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 tl 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 ma 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.

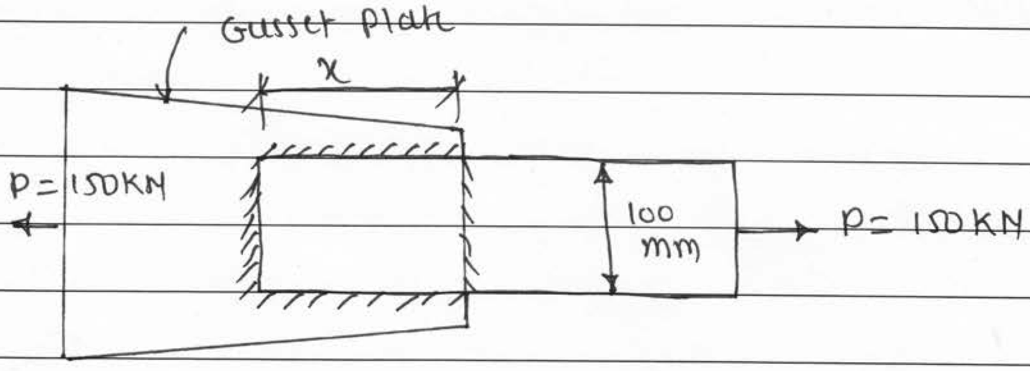
Important notes to examiner

Q .NO	SOLUTION	MARKS
	a)	
Q-1 A)	i) Advantage of steel	
	1) steel members have high strength per unit weight	01 M
	ii) steel being ductile material does not fail suddenly	each
	iii) Steel members are light in weight hence easy	Any
	to handle.	TWO
	iv) steel structure has long life	
	v) The properties of steel mostly not changes with	
	time	
	vi) steel has high scrap value among all building	
	material.	
	ii) Disadvantages	01 M
	i) steel structure are subjected to corrosion hence	each
	required frequent painting	Any
	ii) steel structures required fire proof treatment	TWO
	which increases the cost.	
	iii) Steel is costly material	
	iv) it required skill labour for errection	
	b) Different type of limit state	
	1) limit state of strength.	½ M
	2) limit state of serviceablity.	½ M
	1) limit state of strength	03 M
	the limit state of strength associated with failure	For
	under the action of probable and most unfavourable	any
	combination of factored loads on the structure	ONE

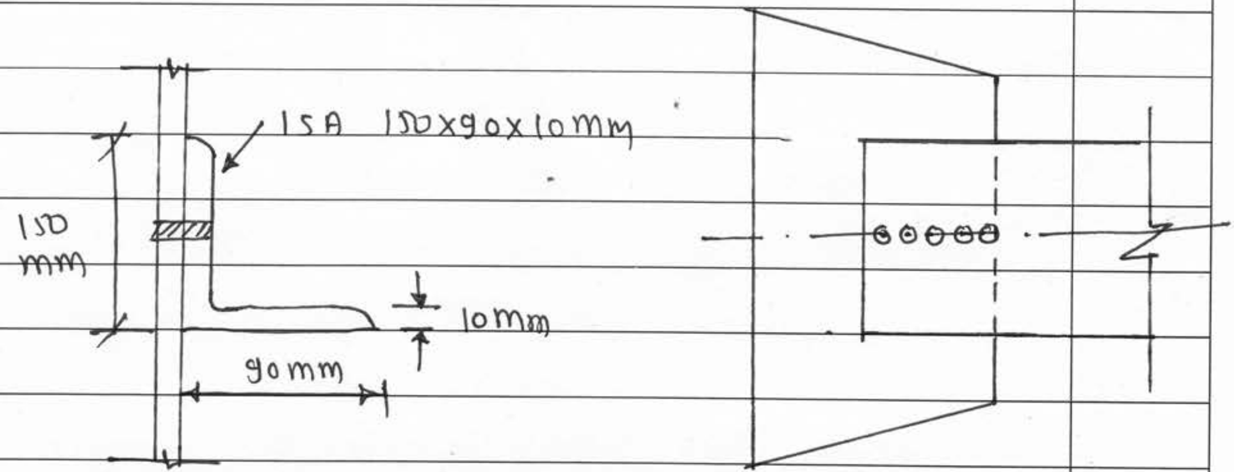
using the appropriate partial safety factors which may endanger the safety of life and property.

Q.NO	SOLUTION	MARKS
	limit state of strength includes	
	i) loss of equillibrium of the structure as a whole or any it's part's or components.	
	ii) loss of stability of the structure (including the effect of sway where appropriate and overturning) or any of it's parts including support's and foundation	
	iii) failure by excessive deformation rupture of the structure or any of it's part's or component	
	iv) ~~fact~~ fracture due to fatigue.	
	v) Brittle fracture	
	or	
	The limit state of serviceablity.	
	i) it includes Deformation and deflection which may adversely affect the appearance or effective use of the structure or may cause improper functioning of equipment or services or may cause damages to finishes and non structural members.	
	ii) vibrations in the structure or any of it's components causing discomfort to people, damages to the structure it's content's or which may limit it's functional effectiveness.	
	iii) Repairable damage or crack due to fatigue	
	iv) corrosion, durablity	
	v) fire	

Q.NO	SOLUTION	MARKS
c)	Types of Load with respective IS code	
	i) Dead load – IS 875-1987 part-I	01M
	ii) Live load – IS875-1987 part-II	for
	iii) Snow load – IS 875-1984 part IV	each
	iv) wind load – IS 875-1984 part IV	
	v) seismic force IS 1893-2002	
d)	List types of failure incase of Tension member	
	i) Due to yielding Gross section	
	ii) Rupture of Net section	
	iii) failure due to block shear (shear lag)	02M
	i) Gross-section yielding : consider deformation of the member in longitudinal direction may take place before it fracture making the structure unserviceable.	02M any one
	ii) Net section Rupture : The rupture of the member when the net cross section of the member reaches the ultimate stress	
	iii) Block shear failure : A segment of block of material at end of member shears out due to the possible use of high bearing strength of the steel and high-strength bolts resulting in smaller connection length.	

