WINTER– 17 EXAMINATION

Subject Name: Design of Steel Structures **Model Answer** Subject Code: 17505

Important Instructions to examiners:

1) The answers should be examined by key words 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 errors such as grammatical, spelling errors should not be given more Importance (Not applicable for subject English and Communication Skills.
4) While assessing figures, examiner may give credit for principal components indicated in the figure. The figures 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 some cases, the assumed constant 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 judgement on part of examiner of relevant answer based on candidate's understanding.
7) For programming language papers, credit may be given to any other program based on equivalent concept.

Q. No.	Sub Q. N.	Answer	Marking Scheme
Q.1	(A)a) Ans	State any four advantages of steel as a construction material. 1. Steel being a ductile material does not fail suddenly it gives visible evidence of impending failure 2. It has high ratio of strength to weight making it to use for the construction of long span bridges, tall buildings etc. 3. Steel can be transported, fabricated and erected at site thus saves time of construction and saves expenses also. 4. Steel as construction material has good earthquake resistor capacity due to its ductility and elastic plasticity. 5. The steel structures can be disassembled and reused wherever required. It can be recycled easily. 6. Steel has high scrap value amongst all building materials. 7. Steel is a gas resistant.	Any four 01 mark for each.
Q.1	(A)b) Ans	Define: .I) Importance factor II) Zone factor III) Response reduction factor IV) Fundamental natural period I) Importance factor (I): The importance factor is a factor used to obtain the design seismic force depending upon the functional use of the structure. II) Zone factor (z): The zone factor is a factor used to obtain the design spectrum depending on the perceived seismic hazard in the zone in which the structure is located. III) Response reduction factor (R): The response reduction factor is the factor by which the actual base shear force should be reduced, to obtain the design force. IV) Fundamental natural period: The fundamental natural period is the first (longest) modal time period of vibration of the structure.	01 mark for each
Q.1	(A)c) Ans	List the values of partial safety factor for material strength in case of resistance by yield, buckling, ultimate stress and bolt connection. Partial safety factor for material.	

Sr. No.	Description	Partial safety factor
1.	Resistance by yielding.	1.10
2.	Resistance to buckling.	1.10
3.	Resistance by ultimate stress.	1.25
4.	Bolted connection.	1.25 (Shop and field fabrication)

01 mark for each

Q.1 (A)d) Explain shear lag.

Ans

Shear lag: While transferring the tensile force from gusset plate to tension member through one leg by bolt or welds, the connected leg of section (such as angle, channel) may be subjected to more stress than the outstanding leg and finally the stress distribution becomes uniform over the section away from the connection. Thus, one part leg behind the other is called as shear lag. — 01 mark

The tearing strength of an angle section connected through one leg is affected by shear lag also. Thus the design strength τ_{dn} governed by tearing of net section is given by,

τ_{dn} = [0.9 (A_{nc} x f_u) / Y_{m1})] + [β x (Ago x fy / Y_{mo})]

Where β = 1.4 – 0.076 (w/t) (f_y/f_u) (b_s/L_c) — 01 mark

Bs = Shear lag width as shown in fig.

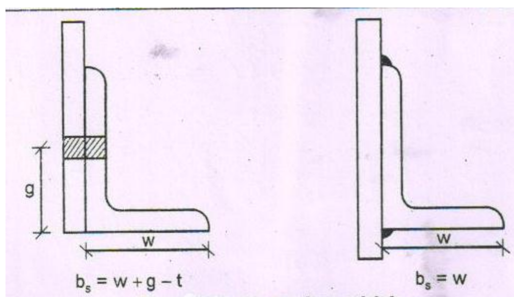


Fig: Shear lag width

02 marks

Q.1 (B)a) Determine bolt value 16mm diameter bolt of 4.6 grade to connect two angles 90 x 60 x 06 mm back to back on opposite side of gusset plate of 8 mm thick. Also determine no. of bolts required for the joint when it carries direct factored load of 110 KN. Draw neat sketch of designed connection.

Ans

The angles are connected on both sides of gusset plate, hence the bolts will be in double shear and bear against 8 mm thick (least of 8 and 2 x 6mm) gusset plate for 4.6 grade bolts, f_{ub} = 400 mPa. For 16 mm diameter bolt, A_{nb} = 0.78 x (π/4) x 162 = 156.83 mm^2.

d_o = d + 2 = 16 + 2 = 18mm

Y_{mb} = Y_{m1} = partial factor of safety for bolt and angles = 1.25 -------------------------------------- 01 mark

Double shear strength of bolts

V_{dsb} = ($2V_{nsb}$ / Y_{mb}) = 2 ($f_{ub}/\sqrt{3}$) (n_n x A_{nb} + n_s x A_{sb}) / 1.25

= 2 x (400 / $\sqrt{3}$)(1 x 156.83 + 0) / 1.25) = 57948 N = 57.95 kN. ------------------------------- 01 mark

Bearing strength of thinner plate

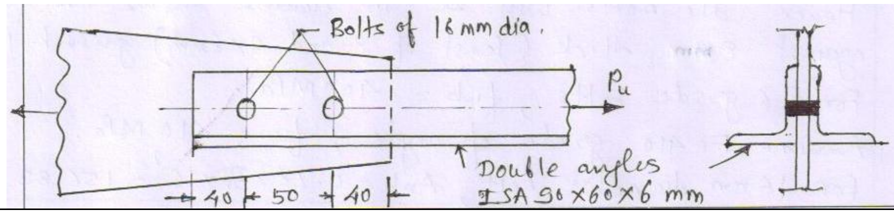
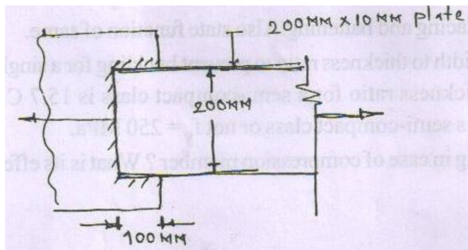
V_{dpb} = V_{npb} / Y_{mb} = 2.5 x (K_b x d x t x f_{ub}) / Y_{mb}

Assume p = 3d = 3 x 16 = 48mm say 50 mm and e = 2d = 2 x 16 = 32 mm say 40 mm.

