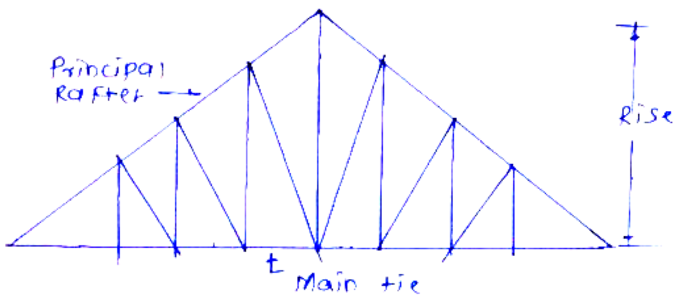
SUMMER- 13 EXAMINATION

Subject code 12140 **Model Answer : Design of Steel Structure** 1/21

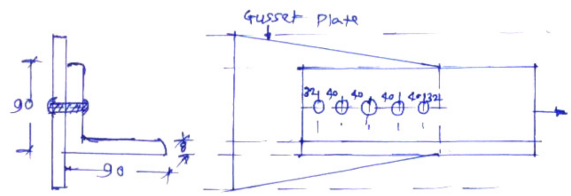
Important instruction to examiners:

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 the candidate and model answer may vary. The examiner may give credit for any equivalent figure drawn.
5) Credits may be given stepwise for numerical problems. In some cases the assumed constant values may be vary and there may be some difference in the candidates answer and model answer.
6) In case of some questions credit may be given by judgment on part of examiner of relevant answer based on candidates understanding.
7) For programming language papers, credit may be given to any other program based on equivalent concept .

Q.No.	Answer	Marks
1. A) a)	Advantage- 1. Extensively useful for large span industrial structures, bridges, towers and communication networks, steel overhead tanks. 2. Steel has many good mechanical properties like malleability, ductility, elastoplasticity (i.e. more ultimate strength and too large strains). 3. It is most appropriate material to construct earthquake resistant structures due to more ductile nature. 4. It is easy to fabricate by riveting or welding to any desired shape. Disadvantages- 1. Steel is a very costly material. 2. It is susceptible to corrosion and hence required corrosion treatment periodically. 3. It requires skill labour for erection. 4. Creates noise and requires electricity during connection of members.	½ ½ ½ ½ ½ ½ ½ ½
b)	Assumptions made in the theory of riveted joint. 1. The tensile stress is uniformly distributed on the portions of the plate between the rivets. 2. The friction between the plates is neglected. 3. The share stress is uniformly distributed on cross-section of the rivets. 4. The rivets fill the holes completely. 5. The rivet in a group share direct load equally. 6. Bending stress in rivets is neglected.	1 each(ANY four)
c)	Lug angle- lug angle is a short piece of an angle section used at a joint to connect the outside leg of a member, thereby reducing the length of the joint. Use- 1. To reduce the length of joint & include the connectivity of outstanding leg to gusset plate. 2. Lug angle is provided at the beginning of a joint so that it can be effective in shearing load.	1 $1\frac{1}{2}$ $1\frac{1}{2}$

d)	 Fig. Howe truss	Label-2 Dia.-2
1. B) a)	P = 100 KN d_n= 16 mm d=16 + 1.5 = 17.5 mm Anet Required= $\frac{p}{\sigma at} = \frac{100x1000}{150}$ = 666.67 mm^2	½
	Add for allowance 30% + 200mm^2	1
	$\therefore$ Required A gross = 666.67 + 200 = 866.67 mm^2 **As it is not mentioned in question whether to selected equal/unequal angle section so both section can be consider for design purpose (w.r.t. to ref. Table 2 or 3 accordingly).**	½
	First Trial Section- ISA 65 x 65 x 6 mm (A = 744 mm^2) $A_{net} = A_1 + KA_2$ A_1 = t (l-d-t/2) = 6 (65-17.5 - 6/2) = 267 mm^2 A_2 = t (b-t/2) = 6 (65-6/2) = 372mm^2 $K = \frac{3A_1}{3A_1 + A_2} = \frac{3x267}{3x267+372} = 0.68$	½
	$A_{net} = A_1 + KA_2$ = 267 + 0.68 x 372 = 519.96 mm^2 Since $A_{net\ Available} < A_{net\ required}$ 519.96 mm^2 < 666.67 mm^2 ……… (Unsafe)	½

	Second Trial Section ISA 90 x 90 x 8 ($A = 1379\ mm^2$) $A_{net} = A_1 + KA_2$ $A_1 = t\ (l-d-t/2)$ $= 8\ (90-17.5 - 8/2) = 548\ mm^2$ $A_2 = t\ (b-t/2)$ $= 8\ (90-8/2) = 688mm^2$ $K = \frac{3A_1}{3A_1 + A_2} = \frac{3x548}{3x548 + 688} = 0.70$	½
	$A_{net} = A_1+KA_2$ $= 548 + 0.7 \times 688$ $= 1029.6\ mm^2$ Since $A_{net\ Available} > A_{net\ required}$ $1029.6\ mm^2 > 666.67\ mm^2$ ……… (Safe)	½
	Strength of member= A x σ_{at} = 1029.6 x 150 Pat = 154.44 KN	½
	End Connection Strength of 16mm diameter PDS rivet In Single shear, $P_s = \tau_{vf} \times \frac{\pi}{4} \times d^2$ $= 100 \times \frac{\pi}{4} \times (17.5)^2 = 24052.8N = 24.05\ KN$ In Bearing $P_b = \sigma_{pf} \times d \times t$ $= 300 \times 17.5 \times 8 = 42000\ N = 42\ KN$ $\therefore$ Rivet Value (R_V)= Min of Ps & Pb = 24.05 KN $\therefore$ No. of rivets = $\frac{p}{R_v} = \frac{100}{24.05} = 4.15 \cong 5$ No.	½
	Pitch = 2.5 dn = 2.5x16 = 40 mm c/c Edge Distance = 2dn = 2x16=32 mm	½

