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.

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 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.NO	SOLUTION	MARKS
Q-1 A	1] objectives of structural Design are	Any
a]	1> Safty	four
	2> Serviceability	2M
	3> Durability	
	4> Economy	
	5> Asthetics	
	2> factor to consider in designing steel	
	structure	Any
	i> stability	four
	2> strength	02
	3> Brittle facture	
	4> fire	
	5> Durability	
Q1A b>	Limit State of serviceability :-	
	Limit state of serviceability is	
	related to the satisfactory performan	
	ce of the structure at working load	
	There are four major types of	
	service ability limit states applica	
	ble to steel structure are as	02M
	follows :-	
	(a) Deflection	½M
	(b) Durability	½M
	(c) Vibration.	½M
	(d) Fireresistance.	½M

Q .NO	SOLUTION	MARKS
Q.1 A c]	partial safty factores for material	

Sr. No	Discription	partial saftey Factor	MARKS
1	Resistance governed by yielding γ_{m0}	1.10	1M
2	Resistance of member to buckling γ_{m0}	1.10	1M
3	Resistance governed by ultimate stress γ_{m1}	1.25	1M
4	Bolted connection in friction and Bearing γ_{mf} & γ_{mb}	1.25 [shop & field fabrication]	1M

Q. 1 A

d] **Shear lag:-**

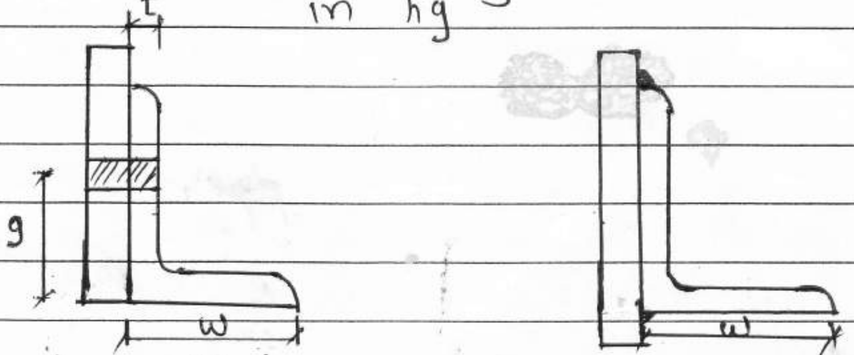
While transferring the tensile force from gusset plate to tension member through one leg by bolts or welds, the connected leg of section (such as angle, channel) may be subjected to more stress than the outstanding leg and finally the stress distribution becomes uniform over the section away from the connection. Thus one part legs behind the other is called as shear lag. 2M

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

$$T_{dn} = 0.9 \frac{A_{nc} f_u}{\gamma_{m1}} + \beta \frac{A_{go} f_y}{\gamma_{m0}}$$ 1M

Where $\beta = 1.4 - 0.076 \frac{w}{t} \times \frac{f_y}{f_u} \times \frac{b_s}{L_c}$

b_s = shear lag width as shown in fig



$b_s = w + g - t$ $b_s = w$ 1M

Fig: Shear Lag width

Q1 B]
a → Given

$f_u = 410\ N/mm^2$ $f_{ub} = 400\ N/mm^2$

$d = 16\ mm$ $d_o = 18\ mm$

$\gamma_{mb} = 1.25$ $P_u = 100\ kN$

Strength of bolt:

Since it is lap joint bolt is in single shear, the critical section being at the root of bolt.

$A_{nb} = 0.78 \times \frac{\pi}{4} \times d^2$

$= 0.78 \times \frac{\pi}{4} \times 16^2 = 156.82$

$\simeq 157\ mm^2$

Design strength of bolt in shear

i.e $V_{dsb} = \dfrac{\frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})}{\gamma_{mb}}$

$= \frac{400}{\sqrt{3}} \cdot \frac{1 \times 157}{1.25} = 29.006 \times 10^3\ N$

$\boxed{\therefore V_{dsb} = 29\ kN}$ 1M

$\therefore$ No of bolts required $= \frac{P_u}{V_{dsb}} = \frac{100}{29}$

$= 3.4 \simeq 4$ No.

$\boxed{\text{No. of bolts required} = 4\ \text{No.}}$

Arranging bolts in single rows

Equating tensile capacity per pitch length

$$T_{dn} = 0.9 \frac{f_u}{\gamma_{m1}} (p - d_o) \cdot t$$

$$29 \times 10^3 = 0.9 \times \frac{410}{1.25} (p - 18) \times 10$$

$$p = \left(\frac{29 \times 10^3 \times 1.25}{0.9 \times 410 \times 10} \right) + 18$$

$$= 27.82 < 2.5 \times d = 2.5 \times 16$$
$$= 40$$ 1M

∴ provide pitch $p = 40$ mm

and edge distance $= 1.7 \times d_o$ [for rough edge]

$$= 1.7 \times 18$$
$$= 30.6 \cong 30$$

k_b is smaller of

i) $\frac{e}{3d_o} = \frac{30}{3 \times 18} = 0.56$

ii) $\frac{p}{3d_o} - 0.25 = \frac{40}{3 \times 18} - 0.25$
$= 0.49$

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

iv) 1

min. value $= 0.49$

$\boxed{\text{Hence } k_b = 0.49}$ 1 M

$\therefore$ design bearing strength

$$V_{dsb} = \frac{V_{npb}}{\gamma_{mb}} = \frac{2.5 \times k_b \cdot d \cdot t \; f_u}{\gamma_{mb}}$$

