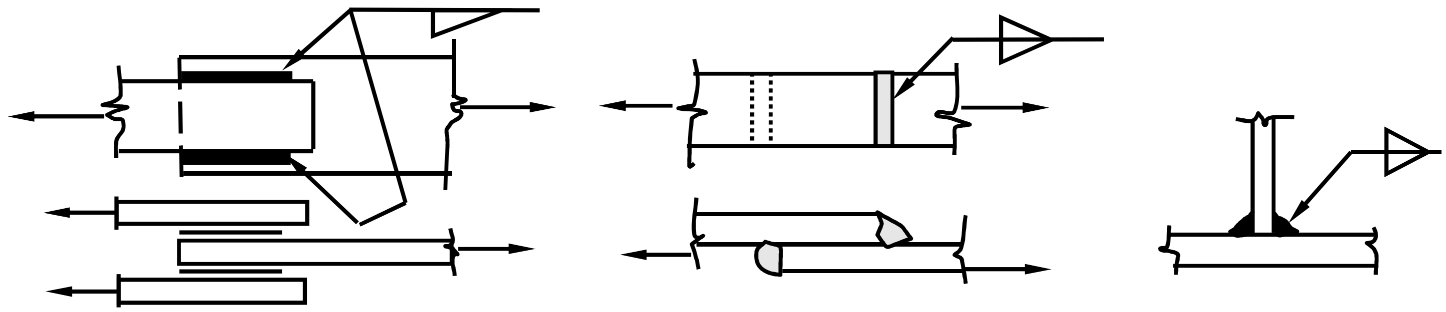
(c) Edge Joint

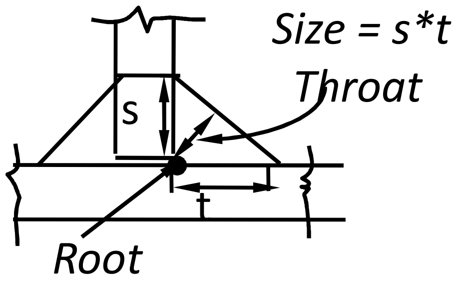
- **Fillet welds**

- **Ease of fabrication and adaptability**
- **Less precision**
- **No special edge preparation**
- **Throat of a weld**
- **Concave and convex surfaces**

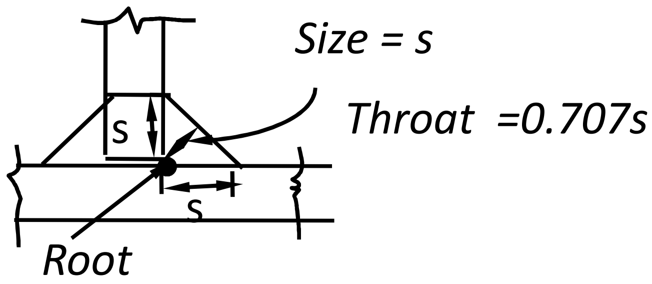


TYPICAL FILLET WELD CONNECTIONS



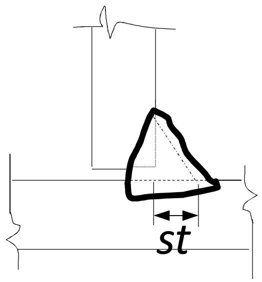


(1) Unequal legs

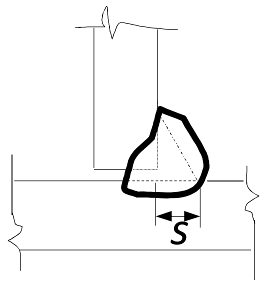


(I1) Equal legs

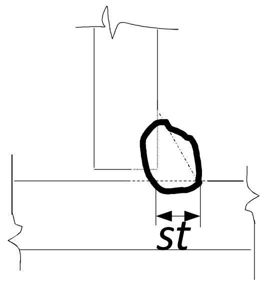
CROSS SECTION OF A FILLET WELD



(a) Concave



(b) Convex

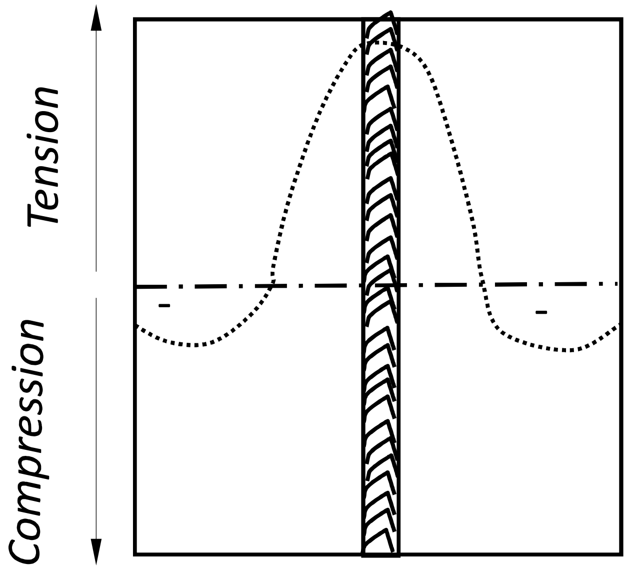


(c) Under cutting

RESIDUAL STRESSES

- **Residual stresses - due to rapid heating and cooling**
- **Yield strength of material is upper limit for residual stresses**

LONGITUDINAL RESIDUAL STRESS DUE TO WELD



WELD DISTORTION

Three basic dimensional changes

- **Transverse shrinkage**
- **Longitudinal shrinkage**
- **Angular distortion**

Methods to overcome distortion

- **Minimize distortion by controlled welding procedures**
- **Acceptable limits**
- **Techniques to remove distortion**

WELD SYMBOLS

Symbolic representation of welds

(Ref. IS:813 - 1986 ‘ Scheme of symbols for welding’)

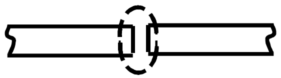
DEFECTS IN WELDS

- **Incomplete fusion**
- **Porosity**
- **Inadequate preparation**
- **Undercutting - Excessive current or long arc**
- **Slag inclusion - Failure to remove slag between runs**
- **Cracks - Breaks in the weld metal**
- **Lamellar tearing - Occurs in the base metal beneath the weld**

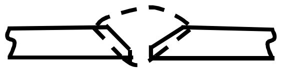
BUTT WELDS

- Critical form of loading - Tension in transverse direction
- Yield stress of weld metal and parent metal in HAZ (Heat affected Zone) is much higher
- Failure always occurs away from the weld
- Toughness and ductility properties are affected

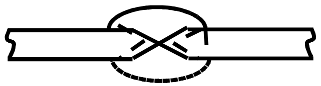
DIFFERENT TYPES OF BUTT JOINTS



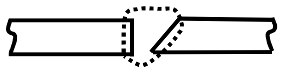
(a) Square



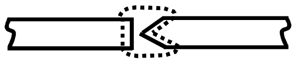
(b) Single V



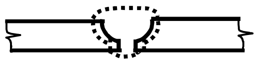
(c) Double V



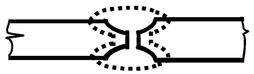
(d)Single Bevel



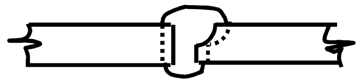
(e)Double Bevel



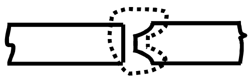
(f) Single U



(g) Single U

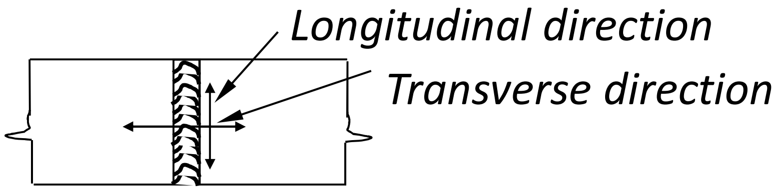


(h) Single J



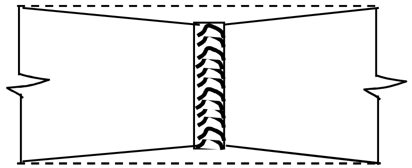
(i)Double J

LOAD APPLIED IN TRANSVERSE DIRECTION OF BUTT WELD



(a)

Load applied in transverse direction



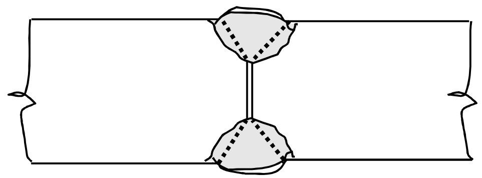
(b)

Longitudinal shrinkage

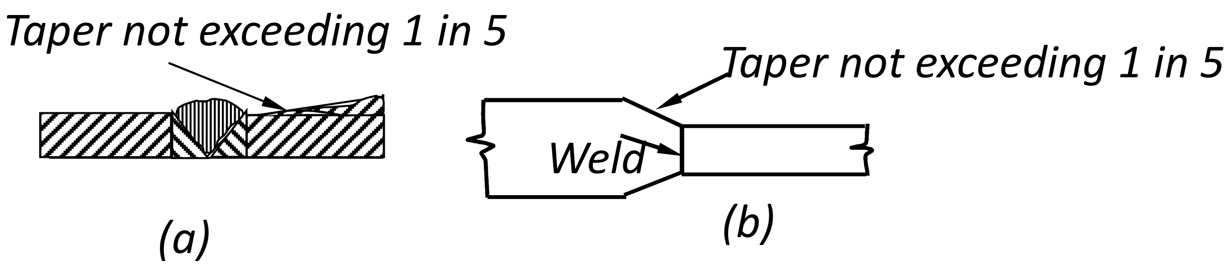
DESIGN

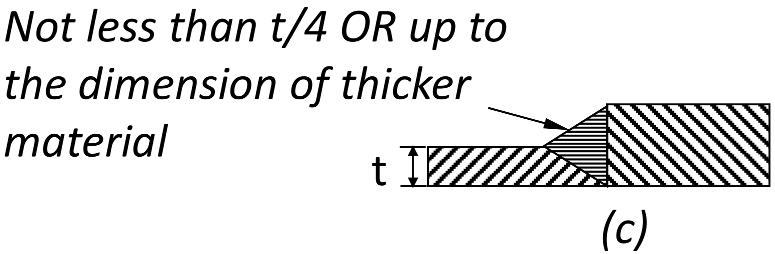
- Direct tension or compression
- Design strength same as parent metal strength
- Effective area equals effective length times throat size
- For full penetration,thickness of weld ,equals thickness of thinner part of connection
- Partial penetration welds are avoided
- Throat thickness - 5/8 thickness of thinner part
- Average stress concept
- Permissible stresses - Parent metal values
- Site welds - 80% of permissible value

PARTIAL PENETRATION WELD



BUTT WELDING OF MEMBERS WITH (A)&(B) UNEQUAL THICKNESS (C) UNEQUAL WIDTH





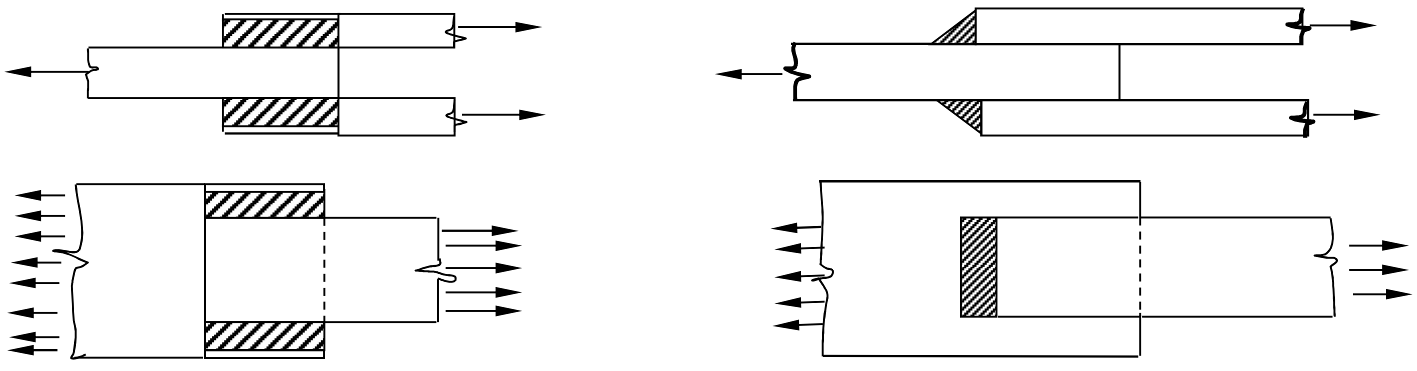
FILLET WELDS

Behaviour

- Lap joints splices
- Shear is the main design consideration
- Side fillets and end fillets
- End fillet loaded in tension - high strength and low ductility
- Side fillet loaded - Limited to weld shear strength (50% tensile strength) Improved ductility
- Average stress in weld throat
- Fillet weld shape is important for end fillets.