Q .NO	SOLUTION	MARKS
Q-1 B		
a)	given data	
	i) axial load = 100KN	
	ii) tie member 100 x 10 mm	
	iii) Shear stress in weld material = 108 Mpa	
		
	Assume gusset plate of 12mm thick	
	let S = size of weld	
	∴ minimum size of weld for 12mm thicker plate = 5 mm	
	also	
	maximum size of weld for 10mm thinner plate	
	= Thkness of Thinner plate − 1.5	
	= 10 − 1.5	
	= 8.5 mm	01M
	∴ we provide size of weld (s) = 6mm	
	[Note –: Students may assume s = 5 to 8 mm according to check the Answer sheet]	
	∴	
	Throat thickness (t) = 0.7 x size of weld	
	= 0.7 x 6	
	= 4.2mm	01M

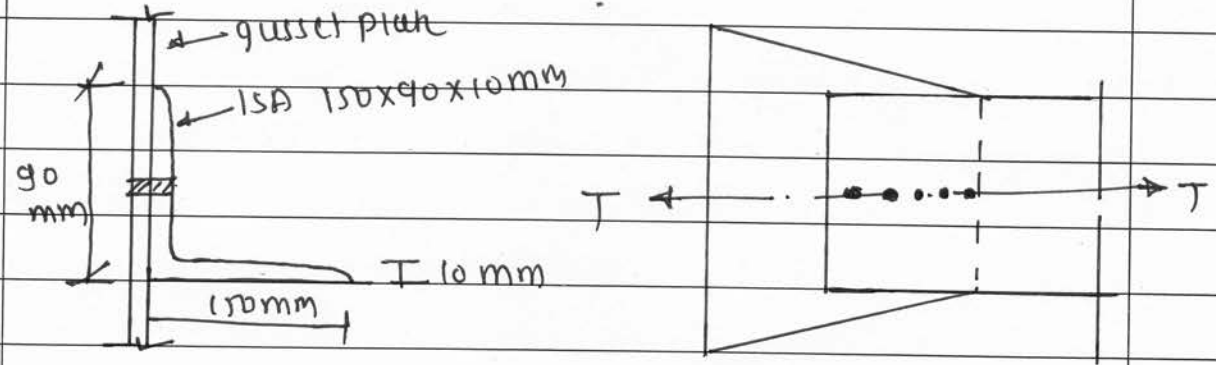
Q .NO	SOLUTION	MARKS
	factored load (P_{wd}) = 150 KN	
	= 150×10^3 N	01M
	for Fe 410 grade steel ---- (Assume grade)	
	ultimate stress (f_u) = 410 N/mm^2	
	let x = lap length	
	∴ Total length of weld (L) = x + x + 100 + 100	
	= 2x + 200	01M
	Now, we know that	
	Design strength of fillet weld = $t \times L \times \frac{f_y}{\sqrt{3} \times \gamma_{mw}}$	
	$150 \times 10^3 = 4.2 \times (2x+200) \times \frac{410}{\sqrt{3} \times 1.5}$	01M
	$\gamma_{mw} = 1.5$ --- (student may assume that)	
	$150 \times 10^3 = 662.79 \times (2x + 200)$	
	26.31 = 2x	
	x = 13.15 mm ---	
	Say **x = 15 mm**	01M
	Note –: Student may assume shopweld or site weld.	

Q .NO	SOLUTION	MARKS
b		
	Diameter of bolt = 18 mm	
	Diameter of hole (dh) = 18+2 = 20mm	
	∴ Net sectional area of connected leg (Anc)	
	$= (150 - t/2)\,t - dh \times t$	
	$= (150 - 10/2) \times 10 - 20 \times 10$	
	$= 1250\ mm^2$	½M
	Gross area of outstanding leg (Ago)	
	$= (90 - t/2) \times t$	
	$= (90 - 10/2) \times 10$	
	$= 850\ mm^2$	½M
	∴ Net sectional area (An) = Anc + Ago	
	$= 850 + 1250 = 2100\ mm^2$	½M
	Now find Net effective area Ane	
	∴ net effective area (Ane) = $\alpha \times An$	
	$= 0.8 \times 2100$	
	$= 1680\ mm^2$	
	∴ shear lag factor $\alpha = 0.8$ for No. of bolts ≥ 4	1M

Q .NO	SOLUTION	MARKS

ii) Case – II

When shorter leg connected to Gusset plate



Diameter of bolt = 18 mm

Diameter of hole $(d_h) = 18 + 2 = 20$ mm — ½M

Net sectional area of connected leg (A_{nc})

$= (90 - t/2) \times t - d_h \times t$

$= (90 - 10/2)\, 10 - 20 \times 10$

$= 650\ mm^2$ — ½M

Gross Area of outstanding leg (A_{go})

$= (150 - t/2) \times t$

$= (150 - 10/2) \times 10$

$= 1450\ mm^2$ — ½M

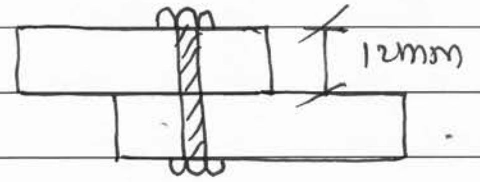
∴ Net sectional area $(A_n) = 650 + 1450 = 2100\ mm^2$ — ½M

∴ $A_{ne} = A_n \times \alpha$

$= 0.8 \times 2100$

$= 1680\ mm^2$ — ½M

Shear lag factor $\alpha = 0.8$ for number of bolt ≥ 4 — 1M

Q.NO	SOLUTION	MARKS
Q-2 (a)		
	Nominal diameter of bolt = 20mm	
	∴ 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 fe410 grade steel plate (assumed)	
	ultimate stress for plate $f_u = 410\ N/mm^2$	
	for 4.6 grade of bolt	
	ultimate stress for bolt $(f_{ub}) = 4 \times 100 = 400\ N/mm^2$	
	yield stress for bolt $(f_{yb}) = 400 \times 0.6 = 240\ N/mm^2$	
		01M
	Now find design shearing strength of bolt (V_{dsb})	
	∴ we know that	
	∴ $V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} [n_n \times A_{nb} + n_s + A_{ns}]$	½M
	Here number of shear plane with thread intercepting the shear plane $n_n = 1$	
	number of shear plane with out thread intercepting the shear plane $n_s = 0$	
	∴ $V_{dsb} = \frac{400}{\sqrt{3} \times 1.25} \times [1 \times 245.04 + 0]$	½M
	γ_{mb} = partial factor of safety for bolt material = 1.25	

Q .NO	SOLUTION	MARKS
	$V_{dsb} = 45.27 \times 10^3$ N	
	Now find Design bearing strength of bolt (V_{dpb})	
	$V_{dpb} = 2.5 \times K_b \times (d \times t) \times \frac{f_y}{\gamma_{mb}}$	½M
	Here coeff. K_b is minimum of	
	① $\left[\frac{e}{3d_h}, \frac{p}{3d_h} - 0.25, \frac{f_{ub}}{f_u}, 1 \right]$	½M
	a) Diameter of hole (d_h) = Nominal dia + 2	
	= 20 + 2	
	= 22 mm	½M
	b) End distance (e) = 2d = 2 × 20 = 40 mm	
	c) pitch (p) = 2.5d	
	= 2.5 × 20 = 50 mm	½M
	i) $\frac{e}{3d_h} = \frac{40}{3 \times 22} = 0.606$	
	ii) $\frac{p}{3d_h} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$	½M
	iii) $\frac{f_{ub}}{f_y} = \frac{400}{410} = 0.975$ & iv) 1	½M
	Hence $K_b = 0.507$ mm	
	take minimum value	
	Now find design bearing strength of bolt (V_{dpb})	
	$= 2.5 \times K_b \times (d \times t) \times \frac{f_y}{\gamma_{mb}}$	½M

