Important Instruction to Examiners:-

1) The answers should be examined by key words & not as word to word as given in the model answers scheme.

2) The model answers & answers written by the candidate may vary but the examiner may try to access the understanding level of the candidate.

3) The language errors such as grammatical, spelling errors should not be given more importance.

4) While assessing figures, examiners, may give credit for principle components indicated in the figure.

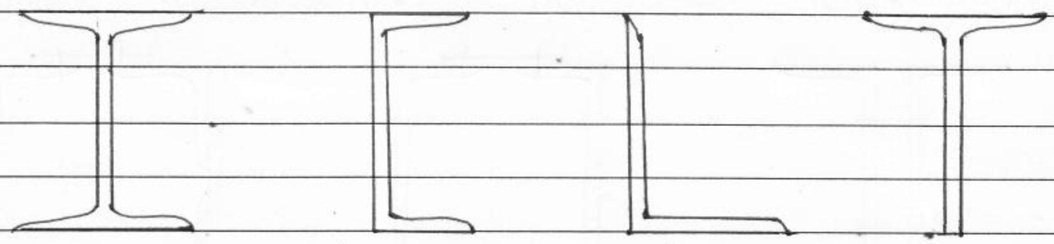
5) The figures drawn by candidate & model answer may vary. The examiner may give credit for any equivalent figure drawn.

5) Credit may be given step wise for numerical problems. In some cases, the assumed contact values may vary and there may be some difference in the candidate's answers and model answer.

6) In case of some questions credit may be given by judgment on part of examiner of relevant answer based on candidates understanding.

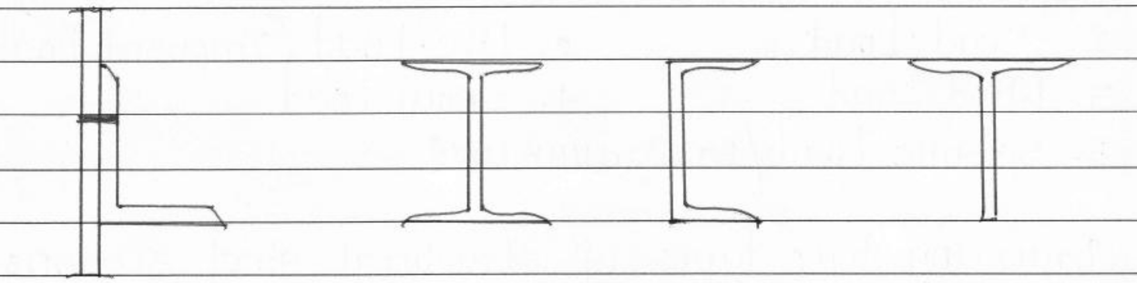
7) For programming language papers, credit may be given to any other programme based on equivalent concept.

Important notes to examiner

Q.1.A)	Attempt any three.	12M
a)	What are the types of loads to be considered while designing the steel structures?	4M
Ans:	Types of loads are- 1. Dead Load 2. Live Load (Imposed Load) 3. Wind Load 4. Snow Load 5. Seismic Load / Earthquake load.	1M each any four
b)	Draw any four types of structural steel sections.	1M each
Ans:	 (a) I section (b) Channel section (c) Angle section (d) T section	
c)	Define Limit state and state different types of limit states.	4M
Ans:	Limit state definition: The acceptable limit for the safety and serviceability requirements before failure occurs is called limit state.	1M
	Types of Limit states- 1. Limit state of strength It includes: i. Plastic Collapse ii. Stability against sway, overturning & sliding. iii. Fatigue	$1\frac{1}{2}$M
	2. Limit state of Serviceability. It includes: i. Deflection ii. Durability iii. Vibration iv. Fire resistance	$1\frac{1}{2}$M

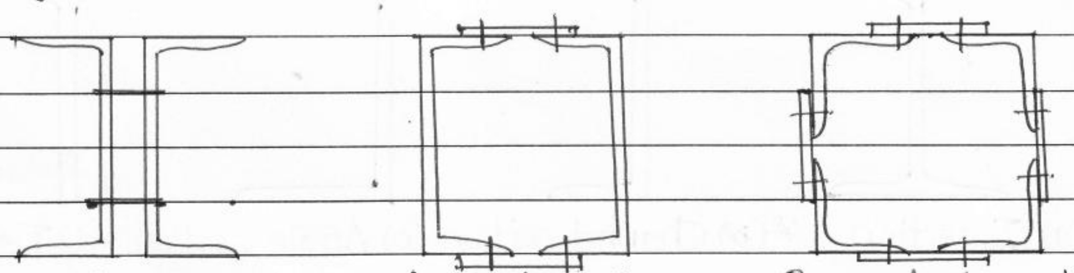
d) State with sketch different single and built-up sections of structural steel members used as tension member.

Ans: 1M each any two



Angle section I section Channel section T section

1M each any two



Channel section back to back Channel section Face to Face Four Angle section

B) Attempt any one: 06M

a) Design a suitable filter weld to connect a tie bar 80mm x 8mm to 10mm thick gusset plate. Design the joint for full strength of the tie and assume welding on all three sides as shown in figure no. 1.