(NOTE:Students may assume slightly different pitch and edge distance. Solution will change accordingly.)

K_b is least of [($e/3d_o$): ($p/3d_o$) – 0.25: (f_{ub}/f_u): 1.0]

i.e. [(40 / 3)x18 = 0.74: (50 / 3x18) – 0.25 = 0.67: 400 / 410 = 0.975: 1.0]

		hence $K_b = 0.67$ V_{dpb} = 2.5 x 0.67 x 16 x 8 x 400 / 1.25 = 68608 N. = 68.61 kN. -------------------------------------- Bolt value, Bv = least of V_{dsb} & V_{dpb} = 57.95 kN. -- No. of bolts required = P_u / B_v = 110 / 57.95 = 1.89 say 2. -- 	 01 mark 01 mark 01 mark 01 mark
Q.1	(B)b) Ans	For a tension member as shown fig. 1. Determine block shear strength. fy = 250 MPa, fu = 410 MPa.  Fig. 1 [Q. 1 (B) b] A_{vg} = 2(100 x 10) = 2000 mm^2 A_{vn} = 2000 mm^2 A_{tg} = 200 x 10 = 2000 mm^2 A_{tn} = 2000 mm^2 Block shear strength (τ_{db}) (τ_{db1}) = [(A_{vg} x f_y) / ($\sqrt{3}$ x Y_{mo})] + [(0.9 x A_{tn} x f_u) / Y_{m1}] = [(2000 x 250) / ($\sqrt{3}$ x 1.10)] + [(0.9 x 2000 x 410) / 1.25] = 852832 N. -- (τ_{db2}) = [(A_{tg} x f_y) / (Y_{mo})] + [(0.9 x A_{vn} x f_u) / ($\sqrt{3}$ x Y_{m1}] = [(2000 x 250) / (1.10)] + [(0.9 x 2000 x 410) / ($\sqrt{3}$ x1.25] = 795413 N. -- Hence τ_{db} = Least of τ_{db1} and τ_{db2} = 795413 N = 795.41 kN. --	 01 mark 02 marks 02 marks 01 mark
Q.2	a) Ans	Design suitable fillet welded connection for ISA 80 x 50 x 08mm with its longer leg connected to gusset plate of thickness 8 mm. The angle is subjected to factored load of 300 KN. Cxx = 27.3 mm. Assume weld applied to all three edges and shop weld. i. Pu = 300 kN. ii. Size of weld minimum size = 3 mm, Maximum size = (3/4)t = (3/4)x8 = 6 mm. So assume 6 mm size fillet weld (shop) --- iii. Design stress of shop weld f_{wd} = f_u / ($\sqrt{3}$ x Y_{mw}) = 410/($\sqrt{3}$ x 1.25) = 189.4 N/mm^2 -------------------------------------- iv. Design strength per mm length of weld p_q = f_{wd} x t_t = 189.4 x 0.7 x 6 = 795.48 N/mm -- v. Effective length of weld required L = P_u/p_q = 300 x 10^3 / 795.48 = 377.13 say 380 mm. --------------------------------------	 01 mark 01 mark 01 mark 01 mark

vi. Let x_1 and x_2 be the lengths of longitudinal weld at upper and lower edges and third edge will be 80 mm long.

$x_1 + x_2 + 80 = 380$

$x_1 + x_2 = 300$ mm — 01 mark

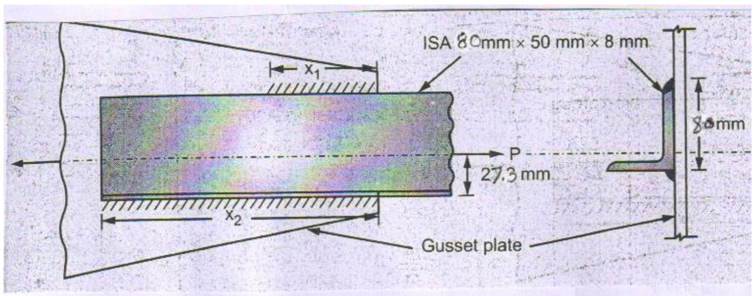
vii. Taking moment about the bottom weld

$795.48 \times x_1 \times 80 + 795.48 \times 80 \times 40 = 300 \times 10^3 \times 27.3$

Hence $x_1 = 88.69$ mm say 90 mm — 02 marks

$x_2 = 300 - 90 = 210$ mm.

viii.



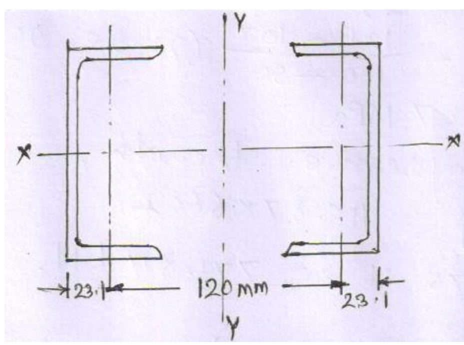
01 mark

Q.2 b) A built up column consist of 2ISMC - 225, placed face to face at 120 mm. The distance is between their centres. The length of column is 6.0 m and both ends are hinged. Find design strength of column.

