WINTER – 14 EXAMINATIONS

Subject Code: 17505 **Model Answer** **Total Pages: 21**

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.

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 b) i) In this question the edge distance (e) and gauge distance (g) are not given, therefore the student may assume other values than given in this solution are also correct. Answers may be assessed accordingly.

Q.1 b) ii) In this question the edge distance (e) and gauge distance (g) are not given, therefore the student may assume other values than given in this solution are also correct. Answers may be assessed accordingly.

Q.2 a) Value of edge distance (e) is not given, student may solve question by using other assume value of e. Answer shall be assessed accordingly.

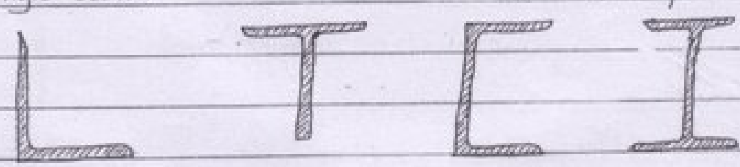
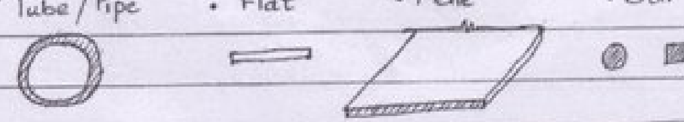
Q.5 a) In this question Dead load (DL) and Live load (LL) per panel point can be obtained by taking total plan area i.e. 16x3.5= 56 sqm and then dividing by total load by no. of panels. . Answers may be assessed accordingly.

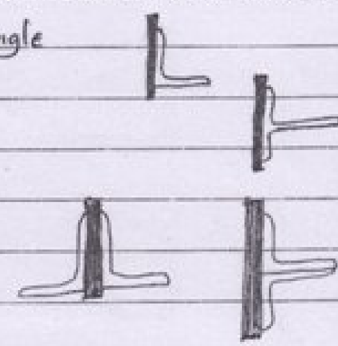
Q.5 b) In this question Live load (LL) and wind load (WL) per panel point can be obtained by considering total plan area and total sloping area respectively and then dividing total load by no. of panel points. . Answers may be assessed accordingly.

Q.5 c) In this question two solutions are possible, examiner may consider any one for giving proportionate marks. Answers may be assessed accordingly.

Solution 1) Considering bearing strength of concrete = 0.6 fck

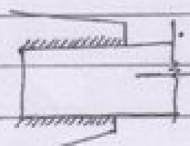
Solution 2) Considering bearing strength of concrete = 0.45 fck

Q.NO	SOLUTION	MARKS
1 a)	Any three	
	(i) Functions of –	1 each ×4 =4
	• Steel towers – to support transmission lines, antennas, radar equipments, tanks, bridge girders	
	• Roof trusses – to provide (roof) cover to structures	
	• Steel bridges – to facilitate transportation across rivers, valleys	
	• Crane girders – to facilitate movement of heavy materials and machinery in industry.	
	(ii) Structural steel sections (any 4) with name & sketches	1 each ×4 =4
	• Angle Section • Tee Section • Channel Section • Iye Section	
		
	• Tube / Pipe • Flat • Plate • Bar	
		
	(iii) Define –	1 each ×4 =4
	1) Importance factor – The factor used to obtain design seismic force based on the functional use of structure.	
	2) Zone factor – The factor used to obtain design spectrum depending upon the perceived seismic hazard in the zone in which the structure is located.	

Q.NO	SOLUTION	MARKS
	Q. 1a contd.	
3)	Response Reduction factor – It is the factor by which actual base shear force (that would be generated if the structure is to remain elastic during its response to the design basis earthquake shaking) should be reduced to obtain design lateral force.	
	Note (*) Mark be given if matter in bracket ( ) even is not written.	
4)	Fundamental Natural period – It is the first modal time period of vibration of the structure.	
(iv) –	Enlist two sections used as Tension member with sketches	1½ each × 2 = 3
	• single (equal or unequal) angle • Tee section • Double angle on either or same sides of GP	
		
–	Function of gusset plate – to facilitate connections at a place where more than one, members are to be jointed.	1

(*) Remark

Q.NO 1 contd..	SOLUTION	MARKS
Q.1 b)	Solve any one	
(i)	Flat size 150 x 10	
	Fillet weld 8 mm	
	Permissible shear stress 108 N/mm²	



Note — Given data represents, question is based on Working Stress Method, not as per Limit State Method.
- External pull or tension force is also not mentioned.
- $\boxed{\text{Can not be solved refering LSM.}}$

Note — Further, if a student wishes to solve by LSM, he has to assume f_u, has to calculate T_d (at least refering T_{dg} or T_{dn}), then only he can solve.

