T.Y. Diploma : Sem. V

[CE/CR/CS/CV]

Design of Steel Structures

Prelim Question Paper Solution

Time: 4 Hrs.] **[Marks : 100**

Q.1(a) Attempt any THREE of the following : [12]

Q.1(a) (i) State objectives and factors to consider by a designer in designing steel structure. [4]

(A) Objectives of Structural Design are

1) Safety
2) Serviceability
3) Durability
4) Economy
5) Aesthetics

Factor to consider in designing steel Structure

1) Stability
2) Strength
3) Brithe Factor
4) Fire
5) Durability

Q.1(a) (ii) Explain the limit states of serviceability applicable to steel structure. [4]

(A) Limit State of serviceability :

Limit state of serviceability is related to the satisfactory performance of the structure at working load. There are four major types of service ability limit states applicable to steel structure are as follows.

a) Deflection
b) Durability
c) Vibrator
d) Fire resistance

Q.1(a) (iii) List the values of partial safety factors for material strength in case of resistance by-yield, buckling, ultimate stress and bolt connection. [4]

(A)

	Descriptions	Partial safety Factor
1	Resistance governed by yielding r_{mo}	1.10
2	Resistance of member to buckling r_{mo}	1.10
3	Resistance governed by ultimate stress r_{m1}	1.25
4	Bolted connection in friction and Bearing r_{mf} and r_{mb}	1.25 [shop and field fabrication]

Q.1(a) (iv) Explain what do you mean by shear lag. [4]

(A) Shear Log

While transferring the tensile force from gussel plate to tension member through one leg by bolts or welds, the connected leg of section (such as angle, channel) may be subjected to more stress than the outstanding leg and finally the stress distribution becomes uniform over the section away from the connection. Thus one port behind the other is called as shear lag.

The tearing strength of an angel section connected through one leg is affected by shear lag also. Thus, the design strength, Td_n governed by tearing at net section is given by

$$Td_n = 0.9\frac{Anc\,fu}{vm_j} + \beta\frac{Ago\,fy}{vm_o}$$

Where β = 1.4 = $0.076\frac{w}{t}\times\frac{fy}{fu}\times\frac{bs}{Lc}$

b_3 = Shear log width as shown in fig

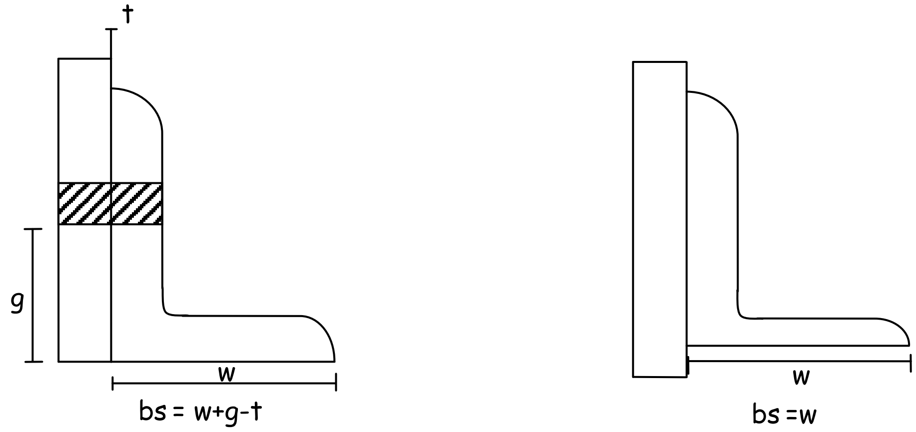


Fig: Shear leg width

Q.1(b) Attempt any ONE of the following : [6]

Q.1(b) (i) Design the Lap joint for the plates of sizes 100 × 16 mm and 100 × 10 mm thick connected so as to transmit a factored load of 100 kN using single row of 16 mm diameter bolts of grad 4.6 and plate of 410 grade. [6]

(A) Given

f_u = 410 N/mm^2 f_{ub} = 400 N/mm^2

d = 16 mm do = 18mm

v_{mb} = 1.25 P_u = 100 kN

Strength of bolt:

Since it is lap joint bolt is in single shear, the critical section being at the root of bolt.

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

$$= 0.78\times\frac{\pi}{4}\times 16^2 = 156.82$$

Design

strength of bolt in shear

i.e. $Vdsb = \frac{Fub}{\sqrt{3}}\frac{(n_n\,Anb + n_s\,A_{sb})}{rmb}$

$$= \frac{400}{\sqrt{3}}\frac{1\times157}{1.25} = 29.006\times10^3\text{ N}$$

$\therefore$ Vdsh = 29 N

$\therefore$ No. of bolts required = $\frac{Pu}{Vdsb} = \frac{100}{29}$

= 3.4 $\cong$ 4 No.

No. of bolts required = 4 no.

Arranging bolts in single rows

Equating tensile capacity per pitch length

$$Tdn = 0.9\frac{F_u}{rm_1}(P - do)\cdot t$$

$$29 \times 10^3 = 0.9 \times \frac{410}{125}(P - 18) \times 10$$

$$P = \left(\frac{29 \times 10^3 \times 125}{0.9 \times 410 \times 10}\right) + 18$$

$$= 27.82 < 2.5 \times d = 2.5 \times 16$$

$$= 40$$

$\because$ Provide pitch P = 40 mm
and edge distance = 17 × do [for rough edge]
= 17 × 18
= 30.6 ≅ 30

kb is smallest of

i) $\frac{e}{3do} = \frac{30}{3 \times 18}$ =0.56

ii) $\frac{P}{3d_0} - 0.25 = \frac{40}{3 \times 18} - 0.25$
= 0.49

iii) $\frac{Fub}{Fu} = \frac{400}{410}$ = 0975

(iv) 1

Min. Value = 0.49

Hence Kb = 0.49
$\therefore$ Design bearing strength

$$Vdsh = \frac{Vnpb}{rmb} = \frac{2.5 \times kb.d.t\ F_u}{rmb}$$

$$= \frac{2.5 \times 0.49 \times 16 \times 10 \times 410}{1.25}$$

= 64288 N = 64.29 kN

Vdsb = 64.29 kN > 29 kN
$\because$ Ok no revision is required

Check for the strength of plate

$$Tdn = \frac{0.9\,An \cdot Fu}{rm} = \frac{0.9 \times (100 - 2 \times 18) \times 10 \times 410}{1.25}$$

= 188.93 kN > 110kN
safe

Provide 4–16 mm ϕ bolts of 40mm
Pitch with edge distance of 30 mm as shown in fig.

