LECTURE NOTES
ON

DESIGN OF STEEL STRUCTURE

FOR DIPLOMA IN CIVIL ENGINEERING

4^{TH} SEMESTER AS PER SCTE&VT SYLLABUS

Prepared by–D. R. Mohapatra

HOD, Department of Civil Engineering

GIET. Polytechnic, Bhubaneswar

Chapter-1

Common steel structures:-

Steel has high strength per unit mass. Hence it is used in constructing large column - free structures.

The following are the common steel structures:-

(i) Roof trusses for factories, cinema halls etc.
(ii) Trussed bents, crane girders, column etc. industrial structures.
(iii) Single layer or double layer domes for auditoriums, exhibitions halls.
(iv) Water tank
(v) Plate girder and Truss - bridges

Advantages and disadvantages of steel structure:-

The advantages of steel over other materials for construction are:

(i) It has high strength per unit mass
(ii) It has assured quality & high durability.
(iii) Speed of construction is another important advantages of steel structure.
(iv) material is reusable.
(v) It joints are taken care it is the best water and gas resistant structure.

Disadvantages:-

(i) It is susceptible to corrosion.
(ii) Maintance cost is high since it needs Point
(iii) steel members are costly.

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Rolled steel section:-

Like concrete steel section of any shape and size cannot be cast on site. steel needs very high temprerature to melt it and rolls into required shape.

→ Steel section of standard section, shapes, size and length are rolled in steel mains and market.

→ Many steel sections are readily available in the market and are in frequent demand such steel sections are known as regular steel sections.

→ Some steel sections are not in use commonly but the steel mills can roll them it order are placed. such steel sections are known as special sections.

various type of rolled steel sections manfracture are listed below:-

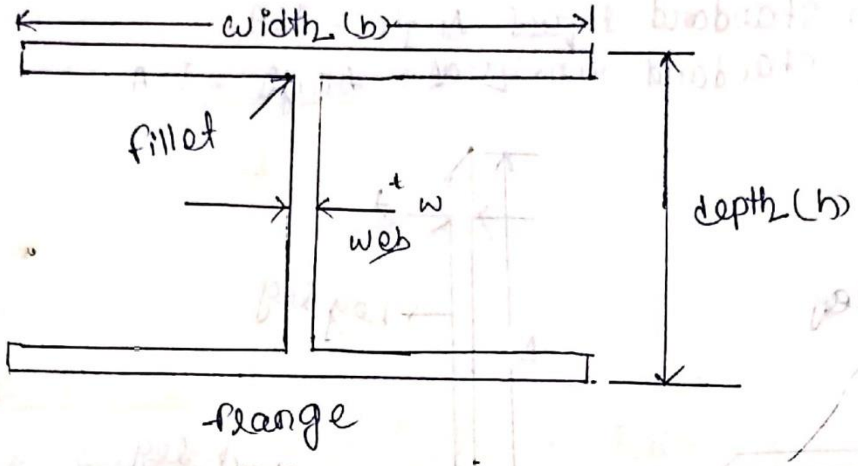
(i) Rolled steel I- section (Beam section).
(ii) Rolled steel cannal section
(iii) rolled steel angle section
(iv) Rolled steel Tee section
(v) Rolled steel bars
(vi) Rolled steel tubes
(vii) Rolled steel plates
(viii) Rolled steel flats
(ix) Rolled steel sheets and strips

Rolled steel I- sections:-

The following five series of rolled steel I-section are manifactured in India.

(a) Indian standard Junior beam - ISJB
(b) Indian standard light beam - ISLB

(c) Indian standard medium beam - ISMB
(d) Indian standard wide - flange Beam - ISWB
(e) Indian standard Heavy beam - ISHB



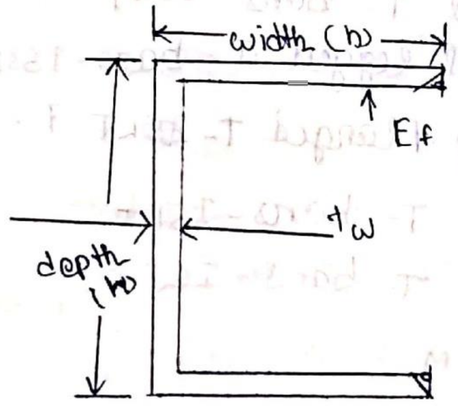
(Rolled steel I section)

Ex:- ISWB 600 @ 1.423 KN/M

Rolled steel channel sections:-

These sections are classified in to the following 4 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



(Rolled steel channel section)

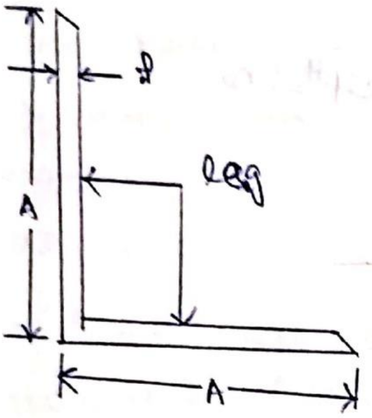
Ex :- ISMC 300 @ 0.351 KN/M

Rolled steel angle sections :-

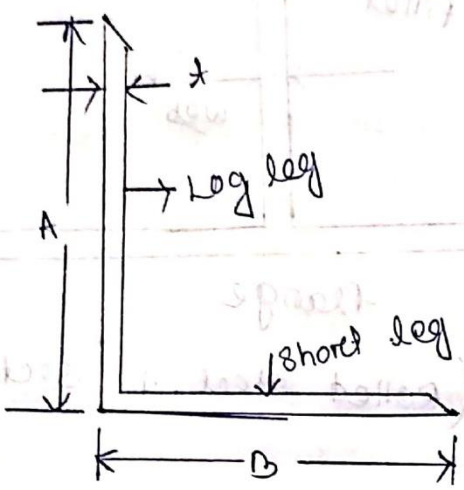
There are classified in to the following two section.

(a) Indian standard Equal Angle - ISA

(b) Indian standard unequal Angle - ISA



(a) Rolled steel equal angle



(b) Rolled steel unequal angle

Ex :- ISA 150, 12 MM thick or ISA 150×150×12 · ISA 150, 115, 10 MM or ISA · 150×150×10

Rolled steel T- Sections:-

following 5 series of rolled steel sections are available

(a) Indian standard normal T- bars- ISNT

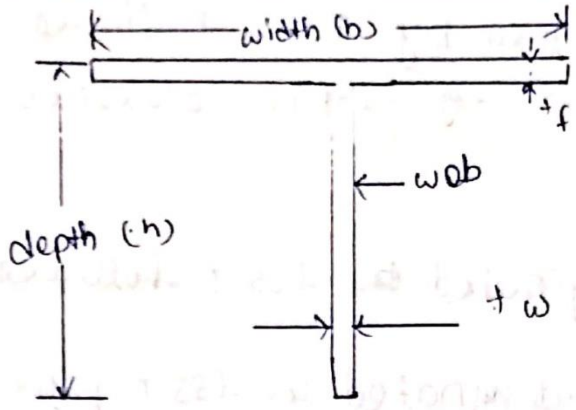
(b) Indian standard special legged T- bars - ISSLT-

(c) Indian standard heavy flanged T- ~~ISDF~~ T - bars-ISHT

(d) Indian standard light T- bars - ISLT

(e) Indian standard Junior T- bars - ISJT

EX:- ISNT 60 @ 53 N/M



(Rolled Steel T- Section)

Rolled Steel Bars:-

Rolled steel Bars classified into the following 2 series.

(a) Indian standard Round bars - ISRO

(b) Indian standard square bares - ISSq

→ Rolled steel bars are designd by ISRO followed by dn diameters in case of round bars and ISSq followed by side width in cas of square.

Ex:- ISRO 16

IS Rq 20

Rolled Steel Tubes:-

These sections are designed by their nomenal bore size

→ In each size their are 3 clames namely light, medium & heavy.

Rolled Steel plates:-

Rolled steel plate s. of the following thickness are available

5, 6, 8, 10, 12, 14, 16, 18, 20, 22, 25, 28, 32, 36, 40, 45

50, 56, 63, 71, 80 MM

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→ These plates are designed by ISPL followed by length, width and thickness. eg. ISPL 2000x1000

Rolled Steel Strips:-

Rolled steel strips is designated as ISST followed by width and thickness.
→ Rolled steel strips is designated as ISST followed by ISST 20 x 25 MM. (width x thickness)

Rolled steel flats:-

flats are differ from strip in the sense the thickness of flats in 5 MM onward and the width is limited.
→ They are disignated by width followed by letter ISF 10 (80 width ~~S~~ 8 10 thickness)

Special consideration in steel design:-

The following special consideration are required in the steel design.

(1) Size and shape
(2) Buckling
(3) Minimum thickness
(4) Connection design

Size and shape:-

steel is manufactured in steel mills and is available in certain shape and size.
→ Hence the member of steel structure should be ~~dsign~~ design to consist of any of the available section or combination of them.

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Buckling consideration:-

The permissible load per unit area in steel is much higher as compared permissible value in concrete

→ As the member in steel structure are more slender, the compression members in steel structure are ~~is~~ liable to buckling

Minimum Thickness:-

Corrosion needs special consideration in steel design.

→ If very thin section are used as a small amount of corrosion may result into a large percentage reduction if effective area

Needs for design of connection:-

A steel design is not complete of the following connections are not designed.

(a) Riveted connection
(b) Bolted connection
(c) Welded connection

Loads

Various loads expected to act on a structure may be classified as below.

(a) Dead load (DL)
(b) Imposed load (IL)
(c) Wind load (WL)
(d) Earthquake loads (EL)
(e) Erection loads (ER)
(f) Accidental loads (AC)
(g) Secondary effects.

(a) Dead load:-

Dead load include the weight of all permanent construction For example, in a building weight of roofs, floor, finishes

(b) Imposed loads:-

IS 800-2007 group the following loads as imposed loads

(i) Live load
(ii) Crane load
(iii) Snow load
(iv) Dust load
(v) Hydrostatic load & earth pressure
(vi) Impact loads
(vii) Horizontal loads

Live load

The load which keep on changing from time to time are called live load.

Crane load

These loads includes loads from crane and other machines acting on the structure.

Snow loads

IS 875 deals with snow loads on roof of the buildings. This load is to be considered for the building to be considered located in the regions where snow is likely to fall.

Dust load

In areas prone to settlement of dust on roof provision for dust load equivalent to probable thickness of accumulation of dust may be made.

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Hydrostatic & Earth pressure:-

IS 875 gives specifications for considering such loads. In the design of structures partly or fully below ground levels.

Impact load:

For structures supporting moving loads suitable aditional allowance of load should be made by increasing imposed load.

Horizontal lads:-

parapets, balustrads and their supporting structure shall be designed for the horizontal forces act at the hand rail or coping level

wind loads:-

The force exerted by the horizontal comopent of wind is to be considered in the design of building, towers etc.

Earth quake load:-

Earth quak shocks cause movement of foundation of structure. The total vibration caused by earth quak may be resolved into 3 mutually perpendicular directions, usually taken as vertical and two horizontal directions.

→ Movement in horizontal direction needs special consideration

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(e) · Erection loads

Prefabricated or precast members are subjected to different type of support and different type of loads. during erection compared to the types of supports and types of supports and loads after erection.