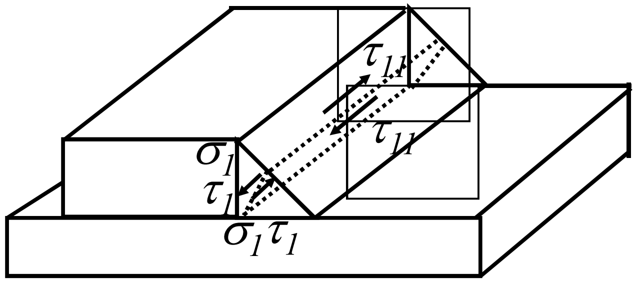
- **Optimum weld shape - Shear leg < 3 times tension leg**
- **Fillet welds are stronger in compression than in tension**

FILLET (A) SIDE WELDS AND (B) END WELDS

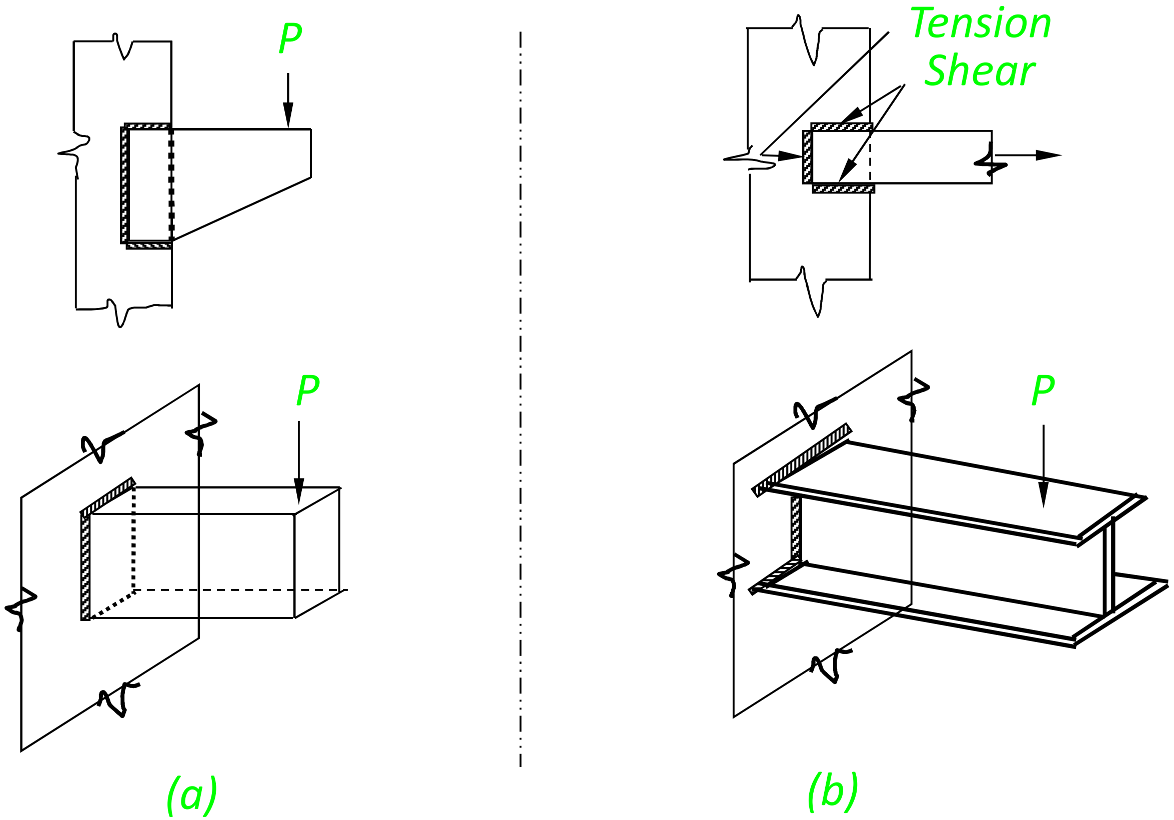


(a) (b)

AVERAGE STRESS IN THE WELD THROAT



(A) CONNECTIONS WITH SIMPLE WELD DESIGN,
(B) CONNECTIONS WITH DIRECTION- DEPENDENT WELD DESIGN



DESIGN

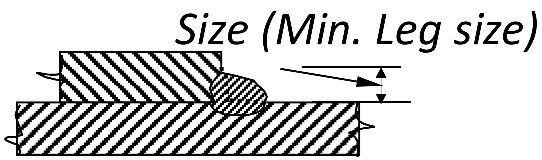
Simple approach - Uniform strength

Size of fillet weld ⩾ 3mm or thickness of thinner part

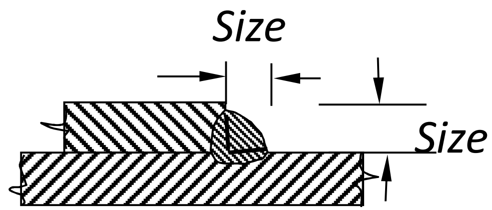
Effective throat thickness ⩾ 3 mm

< 0.7t and 1.0t

= k □ fillet size



Fillets of unequal leg length



Fillets of equal leg length

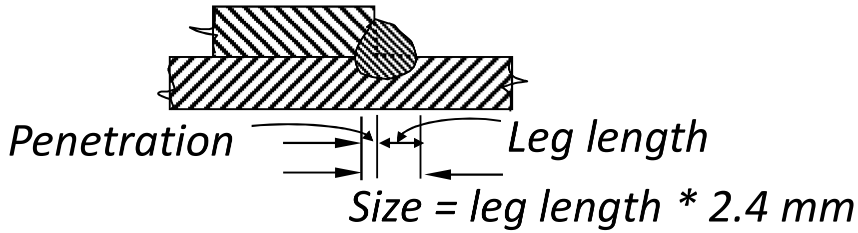
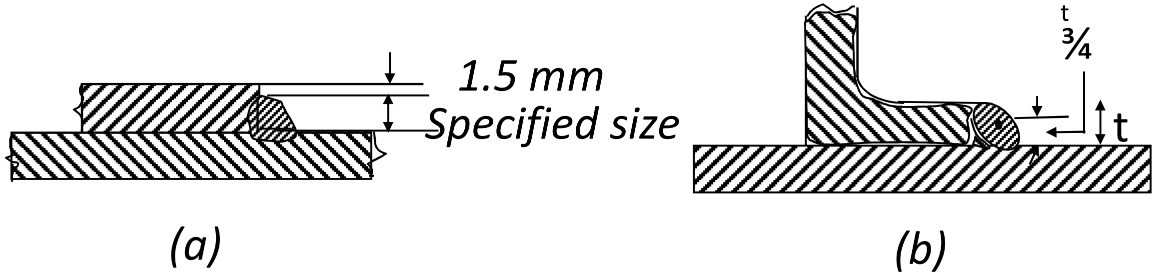


Table 1 Minimum size of first run or of a single run fillet weld

Thickness of thicker part		**Minimum size (mm)**
Over (mm)	Up to and including (mm)	
-	10	3
10	20	5
20	32	6
32	50	8 First run 10 Minimum size of fillet

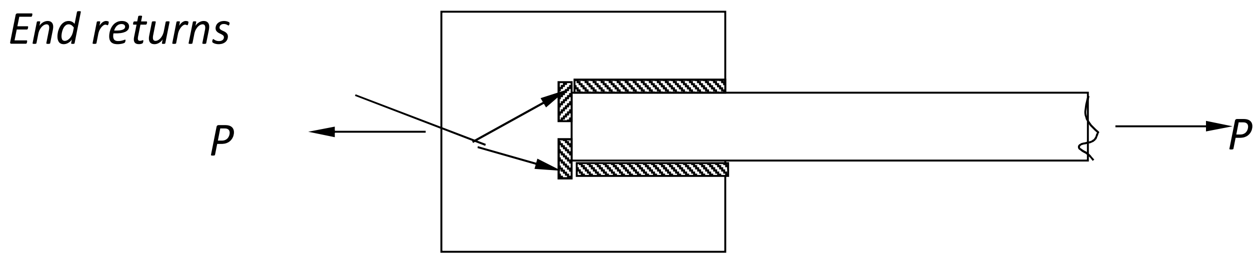


(a) fillet welds on square edge of plate, (b) fillet welds on round toe of rolled section

Table 2. Value of K for different angles between fusion faces

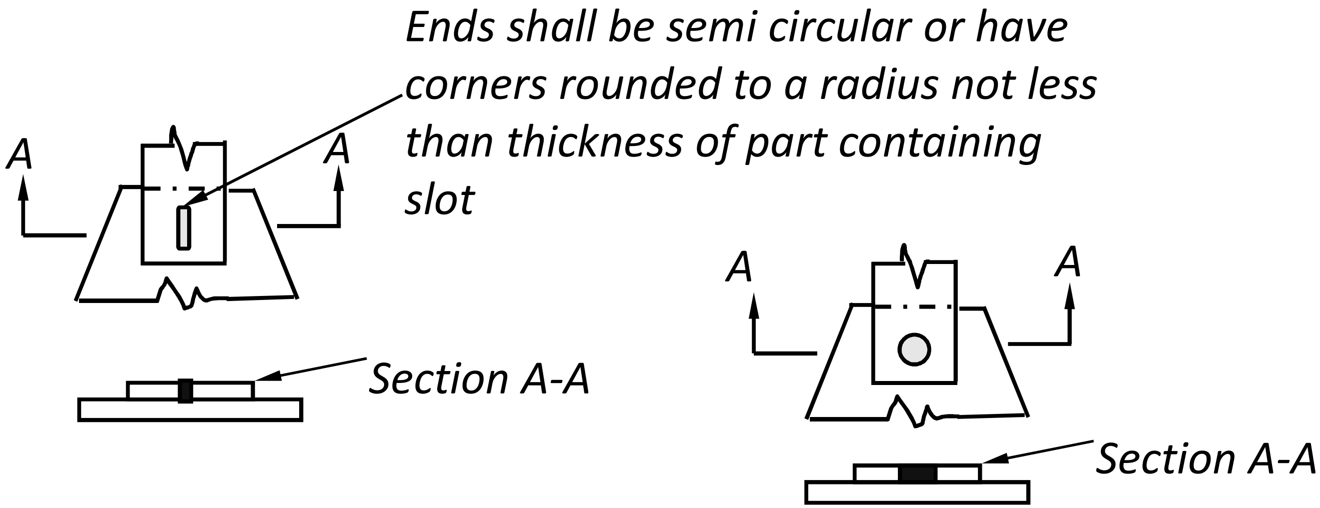
Angle between fusion faces	60° - 90°	91°-100°	101°-106°	107°-113°	114°-120°
Constant K	0.70	0.65	0.60	0.55	0.50