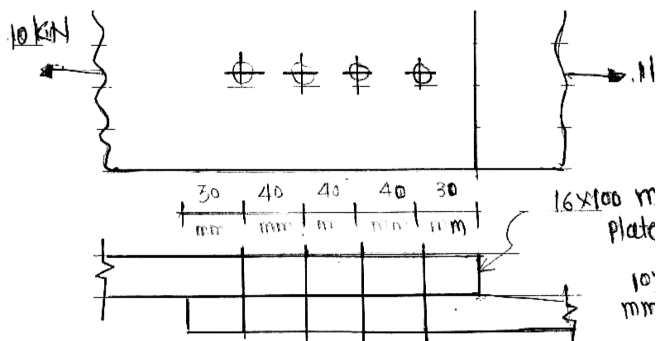
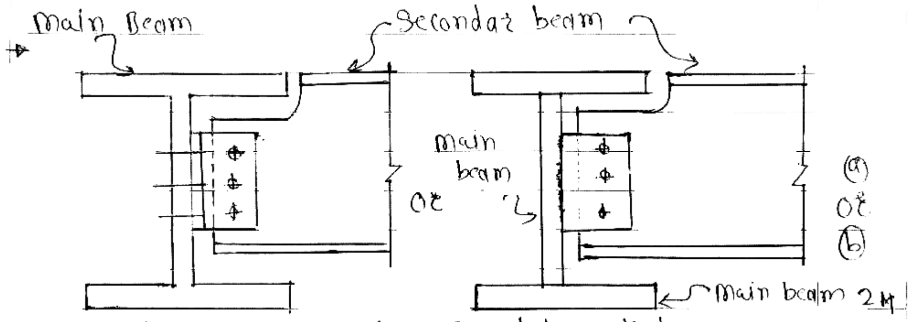


Fig: showing connection of Lap joint

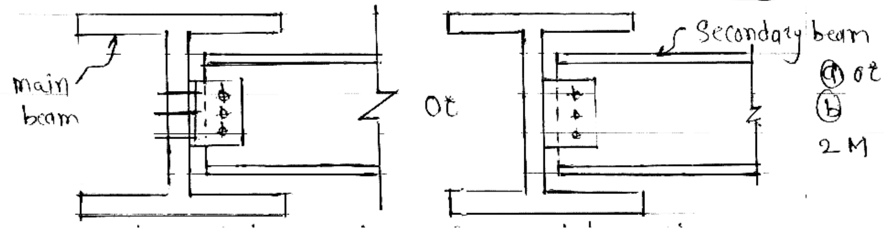
Q.1(b) (ii) Draw neat sketches of bolted connections in case of : **[6]**
(1) Beam to Beam connection when flanges are at same level.
(2) Beam to Beam connection when flanges are not at same level.
(3) Beam to column connection.

(A)



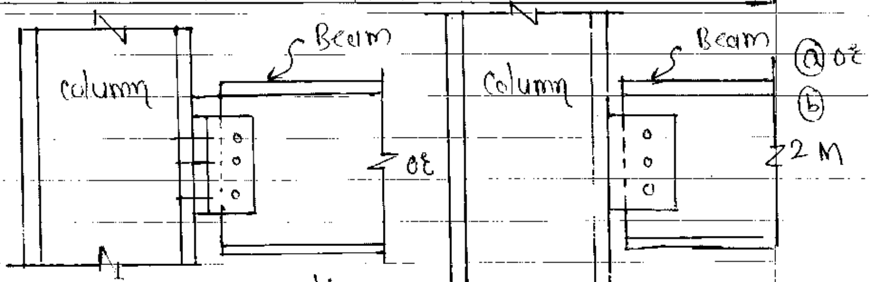
(a) Bolted – Bolted Connection using cleat angle (b) Welded – Bolted Connection Using plate

Fig : Beam to Beam connection when flanges at Same level



(a) Bolted - Bolted Connection using cleat angle (b) Welded - Bolted connection using plate

Fig : Beam to Beam connection when flanges are not at same level



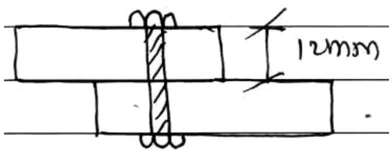
(a) Bolted - Bolted connection using cleat angle (b) Welded - Bolted connection using plate

Fig : Beam to column connection

Q.2 Attempt any TWO of the following : **[16]**

Q.2(a) A lap joint consists of two plates 200 × 12 mm connected by means of 20 mm diameter bolts of grade 4.6. All bolts are in one line. Calculate strength of single bolt and no. of bolts to be provided in the joint. **[4]**

(A)



Nominal diameter of bolt = 20 mm

$\therefore$ Net area of bolt at thread (A_{nb}) $= 0.78 \times \frac{\pi}{4} \times d^2$

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

$A_{nb} = 245.04\ mm^2$

For fe 410 grade steel plate (assumed)
Ultimate stress for plate $f_y = 410\ N/mm^2$
For 4.6 grade of bolt
Ultimate stress for bolt (f_{ub}) $= 4 \times 100 = 400\ N/mm^2$
Yield stress for bolt (f_{yb}) $= 400 \times 0.6 = 240\ N/mm^2$

Now find design shearing strength of bolt (Vdsb)
$\therefore$ we know that

$$\therefore \quad Vdsb = \frac{fub}{\sqrt{3} \times Y_{mb}} \, [n_n \times A_{nb} + n_s + A_{ns}]$$

Here number of shear plane with threat intercepting the shear plane $n_n = 1$
Number of shear plane without thread intercepting the shear plane $n_s = 0$

$$\therefore \quad V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times [1 \times 243.04 + 0]$$

γ_{mb} = partial factor of safety for bolt material = 1.25
$V_{dsb} = 45.27 \times 10^3 N$