Sol: 1. Design strength of 80 X 8 mm plate

$$P_{dw} = \frac{F_y}{\gamma_{mo}} A_g = \frac{250}{1.10} \times 80 \times 8 = 145454\ N$$

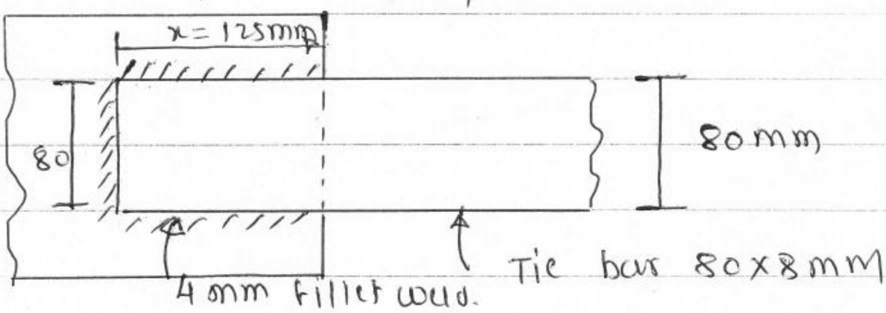
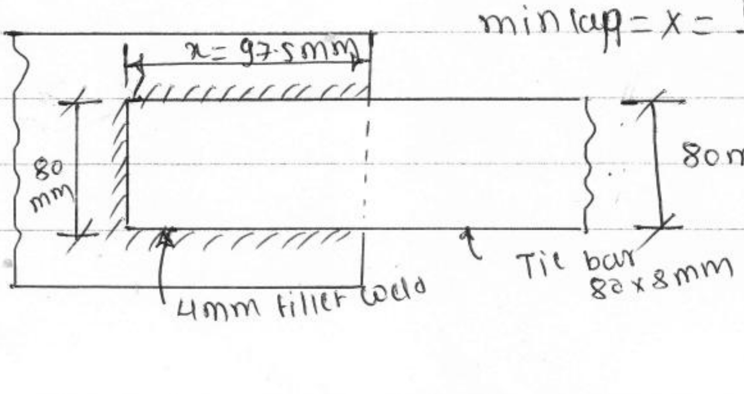
$P_{dw} = 145.45$ kN or $0.91 A_g \cdot f_y$ 1 M

2. Size of weld

Minimum size = 3mm

Maximum size = 8 − 1.5 = 6.5 mm

Provide 4mm site weld 1 M

Q.NO	SOLUTION	MARKS
	3) Design stress for site weld	
	$f_{wd} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.50} = 157.80\ N/mm^2$	1M
	4) Design strength per mm length of weld	
	$P_q = f_{wd} \times t_t = 157.80 \times (0.7 \times 4)$	
	$= 441.84\ N/mm$	1M
	5) effective length of weld required	
	$L = \frac{P_{dw}}{P_q} = \frac{145454}{441.84} = 329.20\ mm$ say 330mm	1M
		1M
	min Lap = $x = \frac{L-80}{2} = \frac{330-80}{2} = 125\ mm$	
	or	or
	3) Design stress for ~~wel~~ shop weld	
	$f_{wd} = \frac{f_u}{\sqrt{3} \times 1.25} = \frac{410}{\sqrt{3} \times 1.25} = 189.37\ N/mm^2$	1M
	Design strength per mm length of weld	
	$P_q = f_{wd} \times t_t = 189.37 \times (0.7 \times 4) = 530.238\ N/mm$	1M
	effective length $= \frac{P_{dw}}{P_q} = \frac{145454}{530.238} = 274.31\ mm$ say 275 mm	1M
	min lap $= x = \frac{L-80}{2} = \frac{275-80}{2}$	
	$\boxed{x = 97.5\ mm}$	
		1M

b)	Two ISA 80X80X6 is connected back to back on either side of 10mm thick gusset plate using fillet weld. Determine tensile strength of member from yield criterion only. For ISA 80X80X6, $A_g = 929\ mm^2$ $C_{zz} = 21.8\ mm$. Take $f_y = 250\ MPa$, $\gamma_{m0} = 1.1$ and $f_u = 410\ MPa$	6 M
Sol:	1. Design tensile strength governed by gross-section yielding	
	$T_{dg} = \dfrac{A_g f_y}{\gamma_{m0}}$	2 M
	$= \dfrac{2 \times 929 \times 250}{1.1}$	2 M
	$= 422{,}272\ N$	2 M
	Tensile strength of member = 422.272 kN (For Yield criterion only - as given in problem)	

b) A discontinuous compression member consists of 2 ISA 90X90X10mm connected backs to back on opposite sides of 12mm thick gusset plate and connected by welding. The length of strut is 3m. It is welded on either side. Calculate design compre--ssive strength of strut. For ISA 90X90X10, $C_{xx} = C_{yy} = 25.9$ mm $I_{xx} = I_{yy} = 126.7 \times 10^4$ mm^4, $r_{zz} = 27.3$ mm values of fcd are given.

Sol:

i. $r_{zz} = 27.3$ mm (Due to symmetry @ zz axis) 1 M

ii. $I_{yy} = 2\left[I_y + A \cdot h^2\right]$

$$= 2\left[126.7 \times 10^4 + 1703\left(25.9 + \frac{12}{2}\right)^2\right]$$ 1 M

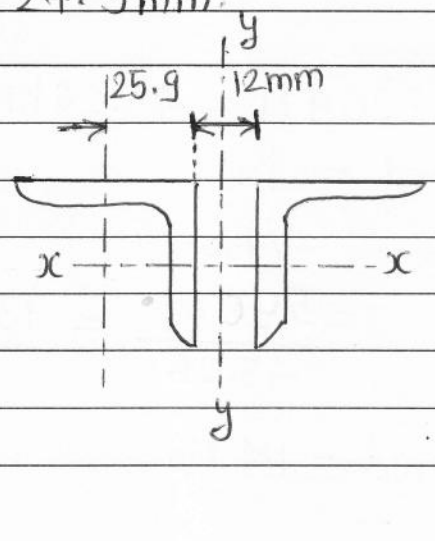
(A is calculated by calculating Area of both leg seperately and then adding them)

$\therefore I_{yy} = 5999979$ mm^4 1 M

iii. $r_{yy} = \sqrt{\dfrac{I_{yy}}{A_g}} = \sqrt{\dfrac{5999979}{2 \times 1703}} = 41.97$ mm 1 M