		½
b)	ISA 50 x 50 x 6 mm, l = 1.25 m (Effective)	
	For single angle ISA 50 x 50 x 6 mm	½
	A = 568 mm^2	
	r_{min}= r_v= 9.6 mm	
	l_{eff} = 0.85 x l = 0.85 x 1.25	1
	= 1.0625 m	
	= 1062.5 mm	1
	$\lambda = \frac{l_{eff}}{r_{min}} = \frac{1062.5}{9.6} = 110.67$	
	Cal, σ_{ac} for λ =110.67	
	From Ref. Table No. 1	
	For λ_1= 110 σ_{ac1} = 72	
	For λ_2= 120 σ_{ac2} = 64	½
	$\therefore \sigma_{ac}=\sigma_{ac1} - \frac{\sigma_{ac1} - \sigma_{ac2}}{\lambda_2 - \lambda_1}(\lambda - \lambda_1)$	1
	$= 72 - \frac{72-64}{120-110}$ (110.67-110)	
	σ_{ac} = 71.464 mPa	1
	P_{ac}= A x σ_{ac}	
	= 568 x 71.464	1
	= 40591.5 N	
	P_{ac}= 40.59 KN	
Q.2 a)	d= dn + 1.5 = 20 + 1.5 = 21.5 mm	½
	Strength of rivet	
	In Single shear, $P_s = \tau_{vf}$ x $\frac{\pi}{4}$ x d^2	
	= 100 x $\frac{\pi}{4}$ x $(21.5)^2$ = 36305.03 N	1