Now find design bearing strength of bolt (V_{dsb})

$$V_{dph} = 25 \times kb \times (d \times t) \times \frac{fy}{\gamma_{mb}}$$

Here coefficient k_b is minimum of

(1) $\left[\frac{e}{3dh}, \frac{p}{3dh} - 0.25, \frac{f_{ub}}{f_u}, 1\right]$

(a) Diameter of hole (dh) = Nominal diameter + 2
$= 20 + 2 = 22$ mm

(b) End distance (e) = 2d = $2 \times 20 = 40$ mm

(c) Pitch (p) = 2.5 d
$= 2.5 \times 20 = 50$ mm

(i) $\frac{e}{3dh} = \frac{40}{3 \times 22} = 0.606$

(ii) $\frac{p}{3dh} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$

(iii) $\frac{fub}{fy} = \frac{400}{410} = 0.975$ &

(iv) 1

Hence Kb = 0.507 mm … take minimum value

Now find design bearing strength of bolt (V_{dpb})

$$= 2.5 \times Kb \times (d \times t) \times \frac{f_y}{r_{mb}}$$

$$= 2.5 \times 0.507 \times (20 \times 12) \times \frac{410}{1.25}$$

$V_{dph} = 99.77 \times 10^3$ N

Now find bolt value i.e. strength of bolt

$\therefore$ Bolt value = minimum strength between shearing & bearing strength of bolt i.e. minimum between V_{dsb} & V_{dpb}

$= 45.27 \times 10^3$ N

Full strength of member $= 0.9 \times \dfrac{fu}{r_m} \times$ Area of plan

$= \dfrac{0.9 \times 410}{1.25}(250 - 1 \times 22) \times 12$

$= 630.54 \times 10^3$ N

Full strength of plan

$\therefore$ No of bolts $= \dfrac{\text{full strength of plate}}{\text{Bolt value}}$

$= \dfrac{630.54 \times 10^3}{45.27 \times 10^3}$

= 13.92 Say 14 Nos

Q.2(b) A discontinuous compression member consists of 2 ISA 90 × 90 × 10 mm connected back to back on opposite sides of 12 mm thick gusset plate and connected by welding. The length of strut is 3 m. It is welded on either side. Calculate design compressive strength of strut. **[4]**

For ISA 90 × 90 × 10, $C_{xx} = C_{yy} = 25.9$ mm $I_{xx} = I_{yy} = 126.7 \times 10^4$ mm^4, $r_{zz} = 27.3$ mm values of fed are

KL/r	**90**	**100**	**110**	**120**
fed (N/mm^2)	**121**	**107**	**94.6**	**83.7**

(A) (i) $r_{zz} = 27.3$ mm (Due to symmetry @ zz axis)

(ii) $I_{yy} = 2[Iy + A \cdot h^2]$

$= 2[126.7 \times 10^4 + 1703\,(25.9 + 12/2)^2]$

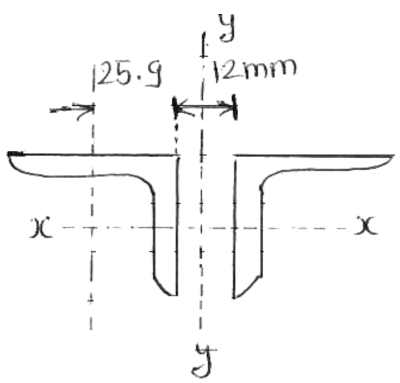
(A is calculated by calculating Area of both leg separately and then adding them)

$\therefore I_{yy} = 5999979$ mm^4

(iii) $\therefore r_{yy} = \sqrt{\dfrac{I_{yy}}{A_g}} = \sqrt{\dfrac{5999979}{2 \times 1703}} = 41.97$ mm

r_{min} = minimum of r_{zz} and r_{yy}

$r_{min} = 27.3$ mm



(iv) For discontinuous double angle, effective length

$KL = 0.85L = 0.85 \times 3 = 2.10$ m = 2100mm

$S.R. = \dfrac{KL}{r_{min}} = \dfrac{2100}{27.3} = 76.92$

KL/r (SR)	fed
70	152
80	136

Hence,

$$fcd = fcd_1 - \frac{fcd_1 - fcd_2}{SR_2 - SR_1}$$

$$fcd = fs_2 - \frac{152-136}{80-70}(76.92-70)$$

$fcd = 140.928\ N/mm^2$

(v) Design compressive Strength

$Pd = fcd \times Ag$

$Pd = 140.928 \times (2 \times 1703)$

$Pd = 480 \times 10^3\ N$

$Pd = 480\ kN$

Q.2(c) Check whether ISMB250@37.4 kg/m is suitable or not as a simply supported beam over an effective span of 6 m. The compression flange of beam is laterally supported throughout the span. It carries udl of 15 kN/m (including selfwt.). Properties of ISMB 250 are b_f = 125 mm, t_f = 12.5 mm, t_w = 6.9 mm, $I_{xx} = 5131.6\times10^4\ mm^4$, $Z_{xx} = 410\times10^3\ mm^3$, r_1 = 13.0 mm, $z_{px} = 465.71 \times 10^3 mm^3$, γ_{mo} = 1.1, β_b = 1 and f_y = 250 MPa. **[4]**

(A) (i) Loads and factored BMS

$w = 15kN/m$

Factored udl, $wd = 15 \times 1.5 = 22.5\ kN/m$

Factored BM, $Md = \dfrac{wd.le^2}{8} = \dfrac{22.5 \times 6^2}{8}$

$= 101.25\ kN/m$

Factored S.F. $Vd = \dfrac{wd.le}{2} = \dfrac{22.5 \times 6}{2} = 67.5kN$

(ii) Plastic modulus of section required

Z_p reqd. $= \dfrac{Md.\gamma_{mo}}{fy} = \dfrac{101.25 \times 10^6 \times 1.1}{250}$

$= 445.5 \times 10^3\ mm^3$

Z_p reqd. $<$ Z_P avil $(=465.71 \times 10^3\ mm^3)$

(iii) Classification of beam section

$d = h - 2(f_t + \gamma_1) = 250 - 2(12.5 + 13)$

$= 199\ mm$

$$\frac{bh}{tf} = \frac{\frac{125}{2}}{12.5} = 5.0 < 9.4$$

$$\frac{d}{tw} = \frac{199}{6.9} = 28.84 \quad < 67$$

As $\dfrac{bh}{tf} < 9.4$ and $\dfrac{d}{tw} < 67$ $\quad\therefore$ Section classification is plastic

(iv) Check for shear

$$Vdr = \frac{f_y \times tw \times h}{\gamma_{mo}\sqrt{3}} \quad OR \quad 0.525\ \ fy.tw.h$$

$$= \frac{250 \times 6.9 \times 250}{1.1 \times \sqrt{3}} = 226348 \text{ N}$$

$= 226.35$ KN $>$ Vd (=67.5kN)

Also, $\frac{Vd}{Vdr} = \frac{67.5}{226.35} = 0.298 < 0.6$

$\therefore$ Check for shear is satisfied.