$$= \frac{2.5 \times 0.49 \times 16 \times 10 \times 410}{1.25}$$

$$= 64288 \text{ N} = 64.29 \text{ kN}$$

$\boxed{V_{dsb} = 64.29 \text{ kN} > 29 \text{ kN}}$ 1 M

$\therefore$ ok No rivision is required

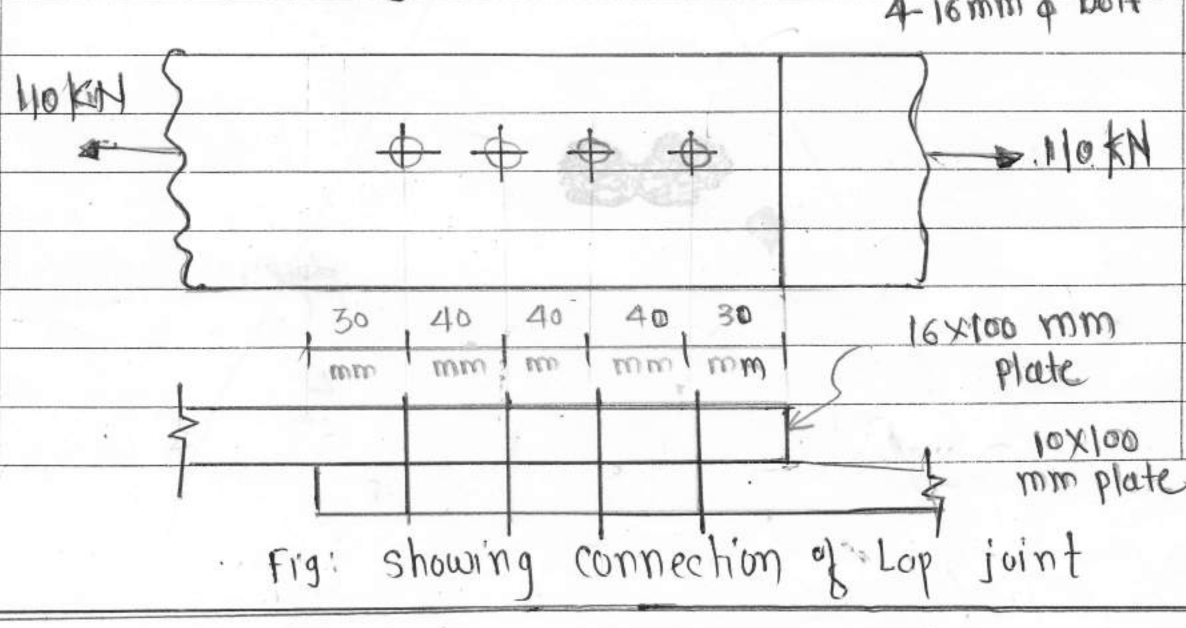
Check for the strength of plate

$$T_{dn} = \frac{0.9 \, A_n \cdot f_u}{\gamma_m} = \frac{0.9 \times (100 - 2 \times 18) \times 10 \times 410}{1.25}$$

$$= 188.93 \text{ kN} > 110 \text{ kN}$$ 1 M

$\therefore$ safe

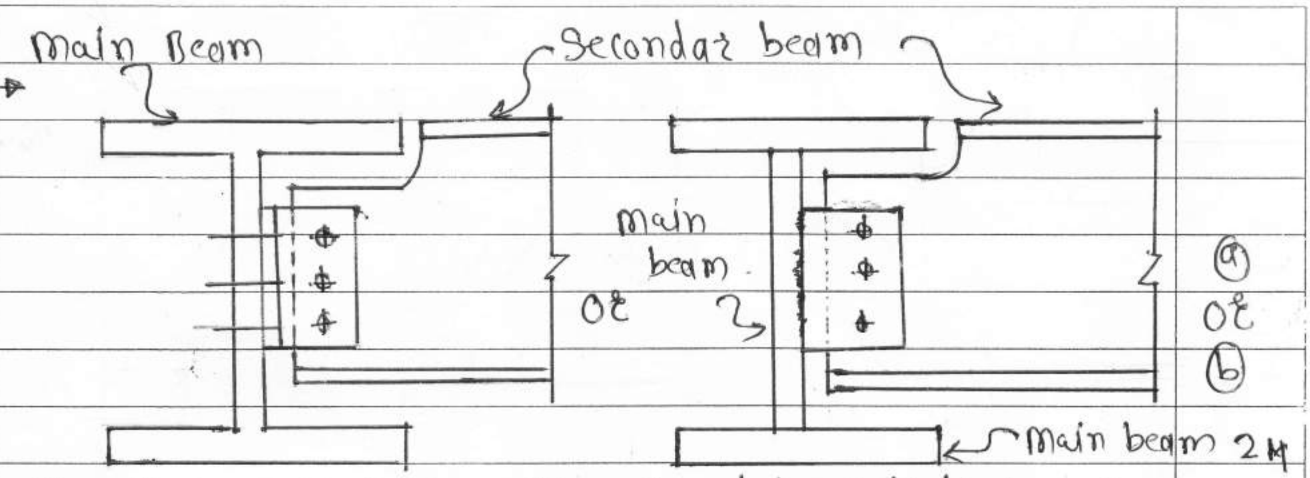
Provide 4-16 mm ϕ bolts at 40 mm pitch with edge distance of 30 mm as shown in fig.