In Bearing $P_b = \sigma_{pf} \times d \times t$ 1

$= 300 \times 21.5 \times 10 = 64500$ N

$\therefore$ Rivet Value (R_V)= Min of Ps & Pb

= 36305.03 N = 36.30 KN 1

$P_{at} = A \times \sigma_{at} = 250 \times 10 \times 150 = 375000$ N 1

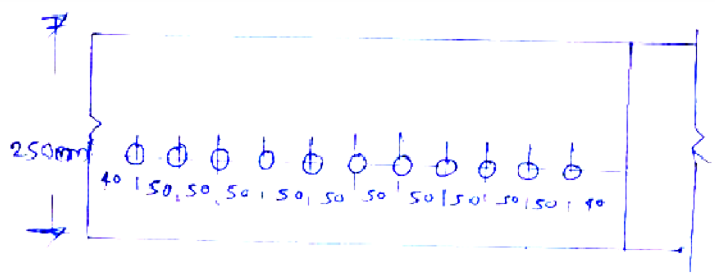
= 375 KN

$\therefore$ No. of rivets $= \dfrac{P_{at}}{Rv}$ 1/2

$= \dfrac{375}{36.30} = 10.32 \cong 11 Nos.$

Pitch = 2.5 dn = 50 mm c/c 1

Edge Distance = 2dn = 40 mm 1

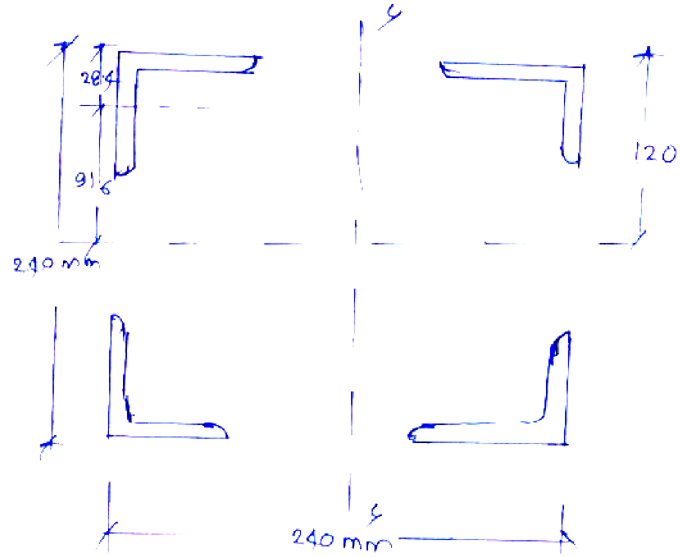


1

2. b) P = 1000 KN, overall dimension = 240 mm x 240 mm 1

$A_{approx.} = \dfrac{p}{\sigma ac} = \dfrac{1000x1000}{120} = 8333$ mm^2

Try 4 ISA 100 x 100 x 10 mm



1

$I_{xx} = 4\,[I_x + A_1h^2]$ ½

$= 4\,[\,177 \times 10^4 + 1903 \times 91.6^2]$

$= 70948942\ mm^4 \cong 70.9 \times 10^6\ mm^4$ 1

$A = 4A_1 = 4 \times 1903 = 7612\ mm^2$ ½

$r_{min} = r_{xx} = r_{yy}$

$r_{min} = \sqrt{\dfrac{I_{xx}}{A}} = \sqrt{\dfrac{70948942}{7612}}$ ½

$r_{min} = 94.54$ mm ½

$\lambda = \dfrac{le}{r_{min}} = \dfrac{4000}{96.54} = 41.43$ 1

Cal, σ_{ac} for λ =41.43

From Ref. Table No. 1

For $\lambda_1 = 40$ $\sigma_{ac1} = 139$

For $\lambda_2 = 50$ $\sigma_{ac2} = 132$

$\therefore \sigma_{ac} = \sigma_{ac1} - \dfrac{\sigma_{ac1} - \sigma_{ac2}}{\lambda_2 - \lambda_1}(\lambda - \lambda_1)$ ½

$= 139 - \dfrac{139-132}{50-40}\ (41.43 - 40)$

$\sigma_{ac} = 137.99$ mPa ½

$P_{ac} = \sigma_{ac} \times A = 137.99 \times 7612$ 1

$= 1050.45$ KN > 1000 KN……. (Safe)

2. c)

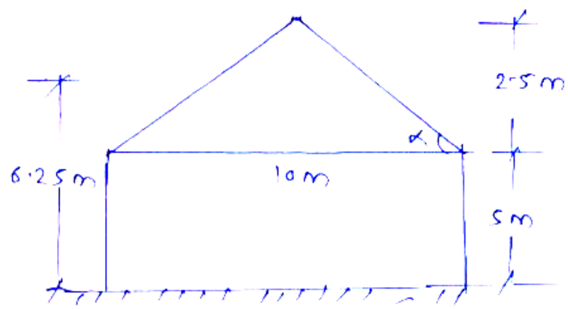
Design wind speed $(V_z) = V_b \times k_1 \times k_2 \times k_3$

$= 44 \times 1.0 \times 0.8 \times 1.0$

$= 35.2$ m/s 1

Design wind pressure pd $= 0.6\ vz^2$

$= 0.6 \times (35.2)^2 = 743.42\ N/m^2 \cong 750\ N/m^2$ ½



Rise = ¼ x 10 = 2.5 m ½

Mean height of roof above G.L. = 2.5 /2 +5

$= 6.25$ m ½

Basic wind pressure P = 750 N/m^2

Slope of roof, $\alpha = \tan^{-1}\left(\frac{2.5}{5}\right) = 26.56$

Case- I Wind normal to ridge external wind pressure coefficient 'C_{pe}' will be as below.

Slope	For wind angle 0^o		Wind Angle 90^o	
	EF	GH	EG	FH
20^o	-0.4	-0.4	-0.7	-0.6
30^o	0	-0.4	-0.7	-0.6

On EF Slope, Cpe = - [0.4 – 0.4 x 6.56/10] ½

$= -0.1376$ ½

On GH Slope $C_{pe} = -0.4$

Internal air pressure co-efficient for normal permeability Cpi = $\pm$ 0.2

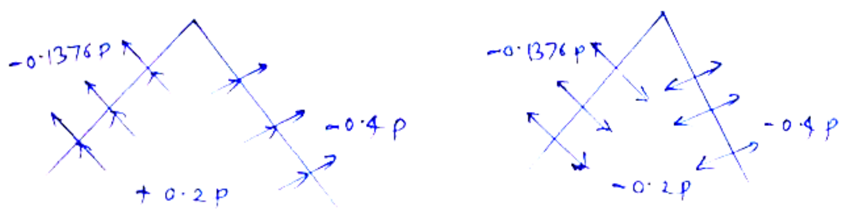
Combined wind pressure = (External wind pressure $\pm$ Internal wind pressure) ½

On EF slope = (-0.1376 – 0.2) x 750 = -253.2 N/m^2 (Uplift)

= $(-0.1376 + 0.2) \times 750 = 46.8$ N/m^2 (Downword) ½

On GH slope = $(-0.4 – 0.2) \times 750 = -450$ N/m^2 (Uplift) ½

= $(-0.4 + 0.2) \times 750 = -150$ N/m^2 (Uplift) ½



a) with +ve internal pressure

b) with -ve internal pressure.

Case-II) Wind Parallel to ridge

External wind pressure coefficient C_{pe}

On both slope for 1/4th length of building = -0.7

On both slope for 1/2nd length of building = -0.6

Cpi = $\pm$ 0.2

$\therefore$ Combined wind pressure = (External wind pressure $\pm$ Internal wind pressure)

Near Gable end (EG) = $(-0.7 + 0.2) \times 750 = -375$ N/m^2 (Uplift)

= $(-0.7 - 0.2) \times 750 = -675$ N/m^2 (Uplift) ½

Internal bays (FH) = $(-0.6 +0.2) \times 750 = -300$ N/m^2 (Uplift) ½

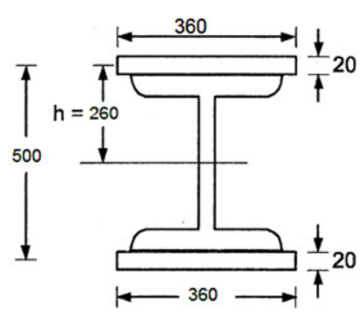
= $(-0.6 - 0.2) \times 750 = -600$ N/m^2 (Uplift) ½

