LEARNING MATERIAL OF STRUCTURAL DESIGN - II

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STRUCTURAL DESIGN-II

Introduction :-

A structure is an assemblage of a group of elements or members capable of withstanding external loads and transmitting them safely to the foundation.

-) Infrastructural development of the country mainly consists of structures like buildings, bridges etc. which mainly comprises of two basic construction materials i.e concrete and steel.

-) depending upon the orientation of structures and their structural use, the members are subjected to axial forces, bending or torsion or a combination there of and are accordingly named based upon their nature of stresses i.e tension, compression or flexural members etc.

-) design of a building structure focuses two aspects namely :- (i) functional design, (ii) structural design. The first part take into consideration the purpose it is to serve like requirements of ventilation, lighting etc. The second part consist in proportionating various elements of the structure for safe transmission of loads with due consideration of economy of materials and labour.

ommon Steel Structures :

n earlier days manufacture of structural steel s confined to a very limited range which has been re come by manufacture of high grade steels with sireable properties and composition through advancement technology.

teel has been extensively used as a building material n various types of structures. Some common example : steel structures are skeleton of high rise buildings, ransmission line towers, over head tanks, chimneys tc.

Steel structures can be broadly subdivided into wo groups.

(I) Framed structures - ex: combination of beams, columns etc.

(II) Sheet structures :- ex- tanks, sheets, chimney etc.

dvantages of Steel structure

Smaller weight to strength ratio :- It has smaller weight to strength ratio resulting in light weight tructures for covering large spans.

Speed of erection :- steel structure can be speedily onstructed due to prefabrication in the workshop.

(3) Addition, alteration and strengthening :- Addition and alteration of steel structures can be easily accomplished by welding and hence steel structures can be strengthened at any later time.

(4) Easy dismantling and transportation :- By using bolted connection, steel structures can be easily dismantled and conveniently handled. It can be easily transported to other sites being light weight & small volume.

(5) Gas & water tight joints :- Carefully made joints result in water and gas resistant construction like water tanks and pipe lines

(6) High scrap & recyclable value :- It has high scrap value for it can be easily reused after dismantling & also can be economically recycled.

Disadvantages of steel structures

1. Corrosion susceptibility :- steel structures when exposed to humid atmosphere are liable to corrosion.

2. High maintainance cost :- They require regular painting & maintainance.

3. Chemical detoriation :- It deteriorates when comes in contact with certain chemicals or gases.

4. Fire & heat susceptibility

5. Costly and susceptible to theft

TYPES OF STEEL :-

Steel is an alloy of iron & carbon and certain special properties can be imparted to it by addition of small percentage of manganese, sulphur, phosphorus chromium etc.

→ The structural steel that is mainly used for manufacture of rolled steel sections can be broadly divided in to -

(1) Standard structural or mild steel

(2) High tensile steel

Properties of structural steel

The properties of steel may be divided in to two groups.

(a). Physical properties

(b) mechanical properties

(a) Physical properties

(i) Unit mass of steel (δ) = $7850\ kg/m^3$

(ii) Modulus of elasticity, $E = 2.0 \times 10^5\ N/mm^2$

(iii) Poisson's ratio, $\mu = 0.3$

(iv) Modulus of Rigidity 'G' $= 0.769 \times 10^5\ N/mm^2$

(v) Coefficient of thermal expansion $\alpha_t = 12 \times 10^{-6} /^\circ C$

(b) Mechanical Properties

The mechanical-properties of steel largely depends on its chemical composition, rolling methods, rolling thickness, heat treatment. Some of the important mechanical properties of structural steel are as follows:-

(i) Yield stress (fy)

(ii) Ultimate stress (fu)

(iii) The maxm percentage elongated on standard gauge length

(iv) Notch toughness

Rolled steel sections

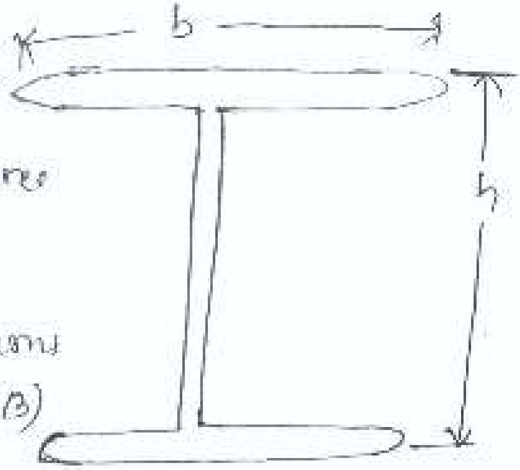
Steel structures are built with steel sections of standard shapes, sizes and length that are rolled in steel mills.

→ Various types of rolled steel sections standardised by BIS & manufactured are listed below:-

(i) Rolled steel I- sections
(ii) Rolled steel channel- sections
(iii) Rolled steel angle sections
(iv) Rolled steel 'T' sections
(v) Rolled steel bars
(vi) Rolled steel tubes
(vii) Rolled steel plates

I- sections

The following 5 series of rolled steel I-section are manufactured in India



(a) Indian Standard Junior Beams (ISJB)

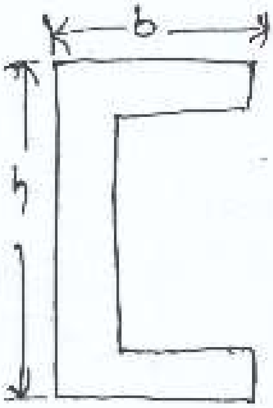
(b) Indian Standard Light Beam (ISLB)

(c) Indian Standard Medium Beam (ISMB)

(d) Indian Standard wide flanged Beam (ISWB)

(e) Indian Standard Heavy Beams (ISHB)

Channel sections



These sections are classified into following four series:-

(a) Indian Standard Junior Channel (ISJC)

(b) Indian Standard Light Channel (ISLC)

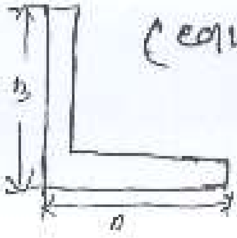
(c) Indian Standard medium weight Channel (ISMC)

(d) Indian Standard Special Channel (ISSC)

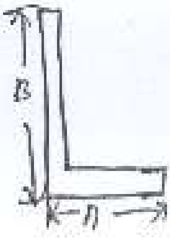
Angle secⁿ

(i) Indian Standard Equal Angle (ISA)

(ii) Indian Standard Unequal Angle (ISA)



(equal Angle A = B)



(Un equal Angle secⁿ A ≠ B)

Special Consideration in steel design :

Steel design differs from other design methods in following aspects

(1) minm thickness — In view of corrosion, the minm thickness of the structural steel members are to be specified, otherwise a very small amount of corrosion may result in to reduction of large percentage of effective area, if very thin sections are used.

(2) Shape and size :

Steel is manufactured in rolling mills and are available in standard shapes sizes. Hence depending upon the site requirements and loading conditions steel structures are designed considering any of the available sections or their combinations.

(3.) Connection design :

during fabrication and assembling, various standard sections in a member & the members themselves in a structure are to be suitably connected by welding, bolting.

Loads & load combination

Loads

The forces that act on a structure are called Loads.

Types of loads

(i) dead load (DL)

(ii) Imposed load (IL)

(iii) Wind load (WL)

(iv) Earthquake load (EL)

(v) Erection load (EL)

dead load:

The loads that are permanently attached to a structure are called dead load & such loads do not change their magnitude, direction or position with time.

Ex:- self weight of the member

Imposed load

The loads that are not permanently attached to a structure or part of the structure but act over a substantial duration of time i.e imposed upon the structure from out-side, are known as imposed loads.

Live load

The loads that are liable to change their position from time to time are called live loads.

Ex: weight of furnitures, movable partitions etc.

Wind load

The forces exerted by horizontal as well as vertical components of wind is known as wind load.

Earthquake load

The forces resulting from both horizontal and vertical components of acceleration imparted to the structures on the ground due to earthquake tremors are known as earthquake loads.

Load combinations

Of the various kinds of loads that are likely to act on a structure, a judicious combination of the probable loads is necessary to ensure safety as well as economy of the structure.

The recommended load combinations are

(1) DL
(2) DL + IL
(3) DL + WL
(4) DL + EL
(5) DL + TL
(6) DL + IL + EL
(7) DL + IL + TL
(8) DL + WL + TL
(9) DL + IL + EL + TL

DL = Dead load
WL = Wind load
TL = Temporary load
IL = Imposed load
EL = Earthquake load

Structural Analysis

In order to find the effect of loads on a structure & its members & connections i.e. the internal forces or moments developed in the members of the structure, the structural analysis is carried out. The IS code permits the following methods of analysis.

(a) Elastic Analysis
(b) Plastic Analysis
(c) Advance Analysis
(d) dynamic Analysis

Elastic Analysis

This method of analysis is also know as working stress analysis. It is based on the assumption that no fibre of the member has yielded for the design load and stress is linearly proportional to strain.

-) The analysis may be carried out into two stages

(1) First order Analysis
(2) Second order Analysis

Plastic Analysis

In this method, it is assumed that a plastic hinge is formed when every fibre at a section reaches yield stress and after plastic hinge is formed, infinite rotation takes place without resisting any additional moment i.e its resistance to moment remain constant.

Advanced Analysis

If the actual behaviour of a frame with full lateral restraints can be accurately modelled in respect of its actual behavior, an advanced structural analysis may be carried out.

Dynamic Analysis

Dynamic Analysis is carried out by seismic coefficient method or by response spectrum method.

Design & design philosophies

Steel structure should be designed and constructed to satisfy the requirements of strength, stability, serviceability, brittle fracture, fatigue, fire with due regard to economy.

The design philosophies are listed below

(i) Working stress method (WSM)
(ii) Ultimate load design (ULD)
(iii) Limit state design (LSD)

Brief Review of principles of limit state design

- A structure may become unfit for used not only when it collapse but also when it violates the serviceability requirements of deflection, vibrations, cracks due to fatigue, corrosion & fire.
- In LSM, various limits are fixed to consider a structure as fit.
- This design is based on both probable load & probable strength.
- Thus philosophy of LSM design is to see that "structure remains fit for use throughout its designed life by remaining within the acceptable limit of safety & serviceability requirements."

CHAPTER-02

Structural Steel Fasteners & connections : Bolts

Introduction

Different elements ore members of steel structures are required to be joined to one another either at their ends or at some intermediate length in order to facilitate the transmission or distribution of member forces or for the purpose of stability as the case may be, which is known as connection.

-> The various elements of a steel structure like beams, columns etc are connected by fasteners or connections.

-> different types of fasteners available in the design are :-

(1) Rivets
(2) Bolts
(3) welds
(4) pins

Bolted Connection

A bolt may be defind as a metal pin with a head at one end and a shank threaded portion at the other end to receive a nut.

-> steel washers are usually provided under the bolts as well as under the nut to distribute the clamping pressure on the bolted members.

→ The washer also prevents the threading to give large bearing pressure on the connecting members.

Types of Bolts

The following types of bolts are in common use.

(1) Unfinished bolts or black bolts

(2) Finished bolts or turned bolts

(3) High strength friction grip bolts (HSFG bolts)

Unfinished or black bolts

These are also known as ordinary or common bolts. These bolts are made from low carbon mild steel round rods with square or hexagonal head and the shank is left unfinished or rough.

Finished Bolts or turned bolts

These are close tolerance bolts which are formed mild-steel hexagonal rods and are made by turning to circular shape. Turned bolts may be either precision bolts or semi-precision bolts.

High strength Friction Grip Bolts (HSFG)

These bolts are made from high strength steel made like black bolts, but the surface of the shank of these bolt is kept unfurnished and these bolts are tightened until very high tensile stresses are developed.

Advantages of Bolted connection

The following are the advantages of bolted connection:-

(i) Use of simple tools & less skilled labour & working area.

(ii) Speedy & noiseless errection

(iii) Economical due to reduced labour & equipment cost.

(iv) minm strength reduction at joint due to less number of holes or bolts

(v) Easy alteration or dismantling of connections.

Dis-advantages of Bolted connection

(i) High cost of material

(ii) Reduced tensile strength due to area reduction at the root of threads

(iii) Gross area is reduced due to presence of bolt holes.

(iv) susceptibility to loosening of bolts under vibration and dynamic loads.

(v) Large joint space, when heavy loads are required.

Classification of bolts based on load transfere mechanism:-

Based on load transfere mechanism, bolted connection may be divided into two groups

(i) Bearing type or slip type connections

(ii) Friction grip type or slip critical connections,

Advantages of HSFG bolts over Bearing type Bolts

(i) Rigidity of joints due to no slip condition

(ii) No shearing or bearing stresses in members as the load transfere mechanism is mainly by friction.

(iii) Large clamping forces provide high static strength of joints.

(iv) Lack of stress concentration in holes leads to high fatigue strength

(v) Smaller length of joint.

Disadvantages of HSFG Bolts over Bearing type bolts

(i) Material cost of HSFG bolt is greater than that of ordinary bolts

(ii) Special workmanship is required

Types of Bolted connections

There are two types of bolted connections
(a) Lap joint (b) Butt joints

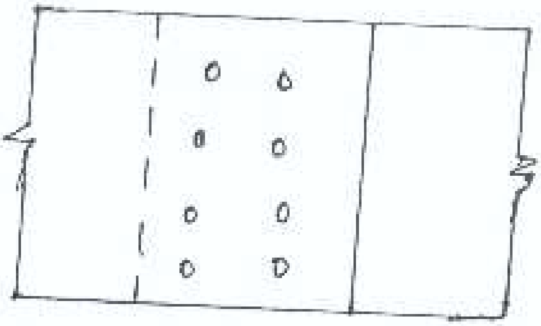
Lap joints

In this type of joints, the two members to be connected over lap one another. This constitutes the simplest type of joint requireing no extra accessories like cover plates.