(v) Check for deflection

$$\delta_{allowable} = \frac{L}{300}$$

$$= \frac{6000}{300}$$

$= 20$ mm

$$d_{max} = \frac{5}{384} \frac{wL^4}{FI}$$

$$= \frac{5}{384} \times \frac{15 \times 6000^4}{2 \times 10^5 \times 5131.6 \times 10^4}$$

As $\delta_{max} > \delta_{allowable}$

$\therefore$ Deflection check is not O.K.

Hence, ISMB 250 is not a suitable section for given loading and span

Q.3 Attempt any FOUR of the following : [16]

Q.3(a) State types of bolted joints and types of failure in case of bolted joints. [4]

(A) i) Types of bolted joints

(a) Lap Joint
- Single line bolting
- Double line bolting

(b) Butt Joint
- Single cover Butt joint
- Double cover Butt joint

ii) Failure of Bolted joint

(a) Failure of plate
- By tearing of plate (shear failure)
- By tensile failure of plate
- By bearing of plate

(b) Failure of bolt
- By shear failure of bolt
- By tensile failure of bolt
- By bearing failure of bolt

Q.3(b) State two advantages of welded joints and two disadvantages of bolted joints. [4]

(A) Advantages of Welded Joints

1) The welded structures are usually lighter than riveted structures. This is due to the reason, that in welding, gussets or other connecting components are not used.
2) The welded joints provide maximum efficiency (may be 100%) which is not possible in case of riveted joint.
3) Alterations and additions can be easily made in the existing structures.
4) As the welded structure is smooth in appearance, therefore it looks pleasing.

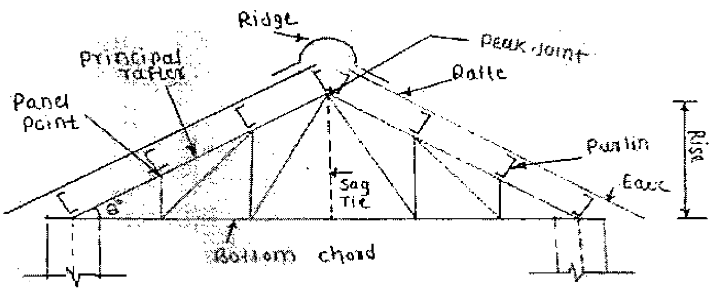
5) In welded connections, the tension members are not weakened as in the case of riveted joints.
6) A welded joint has a great strength. Often a welded joint has the strength of the parent metal itself.
7) Sometimes, the members are of such a shape (i.e. circular steel pipes) that they afford difficulty for riveting. But they can be easily welded.
8) The welding provides very rigid joints. This is in line with the modern trend of providing rigid frames.
9) It is possible to weld any part of a structure at any point. But riveting requires enough clearance.
10) The process of welding takes less time than the riveting.

Disadvantages of bolted joints :

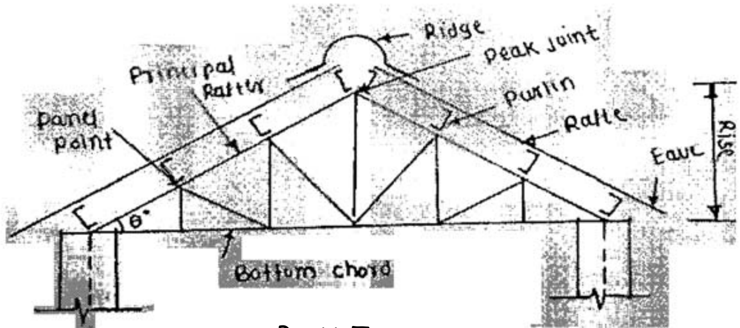
1) Due to holes made in members to be connected, tensile strength of the members is reduced.
2) Rigidity of joint is affected due to loose fit.
3) Deflection may increase due to affected Rigidity of joint
4) Nuts are likely to loose due to moving load vibration.
5) Bolted structures are heavier than welded structure due to use of connecting angles.
6) Circular section can not be bolted.
7) It is not possible to get 100% efficiency in case of bolted connection
8) Problem may arise in case of mismatching of holes.

Q.3(c) Draw sketches of Howe type and Pratt type truss showing pitch, rise, panel point, panel, principal rafters and all members in one of the above types. **[4]**

(A)



Home Truss



Pratt Truss

Q.3(d) State different types of loads and its combination considered during design of roof truss. Explain in brief any one of them along with its relevant IS Code. **[4]**

(A) Types of loads

- Dead Load
- Imposed load or live load
- Snow load
- Wind load
- Earth quack load

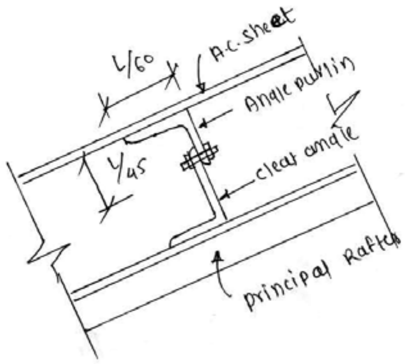
Load Combinations

The following combination of loads with appropriate Partial safety factors may be considered.

a) Dead load + imposed load
b) Dead load + imposed load + wind or earthquake load
c) Dead load + wind or earthquake load
d) Dead load + erection load.