The roof should be designed for the worst effect of wind on studying the calculation. It is evident that the roof will be subjected to maximum uplift when the wind blow parallel to ridge & internal pressure is +ve. ½

Hence roof will be designed for a pressure of 675 N/m^2 on both roof slope. ½

3(a)	Reasons for providing built-up beam section They are required when 1. The available rolled steel sections are insufficient. 2. The span of beam is long and loads are heavy 3. The depth of the beam is fixed and for that depth, section modules of any available section is insufficient 4. When maximum bending moment for a beam is so large that a single available section cannot be provide the necessary moment of resistance.	1 mark each
3(b)	$M_{max} = \frac{wl^2}{8} = \frac{40 \times 4^2}{8} = 80$ kNm $v = \frac{wl}{2} = \frac{40 X 4}{2} = 80$ kN required section modules $Z\ reqd = \frac{M_{max}}{\sigma_{bc}} = \frac{M_{max}}{0.66\ f_y}$ $= \frac{80\ X\ 10^6}{0.66\ X\ 250}$ $= 484.848 \times 10^3\ mm^3$ **Note:** **As per Reference table No.4 in question paper the maximum available $Z_{XX} = 222.4\ cm^3$ for ISLB225. Therefore all the given sections in Reference table No.4 in question paper are not satisfying the requirement of Section modulus and will fail or are unsafe.** **So full marks should be given to the students even if he has attempted the question.** Try ISLB300 giving $Z_{XX} = 488.9 \times 10^5\ mm^3$ $t_w = 6.7$ mm and $I_{xx} = 7332.9 \times 10^4\ mm^4$ **Check for shear** $\tau_{va}\ cal = \frac{V}{t_w X h} = \frac{80\ X\ 10^3}{300\ X\ 6.7} = 39.80$ MPa 39.80 < 0.4fy i.e. 100 MPa	1 1 2

	$\therefore$ safe in shear **Check for deflection** $\delta_{max} = \frac{5}{384}\frac{wl^4}{EI}$ $= \frac{5}{384} X \frac{40\ X\ 10^3\ X\ 4^4\ X\ 1000^3}{2\ X\ 10^5\ X\ 7332.9\ X\ 10^{\ 4}}$ (Assume E= 2 x 10^5 MPa) δmax = 9.09 mm δallowable = $\frac{l}{325} = \frac{4000}{325}$ = 12.30 mm As δmax < allowable **$\therefore$ Provide ISLB300 @ 37.7 kg/m**	
3 (C)	load on purlin DL+ L.L. = 1.0+ 2.3 = 3.3 kN/m $M_{max} = \frac{wl^2}{10} = \frac{3.3\ X\ 3.5^2}{10}$ = = 4.0425 kNm Z reqd $= \frac{M_{max}}{\sigma_{bc}} = \frac{4.0425\ X\ 10^6}{165}$ (0.66 Fy = 165 Mpa) = 24.5 x 10^3 mm^3 Mimi. width of leg (parallel to roof) = L/60 = 3500/60 = 58.33 mm Mimi. depth of leg (perpendicular to roof) = L/45 = 77.77 mm **Note: Selection of any angle section is not possible w.r.t value of section modulus Z as values of Z are not mentioned in the reference tables for angles in question paper. Therefore student should get full marks even if he has attempted up to above step.** $\therefore$ Provide ISA 100 x 75 x 12 mm giving Z_{xx} 27.9 x 10^3 mm^3	1 1 1 1
3(d)	MB = Indian Standard Medium weight Beam SC = Indian Standard Column section MC = medium weight channel with sloping flange ISPG = Indian standard gate channel	1 1 1 1
3(e)	**Four loads acting on steel structure**	1 mark each

	1. Dead Load - it is the self wt of the structural member. It is permanent constant in magnitude and fixed is position.	for any four loads
	2. Live Load - it is known as imposed loads it includes wt. of material stored, movable equipments, furniture, persons using the floor.	
	3. Wind Load - It is the pressure exerted by wind. It depends upon geographical location of the structure, its height, permeability, shape and orientation, wind velocity etc.	
	4. Earthquake Load - When structure is subjected to ground motion in an earthquake, the structure is subjected to vibratory loads in three mutually perpendicular direction	
	5. Show Load - In the areas of show fall structure is subjected to show load which depends upon shape of the roof, is capacity to retain the show.	
	6. Impact Loads - Impact, vibration, temp effect, etc. produce loads on structure.	
	7. Erecting loads - Loads due to placing, or storage and erection equipments are considered as erection loads.	
4A (a)	Ixx = Ixx of section + Ixx of 2 plates	
	$= 38579.0 \times 10^4 + 2(\frac{360 X 20^3}{12} + 360 \times 20 \times 260^2)$	2
	$= 135971 \times 10^4 \ mm^4$	
	$M = \frac{I}{Y_{max}} \times \sigma_{bc} \quad = \frac{135971 \text{ X } 10^4}{270} X165 = 830.93 \times 10^6$ Nmm = 831 kNm	1
	Equate this M to BM_{max}	
	$\therefore$ $M = M_{max}$	
	$831 = wl^2/8$	
	$\therefore$ $831 = w \times 8.5^2 / 8$	
	$\therefore$ **w = 92.00 kN/m**	
		