1 M

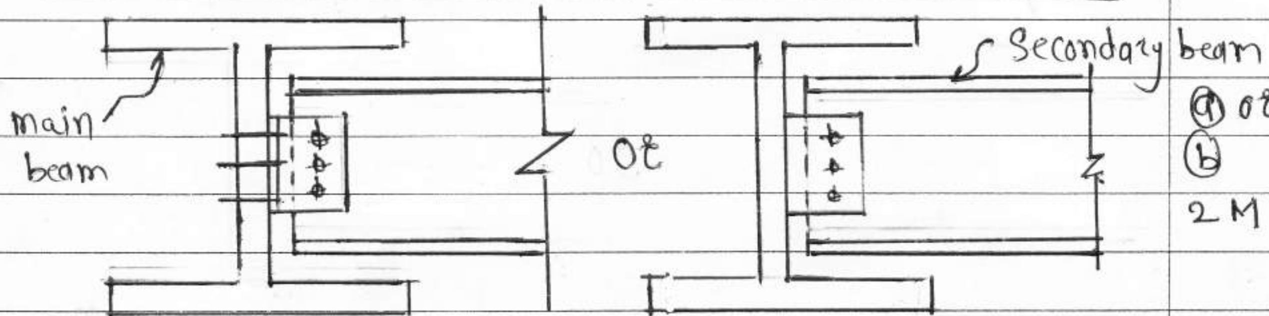
Fig: showing connection of Lap joint

Q.1.B b →



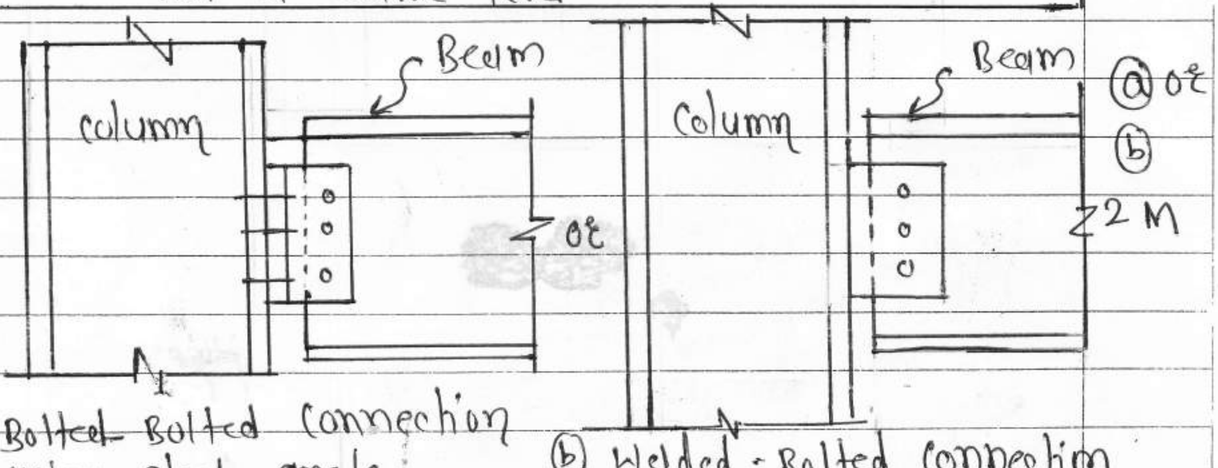
(a) Bolted – Bolted Connection using cleat angle (b) Welded – Bolted Connection Using plate

Fig: Beam to Beam connection When flanges at Same level



(a) Bolted - Bolted Connection using cleat angle (b) Welded - Bolted Connection using plate

Fig: Beam to Beam Connection When flanges are not at same level



(a) Bolted- Bolted Connection using cleat angle (b) Welded - Bolted Connection using plate

Fig : Beam to column connection

Q.2a] Given

$f_y = 250\ N/mm^2 \quad \gamma_{m0} = 1.10$

$f_u = 410\ N/mm^2 \quad \gamma_{mw} = 1.50$

Size of weld = 4 mm
i.e. S

→ Design strength of 80×8 mm plate

$P_{dw} = \frac{f_y}{\gamma_{m0}} A_g = \frac{250}{1.10} \times 80 \times 8$

$= 145454.55\ N$

$= 145.45\ kN$

$\boxed{P_{dw} = 145.45\ kN}$ 1 M

Design stress for site weld

$f_{wd} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.5} = 157.81\ N/mm^2$

$\boxed{f_{wd} = 157.81\ N/mm^2}$ 1M

Design strength per mm length of weld

$P_q = f_{wd} \times t_t$

$= f_{wd} \times 0.7 \times S = 157.80 \times 0.7 \times 4$

$= 441.84\ N/mm$ 1M

$\boxed{\therefore P_q = 441.84\ N/mm}$

Effective length of weld required

$$L = \frac{P_{dw}}{P_q} = \frac{145454.55}{441.84}$$

$\boxed{\therefore\ L = 329.20\ mm \simeq 330\ mm}$ 1 M

In such an arrangement the distance between longitudinal weld shall not exceed 16 t

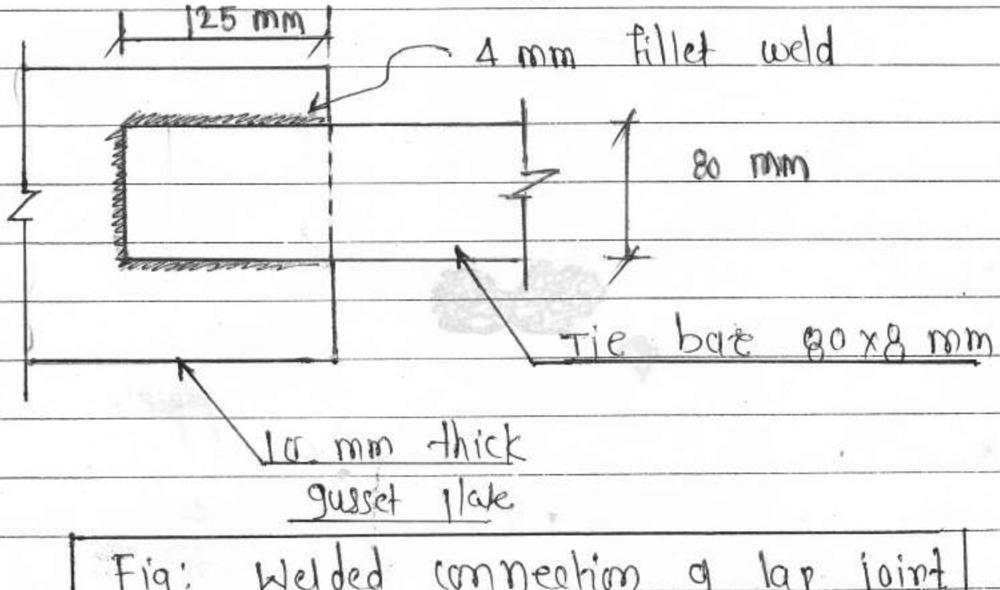
i.e = 16 × 8 = 128 mm 1 M

Let us provide two longitudinal and one transverse weld.

length of transverse weld = 80 mm

Length of each longitudinal weld

$$= \frac{330-80}{2} = 125\ mm$$ 1 M



2 M

Fig: Welded connection of lap joint

Q.2 b] Given

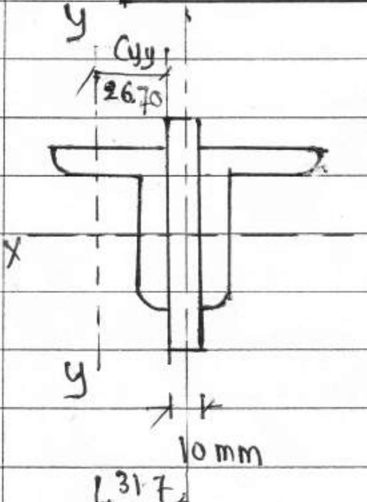
properties of ISA 100 × 100 × 6 mm.

$A = 1167\ mm^2$ $I_{xx} = I_{yy} = 111.3 \times 10^4\ mm^4$

$C_{xx} = C_{yy} = 26.70\ mm$: $L = 2.8\ m = 2800\ mm$

used two bolts.