For single ISMC – 225 A=3301 mm^2 ,$I_{yy} = 1.872 \times 10^6 mm^4$,

I_{xx}: = $26.946 \times 10^6 mm^4$, Cxx: = 23.1 mm. (Refer table no. 1 for f_{cd})

Ans



01 mark

Area of composite section, $A_g = 2 \times 3301 = 6602\ mm^2$

Based on $r_{xx} = r_x = \sqrt{(Ixx / A)} = \sqrt{(26.946 \times 10^6 / 3301)}$

$= 90.34$ mm -- 01 mark

$I_{yy} = 2[I_y + Ah^2]$

$= 2[1.872 \times 10^6 + 3301 \times (120/2)^2]$

$= 27511200\ mm^4$

$r_{yy} = \sqrt{(I_{yy} / A)} = \sqrt{(27511200 / 6602)}$

$= 64.55$ mm. -- 01 mark

Hence $r_{min} = 64.55$ mm

For given end condition, kL = 1.0L -- 01 mark

$SR = kL / r_{min}$

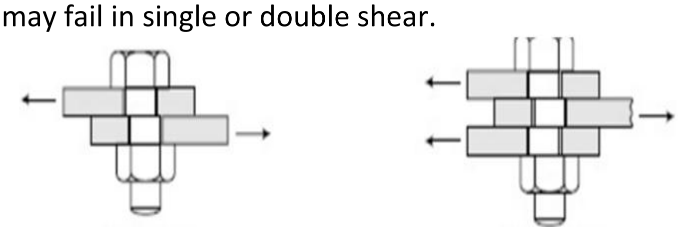
$= 1.0 \times 6000 / 64.55$

$= 92.95$ -- 01 mark

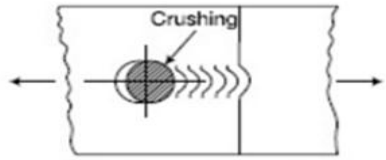
For built up section, buckling class is C for which-

SR	fcd
90	121
100	107

01 mark

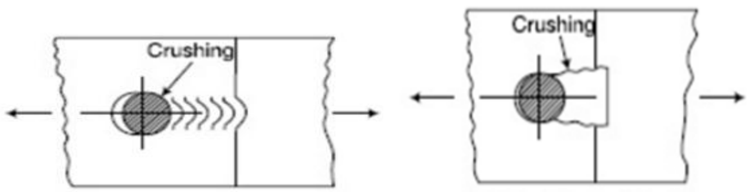
		$f_{cd} = f_{cd1} – [(f_{cd1} – f_{cd2}) / (SR_2 – SR_1)] \times (SR – SR_1)$ $= 121 – [(121 – 107) / (100 – 90)] \times (92.95 – 90)$ $= 116.87$ mPa. --- Design compressive strength $P_d = f_{cd} \times A_g$ $= 116.87 \times 6602$ $= 771575$ N = 771.57 kN. ---	 01 mark 01 mark
Q.2	c) Ans	An ISMB 400 @ 604.3 N/m is used as simply supported beam of span 5.0 M. The compression flange of the beam is laterally supported throughout the span. Determine design flexural strength of member. Also calculate working udl on the beam per meter span. Check the member for deflection. Take $Z_p = 1176.18 \times 10^3 mm^3$, $Y_{mo} = 1.1$, $\beta_b = 1.0$, $f_y = 250$ mPa. Given L = 5 m = 5000 mm $Z_{xx} = Z_p/s = 1176.18 \times 10^3 / 1.14 = 1031.74 \times 10^3 mm^3$ ----------------------------------- $I_{xx} = Z_{xx} \times y_{max} = 1031.74 \times 10^3 \times 400/2 = 206.35 \times 10^6 mm^4$ ---------------------------- Assuming udl = ‘w’ kN/m. i. To calculate design flexural strength, M_d $M_d = (\beta_b \times Z_p \times f_y) / Y_{mo} = (1 \times 1176.18 \times 10^3 \times 250) / 1.10$ $= 267.27 \times 10^6$ N-mm = 267.27 kN-m. -- ii. $M_u = w_u \times L^2/8 = w_u \times 5^2/8 = 3.125 w_u$ kN/m -- iii. Equating M_d and M_u $267.27 = 3.125w_u$ $w_u = 85.53$ kN/m. --- $w = w_u/ Y_f = 85.53/1.5 = 57.02$ kN/m. -- iv. Check for deflection $\delta_{allowable} = L/300 = 5000/300 = 16.67$ mm. -- $\delta_{max} = (5 \times w \times L^4) / (384 \times EI)$ $= (5 \times 57.02 \times 5000^4)/(384 \times 2 \times 10^5 \times 206.35 \times 10^6)$ $= 11.24$ mm. As $\delta_{max} < \delta_{allowable}$, deflection check is O.K. --	 01 mark 01 mark 01 mark 01 mark 01 mark 01 mark 01 mark 01 mark
Q.3	a) Ans	Explain any two types of failure of bolted joints with neat sketches. **Two types of failure of bolted joints:-** **1. Shear failure of bolt:** shear tress are generated when the plates slip due to applied forces. The maximum factored shear force in the bolt may exceed the nominal shear capacity of the bolt. The shear failure of the bolt takes place at the bolt shear plane (interface).the bolt may fail in single or double shear.  **Single Shear** **Double Shear** **Shearing at bolt shank** **2. Bearing failure of bolt:-**the bolt is crushed around half circumferences. The plate may be strong in bearing and the heaviest stressed plate may press the bolt shank. The bearing	 Any Two 02 marks for each

failure of bolt generally does not occur in practice.



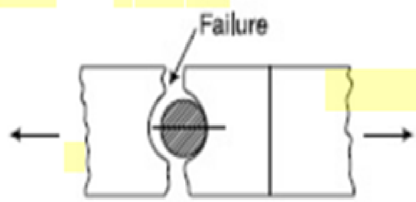
Bearing on bolt

3. Bearing failure of plate: - when an ordinary bolt is subjected to shear forces. The slip takes place and bolt comes in contact with the plate .the plate may get crushed .if the plate material is weaker than the bolt material. The bearing problem can be complicated by the presence of a nearby bolt or the proximity of an edge in the direction of load.