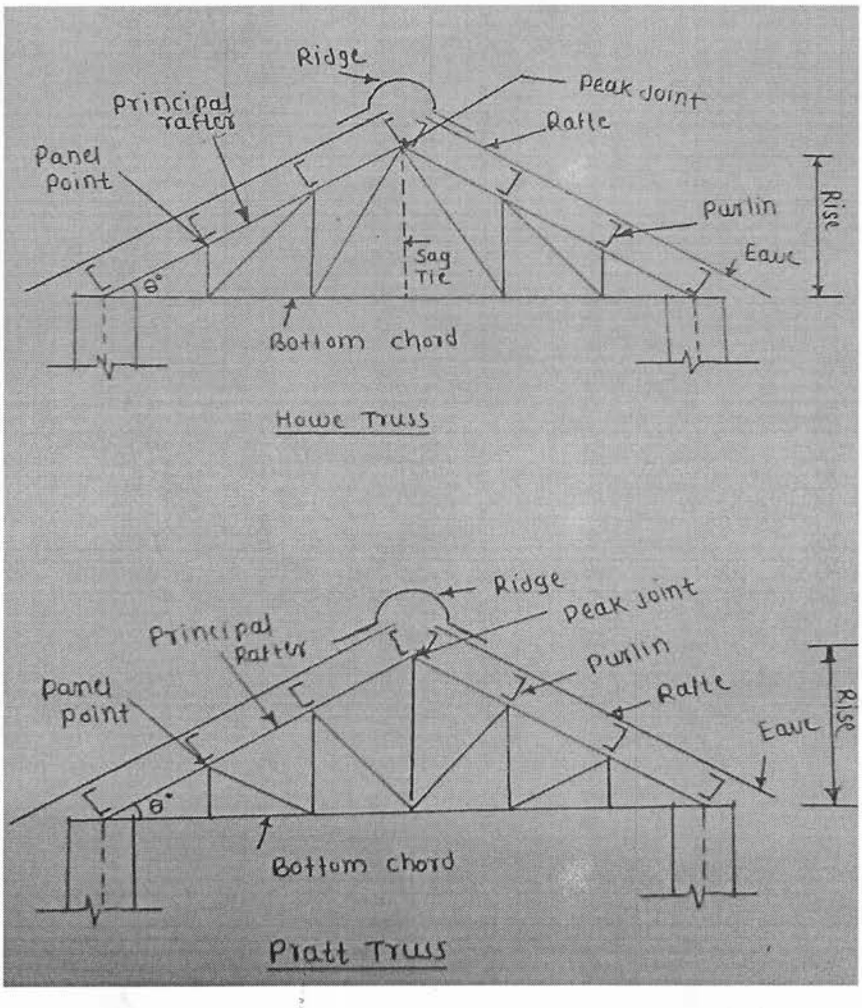
Q .NO	SOLUTION	MARKS
	$= 2.5 \times 0.507 \times (20 \times 12) \times \frac{410}{1.25}$	
	$V_{dpb} = 99.77 \times 10^3$ N	01M
	Now find bolt value i.e. strength of bolt	
	∴ Bolt value = minimum strength bet^n shearing & bearing strength of bolt	
	i.e. minimum bet^n V_{dsb} & V_{dpb}	
	$= 45.27 \times 10^3$ N	½M
	full strength of member $= 0.9 \times \frac{f_u}{\gamma_m} \times$ Area of plate	
	$= \frac{0.9 \times 410}{1.25} (250 - 1 \times 22) \times 12$	
	$= 630.54 \times 10^3$ N	½M
	full strength of plate	
	∴ No. of bolts $= \frac{\text{full strength of plate}}{\text{Bolt value}}$	
	$= \frac{630.54 \times 10^3}{45.27 \times 10^3}$	
	$= 13.92$ say 14 Nos	½M

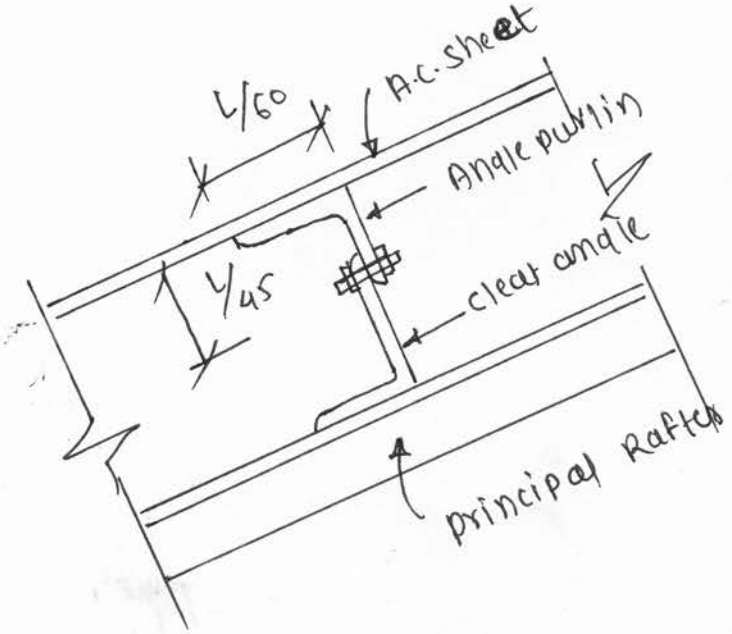
Q .NO	SOLUTION	MARKS
b]	Actual length (L) = 2.4 = 2400 mm	
	∴ effective length (ℓ) = KL	
	= 0.7 × L · for angle connected by weld	
	= 0.7 × 2400	
	ℓ = 1680 mm	1/2M
	least radius of gyration for single angle = r_{min} = r_{vv}	
	∴ $r_{min} = r_{vv} = 17.5$ mm	
	Now find slenderness ratio (λ)	
	$\lambda = \frac{\text{eff. length}}{r_{min}}$	1/2M
	$= \frac{1680}{17.5}$	
	$\lambda = 96$	1/2M
	Now find design compressive stress (f_{cd}) for buckling class 'c' from table of slenderness ratio	
	SR λ | f_{cd} (N/mm²) — 90 | 121; 96 | ?; 100 | 107	
	By interpolation $\frac{121-107}{100-90} \times (100-96) + 107$	
	$\lambda = 112.6$ N/mm²	1/2M
	Now find design strength of strut (P_d)	
	$P_d = A_g \times f_{cd}$	
	$= 1047 \times 112.6$	
	$P_d = 117.89 \times 10^3$ N	1/2M