Such as

- Assuming $f_u = 410$ MPa.
- design strength of weld/mm $P_d = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} \times t$

$$= \frac{410}{\sqrt{3} \times 1.5} (0.7 \times 8)$$

$$= 883.73 \text{ N/mm} \qquad (1)$$

- For plate

$$\rightarrow T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{(150 \times 10)\, 250}{1.1} = 340.90 \times 10^3 \text{ N} \qquad (1)$$

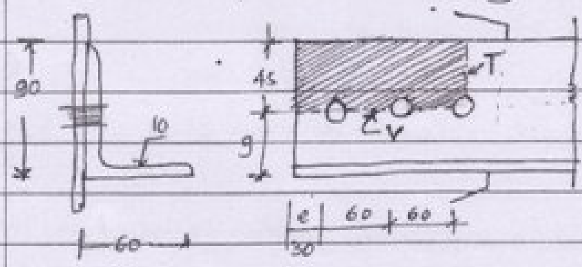
$$\rightarrow T_{dn} = \frac{0.9\, A_n\, f_u}{\gamma_{m1}} = \frac{0.9\,(150 \times 10)\, 410}{1.25} = 442.8 \times 10^3 \text{ N} \qquad (1)$$

$$\therefore T_d = 340.90 \times 10^3 \text{ N} \quad (\text{minimum out of } T_{dg}, T_{dn}) \qquad (1)$$

$$\rightarrow \text{Weld length req}^d = \frac{T_d}{P_d} = \frac{340.90 \times 10^3}{883.73} = 385.75 \text{ mm} \qquad (1)$$

say 390 mm

$$\therefore \text{Required lap length} = \frac{390}{2} = 195 \text{ mm.} \qquad (1)$$

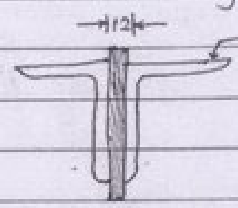
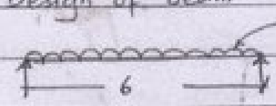
Q.NO 1(b) contd.	SOLUTION	MARKS
	(ii) Determination of block shear Strength.	
		
	$d = 20$	
	$d_h = 22$	
	* Values of 'g' and 'e') are not given.	
	but can be assumed as $g = \frac{90}{2} = 45$ (averaging)	1
	$e = 1.5\, d_o = 1.5 \times 22 = 33$ mm	
	$A_{vg} = (33 + 2 \times 60)\, 10 = 1530 \text{ mm}^2$	
	$A_{vn} = (33 + 2 \times 60 - 2.5 \times 22) 10 = 980 \text{ mm}^2$	2
	$A_{tg} = (45)\, 10 = 450 \text{ mm}^2$	
	$A_{tn} = (45 - 22/2)\, 10 = 340 \text{ mm}^2$	
Using,		
$T_{db_1} = \frac{A_{vg} \cdot f_y}{\sqrt{3}\, \gamma_{mo}} + \frac{0.9\, A_{tn} \cdot f_u}{\gamma_{m_1}}$		
	$= \frac{1530 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 340 \times 410}{1.25} = \boxed{301.13 \times 10^3 \text{ N}}$	1
$T_{db_2} = \frac{A_{tg} \cdot f_y}{\gamma_{mo}} + \frac{0.9\, A_{vn} \cdot f_u}{\sqrt{3}\, \gamma_{m_1}}$		
	$= \frac{450 \times 250}{1.1} + \frac{0.9 \times 980 \times 410}{\sqrt{3} \times 1.25} = 269.29 \times 10^3$ N	1
∴ Block Shear Strength will be minimum out of T_{db_1} & T_{db_2}		1

∴ $\boxed{T_{db} = 269.29 \text{ kN.}}$

* **Note** - As the values of 'e' and 'g' are not given in the question, student may assume other values than given in solution. Answers may be assessed accordingly.