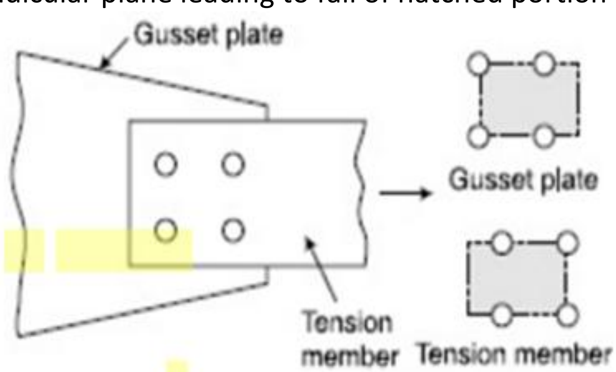
Bearing on plate **Shear tear out of plate**

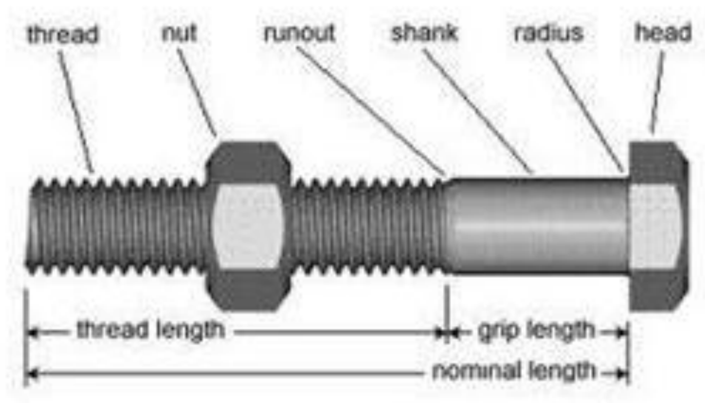
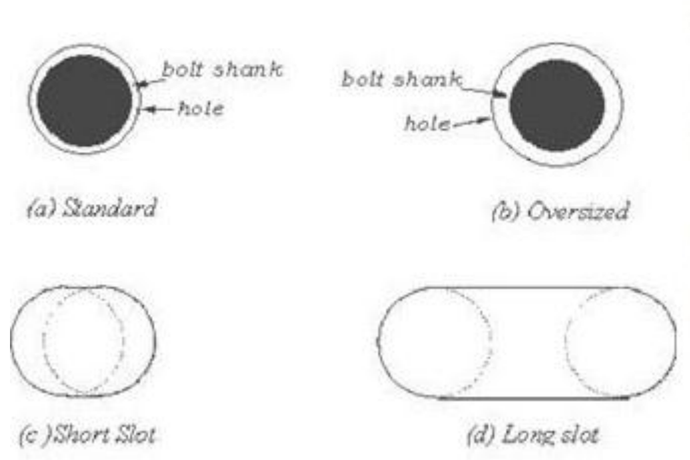
5. Tension or tearing failure of plates: tearing failure occurs when the bolts are stronger than the plate's .tension on both the gross area (yielding) and net effective area (rupture) must be considered.



Tension or tearing failure of plates

6. Block shear failure :-Bolts may have been placed at a lesser end distance than required causing the plates to shear out which, however can be checked by observing the specification for end distance .the failure of connection in block may occur when a block of material within the bolted area breaks away from the remainder bolts are used ,fewer bolts will be used for making connection .this type of failure occurs with the shear on one plane and tension on perpendicular plane leading to fall of hatched portion of the plate.

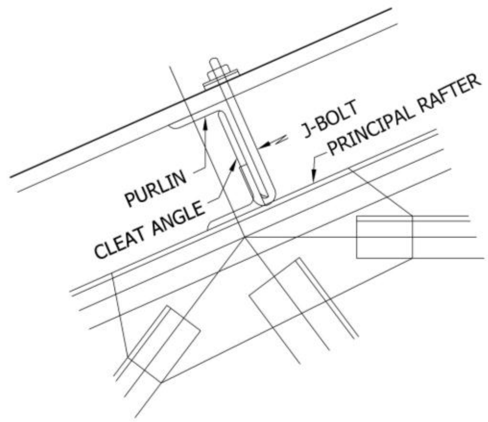


Q.3	b) Ans	List types of bolts and sketch anyone of them. **Types of bolts:-** 1. unfinished(black) bolts 2. High strength friction grip bolts. **1. Unfinished(Black) Bolts**  **2. High strength friction grip bolts.**  	01 mark Fig. Any One 03 Marks
Q.3	c) Ans	Write the IS code provision for design of angle purlin. **IS code provision for design of angle purlin** • Angle purlin should be design for biaxial bending. • Roof angle should be less than 30^0 • Width of angle leg perpendicular to sheeting ≥ L/45	04 marks

		• Width of angle leg parallel to sheeting ≥ L/60 In the above, situation B.M. about z-z axis should be taken as $W_z(L)^2/10$. Where W_z is udl in the direction normal to sheeting L is the spacing of truss, **One leg >L/45 and another leg >L/60** IS 800-1984 (second revision) code provision For roof slopes not exceeding 30^0 based in a min. live load **750 N/m²** **Maximum B.M. =** $W_z(L)^2/10$.	
Q.3	d) Ans	List the factors to be considered in calculation of wind load. Write equations to calculate wind load on roof truss as per IS 875-1987. **Factors to be considered in calculation of wind load:** 1.Location of structure for basic wind speed (V_b) 2.Risk coefficient factor(K_1), Terrain height and structure size factor(K_2) and Topography factor(K_3) for design wind speed (V_z) 3. Building height to width ratio(h/w), roof angle(α) and wind angle(θ) for external wind pressure coefficient(C_{pe}) 4. Percentage of opening in wall (permeability of air) for internal wind pressure coefficient(C_{pi}) • **Equation to calculate wind load on roof truss as per IS-875-1987:** 1. **Design wind speed (V_z)=k_1 k_2 k_3 V_b** i. Risk Coefficient-(**k_1)** ii. Terrain ,Height And Structure Size Factor, **k_2** iii. Topography Factor. **k_3** iv. basic wind speed -V_b 2. **wind pressure (P_z)=0.6 $(V_z)^2$----(N/m²)** 3. **wind load on roof** **$F = (C_{pe} - C_{pi})Ap_z$** C_{pe} - Coefficient of external wind pressure C_{pi} - Coefficient of internal wind pressure A - surface area of structural element in (m^2) p_z - design wind pressure **(N/m²)**	**01 Mark** **03Marks**

Q.3 e) Draw a neat labeled sketch of angle purlin with principle rafter at panel point having roof covering as A.C. sheets.

Ans



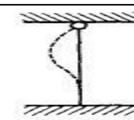
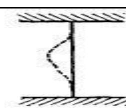
04 marks

Q.4 (A)a) State with sketch the effective length for a compression member as per IS 800/2007 having end conditions as-

i) Translation restrained at both ends and rotation free at one end.

ii) Translation and rotation restrained at both ends.

