Important Instructions to examiners:

1) The answer should be examined by keywords and not as word-to-word as given in the model answer scheme.

2) The model answer and the answer written by candidate may vary but the examiner may try to assess the understanding level of the candidate.

3) The language error such as grammatical, spelling errors should not be given more importance. (Not applicable for subject English and communication skill).

4) While assessing figures, examiner may give credit for principal components indicated in the figure. The figure drawn by candidate and model answer may vary. The examiner may give credit for any equivalent figure drawn.

5) Credits may be given step wise for numerical problems. In the some cases, the assumed constants values may vary and there may be some difference in the candidates answer 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

Question and Model Answers	Marks
Model Answer :→	
Q1. Attempt any three	12M
A) components & corresponding function for	
i) Gantry girder → function of each component	02M
1. Crane wheel → It can move longitudinal dir^n	
2. crane girder → To lift & move heavy material	
3. Crab → load can be lifted and shifted across the width shop	
4. Rail → Rail is mounted on gantry girder.	
The function and gantry girder is to lift and move the heavy materials and machinery from one place to other.	
ii) steel Water tank →	02M

Q	specification/diagram	Mark
	1. Rolled steel section stays – at the junction of plates 2. Mild steel cleats – supported by columns 3. Steel beams – support to tank to distribute load 4. Top tier – Supported to bottom tier Water tank may rest on the ground or be elevated. The function of water tank is to contain material.	
b)	1. Dead load – IS. 875 (Part-1) - 1987 2. Live load – IS 875 (Part-2) - 1987 3. Wind load – IS. 875 (Part-3) - 1987 4. Snow load – IS 875 (Part-4) - 1987 5. Seismic load – IS. 1893 (Part-I) - 2002	(01 M for Each) 4
c)	Limit state of strength →	
	• Limit state of strength, using appropriate factor of safety, are those connected with failures under the, action of probable and most unfavorable combinations of load on the structure which may endanger the safety of life and property. 1. Plastic collapse 2. stability against sway, overturning and ~~st~~ sliding. 3. Fatigue.	02M
	• Limit state of Serviceability. is related to the satisfactory performance of the structure at working load. There are four major types of serviceability limit states applicable to steel structures. They are. i) Deflection ii) Durability iii) Vibration iv) Fire resistance.	02M
d)	Design strength of Tension members → The design strength of members under axial tension, shall be minimum of following three failures. i) due to yielding of the gross-section. ii) rupture of net section. iii) Failure due to block shear.	02M
	Formula for – due to yielding of the gross-section	~~02M~~

Formula for design strength For Tension member

i) due to yielding of the gross-section

$\frac{T}{Ag} < fy$ Where,

$T < Ag \cdot fy$

$Tdg = \frac{Ag \cdot fy}{\gamma mo}$

T = factored tensile force

Tdg = Design strength

A.g = Gross sectional Area

γmo = Partial safety factor (1.1)

ii) rupture of net section,

$T < An \cdot fu$

$Tdn = \frac{T}{\gamma m_1} = \frac{An fu}{\gamma m_1}$

Where,

Tdn = Design st. in rupture

fu = Ultimate stress

γm_1 = Partial safety factor (1.25)

iii) Block shear failure

02M for any (one of above three)

a) for shear yield & tension fracture

$$Tdb_1 = \frac{Avg\, fy}{\sqrt{3}\,\gamma mo} + \frac{0.9\, Atn \cdot fu}{\gamma m_1}$$

b) For shear fracture and tension yield

$$Tdb_2 = \frac{Atg\, fy}{\gamma mo} + \frac{0.9\, Avn\, fu}{\sqrt{3}\,\gamma m_1}$$

minimum of Tdb_1 & Tdb_2 is considered as block shear strength

Q1B a) → Attempt any one — 06m

a) Data

$P = 80 kN$

$f_u = 410 mpa$

$f_{ub} = 400 mpa$

Assume - 16mm dia bolt

Step I →

$d_n = d_o = 16+2 = 18mm$

$A_{nb} = 157 \ mm^2$

Step 2 →

single shear strength of bolt — V_{dsb}

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

$$= \frac{400}{\sqrt{3}} \left[\frac{157}{1.25} \right]$$

$= 29000 \ N = 29 kN$ in single shear — 02m

Step 3 Bearing strength of bolt

Where K_b is smaller

$P = 50mm$,

$e = 40 \ mm$,

$$\frac{e}{3d_o} = \frac{40}{3 \times 18} = 0.74$$

$$\frac{P}{3d_o} - 0.25 = 0.67$$

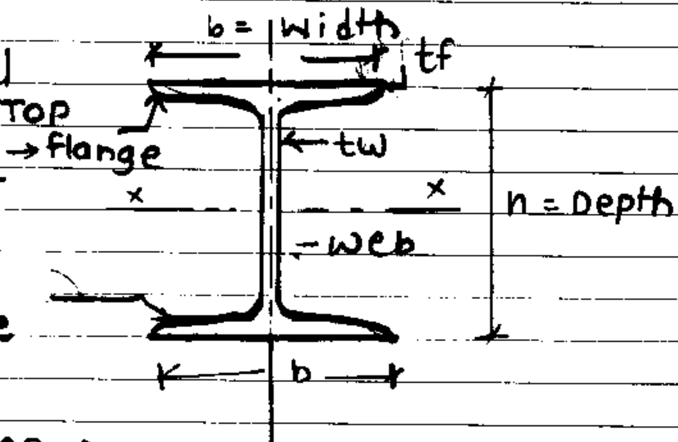
$$\frac{400}{410} = 0.975 = 1.0$$

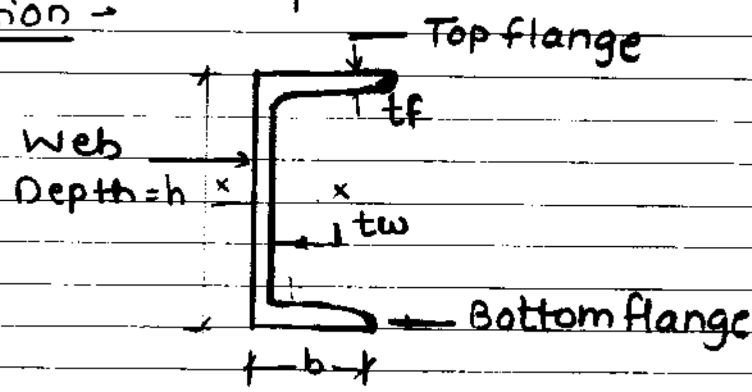
$\therefore K_b = 0.67$

$$V_{dpb} = \frac{2.5 \ K_b \cdot d \, t \cdot f_u}{1.25} = 105.48 > 29 kN$$ — 02m

$= 105.48 \ KN$

Q		specification/diagram	Mark
		step4 – Tensile strength of plate per pitch length	
		$p = 50\ mm$	
		$e = 40mm$	
		$T_{dn} = 0.9 \frac{f_u}{\gamma_{m1}} (p - d_n)t$	
		$= 0.9 \times \frac{410}{1.25} (50 - 18) \times 12$	
		$\boxed{T_{dn} = 113.35\ kN}$	01m
		Least Bolt Value, minimum of three	
		$B_v = 29\ kN$	
		$= \frac{80\ kN}{29} = 2.875$ say 3 No bolts required	01m
b)		Sketches of two rolled steel sections, as tension members.	
	i)	Rolled steel beam I, section	01m
		Channel section →	01m