(f) Accidental loads :-

IS 875 gives certain guidelines to take care of following ~~differ~~ accidental loads on the structures.

(i) Impact & collision
(ii) Explosions
(iii) Fire

(g) Secondary Effect :-

The following types of secondary effects should be looked into the design.

* Differential settlement of foundations
* Differential shortening of columns
* Eccentric connections
* Rigidity of joints.

Load combination :-

A judicious combination of the loads is necessary to ensure the required safety and economy on the design keeping in view the probability of

(a) Their acting together
(b) Their dispsition in relation to other loads and severity of stresses or deformation.

Structural Analysis :-

Structural Analysis is necessary to find the internal & forces developed in the member of the structure

→ IS code permis the following method analysis

(a) Elastic Analysis

(b) plastic analysis

(c) Advanced analysis

(d) Dynamic Analysis

Elastic Analysis :-

It is based on the ~~assumption that~~ assumption that the member has yielded for the design Load and stress is linearly proportional to strain

Plastic Analysis :-

In this method it is assumed that when every fiber at a section reaches yield stress a plastic hinge is formed

Advance Analysis :-

For a fram with full lateral restraints, an advanced structure being designed, will perform satisfactorily during its intened life

→ The design philosophies used are listed below

(i) Working stress method

(ii) Ultimate load design

(iii) Limit state design

Working stress Method :-

This is the oldest systematic analytical design method. Through IS 800-2007 insists for the limit state design, permits use of this method wherever LSD cannot be conveniently adopted

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Ultimate Load Method:-

The limitation of working stress method to asses actual load carrying capacity, mad researchers to develope ultimate load method, which is also known as load factor method.

Limit State Design:-

It is the comprehensive method which will take care of both strength and serve-ceability requirements.

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CHAPTER-2

Structural steel fasteners and connections Bolted connection.

classification of Bolts Based on types of load transfer

On the basic of load transfer in connection bolts may be classified as

(a) Bearing type
(b) friction Grip type

Unfinished bolts and finished bolts belong to bearing type since they transfer shear force from one member to other member by bearing, where as HSFG bolt belongs to friction grip type since they transfer shear friction.

Advantages and disadvantages of bolts connection:—

The following are the advantages of bolted connection over rivited or welded connections.

(i) Making joints is noiseless
(ii) Do not need skilled labour
(iii) Need less labour.
(iv) connection can made quickly.
(v) Working area required in the field is less.
(vi) Structure can be put to use immediately.

Disadvantages:-

(i) Tensile stress is reduced considerably due to stress ~~connections~~ concentration and reduction of area at the root of the theads.
(ii) Rigidity of joints reduced due to loose fit resulting into excessive deflection,
(iii) Due to vibration nuts are likely to loosen.

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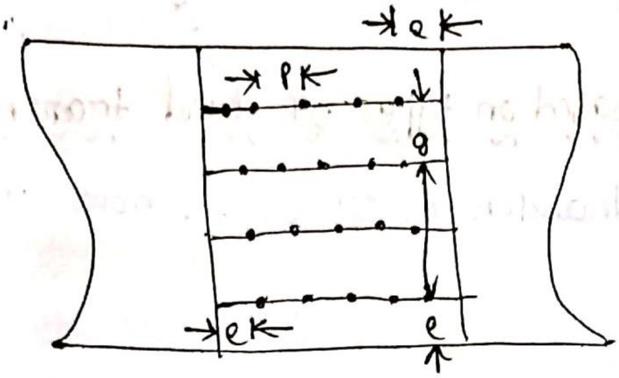
imp spacing

Terminology :-



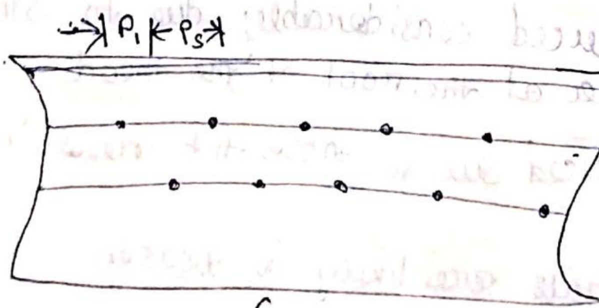
(1) **Pitch of the bolts (p)** :- It is the center to centre spacing of the bolts in a row measured along the direction load It is shown as 'p'

(2) **Gauge distance (g)** :- It is distance between two consecutive bolt of adjacent rows and is measured at right angle to the direction of load.

(3) **Edge distance (e)** :- It is the distance of centre of bolt hole from the adjacent edge of plate.

(4) **End distance (e')** :-
It is the distance of the nearest bolt hole from the end of the plate

(5) **staggered Distance** :-
It is the centre to centre distance of staggered bolts measured obliquely on the member.



(Bolt distance is staggered bolts)

Types of bolted connections :-

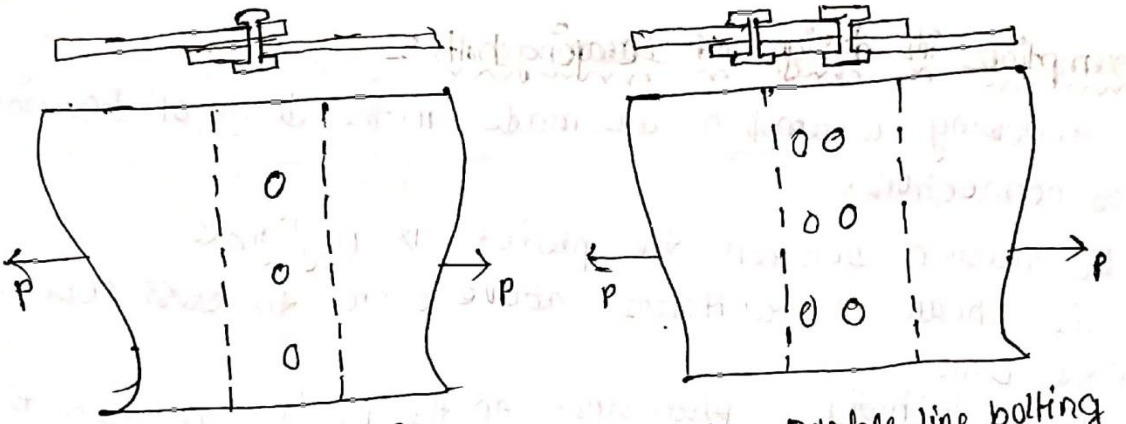
Type of joints may be grouped in to the following two :

(a) Lap joints

(b) Butt joints

(a) Lap joints :-

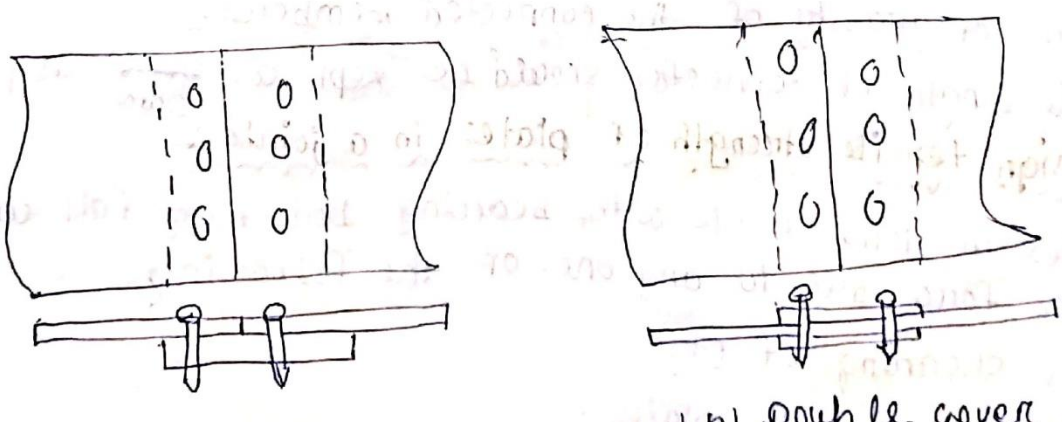
It is simplest type of joints. In this the plates to be connected over lap one another.



(a) single line bolting (b) Double line bolting

(b) But joints :-

In this type of connection the two main plates about against each other and connection is made by providing a single cover plate connected to main plate or by double cover plates, one on either side connected to the main plates.



(a) single cover butt joints (b) Double cover butt joints

Type of Actions on fasteners :-

Depending upon the type of connection and loads, bolts are subjected to the following types of actions.

(a) Only one plane subjected to shear (single shear)
(b) Two plane subjected to shear (Double shear)
(c) pure tension
(d) Pure moment
(e) Shear and tension

Assumption in design of bearing bolts :-

The following assumption are made in the design of bearing bolted connection.

(i) The friction between the plates is negligible
(ii) The shear in uniform ~~above~~ over the cross section of the bolts.
(iii) The distribution of ~~plate~~ stress on the plates between the bolts holes is uniforms.
(iv) Bolt in a group subjected to direct load shears the load equally.

Principles observed in the design :-

The following principle are observed in the design of connections.

(i) The centre of gravity of bolts should coincide with the centre of gravity of the connected members.
(ii) The length of connection should be kept as ~~small~~ same as possible.

Design tensile strength of plates in a joints :-

plates in joint made with bearing bolt may fail under tensile force due to any one of the following

(i) Shearing of edge
(ii) crushing of plates.

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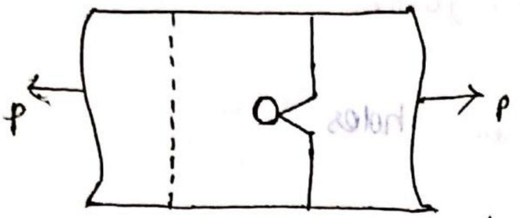
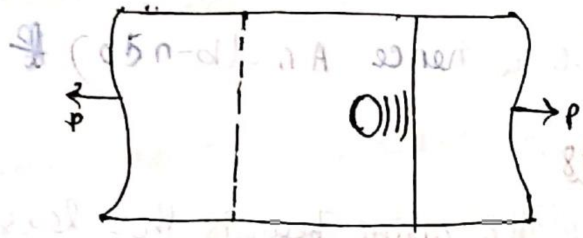
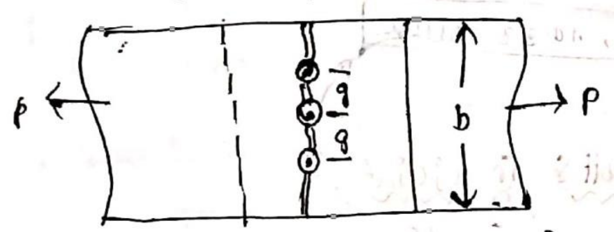
(iii) Rupture of plates



(Shearing of plates)



(crushing of plates)



(Rupture plates)

If the minimum distances are ansured in a joints, the design tensile strength of plate in the joint is the strength of thinerst member against rupture. This strength is given by

$$\boxed{T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m_1}}}$$

page no 32 clause no 6.3 code book

where γ_{m_1} = partial safety factor - page no 30 Table-5

f_u = ultimate stress of materials

A_n = net effective area of plates

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

where·

b = width of plate

t = thickness of thinner plates in joint

do = diameter of bolts holes

g = gauge length between the bolt holes

Ps = Staggered pitch

n = number of bolt holes

Note:-

If there is no staggering, Psi = 0 & hence $A_n = (b - n d_o) t$

Design strength of bearing bolts.