Ans

At one end		At second end		Effective length	sketch
Translation	Rotation	Translation	Rotation		
Restrained	Restrained	Restrained	Free	0.8L	
Restrained	Restrained	Restrained	Restrained	0.65L	

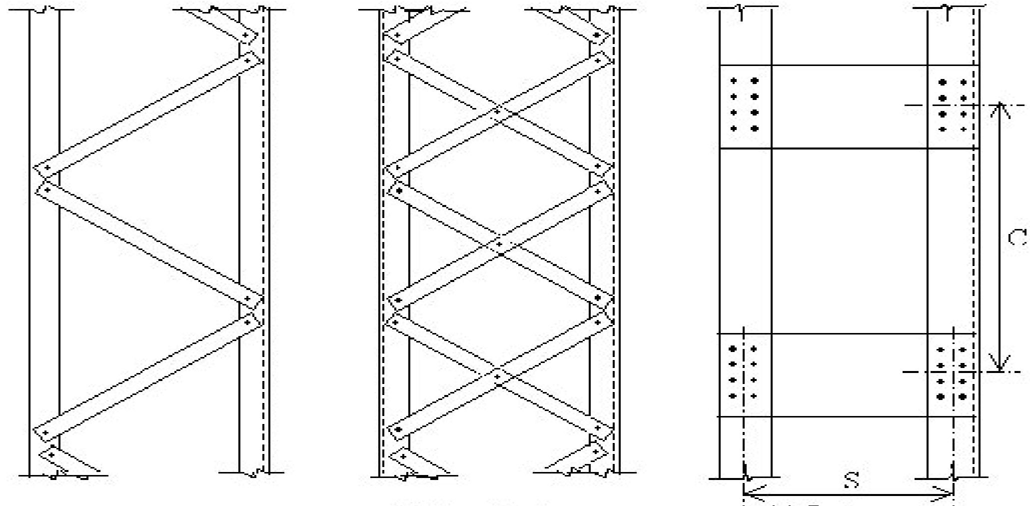
02 marks

02 marks

Q.4 (A)b) Draw neat sketch of lacing and battening. Also state function of same.

Ans

Sketch of lacing and battening:

<table>
<tr><td></td><td></td><td>

(a) Single Lacing (b) Double Lacing (c) Battens

Function: The function of lacing and battening is to hold the various parts of a column straight, parallel at a correct distance apart and to equalize the stress distribution between its various parts.</td><td>03 marks

01 mark</td></tr>
<tr><td>Q.4</td><td>(A)c)

Ans</td><td>Explain "Limits 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 15.7 C. Check whether ISA 90x90x06 nun is semi-compact class or not f_y= 250 MPa.
Limits of width to thickness ration to prevent buckling for a single angle strut
Plate elements of c/s may buckle locally due to compressive stresses. The buckling can be avoided before the limit state is achieved by limiting the width to thickness ratio of each element of c/s subjected to compression due to axial force, moment or shear.
<table>
<tr><td>Component ratio</td><td>ratio</td></tr>
<tr><td>Single angle, or double Angles with the components</td><td>b/t
d/t</td></tr>
</table>

For: ISA 90x90x6 mm thick
b/t_f =90/6=15 between 10.5 ε and 15.7 ε where $\varepsilon = (f_y/250)^{½}$
(hence it belongs to class-3 semi compact section)</td><td>03 marks

01 mark</td></tr>
<tr><td>Q.4</td><td>(A)d)

Ans</td><td>What is local buckling in case of compression member? What is its effect? What is to be done to prevent it?
• **Local buckling in case of compression members:** the individual elements of column i.e. flange or web may buckle locally forming wrinkles. This type of buckling causing column failure is called local buckling.
• **Effect:-**Local buckling reduces overall load carrying capacity of the member
• **Prevention**:-Adopt higher thickness of element that is by controlling width to thickness ratio as per IS –CODE requirement.</td><td>02 marks

01 mark
01 mark</td></tr>
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Q.4 (B)a) Explain gross section yielding and net section rupture in case of design strength of tension member. Also write two measures taken to prevent rupture.

Ans

- **Gross Section Yielding:-** When a tension members is subjected to tensile forces although the net cross sectional yield first, the deformation within the length of connection will be smaller than the deformation in the remainder of tension member.it is because the net section exist within a small length of the member. And the total elongation is the product of the length of the member and the strain. Most of the length of the member will have an unreduced cross section , some attainment of yield stress on the gross area will result in larger total elongation. Here larger deformation is Limit state not the yield . To prevent excessive deformation initiated by yielding the load on the gross section must be small enough so that the stress on the gross section is less than the yield stress.That is

02 marks

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

T = Ag fy

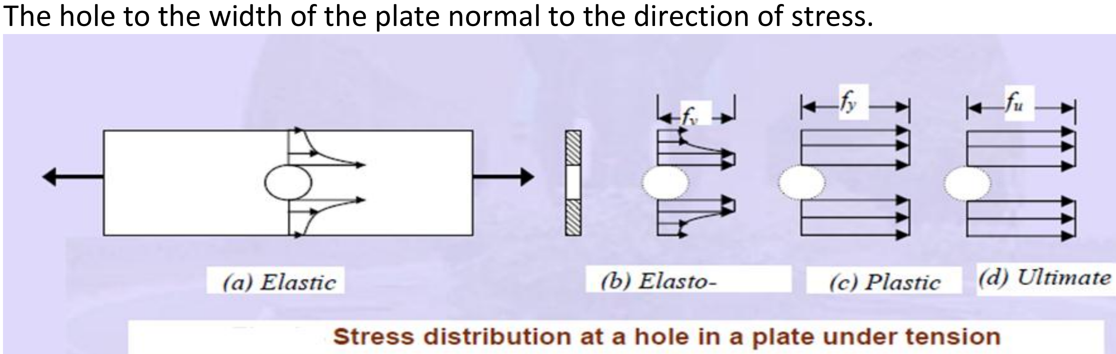
Design strength = Ag fy /ym0

Ym0 = partial safety factor = 1.1

- **Net Section Rupture**

Frequently plates under tension have bolt holes. The tensile stress in a plate at the cross section of a hole is not uniformly distributed in the Tension Member: Behavior of Tension Members elastic range, but exhibits stress concentration adjacent To the whole. The ratio of the maximum elastic stress adjacent to the hole to the average stress on the net cross section is referred to as the Stress Concentration Factor. This factor is in the range of 2 to 3, depending upon the ratio of the diameter of The hole to the width of the plate normal to the direction of stress.