Q1	B	specification/diagram	Mark
	b) ii)	Brief design steps of tension member	
		Step I Determine gross area required from its yield strength, $$\boxed{A_g = \frac{T}{(f_y/\gamma_{m0})}}$$ f_y = yield strength of material $\gamma_{m0} = 1.1$ T = Factored tensile load.	01m
		Step 2 From steel table select suitable rolled steel section providing area matching with calculated gross area in step 1.	
		Step 3 Calculate number of bolts required to connect the member of gusset plate / another member	01m
		Step 4 Calculate design strength T_d of trial section. It should be minimum of T_{dg}, T_{dn} & T_{db} Where, i) Gross section yielding, $$\boxed{T_{dg} = \frac{A_g f_y}{\gamma_{m0}}}$$ ii) Net section rupture a) For plate $T_{dn} = \frac{0.9 A_n f_u}{\gamma_{m1}}$ For angles $T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$ iii) Block shear failure	01m
		Step 5 For shear yield & tension fracture $$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + 0.9 \frac{A_{tn} f_u}{\gamma_{m1}}$$ For shear fracture & tension yield	01m

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

Q		specification/diagram	Mark
		Step 6.	
		minimum of T_{db1} & T_{db2} is Considered as block shear strength	
		Check for T_d check whether $T_d \geq T$ and whether to confirm the Section or to revise	
~~Q2.~~		~~Attempt any two~~	16 m
	a)	$G = 100 \times 75 \times 10$ mm	
		$P = 750$ kN	
		$f_u = 410$ mpa	
		$f_{ub} = 400$ mpa	
		For 16 mm dia. bolt $d_n = d_o = 16+2 = 18$ mm	
		$A_{nb} = 157\ mm^2$	
		Single shear strength of bolt =	
		$V_{dsb} = \frac{f_{ub}}{\sqrt{3}} \left(\frac{n_n \times A_{nb}}{\gamma_{mb}} \right)$	
		$= \frac{400}{\sqrt{3}} \left(\frac{1 \times 157}{1.25} \right)$	
		$\boxed{V_{dsb} = 29\ KN}$	02 m
		Double shear strength of bolt $= 2 \times 29 = 58$ KN	
		Bearing Strength of thinner plate	
		$V_{dpb} = \frac{2.5\ k_b \cdot d \cdot t \cdot f_{ub}}{\gamma_{mb}}$	01 m
		Assum $P = 3d$	
		$= 3 \times 16 = 48 = 50$ mm	
		$e = 2d$	
		$= 2 \times 16 = 32 \approx 40$ mm	
		$p = 50$ mm $e = 40$ mm	01 m

Q	specification/diagram	Mark

Kb is least of,

i) $\frac{e}{3d_0} = \frac{40}{3\times18} = 0.74$

ii) $\frac{p}{3d_0} - 0.25 = \frac{50}{3\times18} - 0.25 = 0.67$

iii) $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975$; 1.0

$\therefore \boxed{K_b = 0.67}$ 01m

$\therefore V_{dpb} = 2.5 \times 0.67 \times 16 \times \cancel{8}\,12 \times \frac{400}{1.25}$ 01m

$= \cancel{68}\ 102912$ N

$\boxed{V_{dpb} = 102.91 \text{ KN}}$

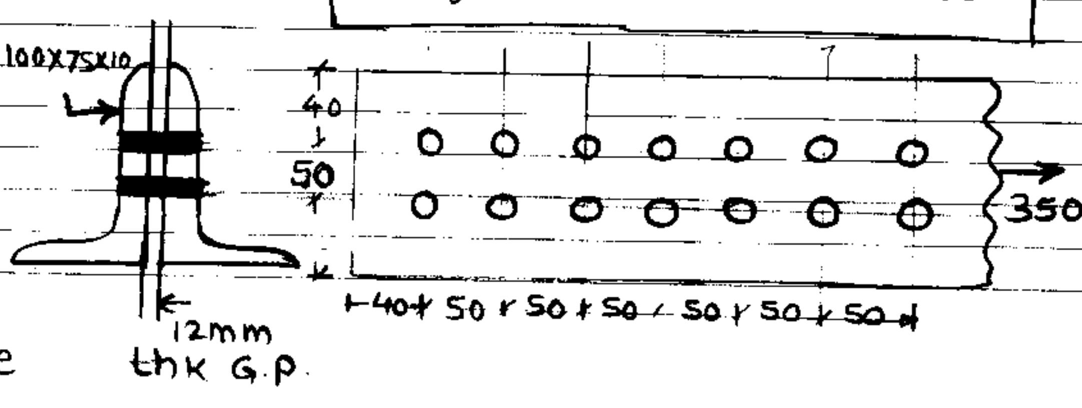
$\therefore$ Least Bolt Value.