Q .NO	SOLUTION	MARKS
(c)	working UDL = 15 KN/m = 15 N/mm	
	effective span (l_e) = 6m = 6000 mm	
	∴ factored load (W_u) = 15×1.5	
	= 22.5 N	½M
	Now find factored moment (M_u)	
	$= \frac{W_u \times l_e^2}{8}$	
	$= \frac{22.5 \times 6000^2}{8}$	
	$= 101.25 \times 10^6$ N/mm	½M
	Now find factored shear force V	
	$V = \frac{W_u \times l_e}{2}$	
	$= \frac{22.5 \times 6000}{2}$	
	$= 67.5 \times 10^3$ N	½M
	Now find required section modulus Z_{px} or Z_e	
	$Z_{p\,req} = \frac{M_u \cdot \gamma_{mo}}{f_y} = \frac{101.25 \times 10^6 \times 1.1}{250}$	
	$= 445.5 \times 10^3\ mm^3 < Z_p$ available	
	$Z_{e\,req} = \frac{445.5 \times 10^3}{1.14} = 390.78 \times 10^3\ mm^3$	
	$< Z_e$ or Z_{px} available	
	--- OK	02M

Q .NO	SOLUTION	MARKS
	* Check for shear	
	Here design shear strength of section (vd)	
	$= 0.525 \times fy \ (tw \times h)$	
	$= 0.525 \times 250 \times 6.9 \times 250$	
	$= 226.40 \times 10^3$ N	
	As design shear strength (vd) > shear strength (V)	
	Hence safe in shear	01M
	* Check for deflection	
	$\delta allowable = \frac{le}{300} = \frac{6000}{300} = 20mm$	
	Now find maximum deflection (δmax)	
	$\delta max = \frac{5 w le^4}{384 EI_{xx}}$	
	$= \frac{5 \times 15 \times 6000^4}{384 \times 2 \times 10^5 \times 5131.6 \times 10^4}$	
	$= 24.66$ mm	
	As $\delta max > \delta allowable$	
	Hence section is not safe in deflection	
	conclusion : the given ISMB-250 section is not safe hence not suitable.	01M

Q .NO	SOLUTION	MARKS
Q-3	**Attempt any Four of the following**	**16**
a)	State types of bolted joints and types of failure in case of bolted joints.	**04M**
	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	01M 01M 01/2 M each any Two 01/2 M each any Two
b	State two advantages of welded joints and two disadvantages of bolted joints.	**04M**
	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 joints. 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.	01 M each any Two

Q .NO	SOLUTION	MARKS
b) Cont.	**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.	01 M each any Two
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.	04M
		02M 02M

Q .NO	SOLUTION	MARKS
(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.	04M
	Types of loads • Dead Load • Imposed load or live load • Snow load • Wind load • Earth quack load	02M
	Load Combinations The following combination of loads with appropriate Partial safety factors (see Table 4) maybe 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.	02M
(e)	**Draw a neat sketch and label of an angle Purlin with principal rafter at Panel Point having root covering is A.C. sheets.**	04M
		04M

Q .NO	SOLUTION	MAR
Q-4	**Attempt any Three of the following**	12M
(a)	**Define stanchion and column. Draw a neat sketch of any one section used showing dimensions with usual notations.**	04M

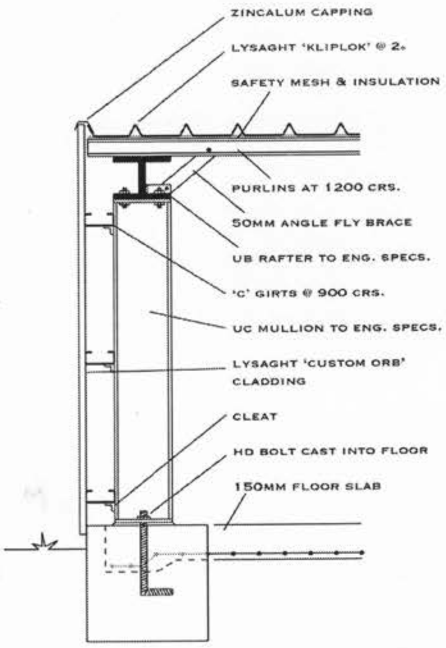
Define stanchion-: an upright bar, post, or frame forming a support or barrier. 01M

Define column -: Column or pillar in architecture and structural engineering is a structural element that transmits, through compression, the weight of the structure above to other structural elements below. 01M

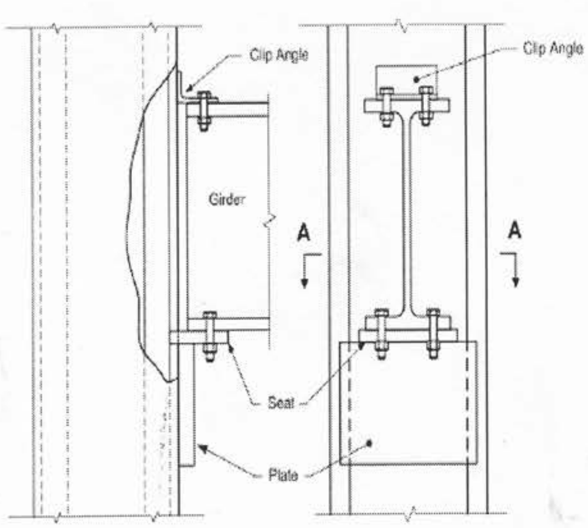
Draw a neat sketch of any one section used showing dimensions with usual notations.

02M ANY ONE FIG

i) sketch of stanchion



ii) sketch of Column



Q .NO	SOLUTION	MARKS
b)	**Define Radius of gyration and Slenderness Ratio. Also state maximum values of slenderness ratio for any two conditions of compression member.**	04M
	i. **Radius of gyration** or gyrations refers to the distribution of the components of an object around an axis. In terms of mass moment of inertia, it is the perpendicular distance from the axis of rotation to a point mass (of mass, m) that gives an equivalent inertia to the original object(s) (of mass, m).	01M
	ii. Slenderness ratio is the ratio of the length of a column and the least radius of gyration of its cross section.	01M
	state maximum values of slenderness ratio	01M FOR EACH WRITE ANY TWO

Table 7.4 *Maximum Slenderness Ratio (λ) for Compression Members*

S. No.	Type of Member	λ
1.	A tension member in which a reversal of direct stress occurs due to loads other than wind or seismic forces.	180
2.	A member carrying compressive loads resulting from dead loads and imposed loads.	180
3.	A member subjected to compressive forces resulting only from combination with wind/earthquake actions, provided the deformation of such members does not adversely effect the stress in any part of the structure.	250
4.	Compression flange of a beam restrained against lateral torsional buckling.	300
5.	A member normally acting as a tie in a roof truss or a bracing system is not considered effective when subjected to possible reversal of stresses resulting from the action of wind or earthquake forces.*	350

Q .NO	SOLUTION	MARKS
Q-4 (A) c)	Explain limit of width to thickness ratio to prevent buckling for a single angle strut, the limiting width to thickness ratio for a semi-compact class is 15.7ϵ. check whether. ISA 90x90x6 mm is of semi-compact class or not fy = 250 Mpa.	
→	compression element — Ratio	
	single angle, or double angles with the components — b/t, d/t	
	ISA 90x90x6 mm $A_g = 1047\ mm^2$, b = 90, d = 90, t = 6mm	
	$b/t = \frac{90}{6} = 15 < 15.7\epsilon$	01$\frac{1}{2}$M
	$d/t = \frac{90}{6} = 15 < 15.7\epsilon$	01$\frac{1}{2}$M
	The given section is in semi compacted class.	