The design strength of bearing bolts under shear is the least of following.

(a) Shear capacity $\boxed{\text{75 page, 10.3.3 clause}}$

(b) Bearing capacity

shear capacity of bearing bolts in a joint:-

Design strength of the bolts; V_{dsb} is given by

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

V_{nsb} = Nominal shear capacity of bolts

γ_{mb} = partial saffety factor of material

$$\boxed{V_{nsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})}$$ $\boxed{\text{75 page, 10.3.3 clause}}$

where,

f_{ub} = Ultimate tensile strength of bolts

n_n = Number of shear planes with ~~thead~~ threads

n_s = Number of shear planes without threads

A_{sb} = Nominal ~~shear~~ shear shank Area of the bolts

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

$A_{nb} = 0.78 \frac{\pi}{4} d^2$

Reduction factor for shear capacity of bolts :-

The code suggests the use of reduction factors for shear capacity in the following situations.

(i) If the joint is too long

(ii) If the grip length is large

(iii) If the packing plates of thickness more than 6mm are used

Reduction factor for long joints (β_{lj})

β_{lj} ~~β_{lj}~~ $= 1.075 - 0.005 \frac{L_j}{d}$ $\boxed{\text{page 75 clause 10.3.3.1}}$

Limits $0.75 \leq \beta_{lj} \leq 1.0$, where 'd' is nominal diameter of bolt.

Reduction factor if grip length is large (β_{lg})

$$\beta_{lg} = \frac{8d}{3d + L_g}$$ $\boxed{\text{10.3.3.2 clause Page 75}}$

L_g = grip length = total thickness of the connected plates.

Reduction factor if packing plates are used (β_{pk})

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

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

Thus ~~Bearing~~ shearing capacity of the bolts in shear

is $$\boxed{\frac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})\, \cancel{\beta_{lj}}\, \beta_{lg}\, \beta_{pk}}$$ $\boxed{\text{Page - 75, clause 10.3.4}}$

Bearing capacity of Bolts (V_{dpb}) :-

IS 800-2007 suggests the following procedure to find bearing strength of bolts

$$\boxed{V_{dpb} = \frac{V_{npb}}{\gamma_{mb}}}$$ $\boxed{\text{75 Page 10.3.4 clause}}$

Where V_{dpb} = design bearing strength

V_{npb} = nominal bearing strength

γ_{mb} = partial safety factor

→ Nominal shearing strength may be found from the following relation

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

Where K_b is smaller of $\frac{e}{3d_0}$, $\frac{p}{3d_0} - 0.25$, $\frac{f_{ub}}{f_u}$, 1.0

in which e, p = end & pitch distances

d_0 = diameter of hole

d = Nominal diameter

t = thickness

f_{ub}, f_u = Ultimate tensile stress of the bolt and plate

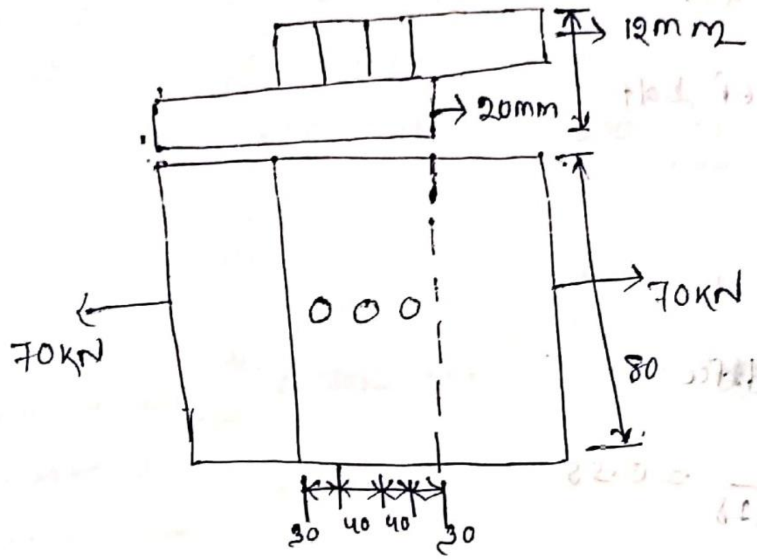
Efficiency of a joints :-

It is defined as the ratio strength of joints and strength of solid plate in tension. It is usually expressed in percentage. Thus

$$\boxed{\text{efficiency } \eta = \frac{\text{Strength of joint}}{\text{Strength of solid plate}} \times 100}$$

Problem-1

Design a lap joint between two plates as shown in figure below. show as to transmit a factor load of 70 KN Ug M16 bolt of grade 4.6 and grade 410 for plate



Solution

Given data

$d = 16$ mm

$d_0 = 16+2 = 18$ mm

$f_u = 410$ N/mm^2

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

shearing strength of bolt

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

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

$$= \frac{400}{\sqrt{3}}(1 \times 157 + 0)$$

$$= 36257.59 \text{ N}$$

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2$$

$$= 0.78 \times \frac{\pi}{4} \times 16^2$$

$$= 156.82$$

$$\simeq 157 \text{ mm}^2$$

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

$= \frac{36257.59}{1.25}$

$= 29006.072N$

Bearing capacity of bolt

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

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

$K_b = \frac{e}{3d_0} = \frac{30}{3 \times 18} = 0.55$

$K_b = \frac{p}{3d_0} - 0.25 = 0.49$

$K_b = \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

$K_b = 1$

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

$= 2.5 \times 0.49 \times 16 \times 12 \times 410$

$= 96432$

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

$= \frac{96432}{1.25}$

$= 77145.6$

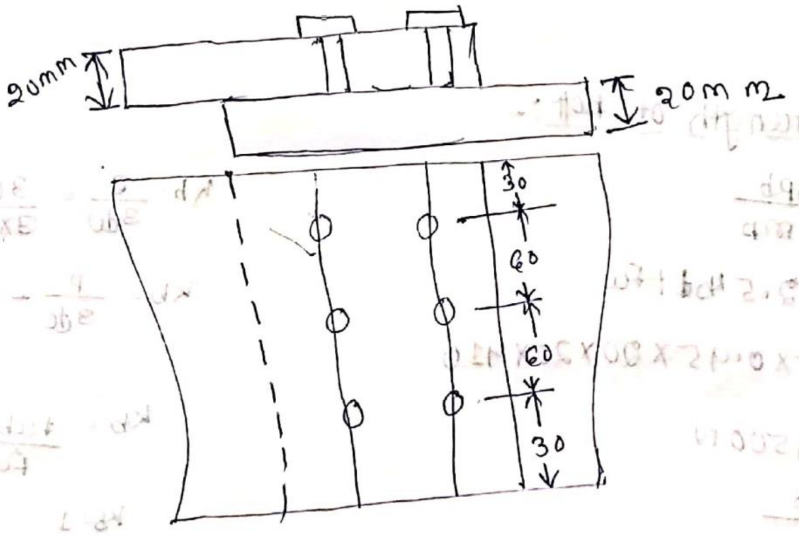
From the above shearing and bearing strength of bolt the result will minimum of those two that is design strength of bolt $= 29006.07N$

So from the above design we clearly know that the strength of plate or given load always be greater the design strength of bolt

$$29006.07N < 70000 N$$
(OK)

Problem-2

Find the efficiency of the lap joint as shown in the diagram below; Given M20 bolt of grade 4.6 and Fe 410 plates are used



Given data.

$d = 20\ mm$
$d_o = 22mm$
$f_{ub} = 400\ N/mm^2$
$f_u = 410\ N/mm^2$

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Strength of plate

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

$$= \frac{0.9 \times 2280 \times 410}{1.25}$$

$$= 673056$$

$A_n = (b - n d_h) t$

$= (180 - 3 \times 22) \times 20$

$= 2280$

$A_{nb} = 0.78 \times \frac{\pi}{4} d^2$

$= 245$

Shearing strength of bolt :-

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

$$= \frac{339481.82}{1.25}$$

$$= 271585.56 N$$

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

$= \frac{400}{\sqrt{3}} \times (1 \times 245 + 0)$

$= 56580.32$

$V_{nsb} = 56580.32 \times 6$

$= 339481.82$

Bearing strength of bolt :-

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

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

$= 2.5 \times 0.45 \times 20 \times 20 \times 410$

$= 184500 N$

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

$$= \frac{184500}{1.25} = 147600$$

$= 147600 \times 6 = 885600$

$k_b = \frac{e}{3 d_0} = \frac{30}{3 \times 22} = 0.45$

$k_b = \frac{p}{3 d_0} - 0.25 = \frac{60}{3 \times 22} \times 0.25$

$= 0.65$

$k_b = \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

$k_b = 1$

Strength of joint = ~~....~~ 271585.86

Here the strength of plate grater than strength of joint which is OK for given data

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

strength of solid plate =

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

$A_g = 180 \times 20$
$= 3600$

$$= \frac{3600 \times 250}{1.10}$$

$$= 818181.81$$

$$\text{efficiency} = \frac{271585.56}{818181.81} \times 100$$

$$= 33.19$$

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Chapter-3 Design of Tensions Member

Tensions member are also known as tie member. The form of a tension member is given to a large ep tinef by the type of thes structure by which. it is apart of by by the method of joining it to the adjucant member of the structure.

Design strength of a tension member:- (Page - 32 code book)

The design strength of a tension member is the lowest of the following.

(a) Design strength due to yielding of gross section (T_{dg})
(b) Rupture strength of critical section (T_{dn})
(c) The block shear (T_{dn})

Design strength of due to yielding of gross section (T_{dg}):-

The strength is given by $T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$ (P-32, clause-6.2)

Where f_y = yield stress of the material.

A_g = Gross area of the cross section

γ_{m0} = Portial safety factor.

Design strength due to Rupture on critical section (T_{dn}):-

This is given by $T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$ 6.3.2

Where A_n = Net affective area of a critical section

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

for thread rods & bolts

than

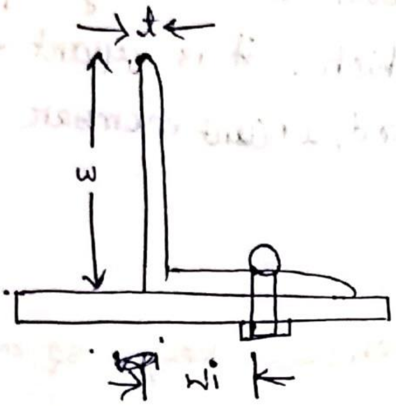
$$T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$$

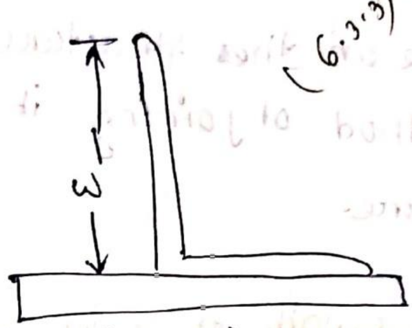
Where, A_n = Net area at the threaded section

$A_n = 0.78 \cdot \frac{\pi}{4} \times d^2$

Single angle :-

(6.3.3)





$b_s = w$

$b_s = w + w_i - t$

As the effectiveness of outstanding leg is less, the design strength as governed by rupture at net section is given by

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_u}{\gamma_{m0}}$$

Where, A_{nc} = Net area of the connected leg.