$B_v = \frac{P_u}{\text{No. of Bolt}}$

No. of Bolts reqd $= \frac{P_u}{B_v}$

$= \frac{750\times10^3}{58}$

$\boxed{\text{No. of Bolt} = 12.93 \approx 14 \text{ Bolt}}$ 01m

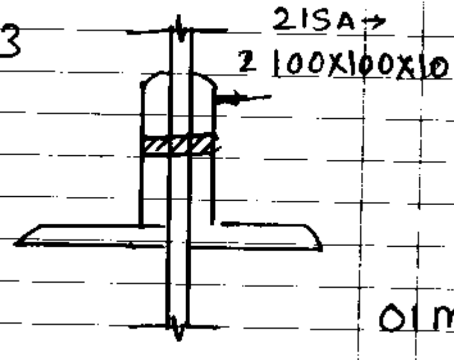


01m

Q2. b) $A = 1903\ mm^2$

$I_{xx} = I_{yy} = 177 \times 10^4\ mm^4$

$Z_{xx} = Z_{yy} = 24.7 \times 10^3\ mm^3$



For a section calculate

Step I

$I_{xx} = 2 \times 177 \times 10^4$

$= 3.54 \times 10^6\ mm^4$ 01m

Step 2

$\therefore r_{xx} = \sqrt{\dfrac{I_{xx}}{A_g}}$

$= \sqrt{\dfrac{3.54 \times 10^6}{2 \times 1903}} = 30.49\ mm$ 01m

$\therefore r_{xx}$ is minimum

Step 3

$SR = \dfrac{KL}{r_{min}} = \dfrac{0.85 \times 3000}{30.49}$

$= 83.631344,\ = 83.63$ 01m

Step 4

(SR)	(Fcd) by interpolation
80	136
83.63	
90	121

$f_{cd} = 136 - \dfrac{15 \times 3.63}{10}$

$= 130.55\ \dfrac{N}{mm^2}$ 01m

Step 5

Design compress load $= f_{cd} \times A_g$

$P_d = f_{cd} \times A_g$ 01m

Q		specification/diagram	Mark
		$f_{cd} = 130.557$	
		Design Compressive load $P_d = f_{cd} \times A_g$	
		$= 130.573.21 \times (2 \times 1903)$	
		$\boxed{P_d = 496.878163 \text{ KN}}$	03m
		$\boxed{P_d = 496.87 \text{KN}}$	
Q2	c.	Data :→	
		$L = 5.2m$	
		$Udl = 50 \text{ KN/m}$	
		$\gamma_{mo} = 1.1$	
		StepI : loads & Factored BMs.	
		Total Udl = 50 KN/m	
		$W_d = 50 \times 1.5 = 75 \text{KN/m}$	
		SF $V_d = \frac{Wle}{2} = \frac{75 \times 5.2}{2} = 195 \text{KN}$	
		$M_d = \frac{W_d L_e^2}{8} = \frac{75 \times 5.2^2}{8} = 253.50 \text{ KNm}$	02m
		StepII	
		Elastic modulus of section required	
		$Ze \text{ reqd} = \frac{Ze \text{ reqd}}{1.14}$	
		Plastic Modulus of section	
		$Ze \text{ reqd} = \frac{M_d \cdot \gamma_{mo}}{f_y}$	
		$= \frac{253.50 \times 10^6 \times 1.1}{250}$	
		$= 1113.20 \times 10^3 \text{ mm}^3$	01m

$$Ze_{reqd} = \frac{1.1132 \times 10^6}{1.14}$$

$$= \underline{976.49 \times 10^3 \, mm^3}$$ 01 m

Step III

Try ISWB 400

$h = 400\,mm$

$bf = 200\,mm$

$tf = 13.0 \quad tw = 8.6\,mm \quad R_1 = 13\,mm$

$Zxx = 1171.30 \times 10^3 \quad Ixx = 23426.70 \times 10^4\,mm^4$

~~d =~~ Classification of beam section.

$$d = h - 2(tf + r_1)$$

$$= 400 - 2(13+13) = 348\,mm$$

$$\frac{bh}{tf} = \left(\frac{200/2}{13}\right) = 7.69 < 9.4$$

$$\frac{d}{tw} = \frac{348}{8.6} = 40.46 < 67$$

Section Classification is Plastic 02m

Step IV,

Check for deflection

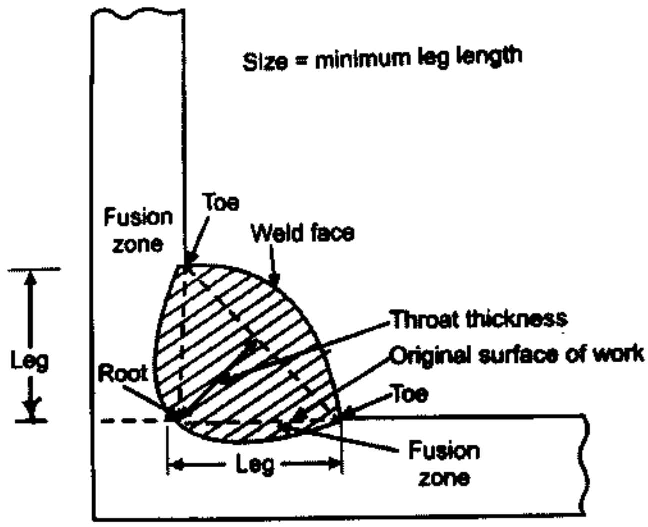
$$\delta_{allowable} = \frac{L}{300} = \frac{5200}{300} = 17.33$$ 01m

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

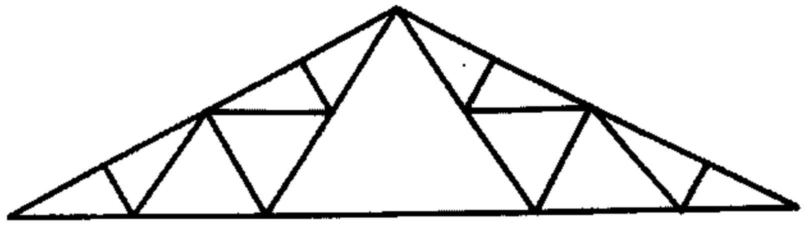
$$= \frac{5}{384} \times \frac{75 \times 5200^4}{2 \times 10^5 \times 23426.70 \times 10^4}$$

$$= 15.23\,mm$$ 01m

$\therefore$ $\delta_{max} < \delta_{allowable}$ Def. Check is O.K.

Question and sub Question	Model Answer	Marks
Q. 3	Attempt any **FOUR** of the following	**16 Marks**
a)	What do you mean by High strength bolts? State the uses of HSB with their commonly used property class. **Answer** **High Strength Bolts:** -The bolts with induced initial tension are called as High strength bolts. In high strength bolt initial pretension in bolts develops clamping force at the interfaces of elements being joined.	2 Marks
	Uses: High strength friction grip bolts are commonly used in practice. High strength bolts in friction type joints are used where slip in serviceability limit state is to be avoided in the connection. It is used when loads are transferred by friction only.	1 Mark
	The bolts of property class 8.8 and 10.9 are commonly used in High strength bolts.	1 Mark
Q. 3 b)	Draw Illustrative sketch of Fillet weld and state following properties with IS Code provisions. I) size of weld ii) Throat Thickness iii) Minimum length of weld. **Answer**  **Details of Fillet weld**	1 Marks
	i) **size of weld-** the size of normal fillet weld is taken equal to its minimum leg length. The size of fillet weld should not be less than 3mm	1 Mark
	ii) **Throat thickness of fillet weld (t)** – It is perpendicular distance from the root of fillet weld to line joining its toes. t = k x size of weld (s) k = 0.7 for right angle fillet weld t = 0.7 x s	1 Mark
	iii) **Minimum length of weld** – Effective length of fillet weld is taken equal to its actual length minus twice the weld size. The effective length of fillet weld should not be less than four times the size of weld.	1 Mark

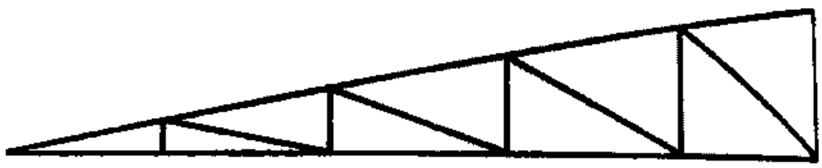
Q. 3 (c) Draw line sketches of i) compound fink truss, ii) North light roof truss.