KL/r	60	70	80	90	100
f_{cd} N/mm^2	163	152	136	121	107



I_{xx} of compound section

$= [I_{xx} \text{ of individual}] \times 2$

$= 111.3 \times 10^4 \times 2$

$\boxed{I_{xx} = 2.226 \times 10^6\ mm^4}$

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

$= 2[111.30 \times 10^4 + 1167 \times 31.7^2]$ 1M

$\boxed{I_{yy} = 4.57 \times 10^6\ mm^4}$

As the angle are equal angles, with calculation we can say that $I_{min} = I_{xx}$

$\therefore r_{min} = \sqrt{\dfrac{I_{xx}[\text{compound}]}{2A}}$

$= \sqrt{\dfrac{2.226 \times 10^6}{2 \times 1167}}$ 1M

$\therefore r_{min} = 30.88\ mm$

	Slenderness ratio $= \frac{kL}{r_{min}}$	
	$= \frac{0.85 \times 2800}{30.88}$	
	$\boxed{\lambda = 77.07}$	1M
	for $\lambda = 77.07$ by interpolation fcd	
	$f_{cd} = 152 - \left[\frac{152-136}{80-70} \times (77.07-70)\right]$	
	$\boxed{f_{cd} = 140.70 \ N/mm^2}$	
	$\therefore P_d = A \times f_{cd}$	
	$= 2 \times 1167 \times 140.70$	
	$= 328393.8 \ N$	
	$\boxed{\therefore P_d = 328.394 \ kN}$	1M
	(b) <u>For welded connection</u>	
	Slenderness ratio $\lambda = \frac{KL}{r_{min}} = \frac{0.7 \times L}{30.88}$	
	$= \frac{0.7 \times 2800}{30.88}$	
	$\boxed{\lambda = 63.47}$	2 M
	for λ 63.47 by interpolation fcd	
	$f_{cd} = 163 - \left[\frac{163-152}{70-60} \times (63.47-60)\right]$	
	$\boxed{f_{cd} = 159.18 \ N/mm^2}$	1M
	$\therefore P_d = A \times f_{cd} = 2 \times 1167 \times 159.18$	
	$= 371526.12 \ N = 371.53 \ kN$	1 m

$\boxed{P_d = 371.53 \ kN}$

Q.2c] → From steel table explore the properties of ISMB 350 @ 514 N/m

$h = 350$ mm $bf = 140$ mm, $tf = 14.2$
$tw = 8.1$ mm
$Ze = 778.9 \times 10^3$ mm^3
$Zp = 889.57 \times 10^3$ mm^3
$r_1 = 14$ mm

Classification of section →

$$d = h - 2(tf + r_1)$$
$$= 350 - 2(14.2 + 14) = 293.6 \text{ mm}$$

$$\frac{bh}{tf} = \frac{(140/2)}{14.2} = 4.93$$

$$\frac{d}{tw} = \frac{293.6}{8.1} = 36.25$$

As $\frac{bh}{tf} < 9.4$ and $\frac{d}{tw} < 84$

Section classificition : Plastic 2 M

∴ Design bending strength →

$$Md = \frac{\beta b \times Zp \times fy}{\gamma mo}$$
$$= \frac{1 \times 889.57 \times 10^3 \times 250}{1.1}$$ [$\beta b = 1$ for plastic]
$$= 202.175 \times 10^6 \text{ N.mm}$$

$\boxed{Md = 202.175 \text{ kN.m}}$ 2 M

To find out u.d.l →

$Md = \frac{wl^2}{8}$

$202.175 = \frac{w \times 5^2}{8}$

$w_f = 64.696$ kN/m

∴ Working u.d.l $w = \frac{64.696}{1.5}$

$\boxed{w = 43.13 \text{ kN/m}}$ 2 M

Check for deflection →

$\delta allow = \frac{L}{300} = \frac{5000}{300} = 16.67$ mm

$\delta max = \frac{5}{384} \frac{wL^4}{EI}$

$= \frac{5}{384} \times \frac{64.696 \times 5000^4}{2 \times 10^5 \times 13630.3 \times 10^4}$

$= 19.3$ mm

$\boxed{\delta max > \delta allowable \quad \therefore \cancel{ok}}$ 2 M

Hence ok.

Q No.3 (a) **In steel construction bolts of grade 4.6 are generally used. What do you mean by grade 4.6** **04 M**

Grade 4.6 are generally used because **02 M**

- This creates a very tightly fitting bolt
- slip is negligible
- Higher stresses can be used.
- limited corrosion

What do you mean by grade 4.6 **02 M**

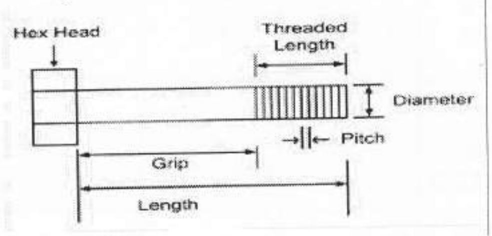
- The first digit relates to the ultimate strength of the material, whilst the second is the ratio of yield stress to ultimate strength.
- Grade 4.6 bolts have an ultimate material strength of 400 N/mm2, and the yield (or proof) stress is 60% of the ultimate strength.(f_{ub}=400Mpa,f_{yb}=240Mpa)

Sketch any one type of bolt. Why drilled holes are preferred over punched holes? **04 M**

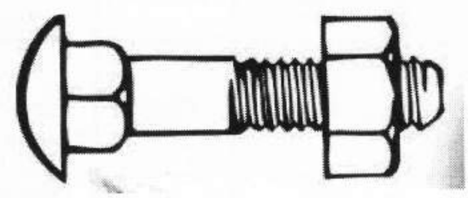
A) Bolts used in steel structures are of three types: **02 M Any ONE Fig.**

1) Black Bolts,
2) Turned and Fitted Bolts and
3) High Strength Friction Grip (HSFG) Bolts.