A_{go} = Gross area of the outstanding

$$\beta = 1.4 - 0.076 \cdot \left(\frac{w}{t}\right) \times \left(\frac{f_y}{f_u}\right) \times \frac{b_s}{L_c} \leq \frac{f_u \gamma_{m0}}{f_y \gamma_m} \ngtr 0.7$$

where w = Outstanding leg width

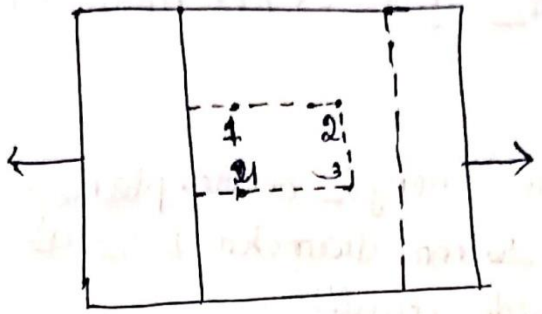
b_s = shear leg width

L_c = Length of the end connection

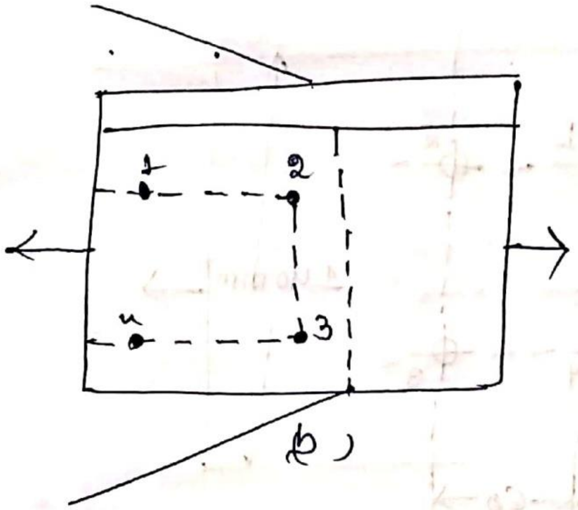
t = thickness of leg

Design strength due to block shear :- (6.4)

At the connected end failure of a tension member may occure along a path involving shear along one plane & tension on a perpendicular plane along the fastener. This type or failure is known as block failure.



(a)



(b)

Referring to figure (a) shear failure occurs along 1-2 & 3-4 where as tensions failure occur along 2-3

Referring to figure (b) shear failure occurs along 1-2 & ~~to~~ tension failure occurs 2-3.

IS 800: 2007 recommended the following block shear strength

T_{db} if bolted connection are used. It shall be smaller of

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

or

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

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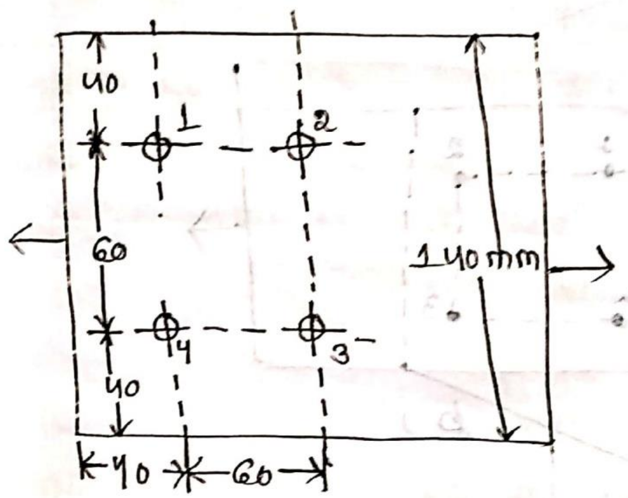
Where Avg, Avn = Minimum gross ~~section~~ & area in shear.

Atg, Atn = Minimum gross & net area in tension

Q-1

Determine the tensile strength of the plate 130mm x 12mm with the holes for 16 mm diameter bolts as shown below steel used is of Fe410 grade quality.



Given data:-

$d = 16$ mm

$d_0 = 16+2 = 18$mm

$f_u = 410 \, N/mm^2$

$t = 16$mm

yielding of gross section:-

$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$ $\qquad A_g = 140 \times 16 = 2240 mm^2$

$= \frac{2240 \times 250}{1.1}$

$= 509090.909 N$

Rupture of critical section:-

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

$A_n = (b - n d_0) t$

$= (140 - 2 \times 18) \times 16$

$= 1664$

$$= \frac{0.9 \times 1664 \times 410}{1.25}$$

$= 491212.8 N$

Block shear strength:-

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

$A_{vg} = 2\{(40+60) \times 16\} = 3200$

$A_{tn} = (60 - 18) \times 16 = 672$

$A_{vn} = (40 + 60 - 1.5 \times 18) \times 16 \times 2 = 2336$

$A_{tg} = 60 \times 16 = 960 mm^2$

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

$$= \frac{3200 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 672 \times 410}{1.25}$$

$= 618265.50 N$

or

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

$$= \frac{0.9 \times 2336 \times 410}{\sqrt{3} \times 1.25} + \frac{960 \times 250}{1.1}$$

$= \cancel{755951.53N}$ 616315.17

∴ Minimum of the above two block shear values = ~~618265.50N~~ 616315.17

∴ The strength of the tension member is the minimum of T_{dg}, T_{dn} and T_{db}

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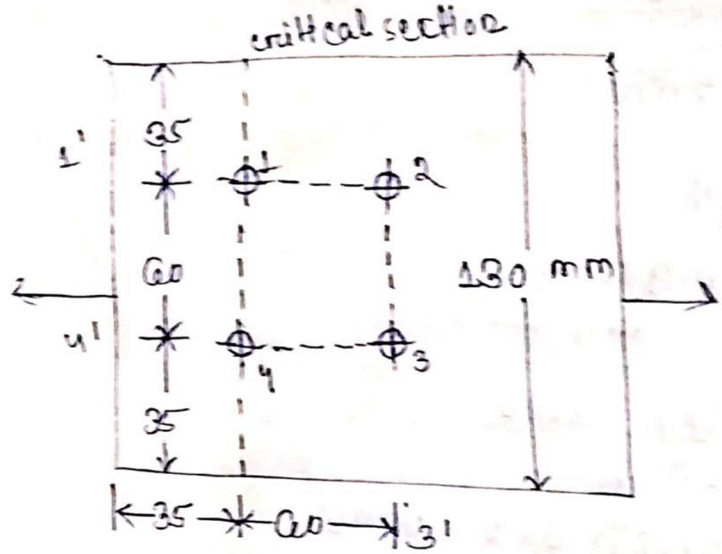
Q-2

Determine the tensile strength of the plate 130 x 12 mm with the holes for 16mm diameter bolts ~~holes~~ as shown below. steel used is of Fe410 grade quality.



Given data

$d = 16mm$

$d_0 = 18mm$

$f_u = 410$

$f_y = 250$

yielding strength of gross section :-

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

$A_g = 130 \times 12 = 1560$

$$= \frac{1560 \times 250}{1.1}$$

$$= 354545.45N$$

Rapture of critical section:-

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

$$= \frac{0.9 \times 1128 \times 410}{1.25}$$

$$= 332985.6$$

$A_n = (b - n d_h) t$

$= (130 - 2 \times 18) 12$

$= 1128$

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Block shear :-

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

$A_{vg} = 2\{(35+60)\times 16\} = 3040$

$A_{tn} = (60-18)\times 16 = 672$

$A_{vn} = (35 + 60 - 1.5\times 18)\, 16\times 2 = 2176$

$A_{tg} = 60\times 16 = 960\ mm^2$

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

$$= \frac{3040\times 250}{\sqrt{3}\times 1.1} + \frac{0.9\times 672\times 410}{1.25}$$

$$= 597270.94$$

or

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

$$= \frac{0.9\times 2176\times 410}{\sqrt{3}\times 1.25} + \frac{960\times \cancel{410}\ 250}{1.1}$$

$$= 589045.76$$

The minimum of the above two block shear value = 597270.94N

$T_{dn} = 332985.6$

$T_{dg} = 354545.45$

$T_{db} = \cancel{597270.94}$ 589045.76

The strength of minimum tension member = 332985.6

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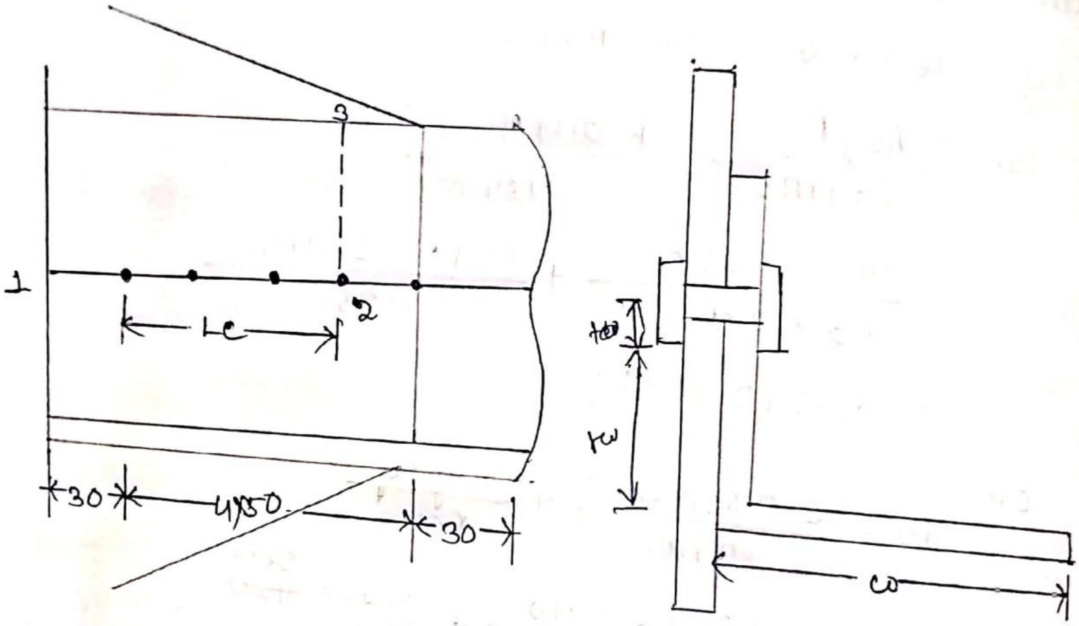
Q3

A single unequal angle ISA 90 60, 6 mm is connected to a 10mm gusset plate at the end with 5 number of 16mm bolt to transfer tension. Determine the design tensile strength of the angle.

(a) If the gusset is connected to 90mm leg

(b) " " " " 60 " .



Given data:-

d = 16mm

do = 18mm

fu = 410

g = 50mm if 90mm leg is connected.

g = 30mm if 60 mm leg is connected

To calculate the strength of angle section that is

(a) yeilding of gross section

(b) Rupture of critical section

(c) Block shear failure.