	The safe UDL the beam can carry (inclusive of self weight) = 92.00 kN/m	1

4A(b)	$\theta = \tan^{-1}$ (Rise/L/2) $= \tan^{-1}$ (3/16/2) $= \tan^{-1}$ (0.375) $\theta = 20.56^0$ Live load intensity for purlins = 750 - (θ-10) x 20 = 750 - (20.56 -10) x 20 = 532.8 N/m^2 Live load intensity for trusses = 2/3 x Live load intensity for purlins = 2/3 x 532.8 = 355.2 N/m^2 Total Live load = Live load intensity for trusses x spacing x span = 355.2 x 3.5 x 16 = 198912 N Live Load per panel = 198912/8 = 24864 N Live Load per panel point = 24864/2 = 12432 N	½ ½ 1 1 ½ ½
4A(c)	$l_{effective}$ = 3000 mm For ISA 65 x 65 x 06 mm r_{mini} = 12.6 mm slenderness ratio $\lambda = l/r_{min}$ = 3000/12.6 = 238.09 since $\lambda > 180$ but $\lambda < 250$ Such section is used to carry comp. load due to wind/earthquake forces only. **Note: slenderness ratio λ for ISA 65 x 65 x 06 for span of 3000 mm is 238.09. For calculation of σ_{ac} corresponding to this value of λ $_{required}$ range of λ is not given in reference table.** **Therefore students who have attempted this question should get full marks irrespective of Answer.**	4

	from table 5.1, IS : 800-1984 σ_{ac} = 21 - (21-20/240-230) (238.09 -230) = 20.19 MPa Increase σ_{ac} by 33.33 due to wind forces σ_{ac} ' = 20.19 x 1.3333 = 26.92 MPa Permissible load = σ_{ac} ' x A = 26.92 x 744 = 20.027 KN > 15 KN $\therefore$ ISA 65 x 65 x 6 is safe for above loading.	
4A (d)	**Necessity of Column bases :** Column bases are required to distribute the concentrated column load over a certain definite foundation area and to ensure connection of the lower column end to the foundation in accordance with the planned design. **Use of steel base plate in column bases:** Steel base plate is used in column bases to provide adequate strength, stiffness and area to spread the load upon the concrete or other foundation or other supports without exceeding the allowable stress an such foundation under any combination of loads and bending movements.	2 2
4B(a)	i) When tack rivets are not provided d = 20 + 1.5 = 21.5 mm Strength of single angle A1 = t (l - d- t/2) = 10 (100 - 21.5 - 5) = 735 mm^2 A2 = t(b-t/2) = 10 (100-5) = 950 mm^2 K = $\frac{3A1}{3A1+A2}$ = $\frac{3 X 735}{3 X 735 + 950}$ = 0.542 A net = $A_1 + A_2$ k = 735 + 950 x 0.542 = 1250 mm^2 strength of single angle = A net x σ_{at} = 1250 x 150 = 187.5 kN (Assume σ_{at} = 150 MPa) $\therefore$ strength of 2 ISA = 187.5 X 2 = 375 x 10^3 kN ii) when tacking rivets are provided A_{net} = 2 (Ag - deduction for holes) = 2 (1903 - 21.5 x 10) = 3376 mm^2 strength of 2 ISA = 3376 x 150 = 506.4 kN Required pull if tacking rivets are provided = 506.4 kN	½ ½ ½ 1/2 1 1 1 1

4B(b)	Assume σ_{ac} = 110 MPa $A_{approximate} = P/\sigma_{ac\ assumed}$ = 198 x 10^3 / 110 = 1800 mm^2 Try 2ISA 100 x 65 x 6 mm From reference table No. 3 in question paper. To Find least radius of gyration $I_{yy} = 2\ (I_{yy} + A1\ .h^2)$ $= 2\ [32.4 \times 10^4 + 955\ (14.7 + 8/2)^2]$ $= 1315908\ mm^4$ $r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{1315908}{2\ X\ 955}}$ = 26.25 mm and r_x = 31.8 from reference table No. 3 in question paper $\therefore r_{mini}$ = 26.25 mm $\lambda = l_{eff}/r_{mini}$ = 2000/26.25 = 76.19 $\therefore \sigma_{ac}$ = 112- (112-101/10) x 6.19 = 105.19 MPa load carrying capacity , $P_{ac} = \sigma_{ac}' \times A$ = 105.19 x 2 x 955 = 200913 N = 200.91 kN > 198 kN : ok **Design of connection** d = 20+1.5 = 21.5 mm strength of one 20 mm PDS rivet $P_b = d \times t \times \sigma_{pf}$ = 21.5 x 8 x 300 (assume σ_{pf} = 300 MPa) P_b = 51.6 kN strength in single shear Ps = Area X τ_{vf} $= 2 \times (\pi/4)\ d^2 \times \tau_{vf}$ (assume τ_{vf} for PDS Rivet =100MPa) Ps = 2 x π/4 x 21.5^2 x 100 = 72.61 kN $\therefore$ Rivet value R_v = 51.6 kN $\therefore$ No. of rivets required = load/R_V = 198 x 10^3 / 51.6 x 10^3 = 3.83 say 4 rivets Mini pitch P = 2.5 d_n = 2.5 x 20 = 50 mm Edge distance, e = $2d_n$ = 2 x 20 = 40 mm	½ ½ ½ ½ ½ ½ ½ ½ ½ ½ ½ ½
5a)	SC-250 , cover plate 300 x 25 mm, P = 3000 KN , (SBC) SOIL = 300 KPa , σ_c = 4 MPa	