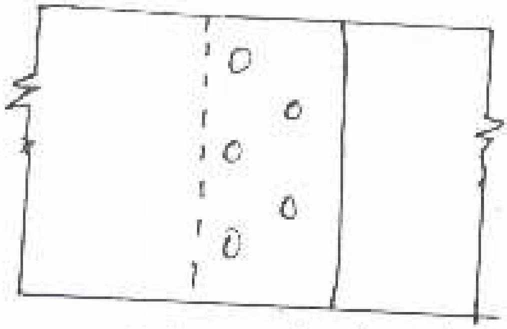
→ If there is one line of bolts, it is called single bolted lap joint.

→ If there is two lines of bolts, it is called a double bolted lap joint.

→ In this case the bolts are subjected to shear in one plane & hence known as bolts in single shear.



chain bolting



zig-zag bolting

Butt joint

In this type of joint, the two members to be connected are placed end to end i.e butt against each other & the connection is made by providing additional plate either on one side (single cover) or on both the sides (double cover butt joint). These additional plates are called cover plates & the members are called main plates.

→ depending upon the number of lines of bolts on either sides of the butting plane, the butt joints are known as single bolted, double bolted or tripple bolted butt joints.

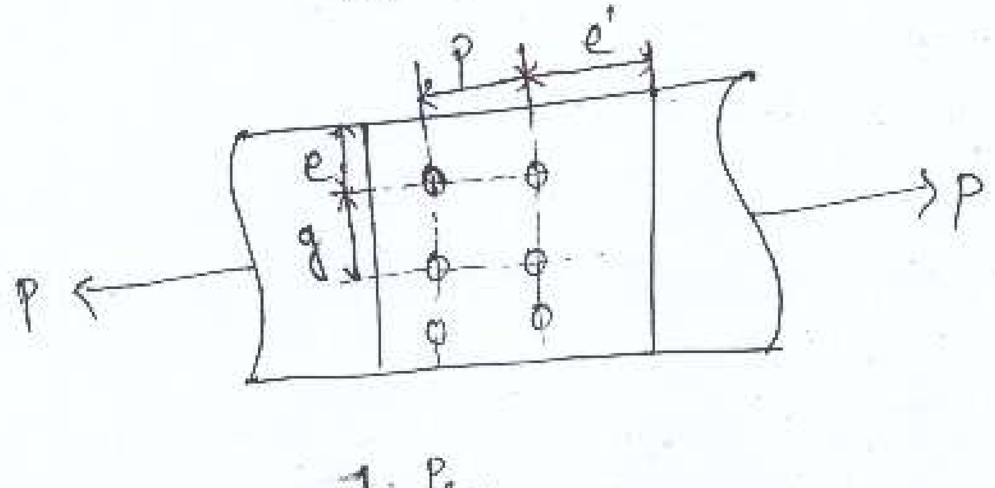
Terminology

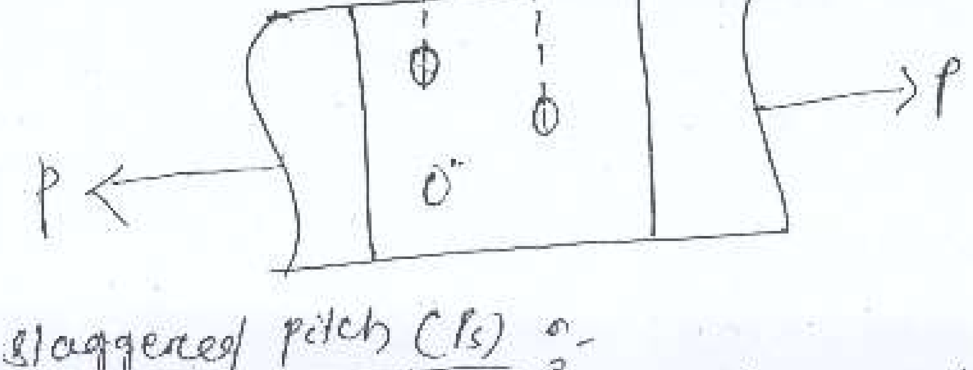
Pitch (P) :- It is the c/c spacing of the bolts in a row measured along the direction of load.

Gauge dist^n (g) :- It is the distance b/n two consecutive bolts of adjacent rows and is measured at right angle to the direction of load.

Edge dist^n (e) :- It is the distance of centre of bolt holes from adjacent edge of the plate measured at right angle to the direction of load.

End distance (e') :- It is the distance of nearest bolt hole from end of the plate measured along the direction of load.





Staggered pitch (P_s) :-
It is the c/c distance of staggered bolts measured obliquely on the member.

Specification for bolted joints

(1) Pitch shall not be less than 2.5 d, where 'd' is the nominal diameter of bolts.

(2) Pitch shall not be more than

(a) 16t or 200mm, whichever is less in case of tension member

(b) 12t or 200mm, whichever is less in case of compression member

where,

t = thickness of thinnest plate

(3) In case of staggered pitch, pitch may be increased by 50% values in specified above, provided gauge distance is less than 75mm.

(4) In case of butt joints maxm pitch is to be restricted to 4.5d for a distance of 1.5 times width of plate from butting surface.

(5) The gauge length 'g' < (100+4t) or 200 whichever is less.

(6) Edge distance

$e > 1.7 \times$ hole diameter (Hand flame cut)

$e > 1.5 \times$ hole dia (rolled, machine flame cut).

(7) $e < 12t\varepsilon$, where $\varepsilon = \sqrt{\frac{250}{f_y}}$

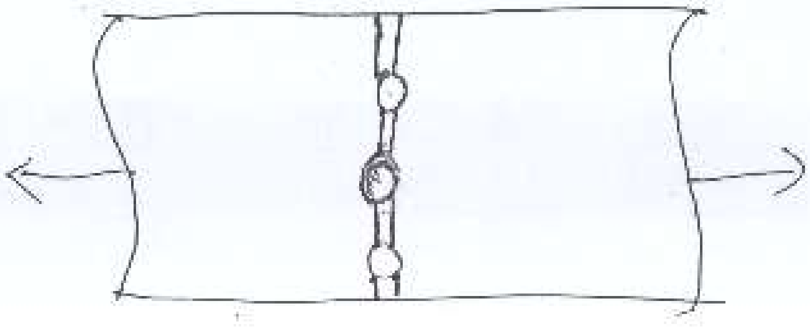
$e < (40 + 4t)$

Nominal dia of bolts (d) in mm	dia of hole (d0) in mm
12	13
14	15
16	18
18	20
20	22
22	24
24	26
27	30
30	33
Over 33 mm	bolt dia + 3mm

Failure of a bearing type bolted joint

A bolted joint may fail in any of the following manner

(1) Rupture of the plate b/n bolt holes :-
The strength of plate is reduced by bolts holes and the plate may tear off along the line of the bolt holes, such type of failure is for tension members only.



(ii) Shearing of Bolt :-

The bolts may fail by shearing, if the shearing stress exceeds their shearing strength. In lap joints & single cover butt joints, the bolts are sheared at one plane only

→ In a double cover butt joint, the bolts are sheared at two planes.

(iii) Bearing of Bolt or plate :-

The plate or bolt is crushed if the compressive stress exceeds the bearing strength of the plate or bolt.

(iv) Bursting or cracking of the edge

The plate will crack at the back of a bolt, if it is placed very near to the edge of the plate.

Design strength of plates in a joint :-

Plates in a joint made with bearing type of bolts may fail due to (i) bursting of the edge, (ii) crushing of plates in bearing or (iii) rupture of plates. The bursting of the edges and crushing failure of plates are generally avoided if the minm edge/end distances are provided.

⇛ The design tensile strength of a plate in the joint is the strength of the thinner member against rupture given by :-

$$\boxed{T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}}$$

γ_{m1} = partial safety factor = 1.25
f_u = ultimate tensile stress

$$\boxed{A_n = (b - n d_0) t}$$ for chain bolting

$$\boxed{A_n = \left[b - n d_0 + \sum \frac{p_{si}^2}{4 g_i} \right] t}$$ for staggered bolting

where,

A_n = net effective area of the plate at critical secn
b = width of the plate
t = thickness of thinner plate of the joint
d_0 = dia. of the bolt hole
g = Gauge distn
p_s = length of the staggered pitch
n = no. of bolt holes at critical secn
i = subscript for summation for all inclined leg.

Design strength of bearing type of bolts in a joint

The design strength of bearing type of bolts is the less of the —

(a) shear capacity or

(b) Bearing capacity

Shear capacity or shear strength (V_{dsb})

The design shear strength of the bolt,

$$\boxed{V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}}}$$

where,

γ_{mb} = partial safety factor of material of bolt

V_{nsb} = nominal shear capacity of bolt

$$\boxed{V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + n_s A_{sb}\right)}$$

where,

f_{ub} = ultimate tensile strength of a bolt

n_n = no. of shear planes with threads intercepting the shear plane

n_s = no. of shear planes without threads intercepting the shear plane

A_{sb} = nominal plain shank area of the bolt $= \pi/4\, d^2$

A_{nb} = net shear area of the bolt at threads

$= 0.78\, \pi/4\, d^2$

Reduction factors for shear capacity of bolts

(i) Reduction factor for long joints (β_{lj})

when the length of the joint l_j exceeds $15d$, the nominal shear capacity V_{nsb} shall be reduced by the factor β_{lj}

$$\boxed{\beta_{lj} = 1.075 - \frac{l_j}{200d}}$$

subjects to the limits $0.75 \le \beta_{lj} \le 1.0$

(ii) Reduction factor for large grip length (β_{lg})

when the grip length l_g exceeds 5 times the diameter 'd' of bolts, the design shear capacity shall be reduced by a factor β_{lg}.

$$\boxed{\beta_{lg} = \frac{8d}{3d + l_g}}$$

(iii) Reduction factor for packing plate (β_{pk}):-

If the thickness of packing plates is more than 6mm in a joint, the shear capacity is reduced by a factor

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

t_{pk} = thickness of the thicker packing in 'mm'.

Thus the complete formula for nominal shear capacity of bolt

$$V_{nsb} = f_{ub}\,(n_n A_{nb} + n_s A_{sb})\,\beta_{lj}\beta_{lg}\beta_{pk}$$

Bearing capacity or bearing strength (V_{dpb}) :-

The design bearing strength of the bolt V_{dpb} is given by

$$\boxed{V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}}$$

where,

V_{npb} : nominal bearing strength of a bolt

$$\boxed{V_{npb} = 2.5\, k_b\, d\, t\, f_u}$$

where,

k_b is smaller of

$$\begin{cases} \text{(i) } \dfrac{e}{3d_0} & \text{(ii) } \dfrac{p}{3d_0} - 0.25 \\ \text{(iii) } \dfrac{f_{ub}}{f_u} & \text{(iv) } 1.0 \end{cases}$$

e & p : end distⁿ & pitch distⁿ

d_0 : dia. of bolt hole

Assumptions of bearing bolts

(1) The stress distribution on the plates between the bolt holes is uniform

(2) The friction between the plates is negligible

(3) The shearing stress is uniformly distributed over the cross-secⁿ of the bolt

(4) The bolts in a group share the direct load equally

(5) Bending stress developed in bolts is neglected

Efficiency of a joint (η)

The efficiency of a joint is the ratio of the strength of the joint and the original strength of the member without bolt holes.

Mathematically,

$$\boxed{\eta = \frac{\text{strength of the joint}}{\text{strength of solid plate}} \times 100}$$

- **Problem-1**

Two steel plates (fe410) of 16mm thick are to be joined by 24mm dia bolts of grade 4.6. Assuming a pitch of 60mm and edge distance of 40mm, calculate the strength of the bolt for the following cases.

- (a) Lap joint.

(b) Single cover butt joint, cover plate being 12mm thick

(c) double cover butt joint, each cover plate 10mm thick.

Soln

Given data

Thickness of the plate, $t = 16mm$, pitch(P) = 60mm

edge distance (e) = 40mm

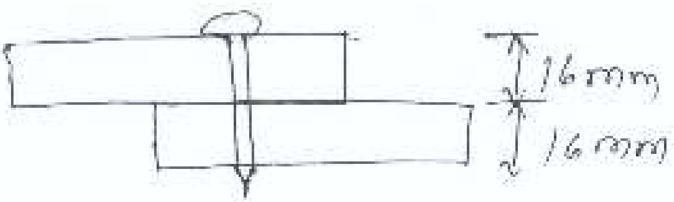
for fe 410 grade of steel, $f_u = 410 N/mm^2$, $f_y = 250 N/mm^2$

dia. of bolt (d) = 24 mm

dia. of hole (d_0) = 24+2 = 26mm

for grade of bolt 4.6, $f_{ub} = 400 N/mm^2$

(a) Lap joint



There is only one plane of shearing at the level of two plates, so the bolt will be in single shear & bearing.