Q .NO	SOLUTION	MARKS
(d)	**Draw a neat labeled sketch of lacing system and state requirements of Lacing to be used.**	04M

Draw a neat labeled sketch — 02M

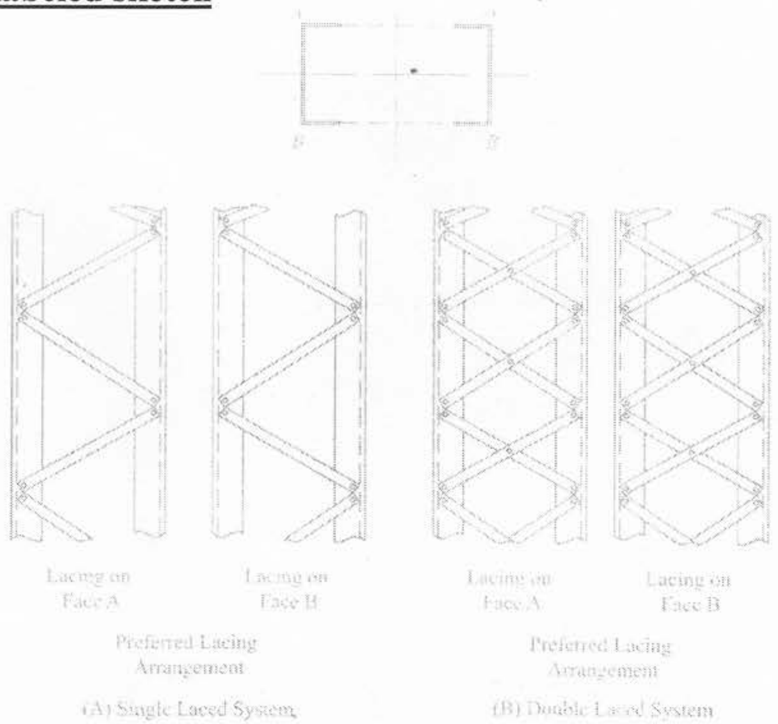


Figure 6.5 Laced column

state requirements of — 01M EACH WRITE ANY TWO

Lacing to be used.

Width of Lacing Bars

In bolted/riveted construction, the minimum width of lacing bars shall be three times the nominal diameter of the end bolts rivet.

Thickness of Lacing Bars

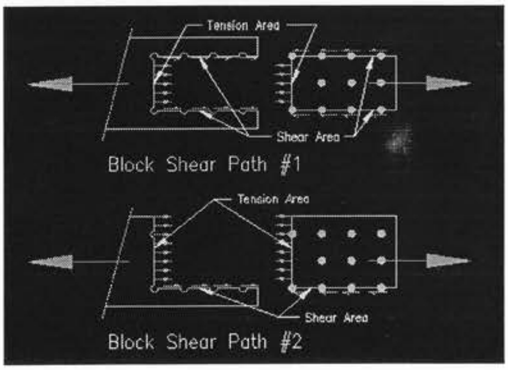
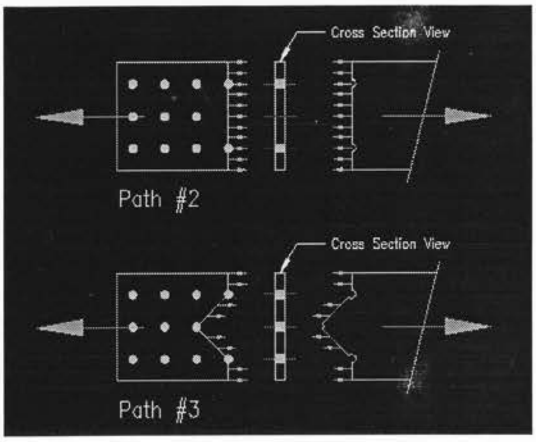
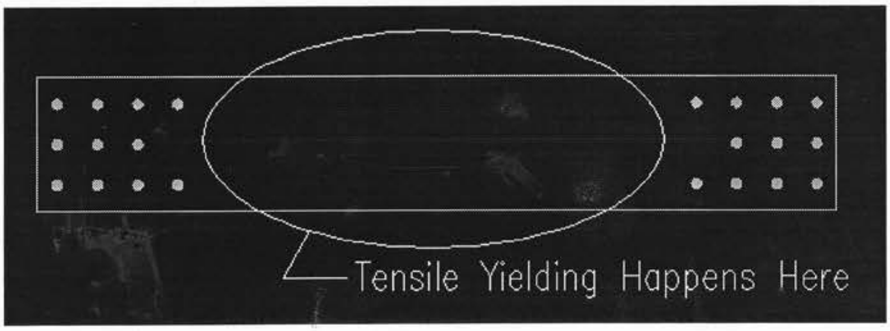
The thickness of flat lacing bars shall not be less than one-fortieth of its effective length for single lacings and one-sixtieth of the effective length for double lacings.

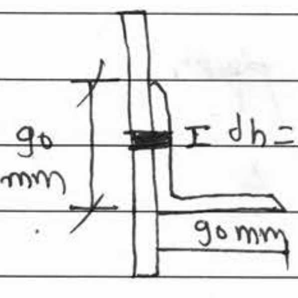
Angle of Inclination

Lacing bars, whether in double. Or single systems, shall be inclined at an angle not less than 40° nor more than 70°

Spacing

The maximum spacing of lacing bars, whether connected by bolting, riveting or welding, shall also be such that the maximum slenderness ratio of the components of the main member (al/rl), between consecutive lacing connections is not greater than 50 or 0.7 times the most unfavorable slenderness ratio of the member as a whole, whichever is less, where al is the unsupported length of the individual member between lacing points, and "r" is the minimum radius of gyration of the individual member being laced together