END RETURNS



SLOT AND PLUG WELDS

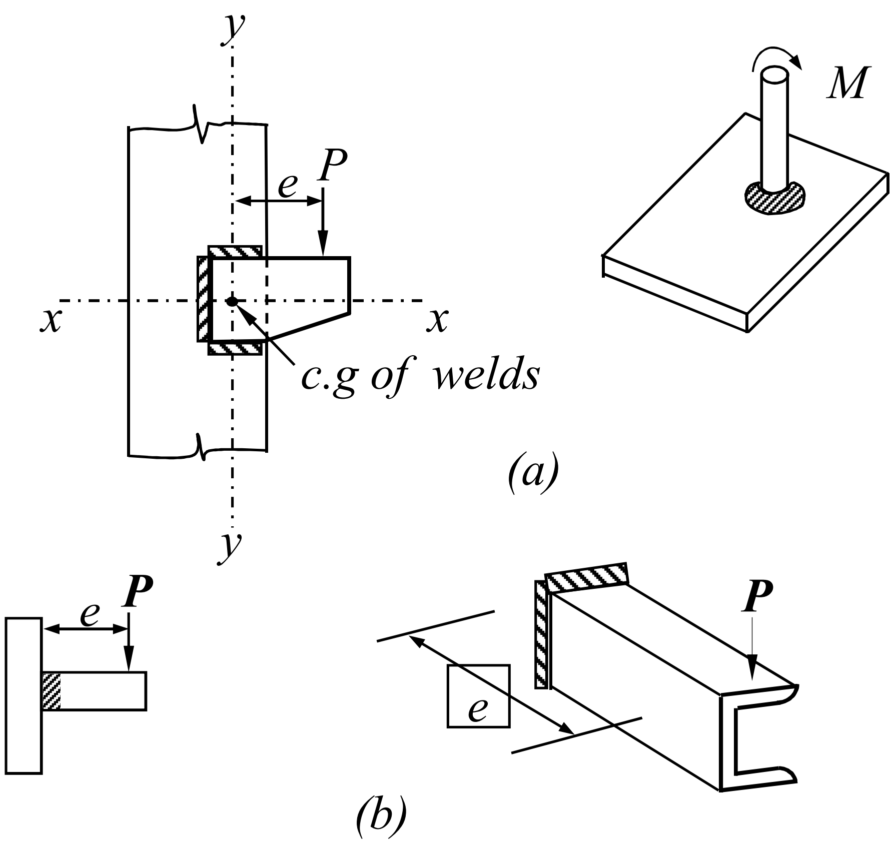
- **Provided along with fillet welds in lap joints**
- **Strength of a plug or slot weld - allowable stress and nominal area in the shearing plane**



(a) Slot weld

(b) Plug weld

(a) WELDS SUBJECTED TO SHEAR AND TORSION,
(b) WELDS SUBJECTED TO SHEAR AND BENDING

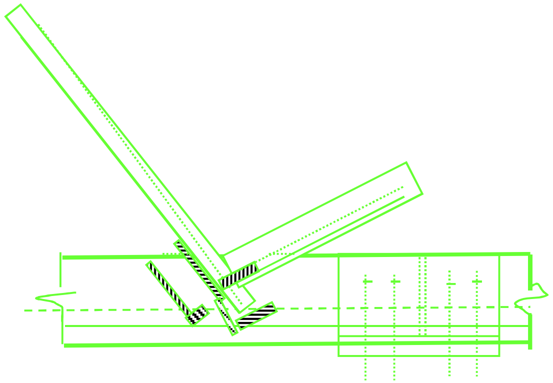


TRUSS CONNECTIONS

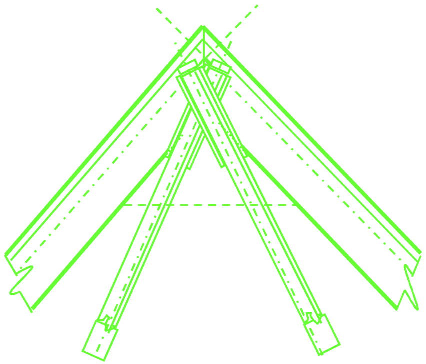
- **Type of connections to be fixed at conceptual stage**
  planar trusses
- **Web members may be directly welded to chord members**

Eccentricities

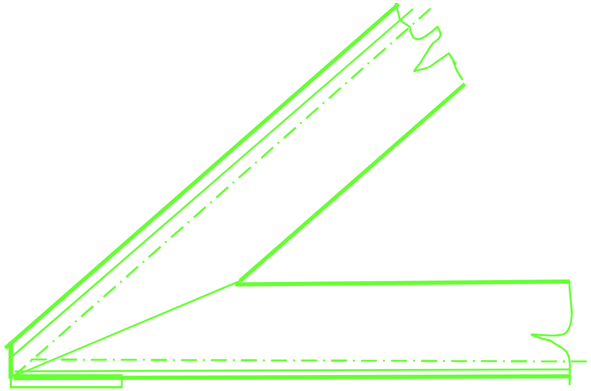
- **Element centroidal axes not intersecting at a point**
- **Connection centroid not coinciding with the**
  element centroid



DIRECT CONNECTION OF WEB MEMBERS

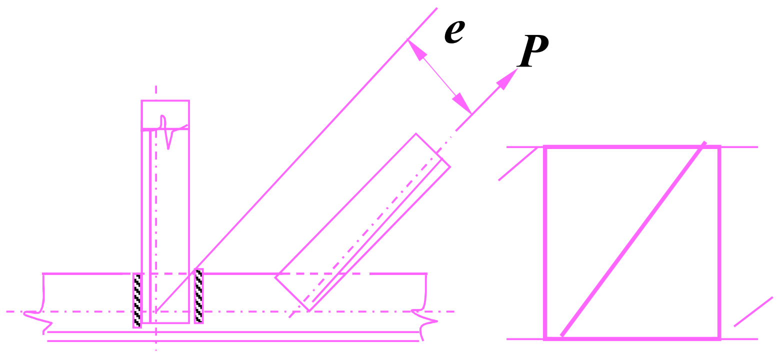


CONNECTION AT THE APEX OF A ROOF TRUSS

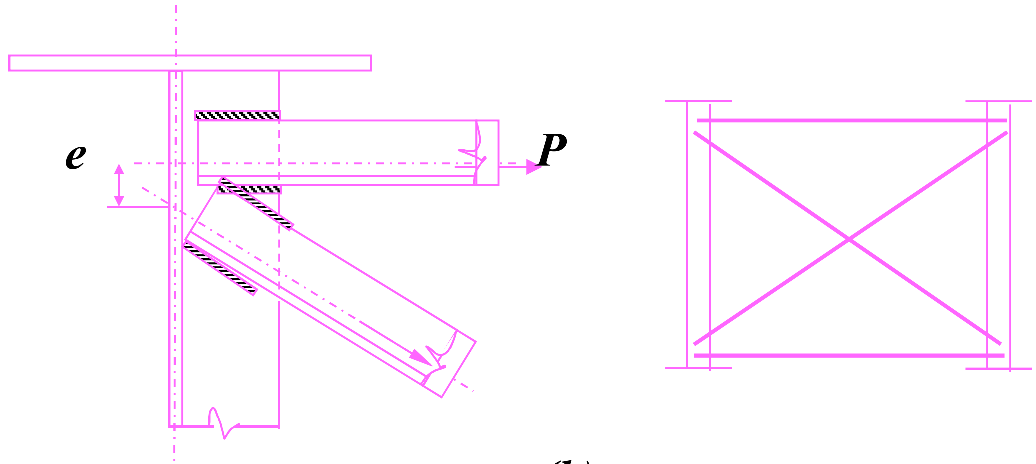


EAVES CONNECTION

ECCENTRICITIES IN TRUSS CONNECTIONS
(a) PRATT TRUSS,
(b) CROSS BRACING BETWEEN PLATE GIRDERS



(a)

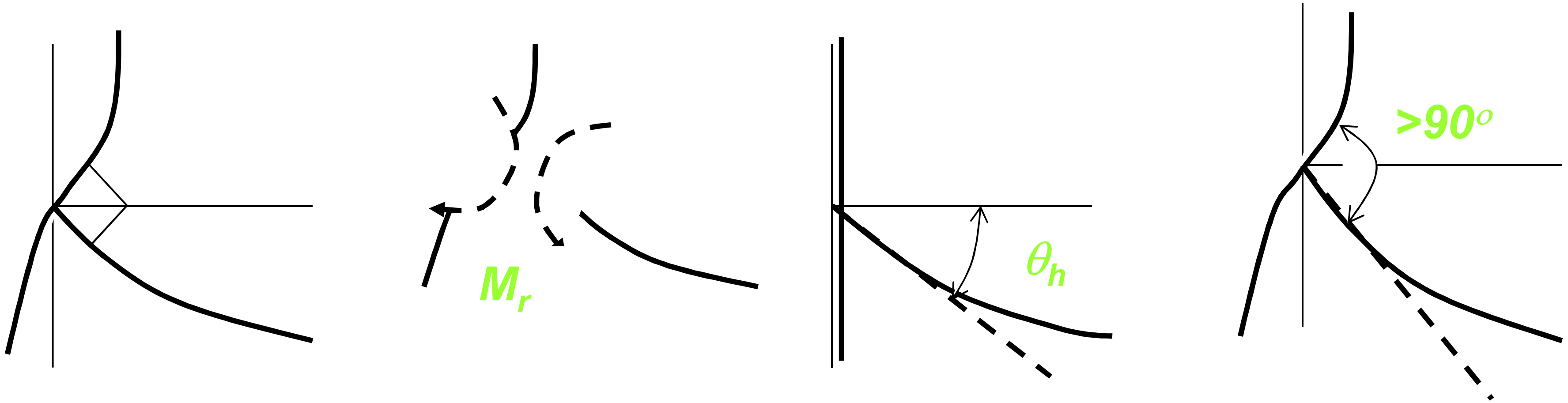


(b)

Connections

RIGIDITY OF MOMENT CONNECTIONS

- Type of connections



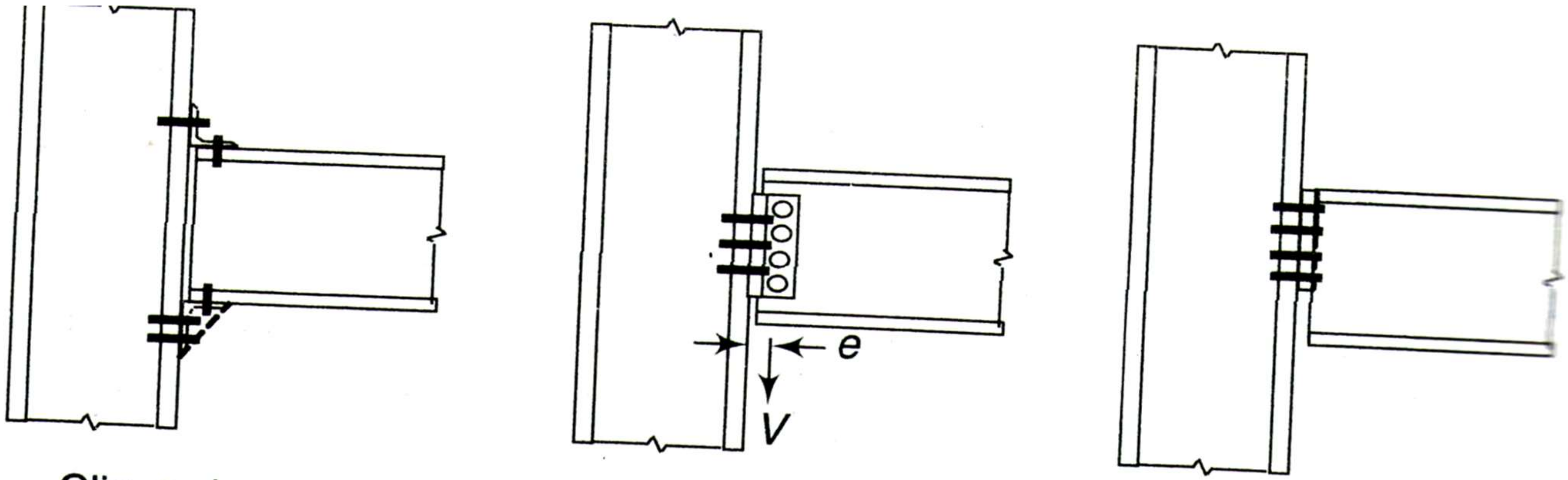
- Rigid

Hinged

Semi-Rigid

Simple Construction

- In simple construction the ends of beams and girders are connected to transmit transverse shear only and are free to rotate under load in the plane of bending. Hence hinged ends are assumed for the beams. Connections are usually made by welding plates or angles to a beam or column in the fabricator's shop and bolted at site to the connecting beam column
- Cls. 4.2.1.3 In simple construction, the connections between members (beam and column) at their junction will not resist any appreciable moment and shall be assumed to be hinged.