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

$r_{min} = 27.3$ mm. 1 M



iv. For discontinuous double angle, effective length

$$KL = 0.85L = 0.85 \times 3 = 2.10\ m = 2100mm$$ 1/2

$$S.R. = \frac{KL}{r_{min}} = \frac{2100}{27.3} = 76.92$$ 1/2

$\frac{KL}{r}$ (SR)	fcd
70	152
80	136

Hence,

$$f_{cd} = f_{cd1} - \frac{f_{cd1} - f_{cd2}}{SR_2 - SR_1}(SR - SR_1)$$

$$f_{cd} = 152 - \frac{152 - 136}{80 - 70}(76.92 - 70)$$

$$f_{cd} = 140.928\ N/mm^2$$ 1 M

v. Design compressive strength

$$P_d = f_{cd} \times A_g$$

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

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

$$P_d = 480\ kN.$$ 1 M

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

Sol: i. Loads and factored BMS

$$w = 15\,kN/m$$

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

Factored BM, $M_d = \dfrac{w_d \cdot l_e^2}{8} = \dfrac{22.5 \times 6^2}{8}$ — 1M

$= 101.25$ kN/m

Factored S.F., $V_d = \dfrac{w_d \cdot l_e}{2} = \dfrac{22.5 \times 6}{2} = 67.5$ kN — 1M

ii. Plastic modulus of section required

Z_p reqd. $= \dfrac{M_d \cdot \gamma_{mo}}{f_y} = \dfrac{101.25 \times 10^6 \times 1.1}{250}$

$= 445.5 \times 10^3$ mm^3 — 1M

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

iii. Classification of beam section

$d = h - 2(f_t + r_1) = 250 - 2(12.5 + 13)$

$= 199$ mm — 1M

$$\frac{b_h}{t_f} = \frac{\frac{125}{2}}{12.5} = 5.0 < 9.4$$

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

As $\frac{b_h}{t_f} < 9.4$ and $\frac{d}{t_w} < 67$ $\therefore$ Section classification is plastic. 1 M

v. Check for shear

$$V_{dr} = \frac{f_y \times t_w \times h}{\gamma_{m0}\sqrt{3}} \quad \underline{or} \quad 0.525 f_y . t_w . h$$

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

$$= 226.35\,kN > V_d\ (= 67.5\,kN)$$

Also, $\frac{V_d}{V_{dr}} = \frac{67.5}{226.35} = 0.298 < 0.6$ 2M

$\therefore$ Check for shear is satisfied.

vii. Check for deflection

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

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

$$= 20\ mm$$

$$\delta_{max} = \frac{5}{384}\frac{wL^4}{EI}$$

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

$$= 24.66\ mm.$$ 1M

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 a) State any modes of failure of bolted joint. 04M

Failure of bolted joint is failure of the plate connected or failure of bolt.

Failure of bolted joint

Failure of plate	Failure of bolt
→ by tearing of plate (shear)	→ by shear failure of bolt
→ by tensile failure of plate	→ by tensile failure of bolt
→ by bearing failure of plate	→ by bearing failure of bolt

02M for failure of plate

02M for failure of bolt

b) State any four advantages and dis-advantages of welded connections over bolted connection

A) Advantages of welded connection

$\frac{1}{2}$M each write Any four

i) Since the process does not involve driving holes, gross sectional area is effective, so more load carry capacity of the member as compared to bolted connection.

ii) Welded structures are lighter than bolted structure

iii) Often a welded joint has the strength of the parent itself.

iv) Repairs and further new connections can be made more easily than bolting.

v) Members of such shapes that afford difficulty for bolting (like circular sections) can be more easily welded.

vi) A welded structure has a better finish and appearance than the bolted structures

vii) Connecting gusset plate, angles can be minimized.

viii) it is possible to weld at any point at any part of the structure. But bolting always require enough clearance.

ix) it is possible to get 100 % efficiency.

x) welded connections are more water tight.

Dis-advantages of welded joints

i) welding require skilled labours and supervision — ½M each write Any four

ii) internal stresses in the weld are likely to set up.

iii) Due to uneven heating and cooling the welded members are likely to get warped.

iv) there is a greater possibility of brittle fracture in welding

v) Testing of a welded joint is Difficult. it needs non-destructive testing.

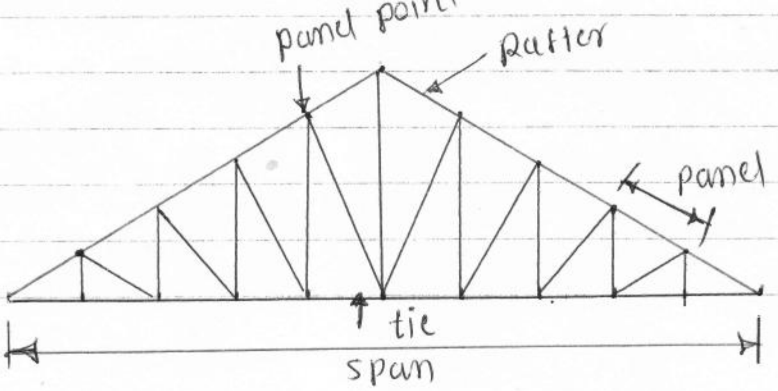
vi) Defects like internal air pockets, incomplete penetration are difficult to detect

vii) welded joints are over rigid.

viii) the fatigue strength is less as compared to bolted joints.

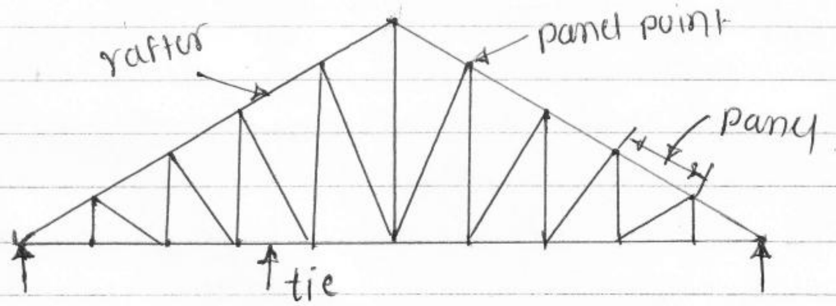
c) Draw neat sketches of Howe & North light trusses. mark panel, panel point, rafter & tie in any one truss.