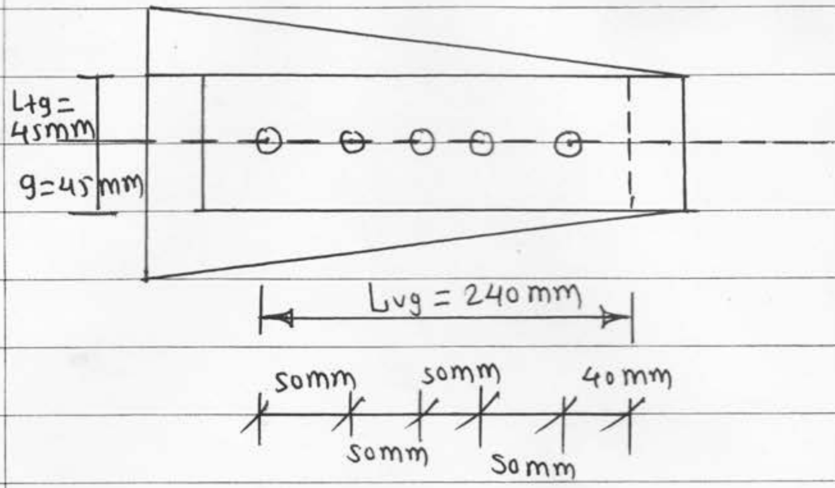
Q .NO	SOLUTION	MARKS
B)	**Attempt any one**	**06M**
a)	**Draw sketches of three different modes of failure in case of members Subjected to Axial Tension.**	**06M**
	Three different modes of failure in case of members Subjected to Axial Tension. **i) Block Shear**  **ii) Net Section rupture**  **iii) Gross section yielding** 	**02M FOR EACH FIG.**

Q .NO	SOLUTION	MARKS
Q-4 (b)		
	Step-I) Calculate A_g req^d =	
	$A_{g\,req^d} = 1.15\left(\frac{T_d}{0.91 f_y}\right) = 1.15\left(\frac{210\times10^3}{0.91\times250}\right) = 1061.53\ mm^2$	½ M
	Step-II) Selection of Trial section	
	Select ISA 90×90×8 mm having	
	$A_g = 1137\ mm^2$	
	Step-III) Nos of bolts req^d	
	Bolt value = 45.3 KN - - (given)	
	Nos. of bolt = 210/45.3 = 4.63 ≃ 5 Nos	½ M
	Step-IV) Check the strength of section for 90×90×8	
	a) Design strength due to yielding of gross-section	
	$T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}}$	
	$= \frac{1137\times250}{1.10}$	
	$T_{dg} = 258.40\ KN$	½ M
	b) Design strength due to rupture of critical section	
		

Q .NO	SOLUTION	MARKS
	$T_{dn} = \alpha A_n \frac{f_u}{\gamma_{m1}}$	½M
	$A_n = A_{nc} + A_{go}$	
	$A_{nc} = \left(B_1 - d_h - \frac{t}{2}\right) \times t$	
	$A_{nc} = \left(90 - 22 - \frac{8}{2}\right) \times 8$	
	$A_{nc} = 512\ mm^2$	
	$A_{go} = \left(B_1 - \frac{t}{2}\right) t$	
	$A_{go} = \left(90 - \frac{8}{2}\right) 8$	
	$A_{go} = 688\ mm^2$	
	$A_n = A_{nc} + A_{go} = 512 + 688$	
	$A_n = 1200 mm^2$	½M
	$T_{dn} = \alpha \cdot A_n \frac{f_u}{\gamma_{m1}}$ $\alpha = 0.8$ Assuming more than 4 bolts	
	$T_{dn} = \frac{0.8 \times 1200 \times 410}{1.25}$	
	$T_{dn} = 314.88\ KN$	½M

Q.NO	SOLUTION	MARKS

c) Design strength due to Block c/c shear



Assume edge distance = 40mm

$g = 45$ mm , $L_{tg} = 45$ mm

spacing of bolt = 50mm

$A_{vg} = 240 \times 8 = 1920 \text{ mm}^2$ — ½M

$A_{vn} = \{ L_{vg} - [\text{numbers of bolts} - 0.5 d_h] \} t$

$= \{ 240 - [(5 - 0.5) 22] \} \times 8$

$A_{vn} = 272 \text{ mm}^2$ — ½M

Q .NO	SOLUTION	MARKS
	$A_{tg} = L_{tg} \times t = 45 \times 8 = 360\ mm^2$	
	$\boxed{A_{tg} = 360\ mm^2}$	
	$A_{tn} = (L_{tg} - 0.5\,d_h) \times t$	
	$= (45 - 0.5 \times 22) \times 8$	
	$\boxed{A_{tn} = 272\ mm^2}$	½M
	$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3} \times \gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$	
	$= \frac{1920 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 272 \times 410}{1.25}$	
	$= \frac{480000}{1.73 \times 1.10} + \frac{100368}{1.125}$	
	$\boxed{T_{db1} = 341.156\ KN}$	½M
	$T_{db2} = \frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{mo}}$	
	$= \frac{0.9 \times 1128 \times 410}{\sqrt{3} \times 1.25} + \frac{360 \times 250}{1.10}$	
	$\boxed{T_{db2} = 274.07\ KN}$	½M
	T_{db} = less of T_{db1} & T_{db2}	
	∴ So take value $\boxed{T_{db} = 274.07\ KN}$	
	The tensile strength of angle = lesser of T_{dg}, T_{dn} & T_{db} (258.40, 314.88 & 274.07)	
	∴ = <u>258.40 KN</u>	½M

This is greater than required 210 KN < 258.40 KN

---- <u>OK</u>

Q .NO	SOLUTION	MARKS
Q 5	**Attempt any two**	**8x2=16**
a)	A column ISMB 300 @ 46.1 Kg/m carries an axial load of 1200 KN. Design a slab base and concrete pedestal for column. The SBC of soil is 180 kN/m^2 . M20 – Concrete is used for concrete pedestal. For ISMB 300 b_f =140 mm, t_f = 13.1 mm, fy = 250 MPa, fu = 410 MPa r_{mo} = 1.1. Draw a neat sketch as per designed details. Solution: Given P = 1200 KN 1) Calculate the required area(A) of base base plate A = column load/ 0.6 f_{ck} A = $(1200 \times 10^3) / (0.6 \times 20) = 100 \times 10^3\ mm^2$.	
		1/2M
	2) To find the size of base plate. L_P & B_p be the sizes of plate D =length or longer length = 300 mm B = width or shorter side of the column = 140 mm. $L_P = (D-B)/2 + \sqrt{[\{(D-B)/2\}^2 + A]}$ $L_P = (300-140)/2 + \sqrt{[\{(300-140)^2/2 + 100 \times 10^3}$ = 406.19= say 410 mm. $B_p = A/L_p = 100 \times 10^3 / 410$ = 243.90 mm say 245mm Larger projection a = $(L_P - D)/2$ = (410 – 300)/2= 55 mm. Shorter projection b = $(B_P - B)/2$ = (243.90 – 140)/2 = 52mm. Area of base plate provided = $L_P \times B_p$ = (D + 2a) x (B + 2b) = (300+2x55) x (140+2x52) = $100 \times 10^3\ mm^2$.	**02M**
	3) Calculate ultimate bearing pressure $w = P/(L_P \times B_p) = 1200 \times 10^3 / (410 \times 245) = 11.95\ N/mm^2$	**1/2M**
	4) Calculate thickness of base plate $t_s = [(2.5 \times w (a^2 - 0.3 \times b^2)\ r_{mo} / fy)]^{0.5}$	