Q.3(e) Draw a neat sketch and label of an angle Purlin with principal rafter at Panel Point having root covering is A.C. sheets. **[4]**

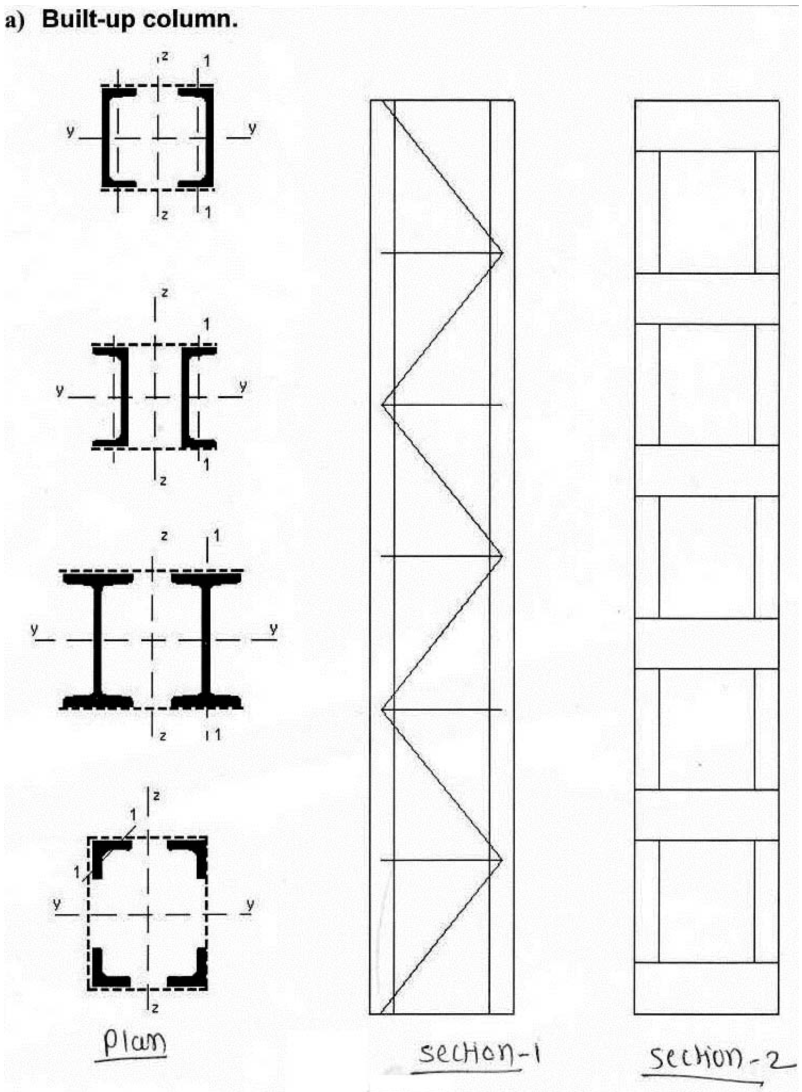
(A)



Q.4(a) Attempt any THREE of the following : **[12]**

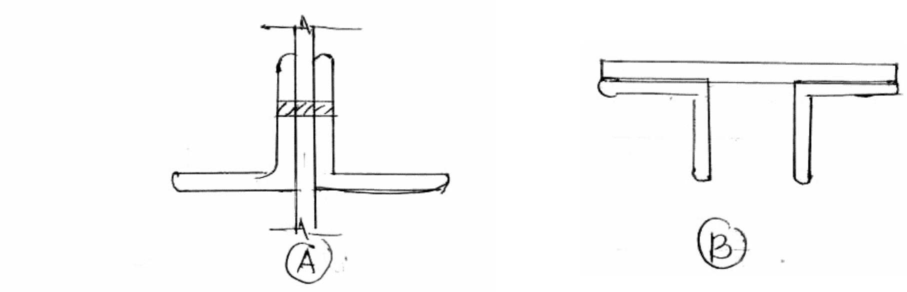
Q.4(a) (i) Sketch different sections used as built-up strut and built-up column. **[4]**

(A)



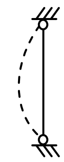
Built-up columns

b) built up strut

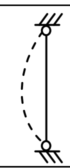


Q.4(a) (ii) State with a sketch the effective length for a compression member as per IS 800 – 2007 having end conditions as **[4]**
(1) Translation restrained at both ends and rotation free at both ends.
(2) Translation and rotation restrained at both ends.

(A) (i) Translation restrained at both ends and rotation free at both ends

Restrained	Free	Restrained	Free		1.0L

(ii) Translation and rotation restrained at both ends

Restrained	Restrained	Restrained	Restrained		0.65 L

Q.4(a) (iii) State the function of lacing and battening. **[4]**

(A) 1. **Function of lacing**
- To connect the different components of built up column together so that they will act as one unit
- To keep the distance between two components of built up column uniform and constant.
- To keep the distance between two components of built up column uniform and constant.

2. **Function of battening**
- The batten is placed opposite to each other at each end of the member and at points where the member is proportioned uniform throughout.
- When battens are used effective length of column should be increased by 10%
- Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force equal to 2.5 percent of the total axial force on the whole compression member, at any point in the length of the member, divided equally between parallel planes of battens.

Q.4(a) (iv) Limiting width to thickness ratio for single beam section of plastic class is 9.4 and d/tw = 84. State whether ISMB 500 @ 852 N/m is of plastic class or not. For ISMB 500 h = 500 mm, b_f = 180 mm, t_f = 17.2 mm, t_w = 10.2 mm, r_1 = 17.0 mm, f_y = 250 MPa. **[4]**

(A) $\frac{h}{bf} < 8.4\,\varepsilon$ ... For class –1 (plastic)

Given section is ISMB500

$\therefore$ h = 500

& bf = 180

$\frac{h}{bf} = \frac{500}{180} = 2.78 < 8.4\ \varepsilon$

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

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

$\varepsilon = 1$

$\therefore\ \frac{h}{bf} = 2.78 < 8.4 \times 1$

$= 2.78 < 8.4$ ... hence the class is plastic

Q.4(b) Attempt any ONE of the following : [6]

Q.4(b) (i) Find the value of permissible stress in axial tension (6 at) for f_y = 250 MPa. State why unequal angles with long legs connected are more efficient? [6]

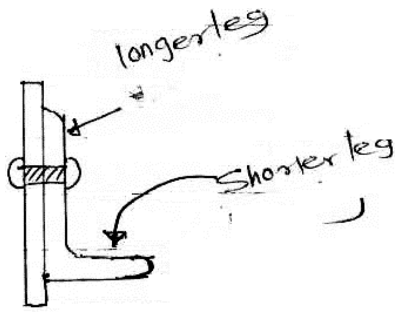
(A) (i) $6at = 0.60 \times fy$

$= 0.60 \times 250$

$6at = 150\ N/mm^2$

(ii) Generally longer legs are connected in case of unequal angle section because of the following reason.