Strength of the bolt in shearing

Assuming that the threads intercept the shear plane, the no. of shear planes at thread $n_n = 1$

the " " " at shank $n_s = 0$

Net shear Area of the bolt $(A_{nb}) = 353\ mm^2$

There is no reduction factor

Nominal shear strength

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + \cancel{n_s A_{sb}}^{\,0}\right)$$

$$= \frac{400}{\sqrt{3}}(1 \times 353)$$

$$= 81.52\ kN$$

design strength in shear (V_{dsb})

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{81.52}{1.25} = 65.22\ kN$$

$$\boxed{V_{dsb} = 65.22\ kN}$$

Strength of bolt in bearing (V_{dpb})

Nominal bearing strength of the bolt (V_{npb})

$V_{npb} = 2.5 k_b d t f_u$

where k_b $\left\{ \begin{array}{l} (i)\ \frac{e}{3d_0} = \frac{40}{3 \times 26} = 0.513 \\ (ii)\ \frac{p}{3d_0} - 0.25 = \frac{60}{3 \times 26} - 0.25 = 0.519 \\ (iii)\ \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975 \\ (iv)\ 1.0 \end{array} \right\}$ least 0.513

So, $k_b = 0.513$

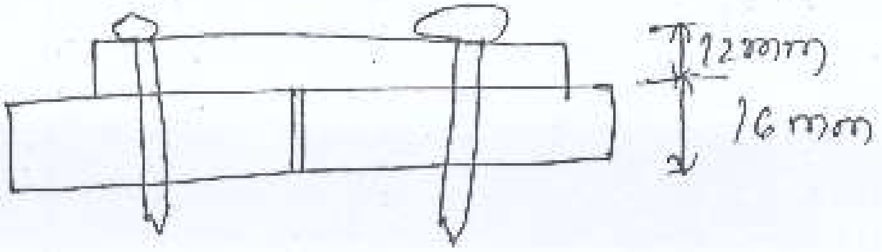
$V_{npb} = 2.5 \times 0.513 \times 24 \times 16 \times 410 = 201.917\ kN$

Design strength in bearing (V_{dpb})

$$\boxed{V_{dpb} = \frac{201.917}{1.25} = 161.533\ kN}$$

The strength of bolt = minm of strength in shear & bearing i.e 65.22 kN

(b) Single cover butt joint



In this case also the bolt will be in single shear and bearing. Strength of the bolt in single shear at the junction of cover plate and main plate

$V_{dsb} = 65.22$ kN.

→ The bearing of the bolt will be calculated against the thinner plate i.e cover plates of thickness $t = 12$mm

strength of bolt in bearing (V_{dpb})

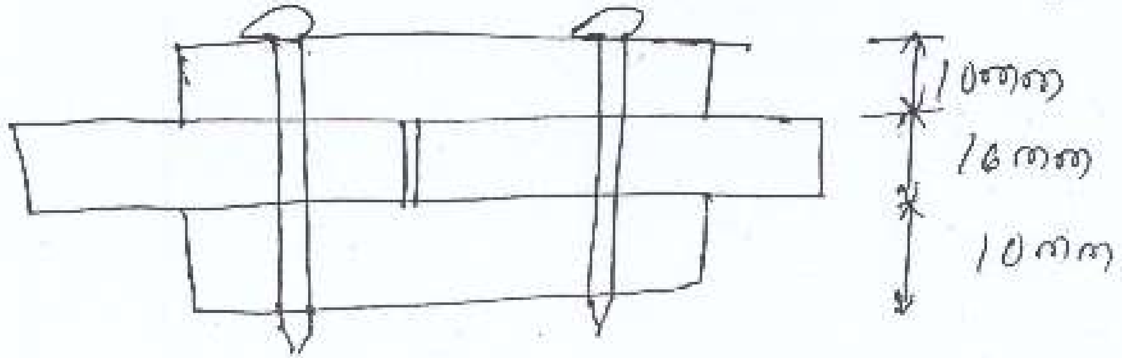
$$V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.513 \times 24 \times 12 \times 410}{1.25} = 121.15 \text{ kN}$$

∴ strength of the bolt = 65.22 kN

(c) <u>Double cover butt Joint</u>

The bolt will be in double shear & bearing



The thickness to be considered for bearing will be the least of the aggregate thickness of cover plates or thickness of the main plate i.e $t = 16$mm

The strength of bolt in double shear

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

(Here $n_n = 1$, $n_s = 1$) $A_{nb} = 353 mm^2$, $A_{sb} = 452 mm^2$

$$= \frac{400}{\sqrt{3}} \left(\frac{1 \times 353 + 1 \times 452}{1.25} \right) = 148.73 kN$$

$$\boxed{V_{dsb} = 148.73 kN}$$

Strength of bolt in bearing (V_{dpb})

$$V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}}$$

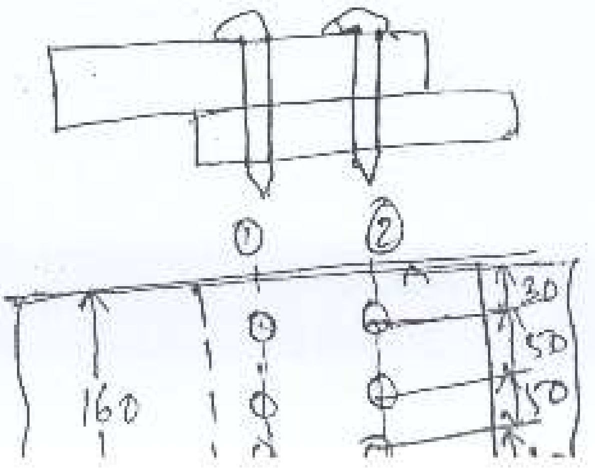
$$= \frac{2.5 \times 0.573 \times 24 \times 16 \times 410}{1.25} = 161.53 kN$$

Strength of bolt $= 148.73 kN$ (Ans)

Problem

Find the maxm force that can be transmitted through a double bolted chain lap joint consisting of 6 bolts in 2 rows. Given that M16 bolts of grade 4.6 & plates of Fe 410 are used. Also find the efficiency of the joint.

Soln



Given data

Thickness of plate $(t_1) = 10mm$, $(t_2) = 12mm$

total no. of bolts $n = 6$

dia. of bolt $(d) = 16mm$

dia. of bolt hole $(d_0) = 18mm$

pitch $(P) = 50mm$

edge $(e) = 30mm$

Grade of bolt $= 4.6$, $f_{ub} = 400 N/mm^2$

Grade of plate fe 410, $f_u = 410 N/mm^2$, $f_y = 250 N/mm^2$

Soln

strength of plate in the joint due to rupture

thickness of thinner plate $(t) = 10mm$

width of plate$(b) = 160mm$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

$$A_n = (b - n d_0) t \text{ (for chain bolting)}$$

$$= (160 - 3 \times 18) \times 10 = 1060 mm^2$$

design strength of plate (T_{dn})

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{me}} = \frac{0.9 \times 1060 \times 410}{1.25} = 312.912 kN$$

Strength of bolt in the joint

(i) design strength in shear (V_{dsb})

No. of shear plane at thread $n_n = 6$

No. of " " " shank $n_s = 0$

$A_{nb} = 0.78\,\pi/4\,d^2 = 157\,mm^2$

Since there is no reduction factor so $\beta_{lj}\,\beta_{lg}\,\beta_{pk} = 1$

Nominal shear strength (V_{nsb})

$$V_{nsb} = \frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + \cancel{n_s} A_{sb}\right)$$

$$= \frac{400}{\sqrt{3}}\,(6 \times 157)$$

$$= 217.516\ kN$$

∴ design strength in shear (V_{dsb})

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{217.516}{1.25} = 174.036\ kN$$

(ii) design strength in bearing (V_{dpb})

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$V_{npb} = 2.5 k_b\,d\,t\,f_u$

$$k_b \begin{cases} \text{(i)}\ \dfrac{e}{3d_0} = \dfrac{30}{3 \times 18} = 0.56 \\ \text{(ii)}\ \dfrac{p}{3d_0} - 0.25 = \dfrac{50}{3 \times 18} - 0.25 = 0.676 \\ \text{(iii)}\ \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.9756 \end{cases} \quad \text{least} = 0.56$$

$\therefore k_b = 0.56$

$V_{npb} = 2.5 \times 0.56 \times 16 \times 10 \times 410 = 91.840$ kN/bolt

design strength in bearing/bolt,

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{91.840}{1.25} = 73.47 \text{ kN}$$

design strength in bearing of 6 bolts $= 6 \times 73.47$ kN

$= 440.832$ kN

design strength of bolts $= 174.036$ kN

$\therefore$ strength of the joint = min^m^ of strength of plate or strength of the bolts $= 174.036$ kN

Efficiency of the joint

$f_y = 250 \text{ N/mm}^2$

$A_g = 160 \times 10 = 1600 \text{ mm}^2$

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = 1600 \times \frac{250}{1.1} = 363.636 \text{ kN}$$

$$\eta = \frac{\text{strength of the joint}}{\text{strength of solid plate}} \times 100$$

$$= \frac{174.036}{363.636} \times 100 = 47.86\%$$

Shear Capacity of HSFG Bolts

The nominal shear capacity of a bolt is given by

$$\boxed{V_{nsf} = \mu_f n_e k_h F_0}$$

where,

μ_f = coefficient of friction

n_e = no. of effective interfaces offering frictional resistance to the slip.

k_h = 1.0 for fasteners in clearance holes
= 0.85 " " oversized and short slotted holes
= 0.70 for fasteners long slotted holes

F_0 = minm bolt tension = $A_{nb} \cdot f_0$

A_{nb} = Net area of the bolt at threads

f_0 = proof stress = $0.70 f_{ub}$

- **Problem**

Two plates of 12mm thick are joined by double-cover butt joint with 20mm dia HSFG bolts of property class 10.9 and cover plates of 8mm thick. Assuming that the fasteners are in clearance holes and slip factor as 0.20, determine the shear capacity of a bolt if slip resistance is designated at (i) service load (ii) ultimate load.

Soln

For 20mm dia HSFG bolts of property class 10.9,

f_{ub} = 1140 N/mm²

$A_{nn} = 245 mm^2$

for fasteners in clearance holes, $k_h = 1.0$

for double cover butt joint $n_e = 2$

slip factor $\mu_f = 0.25$

minm bolt tension at installation)

$F_0 = 0.7 f_{ub} \times A_n = 0.7 \times 1040 \times 245 = 178.36$ kN

nominal shear capacity of bolt (V_{nsf})

$V_{nsf} = \mu_f n_e k_h F_0$

$= 0.25 \times 2 \times 1.0 \times 178360 = 89.18$ kN

(i) shear capacity of a bolt, if slip resistance is designated at service load $\gamma_{mf} = 1.1$

$$V_{sf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{89180}{1.1} = 81.07 \text{ kN}$$

(ii) shear capacity of a bolt, if slip resistance is designated at ultimate load $\gamma_{mf} = 1.25$

$$V_{sf} = \frac{V_{nsf}}{\gamma_{mf}} = \frac{89180}{1.25} = 71.34 \text{ kN}$$

Design Procedure for Bolted joint

(i) The size of the bolt is determined from the unwin's formula $\boxed{d = 6\sqrt{t}}$ where t = thickness of the plate in mm & 'd' is the nominal dia of bolt.

2. The strength of the bolts in shear and bearing are computed assuming suitable value of pitch, edge distance and location of shear plane. The minm of the above is taken as the bolt value and the number of bolts required is obtained by dividing the applied force by bolt value.

3. The bolts are suitably arranged to produce a convenient and efficient joint.

4. The joint is checked for rupture strength of the plate with the assumed arrangement of bolts, which should be more than the applied load.

Problem

Two steel plates of 10mm and 12mm thick are to be joined by a lap joint so as to transmit a load of 120kN using 20mm dia bearing bolts of property class 4.6 and plates of grade fe 410. find the number and arrangement of bolts, if each of the plates are —
(i) 100 mm wide (ii) 200mm wide.

Soln Given data

for M_{20} bolts of property class 4.6, $f_{ub} = 400 N/mm^2$

dia. of bolt (d) 20 mm

dia. of bolt hole (d_0) = 20+2 = 22mm

$A_{nb} = 245 mm^2$, $A_{sb} = 314 mm^2$, $\gamma_{mb} = 1.25$

for fe 410 steel, $f_u = 410 N/mm^2$, $f_y = 250 N/mm^2$

Soln

For a lap joint, the bolts will be in single shear and assuming that the threads intercept the shear-plane, $n_n = 1$, $n_s = 0$

design strength of a bolt in shear

$$V_{dsb} = \frac{V_{nsb}}{\gamma_{mb}} = \frac{1}{\gamma_{mb}}\left[\frac{f_{ub}}{\sqrt{3}}\left(n_n A_{nb} + \cancel{n_s A_{sb}}^{\,0}\right)\right]$$

$$= \frac{1}{1.25}\left[\frac{400}{\sqrt{3}}\left(1 \times 245\right)\right]$$

$= 45.26$ kN

design strength of a bolt in bearing against thinner plate :-

$$V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}$$

$V_{npb} = 2.5 k_b d t f_u$, Assuming $e = 40$mm

$$k_b = \begin{cases} \text{(i)} \ \dfrac{e}{3d_0} = \dfrac{40}{3 \times 22} = 0.606 \\ \text{(ii)} \ \dfrac{p}{3d_0} - 0.25 = \dfrac{60}{3 \times 22} - 0.25 = 0.66 \\ \text{(iii)} \ \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976 \\ \text{(iv)} \ 1.0 \end{cases}$$

least $= 0.606$

so, $k_b = 0.606$

$$V_{dpb} = \frac{2.5 \times 0.606 \times 20 \times 10 \times 410}{1.25} = 99.38\ kN$$

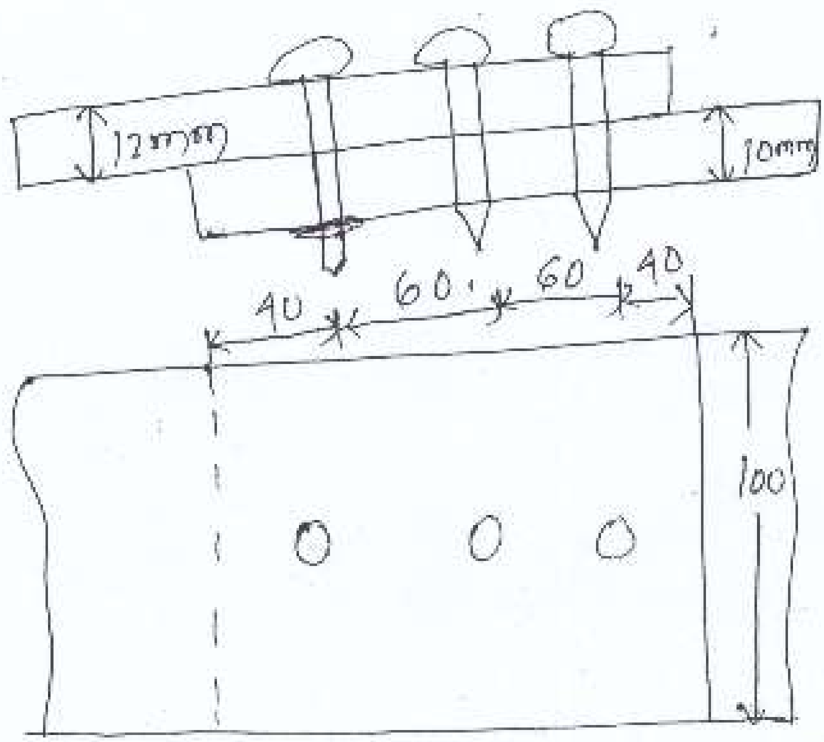
design strength of a bolt = min^m of shear or bearing or bolt value = 45.26 kN

No. of bolts reqd to transmit a load of 120 kN

$$n = \frac{120}{45.26} = 2.65 \simeq \text{say } 3 \text{ nos}$$

(i) when each plate is 100mm wide.