Q.NO	SOLUTION	MARKS
Q.2	Solve any two $(2 \times 8 = 16)$	
	a) Determine bolt value	
	For bolt $d = 20$ mm $\therefore d_0 = 22$ mm $A_{nb} = 245$ mm^2 $p = 50$ mm	
	Grade 4.6 i.e. $f_{ub} = 400$ N/mm^2	
	* value of e not given assumed $e = 1.5\, d_0 = 33$ mm	1
	Note: Student may solve question by using other assumed value of e other than above. Answers shall be assessed accordingly	
	For plate $t = 10$ mm $f_u = 410$ N/mm^2	
	Design strength of bolt –	
	– in single shear	
	$V_{ds,b} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(n_n A_{nb}) = \frac{400}{\sqrt{3} \times 1.25}(1 \times 245) = 45.26 \times 10^3$ N $= 45.26$ kN	1
	– in double shear $V_{ds_2b} = 2\, V_{ds,b} = 2 \times 45.26 = 90.52$ kN	1
	– in bearing over 10 mm thick plate	
	$V_{dpb} = (2.5\, k_b\, d \times t \times f_u)/\gamma_{mb}$	
	k_b is smaller of $\frac{e}{3d_0} = \frac{33}{3 \times 22} = 0.5$	
	$\frac{p}{3d_0} - 0.25 = \frac{50}{3 \times 22} - 0.25 = 0.507$	
	$f_{ub}/f_u = 400/410 = 0.975$	
	and 1	2
	$\therefore k_b = 0.5$	
	$\therefore V_{dpb} = (2.5 \times 0.5 \times 20 \times 10 \times 410)/1.25 = 82.00 \times 10^3$ N	
	$V_{dpb} = 82$ kN	1

contd →

Q.NO 2	Contd SOLUTION	MARKS
	Hence,	
	For bolt in single shear –	
	Bolt value is lesser of V_{dsb} and V_{dpb} i.e. 45.26 kN	1
	For bolt in double shear	
	Bolt value is lesser of V_{ds_2b} and V_{dpb} i.e. 82.00 [redacted] kN	1
b)	Determine design compressive strength	
		
	2 ISA 90 x 90 x 10	
	L = 3 m with one bolt at each end,	1
	• r_{min} of assembly (of 2 equal angles) $= r_{xx}$ of single angle $= 27.3$ mm	2
	• $KL = 0.85\,L = 0.85 \times 3 = 2.55\ m = 2550$ mm	1
	• Slenderness ratio $\frac{KL}{r_{min}} = \frac{2550}{27.3} = 93.40$	1
	• Corresponding $f_{cd} = 121 - \frac{(93.40-90)}{10} \times 14 = 116.24\ N/mm^2$	2
	• Design compressive strength $P_d = f_{cd} \times A_g = 116.24 \times (2 \times 1703) = 395.91 \times 10^3\ N = 395.91$ kN	1
c)	Design of beam	
		
	w = (Dead load + Superimposed load) per m = 25 + 20 = 45 kN/m	

Q.NO 2 c contd.	SOLUTION	MARKS
	$\therefore W_u = 1.5 \times 45 = 67.5$ kN/m	
	$\therefore$ Design moment $M_D = M_u = 67.5 \times 6^2/8 = 303.75$ kNm	1
	& Design shear $V_D = V_u = 67.5 \times 6/2 = 202.50$ kNm.	1
	• For flexure, $Z_p\ reqd = \frac{M_d \cdot \gamma_{m0}}{f_y} = \frac{303.75 \times 10^6 \times 1.1}{250}$	
	$\therefore Z_p\ reqd = 1336.5 \times 10^3\ mm^3$	
	Z_p available $(1401.35 \times 10^3) > Z_p\ reqd$ $\therefore$ OK.	1
	$Z_e\ reqd = \frac{Z_p\ reqd}{1.14} = \frac{1336.5 \times 10^3}{1.14}$	
	$= 1172.37 \times 10^3\ mm^3$	
	Z_e available $(1223.8 \times 10^3) > Z_e\ reqd$ $\therefore$ OK.	1
	i.e. Available ISLB 450 is safe in flexure.	
	• Check for shear,	
	Design shear strength $V_d = \frac{(D \times t_w)\ f_y/\sqrt{3}}{\gamma_{m0}} = \frac{(450 \times 8.3)\ 250/\sqrt{3}}{1.10}$	1
	$= 507.805$ kN. $> V_u$ $\therefore$ OK.	
	and $\frac{V_u}{V_d} = \frac{202.5}{507.805} = 0.4 < 0.6$ $\therefore$ ok $\therefore$ Safe in shear	1
	• Check for deflection,	
	– permissible $y_{max} = L/300 = 6000/300 = 20$ mm.	1
	– actual y_{max} for service condition	
	$= \frac{5\ wL^4}{384\ EI} = \frac{5 \times 45 \times (6000)^4}{384\ (2 \times 10^5) \times (235.36 \times 10^6)}$	1

$= 13.78$ mm

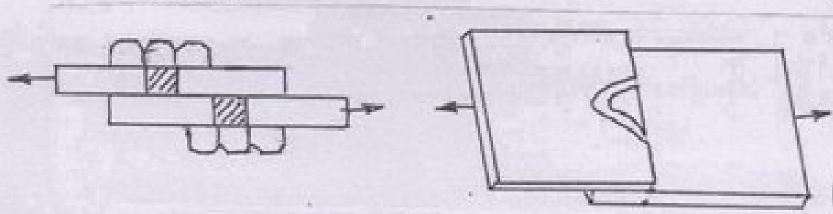
as act y_{max} < per y_{max}, section is safe in deflection