Clip and
seating angle
(a)

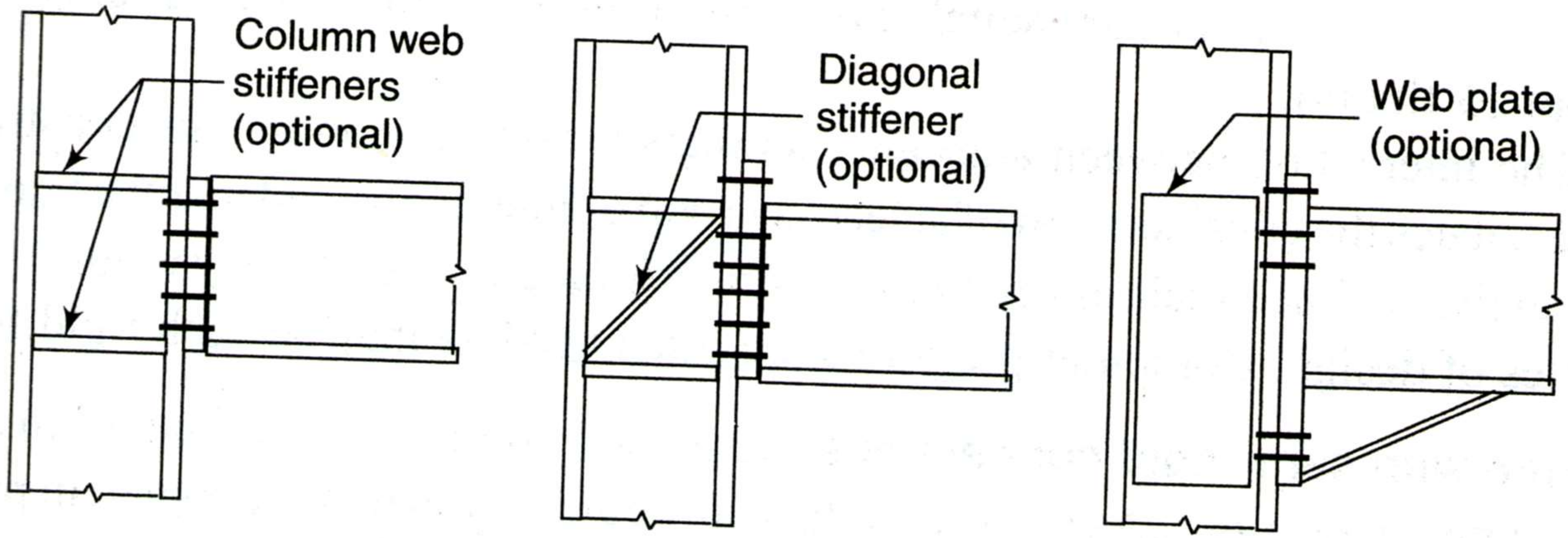
Web cleats
(single or double)
(b)

Curtailed end plate
(header plate)
(c)

Simple beam to column connections

Rigid frame structures

- Assume sufficient rigidity in the beam-column connections, such that under the action of loads the original angles between intersecting members are unchanged (see clause 4.2)
- Fully welded connections can also be considered as rigid beam-to-column connections
- Cls.4.2.1.1 In rigid construction, the connections between members (beam and column) at their junction shall be assumed to have sufficient rigidity to hold the original angles between the members connected at a joint unchanged under loading



Flush end-plate
(a)

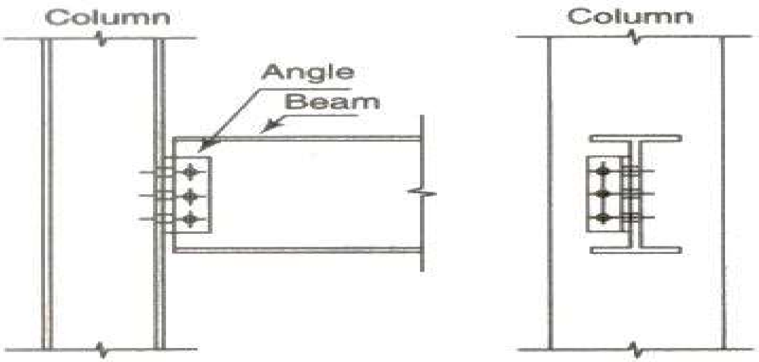
Extended end-plate
(b)

Haunched end-plate
(c)

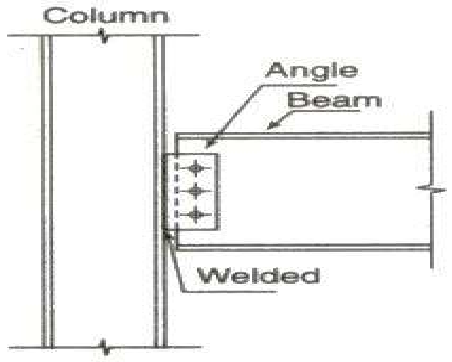
Rigid beam-to-column connections

Semi Rigid Connections

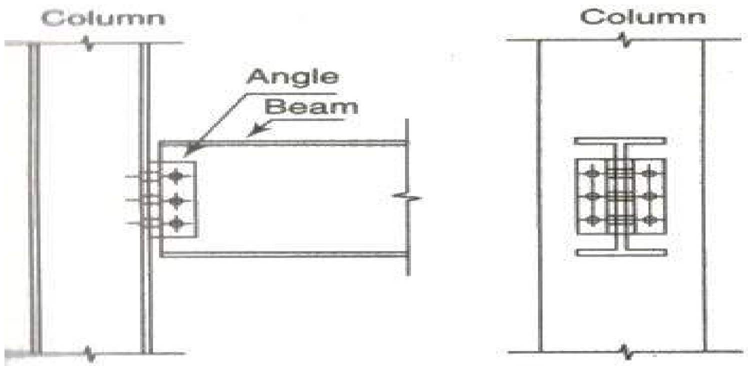
- Fall between simple and rigid connections
- *Semi-rigid construction cls.4.2.1.2*
- In semi-rigid construction, the connections between members (beam and column) at their junction may not have sufficient rigidity to hold the original angles between the members at a joint unchanged, but shall be assumed to have the capacity to furnish a dependable and known degree of flexural restraint. The relationship between the degree of flexural restraint and the level of the load effects shall be established by any rational method or based on test results



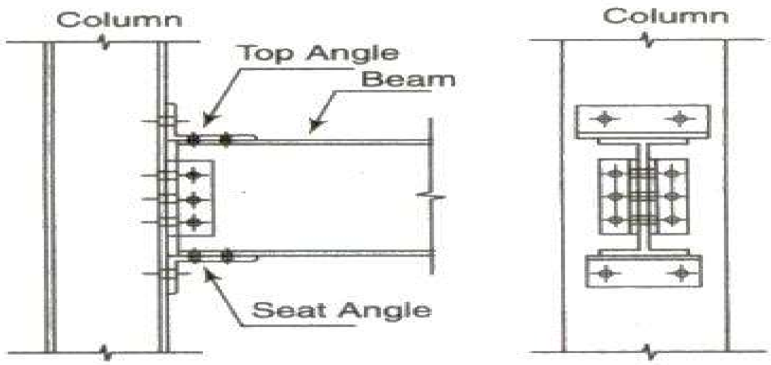
Single web angle
(a)



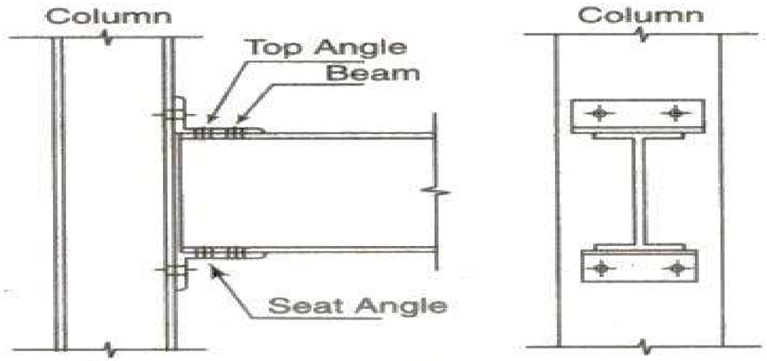
Single plate
(b)



Double web angle
(c)

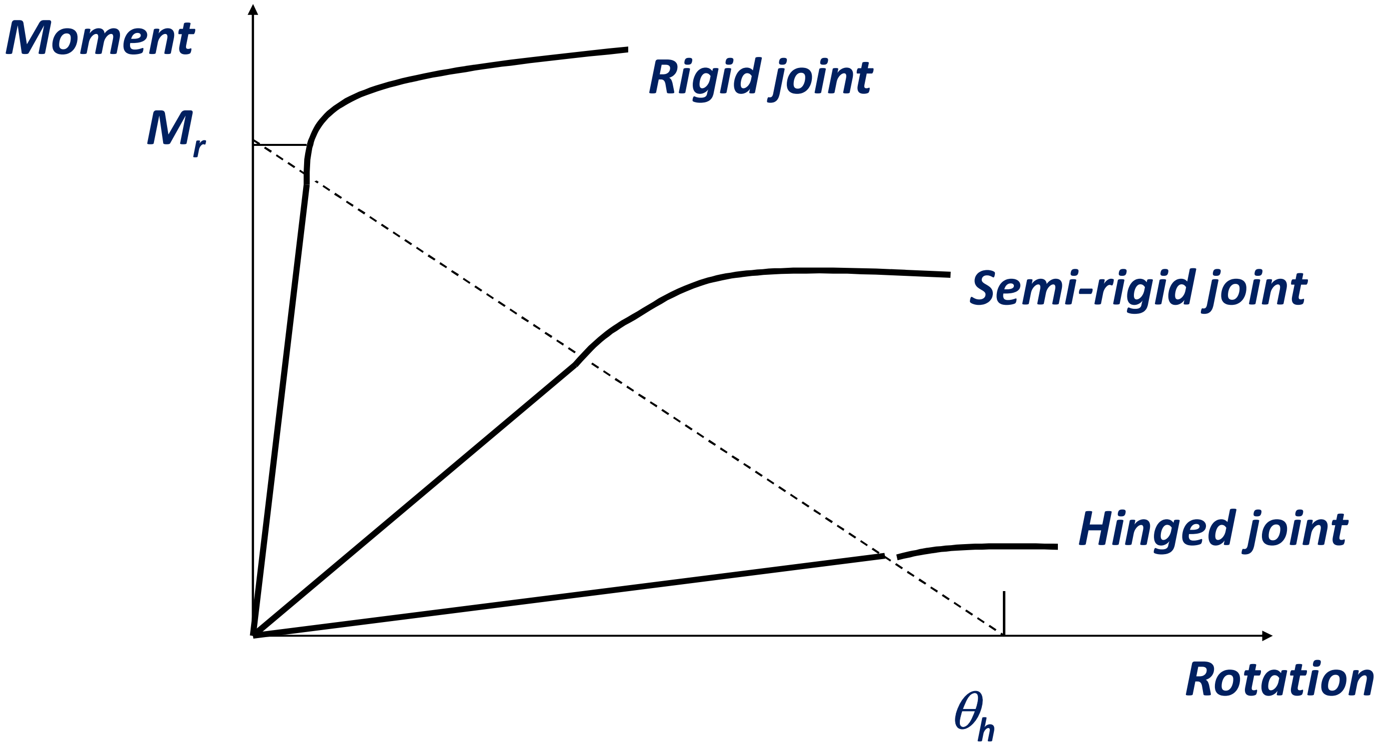


Top and seat angles with double web angle
(d)