Consider angle is connected in the following manner as shown in fig



In the shorter leg is connected to gusset plate, then the bending stress induced in the section is large due to outstanding longer kg, because of which the stress distribution in the section is no–uniform and hence it may lead to fracture of the member prematurity

Q.4(b) (ii) Design a tension member consisting of single unequal angle section to carry a tensile load of 340 kN. Assume single row 20 mm bolted connection. The length of member is 2.4 m. [6]

Take – f_u = 410 MPa, α = 0.80

Section available (mm)	Area (mm^2)
ISA 100 × 75 × 8	1336
ISA 125 × 75 × 8	1538
ISA 150 × 75 × 8	1748

(A) Appropriate gross area required

Reqd $Ag = \frac{1.1 \times Tag}{fy}$

$= \frac{1.1 \times 340 \times 10^3}{250}$

$= 1496\ mm^2$

Try 15A 125 × 75 × 8 mm giving $Ag = 1538\ mm^2$ $r_{min} = 16.1$ mm. Assuming longer leg connected, check the strength of the section

i) Design strength due to yielding of gross section

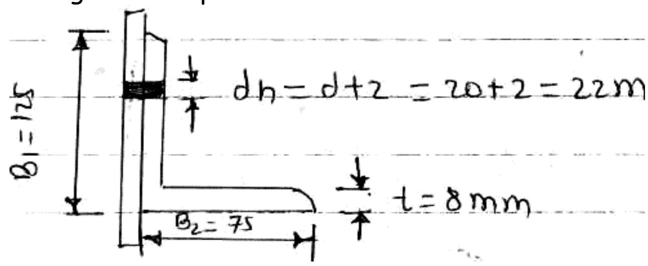
$$Tag = \frac{Ag \times fy}{r_{mo}}$$

$$= \frac{1538 \times 250}{1.10}$$

$$= 349545.4N$$

Tag = 349.54 kN

ii) Design strength due rupture of critical section



$$Tdn = \alpha An \frac{lu}{r_{m1}}$$

$$An = Anc + Ago$$

$$Anc = (B_1 - d_n - t/2) \times t$$

$$= (125 - 22 - 8/2) \times 8$$

Anc = 792 mm^2

$$Ago = (B_2 - t/2)t$$

$$= (75 - 8/2)8$$

$$= 568mm^2$$

An = Anc + Ago

An = 792 + 568

An = 13602 mm^2

Considering more than four bolt's in a raw α = 0.8

$$Tdn = \frac{0.8 \times 1360 \times 410}{1.25}$$

Tdn = 356.864 kN

Design of bolts

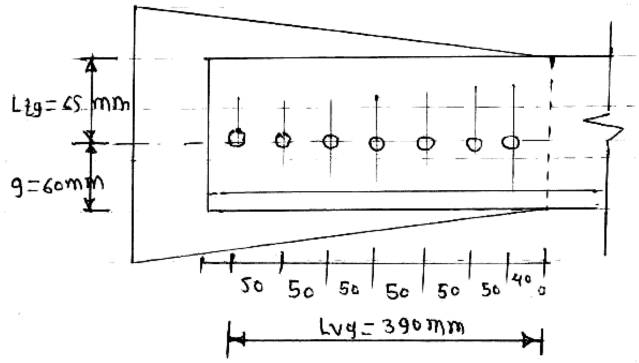
Capacity of bolts in single shear = 45. 3 KN

Capacity of bolt in bearing = $20 \times 8 \times 410 \times 10^{-3}$

= 65.6 kN

least bolt value = 45.3 kN (min of two above)

Number of bolts required = $\frac{340}{45.3}$ = 75 say 8



Assuming edge dist^{n} = 40mm

9 = 60mm

Spacing of bolts = 50mm

$Avg = Lvg \times t = 390 \times 8 = 3120mm^2$

$Avg = 3120\ mm^2$

$Avn = \{Lug - [No.\ of\ bolts - 0.5\ dh]\} xt$

$Avn = \{390 - [(8 - 0.5)22]\} \times 8 = 1800mm^2$

$Avn = 1800mm^2$

$Atg = Ltg \times t = 65 \times 8 = 500\ mm^2$

$Atg = 520\ mm^2$

$Atn = (65 - (0.5 \times 22)) \times 8$

$Atn = 432\ mm^2$

$Tdb_1 = Avg\,fy / (\sqrt{3} \times \gamma_{mc}) + 0.9Atn\,fu / \gamma_{m1}$

$= 3120 \times 250 / (\sqrt{3} \times 1.10) + 0.9 \times 432 \times \dfrac{410}{1.25}$

= 409393.8 + 127526.4

= 5369202 N

Tdb_1 = 536.92 kN

Tdb_2 = 424.962 kN

Tdb = lesser than Tdb_1 and Tdb_2 = 424.96 kN

$\therefore$ The tensile strength of angle = lesser of Tag, Tdn and Tdb

(349.54, 356.86 and 426.96)

= 349.54 kN

This is greater than required 340 kN

Check for slenderness ratio $\lambda = \dfrac{L}{r_{min}} = \dfrac{2400}{16.1}$

$149.06 < 250$

Q.5 Attempt any TWO of the following : [16]

Q.5(a) A hall of size 12m × 18m is provided with Fink type trusses at 3 m c/c. Calculate panel point load in case of Dead load and live load from following data. [8]

(i) Unit weight of roofing = 150 N/m^2
(ii) Self weight of purlin = 220 N/m^2
(iii) Weight of bracing = 80 N/m^2
(iv) Rise to span ratio = 1/5
(v) No. of panels = 6

(A) Span of truss = 12 m
Spacing = 3m /c/c
Types of truss = sink
No. of panel point = 6

Rise = $\frac{\text{span}}{5}$

$= \frac{12}{5} = 24\text{m}$

$\theta = \tan^{-1}\left(\frac{\text{Risc}}{L/2}\right) = \tan^{-1}\left(\frac{2.4}{6}\right) = 21.80^\circ$