02 marks



Stress distribution at a hole in a plate under tension

To prevent the failure of tension member by net section rupture $T<A_n \times f_u$

$T_{dn}=T/ \gamma_{m1}$

Design strength = $T_{dn}= 0.9\, A_n\, fu /\gamma_{m1}$

A_n -net effective area of member

f_u - ultimate stress of material

Y_{m1}- partial safety factor for failure at ultimate stress =1.2

b- width of plate
t – thickness of plate
d- diam. of hole
g- gauge length
p_s- staggered pitch length between bolt hole.
n- no.of bolt hole.

Preventive Measure:-

- To prevent net rupture sufficient amount of edge distance is provided as per IS-800-2007.

As far as possible less nos. of bolt are provided. To reduce the nos. of bolts high strength bolt are provided. — 02 marks

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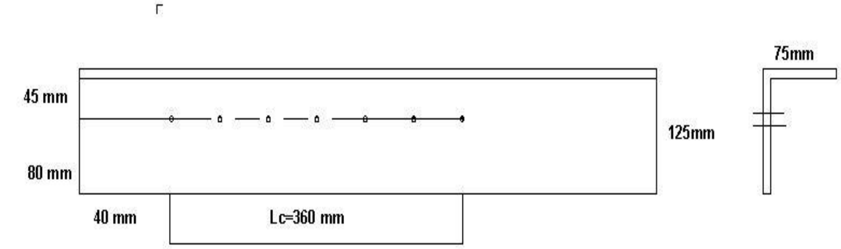
Q.4 (B)b) Design tension member consisting of single unequal angle connected to gusset plate of 12 mm thk. to carry a factored tensile load of 300 kN. Assume single row of 20 mm bolted connection. The length of the member is 2.5 m.
Take f_u = 415 mPa = 0.80

Section (mm)	Area (mm^2)
ISA 100x75x8	1336
125x75x8	1588
150x75x8	1748

Ans
Area required from the consideration of yielding =1.1x300x1000/250
=1320 mm^2
TRY ISA 125X75X8, Which has a gross area **Ag**= **1588 mm^2**
Strength of 20 mm bolt:

a) **In single shear**=[0+0.78x$(20)^2$/4]x 400/1.25x√3
=45272 N

b) Strength in bearing : taking e= 40 mm, p=60 mm — 01 mark
K_b is smaller of 40/3x22, (60/3x22)-0.25,400/410,1.0
i.e. K_b = 0.606
design strength of bolt in bearing = 2.5x K_b x dtx f_u/1.25
design strength of bolt in bearing = 1 x2.5x0.606x20x8x400/1.25
Design Strength of Bolt In Bearing **= 45272 N**
(therefore, Bolt value = 45272 N)
Nos.Of Bolt Required = 300 X1000/**45272**
Nos.Of Bolt Required = **6.62**
(Therefore, provide **7 Nos.** of bolt in a row.)

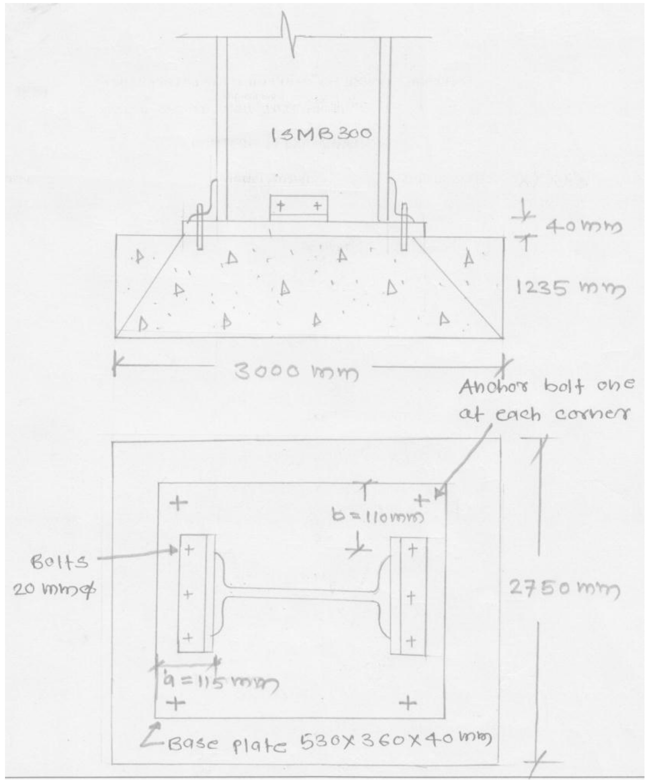
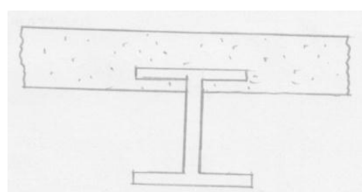
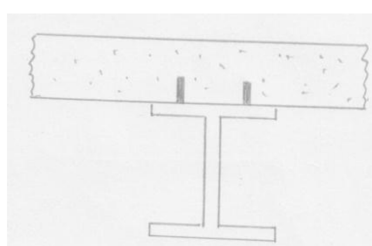
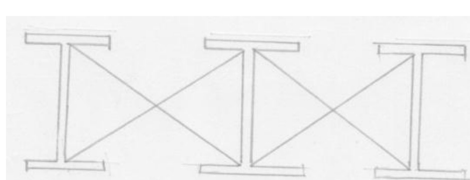