Q. 3 (a) Bolted joint may fail due to-
(i) Shear failure
(ii) Tensile failure
(iii) Bearing failure

01 M

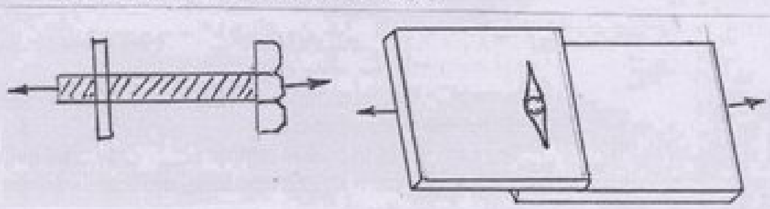
(i) Shear failure occurs due to single / double failure of bolt or tearing failure of plate. This type of failure is avoided by providing minimum edge distance in the bolted joint.

any TWO × $1\frac{1}{2}$ = 03 OR $1\frac{1}{2}$ M (for any two)



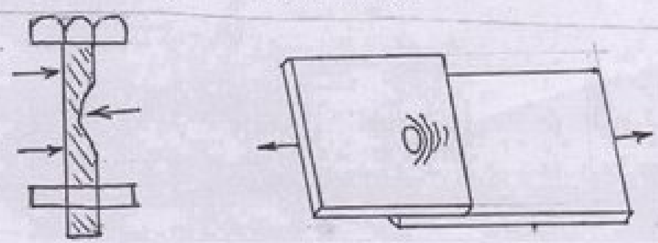
shear failure of bolt (i) tearing failure of plate

(ii) Tensile failure occurs due to tensile failure of bolt or tensile failure of plate. Normally the bolt subjected to tensile force fails if factored tensile force is greater than the tensile capacity of bolt.



tensile failure of bolt (ii) tensile failure of plate

(iii) Bearing failure occurs due to bearing failure of bolt or bearing failure plate. Normally the bolt material is of higher strength than steel plate; as a result bearing failure takes place in the plate material.



Bearing failure of bolt (iii) bearing failure plate

Q.3 (b) Disadvantages of welded connections w.r.t. bolted connections. Any 4x1=4

(i) Welded connections require skilled labour and supervision.

(ii) Welded connections are more brittle and therefore its fatigue strength is less as compared to bolted connections.

(iii) It is difficult to inspect a welded joint as compared to inspect a bolted joint.

(iv) Internal stresses and warping are produced due to uneven heating and cooling in welded connections.

(v) Welded connections require electricity.

(vi) Welded connections are over rigid as compared to bolted connections.

(vii) Member to be jointed may distort due to heat during welding process.

(viii) There is possibility of brittle fracture in case of welded connections.

Q.3 (c) Steel roof truss is a frame structure in which straight members are arranged and pin connected at their ends so that the members generally form triangles. 01

Advantages to use steel roof truss

i. At the places of high rainfall steel roof truss is used to avoid roof drainage problem. 01

ii. For large spans, steel roof truss is used because use of beams will become uneconomical. 01

iii. Use of steel roof truss is advantageous in industrial building, commercial complexes, cinema halls, malls and stadium roofs 01

Q.3 (d) Purlin is a flexural member subjected to transverse loads and is supported at panel points on principal rafter running perpendicular to it over two adjacent roof trusses. 02

IS Code provision for angle purlin -

Bending Moment about both axes should be considered and designed for bi – axial bending requirements. 01

The calculated deflections shall not exceed 01

(i) L / 150 for elastic cladding

(ii) L / 180 for brittle cladding

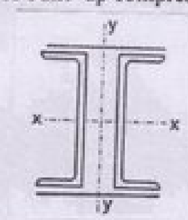
Q.3 (e) The selection criteria of type of roof truss is as listed below- Any 4x0.5 =2 OR $\frac{1}{2}$M for (any four)

(i) Span of roof truss
(ii) Pitch of roof truss
(iii) Purpose for which truss is to be designed- whether pitched roof truss or parallel chord truss
(iv) Roof coverings
(v) Fabrication and Transportation
(vi) Aesthetic
(vii) Climate