Top and seat angles

RIGIDITY OF MOMENT CONNECTIONS



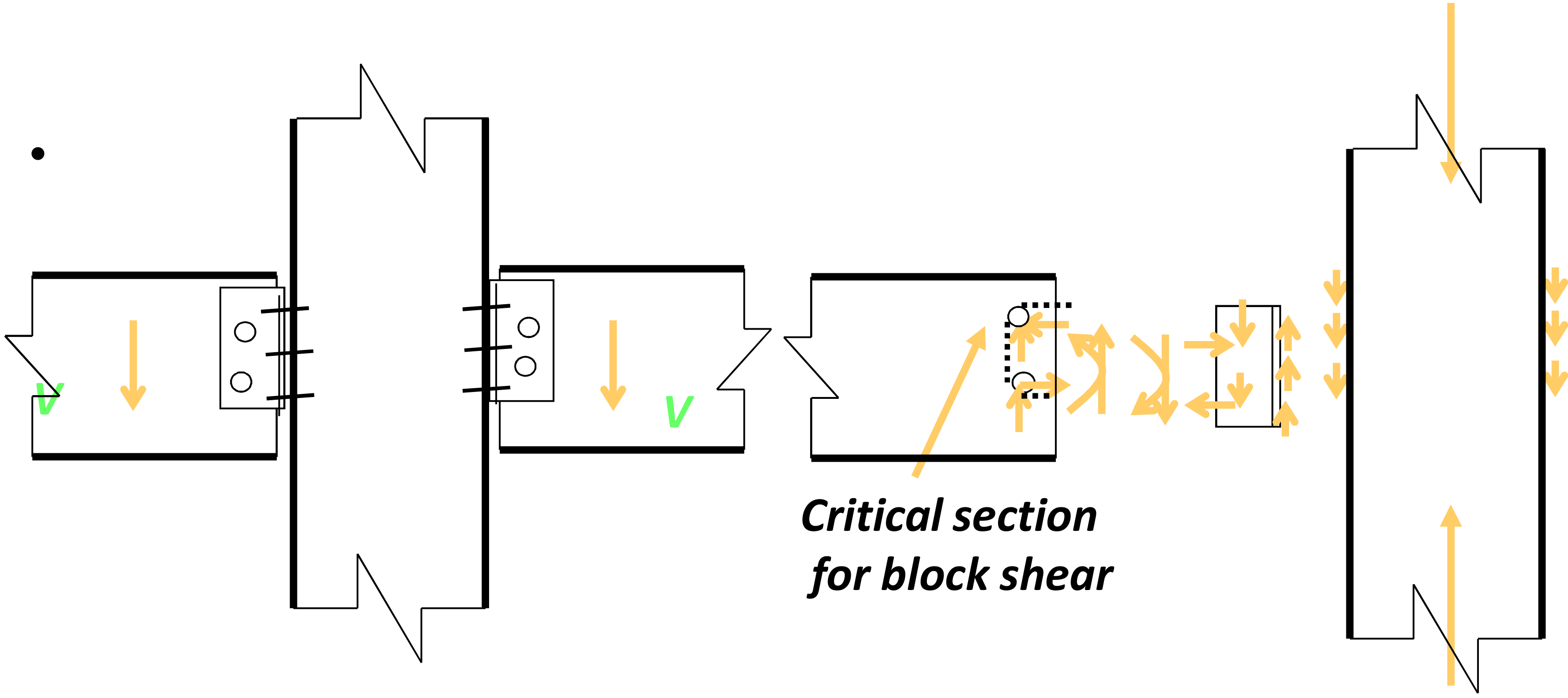
CONNECTION DESIGN PHILOSOPHY

- Exact' analysis:
  - Complex but possible
  - Accuracy depends on assumptions
  - Not practically feasible

- Practical, simplified Methods are Appropriate
  - Should satisfy equilibrium
  - Ductility requirements (static loading)
  - Fatigue strength requirements (cyclic loading)

TRANSFER OF MEMBER FORCES

•



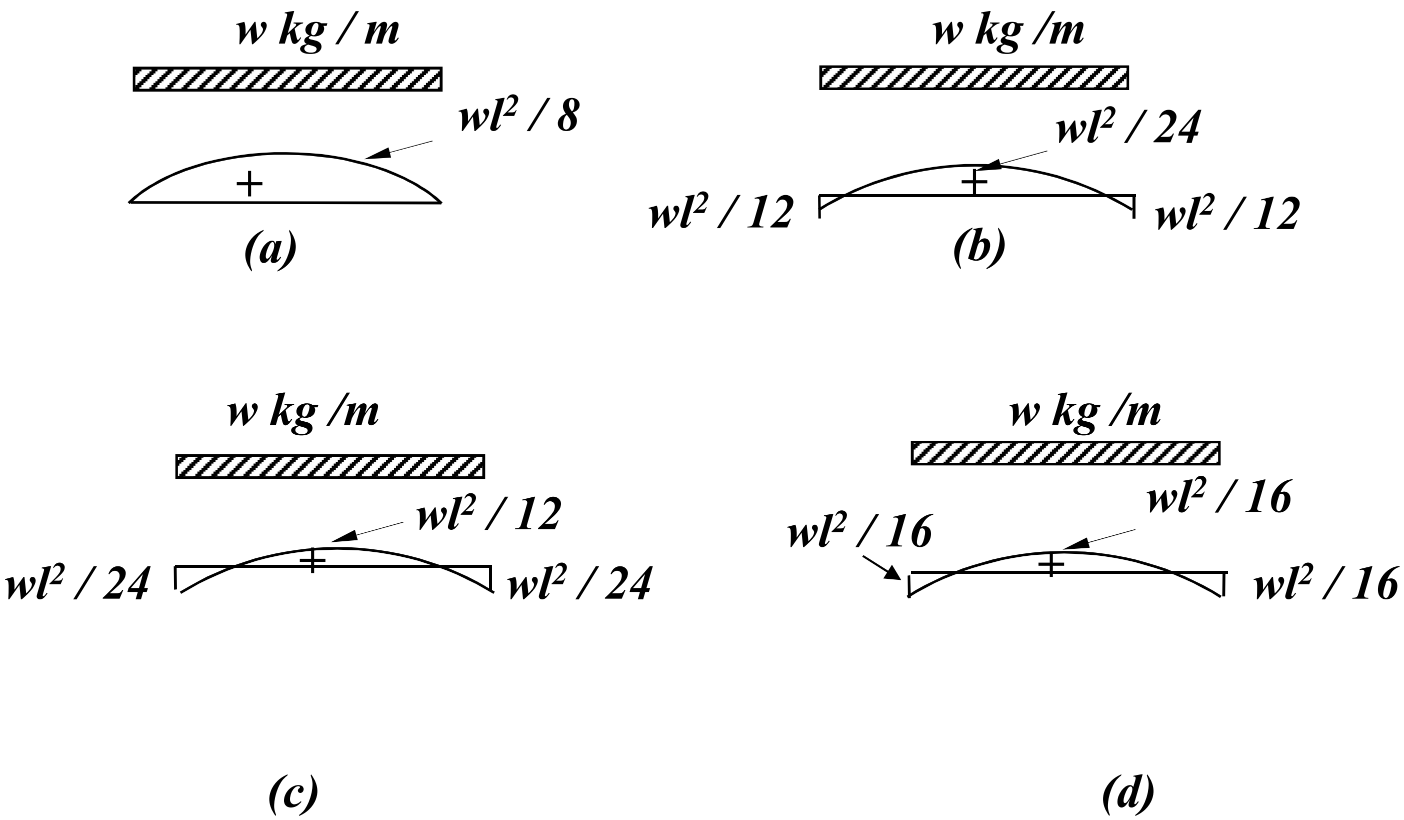
(a) Connection *(b) Freebody Diagram*

BEAM-BEAM AND BEAM-COLUMN CONNECTIONS

Types of beam connections

Rotational characteristics of connections

- **Simple 0-20% moment resistance**
- **Semi rigid 20 -90% “**
- **Rigid >90% “**



(a) Simple Connections (0%)
(b) Rigid Connections (100%)
(c) Semi Rigid Connections (50%)
(d) Semi Rigid Connections (75%)

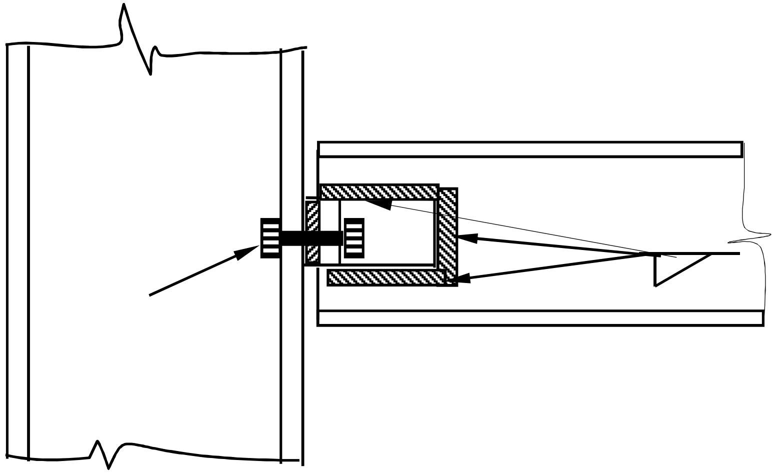
WELDED BEAM CONNECTIONS

- Web angles
- Beam seats — Shear connections
- Stiffened beam seats
- Moment resistant connections

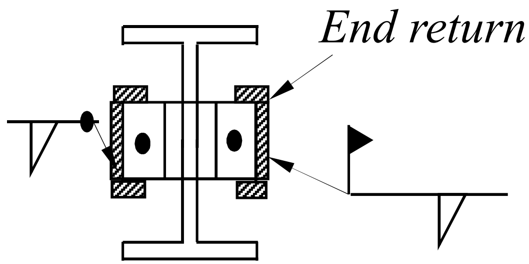
Two common ways of stress transfer at connection

- Bending forces occur in beam flanges and for transfer, welds to be provided at the beam flanges
- For transfer of shear forces welds to be provided at webs