(Used up to Spans 6m to 20m)

Compound fink truss

2 Marks

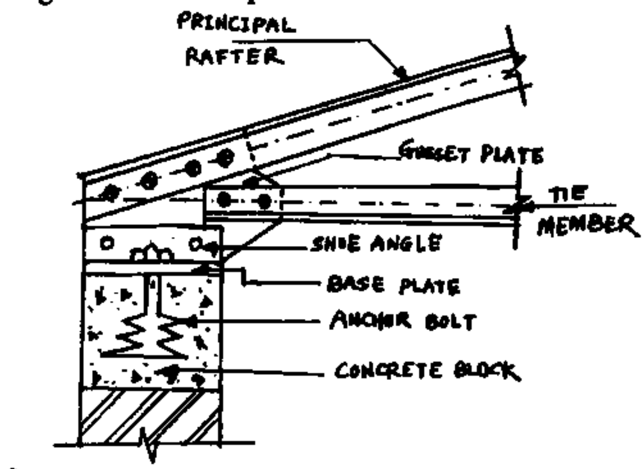


(Used up to Spans 6m to 10m)

North Light Roof Truss

2 Marks

Q3 (d) Draw neat sketch (detailed) of a truss support joint and panel point joint (any one) showing arrangement of members (sketch should include gusset plate connected with angles with the help of rivets/bolts.

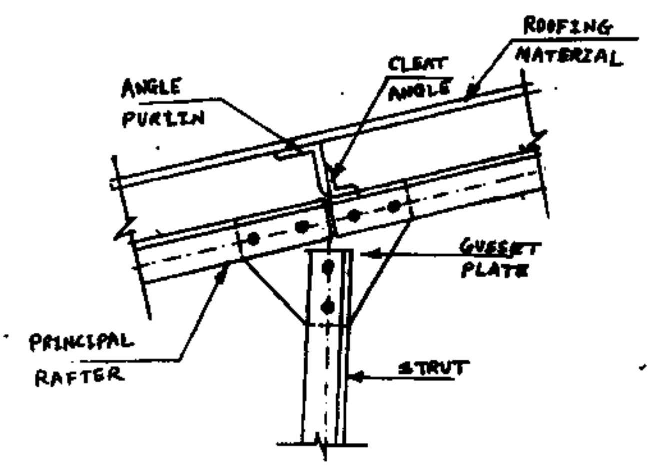


DETAILS OF TRUSS SUPPORT JOINT

4 marks

OR

OR



DETAILS OF TRUSS AT PANEL POINT

4 marks

Q 3 (e) State the necessity of purlins in trusses. State different checks to be taken while

Q. 3 (e)	designing the purlin (No Formula). **Answer-** **Necessity of Purlin is-** 1) To connect the roof trusses to each other and 2) To support the roofing material Following checks are taken while designing the purlin. i) Check for shear ii) Check for Bi axial bending of purlin iii) Check for deflection iv) Check for torsional buckling if required.	2 marks 2 marks (half mark for each)
Q. 4 (A) a)	Attempt any **THREE** of the following Define radius of gyration and slenderness ratio with maximum limit **Answer** **Radius of Gyration (K)**-The radius of gyration of a given area about a given axis is that distance from the given axis at which all elemental areas of given area should have to be placed so as not alter the moment of ineria about given axis. OR It is the square root of ratio of moment of inertia to the cross sectional area. **Slenderness Ratio (λ)** - It is the ratio of effective length of column to its least radius of gyration. $\lambda = Leff./r_{min}$ **Maximum values of slenderness ratios.**	**12 Mark** 1 Mark 1 Mark

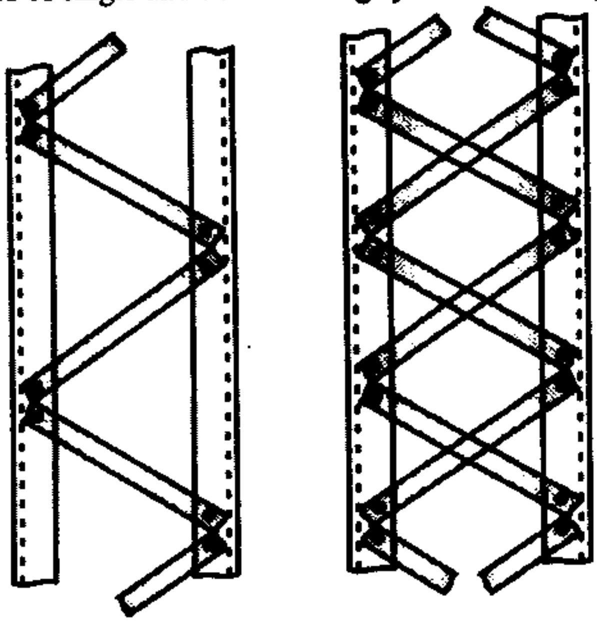
Type of member	**Maximum slenderness Ratio**
i) A member carrying compressive load resulting from dead loads and imposed loads	180
ii) A tension member in which reversal of direct stress due to loads other than wind or seismic forces occurs.	180
iii)A member subjected to compressive forces resulting from wind or earthquake forces.	250
iv)compression flange of beam.	300
v) A member normally acting as tie in a roof truss or bracketing system but subject to reversal of stresses resulting from wind or earthquake forces.	350
vi) Tension members	400

Any Four 2 Marks

b)	State in brief design steps of simple compression member **Answer- Design steps** 1) Calculate the ultimate axial load to be resisted by the member. 2) Assume suitable trial section. For single and double angles assume the design stress fcd = 90 N/mm^2 and for I section = 150 N/mm^2 3) Arrive at the effective length of column considering end condition. 4) Calculate the slenderness ratio. Check that they satisfy the maximum limits. 5) Check the buckling class of the cross section and select proper buckling class a, b, c or d. corresponding to the selected buckling class obtain the design compressive stress f_{cd}. 6) The allowable compressive load = f_{cd} x Ag	1 Mark 1 Mark 1Mark

7) Check that the design compressive load is equal to or greater than the applied load to be resisted. — 1Mark

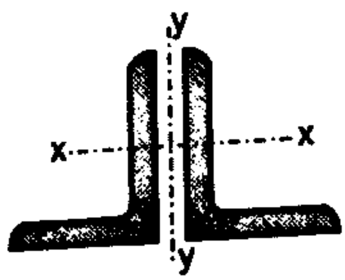
c) Draw neat sketches of single and double lacing system. State its purpose.



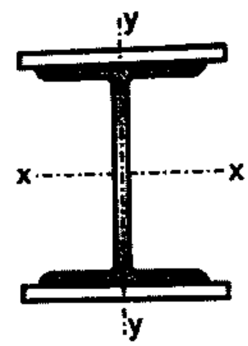
i) Single lacing System **ii) Double lacing system** — 2 Marks

Purpose of lacing-Purpose of lacing is to connect the individual components of built up compression members so that they will act as a single unit. — 2 Mark

d) Draw and label any four types of built up compression member.