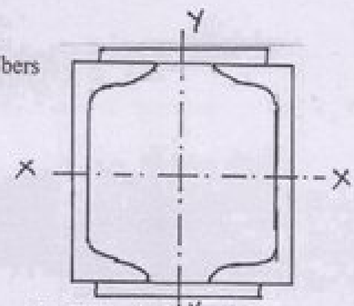
Pitch: - It is the ratio of rise to span of truss. 01

Slope: - It is the ratio of rise to half span of truss. 01

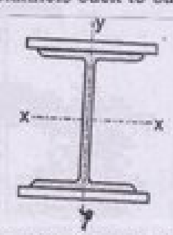
Q.4 (a) (i) Forms of built- up compression members Any 4x1=4



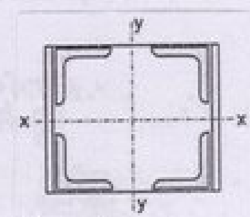
(a) Channels back to back



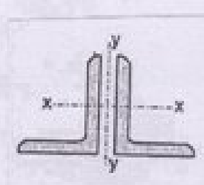
(b) Channels face to face



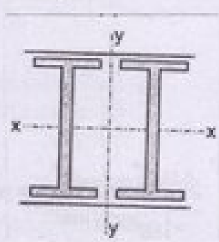
(c) I – Section with flange plates



(d) four angles



(e) Double Angle Section



(f) Double I Section

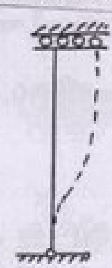
(ii)general requirements for lacing:-

Any 4x1=4

(a) Thickness of flat lacing bars shall not be less than l/40 for single lacing and 1/60 for double lacing.
(b) Lacing bars shall be inclined at an angle not less than 40° nor more than 70° to the axis of the built-up member.
(c) Lacing system shall be uniform through act the length of the column.
(d) The minimum width of lacing bars shall be three times the nominal diameter of the end bolt.
(e) Lacing flats to be designed for transverse shear of 2.5 % of axial force in the member.
(f) Slenderness ratio of lacing flats shall not exceed 145.

(iii) Effective length of column

The effective length of a column is the distance between the points of zero moment or points of contra flexures of a buckled column. 02



01

01

Effective length = 1.2L

(iv) Local buckling in compression member-

The buckling of the plate element of the cross – section under compression or shear may take place before over all bucking. This phenomenon is called local buckling. Hence local buckling involves distortion of cross – section. 02

Local buckling effect :-

Local buckling reduces over all load carrying capacity of the member. 01

Hence to prevent local buckling effects – adopt higher thickness of elements i.e.by controlling width to thickness ratios as per IS requirements. 01

Q.4	(b)	(i) Gross –section yielding –	
		• Deformation of the tension member in longitudinal direction may take place before it fractures, making the structure unserviceable, is called gross – section yielding.	01
		• Hence, to prevent deformation due to yielding the stress on gross – section shall be less than yield stress. i.e. $\frac{T}{Ag} < Fy$	01
		• $T_{dg} = \frac{Ag.Fy}{\gamma m0}$	
		Net – section rupture –	
		• Tension member is connected to gusset plate by means of bolts, and when member is loaded, the fibers adjacent to the bolt hole yield due to stress concentration. When load increase in the member the entire net section of the member reaches the ultimate stress, which is called as net section rupture.	01
		• Hence, to prevent failure of tension member due to net section rupture, ultimate strength of tension member should be greater than design force in the member. i.e. $A_n f_y > T$	01
		• $T_{dn} = \frac{T}{\gamma m1} = \frac{An.fy}{\gamma m1}$	01
		• as there is no reserve strength of any kind beyond ultimate strength of tension member factor 0.9 should be applied to calculate Hence, $T_{dn=0.9} \frac{An.Fy}{\gamma m1}$	01

Q.4 b)

(ii) (i) $T_{dg} = \frac{A_g \cdot f_y}{\gamma_{m0}} = \frac{978 \times 250}{1.10}$

$= 222272.7\ N$

$= 222.27\ KN$ 01

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

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

$= 1.4 - 0.076 \left(\frac{80}{8}\right)\left(\frac{250}{410}\right)\left(\frac{80}{170}\right)$

$\beta = 1.182$ 01

$\therefore T_{dn} = \frac{0.9 \times 368 \times 410}{1.25} + \frac{1.182 \times 608 \times 250}{1.10}$

$= 271964.51\ N$ 01

$= 272\ KN$

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

where, A_{vg} = total welding $l \times t$

$= 340 \times 8 = 2720\ mm^2$

$= A_{vn}$

$A_{tg} = 50 \times 8 = 400\ mm^2$

$= A_{tn}$

P.T.O.