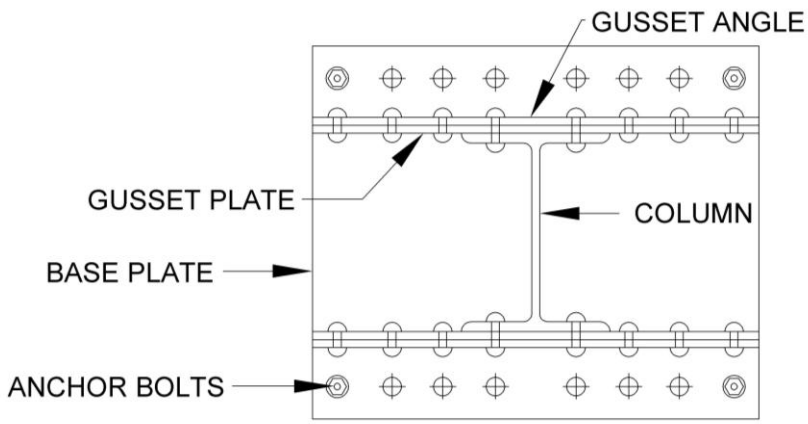
01 mark

		Checking the design : **(a)strength against yielding =**$A_g f_y / y_{m0}$ **=**1588x250/1.1 **=**360909.09 N **>300000 N** **(OK)**	01 mark
		(b) strength of plate in rupture : **Area of connected leg** **Area of connected leg** $\mathbf{A_{nc}}$ **=** (125-22-4)x8=**792 mm²** $\mathbf{A_{go}}$ **=** (75-4)x8=**568 mm²**	01 mark
		β=1.4-0.076x(w/t)x(fy/fu)x(b_s/L_c) β = 1.4-0.076 x(75/8)x(250/410)x(112/360) **β=1.264** T_{dn} = (0.9 fu A_{nc}/y_{m1})+(β A_{go} fy/ y_{m1}) T_{dn} = (0.9x410x**792** /1.25)+(**1.264x568** x 250/ 1.1) $\mathbf{T_{dn}}$ **= 233798.4+163279=397077.4 N > 300000 N -------- (OK)**	
		(c) strength against block shear failure A_{vg} =(40+60 X6)X8=**3200 mm²** A_{vn} =(40+60X6-6.5X22)X 8=**2056 mm²** A_{tg} =(125-45)X8=**640 mm²** A_{tn} =(125-45-0.5X22)X8 =**552 mm²** Smallest of two =(A_{vg} x fy /1.732 y_{mo})+(0.9 x A_{tn} x fu /y_{m1}) =(**3200**x 250 /1.732x 1.1)+(0.9 x **552** x 410 /1.25) =(419903.422)+(162950)=**582853.4 N** Smallest of two =(A_{tg} x fy /y_{mo})+(0.9 x A_{vn} x fu /1.732 Xy_{m1}) Smallest of two =(**640**x 250 /1.1)+(0.9 x **2056**x 410 /1.732 X1.25) =145454+350422=**495876 N** **Hence strength of two angles against block failure = 495876 N > 300000 N** **(OK)** **(Hence,use ISA 125X75X8 with 7 Nos. of 20 mm bolt)**	02 marks
Q.5	a) Ans	A hall of size 12m x 20 m is provided with Howe type roof trusses at 4 m c/c. Calculate panel point load in case of DL and LL for following data- i) unit wt. of roof covering = 165N/m² ii) self-wt. of purlin = 100 N/m² iii) wt. of bracing = 60 N/m² iv) rise to span ratio = 1/5 v) total no. of panels = 08 Given: i) Unit wt. of roof covering = 165N/m² ii) Self-wt. of purlin = 100 N/m² iii) Wt. of bracing = 60 N/m² iv) Rise to span ratio = 1/5 v) Total no. of panels = 08 vi) Span = 12 m. **a. Calculation of Dead load:** i. Self-weight of truss = [(L/3) + 5] x 10 = [(12/3) + 5] x 10 = 90 N/m² ---	01 mark
		ii. Unit weight of roof covering = 165 N/m² iii. Self-weight of purlin = 100 N/m² iv. Weight of bracing = 60 N/m² Hence Total Dead load per m² = 90 + 165 + 100 + 60 = 415 N/m² -----------------------------------	01 mark
		Dead load per intermediate panel point = Dead load per m² x plan area of roof per panel	

		point Dead load per intermediate panel point = 415 x 4 x (12/8) = 2490 N. ------------------------------- Dead load per end panel point = 2490/2 = 1245 N. ------------------------------- **b. Calculation of Live load:** Angle of truss (θ) = $\tan^{-1}[2.5/(12/2)] = 21.80^0$ Live load on purlin = 750 – [(θ – 10) x 20] = 750 – [(21.8 – 10) x 20] = 514 N/m^2 > 400 N/m^2 Hence OK -- Live load on truss = (2/3) x 514 = 342.67 N/m^2 -- Live load per intermediate panel point = Live load per m^2 x plan area of roof per panel point Live load per intermediate panel point = 342.67 x 4 x (12/8) = 2056 N --------------------------- Live load per end panel point = 2056/2 = 1028 N	 01 mark 01 marks 01 marks 01 mark 01 mark 01 mark
Q.5	b) Ans	A industrial building has trusses for 14m span. Trusses are spaced at 3.5m c/c and rise of truss is 3.50m. Calculate panel point load in case of live load and wind load using following data- i) Coefficient of external wind pressure (C_{pe})= - 0.7 ii) Coefficient of internal wind pressure (C_{pi})=± 0.2 iii) Design wind pressure = 1200 N/m^2 iv) No. of panels = 08 Given data: Span = 14 m. Rise = 3.5 m Coefficient of external wind pressure (C_{pe})= - 0.7 Coefficient of internal wind pressure (C_{pi})=± 0.2 Design wind pressure (p) = 1200 N/m^2 No. of panels = 08 **a) Wind load:** i. Design wind pressure $p_d = (C_{pe} - C_{pi}) \times p$ = (-0.7 – 0.2) x 1200 = -1080 N/m^2 -- ii. Angle of truss (θ) = $\tan^{-1}[3.5/(14/2)] = 26.56^0$ --- iii. Inclined length of panel = $(14/8)/\cos 26.56^0$ = 1.956 -- iv. Wind load per intermediate panel point = Design wind pressure (p_d) x inclined panel length x spacing = -1080 x 1.956 x 3.5 = -7393.7 N v. Wind load per end panel point = -7393.7/2 3696.85 N **b) Live load:** Live load on purlin = 750 – [(θ – 10) x 20] = 750 – [(26.56 – 10) x 20] = 418.8 N/m^2 > 400 N/m^2 Hence OK -- Live load on truss = (2/3) x 418.8 = 279.2 N/m^2 --- Live load per intermediate panel point = Live load per m^2 x plan area of roof per panel point Live load per intermediate panel point = 279.2 x 3.5 x (14/8) = 1710 N --------------------------- Live load per end panel point = 1710/2 = 855 N	 01 mark 01 mark 01 mark 01 mark 01 mark 01 mark 01 mark 01 mark