i) Area of base plate = $\frac{P}{\sigma c}$ = $\frac{3000}{4}$ = 0.75 m^2 ½

ii) Side of square base = $\sqrt{0.75}$ = 0.886 m

ii) thickness of base plate

$$t = \sqrt{\frac{3w}{\sigma bs} - (a^2 - \frac{b2}{4})}$$ ½

$$w = \frac{3000 \times 10^3}{870 \times 870} = 3.96 \text{ N / mm}$$

½

a = b = 570/2 = 285 mm

$$t = \sqrt{\frac{3 \times 3.96}{185} (285^2 - \frac{285^2}{4})}$$ t = 62.54 mm = 65 mm ½

½

provide 870 x 870 x 65 mm slab base

Cleat angle

½

Provide cleat angle of 2 ISA 100 X 100 X 10 mm to secure column with base plate by 20 mm dia. rivet in each leg of angles.

Concrete base

Axial load on column = 3000 KN

10% OF self weight of foundation = 300 KN

Total = 3300 ½

Area of concrete slab = $\frac{3300}{300}$ = 11 m^2

½

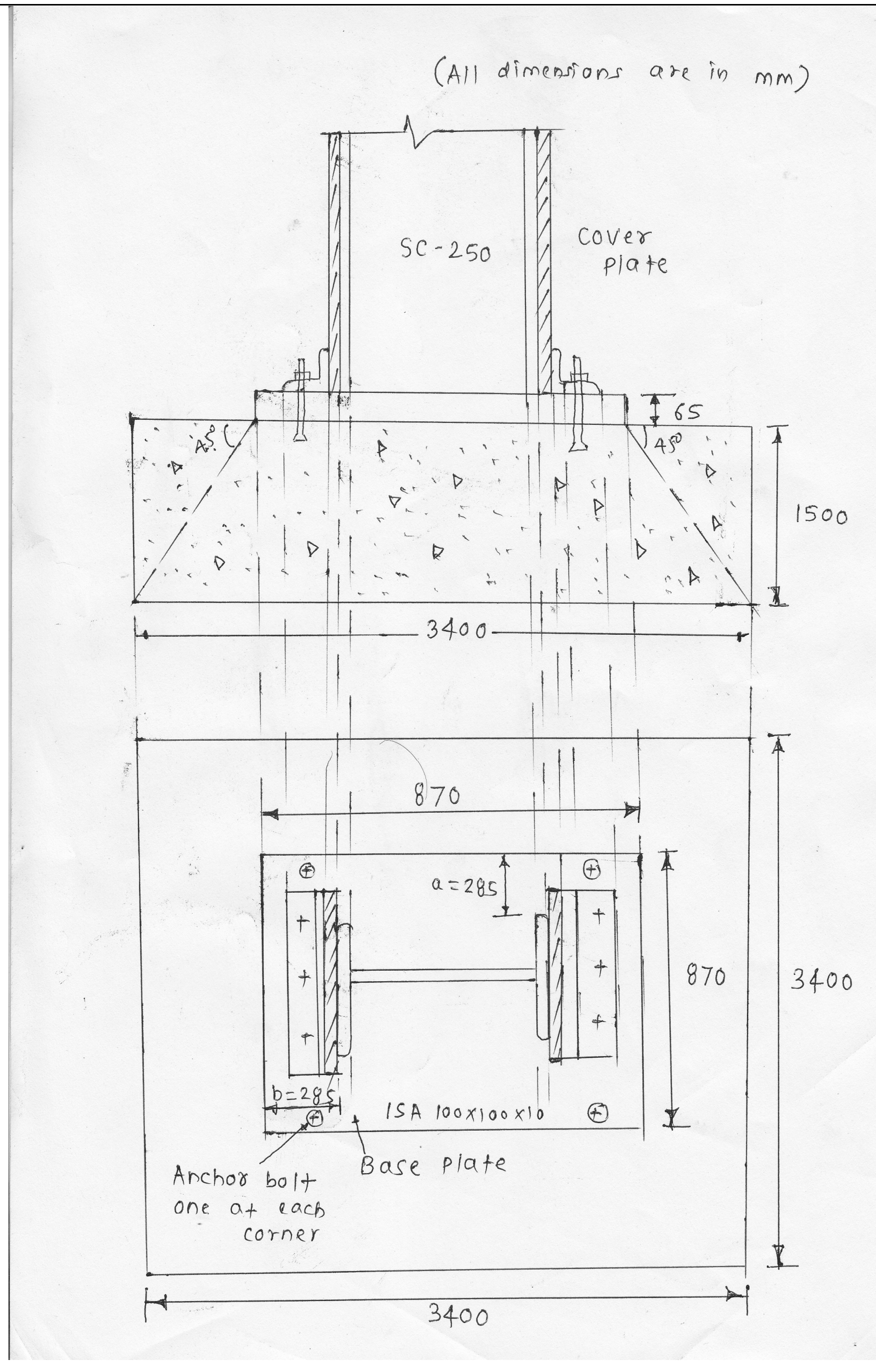
Side of square concrete slab = $\sqrt{11}$ = 3.316 m = 3.4 mm