The bolts are to be arranged along the length in a row because width is not sufficient to accomodate them in a row along the width.



check for rupture strength of the plate

$b = 100, \quad n = 1$

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}, \qquad A_n = (b - n d_0) t$$

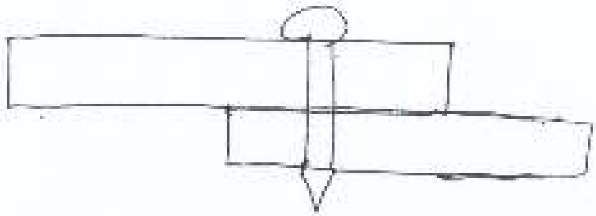
$$= (100 - 1 \times 22) 10 = 780 mm^2$$

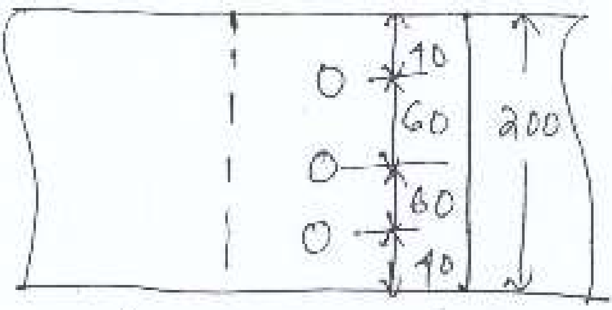
$$T_{dn} = \frac{0.9 \times 780 \times 410}{1.25} = 230.26\,kN > 120\,kN$$

230.26 kN > 120 kN (Hence safe)

(ii) when each flat is 200 mm wide

To reduce the length of the joint, the bolts may be arranged along the width in a row.





Check for rupture strength of the plate

design strength of the plate (T_{dn})

$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{ml}} = \frac{0.9 \times 1340 \times 410}{1.25}$$

$$= 395.57\,kN > 120\,kN \text{ (Hence ok)}$$

Welded Connection

welded consist of joining two pieces of metal by establishi a metallurgical bond between them through the applicati of pressure ore through fusion.

-) In othere words, welding is a method of connecting two pieces of metal by heating to a plastic ore fluid state.

Types of weld and welded joints

The basic types of welded joints are classified depending upon the types of weld. There are 3 types of welds

(1) Butt weld
(2) fillet weld
(3) slot weld & plug weld

Butt weld

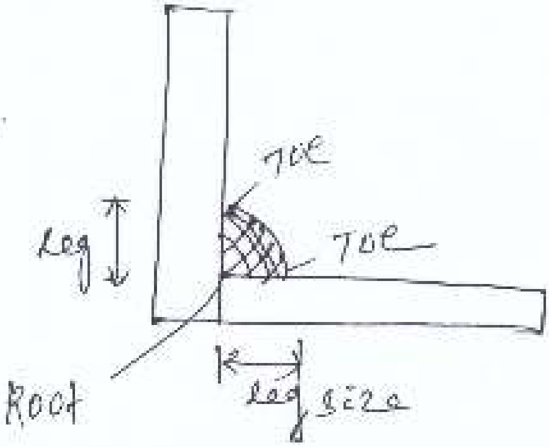
This is also known as groove weld. Butt welds are provided when the members to be joined are placed end to end ore aligned in the same plane.

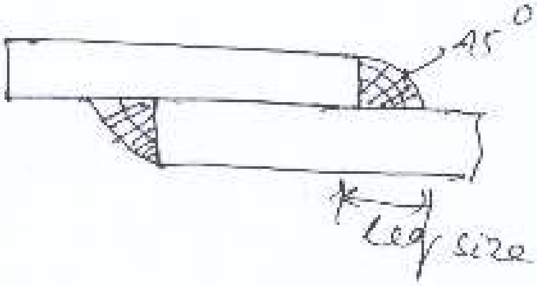
-) depending upon the shape of the groove made for welding, various types of groove welds are listed as follows.

Fillet weld

Fillet welds are provided when two members to be joined are in different planes.

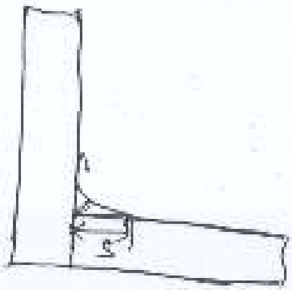
-> Fillet weld is a weld of approximately triangular cross-section joining two surfaces nearly at right angles to each other in lap, tee or corner types of joint.



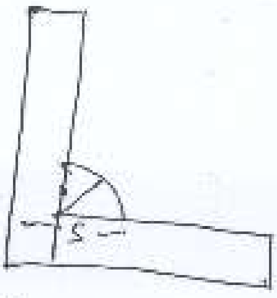


-> When the cross-section of fillet weld is isoceles triangle with face at 45°, it is called as standard fillet weld.

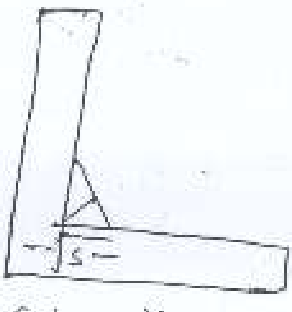
-> In special circumstances 30° & 60° angles may be used. Depending upon the shape of weld face, a fillet weld is known as concave fillet weld, convex fillet weld or as mitre fillet weld.



(a) concave



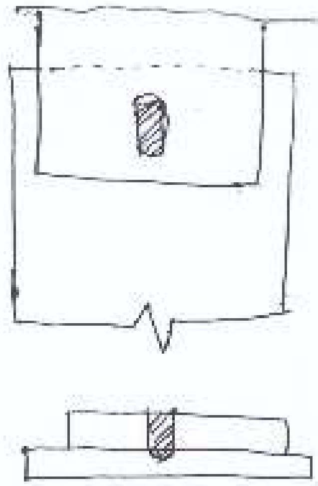
(b) convex



(c) mitre

Slot and plug welds

Slot and plug welds are used to supplement the fillet welds, when the reqd length of fillet weld can not be provided. The penetration of these welds into base metal is difficult to ascertain and the inspection of these welds is difficult.



Advantages of welded connections

(1) welding is more adaptable than bolting or riveting, as even circular sections can be easily connected by welding.

2. Full strength of a joint can be developed i.e 100% efficiency can be achieved in contrast to bolted or rivetted connection which can reach a maxm of (70-80)% efficiency.

3. Since there is no deduction for holes, the gross section is effective in carrying loads and there is no problem of mismatching.

4. Better resistance against fatigue, impact load

5. Results in lighter structures, due to absence of connecting plates, gusset plates etc.

6. Noise pollution is nearly eliminated

7. Presents good aesthetic appearance

8. Connections are water & air tight

Disadvantages of welded connections

1. Skilled labour & electricity is necessary for welding.
2. Due to uneven heating and cooling, internal stresses and warping develops.
3. Welded joints are more brittle & their fatigue strength is less.

I.S Code Provisions for Welding

Butt weld

1) Reinforcement ?-

1) Size of butt weld shall be specified by the throat thickness. In double 'U', double 'V', double 'J' butt welds, which gives complete penetration of welding. Size of butt weld shall be taken as thickness of thinner plate connected.

2) In case of incomplete penetration of welding effective throat thickness = minm thickness of weld metal.

2) In absence of appropriate data,
throat thickness = $\frac{5}{8}$th of thickness of thinner material

3) Effective length of butt weld = length of full size weld

4) minm length of weld
= 4 x size of weld

(5) for intermittent butt weld,

effective length $\geq$ 4 x size of weld

Space b/n two welds < 16 x thickness of thinner plate

Fillet weld

(1) Size

(a) The size of normal fillet weld shall be taken as the minm weld leg size

(b) for deep penetration weld with not less than 2.4mm

size of weld = minm leg size + actual penetration

(2) minm size of weld = 3mm

(It is provided to avoid risk of cracking)

plate thickness		minm size of weld
< 10mm	—	3mm
10-20mm	—	5mm
20-32 mm	—	6mm
32-50mm	—	8mm

* The maxm size of filled weld should be 1.5mm less than the nominal thickness of the edge.

(3) Effective throat thickness

= > 3mm & < 0.7t or t

The throat of a fillet is the length of perpendicular from the right angle corner to the hypotenuse.

Throat thickness = K x fillet size

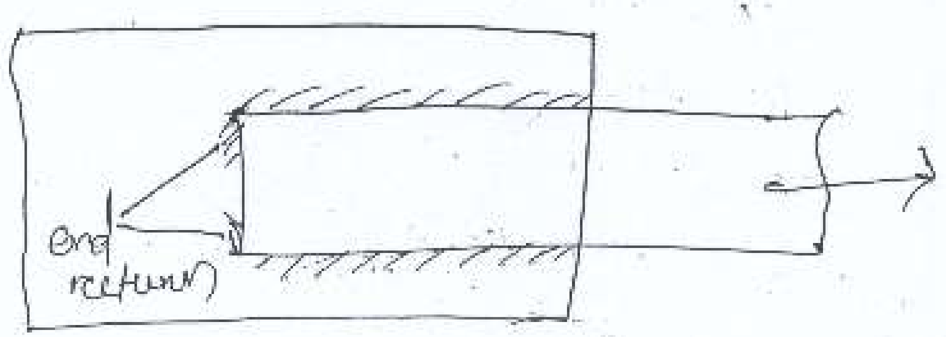
Angle b/n fusion faces	60-90°	91-100	101-106	107-113	114-120
Constant K	0.7	0.65	0.60	0.55	0.50

(4) Effective length

The effective length of a fillet weld is equal to its overall length minus twice the weld size. The effective length of a fillet weld designed to transmit load should not be less than 4 s

(5) End return

The fillet weld terminating at the end or side of a member should be returned around the corner when-ever practicable for a distance not less than twice the weld size.



(6) Over lap

The minm lap in a lap joint should not less than 4t or 40mm, which ever is more.

Plug and slot welds

(i) Size

(a) width or diameter should be not less than 3t or 25 mm whichever is more.

(b) corner radius in slotted hole should not be less than 1.5t or 12mm whichever is greater.

(ii) spacing

spacing should be 2t or 25mm which is more.

design stresses & design strength of welds

fillet weld, slot or plug welds :-

$$\boxed{\text{design strength of weld } f_{wd} = \frac{f_{wn}}{\gamma_{mw}}}$$

where,

f_{wn} = nominal strength of the fillet weld

$$f_{wn} = \frac{f_u}{\sqrt{3}}$$

Butt weld

$$\boxed{\text{design stress of the butt weld } f_{dw} = \frac{f_y}{\gamma_{mw}}}$$

design stress of butt weld in shear is given by

$$\boxed{\tau_{dw} = \frac{f_{yw}}{\sqrt{3}\,\gamma_{mw}}}$$

Problem

A steel plate 200mm x 12mm is welded to a 10mm thick gusset plate such that the overlap of the member is 250mm. If fillet weld of size 6mm is used for the connection, determine the design strength of the joint. Given that shop welding is to be done on three sides & grade of steel is fe 410.