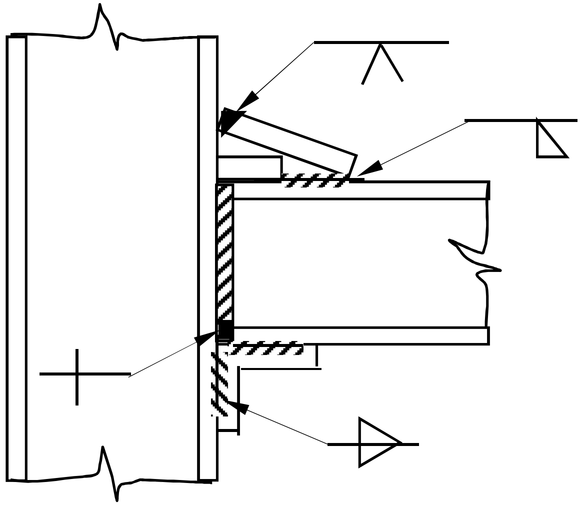
FRAMED SIMPLE CONNECTION



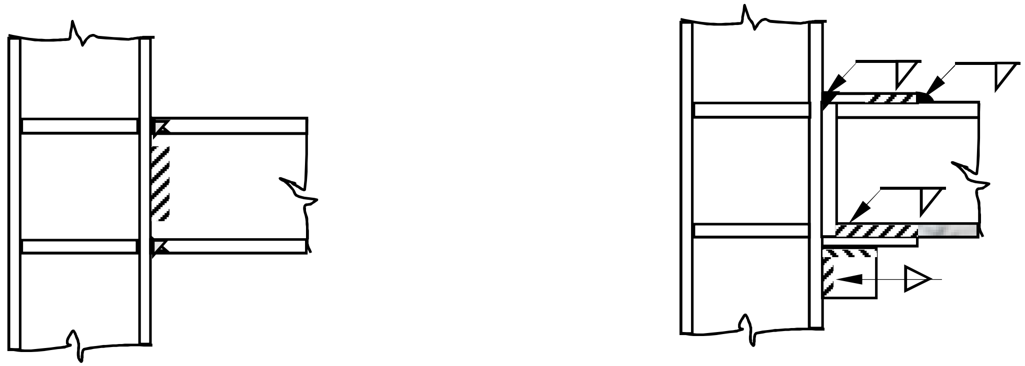
Erection bolt



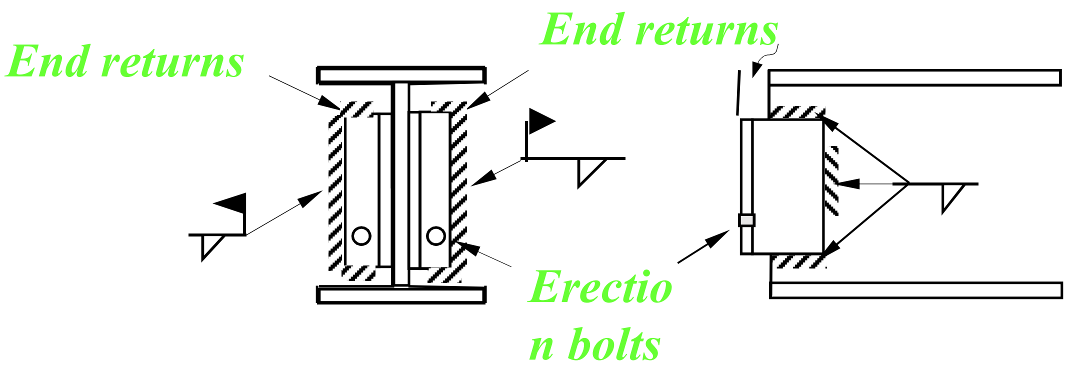
WELDED SEMIRIGID CONNECTION



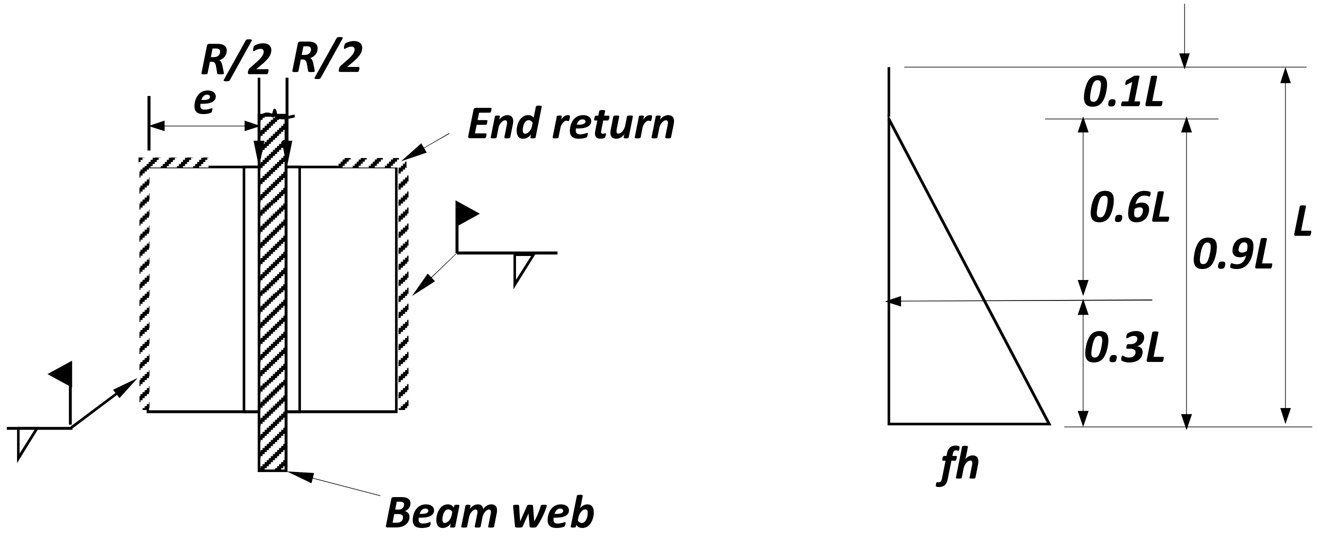
WELDED MOMENT - RESISTING CONNECTION



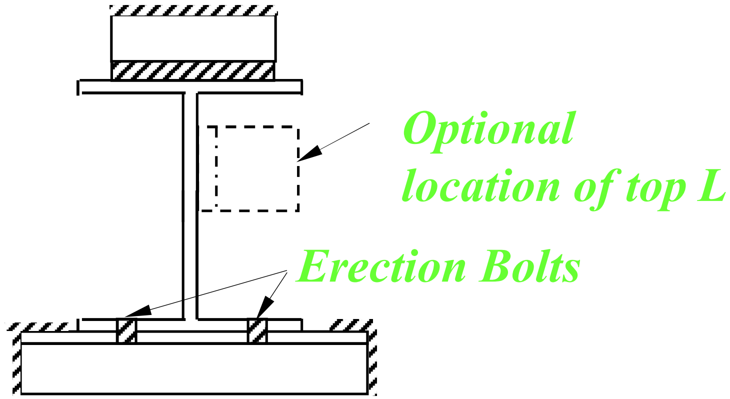
WELDED WEB ANGLES

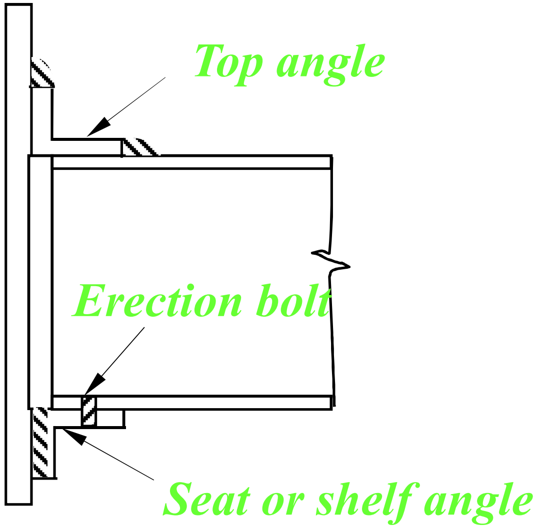


ECCENTRICITY OF REACTION FORCES

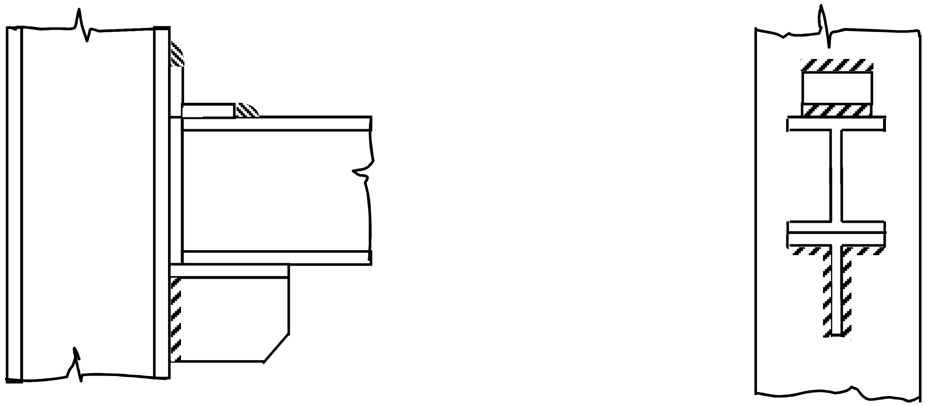


WELDED SEATED- BEAM CONNECTIONS





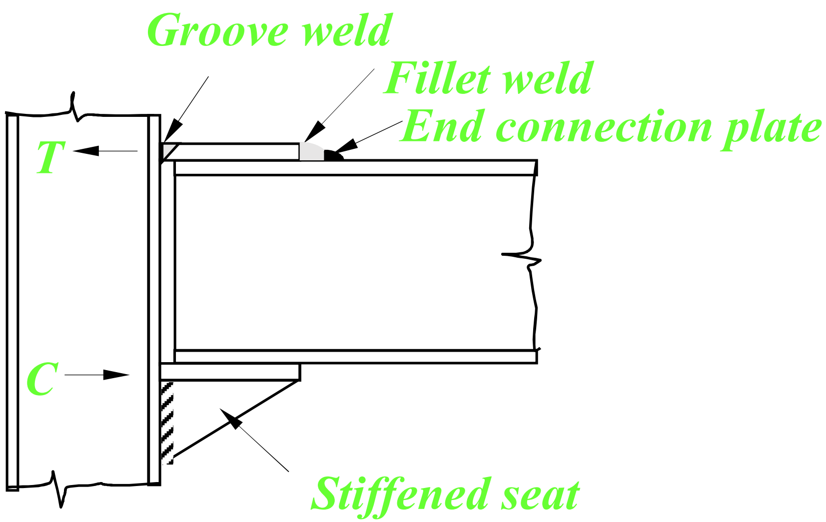
STIFFENED BEAM SEAT CONNECTION



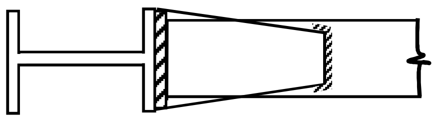
MOMENT RESISTANT CONNECTIONS

- **Continuous structures**
- **Connections are designed to resist full moments**
- **Efficient connections**
- **Moment resistance may reduce**
  - **Bending of the column at the connection point**
  - **Top connection plate tries to bend column flange**

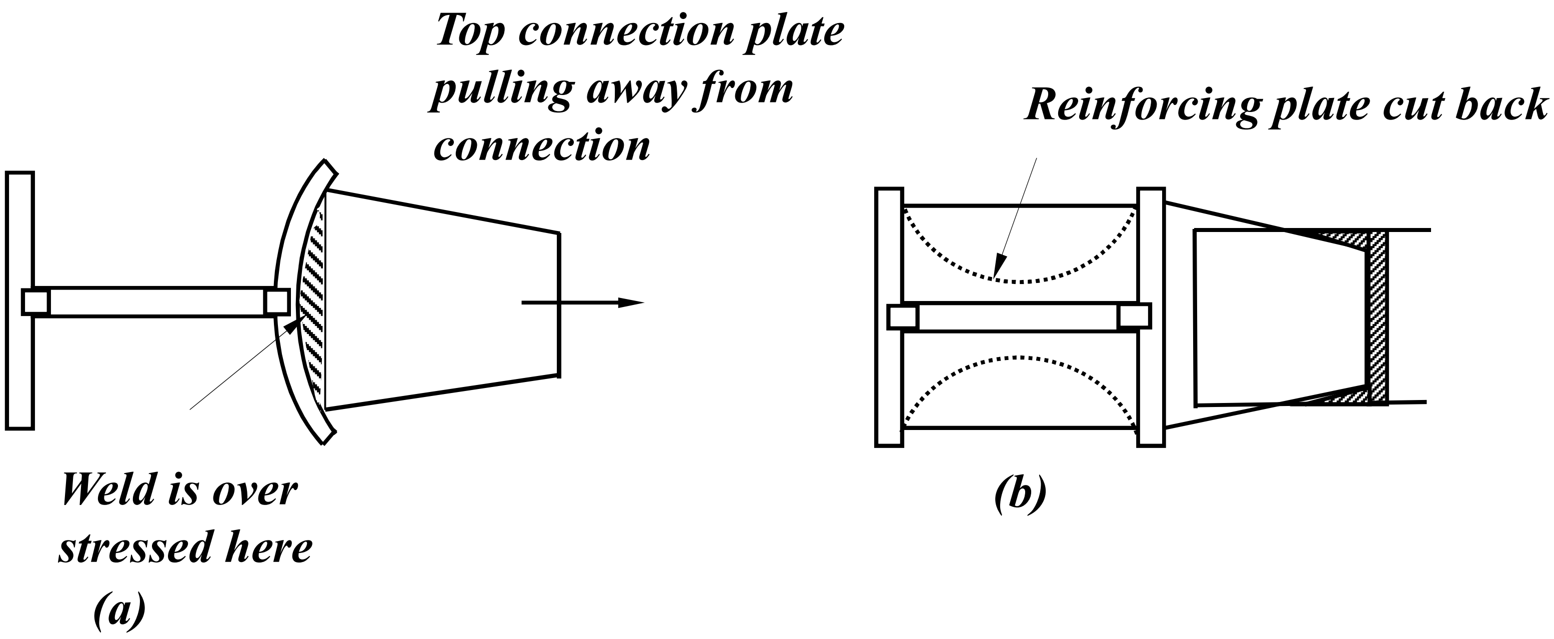
MOMENT RESISTING CONNECTIONS -1



(a)



(b)



(a) OVERSTRESSING OF THE WELD,
(b) COLUMN FLANGE STIFFENED WITH PLATES

BEAM – COLUMN CONNECTIONS

BEAM – COLUMN CONNECTIONS

- Beam-to-column connections are neither ideally pinned nor ideally fixed and posses a finite non-zero stiffness.
- However they are classified as simple (pinned), semi-rigid and rigid (fixed) depending on the connection stiffness
- Such a classification helps in simplifying the analysis of frames.
- A connection having a small stiffness can be assumed as pinned while a connection having a large stiffness can be assumed as fixed.
- In the former case, the actual mid-span bending moments will be less than what is designed for while in the latter case the mid-span deflection will be more than what is calculated.