Assume angle of dispersion = 45^0

Depth of concrete slab = $\frac{1}{2}$ (3.4 – 0.87)

= 1.265 m = 1.5 m

Overall dimension of pedestal = 3.4 m x 3.4 m x 1.5 m 1

		1.5 1.5
5(b)	Span = 20m, spacing = 4m, rise = 3m base point = 10 no. tan ϴ = 3/10 = ϴ = $\tan^{-1}$ 3/10 ϴ = 16.69^0 sloping area = 10.44 x 2 x 4 = 83.52 m^2 dead load due to G.I Sheet = 100 x 83.52 = 8352 N	1 ½ 1

	Due To Purline = Span x spacing of truss x wt/mt = 20 x 4 x150 = 12000 N Plan area = 20 x 4= 80 m^2 due to truss = 105 x plan area = 105 x 80 = 8400 N DUE TO bracing = 75 x 80 = 6000 N Total D.L. = 34752 N = 34.75 KN D.L / Panel point = 3.475 KN <u>Live load calculation</u> L.L. on purline = 750 – (ө - 10) X 20 = 750 – (16.69 – 10) X 20 = 616.2 N/m^2 L.L. on truss = 2/3 x 616.2 = 410.8 N/m^2 total L.L = L.L Intencity X plan area = 410.80 x 80 = 32864 N = 32.86 KN L.L per panel = 32.86 / 10 = 3.286 KN	½ ½ 1 ½ 1 ½ ½ 1
5(c)	Given data I. ISA = 125 X75X12 mm II. Weld = 6mm III. Permissible shear stress = 108.5 Mpa IV. A = 1902mm^2 **SOLUTION :** i) Maximum force in angle = σat x A = 150 X 1902 = 285300N = 285.3 KN ii) Effective Length of weld = $\frac{285 \times 10^3}{108.5 \times 0.7 \times 6}$ = 626.06 says 630 mm	 1 1

from the above figure , the total effective length of weld

$$X1 + X2 + 125 = 630$$ 1

$$X1 + X2 = 505$$

Force in weld per mm length = p_q x 1 x t

$$= 108.5 \times 1 \times 0.7 \times 6$$ 1

$$= 455.7 \text{ N/mm}$$

taking moment of forces about the bottom the bottom edge of the member

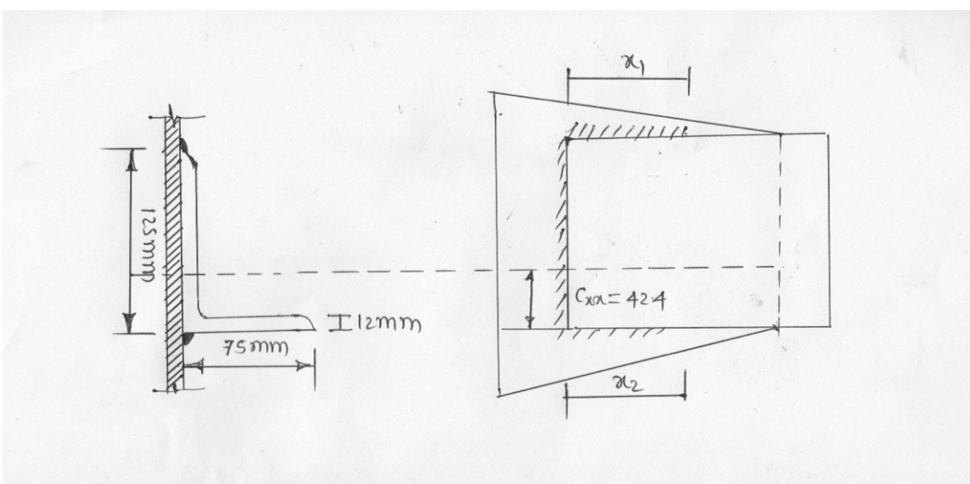
$$455.7 \times X_1 \times 125 + 455.7 \times 125 \times 125/2$$ 1

$$= 285300 \times 42.4$$

$$56962.5\, X1 + 3560156.25 = 12.09 \times 10^6$$ 2

$$X1 = 149.86 \text{ mm} = 150\text{mm}$$

$$X2 = 355 \text{ mm}$$



1

6(a)

i) The process does not involve driving holes the gross sectional area of welding member is effective

ii) Welded structures are comparatively lighter than corresponding Riveted structure.

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

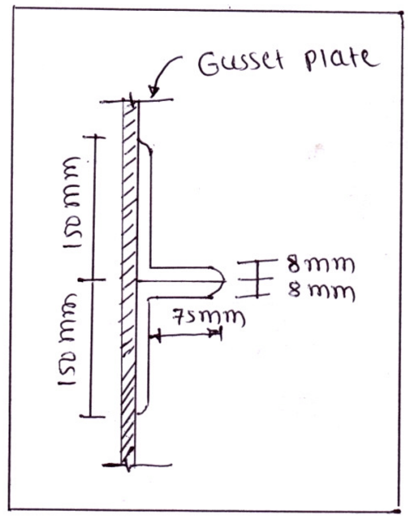
iv) Repair and further connections can be done more easily than riveting.