$$\therefore T_{db_1} = \frac{A_{vg} \cdot f_y}{\sqrt{3} \cdot \gamma_{m0}} + \frac{0.9\ A_{tn} \cdot f_u}{\gamma_{m1}}$$

$$= \frac{2720 \times 250}{\sqrt{3} \times 1.10} + \frac{0.9 \times 400 \times 410}{1.25}$$

$$= 475 \text{ KN}$$ 01

$$\& \quad T_{db_2} = \frac{0.9\ A_{vn}\ f_u}{\sqrt{3} \times \gamma_{m1}} + \frac{A_{tg} \cdot f_y}{\gamma_{m0}}$$

$$= \frac{0.9 \times 2720 \times 410}{\sqrt{3} \times 1.25} + \frac{400 \times 250}{1.10}$$

01

$$= 554.50 \text{ KN}$$

$\therefore$ Design strength of Tension Member is the lesser value of T_{dg}, T_{dn}, T_{db}

$\therefore$ Design strength of member is = 222.27 KN

01

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Q.5(a) slope $\theta = \tan^{-1}\left(\frac{3}{8}\right) = 20.556°$ —— ① — 1M

Dead Load Calculation:

- Weight of roof covering $= \frac{120}{\cos 20.556} = 128.16\ N/m^2$
- self wt. of truss $= \left(\frac{L}{3}+5\right)\times 10$

$= \left(\frac{16}{3}+5\right)\times 10 = 103.33\ N/m^2$

- Weight of bracing $= 75\ N/m^2$
- Weight of purlin $= 80\ N/m^2$

Total DL $= 386.49\ N/m^2$ —— ② 2M

∴ Dead Load on one panel point

= Intensity of DL × Area under one panel point

$= 386.49 \times 2 \times 3.5$

$= 2705.43\ N = \underline{2.705\ kN}$ —— ① 1M

Live Load Calculation:

Live Load for purlin $= 750 - (\theta - 10)\times 20$

$= 750 - (20.556 - 10)\times 20$

$= 538.88\ N/m^2 \nless 400\ N/m^2$ —— ② 2M

∴ Live load for truss $= \frac{2}{3}\times 538.88$

$= 359.27\ N/m^2$

Live load on one panel point $= 359.27 \times 2 \times 3.5$

$= 2514.9\ N$

$= \underline{2.515\ kN}$ —— ② 2M

Note: DL & LL per panel point can be obtained by taking total plan area i.e. $16\times 3.5 = 56\ m^2$ and then dividing by total load by no. of panel points.

★ Note:- D.L & L.L. Per Panel Point can be obtained by Taking total plan area i.e. $16\times 3.5 = 56\ m^2$ and then dividing by total load by number of panel points. Answers may be assessed accordingly.

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Q.5(b) for worst combination

$$P_d = (C_{pe} - C_{pi})\,p$$
$$= (-1.0 - 0.5)\,1650 = -2475\ N/m^2 \quad \text{— ② - (2M)}$$
$$\theta = \tan^{-1}\left(\frac{4.5}{10}\right) = 24.23°$$

Wind load per panel point = Design wind pressure × spacing × Inclined panel length

$$= -2475 \times \frac{2}{\cos 24.23} \times 5$$
$$= -27140.97\ N \ (\text{uplift})$$
$$= \underline{-27.14}\ kN \quad \text{— ② - (2M)}$$

Live load on purlin $= 750 - (\theta - 10) \times 20$ — ① - (1M)

$$= 750 - (24.23 - 10) \times 20$$
$$= 465.4\ N/m^2 \nless 400\ N/m^2$$

∴ Live Load on truss $= \frac{2}{3} \times 465.4$

$$= 310.27\ N/m^2 \quad \text{— ① - (1M)}$$

Live load on panel point $= 310.27 \times 2 \times 5$

$$= 3102.7\ N$$
$$= \underline{3.1027}\ kN \quad \text{— ② - (2M)}$$

Note: Live Load and Wind Load per panel point can be obtained by considering total plan area and total sloping area respectively and then dividing by total load by no. of panel points.

Note:- LL and W.L per panel point can be obtained by considering total plan area and total sloping area respectively & then dividing total load by number of panel points.