Yeilding of gross section

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

$= \frac{1560 \times 250}{1.1}$

$= 354545.45 \text{ N}$

$A_g = 260 \times 6$

$= 1560$

Rupture of critical section

strength as goven by rupture of critical section

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

$$A_{nc} = \left(90 - \frac{6}{2}\right) \times 6 = 522$$

$$A_{go} = \left(60 - \frac{6}{2}\right) \times 6 = 342$$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \leq \left(\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}}\right) \geq 0.7$$

$t = 6\ mm$

$L_c = 4 \times 5 = 200$

$b_s = w + w_1 - t$

$= 50 + 60 - 6 = 104$

$$\beta = 1.4 - 0.076 \times \left(\frac{60}{6}\right)\left(\frac{250}{410}\right)\left(\frac{104}{200}\right)$$

$= 1.15$

$$\frac{F_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.47$$

$$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 522 \times 410}{1.25} + \frac{1.15 \times 342 \times 250}{1.1}$$

$= \cancel{243480.76}$ $239594.4N$

Block shear strength

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

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$A_{vg} = 230 \times 6 = 1380$

$A_{vn} = (230 - 4.5 \times 18) \times 6 = 894$

$A_{tg} = 40 \times 6 = 240$

$A_{tn} = (40 - 0.5 \times 18) \times 6 = 186$

$= \cancel{1380 \times 2}$

$$T_{db} = \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25}$$

$$= 235985.239 \text{ N}$$

or

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

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{240 \times 250}{1.1}$$

$$= 206913.271 \text{ N}$$

~~From the above~~
Minimum of the above two block shear values =

206913.27 N

60mm connected leg

Yeilding of gross section

$T_{dg} = \frac{A_g f_y}{\gamma_{m0}}$ $\quad A_g = 865 mm^2$

$= \frac{865 \times 250}{1.1}$

$= 196590.909 N$

Rupture of critical section:-

strength as governed by rupture of critical section

$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$

$A_{nc} = \left(60 - \frac{6}{2}\right) \times 6 = 342 mm^2$

$A_{go} = \left(90 - \frac{6}{2}\right) \times 6 = 522 mm^2$

$\beta = 1.4 - 0.076 \left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)$ $\quad b_s = 90 + 30 - 6 = 114$

$= 1.4 - 0.076 \left(\frac{90}{6}\right)\left(\frac{250}{410}\right)\left(\frac{114}{4 \times 50}\right)$

$= 1.00$

$\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44$

$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$

$= \frac{0.9 \times 342 \times 410}{1.25} + \frac{1.00 \times 522 \times 250}{1.1}$

$= 219594.763 N$

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Block shear

Failure may taken along section 1,2,3

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

$A_{vg} = 230 \times 6 = 1380$

$A_{vn} = (230 - 4.5 \times 18) \times 6 = 894$

$A_{tg} = 30 \times 6 = 180$

$A_{tn} = (30 - 0.5 \times 18) \times 6 = 186$

$$T_{db} = \frac{1380 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 186 \times 410}{1.25}$$

$= 235985.239 N$

or

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

$$= \frac{0.9 \times 894 \times 410}{\sqrt{3} \times 1.25} + \frac{180 \times 250}{1.1}$$

$= 193276.907 N$.

Minimum of the above two block shear value $= 193276.907N$

(a) Design a single angle section for a tension member of a roof truss to carry a factor tensile force of 225 kN. The member is subjected to the possible reversal of stress due to action of wind. The effective length of the member is 3m. Use 20mm shop bolts of grade 4.6 for the connection.

Solution

Given data -

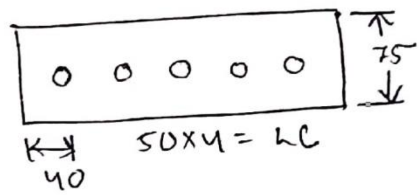
$d = 20mm$

$d_o = 22mm$

$\cancel{f_u = 410\ N/mm^2}$

$T_u = 225\ N$

$L_{eff} = 3\ m$



From the consideration of yield strength, gross area of the angle section.

$$A_g = \frac{T_u \times \gamma_{m0}}{f_y}$$

$$= \frac{225 \times 10^3 \times 1.1}{250}$$

$$= 990\ mm^2$$

Try ISA 75x75, 10 mm which has gross area $A_g = 1402 mm^2$

Number of bolt required.

Use gusset plate of thickness 12mm

strength of 1 bolt in single shear

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

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times 20^2$$

$$= 245.04$$

$$\simeq 245$$

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

$$= \frac{400}{\sqrt{3}}(1 \times 245)$$

$$= 56580.32$$

$$V_{dsb} = \frac{56580.32}{1.25}$$

$$= 45264.25$$

Bearing strength

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

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

$= 2.5 \times 0.5 \times 20 \times 12 \times 410$

$= 102500$

$V_{dpb} = \frac{102500}{1.25}$

$= 82000$

$p = d \times 2.5$

$= 20 \times 2.5 = 50mm$

$e = d_0 \times 1.5$

$= 22 \times 1.5 = 30 = 40mm$

$k_b = \frac{e}{3d_0} = \frac{40}{3 \times 22} = 0.6$

$k_b = \frac{p}{3d_0} - 0.25$

$= \frac{50}{3 \times 22} - 0.25 = 0.5$

$k_b = \frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$

$k_b = 1$

The strength of one bolt will be the minimum of the above two values : 45264.25

Number of bolt required = $\frac{\text{given load}}{\text{strength of 1 bolt}}$

$= \frac{225 \times 10^3}{45264.25}$

$= 4.97$

$\simeq 5$

Check the design

yailding of gross section

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

$= \frac{1402 \times 250}{1.1} = 318636.36N$

Rupture of critical section

$T_{dn} = \frac{0.9 A_{nc} f_u}{\gamma_{m1}} + \frac{\beta A_{go} f_y}{\gamma_{m0}}$

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

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$t = 10$

$w = 75$

$Lc = 50 \times 4 = 200$

$bs = w + w_1 - t = 75 + 40 - 10 = 105$

$A_{nc} = \left(75 - \frac{10}{2}\right) \times 10 = 700$

$A_{go} = \left(75 - \frac{10}{2}\right) \times 10 = 700$

$\beta = 1.4 - 0.076 \cdot \left(\frac{75}{10}\right)\left(\frac{250}{410}\right)\left(\frac{105}{200}\right)$

$= 1.217$

$\frac{f_u \gamma_{m0}}{f_y \gamma_{m1}} = \frac{410 \times 1.1}{250 \times 1.25} = 1.44$

$T_{dn} = \frac{0.9 \times 700 \times 410}{1.25} + \frac{1.217 \times 700 \times 250}{1.1}$

$= 400253.63 N$

<u>Block shear strength</u> :-

$T_{db} = \left(\frac{A_{vg} \times f_y}{\sqrt{3} \times \gamma_{m0}} + \frac{0.9 \times A_{tn} f_u}{\gamma_{m1}}\right)$

$A_{vg} = 240 \times 10 = 2400 mm^2$

$A_{vn} = (240 - 4.5 \times 22) \times 10 = 1410 mm^2$

$A_{tg} = 40 \times 10 = 400 mm^2$

$A_{tn} = (40 - 0.5 \times 22) \times 10 = 290 mm^2$

$T_{db} = \left(\frac{2400 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 290 \times 410}{1.25}\right)$

$= 400526.32 N$

or

$$T_{db} = \frac{0.9 A_{vn} f_u}{\sqrt{3} \times 1.25} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 1410 \times 410}{\sqrt{3} \times 1.25} + \frac{400 \times 250}{1.1}$$

$$= 331220.74 N$$

$\therefore T_{db} = 331220.74 > 225000.00 N$ (ok)

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Chapter 3.2

Welded connection :-

Welding consist of joining two pieces of metal by establishing a metallurgical bond between them.

Imp. 10 mark in 5 marks

Advantages & disadvantages of welded connection

Advantages

(i) Due to the absence of gusset plate connecting angle etc, welded structures are lighter. Noise produce in welding process is relatively less.

(ii) Welded joints are rigid

(iii) Alternations in connection can be easily made in the design of welded connections.

(iv) Welded connection have good as aesthetic appearance

(v) Welded connection is air tight & water tight.

Disadvantages:-

(i) Due to uneven heating & colling members are likely to distort in the process of welding.

(ii) There is a greater possibility of brittle fracture in welding.

(iii) Proper welding in field condition is difficult.

(iv) Highly skilled person is reqired for welding.

(v) Welded joints are over rigid.

Types of welded joint :-

There are 3 types of welded joints.

(1) Butt weld
(2) Fillet weld
(3) Slot weld & plug weld

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(1) Butt well :- Butt weld is also known as groove weld.

Types of butt well

Fillet weld :-

Fillet weld is a weld of approximately triangular cross-section joining two surfaces; approximately at right angle two each other in lap joint, Tee-joint or corner joint.

→ When the cross-section of fillet welded is iso-scales triangle with face at 45°, it is known as a standard fillet weld.

In special circumcamstances 60° & 30° angle are used.

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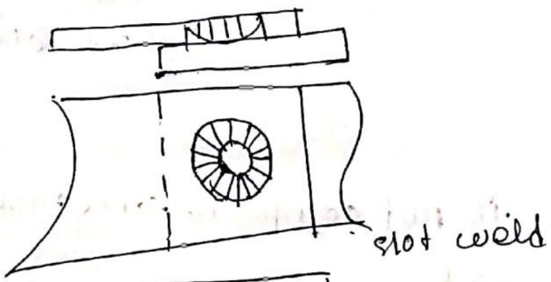
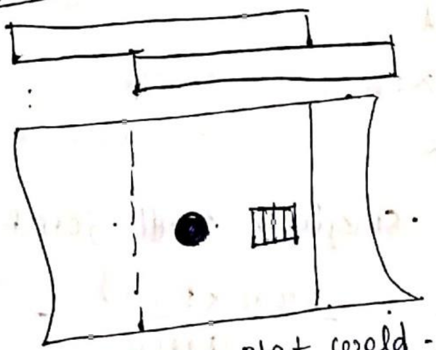
A fillet weld is known as concave fillet weld or as mitre fillet weld depending upon the shape of weld face

(3) Slot weld & plug weld :—

(i) As shown as typical slug weld in which a plate with circular hole is kept with anather plate to be joint & then fillet welding is made along the perifarey of the hole.

(ii) As shown typical plug welds in which small holes are made in one plate & is kept over anather plate to be connected & then the entire hole is filled with filler material





Q-1 A 18 mm thick plate is joint to a 16 mm plate by 200 mm long butt weld. Determined the strength of joint if

(1) A double V butt weld is used

(2) A single V butt weld is used

Assume that Fe410 grade plate & soft welds are used.

1) Double V weld

Given data :-

$t = 16mm$

effective length $(L_w) = 200mm$

$\gamma_{mw} = 1.25$

$f_u = 410 N\text{-}mm^2$

$F_{wn} = \frac{f_u}{\sqrt{3}} = \frac{410}{\sqrt{3}}$ (79)

$= 236.71N$

stress $= \frac{load}{area}$

load $=$ stress $\times$ area

$= f_{wn} \times L_w \times t$

$= 236.71 \times 200 \times 16$

$= 757472$

$\therefore$ Design strength of weld $= f_{wd} = \frac{f_{wn}}{\gamma_{mw}}$

$= \frac{757472}{1.25}$

$= 605977.6N$

(2) single V weld :-

since the penetration is not complete effective.

throat thickness $= \frac{5}{8} \times t$

$= \frac{5}{8} \times 16$

$= 10$

$\therefore$ The design strength of single V butt joint.