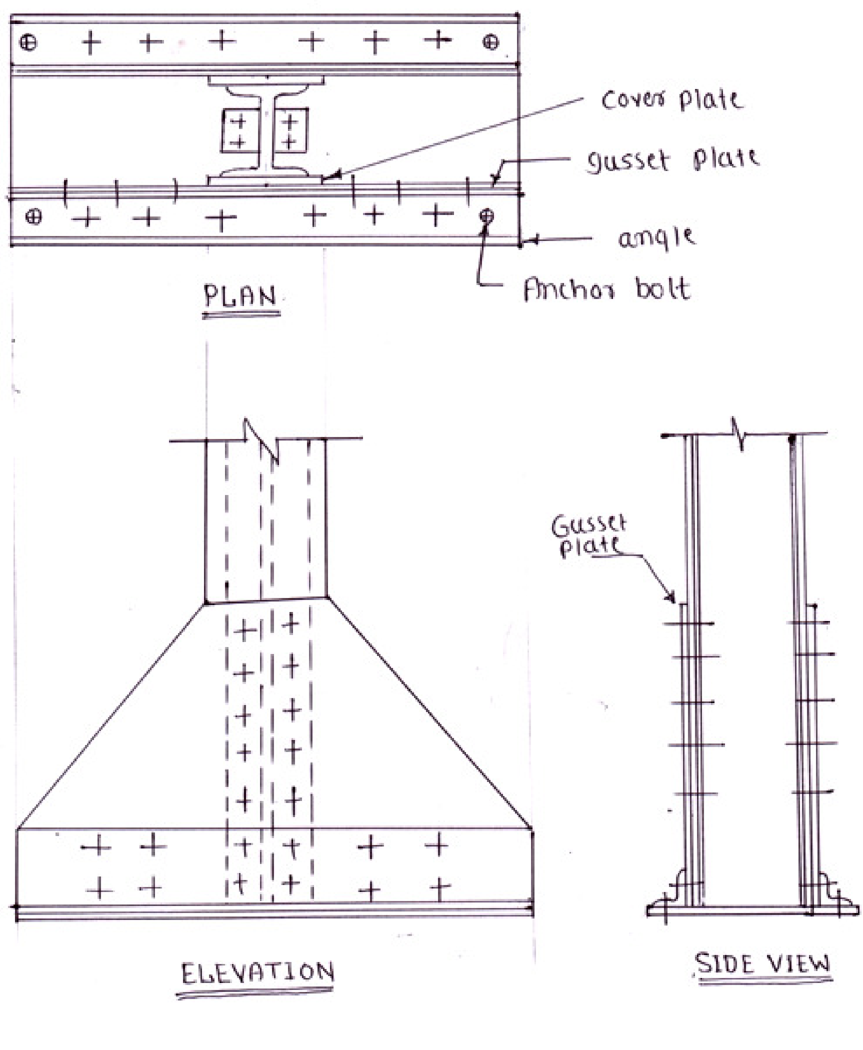
v) Members of such shapes that affored difficulty for riveting can be More easily welded.

vi) A weled connection has better finish and apperance than the Corresponding riveted structure.

vii) Connecting gusset plate,angle can be minimized

1 each to any four

	viii) It is possiable to weld at any point at any part of the structure.but Riveting will always require enough clearance. ix) The process of welding takes less time than riveting. x) The process of welding is less noisy than riveting.	
6 (b)	**Given-:** i) 2ISA -150mm X 75mm X 8mm ii) Diameter of rivets= 20mm dn=20mm d=20+1.5=21.5mm A_1=2t(b1-d-t/2) =2 X 8(150-21.5-8/2) =1992 mm2 A_2=2t(b2-t/2) =2 X 8(75-8/2) =1136 mm2  K=$5A_1/(5A_1+A_2)$= (5 x 1992)/((5 X 1992)+(1136)) = 0.89 $A_{net} = A_1+KA_2$=1992+ 0.89 x 1136 Anet =3003.04 mm^2 Strength (P) = Anet X 6at =3003.04 X 150 **<u>P = 450.45 KN</u>**	½ ½ ½ ½ 1 1
6(c)	**Given-:** i) <u>Sc140@33.21kgf/m</u> ii) L=4m iii) A=4240mm2 iv)r_x =58.5mm v) r_y =32.1mm vi)F_y =250Mpa For column effective held in position and restrained against rotation at one end and other end restrained against rotation but not in position Leff=1.2 X 4 =4.8m =4800mm λ =(Leff/r_{min})=4800/32.1 =149.53 From reference table No-1 λ_1 =140 6_{ac1} λ_2 =140 6_{ac2}	1 1

	6ac =45.285 Mpa Pac = A X 6ac Pac = 4240 X 45.282 **<u>Pac = 191.98 KN</u>**	1 1
6(d)	 Gusseted Base	2 2
6(e)	**Plate Girder –** A plate girder is a built up beam fabricated mainly by plate section.angle section are usally used to connect falnge plates and we plates by rivets. For weled plate girders, flange plates may be weled directly to web plates without angle sections.	

	A plate girder is provide when the span is very large (more than 20m) **Functions of Web plate and Bearing stiffner** a) **Web Plate** – To take shear force b) **Bearing stiffeners-** these are provided at points of concentrated load so the function of bearing stiffners is to take point loads.	2 1 1

Note for Examiner: In the above answers if students are writing some additional points or information which may be correct but not included in the model answer sheet. Examiners are requested to go through each answer carefully.