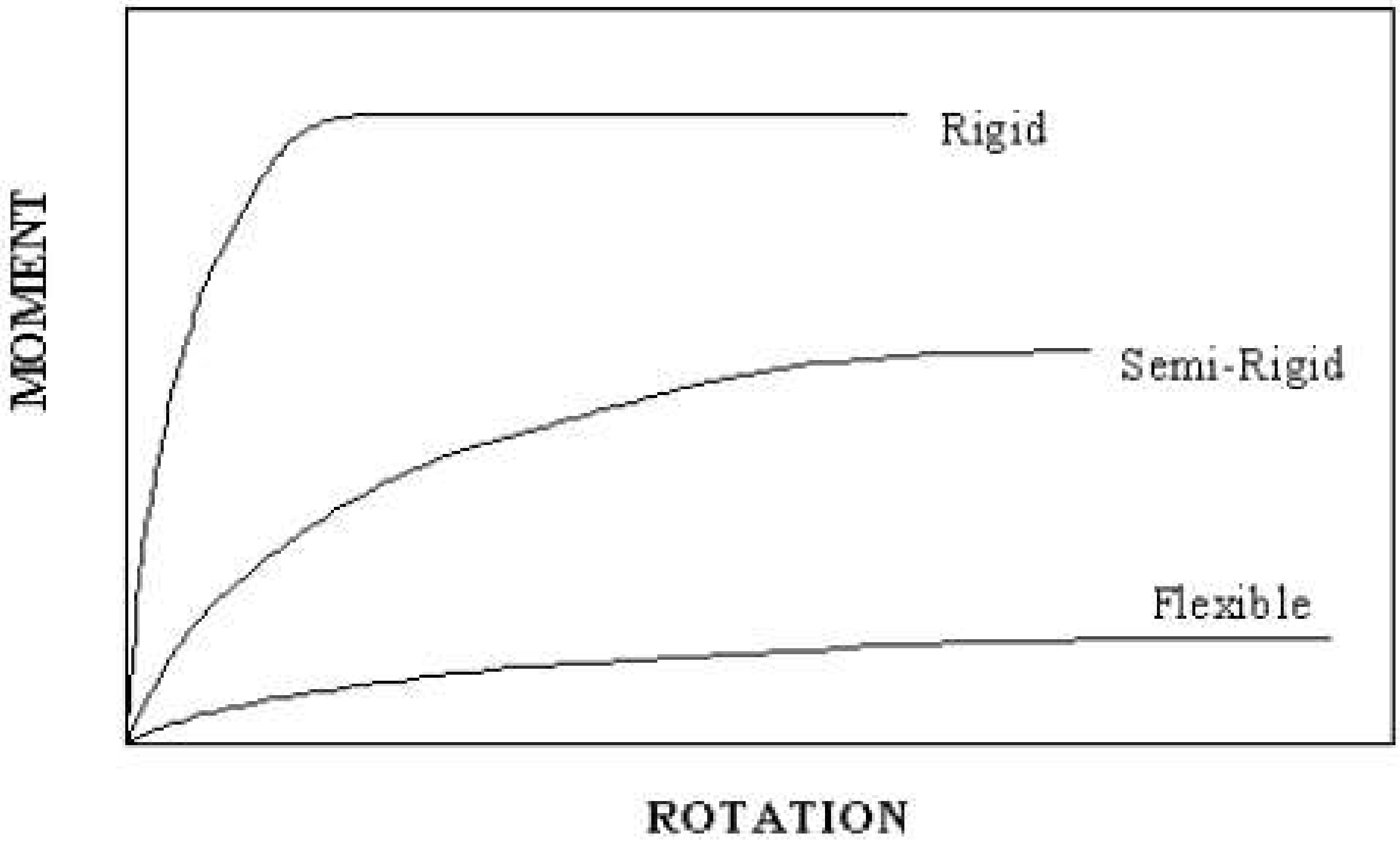


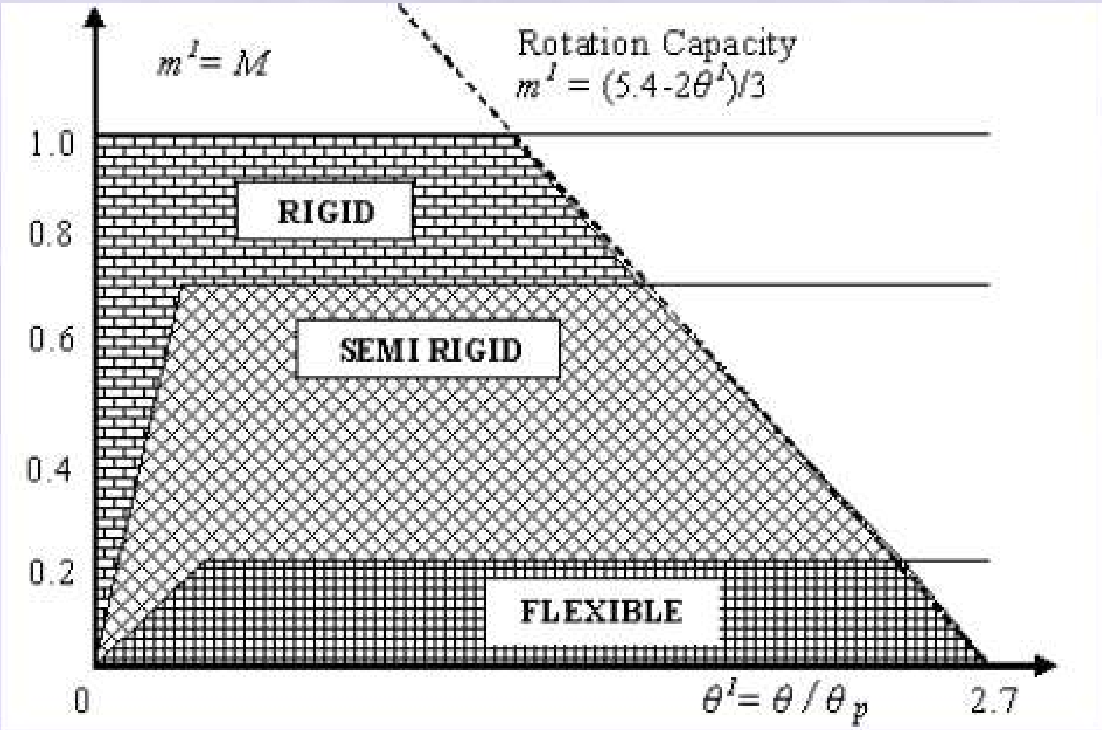
Fig. 1.1 Moment-rotation relationships for connections

Connection classification

- Classification proposed by Bjorhovde et al. (1990) is recommended by the IS 800 code.
- Connections are classified according to their ultimate strength or in terms of their initial elastic stiffness.
- classification is based on the non-dimensional moment parameter ($m^1 = M_u / M_{pb}$) and the non-dimensional rotation ($q^1 = q_r / q_p$) parameter, where q_p is the plastic rotation.
- Bjorhovde's classification is based on a reference length of the beam equal to 5 times the depth of the beam.

Table.1.1 Connection classification limits: In terms of strength

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	$m^1 \leq 0.2$	$m^1 \leq 0.5\theta^1$



Classification of Connections according to Bjorhovde

Connection configurations

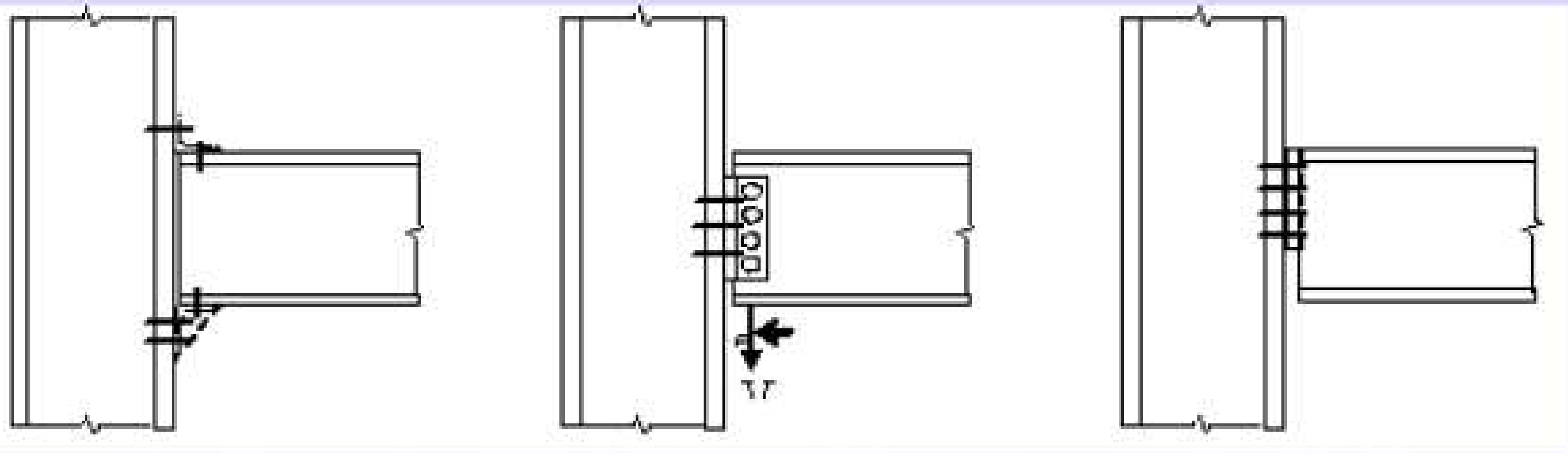
- **Simple connections:**
- Simple connections are assumed to transfer shear only
- can be used only in non-sway frames where the lateral loads are resisted by some alternative arrangement such as bracings or shear walls.
- Simple connections are typically used in frames up to about five storey in height, where strength rather than stiffness govern the design.

Simple Connection…

- The clip and seating angle connection
- In the case of unstiffened seating angles, the bolts connecting it to the column may be designed for shear only assuming the seating angle to be relatively flexible.
- If the angle is stiff or if it is stiffened in some way then the bolted connection should be designed for the moment arising due to the eccentricity between the centre of the bearing length and the column face in addition to shear.
- The clip angle does not contribute to the shear resistance because it is flexible and opens out but it is required to stabilise the beam against torsional instability by providing lateral support to compression flange.