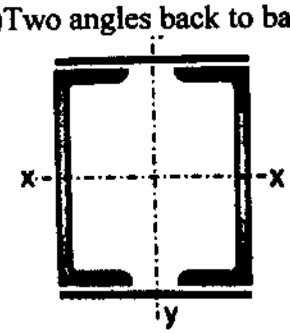


i)Two angles back to back

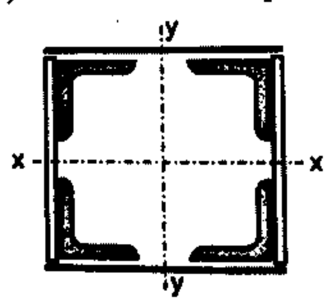


ii) I section with two plates on flanges

Any four section 1Mark each



iii) Two channels placed toe to toe



iv)Four angles placed toe to toe with two plate

if any other four built up sections drawn credit is given to students accordingly

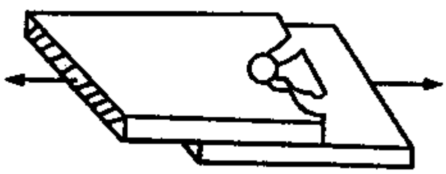
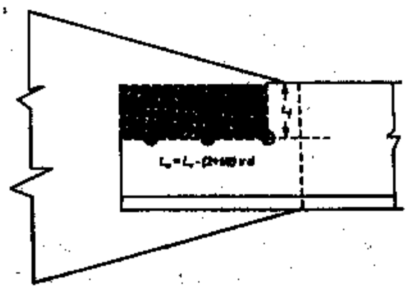
Q. 4 (B) Attempt any **ONE** of the following.

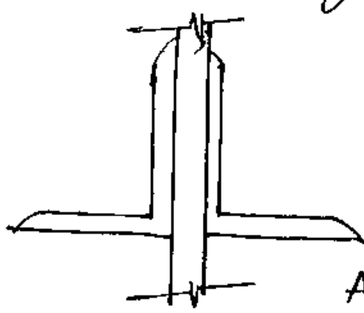
a) State with sketches different modes of failure member in axial tension. — **6 Marks**

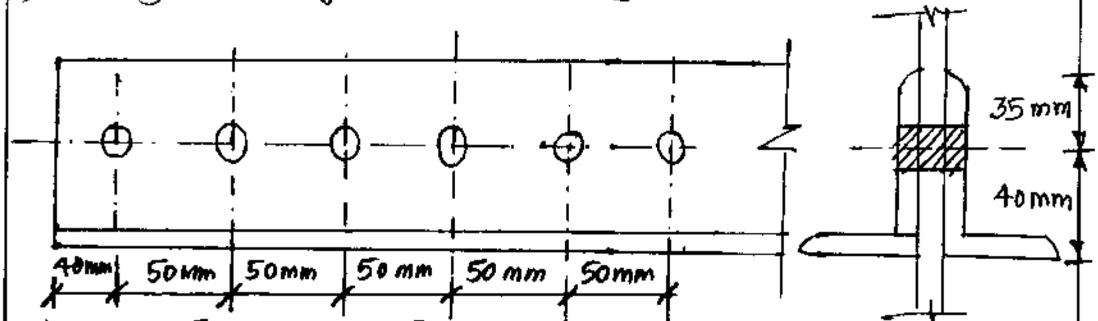
Ans-

Modes of failure of member in axial tension-

i) **Yielding of gross cross section** - In a member subjected to uniaxial tension, a stage is reached at which elongation increases without

	increase in load. At this stage yielding of gross section causes excess elongation and member fails in gross yielding of cross section.	1 Mark
	ii) **Rupture of net cross section-** A tension member is usually connected to other members by bolt or welds. The fibres adjacent to the bolt hole yield due to stress concentration. However the ductility of steel permits the initially yielded zone to deform without fracture. At this stage the entire net section reaches the ultimate stress and section fails in rupture.	1 Mark
	iii) **Block shear failure-** This type of failure occurs due to tearing of segment or block of the material at the end of the member. It occurs along a path involving tension on one plane and shear on a perpendicular plane.	1 Mark
	 **Rupture of the cross section**  **Block shear Failure**	3 Marks (one and half mark for each sketch)

Question and sub Question	Model Answer	Marks
4(B)(b)	A tension Member consists of two angles ISA 75 X 75 X 8 mm bolted to 10 mm thick gusset plate one on each side using single row of bolt and tack bolted. Determine the Maximum load that the member can carry Take i) Area of angle = $1140\,mm^2$ ii) Gauge distance as per IS clause	
→	Given 2 ISA 75 X 75 X 8 mm with 10 mm thk Gusset plate. Assume 20mm dia. 4.6 Grade bolts. If students consider other dia. bolts Credit should be given to students accordingly.  $Ag = 1140\,mm^2$ Dia. of bolt hole $d_h = 20 + 2 = 22\,mm$ Gross area of two angles. $Ag = 2 \times 1140\,mm^2 = 2280\,mm^2$	
	1) Design strength Governed by Gross section Yielding. $T_{dg} = \frac{Ag \cdot fy}{\gamma_{m0}} = \frac{2280 \times 250}{1.1}$ $T_{dg} = 518.18\,KN$	1 Mark.
	2) Design strength Governed by Net section Rupture Net Area $A_n = 2280 - 2(22 \times 8) = 1928\,mm^2$. Assuming bolts ≥ 4 in Connection $\alpha = 0.8$ Approximate Rupture strength. $T_{dn} = \alpha \frac{A_n \cdot f_u}{\gamma_{m1}} = \frac{0.8 \times 1928 \times 410}{1.25}$ $T_{dn} = 505.90\,KN$	1 Mark.

Question and sub Question	Model Answer	Marks
	Design tensile strength = Minimum of T_{dg} & T_{dn} $= 505.90\ kN$. Design shear stress for a bolt $= \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} = \frac{400}{\sqrt{3} \times 1.25}$ $= 184.75\ N/mm^2$. Net Area of bolt $A_{nb} = 0.78 \times \pi/4 \times 20^2 = 245.04\ mm^2$ Design shear strength of bolt in double shear $= 2 \times A_{nb} \times 184.75 = 2 \times 245 \times 184.75$ $= 90.52\ KN$ No. of Bolts reqd. $= \frac{505.90}{90.52} = 5.58 \simeq 6$ Nos. Minimum pitch $= p = 2.5d = 2.5 \times 20 = 50mm$ Edge distance $e = 1.7 d_o = 1.7 \times 22 = 37.4 \simeq 40mm$	1 Mark.
	3) Design strength Governed by Block shear  $A_{vg} = [5 \times 50 + 40] \times 8 = 2320mm^2$ $A_{vn} = [5 \times 50 + 40 - 5.5 \times 22]\, 8 = 1352\ mm^2$ $A_{tg} = 35 \times 8 = 280mm^2$ $A_{tn} = [40 - 0.5 \times 22]\, 8 = 232\ mm^2$ Block shear strength $T_{db1} = \frac{A_{vg} \cdot f_y}{\sqrt{3}\,\gamma_{mo}} + \frac{0.9\, A_{tn} \cdot f_y}{\gamma_{m1}}$	

Question and sub Question	Model Answer	Marks
	$= \dfrac{2320 \times 250}{\sqrt{3} \times 1.1} + \dfrac{0.9 \times 232 \times 410}{1.25}$	
	$T_{db1} = 304.421 + 68.486$	
	$\boxed{T_{db1} = 372.907\ kN}$	1 Mark.
	$T_{db2} = \dfrac{A_{tg} \cdot f_y}{\gamma_{m0}} + \dfrac{0.9 A_{vn} \cdot f_u}{\sqrt{3}\ \gamma_{m1}}$	
	$T_{db2} = \dfrac{280 \times 250}{1.1} + \dfrac{0.9 \times 1352 \times 410}{\sqrt{3} \times 1.25}$	
	$= 63.636 + 230.426$	
	$T_{db2} = 294.062\ kN$	
	For two Angles Block shear strength (T_{db})	
	$T_{db} = 294.062 \times 2$	
	$T_{db} = 588.124\ kN$	1 Mark
	Design Tensile strength for Double Angle Section = Minimum of T_{dg}, T_{dn}, T_{db}	
	$= 505.90\ kN$.	1 Mark.
	IF student has solved the problem by assuming no. of bolts, asses the problem for full Marks (marks are given Accordingly)	

Q.5 a) Panel point load in case of Live Load & Wind Load.