i) Howe truss —



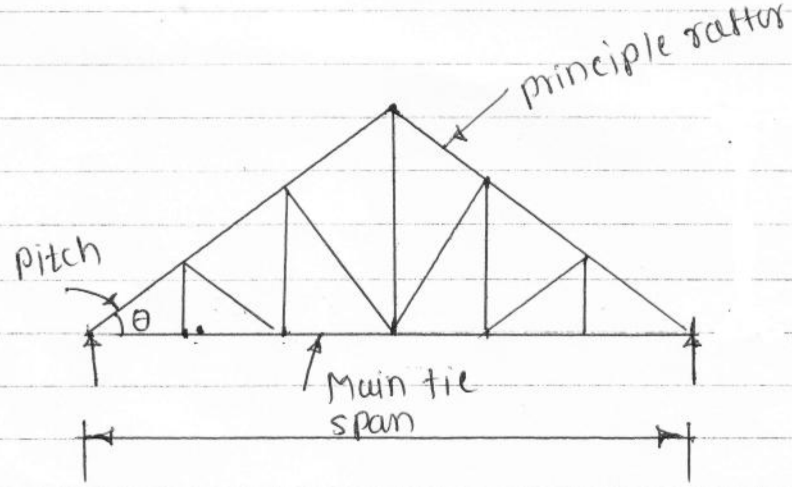
02M

ii) North light Roof truss



02M

d)



02M for Dia

use of steel roof truss

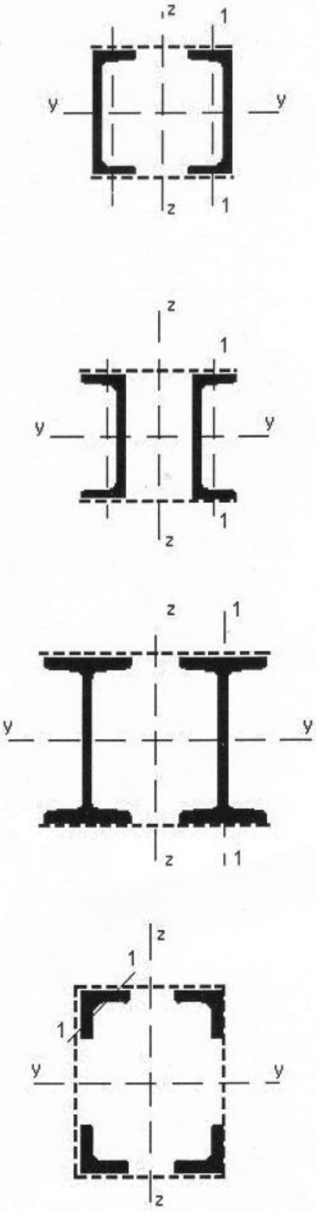
½M each write Any four

i) At the place of high rainfall to avoid the roof drainage problems.
ii) where roofs have to support an additional load due to snowfall
iii) for very large span, where use of beams will make the construction most uneconomical.
iv) for roofs of multistory buildings, industrial building auditoriums, cinema halls, malls, comericial complexes. stadium etc
v) In the form of bracing in horizontal plans and vertical planes in industrial building to resist lateral loads & wind load.

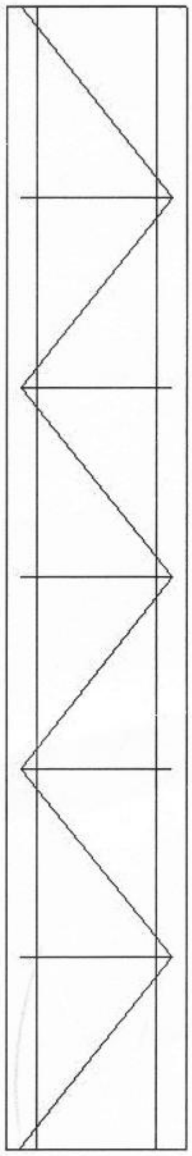
Q. NO	SOLUTION	MARKS
Q-3 e)	what is purlin? state IS 800-2007 procedure for design of angle purlin.	
→	purlin is a horizontal member consisting of unequal angle or channel section supported on principal rafter	01 M
	Design procedure	
	i) Load calculation	
	ii) calculate the component of load along x-x & along y-y axis	
	iii) calculate factored load due to combination.	03 M
	iv) calculate Bi-axial moment	
	v) selection of angle section.	
	vi) calculate of design moment	
	vii) check for biaxial bending for out standing toe of purlin. $\frac{M_x}{M_{dx}} + \frac{M_y}{M_{dy}} \leq 1$	

Q No.4 A		
a)	**Sketch different sections used as built up strut and built-up column.**	04M

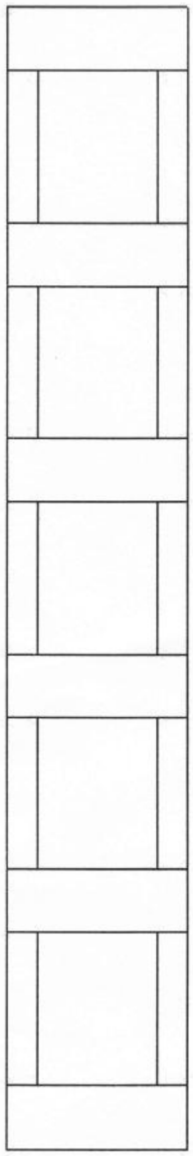
a) **Built-up column.**



Plan



section-1

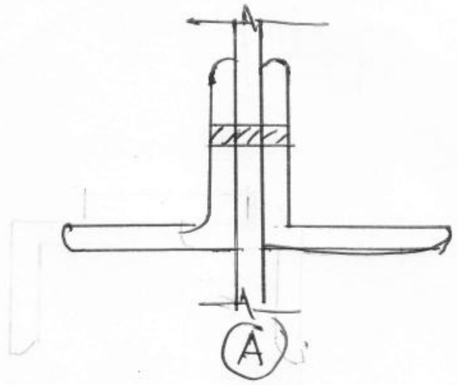
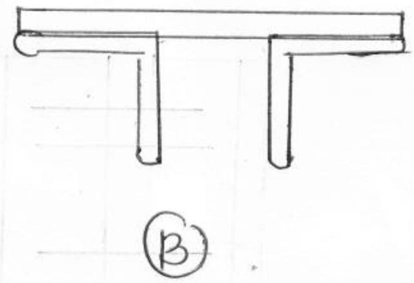


section-2

Built-up columns

1M for Any ONE Plan

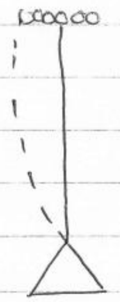
1M for section Any ONE

	b) built up strut	
		01M
		01M

Q-4 (A)