Simple Connection…

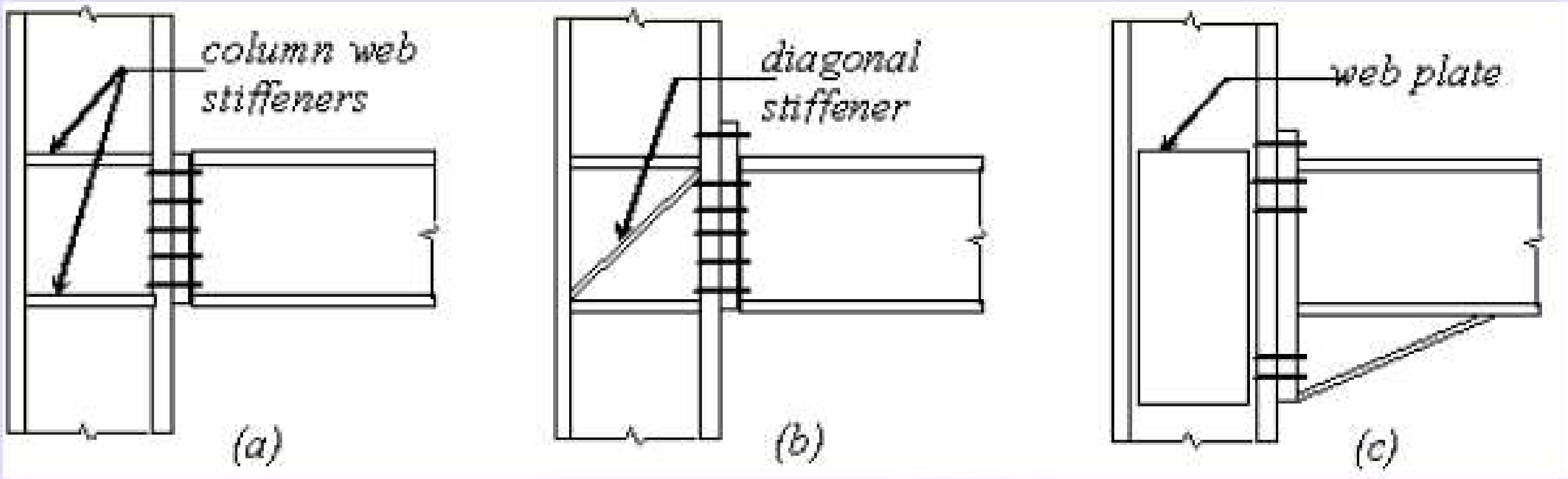
- The connection using a pair of web cleats, referred to as framing angles, is also commonly employed to transfer shear from the beam to the column.
- Here again, if the depth of the web cleat is less than about 0.6 times that of the beam web, then the bolts need to be designed only for the shear force.
- Otherwise by assuming pure shear transfer at the column face, the bolts connecting the cleats to the beam web should be designed for the moment due to eccentricity. The end plate connection eliminates the need to drill holes in the beam. A deep end plate would prevent beam end rotation and thereby end up transferring significant moment to the column. Therefore the depth of the end plate should be limited to that required for shear transfer.
- However adequate welding should be provided between end plate and beam web. To ensure significant deformation of the end plate before bolt fracture, the thickness of the end plate should be less than one half of the bolt diameters for Grade 8.8 bolts and one-third of the bolt diameter for Grade 4.6 bolts.



g. 1.3 Simple beam-to-column connections (a) Clip and seating angle
(b) Web cleats (c) Curtailed end plate

Rigid Connections

- Rigid connections transfer significant moments to the columns and are assumed to undergo negligible deformations.
- Rigid connections are necessary in sway frames for stability and also contribute in resisting lateral loads.
- In high-rise and slender structures, stiffness requirements may warrant the use of rigid connections.
- Extended end-plate connections have become the popular method for rigid connections.
- It is fairly easy to transfer about 0.7 to 0.8 times the yield moment capacity of the beam using these connections.
- Column web stiffening will normally be required and the bolts at the bottom are for preventing the springing action. These bolts can however be used for shear transfer.
- In the case of deep beams connected to relatively slender columns a haunched connection



.4 Rigid beam-to-column connections (a) Short end plate (b) Extended end plate (c) Haunched

Semi- Rigid Connections

- Semi-rigid connections are those fall between simple and rigid connections.
- Use of semi-rigid connections makes the analysis somewhat difficult but leads to economy in member designs. The analysis of semi-rigid connections is usually done by assuming linear rotational springs at the supports or by advanced analysis methods, which account for non-linear moment-rotation characteristics.
- The moment-rotation characteristics will have to be determined based on experiments conducted for the specific design. These test results are then made available as data bases. Simple models are proposed in the form of equations with empirical constants derived based on test results. Depending on the degree of accuracy required, the moment-rotation characteristics may be idealized as linear, bilinear or nonlinear curves.

Semi- Rigid Connections

- For obtaining the moment rotation relationship the Frye-Morris polynomial model is recommended by IS 800. The model has the form shown in the following equation
- $\theta_r = C_1\,(KM)^1 + C_2\,(KM)^3 + C_3\,(KM)^5$
  - Where, K = a standardization parameter dependent upon the connection type and geometry and C_1, C_2, C_3 = curve fitting constants.
- Table.1.2. shows the curve fitting constants and standardization constants for Frye-Morris Model. (All size parameters are in mm) Depending on the type of connection, the stiffnesses given in Table.1.3 may be assumed either for preliminary analysis or when using a linear moment curvature relationship. The values are based on the secant stiffenesses at a rotation of 0.01 radian and typical dimension of connecting angle and other components as given in the Table 1.3.

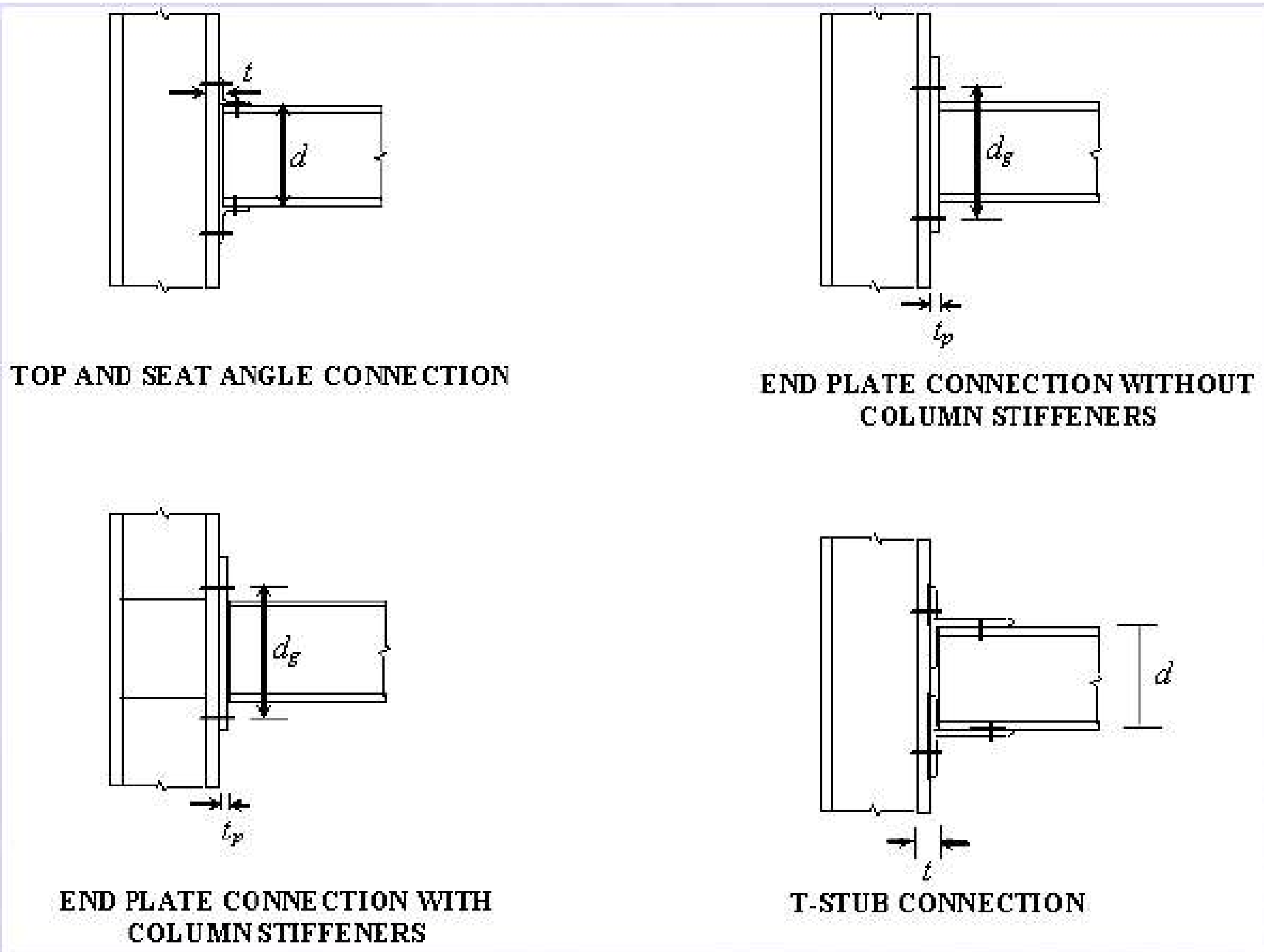


Fig. 1.5 Semi-rigid beam-to-column connections

Table 1.2. ***Connection constants in frye –morris model***

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	$C_1 = 2.1 \times 10^{-4}$ $C_2 = 6.2 \times 10^{-6}$ $C_3 = -7.6 \times 10^{-9}$	$K = 4.6 \times 10^{-6} d^{-1.5} t^{-0.5} l_t^{-0.7} d_b^{-1.1}$

Where

d_a = depth of the angle in mm t_a= thickness of the top angle in mm

l_a= length of the angle in mm d_b= diameter of the bolt in mm

d_g= center to center of the outermost bolt of the end plate connection in mm

t_p= thickness of ends- plate in mm

t= thickness of column flange and stub connector in mm

d= depth of the beam in mm l_t= length of the top angle in mm

Table 1.3 Secant stiffnesses

Sl No	Type of Connection	Dimension in mm	Secant Stiffeness kNm/radian
1.	Single Web Connection Angle	d_a=250 t_a=10 g=35	1150
2.	Double Web –Angle Connection	d_a=250 t_a=10 g=77.5	4450
3	Top and seat angle connection without double web angle connection	d_a=300 t_a=10 l_a=140 d_b=20	2730
4	Header Plate	d_p=175 t_p=10 g=75 t_w=7.5	2300

Semi- Rigid Connections

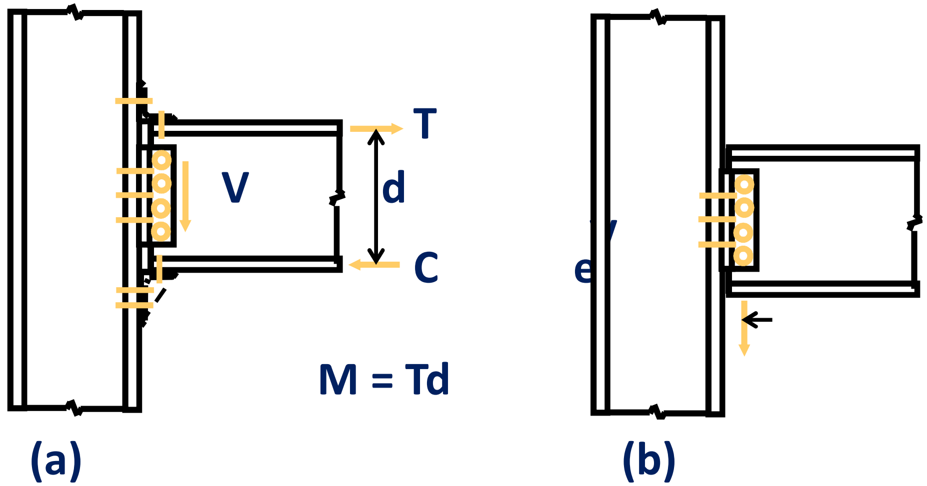
- The major advantage of semi-rigid connections is that they are cheaper than rigid connections and allow the optimum utilization of the beam member.

- To understand the second point, consider a beam with simple supports over a span L, subjected to a concentrated load W at mid-span. The mid-span bending moment will be WL/4. On the other hand, if the beam is provided with rigid supports, the maximum moment is WL/8 and occurs at the mid span as well as the support. The moment at the support gets transferred to the column and so may not be desirable.
- By using a semi-rigid connection we can control the mid span and support moments to the desired value.

GENERAL ISSUES IN CONNECTION DESIGN

Assumptions in traditional analysis

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



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

BEAM-TO-COLUMN CONNECTIONS

(a) Simple – transfer only shear at nominal eccentricity
Used in non-sway frames with bracings etc.
Used in frames upto 5 storeys

(b) Semi-rigid – model actual behaviour but make analysis difficult (linear springs or Adv. Analysis). However lead to economy in member designs.

(c) Rigid – transfer significant end-moments undergoing negligible deformations.
Used in sway frames for stability and contribute in resisting lateral loads and help control sway.