Given: L = span of Truss = 12 m

S = Spacing of Truss = 4 m c/c

R = Rise of Truss = 3 m

No. of Panels = 8

Slope of truss $\tan\theta = (R/L/2)$ ∴ $\theta = \tan^{-1}\left(\frac{3}{12/2}\right)$

∴ $\theta = 26.56^\circ$ (½ M)

∵ Panel point Plan Area = length of each panel × spacing of truss

$= \frac{12}{8} \times 4 = 6\ m^2$ (½ M)

& Panel point Actual Area $= \frac{6}{\cos\theta} = \frac{6}{\cos 26.56} = 6.71\ m^2$ (½ M)

Live Load / Imposed Load Calculations. (min. 400 N/m²)

LL on purlins $= 750 - (26.56 - 10)\,20 = 418.8\ N/m^2$ (1 M)

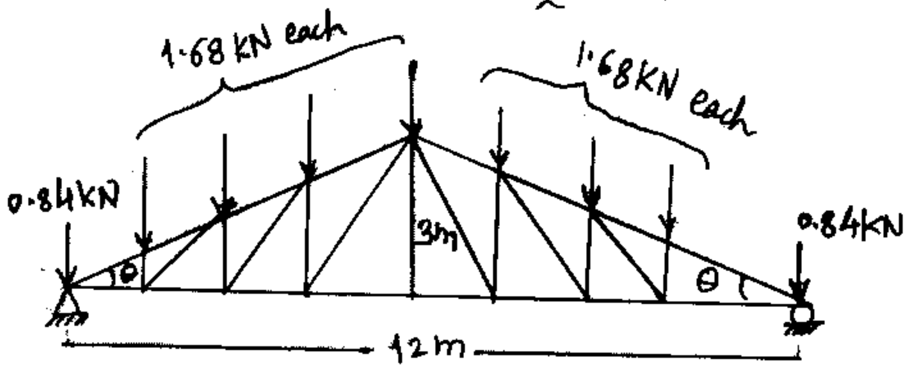
LL on truss supporting Purlins $= \left(\frac{2}{3}\right)(418.8) = 279.2\ N/m^2$ (½ M)

LL on each interior panel point = Load × plan Area

$= 279.2 \times 6$

$= 1675.2\ N = 1.68\ kN$ (½ M)

LL on end panel points $= \frac{1.68}{2} = 0.84\ kN$



LL on Roof Truss (½ M)

Wind load Calculations :-

Design Wind Pressure = P_d = 1.15 kPa = 1.15 kN/m^2 (½ m)

given :- Coeff. of external wind = C_{pe} = −0.7
& Coeff. of internal wind = C_{pi} = ±0.2

∴ Max. Wind pressure distribution on truss

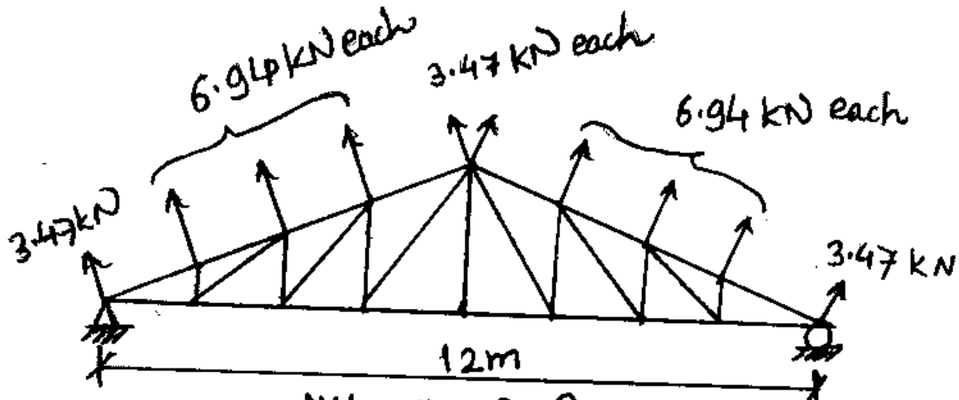
$$C_{pe} - C_{pi} = -0.7 - (+0.2) = -0.9$$
or
$$= -0.7 - (-0.2) = -0.5$$
∴ max $\underline{-0.9}$ (1 m)

∵ Panel point wind pressure on actual Area (intermediate Panels.)

$$F = [C_{pe} - C_{pi}] \cdot P_d \cdot A = -0.9 \times 1.15 \times 6.71$$ (1 m)

∴ $F = \underline{-6.94}$ kN. (uplift) (½ m)

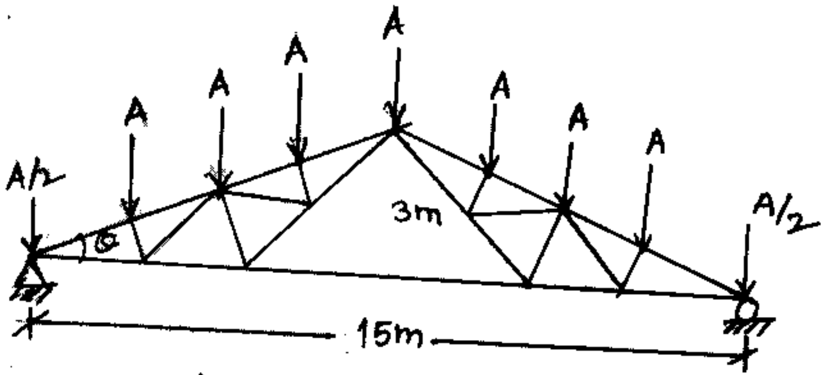
& on end panels $= \frac{F}{2} = \underline{-3.47}$ kN (uplift) (½ m)



WL on Roof Truss (½ m)

(* For any Truss of 8 pannel full marks are given)