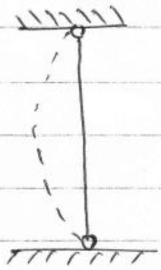
b) State effective length for a compression members having end conditions as 04M

i) Restrained against translation and free against rotation at one end but roller supported at the other end.



effective length = 2L 02M

ii) Restrained against translation and free against rotation at both ends



effective length = 1L 02M

Q-4 (c) Draw neat sketch showing single lacing system. why lacing is used 04M

i)

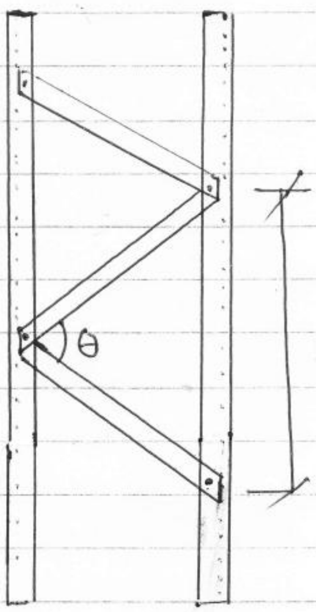


Diagram of single lacing system

02M for fig

ii) use of lacing

The purpose of lacing is to hold the variouse parts of a column straight; parallel at a correct distance apart and to equalize the stress distribution betn it's various part's 02M for use

d) Limiting width to thickness ratio for single angle strut of semi compact class is 15.7 ϵ. State whether ISA is 100x100x6 mm is of semi compact class or not take $f_y = 250$ Mpa. 04M

i) $D/t < 15.7\epsilon$

ii) $d/t < 15.7\epsilon$

01M

where $\epsilon = \sqrt{\frac{250}{f_y}}$ ... $f_y = 250\ N/mm^2$ 0.1M

$\epsilon = \sqrt{\frac{250}{250}} = 1$ $\boxed{\epsilon = 1}$

i) $b/t = \frac{100}{6} = 16.67 > 15.7$ --- (1) 01M

ii) $\frac{d}{t} = \frac{100}{6} = 16.67 > 15.7$ --- (2) 01M

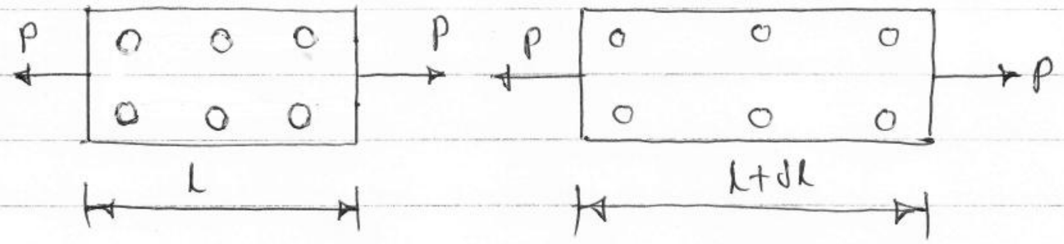
from above two eqn it conclude that the given single angle strut is not in semi compact class.

Q .NO	SOLUTION	MARKS

Q-4(B) a) State and explain three modes of failure of axial tension member. 06M

i) gross-section yielding

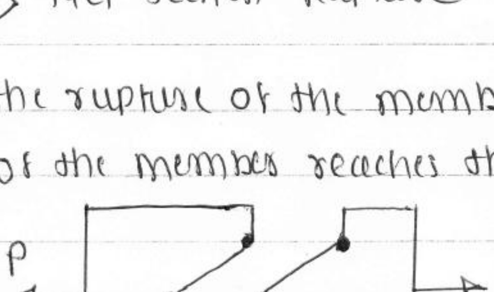
considerable deformation of the member in longitudinal direction may takes place before it fractures making structure unserviceable. 1M



1M

ii) Net section Rupture

the rupture of the member when the net cross-section of the member reaches the ultimate stress. 1M

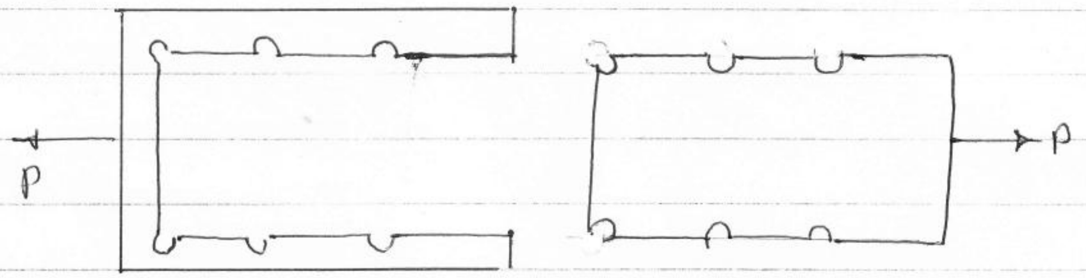


1M

iii) Block shear failure

A segment of block of material at end of member shear out due to the possible use of high bearing strength of the steel and high-strength bolts resulting in smaller connection length.

the factored design tensile load 'T' in the member should be less than the design strength Td of the member