(Given data

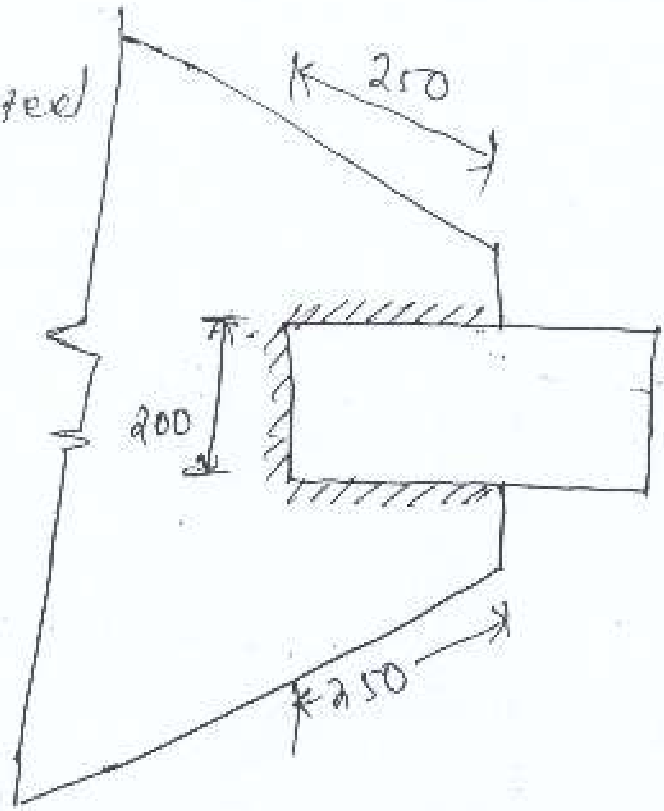
for fe410 grade steel

$f_u = 410 N/mm^2$

$f_y = 250 N/mm^2$

for shop welding

$\gamma_{mw} = 1.25$



Soln

Effe length of the weld (l_w)

$= 2 \times 250 + 200 = 700mm$ (Assuming end return)

Effective throat thickness $t_e = ks = 0.7 \times 6 = 4.2mm$

design strength of the weld $P_{dw} = l_w t_e \dfrac{f_u}{\sqrt{3}\gamma_{mw}}$

$= 700 \times 4.2 \times \dfrac{410}{\sqrt{3} \times 1.25} = 556.75 kN$

design strength of the plate

$A_g = b \times t = 200 \times 12 = 240 mm^2$

strength of the plate on yielding

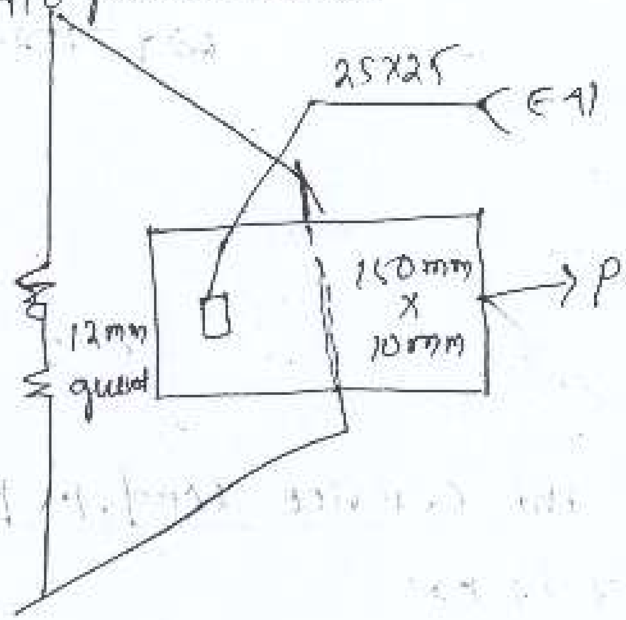
$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{200 \times 12 \times 250}{1.1} = 545.45 \text{ KN}$$

strength of the joint = minm of weld or plate strength

= 545.45 KN

Problem

determine the service load that can be transmitted through the connection shown in the fig. Assume fillet welding & fe 410 grade steel.



Given data.

for electrode E 41

$f_y = 330$ mpa

$f_u = 410$ to $510 \ N/mm^2$

$f_{ub} = 410 \ N/mm^2$, $f_y = 250 N/mm^2$

Soln

design strength of fillet weld, $P_{dw} = l_w t_e \frac{f_y}{\sqrt{3}\gamma_{mw}}$

$$= \frac{400 \times 3.5 \times 410}{\sqrt{3} \times 1.5} = 220.93\,kN$$

design stress of plug weld $f_{wd} = \dfrac{f_u/\sqrt{3}}{\gamma_{mw}}$

$$= \frac{410}{\sqrt{3} \times 1.5} = 157.81\,N/mm^2$$

Area of the plug weld $= 25 \times 25 = 625\,mm^2$

design strength of plug weld $= 625 \times 157.81 = 98.63\,kN$

total design strength =

$= 265.12 + 98.63 = 363.75\,kN$

Strength of the plate

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{150 \times 10 \times 250}{1.1} = 340.91\,kN$$

If P is the service load, $1.5P = 340.91\,kN$ or

$P = 227.27\,kN$

design procedure for butt weld

In case of complete penetration, butt weld design calculations are not reqd.

design procedure for fillet weld

1. Size of the weld is selected based on the thickness of the members to be joined
2. depending on the angle b/n fusion faces, the effective throat thickness is calculated.
3. If force to be transmitted is not given, design strength should be taken as the rupture strength
4. strength of the weld per mm length is calculated.
5. $le = \dfrac{\text{strength of weld per mm}}{\text{factored load}}$
6. Length of weld arranged suitably
7. Check for minm lap of the joint
8. End returns of length equal to twice the size of the weld at each end of the longitudinal fillet weld ~~are~~ provided.

Problem

design a suitable fillet weld to connect a tie bar 60mm x 8mm to a 12mm thick gusset plate so as to develop maxm force if (i) shop welding is done on two sides & (ii) fillet weld is done on three sides.

Given data

grade of steel Fe 410, $f_u = 410\ N/mm^2$, $f_y = 250\ N/mm^2$

for 12mm thick gusset plate,

minm size of weld = 5mm

for 8mm thick tie bar

maxm size of weld = 8 − 1.5 = 6.5mm

Hence let us provide a weld size of (s) = 6mm with 90° fusion faces.

Effective throat thickness (t_e) = 0.7s = 0.7×6 = 4.2mm

To develop maxm force, the design strength of weld be equal to strength of the plate.

<u>Strength of the plate in yielding</u>

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{60 \times 8 \times 250}{1.1} = 109.09\,kN$$

(i) For shop welding on two sides, partial safety factor $\gamma_{mw} = 1.25$

Strength of the weld per mm length

$$= \frac{l_w t_e f_u/\sqrt{3}}{\gamma_{mw}} = 1 \times 4.2 \times \frac{410/\sqrt{3}}{1.25} = 795.66\,N/mm$$

$$= 0.795\,kN/mm$$

Effective length of weld reqd $= \frac{109.09}{0.795} = 137.22mm$

$\simeq 140mm$

Length of weld on each side = 140/2 = 70mm

$\nless b$ = 60mm (transverse spacing)

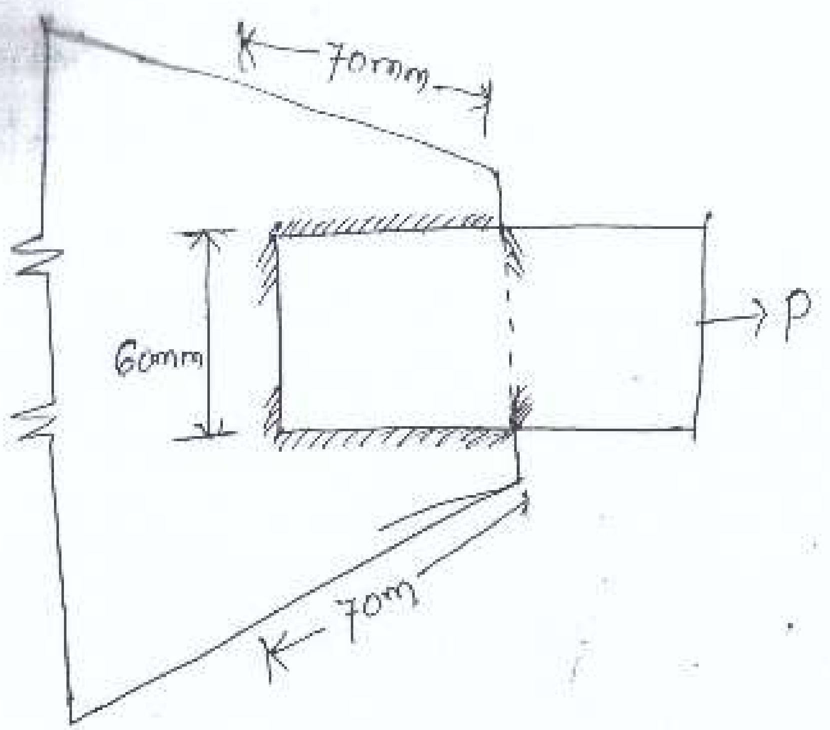
$\nless 4c$ = 4×6 = 24mm

minm lap = 4 × tmin = 4 × 8 = 32 mm or 40mm which is more = 40mm

Hence provide 6mm size & 70mm long shop fillet weld on both sides of the plate with end return of 2×6 = 12mm

∴ Length on each side = 70/2 × 12 = 94 mm

Total length = 94 × 2 = 188 mm



(II) For field welding on three sides

partial safety factor $\gamma_{mw} = 1.5$

Strength of the weld per mm length

$= l_w t_e \frac{f_u/\sqrt{3}}{\gamma_{mw}} = 1 \times 4.2 \times \frac{410/\sqrt{3}}{1.5} = 0.663 \text{ kN/mm}$

∴ Effective length of weld reqd $= \frac{109.09}{0.663} = 164.5 \text{ mm} \simeq 165 \text{ mm}$

$\not< 4s = 4 \times 6 = 24 \text{ mm}$ (ok)

Length of the end weld = 60 mm

Length of the weld reqd on both the sides

= 165 − 60 = 105 mm

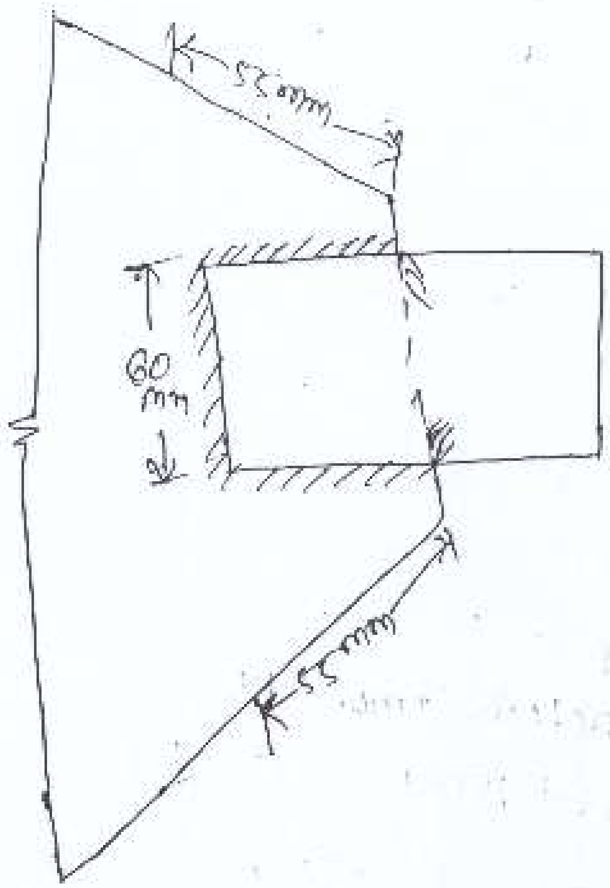
Length weld reqd on each side

= 105/2 = 52.5 mm ≃ 55 mm

$\not<$ minm lap $\not< 4 \times t_{min} = 4 \times 8 = 32$ mm & $\not<$ 40 mm

Hence let us provide a lap of 55mm & 6mm size shop welds on three sides with end return of $2\times6=12mm$

$\therefore$ total length $=60+2\times55+2\times12=194mm$



Tension Members

Tension members are linear members predominatly subjected to pulling which tend to stretch/elongate the member.

-> Tension members in a truss is known as tie.

Common shapes of tension members

design strength of a tension member

design strength of a tension member is the lowest of the following :-

(i) design strength due to yielding of gross secⁿ (T_{dg})

(ii) Rupture strength of the critical secⁿ (T_{dn})

(iii) Block shear strength (T_{db})

design strength due to yielding of gross secⁿ (Tdg)

$$\boxed{T_{dg} = \frac{A_g f_y}{\gamma_{m0}}}$$

where,

f_y = yield stress of the material

A_g = gross area of the c/s

γ_{m0} = partial safety factor for failure in tension by yielding = 1.1

design strength due to rupture of critical secⁿ (Tdn)

for plates

$$\boxed{T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}}$$

where,

γ_{m1} = partial safety factor = 1.25

A_n = Net effective area

$$= \left[b - n d_0 + \sum \frac{p_{si}^2}{4 g_i} \right] t$$

b = width of the plate

d_0 = dia of bolt hole

g = gauge length

(b) For threaded rods

$$\boxed{T_{dn} = 0.9 A_n f_u / \gamma_{m1}}$$

(c) Single angle

An angle connected through one leg is affected by shear lag and the effectiveness of outstanding leg reduces.

$$\boxed{\text{Rupture strength } (T_{dn}) = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}}$$

where,

A_{nc} = Net area of connected leg

A_{go} = Gross area of outstanding leg

$$\boxed{\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)}$$

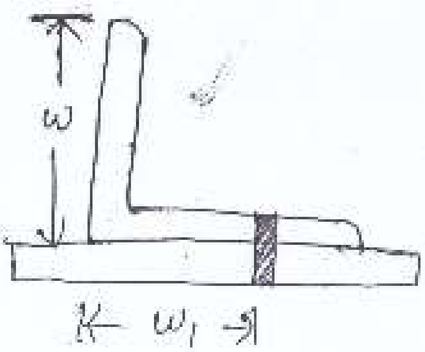
where,

w = outstanding leg width

b_s = shear lag width

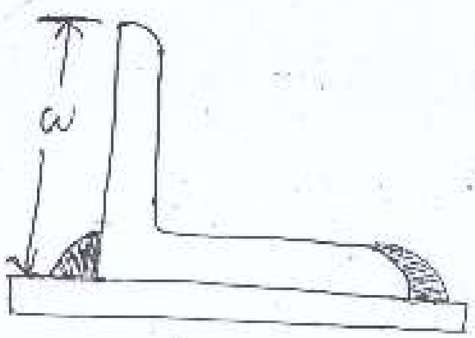
L_c = length of end connection

t = thickness of leg



$b_s = w + w_1 - t$

(Bolted connect^n)