$= f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_{wn} \times L_w \times t}{\gamma_{mw}}$

$= \frac{236.71 \times 200 \times 10}{1.25} = 378736N$

Q-2

Design a suitable Longitudinal fillet weld to connect the plates as shown below to transmit a pull equal to the full strength of small plate. Given plates are 12 mm thick grade of plates is Fe 410 and welding to be made in workshop.

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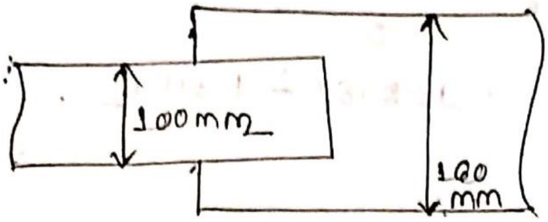

Ans:-

Given data

Thickness of plate (t) = 12mm

$F_u = 410$

Breadth of the plate (b) = 100mm

Minimum sized to be used = 5mm

The maximum size = The size of the plate or the thickness of the plate − 1.5

= 12 − 1.5

= 10.5mm ≃ 10mm

So = 10mm

Full design strength of smaller plate

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

$A_g = b \times t$

$= 100 \times 12$

$= 1200$

$$= \frac{1200 \times 250}{1.1}$$

$= 2727.27.27 N$

Assume normal weld, through thickness.

$t = 0.7 \times S = 0.7 \times 10 = 7$

Design strength of weld (F_{wd}) $= \frac{F_{wn}}{\gamma_{mw}}$

$$\Rightarrow F_{wd} = \frac{f_u/\sqrt{3}}{\gamma_{mw}}$$

$$\Rightarrow F_{wd} = \frac{L_w \times t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$\Rightarrow 272727.27 = \frac{L_w \times t \times f_u}{\sqrt{3} \times \gamma_{mw}}$$

$$\Rightarrow L_w = \frac{272727.27 \times \sqrt{3} \times \gamma_{mw}}{7 \times 410}$$

$= 205.79$ mm.

Each length of the welding = $\frac{617.74}{2}$

$= 308.87 \approx 309\ m$.

Chapter-4

Design of steel compression Member

A compression member is a structural member which is subjected to two equal and opposite compressive forces applied at its ends.

→ Strut is a compression member used in the roof truss and bracing. They are of a small ~~smo~~ span and may be vertical or inclined.

→ Column, stanchion or post is a verticle compression member supporting floors or girders in a building. These compression members are subjected to heavy loads.

→ The principal rafter is a top chord member in a roof truss and boom is the principal compression member in a crane

SHAPES OF COMPRESSION MEMBERS:-

→ Since the design stress in compression member decreases with the least radius of ~~grea~~ gyration, the section should be proportioned to have maximum moment of inertia for the same sectional area. This can be achived by concentrating the area away from centroid of the section

→ As far as possible the section should have approximately the same radius of gyration about any axis. This requirement is fulfilled by circular tubes.

→ Due to difficulties in making end connections, they were not commonly used earlier. But now-a-days due to improvements in welding technology tubular sections are getting popular as compression member.

→ Next best shape may be square tubing. Among I-section ISHB section are preferable as colum since they have better r_{min} values for the same area of cross-sections.

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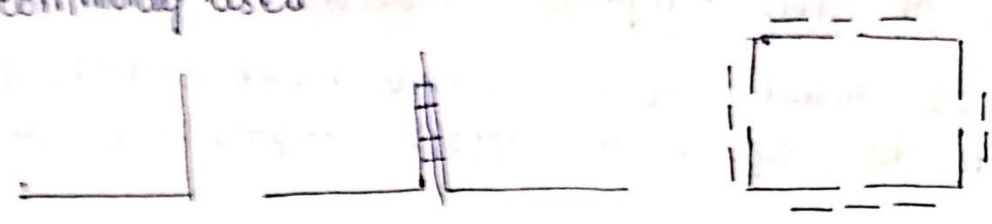
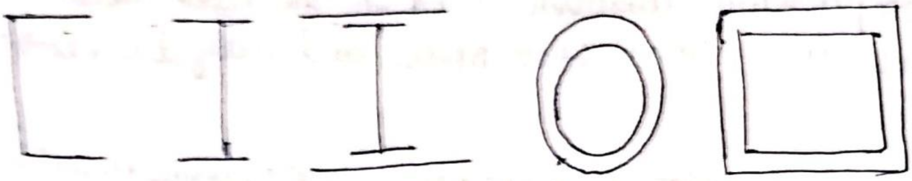
→ In roof trusses and transmission towers angle sections are commonly used.





Buckling class of cross-section Members And slenderness ratio

→ When an axially loaded compression member becomes unstable overall it can be buckle in 3 ways.

(a) Flexural Buckling

(b) Torsional Buckling

(c) Flexural - torsional Buckling.

Flexural Buckling :-

It is a deflection caused by bending or flexure, about the axis corresponding to the largest slenderness ration.

Torsional Buckling :-

The Flexural buckling considered above is due to bending alone that is the sections displace from their original position by translation without rotation.

Flexural torsional buckling :-

This type of failure is caused by a combination of flexural buckling & torsional buckling.

→ The member bends and twist simultaneously. This type of failure can occur only with unsymmetrical cross-sections

Ex:- channels, structural tees, double angle.

Slenderness ratio

Slenderness ratio of a column is defined as the ratio of effective length to corresponding radius of gyration of the section. Thus

$$\text{Slenderness ratio} = \frac{L_e}{r} = \frac{KL}{r}$$

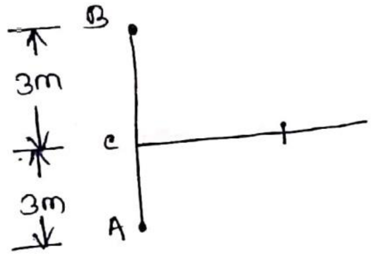
where L = actual length of compression member

$L_e = KL$, effective length

r = appropriate radius of gyration

Actual Length :-

It is the centre to centre distance of compression member between the restrained ends.



Effective length

The effective length KL is calculated from the actual length L of the member considering the rotational and relative translational boundary conditions at the ends.

Design compressive stress and strength

The design compressive stress, f_{cd} of axially loaded compression members shall be calculated using following equation.

$$f_{cd} = \frac{f_y/\gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{0.5}} \le \frac{f_y}{\gamma_{m0}} \quad \text{(34 page)}$$

where $\phi = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right]$

λ = non-dimensional effective slenderness ratio

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

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F_{cc} = Euler buckling stress

α = imperfection factor

γ_{m0} = 1.1 for fe 415 steel

The design compressive strength Pd of a member is given by $\boxed{P_d = A_e f_{cd}}$

Where A_e = effective sectional area

Q-1 Determine the design axial load capacity of the column ISHB 300 @ 577 N/m. If the length of column is 3m and its both end pinned.

Solution:-

Given data:-

Section - ISHB 300@577 N/m

$h = 300mm$

$b = 250mm$

$t_f = 10.6mm$

$t_w = 7.6mm$

for both end pinned. $\lambda = \frac{L_e}{r} = \frac{KL}{r}$

$KL = 1.0L$

$= 1.0 \times 3$

$= 3m$

For the calculation of design strength $P_d = A_e f_{cd}$

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

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

so from IS 800 table 10 we find that

$\frac{h}{b_f} > 1.2$

$\therefore \frac{300}{250} = 1.2$

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Try ISA · tf ≤ 40

tf = 10·6

so here the above condition is satisfied and the buckling axis will y-y axis and buckling class is 'b'

So according to buckling class, as per IS 800 : 2007 $\alpha = 0.34$

From SP6 · r minimum = r_{yy} = 5.41 cm = 54.1 mm

Now $\lambda = \frac{KL}{r} = \frac{3000}{54.1} = 55.45$

$\boxed{E = 2 \times 10^5}$

$\lambda = \sqrt{\frac{f_y}{f_{cc}}}$

$f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{\pi^2 \times 2 \times 10^5}{\left(\frac{3000}{54.1}\right)^2} = 641.92$

$\lambda = \sqrt{\frac{250}{641.92}} = 0.629$

$\phi = 0.5\left[1 + \alpha \cdot (\lambda - 0.2) + \lambda^2\right]$

$= 0.5\left[1 + 0.34 \cdot (0.62 - 0.2) + 0.62^2\right]$

$= 0.763$

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

$= \frac{\frac{250}{1.10}}{0.76 + [0.76^2 - 0.62^2]^{0.5}}$

$= 189.46$

$A_e = 74.85$ cm

$= 7485$ mm

$P_d = A_e f_{cd}$

$= 7485 \times 189.46$

$= 1418108.1$ N

Design of a compression member :-

The following are the usual steps in the design of compression member.

(i) Design stress in compression is to be assumed

For rolled steel beam section of the slenderness ratio varies from 70 to 90. Hence design stress may assumed as 125 N/mm². For angle struts the slenderness ratio varies from 110 to 130. Hence desing stress for such members may be assumed as 90 N/mm². For compression members carrying large loads, the slenderness ratio is comparatively small. ~~Large loads from~~ For such member design stress may be assumed as 200 N/mm².

(ii) Effective sectional area required is $A = \frac{Pd}{Fcd}$

(iii) Select a sectional area ~~required~~ to give effective area required and calculate r minimum

(iv) Knowing the end conditions and deciding the type of connection Determine effective length

(v) Find the slenderness ratio and hence design stress Fcd and Load carrying capacity Pd

(vi) Revise the section If calculated pd differs considerably from the design load.

Q-2

Design a single angle strut connected to the gusset plate to carry 180 KN factor load. The length of the strut between centre to centre connection is 3m.

Solution :-

Given data

$L = 3\,m$
$= 3000\,mm$

$P_d = 180\,KN = 180 \times 10^3\,N$

Assuming compressive stress $f_{cd} = 90\,N/mm^2$

So, now we are proceeding to calculate area $A = \frac{P_d}{f_{cd}}$

$$= \frac{180 \times 10^3}{90} = 2000\,mm^2$$

Try ISA 125 X 95, 12mm, which has area = $2498\,mm^2$

$r_{minimum} = 20.1\,mm$

Assuming the end condition that is one side fix and other hinged so $KL = 0.8L$

$= 0.8 \times 3000$

$= 2400$

Now slenderness ratio (λ) $= \frac{KL}{R}$

$$= \frac{2400}{20.1}$$

$$= 119.40$$

From IS 800: 2007 we know that from angle section buckling class will be (b) so $\alpha = 0.49$

$\frac{KL}{r}$	fy 250	Fcd
		94.6
110		84.35
119.4		83.7
120		

$$y = y_1 + \frac{(y_2 - y_2)}{(x_2 - x_1)} \times (x - x_1)$$

$$= 94.6 + \frac{(83.7 - 94.6)}{(120 - 110)} \times (119.4 - 110)$$

$$= 84.35$$

$P_d = A_e \times F_{cd}$

$= 24.98 \times 84.35$

$= 210706.3N$

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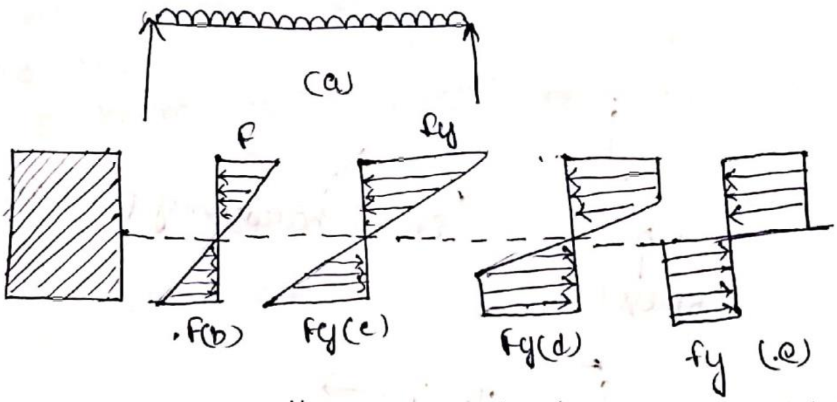
Design of steel beam Chapter. 5

Beam is a structural member with length considerable larger than for cross sectional dimension subjected to lateral load which give rise to bending moment, shear foces in the members.