Q.NO	SOLUTION	MARKS
	the design strength of the member should be less than the design strength of the member under the axial tensile load is the lowest of the design strength due to yielding of gross-section Tdg rupture of critical (net) section Tdn & block shear Tdb.	1M
		1M

b) Design a suitable angle section as a tie member in a truss to carry factored load of 215 KN.
use double angle section connected back to back on either sides of 12 mm thick gusset plate by means of 4-20 mm dia. bolt in one line. assume design strength of 20 mm dia Bolt = 45.3 KN. 06M
$\alpha = 0.8$, $\beta = 1.08$, $\gamma_{mo} = 1.1$, $\gamma_{m1} = 1.25$, $f_y = 250$ Mpa
$f_u = 410$ Mpa.

i) Approximate gross-area required Ag

$$\text{Reqd } A_g = \frac{1.1 \times T_{dg}}{f_y} = \frac{1.1 \times 215 \times 10^3}{250} = 1100\,mm^2$$

{ some student Assume 10 to 15 %. more area for selection of section }

Area of each angle reqd $= \frac{1100}{2} = 550\,mm^2$

Try ISA 80×50×8 giving $A_g = 2 \times 978 = 1956\,mm^2$

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

$$T_{dg} = \frac{A_g \times f_y}{\gamma_{mo}} = \frac{1956 \times 250}{1.1} = 444.54\text{ KN}$$ ½M

ii) Design strength due to rupture of critical section T_{dn}

$$T_{dn} = \alpha A_n \frac{f_u}{\gamma_{m1}}$$ ½M

Diameter of rivet = 20 mm

$\therefore D_n = 20 + 2 = 22$ mm

$A_n = 2(A_{nc} + A_{go})$

$A_{nc} = \left[B_1 - d_h - \frac{t}{2}\right] \times t$

$= \left[80 - 22 - \frac{8}{2}\right] \times 8$

$A_{nc} = 432\ mm^2$

$A_{go} = \left[B_2 - \frac{t}{2}\right] \times t$ ½ M

$= \left[50 - \frac{8}{2}\right] \times 8$

$= [50 - 4] \times 8$

$= 368\ mm^2$

$\therefore A_n = 2[432 + 368]$

$A_n = 1600\ mm^2$ ½ M

let $T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$ or $0.8\alpha . A_n f_u = T_{dn}$

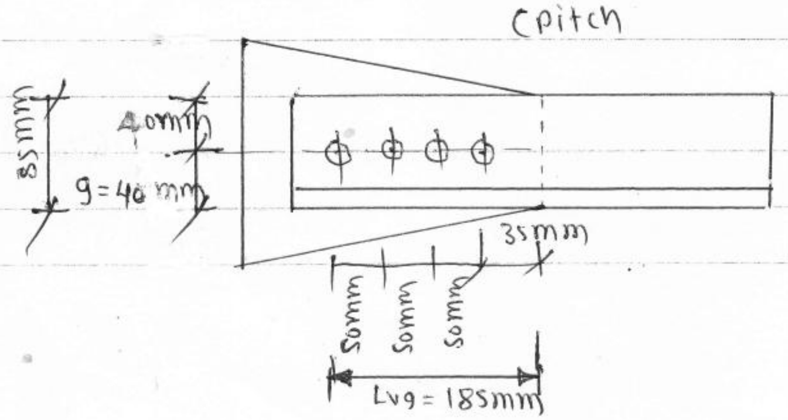
$T_{dn} = \frac{0.8 \times 1600 \times 410}{1.25}$

$\boxed{T_{dn} = 419.84\ KN}$ ½ M

Design strength due to block shear.

let assume edge distance = 35 mm

g = 40 mm P = 2.5 × d = 50 mm (pitch)



$A_{vg} = L_{vg} \times t$ ½ M

$= 185 \times 8$

$= 1480 mm^2$

$A_{vg} = L_{vg} \times t = 185 \times 8 = 1480 mm^2$

$\boxed{A_{vg} = 1480 mm^2}$

$A_{vn} = \left\{ L_{vg} - \left[(\text{no. of bolts} - 0.5) d_h\right] \right\} \times t$

$A_{vn} = \left\{ 185 - \left[(4 - 0.5) \times 22\right] \right\} \times 8$

$\boxed{A_{vn} = 864 mm^2}$ ½M

$A_{tg} = L_{tg} \times t = 40 \times 8 = 320 mm^2$

$\boxed{A_{tg} = 320 mm^2}$ ½M

$A_{tn} = \left[(L_{tg} - 0.5 d_h) \times t\right]$

$= \left[\left[40 - (0.5 \times 22)\right] \times 8\right]$

$\boxed{A_{tn} = 232 mm^2}$ ½M

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

$= \frac{1480 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 232 \times 410}{1.25}$

$= 194.199 \times 10^3 + 68.486 \times 10^3$

$T_{db1} = 262.68 \times 10^3 N$

$\boxed{T_{db1} = 262.68 KN}$ ½M

$$T_{db2} = \frac{0.9\,A_{vn}\,f_u}{\sqrt{3} \times \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$ ½M

$$T_{db2} = \frac{0.9 \times 864 \times 410}{\sqrt{3} \times 1.25} + \frac{320 \times 250}{1.1}$$

$$= 147.25 \times 10^3 + 72.72 \times 10^3$$

$$T_{db2} = 219.97 \times 10^3$$

$$\boxed{T_{db2} = 219.97 \text{ KN}}$$ ½M

for double angle block shear strength = 2 × 219.97
= 439.95 KN
= 434.45 KN > 215 KN
∴ OK.