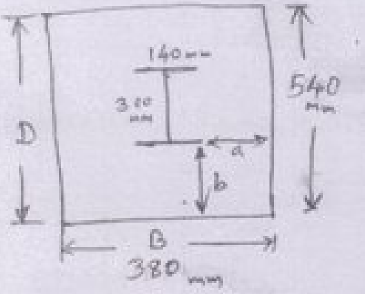
In this Question Two Solutions are Possible.
Solution (I) - considering Bearing strength of concrete $= 0.6 f_{ck}$

Q.5 (c) Axial load $= 1.58$ MN $= 1580$ kN

Ultimate load $= 1.5 \times 1580 = 2370$ kN

Bearing area of base plate $A = \frac{P_u}{0.6 f_{ck}} \cdot \frac{P_u}{0.6 f_{ck}}$

$$A = \frac{2370 \times 10^3}{0.6 \times 20} = 197500 \text{ mm}^2 \text{ — (1) — (1M)}$$



$B \times D = 197500$ — (1) eq^n

$a = b$

$$\frac{B-140}{2} = \frac{D-300}{2}$$

$B = D - 300 + 140$

$B = D - 160$ — (2) eq^n

$(D-160)D = 197500$

$D^2 - 160D - 197500 = 0 \quad \therefore D = 531.5$ mm

Adopt $D = 540$ mm

$\therefore B = 380$ mm & $a = b = 120$ mm — (2) — (2M)

$$w = \frac{2370 \times 10^3}{540 \times 380} = 11.55 \text{ N/mm}^2 \qquad t = \sqrt{\frac{2.5w(a^2 - 0.3b^2)\gamma_{m0}}{f_y}} \text{ — (1) — (1M)}$$

Thickness of plate $t = \sqrt{\frac{2.5 \times 11.55(120^2 - 0.3 \times 120^2) \times 1.1}{250}}$

$t = 35.78$ mm $\simeq 36$ mm $< t_f \quad \therefore$ ok

Provide base plate of size 540 mm × 380 × 36 mm — (2) — (2M)

Area of concrete block $A = \frac{P_u \cdot \gamma_{m0}}{SBC \cdot \gamma_f}$

$$A = \frac{2370 \times 10^3 \times 1.1}{200 \times 10^3 \times 1.5} = 8.69 \text{ m}^2$$

For equal projections $B \times D = 8.69 \text{ m}^2 = 8.69 \times 10^6 \text{ mm}^2$

$\frac{B-440}{2} = \frac{D-600}{2} \quad \therefore B = D - 160 \quad \therefore D^2 - 160D - 8.69 \times 10^6 = 0$

$D = 3028.9$ mm — (1) (1M)

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Q.5(c) Contd.

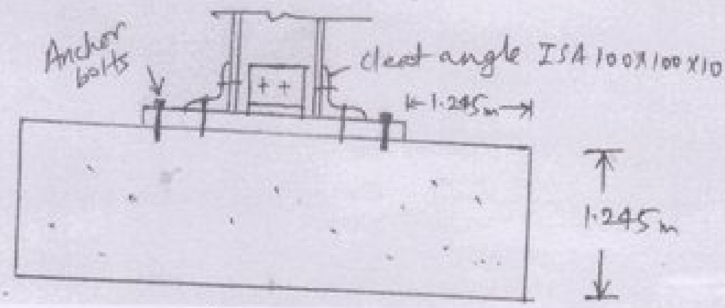
Adopt D = 3030 mm = 3.03 m

B = 2870 mm = 2.87 m

∴ Actual projections a = b = 1245 mm

Considering 45° angle of dispersion of load

The depth of concrete block = 1.245 m —① (1M)



* Alternate solution by taking Bearing Stress= $0.45f_{ck}$

solution ② considering Bearing strength of concrete = $0.45 f_{ck}$

(P.T.O)

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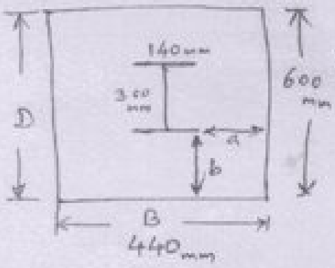
Q.5 (c) Axial load = 1.58 MN = 1580 kN

Ultimate load = 1.5 × 1580 = 2370 kN

Bearing area of base plate $A = \frac{P_u}{0.45 f_{ck}} = \frac{P_u}{0.45 f_{ck}}$

$$A = \frac{2370 \times 10^3}{0.45 \times 20} = 263333 \text{ mm}^2 \text{ --- (1)}$$



$B \times D = 263333.33$ — (1) eq^n

$a = b$

$$\frac{B-140}{2} = \frac{D-300}{2}$$

$B = D - 300 + 140$

$B = D - 160$ — (2) eq^n

$(D-160) D = 263333.33$

$D^2 - 160D - 263333.3 = 0 \quad \therefore D = 600$ mm

Adopt $D = 600$ mm

$\therefore B = 440$ mm & $a = b = 150$ mm — (2)

$$w = \frac{2370 \times 10^3}{600 \times 440} = 8.98 \text{ N/mm}^2$$

Thickness of plate $t = \sqrt{\frac{2.5 \times 8.98 (150^2 - 0.3 \times 150^2) \times 1.1}{250}}$