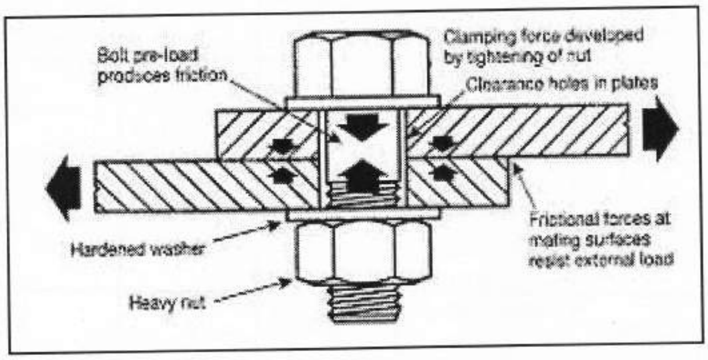
1) **Black Bolts**



2) **Turned and Fitted Bolts**



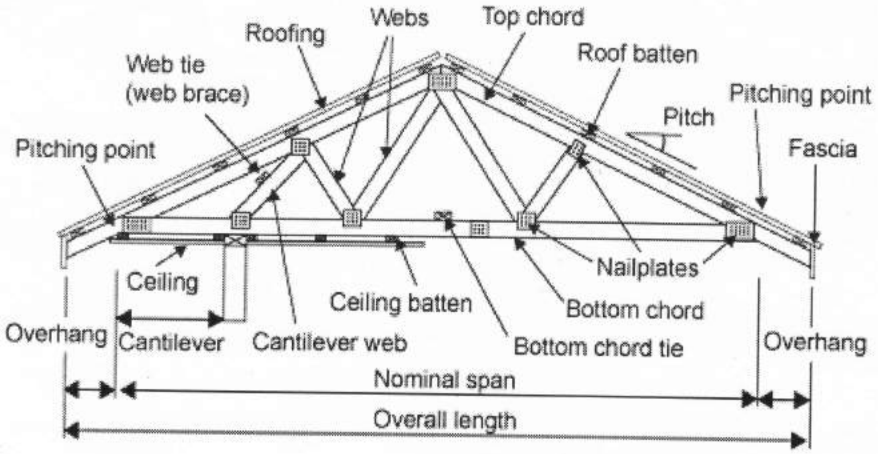
3) **High Strength Friction Grip (HSFG) Bolts.**



B) Drilled holes are preferred over punched holes because punching of holes reduces ductility and toughness and it may lead to brittle failure. 02M

Q No.3 (c) **Define component parts of a roof truss with a labeled sketch.** 04M

a) **sketch**



02M Fig

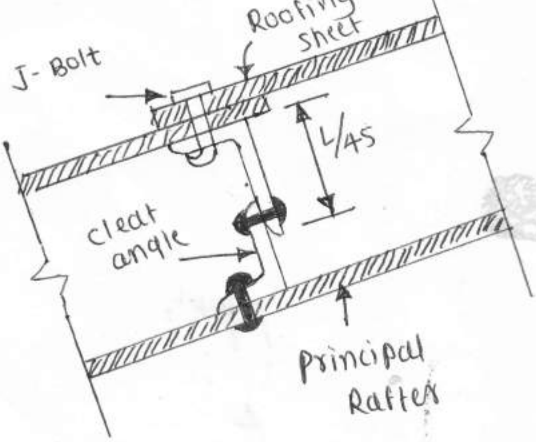
b) **Component parts**

- **Apex -** the highest point of the truss
- **Top cord -** the piece of timber which runs to the top of the truss
- **Web -** is a short timber which runs from the bottom chord to the top chord
- **Panel point -** is where the web meets the top chord. It is the strongest point for lifting the truss
- **Heel -** is where the bottom chord meets the top chord
- **Bottom chord -** is the large horizontal member (timber or steel) at the bottom of the truss
- **Truss span -** is the length of the bottom beam that spans the wall frames
- **Pitch -** is the angle the top chord makes with the bottom chord
- **Eave overhang -** is the horizontal distance the top chord extends from the wall.

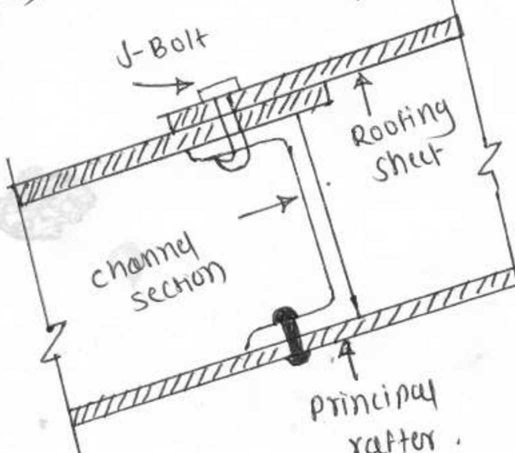
½M each write Any four

Q No.3 (d) **Draw neat sketches of connection of an angle purlin with principal rafter at panel point and the correct orientation of placement of channel section pulin over principal rafter** 04M

a) Angle purlin



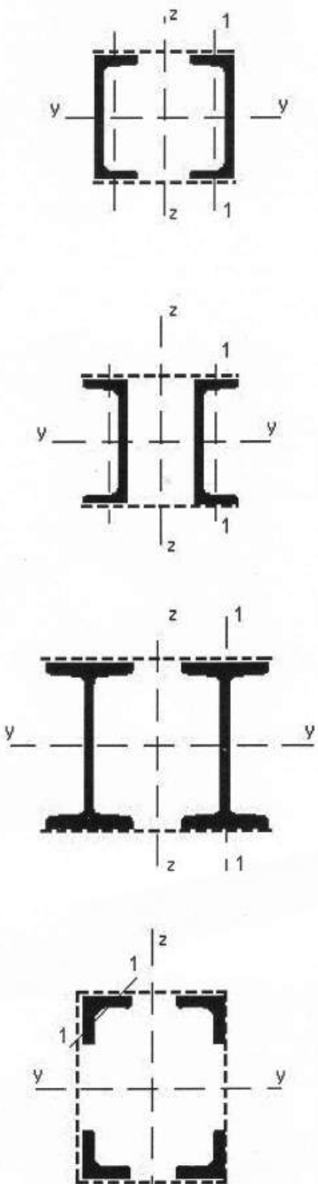
b) channel section purlin



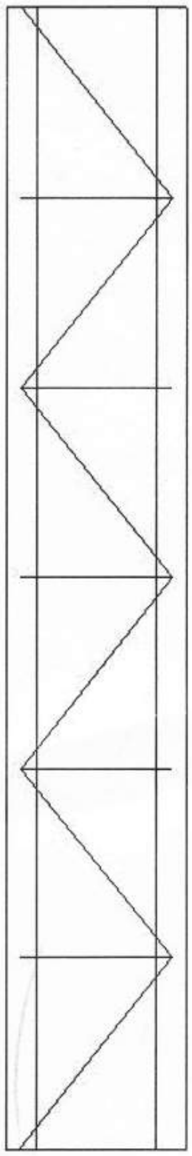
02M each Dia

Q No.4 A		
a)	**Sketch different sections used as built up strut and built-up column.**	04M