$= [(2.5 \times 12 (55^2 - 0.3 \times 52^2)\ 1.1/245)]^{0.5}\ 17.26$ = say 18 mm t_f = 13.1 mm. **01M**

Size of base plate provided 410 x 245 x 18 mm

5) Calculate the size of concrete pedestal

Area of concrete pedestal = 1.1 P/(SBC X 1.5)

= 1.1 X 1200 /(180 X 1.5) = 4.88 m^2

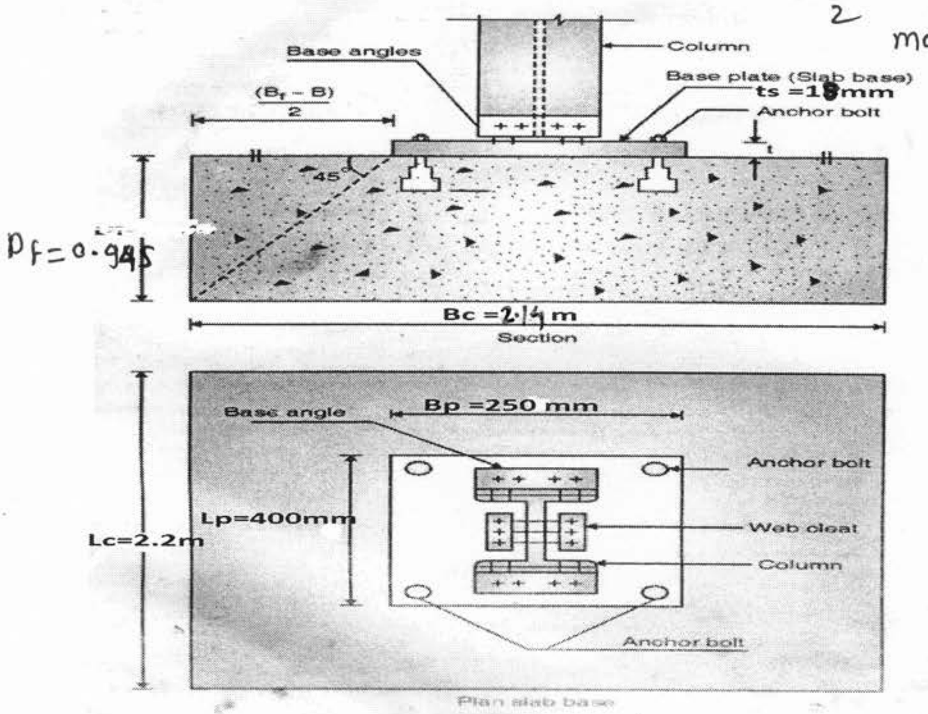
$$L_f = \frac{L_p - B_p}{2} + \sqrt{\left(\frac{L_o - B_p}{2}\right)^2 + A_f} = \frac{0.4 - 0.250}{2} + \sqrt{\left(\frac{0.4-0.25}{2}\right)^2 + 4.88}$$

$L_f = 2.28$ m

$B_f = \frac{A_f}{2.28} = \frac{4.88}{2.28} = 2.14$ m, $D_f = \frac{L_f - L_p}{2} = 940$ mm or $\frac{B_f - B_p}{2} = 945$ mm **02M**

max $D_f = 945$ mm

6) Neat sketch



02M

Note : Student can also take square shape of base plate. Accordingly full marks should be given

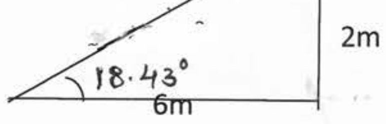
b) An industrial bldg. has Howe roof truss having 12 m span. Take A/C sheet covering weighing 175 N/m^2 , eight panel lengths along the tie member, pitch of roof =1/6 and weight of purlin is 55 N/m^2. Assume self wt of truss as 90 N/m^2. Calculate the panel point loads for dead load and live load.

Solution :

Span = 12 m

Pitch of roof = 1/6

Rise = pitch x span = 1/6 x12 = 2m.



	Ø =18.43°	01M
	Sloping length = √ ($6^2 + 2^2$) = 6.32 m	
	We have 8/2 = 4 panel along each sloping length.	
	c/c spacing of purlins at panel points = 6.32/4 = 1.58 m.	01M
	Assume spacing of truss = 4m	
	A) Dead load calculation	
	i) Weight of A/C sheet = 175 N/m²	
	ii) Weight of purlin = 55 N/m²	
	iii) Self wt of truss = 90 N/m²	
	Total Dead load = i+ii+iii = 175+55+90 = 320 N/m²	
	spacing of trusses = 4m ....assumption	01M
	Length of each panel = 1.58 x cos 18.43 = 1.5 m	
	Load at each panel point = 320 x plan area =320 x (1.5 x 4)/1000	01M
	=1.92 KN	½M
	Load at each panel point = 1.92/2 = 0.96 kN.	
	B) Live load calculation	½M
	As slopeis 10^0 , the imposed load on purlin is 750 N/m² less 20 N/m² for every degree increase in slope in excess of 10^0 but > 400 N/m²	
	Imposed load on purlin = 750- (18.43-10) x 20	
	= 581.4 N/m² > 400 N/m²	01M
	Imposed load on truss supporting purlins = (2/3) x 581.4 =387.6 N/m²	
	Imposed load at each interior panel = 387.6 x (1.41 X 4)/1000	01
	= 2.186 KN.	½M
	Imposed load at each end panel = 2.186/2 = 1.093 KN.	
	Note: as c/c spacing of trusses is not mentioned, student can take any appropriate value, accordingly full marks should be given.	½M
C)	Find the wind load per panel point for designing a roof truss of span 10 m and pitch as ¼. The height of eaves is 5 m above ground. Assume V_b = 4 m/s, probability factor K_1 = 1, Size factor K_2 = 0.8, topography factor K_3 =1.	