$t = 39.44$ mm $\cong 40$ mm $< t_f$ $\therefore$ ok

Provide base plate of size 600 mm × 440 × 40 mm — (2)

Area of concrete block $A = \frac{P_u \cdot \gamma_{mo}}{SBC \cdot \gamma_f}$

$$A = \frac{2370 \times 10^3 \times 1.1}{200 \times 10^3 \times 1.5} = 8.69 \text{ m}^2$$

For equal projections $B \times D = 8.69 \text{ m}^2 = 8.69 \times 10^6 \text{ mm}^2$

$\frac{B-440}{2} = \frac{D-600}{2} \quad \therefore B = D - 160 \quad \therefore D^2 - 160D - 8.69 \times 10^6 = 0$

$D = 3028.9$ mm — (1)

Q.5(c) contd.

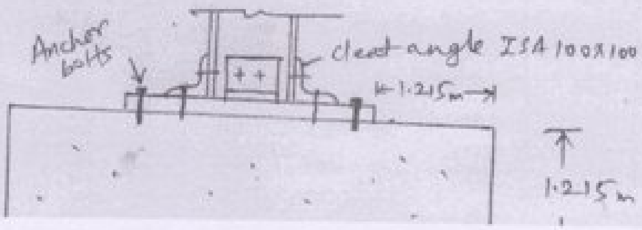
Adopt $D = 3030$ mm $= 3.03$ m

$B = 2870$ mm $= 2.87$ m

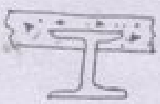
Actual projections $a = b = 1215$ mm

Considering 45° angle of dispersion of load

The depth of concrete block $= \underline{1.215\,m}$ — (1)



Q.6(a) A Laterally supported beam is a beam whose compression flange is restrained from buckling. — (2)



i) by casting flange into RCC slab

ii) by connecting concrete floor by shear connectors.

iii) flange may be restrained by its connection to cross-beams or bracings — (2)

(b) Design bending strength,

$$Md = \frac{Z_p \cdot f_y \cdot \beta_b}{\gamma_{mo}} \qquad \frac{b_f}{t_f} = \frac{200/2}{9} = 11.11 < 15.7$$

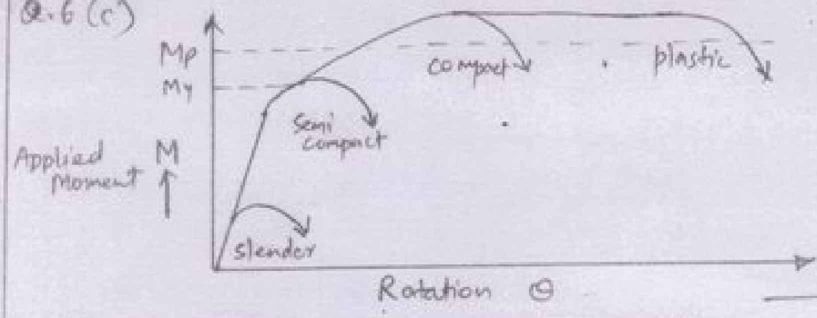
where $\beta_b = Z_e/Z_p$

$$d = 250 - 2(10+9) = 212\,mm \qquad \frac{d}{t_w} = \frac{212}{6.7} = 31.64 < 84$$ — (2)

∴ Section is semi-compact

$$Md = \frac{Z_p \cdot f_y \cdot \frac{Z_e}{Z_p}}{\gamma_{mo}} = Md \Rightarrow \frac{Z_e \cdot f_y}{\gamma_{mo}} = \frac{475.4 \times 10^3 \times 250}{1.1} = \underline{\underline{107.8\,kNm}}$$ — (2)

Q.6 (c)



——— (2)

Plastic sections has sufficient ductility. compact sections have relatively lower rotation, Semi compact sections, bending stress is limited to yield stress. For slender members local or lateral buckling occurs in the elastic range. ——— (2)

Q.6(d)

Slab base	Gusseted base
i) thickness of base plate required more	i) thickness of base plate required less compared to slab base.
ii) Cleat angles are used to fasten column section to base plate	ii) Cleat angles are used to fasten gusset plate to base plate, so stiffness of joint increases.
iii) simple in construction	iii) complex in construction
iv) Economical	iv) Expensive but stronger.

——— (4)

Q.6(e) Column bases spreads the load on wider area so that intensity of bearing pressure on concrete block is within limit. The load on column is transfered to base plate through bearing. Cleat angles holds the column in position and make easy connection of column section with base plate. Plan area of concrete depends on SBC of soil). Cleat angles are provided to hold column and connect it to base plate. (4)