	Marks
Q.5 b) DL & LL on Panel point of Fink Truss	
Given :- Hall of size = 15m x 30m.	
∴ Span of Truss = L = 15m	
Spacing of truss = s = 3.75m c/c	
Pitch of Truss = $\frac{1}{5} = \frac{R}{L}$	
∴ Slope of Truss $\theta = \tan^{-1}\left(2 \times \frac{1}{5}\right) = 21.80^{\circ}$	(½M)
* Assume above truss with 8 panels *	
Pannel point Plan Area = $\frac{15}{8} \times 3.75 = 7.03\,m^2$	(½M)
DL Calculations :-	
DL on plan area of Roof Truss is as bellow	
1) Wt. of roof covering material = 175 N/m^2	
2) Self wt. of purlin = 100 N/m^2	
3) Wt of bracing = 60 N/m^2	
4) Self wt of roof Truss $\left(\frac{Span}{3}+5\right)10$	(½M)
$= \left(\frac{15}{3}+5\right)10 = 100\ N/m^2$	(½M)
Total DL = 435 N/m^2.	(½M)
DL on each interior panel point = 7.03 x 435 = 3058.05N	(½M)
(A) = 3.06 kN	(½M)
DL on each end panel point = $\frac{3.06}{2}$ = 1.53 kN	(½M)
LL Calculations :- (min 400 N/m^2)	
LL on Purlins = 750 − (21.80 − 10) 20	(½M)
= 514 N/m^2.	(½M)
LL on Truss supporting purlin = $\frac{2}{3}(514)$	(½M)
= 342.62 N/m^2	(½M)

Question and Model Answers	Marks
LL on each interior Panels $=(A) = 7.03 \times 342.62$	
$\therefore$ $A = 2408.9\,N = 2.41\,kN$	(½m)
LL on each end panels $= \frac{2.41}{2} = 1.205\,kN$	(½m)
 DL & LL on Finks Truss	(1m)
for DL $(A) = 3.06\,kN$	
for LL $(A) = 2.41\,kN$	
* Note :- Any other assumption for no. of panels may change answer	
Q.5 c) Design of Slab base & Concrete Pedestal	
Given. Axial load on column $= 1600\,kN = P$	
$\therefore$ Factored Load $= P_u = 1.5 \times 1600 = 2400\,kN$	(½m)
A) Design of Base Plate :	
Required Base Area $= \frac{2400 \times 10^3}{(0.6 \times 20)} = \frac{\text{Factored load}}{\text{Bearing strength of concrete}}$	
$A_b = 200 \times 10^3\,mm^2$	(½m)
for equal Projections length of Base plate.	

$L_p = \left(\frac{400-250}{2}\right) + \sqrt{\left(\frac{400-250}{2}\right)^2 + (200\times10^3)}$ (½ m)

$L_p = 528.46\,mm \approx \underline{530\,mm}$ (½ m)

& width of base plate (B_p) :-

$B_p = \frac{A_p}{L_p} = \frac{200\times10^3}{530} = 377.35\,mm \approx 380\,mm$ (½ m)

for thickness of Base plate (t_p)

larger projection $= a = \frac{530-400}{2} = 65\,mm$

& smaller projection $= b = \frac{380-250}{2} = 65\,mm$

& net upward pressure on base plate $w = \frac{2400\times10^3}{(530\times380)}$

$\therefore w = 11.92\,N/mm^2$ (½ m)

$\because t_p = \sqrt{\frac{2.5\,w(a^2-0.3b^2)}{fy/1.1}}$ (½ m)

$\therefore t_p = \sqrt{\frac{2.5\times11.92\times(65^2-0.3(65)^2)}{250/1.1}}$

$\therefore t_p = 19.69\,mm > min\ t_f = 12.7\,mm$ ∴ ok. (½ m)

$\therefore \underline{t_p \approx 20\,mm}$

Size of Bearing or Base Plate $= (530\times380\times20)\,mm$ (½ m)

B] Concrete Pedestal Design :- for service load P = 1600 kN

Increase self wt. Appx. 10% & SBC of soil $200\,kN/m^2$

$\therefore$ Area of concrete pedestal $= \frac{(1600)\times1.1}{200} = 8.8\,m^2$ (½ m)

for equal projection, - length of concrete Pedestal

$L_c = \left(\frac{0.530-0.380}{2}\right) + \sqrt{\left(\frac{0.530-0.380}{2}\right)^2 + 8.8}$ (½ m)

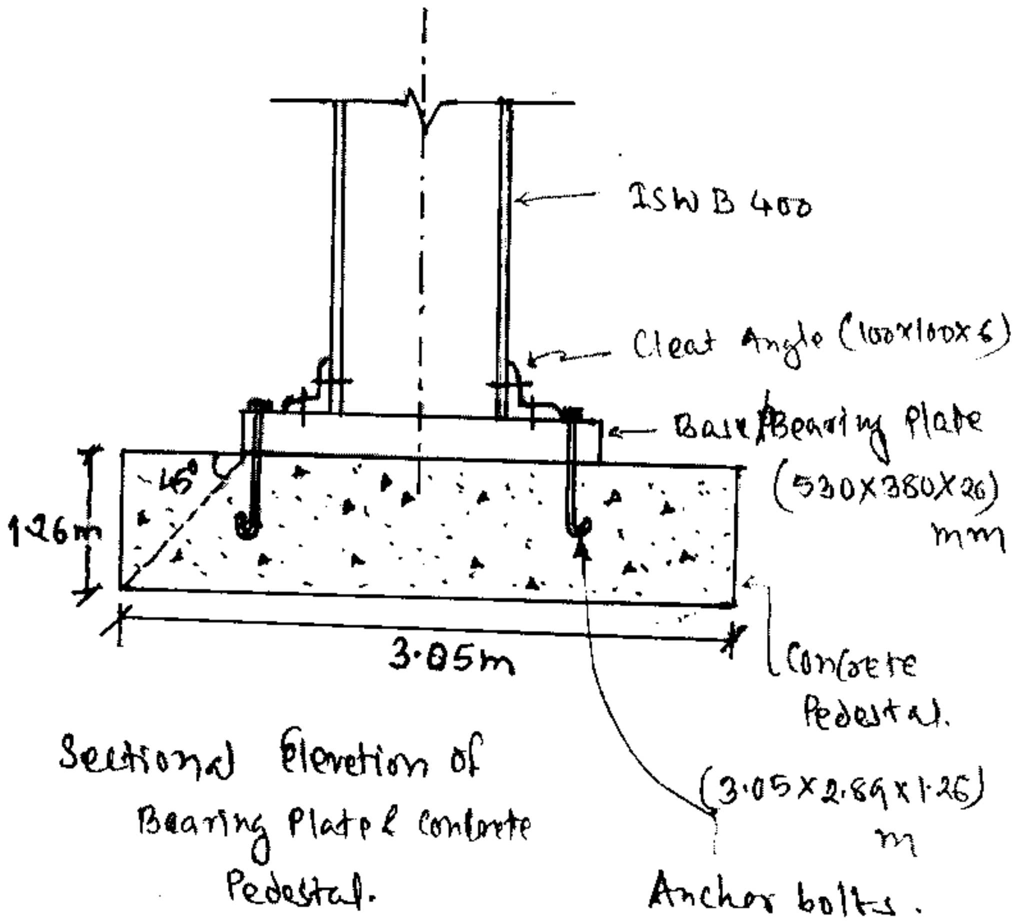
$\therefore L_c = 3.042m \approx \underline{3.05m}$ (½ m)

& width of concrete pedestal $B_c = \frac{8.8}{3.05} = 2.89m$ (½ m)

for Depth of concrete pedestal, using 45° distribution

D_c = Projection on larger side $= \left(\frac{3.05}{2} - \frac{0.530}{2}\right) = \underline{1.26m}$ (½ m)

Hence concrete pedestal is $\underline{(3.05 \times 2.89 \times 1.26)\,m}$



(1m)

Sectional Elevation of Bearing Plate & Concrete Pedestal.