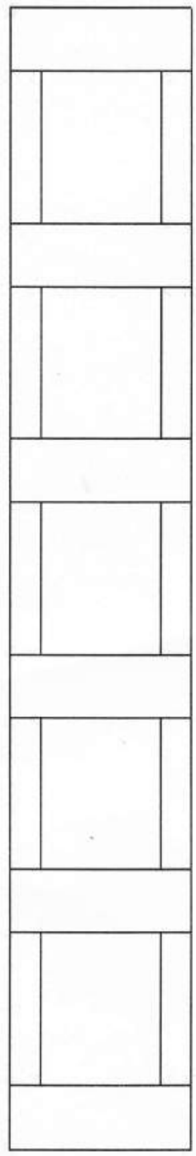
a) **Built-up column.**



Plan



section-1

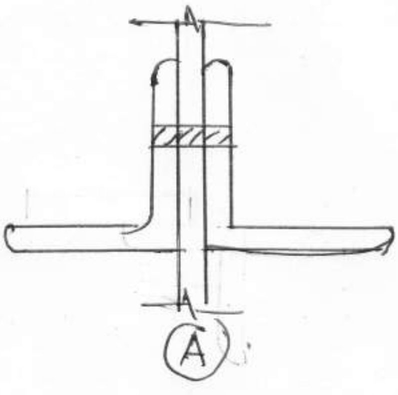
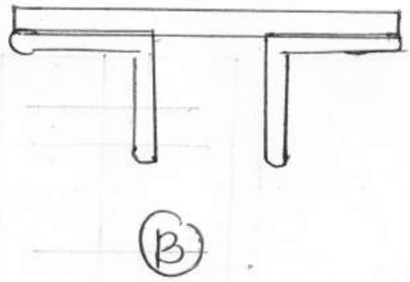


section-2

Built-up columns

1M for Any ONE Plan

1M for Section Any ONE

	b) built up strut	
		01M
		01M

b)	**State with sketch the effective length for compression member as per IS 800 -2007 having end condition as** **i) Translation restrained at both ends and rotation free at both ends** **ii) Translation and rotation restrained at both ends**	04M

i. Translation restrained at both ends and rotation free at both ends

Restrained	Free	Restrained	Free		1.0L

02M

ii. Translation and rotation restrained at both ends

Restrained	Restrained	Restrained	Restrained		0.65 L

02M

c)	**State the function of lacing and battening.**	

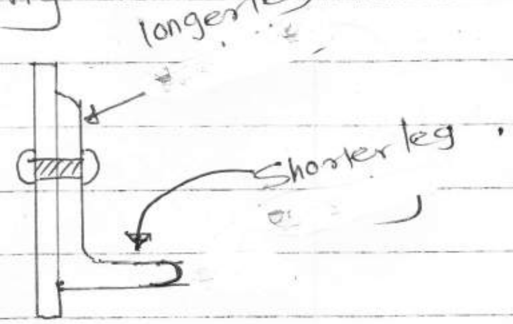
1. **function of lacing**
   - To connect the different components of built up column together so that they will act as one unit
   - To keep the distance between two components of built up column uniform and constant.
   - To keep the distance between two components of built up column uniform and constant.

02M

2. **function of battening**
   - The batten is placed opposite to each other at each end of the member and at points where the member is proportioned uniform through out.
   - When battens are used effective length of column should be increased by 10 %
   - Battens shall be designed to carry the bending moments and shear forces arising from transverse shear force equal to 2.5 percent of the total axial force on the whole compression member, at any point in the length of the member, divided equally between parallel planes of battens.

02M

Q.NO	SOLUTION	MARKS
Q-4 A (d)	$\frac{h}{b_f} < 8.4\varepsilon$ --- for class-1 (plastic)	01M
	∴ given section is ISMB 500	
	∴ $h = 500$	
	& $b_f = 180$	
	∴ $\frac{h}{b_f} = \frac{500}{180} = 2.78 < 8.4\varepsilon$	1M
	but $\varepsilon = \sqrt{\frac{250}{f_y}}$	
	$\therefore \varepsilon = \sqrt{\frac{250}{250}}$	
	$\varepsilon = 1$	1M
	$\therefore \frac{h}{b_f} = 2.78 < 8.4 \times 1$	
	$= 2.78 < 8.4$ --- hence the class is plastic	1M

Q .NO	SOLUTION	MARKS
Q-4 B (a)		06M
	i) $\sigma_{at} = 0.60 \times f_y$ for fe250	
	$= 0.60 \times 250$	
	$\sigma_{at} = 150\ N/mm^2$	02M
	ii) generally longer legs are connected in case of unequal angle section because of the following reason.	
	consider angle is connected in the following manner as shown in fig	
		
	if the shorter leg is connected to gusset plate, then the bending stresses induced in the section is large due to outstanding longer leg, because of which the stress-distribution in the section is no-uniform and hence it may lead to fracture of the member prematurely	04M

Q.NO	SOLUTION	MARKS

Q-4 (B)

b) Approximate gross area required A_g

$$\text{Req}^d\ A_g = \frac{1.1 \times T_{dg}}{f_y}$$

$$= \frac{1.1 \times 340 \times 10^3}{250}$$

$$= 1496\ mm^2$$

Try ISA $125 \times 75 \times 8$ mm giving $A_g = 1538\ mm^2$ $r_{min} = 16.1$ mm. Assuming longer leg connected, check the strength of the section.

i) Design strength due to yielding of gross section

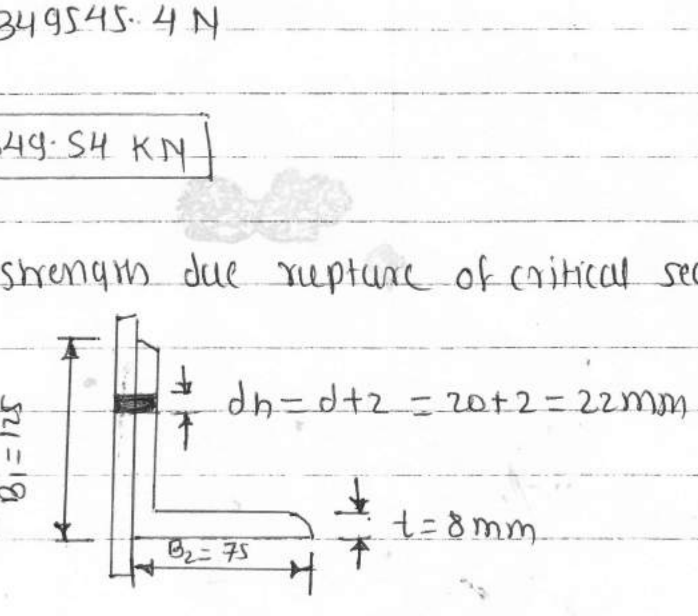
$$T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}}$$