Calculation of dead load
(i) Weight of roofing = 150 N/m^2
(ii) Weight of Purlin = 220 N/m^2
(iii) Weight of truss = $\left(\frac{L}{3}+5\right)\times 10$
= 90 N/m^2
(iv) Weight of bracing = 80 N/m^2

Total dead load = 540 N/m^2
Total dead load on one truss = 540 N/m^2
= 540 × 12 × 3
= 19.44 kN

Dead load on each panel point = $\frac{19.44}{2}$ =3.2H kN

D on end panel point = $\frac{324}{2}$
= 1.62 kN

Live load calculation
L.L. on purlin = 750 − (0 − 10) × 20)
= 750 − [2180 − 10) × 20]
= 514 N/m^2 > 400 N/m^2
L.L of truss = 2/3 × 514 = 342.67 N/m^2
∴ Total L. L. = L.L. of truss × span × spacing
= 342.67 × 12 × 3
= 12336 N

L.L. m each panel = $\frac{12336}{6}$ = 2056 N

L.L. m end panel = $\frac{2056}{2}$ = 1028 N

Q.5(b) An industrial building has trusses for 14 m span. Trusses are spaced at 4m c/c and rise of truss in 3.6m. Calculate panel point load in case of live load and wind load using following data : **[8]**

(i) Coefficient of external wind pressure = – 0.7
(ii) Coefficient of internal wind pressure = ± 0.2
(iii) Design wind pressure = 1.5 kPa
(iv) Number of panels = 08

(A) Span of trus = 14m
Spacing of truss = 3.6 m
No. of panels = 8

Design wind pressure = 1.5 kpa
$= 1.5 \times 10^3$ N/m^2

$$\theta = \tan^{-1}\left(\frac{\text{Rise}}{\text{Span}/2}\right) = \frac{3.6}{14/2} = 27.22°$$

$\therefore\ \theta = 27.22°$

Wind load calculation
Coefficient of external wind pressure
Cpe = -0.7

Coefficient of internal wind pressure
Cpi = ±02

Total wind press = [Cpe – Cpi] × P_2

Wind load combination
i) w.c = [–0.7 – (0.2)] × 1500 = 750 N/ m^2
ii) w.c = [–0.7 – (+0.2)] × 1500 = 1350 N/m^2

Max. intensity = –1350 N/m^2

Length of principle dafter = $\dfrac{L/2}{\cos\theta}$
= [1412]
cos 27.22

Length of principle dafter = 7.87 m
$\because$ Sloping area = 2 × 7.87 × 4
= 62.96 m^2
$\because$ Total wind load = Max. intensity × sloping area
= 1350 × 62.96
= 84996 N

Wind
$\because$ load an each panel = $\dfrac{84996}{8}$
$\therefore$ wind load on end panel = -10624.5 N
$\therefore$ wind load on end panel = $\dfrac{-10624.5}{2}$
= 5312.25 N

Live load calculation
Live load on purlin = 750 - [(θ - 10) × 20]
= 750 – [(27.22 – 10) × 20]
= 405.6 × 4v N/m^2
Hence ok

L.L. on truss

$= 2/3 \times 405.6$

$= 270.4 \ N/m^2$

$\because$ Total L.L $=$ L.L. intensity $\times$ Span $\times$ spacing

$= 270.4 \times 14 \times 4$

$= 15142.4$ N

$\because$ load on each panel $= \dfrac{\text{T.L}}{\text{No. of Panel}}$

$= \dfrac{15142.4}{8}$

$= 1892.8$ N $= 1.892$ kN

and load on end panel $= \dfrac{1892.8}{2}$

$= 946.4$ N $= 0.926$ kN

Q.5(c) Design a slab base for column ISHB 400 @ 82.2 kg/m to carry factored axial compressive load of 2000 kN. The base rests on concrete pedestal of grade M_{20}. For ISHB 400, b_f = 250 mm, f_y = 250 MPa, f_u = 410 MPa, γ_{mo} = 1.1, t_f = 12.7 mm. **[8]**

(A) Given

Factored load pu $= 2000$ kN

$= 2000 \times 10^3$ N

Fck = 20

D = 400

B = 250 i.e bf

$v_{mo} = 1.1$

tf = 12.7

fy = 250 N/mm^2

Bearing Strength of conc

= 0.6 fek

$= 0.6 \times 20 = 12 \ N/m^2m$

Bearing area of base plate

$$A = \frac{Pu}{\text{Bearing strength of conc}}$$

$$A = \frac{2000 \times 10^3}{12} = 166.67 \times 16^3$$

Size of base plate

length of plate

$$Lp = \frac{D-B}{2} + \sqrt{\left(\frac{D-B}{2}\right)^2 + A}$$

$$= \frac{400-250}{2} + \sqrt{\left(\frac{400-250}{2}\right)^2 + 1666.67 \times 10^3}$$

$= 490.08 \cong 500$

$$Bp = \frac{A}{Lp} = \frac{166.67 \times 10^3}{500} = 333.34 \cong 350$$

Larger Projection

$$a = \left(\frac{Lp - p}{2}\right) = \frac{500 - 400}{2} = 50 \text{ mm}$$

Smaller Projection

$$b = \left(\frac{Bp - B}{2}\right) = \frac{350 - 250}{2} = 50 \text{ mm}$$

Area of base plate

$$Ap = 500 \times 350 = 175 \times 10^3$$

Ultimate Pressure from below m the
Slab base

$$w = \frac{Pu}{A} = \frac{2000 \times 10^3}{175 \times 10^3} = 11.42 \text{ N/mm}^2$$

Thickness of slab base

$$f_s = \sqrt{\frac{2.5\,w\left(a^2 - 0.3b^2\right)vm_0}{fy}}$$

$$= \sqrt{\frac{2.5 \times 11.42\left(50^2 - 0.3 \times 50^2\right) \times 1.10}{250}}$$

$= 14.82$ mm $>$ tf l.e. k – 7

$\cong 15$ mm

Hence provide slob base plate having dimension
$500 \times 350 \times 15$

Q.6 Attempt any FOUR of the following : [16]