$b_s = w$

(welded connect^n)

(b) For threaded rods

$$\boxed{T_{dn} = 0.9 A_n f_u / \gamma_{m1}}$$

(c) Single angle

An angle connected through one leg is affected by shear lag and the effectiveness of outstanding leg reduces.

$$\boxed{\text{Rupture strength } (T_{dn}) = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}}$$

where,

A_{nc} = Net area of connected leg

A_{go} = Gross area of outstanding leg

$$\boxed{\beta = 1.4 - 0.076 \left(\frac{w}{t}\right) \left(\frac{f_y}{f_u}\right) \left(\frac{b_s}{L_c}\right)}$$

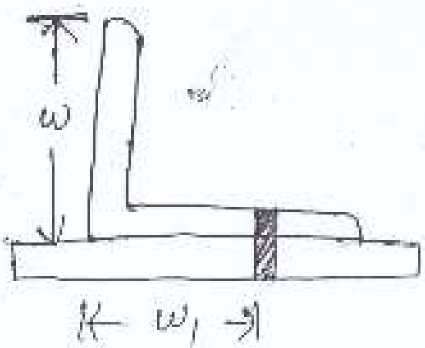
where,

w = outstanding leg width

b_s = shear lag width

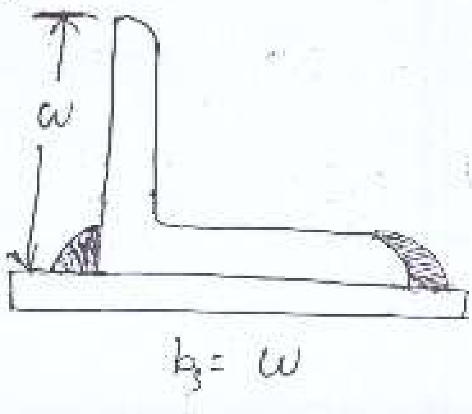
L_c = length of end connection

t = thickness of leg



$b_s = w + w_1 - t$

(Bolted connect^n)



$b_s = w$

(welded connect^n)

for preliminary sizing, the rupture strength of net section may be approximately taken as

$$\boxed{T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}}$$

where $\alpha = 0.6$ for one or two bolts

$= 0.7$ for three bolts

$= 0.8$ for four or more bolts

design strength due to block shear

(i) for shear yield & tension fracture

$$\boxed{T_{db_1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{m0}} + 0.9\frac{A_{tn} f_u}{\gamma_{m1}}}$$

(ii) for tension yield and shear fracture

$$\boxed{T_{db_2} = \frac{A_{tg} f_y}{\gamma_{m0}} + 0.9\frac{A_{tn} f_u}{\sqrt{3}\gamma_{m1}}}$$

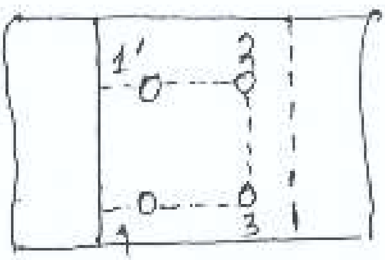
where,

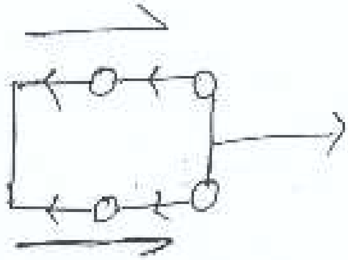
A_{vg} & A_{vn} = min^m^ gross & net area

A_{tg} & A_{tn} = min^m^ gross & net area

Block shear failure

At the connected end, failure of tension members may occure along a path involving shear along one plane & tension on perpendicular plane along the fasteners this type of failure is known as block shear failure.





Slenderness Ratio (λ)

The effective slenderness ratio of a tension member is the ratio of its effective length (KL) to its least radius of gyration.

design of tension members subjected to axial load

The following procedure may be adopted in the design of axially loaded tension members.

1. The gross area 'Ag' reqd to carry the factored load Tu from consideration of yielding is given by

$$\boxed{A_g = \frac{T_u}{f_y/\gamma_{m0}}}$$

$$\boxed{A_g = \frac{1.1\,T_u}{f_y}}$$

2. Select suitable shape of secn depending upon the type of structure & location of member such that gross area is (25 to 40)% more than calculated Ag.

3. determine the no. of bolts or the welding reqd & arranged.

4. find the strength considering
   (1) strength in yielding
   (2) " " Rupture
   (3) Block shear

5. Check of the strength is more than external factored tensile force

6. Check for slenderness ratio from table-3 IS 800:2007

Problem

A tension member is to carry a factored load of 250kN. design a suitable plate secⁿ for it assuming the connectⁿ to consist of lap joint with bearing type bolts of property class 4.6. Given grade of structural steel is Fe 410 & the effective length of the member is 0.75 m subject to possible reversal of stress due to earthquake

Given data

for steel grade Fe 410, $f_u = 410\ N/mm^2$, $f_y = 250\ N/mm^2$

$\gamma_{m0} = 1.1$, $\gamma_{m1} = 1.25$

Solⁿ

(1) Calculation of sectional area reqd

Net area reqd on the basis of rupture of critical secⁿ

$$A_n = \frac{T\gamma_{m1}}{0.9 f_u} = \frac{250\times10^3 \times 1.25}{0.9\times410} = 846.88\,mm^2$$

Assuming 25% excess, gross area $A_g = 1.25 \times 846.88$

$= 1058.60\,mm^2$

Gross area reqd on the basis of gross secⁿ yielding

$$A_g = \frac{T\gamma_{m0}}{f_y} = \frac{250\times10^3\times1.1}{250} = 1100\,mm^2$$

Hence let us provide a plate of 140mm × 8 mm, giving

(ii) From Unwin's formula, dia of bolt $d = 6\sqrt{t}$

$$d = 6\times\sqrt{8} = 16.97\,mm$$

Hence let us provide single row bolts of 16mm dia, $d = 16mm$

$d_0 = 18mm$, $\gamma_{mb} = 1.25$, $A_{nb} = 157\,mm^2$, $A_{sb} = 201\,mm^2$

$f_{ub} = 400\,N/mm^2$

Assuming the shear plane, $n_n = 1$, $n_s = 0$, assuming $\beta_{lj}, \beta_{lg}, \beta_{pk} = 1$

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}\left(n_n A_{nb} + \overset{0}{\cancel{n_s A_{sb}}}\right)$$

$$= \frac{400}{\sqrt{3}\times 1.25}\left(1\times 157\right) = 29.01\,kN$$

Strength of a bolt in bearing

Assuming $e = 30mm$, $p = 40mm$

$$k_b = \text{least of} \begin{cases} \dfrac{e}{3d_0} = \dfrac{30}{3\times 18} = 0.555 \\[2ex] \dfrac{p}{3d_0} - 0.25 = 0.491 \\[2ex] \dfrac{f_{ub}}{f_u} = \dfrac{400}{410} = 0.976 \\[2ex] 1.0 \end{cases}$$

so $k_b = 0.491$

$$V_{dpb} = \frac{2.5 k_b d t f_u}{\gamma_{mb}} = \frac{2.5\times 0.491\times 16\times 8\times 410}{1.25}$$

∴ Bolt value = 29.01 kN

∴ No of bolts reqd = $\frac{250}{29.01}$ = 8.62 ≃ 10

Hence let us provide 16mm dia. bolts in 2 rows

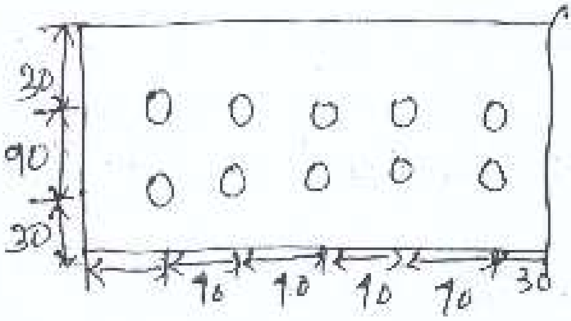
Check for long joints

l_j = distn b/n the first & last rows of bolts in the joint.

= 4×40 = 160mm < 15d = 15×16 = 240mm (ok)

(i) Net section in rupture

Here P_{si} = 0, n = 2



$$A_n = \left[b - n d_0 + \sum \frac{P_{si}^2}{4g_i}\right] t$$

$= [140 - 2×18+0] × 8 = 832\ mm^2 < 816.88\ mm^2$

Hence let us revise the secn, to 150mm×8mm, giving

A_g = 1200 mm²

A_n = (150 − 2×18+0)×8 = 912 mm² > 816.88 mm² (ok)

$$T_{dn} = \frac{0.9 f_u A_n}{\gamma_{ml}} = \frac{0.9×410×912}{1.25} = 269.222\ kN$$

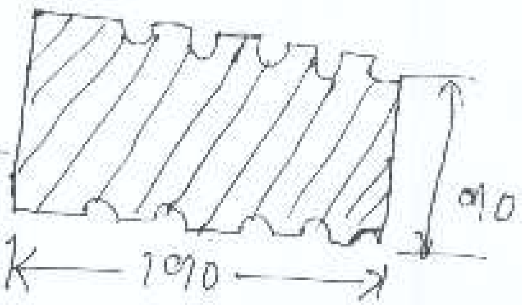
(ii) <u>Gross secn yielding</u>

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}} = \frac{1200×250}{1.1} = 272.727\ kN >$$

250 kN

(iii) Strength of the plate against block shear

The shaded portion of the plate may shear off



Gross area in shear,

$A_{vg} = 2(2 \times 40 + 30) \times 8 = 3040\,mm^2$

Net area in shear, A_{vn}

$A_{vn} = 2\left[(4 \times 40 + 30) - \left(4 + \frac{1}{2}\right) \times 18\right] \times 8 = 1744\,mm^2$

Gross area in tension, $A_{tg} = 90 \times 8 = 720\,mm^2$

Net area in tension, $A_{tn} = \left(90 - 2 \times \frac{1}{2} \times 18\right) \times 8 = 576\,mm^2$

For tension yield & shear failure (T_{db1})

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + 0.9\frac{A_{tn} f_u}{\gamma_{m1}}$$

$$= \frac{3040 \times 250}{1.1 \times \sqrt{3}} + \frac{0.9 \times 576 \times 410}{1.25}$$

$$= 460.872\ kN$$

For tension yield & shear fracture

$$T_{db2} = \frac{A_{tg} f_y}{\gamma_{m0}} + \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}}$$

$$= \frac{720 \times 250}{1.1} + \frac{0.9 \times 1744 \times 410}{\sqrt{3} \times 1.25} = 460.672\ kN$$

Strength against block shear $= 460.872\ kN > 210\ kN$

(4) Check for slenderness ratio

min^m radius of gyration, $r = \sqrt{\frac{I}{A}}$

$$= \sqrt{\frac{bt^3/12}{bt}} = \frac{8}{\sqrt{12}} = 2.309\,mm$$

Effective length $KL = 0.75\,m$ (given) $= 750\,mm$

max^m slenderness ratio $\lambda = \frac{KL}{r} =$

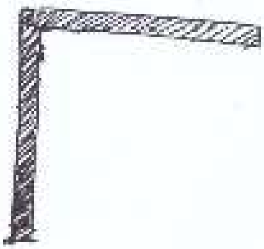
$$= \frac{750}{2.309} = 324.8 < 350\,mm$$

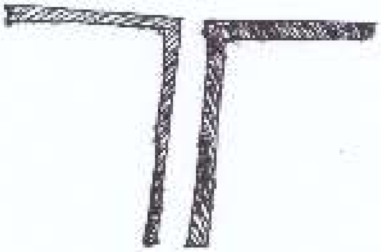
$\therefore$ Hence design ok

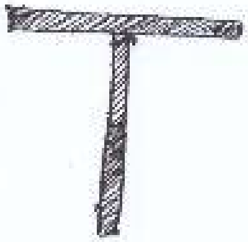
Compression Members

Many structural members are in compression. Vertical compression members in buildings are called column & compression members in trusses are called struts.

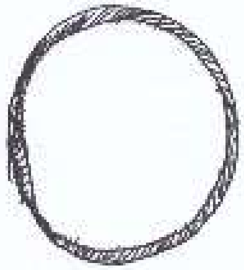
Common shapes

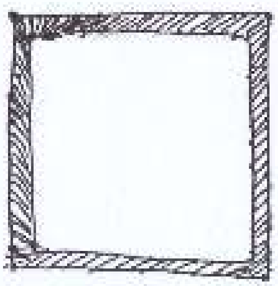

(a) single angle

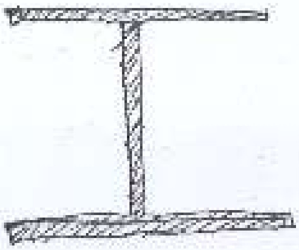

(b) double angle

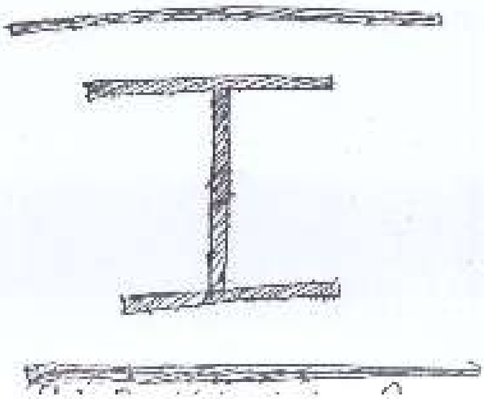

(c) Tee


(d) channel


(e) circular hollow secⁿ


(f) Rectangular Hollow secⁿ


(g) I-secⁿ



Buckling class of cross secⁿ

It is a common practice to transfer load axially through any member. But due to some imperfection, unexpected eccentricity may be imposed.