5 (a)	Span of truss = 12 m Spacing = 3 m / c/c Types of truss = fink No of panel point = 6	08 M
	Rise = $\frac{\text{span}}{5}$ $= \frac{12}{5} = 2.4$ m	
	$\theta = \tan^{-1}\left(\frac{\text{Rise}}{L/2}\right) = \tan^{-1}\left(\frac{2.4}{6}\right) = \underline{21.80^\circ}$	1M
	Calculation of Dead load i) wt of roofing = 150 N/m^2 ii) wt of purlin = 220 N/m^2 iii) wt of truss = $\left(\frac{L}{3}+5\right) \times 10$ $= 90$ N/m^2 iv) wt of bracing = 80 N/m^2	1M
	Total dead load = 540 N/m^2	1M
	Total dead load on one truss = 540 x plane area = 540 x 12 x 3 = 19.44 KN	1M
	Dead load on each panel point = $\frac{19.44}{2}$ = 3.24 KN	
	D L on end panel point = $\frac{3.24}{2}$ = 1.62 KN.	1M

Live load calculation

L.L on purlin $= 750 - ((\theta - 10) \times 20)$
$= 750 - [(21.80 - 10) \times 20]$
$= 514\ N/m^2 > 400\ N/m^2$
$\therefore$ ok

L.L of truss $= \frac{2}{3} \times 514 = 342.67\ N/m^2$ 1M

$\therefore$ total L.L = L.L of truss × span × spacing.
$= 342.67 \times 12 \times 3$
$= 12336\ N$ 1M

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

L.L on end panel $= \frac{2056}{2} = 1028\ N$ 1M

Q-5 (b)	Span of truss = 14m Spacing of truss = 3.6m No. of panels = 8 Design wind pressure = 1.5 KPa $= 1.5 \times 10^3$ N/m2 $\theta = \tan^{-1}\left(\frac{Rise}{span/2}\right) = \frac{3.6}{14/2} = 27.22^\circ$ $\therefore \theta = 27.22^\circ$ wind load calculation coeff of external wind pressure $C_{pe} = -0.7$ coeff. of internal wind pressure $C_{pi} = \pm 0.2$ Total wind press = $[C_{pe} - C_{pi}] \times P_z$ wind load combination i) $w.L = [-0.7-(-0.2)] \times 1500 = 750$ N/m^2 ii) $w.L = [-0.7-(+0.2)] \times 1500 = 1350$ N/m^2	08M 1M 1M

Max. intensity = $-1350\ N/m^2$ [uplift]

Length of principal rafter = $\frac{L/2}{\cos\theta}$

$= \frac{[14/2]}{\cos 27.22}$

$\boxed{\text{length of principal rafter} = 7.87\ m}$

∴ Sloping area = $2 \times 7.87 \times 4$

$= 62.96\ m^2$ 1M

∴ Total wind load = max. intensity × sloping area

$= -1350 \times 62.96$

$= 84996\ N$

∴ Wind load on each panel = $\frac{84996}{8}$

$= -10624.5\ N$

∴ Wind load on end panel = $\frac{-10624.5}{2}$

$= -5312.25\ N$ 1M

Live load calculation →

Live load on purline = $750 - [(\theta - 10) \times 20]$

$= 750 - [(27.22 - 10) \times 20]$

$= 405.6 \nless 400\ N/m^2$ 1M

Hence ok

L.L. on truss

$= 2/3 \times 405.6 = 270.4\ N/m^2$ 1M

∴ Total L.L = L.L. intensity × span × spacing

$= 270.4 \times 14 \times 4$

$= 15142.4$ N

$\therefore$ load on each panel $= \frac{T.L}{No.\ of\ panel}$

$= \frac{15142.4}{8}$

$= 1892.8$ N $= 1.892$ KN 1M

& load on end panel

$= \frac{1892.8}{2}$

$= 946.4$ N $= 0.926$ KN 1M

5C] Given

factored load $Pu = 2000$ kN

$= 2000 \times 10^3$ N

$f_{ck} = 20$

$D = 400$

$B = 250$ i.e b_f

$\gamma_{m0} = 1.1$

$t_f = 12.7$

$f_y = 250$ N/mm^2

→ Bearing strength of conc.

$= 0.6\, f_{ck}$

$= 0.6 \times 20 = 12$ N/mm^2 1M

Bearing area of base plate

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

$$A = \frac{2000 \times 10^3}{12} = 166.67 \times 10^3$$ 1M

Size of base plate

length of plate

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

$$= \frac{400-250}{2} + \sqrt{\left(\frac{400-250}{2}\right)^2 + 166.67 \times 10^3}$$

$$= 490.08 \cong 500$$ 1M

$$B_p = \frac{A}{L_p} = \frac{166.67 \times 10^3}{500} = 333.34 \cong 350$$ 1M

Larger projection

$$a = \left(\frac{L_p - D}{2}\right) = \frac{500-400}{2} = 50 \text{ mm}$$ 1M

smaller projection

$$b = \left(\frac{B_p - B}{2}\right) = \frac{350-250}{2} = 50 \text{ mm}$$ 1M

Area of base plate

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

ultimate pressure from below on the slab base →

$$w = \frac{P_u}{A} = \frac{2000 \times 10^3}{175 \times 10^3} = 11.42 \text{ N/mm}^2$$ 1M