$$= \frac{1538 \times 250}{1.10}$$

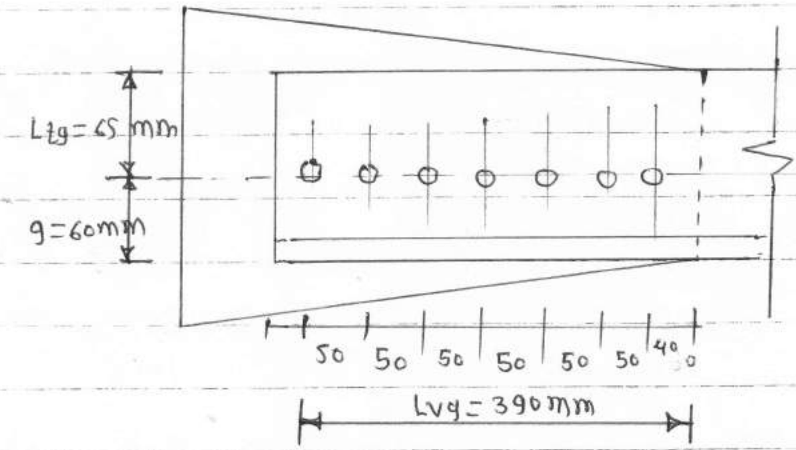
$$= 349545.4\ N$$

$$\boxed{T_{dg} = 349.54\ KN}$$ ½ M

ii) Design strength due rupture of critical section

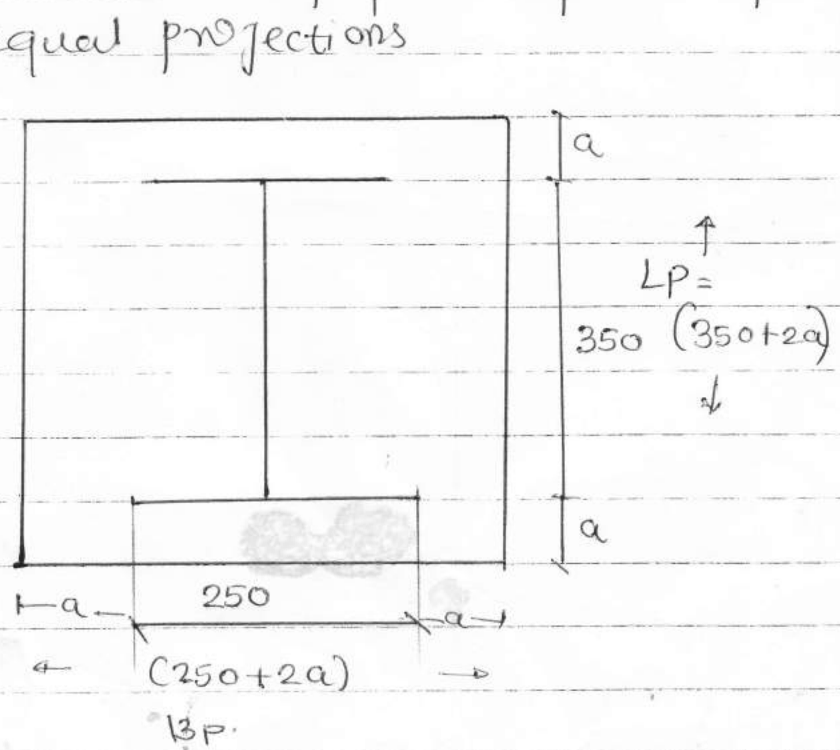


Q. NO.	SOLUTION	MARKS
	$T_{dn} = \alpha A_n \dfrac{f_u}{\gamma_{m1}}$	
	$A_n = A_{nc} + A_{go}$	
	$A_{nc} = (B_1 - d_h - t/2) \times t = (125 - 22 - 8/2) \times 8$	
	$A_{nc} = 792\ mm^2$	
	$A_{go} = (B_2 - t/2)\,t = (75 - 8/2)\,8 = 568\ mm^2$	
	$A_n = A_{nc} + A_{go}$ $A_n = 792 + 568$ $A_n = 1360\ mm^2$	1/2 M
	considering more than four bolt's in a row $\alpha = 0.8$	
	$T_{dn} = \dfrac{0.8 \times 1360 \times 410}{1.25}$	
	$\boxed{T_{dn} = 356.864\ KN}$	1/2 M
	Design of bolt's capacity of bolts in single shear $= 45.3\ KN$ capacity of bolt in bearing $= 20 \times 8 \times 410 \times 10^{-3}$ $= 65.6\ KN$ least bolt value $= 45.3\ KN$ (min. of two above)	

Q.NO	SOLUTION	MARKS
	Number of bolts required $= \frac{340}{45.3} = 7.5$ say 8	
		½M
	Assuming edge dist^n = 40mm	
	g = 60mm	
	spacing of bolts = 50mm	
	$A_{vg} = L_{vg} \times t = 390 \times 8 = 3120 mm^2$	
	$\boxed{A_{vg} = 3120 mm^2}$	½M
	$A_{vn} = \{ L_{vg} - [\text{No. of bolts} - 0.5\, d_h] \} \times t$	
	$A_{vn} = \{ 390 - [(8-0.5)22 \} \times 8 = 1800 mm^2$	
	$\boxed{A_{vn} = 1800 mm^2}$	½M
	$A_{tg} = L_{tg} \times t = 65 \times 8 = 520 mm^2$	
	$\boxed{A_{tg} = 520 mm^2}$	½M

Q.NO	SOLUTION	MARKS
	$A_{tn} = (L_{tg} - 0.5d_h) \times t$	
	$A_{tn} = [65 - (0.5 \times 22)] \times 8$	
	$\boxed{A_{tn} = 432 mm^2}$	½M
	$T_{db1} = A_{vg} f_y / (\sqrt{3} \times \gamma_{m0}) + 0.9 A_{tn} f_u / \gamma_{m1}$	½M
	$= 3120 \times 250 / (\sqrt{3} \times 1.10) + 0.9 \times 432 \times \frac{410}{1.25}$	
	$= 409393.8 + 127526.4$	
	$= 536920.2$ N	
	$\boxed{T_{db1} = 536.92 KN}$	01½M
	$T_{db2} = (0.9 A_{vn} f_u / \sqrt{3} \times \gamma_{m1}) + A_{tg} f_y / \gamma_{m0}$	
	$= 0.9 \times 1800 \times 410 / (\sqrt{3} \times 1.25) + 520 \times \frac{250}{1.10}$	
	$= 306780.8 + 118181.8$	
	$= 424962.8$ N	
	$T_{db2} = 424.962$ KN	01½M
	T_{db} = lesser than T_{db1} & T_{db2} = 424.96 KN	
	∴ The tensile strength of angle = lesser of T_{dg}, T_{dn} & T_{db}	
	(349.54, 356.86, & 426.96)	
	= 349.54 KN	
	This is greater than reqd 340 KN	
	Check for slenderness ratio. $\lambda = \frac{l}{r_{min}} = \frac{2400}{16.1}$	½M
	149.06 < 250	