→ Buckling is defined as the sudden bending, warping or crumpling of the compression members under compression.

→ Due to buckling, deformation developed in a column occures in a direction or plane normal to the direction of the loading.

Slenderness Ratio (λ)

It is defined as the ratio of effective length to the corresponding radius of gyration of the secⁿ.

$$\boxed{\lambda = \frac{KL}{r}}$$

design compressive stress & strength

The design compressive strength of a member is given by

$$\boxed{f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + [\phi^2 - \lambda^2]^{0.5}}}$$

where,

$$\phi = 0.5\,(1 + \alpha(\lambda - 0.2) + \lambda^2)$$

$$\lambda = \sqrt{f_y/f_{cc}} = \sqrt{\left(\frac{KL}{r}\right)^2 f_y/\pi^2 E}$$

f_{cd} = design compressive stress

α = imperfection factor

Imperfection Factor

Buckling class	a	b	c	d
α	0.21	0.34	0.49	0.76

design compressive strength P_d =

$$\boxed{P_d = A_e\, f_{cd}}$$

Problem

Calculate factored axial load on the column sec^n. ISHB 400 @ 806.38 N/m. The height of the column is 3.0 m and it is pin-ended. Use steel of fe410 grade.

(1) for steel grade Fe410

$f_y = 250\ N/mm^2$, $\gamma_{m0} = 1.1$, $E = 2\times10^5\ N/mm^2$

(2) For ISHB 400 @ 806.38 N/m (From code book)

$h = 400mm$, $b_f = 250mm$, $t_f = 12.7mm$, $t_w = 10.6mm$

$a = 10466mm^2$, $r_{zz} = 166.1mm$, $r_{yy} = 51.6mm$ (fn)

(3) Buckling class

$$\frac{h}{b_f} = \frac{400}{250} = 1.6 > 1.2,\ t_f = 12.7mm \leq 40mm$$

∴ Buckling class about z-z axis = a, about y-y axis = b

(4) Effective sectional Area

$A_e = a = 10466mm^2$

(5) Effective length of columns

for column pinned at both ends, $KL = 1.0L$

(6) Check for limiting thickness by comparing with semi-compact secn parameters.

Here, root radius $R_1 = 14mm$

$$b = \frac{b_f}{2} = \frac{250}{2} = 125mm$$

$$d = h - 2(t_f + R_1)$$

$$d = 400 - 2(12.7 + 11) = 346.6mm$$

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

for rolled secn, outstand of compression flange, (11)

$\frac{b}{t_f} = \frac{125}{12.7} = 9.84 < 10.5\varepsilon = 10.5 \times 1 = 10.5$ (class 2, compact)

$< 15.7\varepsilon = 15.7 \times 1 = 15.7$ (class 3, semicompact)

$< 34\varepsilon = 84 \times 1 = 84$ (class 1, plastic)

Hence the sec^n is not slender & full sec^n is available for design strength.

(7) <u>design compressive strength</u>

(i) About y-y axis

$$\lambda_y = \frac{KL}{r_{yy}} = \frac{1.0 \times 3000}{51.6} = 58.14 < 180$$

for $f_y = 250\ N/mm^2$ & buckling class 'b'

By interpolation

KL/r	fcd
50	194
58.14	?
60	181

$$f_{cd} = 194 - \left[\frac{(194-181)}{(60-50)} \times (58.14-50)\right] = 183.41\ N/mm^2$$

∴ factored axial load $P_d = A_e \times f_{cd}$

$= 10466 \times 183.49$

$= 1920.65$ KN

ii) About z-z axis

$$\lambda_z = \frac{KL}{r_{zz}} = \frac{1.0 \times 3000}{166.1} = 18.06 < 180$$

for f_y = 250 N/mm² & buckling class a

By interpolation,

KL/r	f_{cd}
10	227
18.01	?
20	226

$$f_{cd} = 227 - \left[\frac{(227-226)}{(20-10)} \times (18.01-10)\right] = 226.2\ N/mm^2$$

∴ factored axial load $P_d = A_e \times f_{cd}$
$= 10466 \times 226.2$
$= 2367.41$ kN

∴ design factored axial load = minm of the two
$= 1919.65$ kN (Ans)

design of axially loaded compression member

The following procedure may be adopted in the design of compression member.

1. Assume slenderness ratio and determine design compressive stress considering grade of steel and assuming buckling class
2. Calculate effective sectional area reqd $A_e = P_d/f_{cd}$ choose a trial secn from steel table
3. find effective length & maxm slenderness ratio i.e $\lambda_{max} = l/r_{min}$
4. determine permissible compressive stress f_{cd}
5. Redesign if P_d differs considerably from the design load.

6. The section may be checked for limiting thickness also.

Problem

Design a column section (using channel sec^n only) to carry a factored axial load of 400 kN. The column is 4m long & is effectively held in position at both ends but restrained against rotation at one end only. Consider $f_y = 250$ N/mm^2. Assume earthquake actions.

Sol^n

(1) Assuming permissible design compressive stress 80 N/mm^2

$$A_{req} = \frac{400 \times 10^3}{80} = 5000 \text{mm}^2$$

(2) Try 2 ISMC 350 @ 413 N/m, having

$A = 5366$ mm^2 (from steel table)

$r_{zz} = 136.6$ mm, $r_{yy} = 28.3$ mm

$r_{min} = r_{yy} = 28.3$ mm

(3) For one end fixed & other end pinned

$$KL = 0.8L = 0.8 \times 4000 = 3200 \text{mm}$$

$$\lambda_{max} = \frac{KL}{r_{min}} = \frac{3200}{28.3} = 113.07 < 250$$

(4) The buckling class is 'c' for channel sec^n

for $\frac{KL}{r} = 113.07$ & $f_y = 250$ N/mm^2

Peremissible comp. stress fcd =

$f_{cd} = 91.25\ N/mm^2$ (By interpolation)

(5) design strength $P_d = A_e f_{cd}$

$P_d = 5366 \times 91.25 = 489.65\ kN$

$489.65\ kN > 400\ kN$ (Hence safe)

(6) Check for limiting thickness

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1$$

from steel table, $b_f = 100\,mm$, $h = 350\,mm$, $t_f = 13.5\,mm$
$t_w = 8.1\,mm$, $R_1 = 14\,mm$

Here $b = 100\,mm$, $d = h - 2(t_f + R_1)$
$= 350 - 2(13.57 + 14) = 295$

for channel secn, $\frac{b}{t_f} = \frac{100}{8.1} = 12.35 < 15.7\varepsilon = 15.7$

$\frac{d}{t_w} = \frac{295}{8.1} = 36.42 < 42\varepsilon = 42$

(Ok)

Design of Steel Beams

Beams are those structural members, whose length is considerably larger than the cross-sectional dimension.

Common cross sections

For beams, angles, I-sections, channels etc are commonly used. For heavier loads I-sections with additional plates connected on flanges are used.

Classification of cross-section

during plastic analysis, it has been found that when all fibres of a beam cross-section reach yield point, then plastic hinge is formed which doesn't allow the beam to take any extra load & beam fails due to rotation w.r.t the plastic hinge.

→ But during this mechanism the beam should be capable of sufficient rotation capacity without local buckling.

→ Buckling in any small part of a member is called local buckling & buckling of whole beam is called global buckling.

→ If local buckling occurs before reaching the formation of plastic hinge then beam fails without developing full plastic moment or full rotation about plastic hinge.

→ Hence it is necessary to see that plate elements of a cross-section do not buckle locally due to compressive stresses before plastic hinges are formed.

→ Local buckling can be achieved by providing proper width to thickness ratio. Based upon this criteria beam cross-sections are divided into following 4 categories.

(1) Class-1 (Plastic cross-secⁿ)

Those are the secⁿ that can develop plastic hinges and also have full rotation capacity for failure of the structure by plastic mechanism.

(2) Class-2 (Compact) cross-secⁿ

Such secⁿ can develop plastic moment & rotation capacity in inadequate amount due to local buckling.

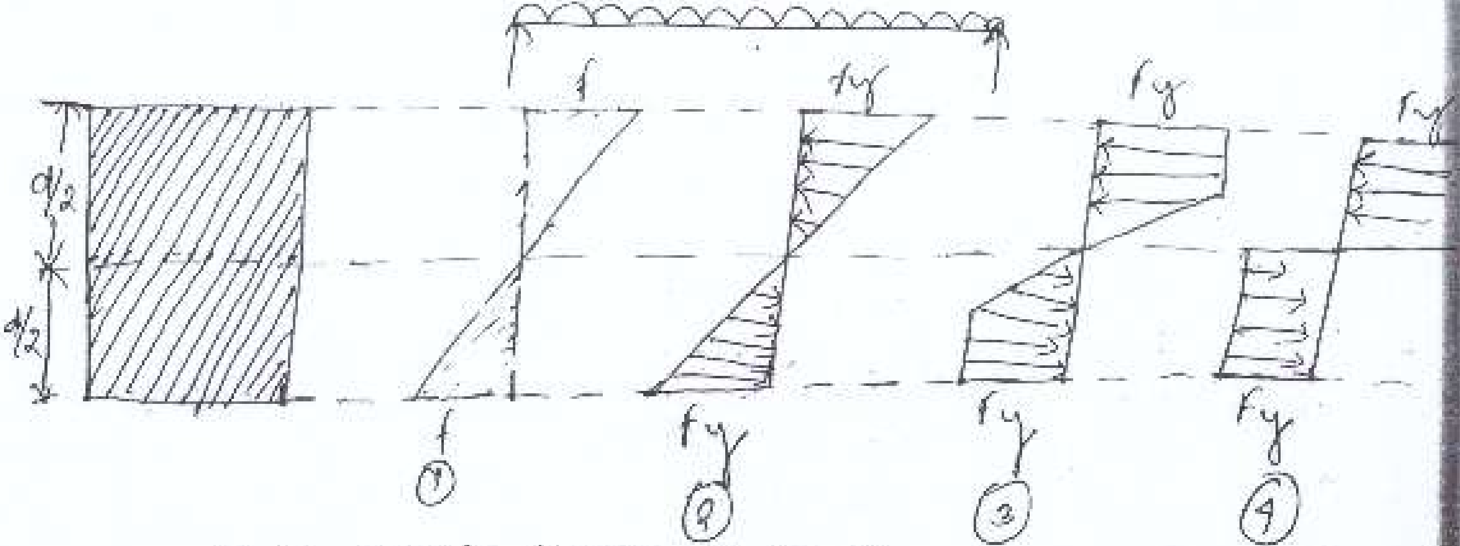
(3) Class-3 (semi-compact) cross-secⁿ

These are the secⁿ in which extreme fibres in compression can reach yield stress, but can't develop the plastic moment of resistance, due to local-buckling.

(4) Class-4 (Slender) cross-secⁿ

These cross-secⁿ in which the elements buckle locally even before reaching yield stress belong to this category.

consider the c/s of simply supported with UDL imposed on it.



within elastic limit in fig(1), where stress varies linearly from compression to tension.

Design of tubular structure

Introduction

The steel tubular or tubular steel sectional are commonly being used as structural components and large number of such structure like truss member, airplane hanger, cross branchings and beams. They are also used for scaffolding of buildings. The steel tubular sections are effectively used in large space frames, lattice structures of arenas, stadium and exhibition halls. The masts and transmission towers are the example where tubular sections are utilized efficiently.

Classification

→ Depending upon the manufacturing process, the steel tubular categorised as,

(a) Hot finished seamless (HFS)
(b) Cold finished seamless (CFS)
(c) Hot finished welded (HFW)
(d) Electric resistance welded (ERW) or high frequency induction welded (HFI

→ The standard sizes, their masses / weight and relevant geometrical properties are given table-1 of IS 1161 : 1998.

Designation of steel tubes:

→ Steel tubes are designated by their nominal bore and shall be classified as light, medium and heavy depending upon the wall thickness.

→ They shall be graded as Yst 22, Yst 25, Yst 32 depending on their yield stress of the material.

<u>Permissible stresses</u> -

- The magnitude of permissible stress under various loading condition as per IS 806 : 2002 which are follows -

<u>Axial stress in tension</u> - (Table - 1 of IS 806 - 1968) may be refferred

<u>Axial stress in compression</u> -

(Table - 2 of IS - 806 - 1968 is to be followed)

<u>Bending stress</u> -

- Table - 3 of IS - 806 - 1968 may be followed

<u>Shear stress</u> -

Table - 4 of IS 806 - 1968 may be followed

<u>Bearing stress</u>

Table - 5 of IS 806 - 1968 may be followed.

<u>Connection</u> -

14 → connections in structures using steel tubes are provided by weldings, riveting or bolting.

→ connections between the tubes are made directly tube to tube without gusset plates or other attachments.

→ Ends of the tubes may be flattened or otherwise formed to provided for welded riveted or bolted connections.

→ Generally welding is adopted for connections in tubular steel construction, which is rigid and greater overall economical.

→ Actual condition of rigidity should be taken in to consideration while designing these type of joints.

→ The weld connecting two tubes ends to should be full penetration butt weld.

→ The weld connecting the end of one tube (branch tube) surface of another tube (main tube) with their axis at angle of not less than 80° shall be of any one of the following.