Based on the lateral supports to compression flanges there are mainly two types of beams.

(i) Laterally supported beam
(ii) Laterally unsupported beam

Plastic moment carrying capacity of a section



(i) Consider the cross section of simply supported beam where the bending moment is maximum for the given loading. Within the elastic limit the stress varies linearly from compression to tension as shown is the diagram that is (b)

(ii) As the load is gradually increased stresses increase proportionality till extream fiber is subjected to yield stress. The extream fiber yield at (c). For simplycity of analysis stress strain for steel is assumed, in which strain hardnig part of the curve is ignored.

(iii) Hence according to theory of plastic analysis highly stressed fiber once yields is not capable of resisting any moment. But interior fiber are not yet yielded and hence additional loads are resisted by unyielded portion of the section.

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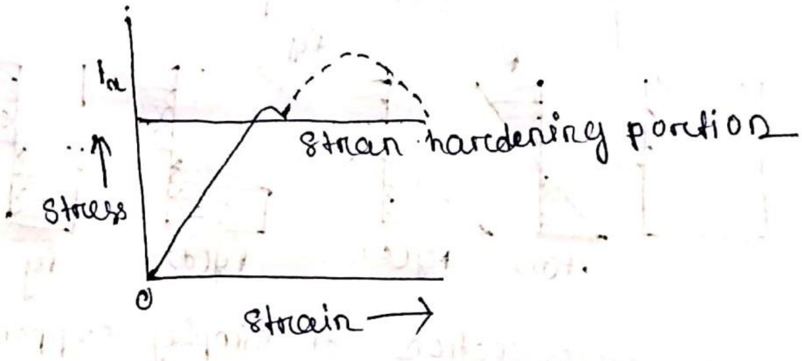
(iv) As the load is gradually increase ~~fiber~~ one by one fiber reach yield stress and stop resisting additional load.

(v) However resistance to load continue till all fibers are yielded as shown in the diagram (e). After this condition the section will not resist further moment due to increase in load. This condition when all fibers at a section yield is called formation of plastic hinge.

(vi) After this stage the rotation at section will take place without resisting additional moment but the moment corresponding to yielding of all fiber is resisted. This moment capacity is called plastic moment capacity of the section and is denoted by Mp



Classification of cross section :-
Page No 17 IS 800 : 2007

Diflection limit :-

Diflection limit should be check before accepting a design. In special situations the other serviciablity limits like vibration limit. durability consideration and fire resistance also should be checked. (Page No-31 IS 800:2007)

✓ ~~stop~~ short note
~~Web~~ buckling

Shorter ~~poor~~ portion of beam at supports ~~it~~ acts as a column to transfer the load from beam to the support.

Hence under this compressive force the ~~tensile~~ web may buckle. This may happen under a concentrated load on the beam also.

→ The load dispersion angle may be taken as 45°. Hence there is need to check for web buckling.

→ However the rolled section are provided with suitable thickness for web. so that web buckling is avoided.

Web crippling:-

Near the support web of the beam may cripple due to lek of bearing capacity. The cripple occurs at the root of the radius.

(ii) IS 800: 2007 has ~~excepted~~ excepted the following formulae to find crippling strength of web. (Page No - 67 clause No 8.7.3.1)

(iii) The Kr is taken in fining the web thickness of rolled still sections to avoid such failures. Hence if rolled ~~still~~ steel section is selection as a beam section there is no need to check for this failure.

Design proccedure of beam

Page No - 52 IS 800: 2007

Q) A laterally supported beam ISMB 600 @ 1202.71 N/m is, placed between two supports. Determine the safe uniformly distributed load that the beam can carry for an effective span of 12m. Take $f_y = 250\ N/mm^2$

Given data:-

ISMB 600 @ 1202.71 N/m

$h = 600$

$b = 210$

$t_f = 20.8$

$t_w = 12.0$

$L = 12m = 12 \times 10^3\ mm$

Step-1

Section classification

$\frac{b}{t_f} = \frac{210}{20.8} = 10.09$

For plastic section as per the IS 800:2007 code book

$\frac{b}{t_f} < 9.4\varepsilon$

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

$= \sqrt{\frac{250}{250}}$

$= \sqrt{1} = 1$

$9.4\varepsilon = 9.4 \times 1 = 9.4$

Further we check the section according to criteria of compact -

$\left(\begin{array}{l} \frac{b}{t_f} \leq 10.5\varepsilon \\ \frac{b}{t_f} > 9.4\varepsilon \end{array} \right)$ $= 10.09 < 10.5$

$= 10.09 > 9.4$

The Section hence it is proved to be compact section

Hence we proceed to calculate the bending moment.

$\frac{d}{t_w} \leq 67\varepsilon$

$\Rightarrow \frac{600}{12.0} = 50$

$\Rightarrow 50 < 67\varepsilon$ (proved)

So, Now we checked the condition that given in the IS 800:2007 codebook. Here from IS 800: 2007 codebook we have two condition that is $V \leq 0.6 V_d$ & another $V > 0.6 V_d$

Now we assumed one condition to calculate M_d.

$v \leq 0.6 V_d$

$$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}}$$

As the section is compact $\beta_b = 1$

$$Z_p = 3510.63 \text{ cm}^3$$
$$= 3510.63 \times 10^3 \text{ mm}$$

$$M_d = \frac{\beta_b z_p f_y}{\gamma_{m0}}$$

$$= \frac{1 \times 3510.63 \times 10^3 \times 250}{1.1}$$

$$= 797870454.5 \text{ N-mm}$$

So after calculating Md we need to check according to the condition given in IS 800:2007,

$$M_d < \frac{1.2 z_e f_y}{\gamma_{m0}}$$

$$\Rightarrow 797870454.5 < \frac{1.2 \times 3510.63 \times 10^3 \times 250}{1.1}$$

$$\Rightarrow 797870454.5 < 834654545.5 \text{ N-mm}$$

Design moment and load carrying capacity from the codebook IS 800, 2007, we know that,

$$m \leq M_d$$

$$\frac{WL^2}{8} \leq 797870454.5$$

$$\Rightarrow \frac{w \times (12 \times 10^3)^2}{8} = 797870454.5$$

$$\Rightarrow w = \frac{797870454.5 \times 8}{(12 \times 10^3)^2}$$

$$= 44.326 \text{ N/mm}$$

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Q.B

Design a simply supported beam of effective span 1.5 m carrying a factored load of 300 kN at mid span.

Given data

Load = 360 kN

effective span (l) = 1.5 m

Maximum moment occurs at mid span is given by $M = \frac{wl}{4}$

$$M = \frac{300 \times 10^3 \times 1.5 \times 10^3}{4} = 335 \times 10^6 \text{ N-mm}$$

For the calculation of Z_p we need to put the formula that

is $M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}}$

For β_b we assume that the section is plastic

$\beta_b = 1$

$$\Rightarrow Z_p = \frac{M \times \gamma_{m0}}{\beta_b \times f_y}$$

$$= \frac{135 \times 10^6 \times 1.1}{1 \times 250} = 594 \times 10^3 \text{ mm}^2$$

Try ISMB 300 which has $Z_p = 651.74 \times 10^3 \text{ mm}^3$

The sectional property of ISMB 300 are

$h = 300$ mm

$b_f = 140$ mm

$t_f = 12.4$ mm

$t_w = 7.5$ mm

$Z_e = 573.6 \times 10^3 \text{ mm}^3$

Depth of web (d) = $h - 2 \times (t_f \times r_1)$

$= 300 - 2 \times (12.4 + 14.0)$

$= 247.2$ mm

self weight of beam = 44.2 kg/m

= 0.442 kN-m

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Factor weight $= 1.5 \times 0.442$

$w = 0.663$ kN-m

moment due to self weight $= \frac{wl^2}{8}$

$= \frac{0.663 \times 1.5^2}{8}$

$= 0.186$ kN-m

Total moment $= 135 + 0.186$

$= 135.186$

Factor shear force due to self weight $SF = \frac{wL}{2}$

$= \frac{0.663 \times 1.5}{2} = 0.497$ kN

Total shear force $= \frac{\omega}{2} + 0.497$

$= \frac{360}{2} + 0.497$

$= 180.497$ kN

section classification :-

Over hange $b = \frac{140}{2}$

Hence assumption which we are take is correct as per IS 800: 2007.

shear capacity of the section :- (Page no 59)

$V \le V_d$ so, $V_d = \frac{V_n}{\gamma_{m0}}$

$V_n = V_p$

$V_p = \frac{A_v f_{yw}}{\sqrt{3}}$

$A_v = h t_w$

$= 300 \times 7.5$

$V_p = \frac{300 \times 7.5 \times 250}{\sqrt{3}}$

$V_n = 324759.52$

$V_d = \frac{V_n}{\gamma_{m0}} = \frac{324759.52}{1.1}$

$= 295235.92$ N (Hence the condition $V \le V_d$ satisfied)

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From the relation $v \leq 0.6V_d$ and $v > 0.6V_d$ the condition $v > 0.6V_d$ is satisfied. for that reason we follow the formula according to the condition that is

$$M_d = M_{dv} \text{ (page no 70)}$$

$$M_{dv} = M_d - \beta(M_d - M_{fd}) \leq 1.2 \cdot Z_e f_y / \gamma_{m0}$$

$$\beta = \left(\frac{2v}{V_d} - 1\right)^2$$

$$M_{fd} = F_{cd} \times A$$

$$M_d = \frac{\beta_b \times Z_p f_y}{\gamma_{m0}}$$

$$= \frac{1 \times 651.74 \times 10^3 \times 250}{1.1}$$

$$= 1481227.27.3$$

$$\beta = \left(\frac{2v}{V_d} - 1\right)^2$$

$$= \left(\frac{2 \times 180.497 \times 10^3}{295235.92} - 1\right)^2$$

$$= 0.04$$

$$M_{fd} = F_{cd} \times A$$

$$\frac{KL}{r} = \frac{1.5 \times 10^3}{28.4} = \frac{1500}{28.4} = 52.81$$

$$\frac{h}{t} = \frac{300}{12.4} = 24.19$$

Chapter-6 Tubular section

Round Tubular section :-

Tubular section from the most efficient section from some of the structural element however in past few decades use was restricted for the following reason.