Solution:

Rise = ¼ x 10 = 2.5 m

tan θ = 2.5/5 = 0.5,

θ = 26.56^0 ½M

I) wind load calculation

$V_Z = V_b \times K_1 \times K_2 \times K_3$

= 4 X 1 X 0.8 X 1

= 3.2 m/s ½M

II) Design wind speed

$P_d = 0.6\,(V_Z)^2$

$= 0.6 \times 3.2^2$

= 6.144 N/m^2

III) Wind normal to ridge ½M

IV) Wind force calculation

Normal permeability C_{pi} = +- 0.2 ....Given

To find C_{pe}

building height ratio h/w =5/10 =0.5 ½M

for θ = 26.56° h/w=0.5

Cpe Note full mark's give to student's if up to above steps

h/w	Roof angle θ	Wind angle 0°		Wind angle 90°	
		EF	GH	EG	FH
0.5	20°	-0.4	-0.4	-0.7	-0.6
	30°	0	-0.4	-0.7	-0.6

01M

1. Wind normal to ridge, θ = 0°

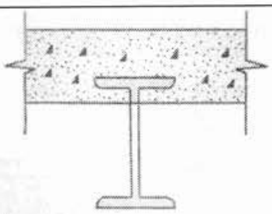
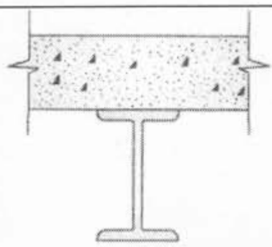
for θ = 20°, Cpe =-0.4 and θ = 30°, Cpe = 0

by interpolation Cpe = -0.1376 for θ = 26.56°

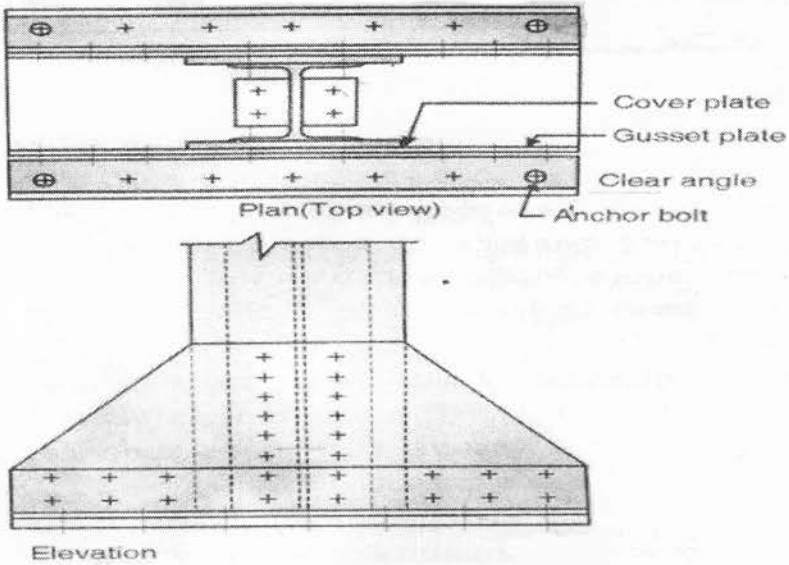
	Wind force = F = (Cpe – Cpi) x P_d = (-0.1376 - 0.2) x 6.144 = - 2.07 (Upward pressure) = (-0.1376 + 0.2) x 6.144 = + 0.38 (Downward pressure) = (- 0.4 -0.2) x 6.144 = - 3.68 (Upward pressure) = (- 0.4 + 0.2) x 6.144 = -1.22 (Upward pressure) 2. Wind parallel to ridge $\theta = 90^\circ$ Wind force = F = (Cpe – Cpi) x P_d = (- 0.7 – 0.2) x 6.144 = - 5.52 (Upward pressure) = (-0.7 + 0.2) x 6.144 = - 3.07 (Upward pressure) = (-0.6 – 0.2) x 6.144 = - 4.91 (Upward pressure) = (-0.6 + 0.2) x 6.144 = - 2.45 (Upward pressure) Design wind pressure maximum value + 0.38 (Downward pressure) - 5.52 (Upward pressure) as c/c spacing of trusses are not given. Take c/c spacing as 4 m ....Assumption Then sloping line will be = $[5^2+2.5^2]$ = 5.59 m. Then plan area = 5.59 x 4 =22.36 m^2. Downward wind load on each intermediate panel = wind pressure x area exposed = 0.38 x 22.36 = 8.49 N Downward wind load on each end panel = 8.49/2 =4.24 N. Upward wind load on each intermediate panel = 5.52 x 22.36 = 123.42 N Upward wind load on each end panel = 123.42/2 =61.71 N. **Note: as Cpe values are not given student may write Cpe value more or less nearer to actual value, full marks should be given. Also spacing is not mentioned, student can take any appropriate value, accordingly full marks should be given.**	01M 01M ½M 01M ½M 01M

Q.6	Attempt any four	4x4 =16
a)	Write the steps to calculate the thickness of base plate used in slab base. Why anchor bolts are used in slab base.	
	Solution: 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 $L_P = (D-B)/2 + \sqrt{[\{(D-B)/2\}^2 + A]}$ $B_p = A/L_p$ Larger projection $a = (L_P - D)/2$ Shorter projection $b = (B_P - B)/2$ Area of base plate provided = $L_P \times B_p = (D + 2a) \times (B + 2b)$ 3) Calculate ultimate bearing pressure $w = P/(L_P \times B_p)$ 4) Calculate thickness of base plate $t_s = [(2.5 \times w\ (a^2 - 0.3 \times b^2)\ \gamma mo / fy)]^{0.5}$ **Function of anchor bolt** : To connect concrete pedestal and base plate anchor bolts are used.	 01M 01M 01M 01M

b)	Differentiate between Laterally supported and unsupported beam with neat sketch showing all details	04M

Laterally supported beam	Laterally unsupported beam	
In laterally supported beam, compression flanges are embedded in concrete	In laterally unsupported beam, compression flanges are not embedded in concrete	01M
Compression flange of Beam is restrained against rotation	Compression flange of Beam is free for rotation	01M
Lateral deflection of compression flange is not occur	Lateral deflection of compression flange is is occur	01M
 Laterally supported. (It means compression flange is restrained)	 Laterally unsupported.	01M

c)	Define Gusseted base. Also draw its neat lebelled sketch showing all details. Defination: 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.	01M

	 **Fig Gusseted Base.**	03M
d)	How beam section are classified for bending as per IS 800:2007. Describe any two of them.	04M
	Solution: Classification beam: 1) Plastic or class-I 2) Compact or class –II 3) Semi compact or class – III 4) Slender or class- IV Explanation 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 semiplastic 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.	02 01 for each (any two)

e)	A simply supported beam of 6 m span supports on RCC slab where in comp. flange is embedded. The beam is subjected to a dead of 25 KN/m and super imposed load of 20 KN/m, over entire span. Calculate plastic and elastic modulus required. Assume r_f = 1.5, r_m = 1.1 f_y =250 N/mm^2	04M
	Solution: 1)Calculation of factored load Dead load = 1.5 x 25 =37.5 KN/m Live Load = 1.5 x 20 = 30 KN/m 2) Calculate Maximum bending moment and shear force B.M =$WL^2/8$ + $WL^2/8$ =37.5 x $6^2/8$ + 30 x $6^2/8$ =303.75 KN.m S.F =WL/2 +WL/2 = 37.5 X 6/2 +30 X 6/2 = 202.5 KN.m 3) Plastic modulus = Z_p =M X r_{mo}/f_y =303.75 X 10^6x1.1 /250 =1.3365 x 10^6 mm^3 4) Elastic modulus = Z_e = Z_p/1.14 =1.3365 x 10^6 /1.14 =1.17236 x $10^6 mm^3$	01 02 ½ ½