Q.6 (a) Beams cross section classification.

Classification of cross section of Beam on moment-rotation behaviour is as -

(1) Plastic (Class-I) (2) Compact (Class-II) (2M) or ½M each

(3) Semi-compact (Class-III) (4) Slender (Class IV)

Explaination (any one) (2M)

(1) Plastic or Class-I - Develops plastic mechanism (plastic hinges with large rotation capacity)

($M \geq M_p$ & $\theta \geq \theta_p$)

Unaffected by local buckling.

(2) semicompact or Class-II :- Develops full plastic moment (M_p) but fails by local buckling due to inadequate rotation capacity (θ_p)

($M = M_p$ but $\theta < \theta_p$)

(3) Semicompact or Class-III :- Extreme fibres reaches the yield stress but local buckling prevents further moment (M_p)

($M = M_y$)

(4) Slender or Class IV :- Premature local buckling prevents yield moment also. ($M < M_y$)

6 (b) Steps for design of laterally supported Beam :-

Step 1 :- Calculate max bending moment & shear force i.e (M & V resp.) from factored load (½M)

Step: 2 :- Obtain Section Modulus (Z_p) required

$Z_{p\,reqd.} = \frac{f_y}{\gamma_{m0}} \times M$ where $\gamma_{m0} = 1.1$ ... PFOS (½ M)

Step 3:- Select suitable section so that Z_p provided $> Z_{p\,req}$ (½ M)

Step 4:- Check the section classification from b/t_f and d/t_w. (½ M)

Step 5:- Calculate design shear for web.

$V_d = \frac{f_y \times D \times t_w}{\gamma_{m0} \times \sqrt{3}}$ ... check $V_d > V$ and $V < 0.6 V_d$ (½ M)

Step 6:- Calculate moment resisted by the section

$M_d = \frac{\beta_b Z_p f_y}{\gamma_{m0}}$ $\beta_b = 1$ for plastic sections & compact sections (½ M)

$\beta_b = \frac{Z_e}{Z_p}$ for semicompact sections

Step 7: Check for deflection i.e. max deflection at service load < permissible values given. (½ M)

Step 8: Check for web buckling $[d/t_w < 67\epsilon]$ (½ M)

Q.6(c) 6 (c) Design bending strength of laterally supported beam (M_d)

Given- ISWB 400 @ 667.3.

Section classification:- $\epsilon = \sqrt{\frac{250}{f_y}} = 1$

$b/t_f = \frac{(200/2)}{13} = 7.69 < 9.4\epsilon$ ... plastic (½ M)

& $d = 400 - 2(t_f + R_1) = 400 - 2(13+13) = 348$ mm.

$\therefore d/t_w = \frac{348}{8.6} = 40.46 < 84\epsilon$ ... plastic (½ M)

$\therefore$ Beam section is 'plastic'

$\therefore \beta_b = 1$ (1 M)

Assuming $V < 0.6 V_d$, -

Design moment or Design bending strength (M_d)

$$M_d = \beta_b \cdot Z_p \cdot f_y / \gamma_{m0}$$ (½ m)

$$\therefore M_d = 1 \times 1240 \times 10^3 \times \frac{250}{1.1}$$

$$\therefore M_d = 281.82 \times 10^6 \text{ N.mm.}$$ (1m)

$$\therefore \underline{M_d = 281.82 \text{ kN.m}}$$ (½ m)

6(d) Basic concepts to decide plan area of slab base & concrete block.

① Base plate area should sufficiently bear the load coming from column & spread it on wider area (the intensity on concrete block within the limit of its bearing pressure). (1½ M)

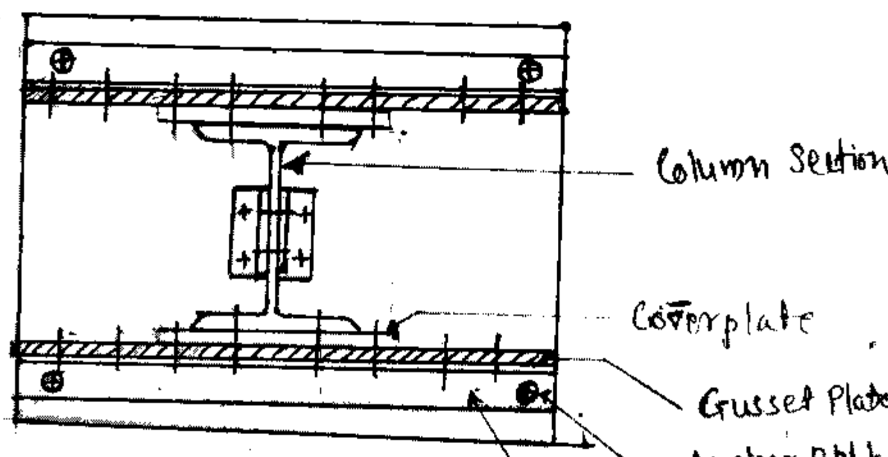
② The area of concrete block should be such that it will spread & transfer the load so that the intensity of bearing pressure of soil (SBC) is within its limit (1½ M)

③ Function of cleat angle :- To secure column section with base plate. (1)

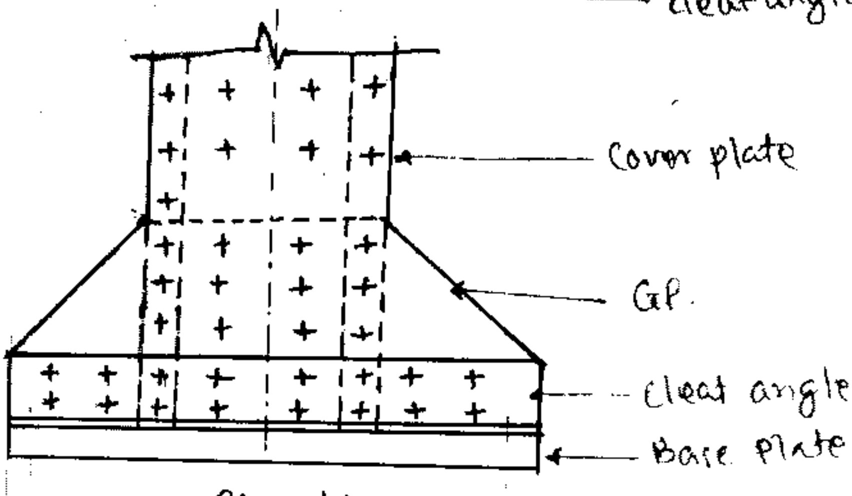
6(e) Gusseted Base:-

Defination:- The column base with gussete plates to transfer heavy loads or moment with axial loads within bearing limit is called as 'gusseted base'. (01 M)

Labelled sketch of Gusseted base:-



(1½ M)



(1½ M)