(i) It was difficult to have a smooth curve surface.
(ii) It was difficult to have ~~a smooth~~ cut the tube to a correct profile for marking joint between steel tubes and at angle connection were a problem.

The first two problem were overcome with the development of the oxy acetyle cutting machine which called the surface to the correct profile giving a bevelled and faciliate welding operation. connecting are now not a problem with the development with of welding technique.

(i) The economy of steel tube construction is an comparable The large span roof truss with tube section have smaller self weighed and many a time the supporting RCC.

(ii) column may even be replaced by mean column deriving considerable economy.

(iii) These are suitable for roofing system of industrial building and ware house and for large space,

(iv) Since tubular section subjected to wind and water current have a low drag coefficient they provide an ideal section for transmission line tower. and of drilling installation

(v) IS 1161 SP specified the use of following types of tubes for structural purposes.
(i) HFW (ii) HFS (iii) ERW

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Classification

steel tubes are classified as light medium and heavy depending upon the wall thickness on the basis of yield stress. these are classified as IS 1161, table-2 (refer clause - 3.1 and 11.2)

Grade	Tensile stress	Yield stress
Yst 210	330	210
Yst 240	410	240
Yst 310	450	310

Advantages and disadvantages :-

(i) Thes have a small self weighed also because of direct connection, gusests plates are eliminated further reducing dead load.

(ii) Tubes have uniform radius of gyration and for the same weighed they are tursonal spread is more than any other rolled section.

(iii) The dynamic load tubes have higher frequency of vibration than other rolled section.

(iv) Due to a smooth finish surface dirt and moisture donot collected over the surface reducing the possibility of corrosion.

(v) The change in load with the floor level can be econodated by varying the tube thickness and the external tube dimenssion may be maintain.

(vi) The only disadvantages of the tubular section is i/s high manufactoring costs.

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Minimum thickness :-

For tubes painted with one prime coat of red oxide and than painted periodecally the thickness should not less than than

SLNO	condition	Thickness (mm)
1.	For construction exposed to weather	4
2.	For construction not exposed to weather	3.2
3.	For member not readily accessible for maintenance	5
4.	For construction exposed to weather	3.2
5.	For construction not exposed to weather	2.6

Tubular compression and tension :-

1. In a tubular section the material is disposed off symmeterically about its centre of gravity. Because of uniform radius of gyration a tubular section is the most ideal section of compression members.

(ii) The diameter of the table should be as large as possible with the ratio d/t as small as possible to avoid local buckling

d = mean diameter of the tube

t = thickness of the table

(iii) The specification for effective length and slenderness ratio are same as that for other rolled section.

(iv) the effective length of truss member is same as that of condinous struts.

(v) The permissible stress is compression

(i) The design criteria for tension member is the net cross sectional area tubes cast more than the other rolled section and hence economical.

(ii) The design procedure is same as that for other rolled section

(iii) Permissible stresses for tube in tension are given in table.

Grade	Axial stress in tension MPa	Bending stress MPa	Minimum shear stress MPa	Minimum bearing stress MPa
Yst - 22	125	140	90	170
Yst - 25	150	165	110	190
Yst - 32	190	205	135	250

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Beam

The torsonal strength of tubular section is 200 to 300 times that of an open section. also their deformation is about 100 times smaller than that of open section of the same size.

(i) When used as beam, the risk of later instability is the least

(ii) The guideline for the design of beams and particle are same as for other rolled section.

Problem-1

A Tubular column consist of IS 1161 grade yst 32 of column is hinged at both the end the outside diameter of tube is 219.1 mm the weight of 1m length of tube is 310 N, the length of column is 4.5m. Determine the safe load carrying capacity of the column.

Solution:-

Given data-

d = 219.1 mm

length of the column = 4.5m

weight of 1m length = 310N

Grade = Yst.32

From table 1 we can take nominal bore = 200m ∴ As there is no given date about the thickness so we can assume that is the section is a heavy section.

$t = 5.9 mm$

So now area = $39.5 \times 10^2 mm$

So the length of the column = 4.5 m = 4.5×10^3 mm

Radius of gyration = 7.54 cm = 7.54×10 mm

For effective length of the column the condition which we have that is both side hinged so $L_{eff} = 1.0L$.

$= 1 \times 4.5 \times 10^3$

$= 4.5 \times 10^3$.

So Now $\lambda = \frac{L_{eff}}{r_{min}}$

$= \frac{4.5 \times 10^3}{75.4} = 59.68$

λ/r		F_c
x_1 50	————	153.9 y_1
x 59.68	————	y
x_2 60	————	146.8 y_2

$y = y_1 + (x - x_1) \times \left(\frac{y_2 - y_1}{x_2 - x_1}\right)$

$= 153.9 + (59.68 - 50) \times \left(\frac{146.8 - 153.9}{60 - 50}\right)$

$= 147.02$.

$P_d = A_e F_c$

$= 395 \times 10^2 \times 147.02$

$= 580729$ N

Problem - 2

A steel tubular column of 4.8 m length is hinged at both ends, it has nominal diameter of 225 mm and confirm to Yst 25 grade. Determine the safe load carrying capacity of the column.

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Given data

d = 225 mm

length of the column = 4.8 m

Grade = YST 25

From table 1 we can take the outside diameter = 244.5 mm

As there is no given data about the thickness so we can assume that is the section is a heavy section.

t = 5.9 mm

So now area = 44.2 cm^2 = 44.2×10^2 mm^2

Length of the column = 4.8 m = 4.8×10^3 mm

Radius of gyration = 8.44 cm

= 8.44×10

For effective length of the column the condition which we have that is both side hinged so leff = 1.0L

$= 1.0 \times 4.8 \times 10^3$

$= 4.8 \times 10^3$

$$\lambda = \frac{Leff}{r_{min}}$$

$$= \frac{4.8 \times 10^3}{84.4} = 56.87$$

l/r	Fe
x_1 50	153.9 y_1
x 56.87	y
x_2 60	146.8 y_2

$$y = y_1 + (x - x_1) \times \left(\frac{y_2 - y_1}{x_2 - x_1}\right)$$

$$= 153.9 + (56.87 - 50)\left(\frac{146.8 - 153.9}{60 - 50}\right)$$

$= 149.02$ N-mm^2

$P_d = A_e F_e$

$= 44.2 \times 10^2 \times 149.02 = 658668.4$ N.

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Design considerations for Masonarry ~ chapter-7

Masonary :-

An assembly of masonary units properly bonded together with moter.

Masonarry units :-

Individual unit which are bonded together with the help of moter to forma masonary element, such as wall column, Pier and butterss.

Wall :-

(i) It is defined as upright construction heaving length much greater than the thickness and presenting a continous surface except where pierced by doors window etc.

(ii) It is sed for shelter, protection or privacy or to subdivide interior space to support floors - roofs or the like to retain earth, to fence in an area etc.

Coloumn :-

An isolated verticle beam bearing member, width of which does not exceed four times the thickness.

Piers :-

A thickened section forming intigrated part of wall placed at intervals along the wall to increase the stiffness of the wall or to carry a verticle concentrated load.

Buttress :-

A pier of masonary built as an intigral part of wall and projecting from either or both surfaces, decreasing in cross sectional area from base to top.

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Types wall

Load bearing wall
(a) Solid wall
b) cavity wall
c) Faced wall
d) veneered wall

Non- load bearing wall
(a) Partition
(b) curtain wall
(c) Free standing wall
(d) veneered wall
(e) Parapet wall
(f) shear wall
(g) Buttress.

Load bearing wall :-
Load bearing wall support the weight of a floor or roof structure above and are so named because they can support a significant amount of weight.

Cavity wall:-
A wall constructed from two skins of masonary the outer skin of which be brickwork or block work and the inner skin of whiches generary of block work, separated by a ~~capacity~~ cavity to prevent the penetration of moisture and to allow for the installation of thermal insulation.

Veneered :-
A wall in which the facing attached to the backing but not so bonded as to resulting common action under load.

Solid wall:-
A wall constructed of one skin of masonary which can consist of brick or block and does not include a cavity between the interior and exterior.

Non load bearing wall:-
A non load bearing, some times called a partition wall is responsible only for holding up itself.

Partition wall :-

A non-load bearing wall that separates the internal space of a buiding

Parapet wall :-

the upper most reaches of a wall that extend above the roof level and provides a degree of production to roof, gutters balconies and walkways.

Curtain wall :-

A non structural cladding system for the external walls buildings.

Design consideration of walls and column :-

General :-

(i) Masonary structure gain stability from the support offered by cross walls floors, roof and other element such as piers and buttresses load bearing walls are structural more efficient when the load is uniformly distributed and the structure is so planned that eccentricity of loading on the member is as small as possible.

(ii) Avoidance of eccentric loading by providing adequate bearing of floor/roof on the walls providing adequate stiffness in slab and avoiding cavity and the supports etc. is specilly important in load bearing walls in multistorey structures. These matters should recieve carefull consideration during the planning stage of masonary structures.

Laterally supports and stability :-

(i) Laterally supported for masonarey element such as load bearing wall or coloumn are intended to.

(a) limit slenderness of a masonary element so as to Prevent or reduce possibility of bucksing of the member due to verticle loads.

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(b) Resist horizontal components of force so as to ensure stability of structure against over turning.

(ii) Lateral support may being the verticle or horizontal direction the former consisting of floor/roof bearing on the wall or properly anchored to the same and latter consisting of cross walls, piers or buttresses.

(iii) In case of wall, where slenderness ratio is based on effective height any of the following construction are provided.

- Rcc floor/roof slab irrespective of the direction of spans. Bars on the supported wall as well as cross walls to extent of at least 9cm.
- Rcc floor/roof slab not bearing on the supporting wall or cross wall is anchored to it with non corrodable metal ties of 60cm length and of section not less than 6x30mm and intervals no exceeding 2m.

Stability:-

1. A wall or column subjected to vertical and lateral loads may be considered to be provided with adequate lateral support from consideration of stability, if the construction providing the support in capable of resisting the following forces.

(a) Simply static reaction at the point of lateral support to all the lateral loads

(b) 2.5 pecent of the total vertical load that the wall or column is designed to carry at the point of lateral support.

(2) In case of load bearing building upto four storeys, stability requirement of may be deemed to have been met with it.

1) (a) Height to width ratio of building does not exceed 2

(b) Cross wall acting as stiffening walls continuous from outer wall or outer wall to a load bearing inner wall, and of thickness and spacing.

(c) Floors and roof either bear on cross wall.

External walls of basement and plinth stability requirements:-

1- Bricks used in basement and plinth have minimum crushing strength of 5 N/mm^2 and motar used in masonarey is of grade MI or better.

2- Clear height of ceilling in basement does not exceed 2.6 m

3- In the zone of action of soil pressure on basement walls traffic load including any surcharge due to adjoining building does not exceeds 8 kN-m^2 and terrian does not rise.

4- Minimum thickness of basement walls in an accordance.

Sl NO	Minimum thickness of basement wall (Normal) Normal	Height of the grade above basement floor level with wall loding (permanent load) More than 50 KN/m	Less than 50 kN-m
(1)	(2) cm	(3) m	(4)
1	40	2.50	2.00
2	30	1.75	1.40

Effective length of wall:-

Sl NO	condition of support	effective height
1	Lateral as well as rotational restraint top and bottom	0.75 H
2		