Thickness of slab base

$$t_s = \sqrt{\frac{2.5\, w\, (a^2 - 0.3 b^2)\, \gamma_{m0}}{fy}}$$

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

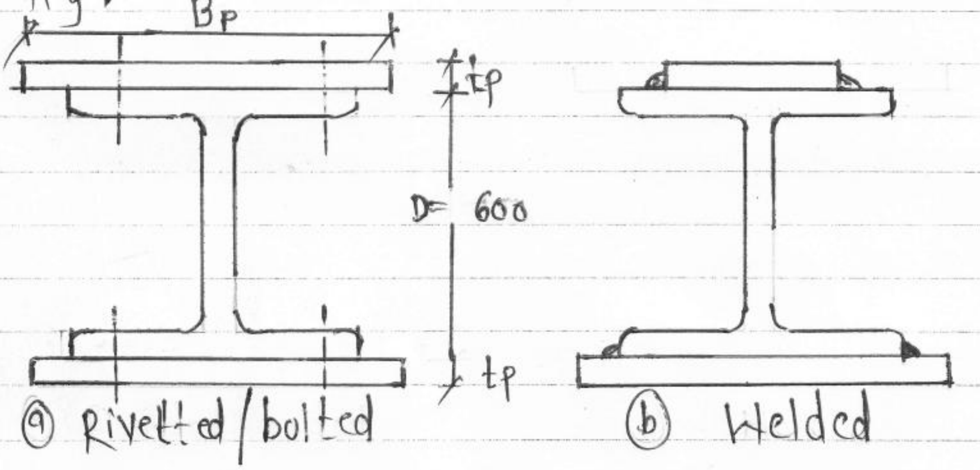
$= 14.82$ mm $>$ tf i.e 12.7

$\simeq 15$ mm 1M

Hence provide slab base plate having dimenssion

$500 \times 350 \times 15$

6 a] For a beam ISWB 600, if it is insufficient to resist the external bending moment it may be reinforced along the entire length or part of it. Also, the depth of the beam may be restricted due to the head requirement. In both above situation it becomes necessary to provide built up or compound beam as shown in fig. 2M



2M.

Fig: Built up beam | For ISHB 400

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6 b] Based on moment rotation behaviour the beam section are classified as follows in accordance with their behaviour in bending as per IS 800 : 2007

ⓐ Class 1 (plastic) :→
Cross section which can develope plastic hinges and have the rotation capacity required for failure of the structure by formation of the plastic mechanism. 1M

(b) class 2 (compact) :→
Cross-sections which can develope plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of plastic mechanism, due to local buckling 1M

(c) class 3 (semi-compact) :→
Cross-section in which the extreme fibre in compression can reach yield stress but cannot develope the plastic moment of resistance, due to local buckling 1M

(d) class 4 (slender) :→
Cross-section in which the elements buckle locally even before reaching yield stress 1M

Q 6C] Plate Girder :→

- Plate girder are built up beams comprising of plate sections for web and flanges when welded connection are used, and plate sections for web and angle sections with or without cover plates for flanges when bolted connections are used
- plate girder are flextural members. 2M
- plate girder is economical when the span is very large say 20 m.

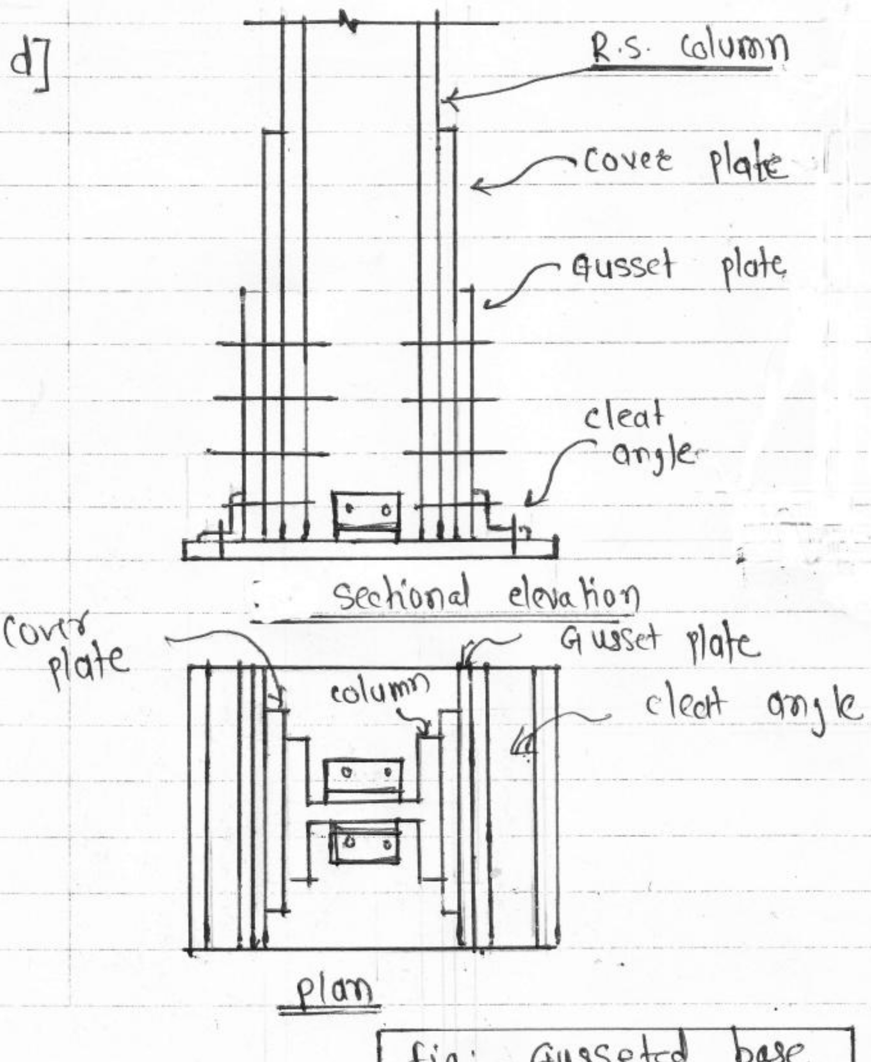
6 c] Functions of web plate :→

The web plate girder resists the entire shear force. The web plate sometime buckles due to excess load. To avoid Web buckling, vertical and horizontal stiffeners are provided. 1M

Functions of Bearing stiffeners :→

Bearing stiffeners function is to take point loads, hence are provided where point load are action or wt +ve point of support 1M

6 d]



02M Section

02M Plan

fig: Gusseted base

6(e)	the basic concept to decide the plan area of slab base and concrete block is that the column tranfere their loads to the soil through column base and distribute the loads over a greater area so that the pressure on the concrete block and soil does not exceed's the design bearing stress.	02M
	Function of cleat Angle :- To secure the column section with base plate providing cleat angle.	1M
	Function of Anchor bolt :- To connect concrete pedestal & base plate anchor bolts are used	1M