Q.5	c)	A column ISMB - 300 carries an axial load of 1.5 MN. Design a slab base and concrete pedestal for the column. Take SBC of soil as 200 kPa and M20 grade of concrete is used for concrete pedestal. For ISMB-300 consider b_f= 140 mm, t_f= 13.1 mm. Take f_y=250 MPa, γ_{m1} =1.1.	
	Ans	Given: ISMB 300, P = 1500 kN, SBC = 200 kPa, M20 – fck = 20 N/mm^2, f_y = 250 mPa, b_f = 140 mm, t_f = 13.1 mm P_u = 1500 x 1.5 = 2250 kN. Bearing area of base plate (A) = $P_u/(0.6f_{ck})$ $= (2250 \times 10^3) / (0.6 \times 20) = 187500$ mm^2	01 mark
		Size of plate for equal projections a and b $L_p = [(D - B)/2] + \sqrt{[(D - B)/2]^2 + A}$ $= [(300 - 140)/2] + \sqrt{[(300 - 140)/2]^2 + 187500}$ L_p = 520.34 mm say L_p = 530 mm $B_p = A / L_p$ = 187500 / 530 = 353.77 say 360 mm Larger projection a = $(L_p – D) / 2$ = (530 – 300) / 2 = 115 mm Smaller projection b = $(B_p – B) / 2$ = (360 – 140) / 2 = 110 mm	01 mark
		Ultimate pressure from below on the base plate- $W = P_u / (L_p \times B_p) = 2250 \times 10^3 / (530 \times 360) = 11.79$ N/mm^2	01 mark
		Thickness of base plate $t_s = \sqrt{[2.5 \times w \times (a^2 – 0.3b^2) \times \gamma_{mo} / f_y]} > t_f$ $= \sqrt{[2.5 \times 11.79 \times (115^2 – 0.3 \times 110^2) \times 1.1 / 250]}$ = 35.27 mm say 40 mm > 13.1mm (t_f)	02 marks
		Size of concrete block- $A_f = (P_u \times \gamma_{mo}) / SBC \times \gamma_f) = (2250 \times 1.1) / (200 \times 1.5) = 8.25$ m^2	01 mark
		For equal projection- $L_f = [(L_p - B_p)/2] + \sqrt{[(L_p – B_p)/2]^2 + A_f}$ $= [(0.53 – 0.36)/2] + \sqrt{[(0.53 – 0.36)/2]^2 + 8.25}$ L_f = 2.95m say L_f = 3m $B_f = A_f / L_f$ = 8.25 / 3 = 2.75 m Provide M20 concrete pedestal of size 3 m x 2.75 m Actual projection- = $(L_f – L_p) / 2$ = (3000 – 530) / 2 = 1235 mm and = $(B_f – B_p) / 2$ = (2750 – 360) / 2 = 1195 mm Considering 45^0 angle of dispersion, Df = 1235 mm.	01 mark

			01 mark
Q.6	a) Ans	Define-laterally supported beam along with suitable sketch. State any three methods of providing lateral support to the beam. A laterally supported beam is a beam whose compression flange is restrained from buckling. Methods of providing lateral support to the beam. 1. Compression flange embedded in slab.  2. Compression flange connected to slab by shear connectors.  3. Beams connected by braces. 	01 mark 01 mark 01 mark 01 mark

Q.6	b) Ans	State four classification of c/s of beam based on moment-rotation behavior as per IS- 800/2007 Classification of c/s of beam based on moment – rotation behavior as per IS 800-2007 1. Class 1 – Plastic 2. Class 2 – Compact 3. Class 3 – Semi compact 4. Class 4 – Slender	01 mark for each
Q.6	c) Ans	An ISMB - 250 is used for simply supported span of 4m to carry a factored load of 30 KN/m. Check the section for shear only. Take f_y = 250 mPa, t_w = 6.4 mm. Given data- ISMB 250, Span (l_e) = 4.0 m, Factored load (w_d) = 30 kN/m, f_y = 250 mPa, t_w = 6.4 mm Factored shear force (V_d) = w_d x l_e / 2 = 30 x 4 / 2 = 60 kN Check for shear- $V_{dr} = (f_y \times t_w \times h) / (\Upsilon_{mo} \times \sqrt{3})$ $= (250 \times 6.4 \times 250) / (1.1 \times \sqrt{3})$ = 209.94 kN > 60 kN(V_d) hence shear check is satisfied.	01 mark 01 mark 01 mark 01 mark
Q.6	d) Ans	Draw plan of gusseted base showing all components.  PLAN OF GUSSETED BASE	02 marks for fig 02 marks for labeling
Q.6	e) Ans	Write steps to calculate the thickness of base plate used in slab base. Why anchor bolts are used in slab base. Steps to calculate thickness of base plate used in slab base. $t_s = \sqrt{[2.5 \times w \times (a^2 - 0.3b^2) \times \Upsilon_{mo} / f_y]}$ Where- W = Ultimate pressure from below on slab base = P_u / (L_p x B_p) P_u = Factored load. L_p = Length of base plate. B_p = Width of base plate. a = Larger projection. b = Smaller projection. Υ_{mo} = Partial safety factor = 1.1 f_y = Yield stress (250 N/mm^2)	01 mark 01 mark

		Function of anchor bolts: To connect the base plate to concrete block so that stability, stiffness and strength of foundation is achieved.	02 marks