Q.6(a) Write steps to calculate the thickness of base plate used in slab base. Why anchor bolts are used in slab base. [4]

(A) Design steps to find thickness

1) To calculate area (A) of base plate
   A = Column load/Bearing strength
   Bearing strength of concrete = $0.6\ f_{ck}$
2) Select the size of base plate.
   L_p & B_p be the sizes of plate
   D = length or longer length
   B = width or shorter side of the column

   Consider square plate

$$L_P = \frac{(D - B)}{2} + \sqrt{\left[\left\{\frac{(D - B)}{2}\right\}^2 + A\right]}$$

$$B_P = \frac{A}{L_P}$$

Large projection $a = \frac{(L_P - D)}{2}$

Shorter projection $b = \frac{(B_P - B)}{2}$

Area of base plate provided = $L_P \times B_P = (D + 2a) \times (B + 2b)$

3) Calculate ultimate bearing pressure

$$w = \frac{P}{(L_p \times B_p)} \qquad \text{[1 mark]}$$

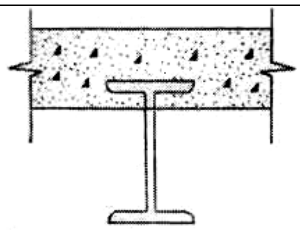
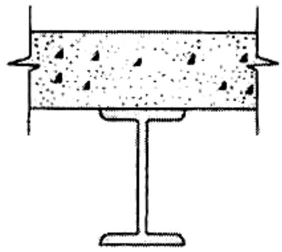
4) Calculate thickness of base plate

$$t_s = \left[\left(\frac{2.5 \times w(a^2 - 0.3 \times b^2)r_{mo}}{f_y}\right)\right]^{0.5} \qquad \text{[1 mark]}$$

Function of anchor bolt : To connect concrete pedestal and base plate anchor bolts are used.

Q.6(b) Differentiate between Laterally supported and unsupported beams with a neat sketch. **[4]**

(A)

	Laterally supported beam	Laterally unsupported beam
1)	In laterally supported beam, compression flanges are embedded in concrete.	In laterally unsupported beam, compression flanges are not embedded in concrete.
2)	Compression flange of Beam is restrained against rotation	Compression flange of Beam is free for rotation.
3)	Lateral deflection of compression flange is not occur.	Lateral deflection of compression flange is occur.
4)	 Laterally supported.(it means compression flange is restrained)	 Laterally unsupported.

Q.6(c) Define Gusseted base. Also draw its labelled sketch showing all details. **[4]**

(A) Definition

When the load on column is large or column subjected to moment along with axial load, base is provided called gusseted base.

It consists of base plate, gusset angle, connecting angle on either side of column.

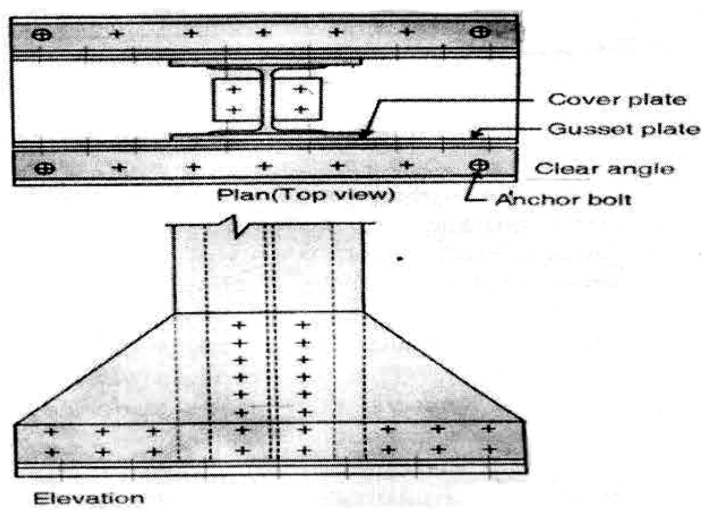


Fig. : Gusseted Base

Q.6(d) How beam sections are classified for bending as per IS : 800 – 2007. Describe any two of them. [4]

(A) Classification beam:

1) Plaster or class – I
2) Compact or class – II
3) Semi compact or class – III
4) Slender or class – IV

Explain in detail

1) Plastic or class – I

Cross section which can develop plastic hinge, sustain large rotation capacity required to develop plastic mechanism are called as plastic section. These sections are unaffected by local buckling and are able to develop their full plastic moment capacities until a collapse mechanism is formed.

2) Compact or class – II

In compact section, the full cross section forms first plastic hinge but local buckling prevents subsequent moment redistribution. These sections develop full plastic moment capacities M_P but fails by local buckling due to inadequate plastic hinge rotation capacity.

3) Semi compact or class – III

In semi plastic section the extreme fibres reach the yield stress but local buckling prevents the development of plastic moment resistance.

4) Slender or class – IV

The slender section cannot attain even the first yield moment because of premature local buckling of web or flange.

Q.6(e) A simply supported beam of 6 m span supports on R. C. C. slab where in comp flange is embedded. The beam is subjected to a dead load of 25 kN/m and super imposed load of 20 kN/m, over entire span. Calculate plastic and elastic modulus required. [4]

Assume r_f = 1.5, γ_m = 1.1 f_y = 250 N/mm^2.

(A) 1) Calculation of factored load

Dead load = 1.5 × 25 = 37.5 KN/m

Live load = 1.5 × 20 = 30 KN/m

2) Calculate Maximum bending moment and shear force.

$$\text{B.M.} = \frac{WL^2}{8} + \frac{WL^2}{8} = \frac{37.5\times 6^2}{8} + \frac{30\times 6^2}{8} = 303.75 \text{ KN.m}$$

$$\text{S.F.} = \frac{WL}{2} + \frac{WL}{2} = \frac{37.5\times 6}{2} + \frac{30\times 6}{2} = 202.5 \text{ KN.m}$$

3) Plastic modulus

$$Z_P = \frac{M\times r_{mo}}{f_y} = \frac{303.75\times 10^6\times 1.1}{250} = 1.3365\times 10^6 \text{ mm}^3$$

4) Elastic modulus

$$Z_e = \frac{Z_P}{1.14} = \frac{1.3365\times 10^6}{1.14} = 1.17236\times 10^6 \text{ mm}^3$$

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