(a) Butt weld throughout
(b) Fillet weld throughout
(c) Fillet butt weld, the weld being a fillet weld in one part and a butt weld in another with a continuous change from one from to the other.

<u>Joints</u> -

In case of of joints in compression members the ends of the members are faced for complete bearing over their whole area. The welding are joining materials are kept sufficient to the tolen the members accurately in place to resist all forces other than direct compression including those arising during transit unloading and erection.

Permissible stress in welds -

For buttweld, tensile stress = 125 N/mm² (for Ys t 22)
= 150 N/mm² (for Ys t 25 or Ys t 32)

compression stress = σ_{ac} (Fc) (up to Ys t 25)

Shear stress = 90 N/mm² for Ys t 22
= 110 N/mm² for Ys t 25 or Ys t 32

For fillet welds, shear stress = 90 N/mm² for Ys t 22
= 110 N/mm² for Ys t 25 or Ys t 32

Tubular columns -

→ Round tubular sections provide the most efficient crosssectional shape for the columns and compression members having lateral restrain in all directions normal to the axis of the member.

→ The diameter of such member should be as large as possible with the additional requirement that the mean diameter to thickness ratio (dm/t) should also be small enough to ensure that the stress failure by local buckling does not take place.

→ In design of tubular columns, two factors namely "crincling" and heat treatment.

Effective length of compression members

Table - 7 of IS - 806 - 1968 may be followed.

Maximum

Crinkling of tubes -

→ When a steel tube is subjected to excessive compression then the tube will have a change to crinkling. Crinkling means caving in and formation of after the inner of the concentina of walls of tubes under compressive stress such folds may be circular oval or polygonal and they may occur after or before the constitutional stress reaches yield point.

→ This stress is a function of the mechanical properties of the material and of the geometrical shape of the cross section.

Mathematically,

The stress causing collapse $= P = E \cdot \frac{t}{R} \left(\frac{m^2}{\frac{3}{m^2} - 1} \right)^{1/2}$

where, t = thickness of the tube

R = mean radius of the tube

$\frac{1}{m}$ = poisson's ratio of the tube material

E = Young's modulus

Tubular

The tubular tension members donot have any advantage as tension member and rather they have higher cost of production than other rolled steel sections.

Design of tubular b

The tensile and compressive strength in the extreme fibres of tubes in bendings should not exceed the permissible values as given Table-3, table-4 of the code.

Q.1 A tubular steel column of 4.8m length in hinged at both ends. It has nominal diameter of 225 mm and conforms to Yst 25 grade. Determine the safe load carrying capacity of the column.

Solution - Given data -

$l = 4.8m = 4800mm$, $d = 225$ mm

End condition, so, $L = le = 4800$ mm

Radius of gyration of the corresponding to nominal diameter of the 225 mm (heavy), $r = 8.44 cm$
$= 84.4$ mm

so, Slend ratio - $\lambda = \frac{le}{r} = \frac{4800}{84.4} = 56.87 < 180$

Maximum slen ratio $= \lambda = 180$

Again, for Yst 25 and $l/r = 56.87$,

$f_c = 114.96 \ N/mm^2$ (using interpolation from Table - 2)

so, the safe load carrying capacity of the members

$\Rightarrow F = Afc$

$\Rightarrow F = 4420 \times 114.96$
$= 508.14$

Area of the tube $= 4420 \ mm^2$ from Table - 1 of IS 1161 - 1998

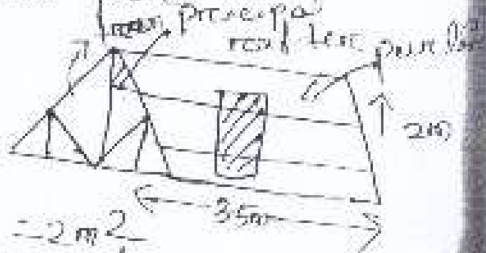
Tubular

Various members of the roof truss are subjected to axial compressive and tensile forces only. The elements of the truss are generally joined by welding.

(2) Design a tubular steel purlin for the following data spacing of roof truss = 3.5 m spacing of purlin along the slope of the roof = 2 m Vertical load from roof sheeting etc = 150 N/m² Live load on the roof = 0.75 kN/m². The purlin is effectively continuous over the rafters. Assume all loads acting normal to the roof and use Yst 22 grade.

Soln

Vertical load on the purlin per meter length



Area of the roof load coming to the purlin per meter run = 2×1 = 2 m²

Vertical load from roof sheeting/mtr = 150×2 = 300 N/m

Assuming self wt. of tubular purlin = 50 N/m

Live load on the purlin 750×2 = 1500 N/m

Total load = w = 300+50+150 = 1850 N/m

Total load on the purlin = wL = 1850×3.5 = 5550 N

Maximum bending moment in the purlin $= \frac{WL}{12}$

$= \frac{5550 \times 3.5}{12}$

$= 1618.75$ Nmm

Allowable bending stress in the purlin = Fb = 140 N/mm²

Required section modulus $= Z = \frac{M}{Fb} = \frac{1618.75 \times 100}{140}$

$= 11562.5 \text{ mm}^3$

$= 11.56 \text{ cm}^3$

Let us provide a 65 mm nominal dia light steel tube 5.71 kg/m and section modulus = 12.82 cm³ check for deflection -

Minimum outside dia $= D = \frac{L}{70}$

$= \frac{3500}{70}$

$= 50 \text{ mm} < 65 \text{ mm}$ OK

Minimum section modulus $: Z = \frac{WL}{16800} = \frac{\frac{5550}{9.81} \times 40}{16800}$

$= 13.49 > 12.82 \text{ cm}^3$

Hence, adopt a 65mm nominal dia medium stress tube having section modulus 14.20 cm^3 @ 6.42 Kg/m and $A = 8.20 \text{ cm}^2 = 820 \text{ mm}^2$

check for stress developed.

self wt of purlin = 6.42 Kg/m

check for bending stress -

Total UDL on the purlin $= 300 + 63 + 1500$

$= 1863 \text{ N}$

Total load/m $= 1863 \times 3.5 = 6520.5 \text{ N/m}$

Maximum bending moment in the purlin

$m = \frac{wL}{12}$

$= \frac{6520.5 \times 3.5}{12}$

$= 1901.81 \text{ N/m}$

Maximum bending stress in the purlin

$= f = \frac{m}{z}$

$= \frac{1901.81}{14200} = 133.93 < 140 \text{ N/m}^2$ OK check for shear stress force $F = \frac{W}{2} = \frac{6520.5}{2}$

$= 3260.25\ N$

Maximum shear stress $= \frac{F}{A/2}$

$= \frac{3260.25}{820/2}$

$= 7.91\ N/mm^2$

Introduction -

A masonry structure is an assemblance of masonry units or blocks properly added together with mortar. The masonry units are solid or perforated burnt clay bricks, sand-lime bricks, stones, concrete blocks, lime based blocks or burnt clay hollow blocks. The basic advantages of masonry construction lies in the fact that in load bearing structures it performs a variety of function such as supporting loads, subdividing space, provide thermal and acoustic insulation, affording fire and weather protection etc. It is suited for buildings where the floor area is subdivided in to a large number of rooms of smaller medium size and the floor plan is repeated in each story throughout the height of the buildings, ho , nursing home, hospitals, schools and certain type of administrative buildings.

Masonry units. -

Masonry units used in construction are properly bonded together with some cementing material say matter. Many masonry units are used in construction, but bricks and concrete blocks are largely used for structural units choice of generally made from the consideration of local availability, compressive strength

, durability, cost and of construction.

The relationships between compressive strength of bricks and maximum number of storage in case of simple residential building having one brick thick walls and rooms of the medium size in given below.

Comp. strength (N/mm^2)	No. of storage
3 - 3.5	1 to 2
7	2 to 3
10	3 to 4
15	4 to 5

Mortar -

Mortars are in mixtures of some cementing materials such as cement lime and fine aggregate (such as sand, burnt clay cinder etc). Mortars are broadly classified into three types such as.

(i) Cement mortar

(ii) Lime mortar

(iii)

Cement Mortar -

These consists of cement and sand, varying in proportion from 1:8 to 1:3 strength and workability in preming with the in are in the proportion of cement. Rich mortars ~~thde~~ through having good strength have high shi and are thus more ~~tob~~ to cracking.

Lime Mortar - These consists of intimate mixtures of lime as binder and sand, burnt clay / Su linder as fine aggregate in proportion 1:2 to 1:3 lime mortar gain strength slowly and have low ultimate strength. Mortar having hydralic lime altain some what be strength than flat lime. Lime mortar in good workable, having good water red and low

Cement - lime mortar -

These mortars combine good qualiting of constant as well as lime mortar, that is medium strength along with good workability, good water retentivity, freedom from cracks and good resistance against crack and good resistance against rain penetration commonly used proportion are (cement : lime : sand) 1:1:6, 1:2:9 and 1:3:12. It is much better than cement mortar for masonry work in most of the structures.

Grades of Mortar (refer table-1 of IS-1905)

Grade of mortars	Mix proportion by lose volume			Minimum compressive strength (N/mm2) at 28-days
	Cement	Lime	Sand	
H1	1	1/4 C or B	3	10.00
H2	1	1/4 C or B	4	7.50
	1	1/2 C or B	4 1/2	6.00
M1	1	—	5	5.00
M2	1	—	6	3.00
M3	1	—	7	1.50
	1	—	8	0.7
L1	1	1B	3	0.5
L2	—			

where; A = Hydra lime
B = Semi - hydraulic lime
C = Fat lime

Design of Masonry walls -

From the structural design consideration walls can be classified into types such as
(a) Load Bearing walls
(b) Non - load being walls

Load - bearing walls -

A wall deb carry an imposed vertical load in addition to its own weight together with any lateral load.

Non - load bearing walls -

A wall dunot resist or support any load such that it can be removed with the approval of a structural engineer without hampering the intergrity of the remaining structure.

Design considerations for load

(i) Masonry buildings are mainly constracted of load bearing walls where walls are used to transfer gravity as well as lateral loads to the foundation in additional to its of sub-dividing space providing thermal and accoustic inculation providing fire resistance and providing weather protection.

(ii) while transforming design loads the masonry is subjected to mainly compressive, tensile and shear strength which should be well within permissible limits and the wall should not buckle or overturn.

(iii) Load being walls are structurally more efficient when the load in uniformly distributed and the structure in so planed that the eccentricity of loading on the members is as small as possible.

(iv) Advance of eccentric loading by providing adequate bearing of roof slab on the walls providing adequate stiffens of especially important in load bearings walls in masonry structure.

(v) In order to ensure uniformly loading openings in walls should not be too large and these should be of 'hole in the wall' type as far as possible.

(vi) Beings for lintels and bed blocks under beames should be liberal in since heavy uncertain of loads should be varied where fesible with the loadings so as to obtain more or less uniform stress in adjoining points of the members.

Design loads–

The loads to be taken in consideration fore design of masonry walls are (i) Gravity loads– Vertical loads such as dead load (DL), (LL) Live load of the super structure.

(ii) Lateral loads – Horizontal loads like – accidental load (AL), wind load (WL) and earthquake loads (EL).

Permissible structures.

(clause - 5.4 of IS 1905-1987) to be followed

Permissible compressive stress - The permissible stress (Fc) shall be based on the value of the basic compressive stress (FC) taking in to account the influence of slenderness ratio of the wall, eccentricity of loading, area of cross section of the wall, shape of the masonry units and the type of loading (uniform and concentrated) (Clause - 5.4.1 of IS-1905-1987) ←

Design consideration for non-load bearing walls -

→ A non-load bearing walls is often designed to resist only to lateral loads. It may be provided an exterior wall to protect against weather and as an interior wall for the purpose of partitioning. Hence a non-load bearing wall may be called a panel wall curtain wall/partition wall.

→ Panel wall are non-load bearing exterior walls in framed construction wholly supported on each storey and subject to lateral loads only.

→ curtain walls are supported by horizontal and vertical structural member where necessary and subjected to lateral loads only.

Effective height of masonry walls (Table - 4 of IS - 1905 : 1987)

Effective length of masonry walls (Table - 5 of IS - 1905 - 1987)

Effective thickness -

Effective thickness (t) of a solid wall shall be its actual thickness, including the thickness of joint between masonry units.

Slenderness Ratio (SR) - $\frac{\text{Effective height}}{\text{Effective thickness}}$ or $\frac{\text{Effective length}}{\text{Effective thickness}}$,

For walls, whichever is smaller

Max^m SR - (Refer table - 7 of IS - 1905 - 1987)

The angle of dispersion of vertical load on walls shall be taken as not more than 30° from the vertical

Free standing wall (Table - 11 of IS - 1905 - 1987)

P

Q① A ground floor masonry wall 4m clear ht up to bottom of the roof slab. Ht of plinth above foundation footing = 0.8 m. If the walls thickness 30 cm calculate effective ht and slenderness ratio for partial restrain and both ends cond^n.

Ht of wall measured from top of the footing = 4 + 0.8 = 4.8 m
(from note - 2, clause - 4.3.1)

From ~~table~~ - 4 of IS - 1905 - 1987,

effective ht of wall = 1.0 H = 1×4.8 = 4.8 m

slenderness ratio (SR) $= \frac{h}{t} = \frac{4.80}{0.3} = 1.6$

(2) A masonry wall is 4.0 m ht and 6.0 m length calculate effective length of the wall for the support conditions wall is supported by a cross wall at one end continuous with cross wall at the other hand.

Soln :-

For the case as given in question,

length = 6 m, ht = 4 m

Effective length = 0.9 l (Sl no - 2 of Table - 5 of IS - 1905 - 1987)

= 0.9×6

= 5.4 m