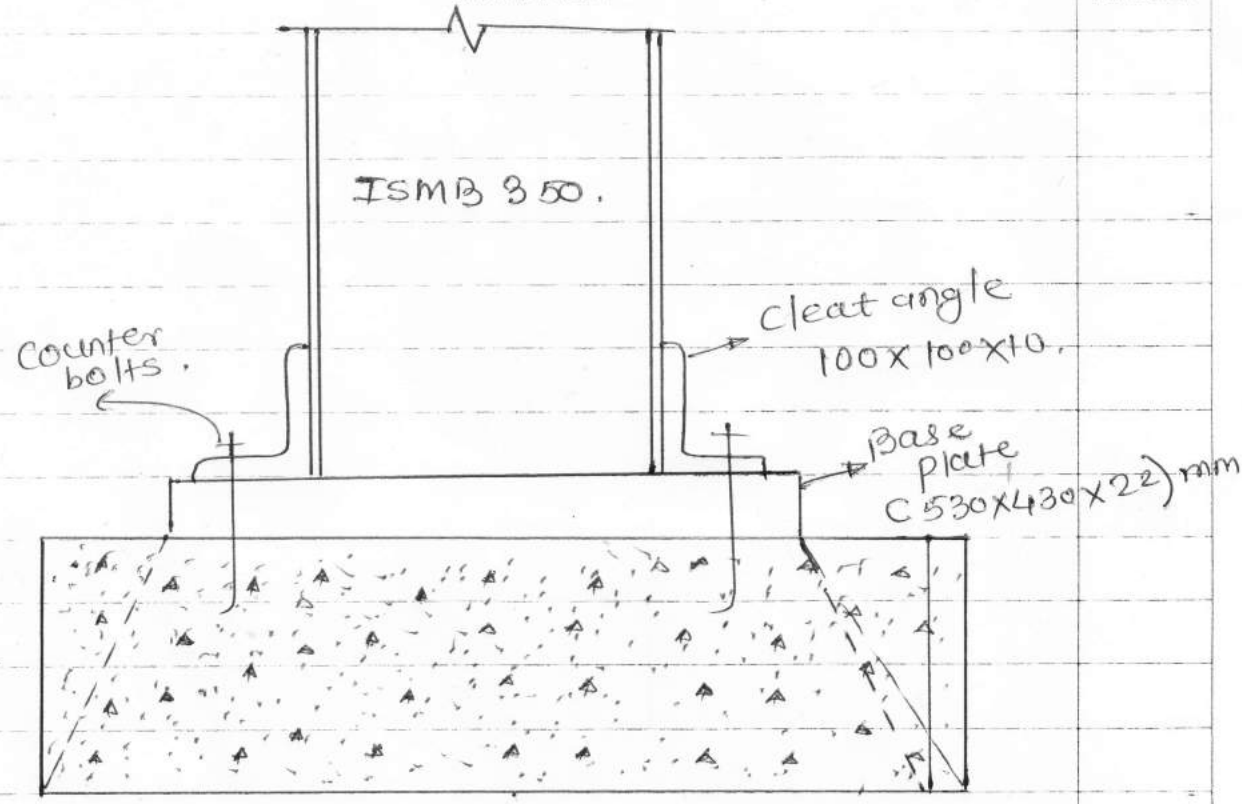
Q. NO	SOLUTION	MARKS
5 (A).	Attempt any 2. To design a slab base.	
Soln:-		
	Given data :-	
Step 1]	ISHB - 350	
	$P_u = 1500 kN$.	
	$f_{ck} = 20 N/mm^2$	
	Bearing strength of concrete $= 0.45 \times 20 = 9 N/mm^2$	(1m).
Step 2]	To calculate area required for base plate	
	$A_{required} = \frac{P_u}{0.45 f_{ck}} = \frac{1500 \times 10^3}{0.45 \times 20} = 166666.67 mm^2$	
Note:-	[Students can also write the above formula as $A_{req} = \frac{P_u}{0.60 f_{ck}}$]	
Step 3]	Select the size of plates L_p and B_p for equal projections	



Q.NO	SOLUTION	MARKS
	∴ Area of baseplate = $(350+2a) \times (250+2a)$	
	$166666.67 = (350+2a) \times (140+2a)$	
	$16666.67 = 49\times10^3 + 280a + 700a + 4a^2$	
	$16666.67 = 49\times10^3 + 980a + 4a^2$	
	$16666.67 = 4a^2 + 980a + 49\times10^3$	
	$166666.67 = 4a^2 + 980a + (49\times10^3)$	
	Re-arranging the above equation we get	
	$4a^2 + 980a + (49\times10^3 - 166666.67) = 0$	
	$4a^2 + 980a - 117.66\times10^3 = 0$	
	Solving the above quadratic eqn we get the value of a as follows.	
	$a = 88.26$ say $a \approx 90$ mm	
	∴ $L_p = (350) + (2\times90) = 530$ mm	(2m)
	$B_p = (250) + (2\times90) = 430$ mm	
	Step (4) Area of base plate provided	
	Area $= 530\times430$	
	Area $= 227.9\times10^3\ mm^2 > 166666.67\ mm^2$	(1m)
	Hence safe.	
	Step (5) Calculate ultimate pressure from below on base	
	$W = \dfrac{P_u}{L_p \times B_p}$	(1m)
	$W = \dfrac{1500\times10^3}{(430\times530)} = 6.5\ N/mm^2$	

Q.NO	SOLUTION	MARKS
	Step 6:- Calculate thickness of slab-base	
	$t_p = \sqrt{\dfrac{2.5w(a^2 - 0.3b^2)\gamma_{m0}}{f_y}}$ Assume $f_y = 250\ N/mm^2$	
	$t_p = \sqrt{\dfrac{2.5\times6.5(90^2 - 0.3\times90^2)\times1.1}{250}}$	(1m).
	$t_p = \sqrt{\dfrac{101.35\times10^3}{250}}$	
	$t_p = 20.13\,mm$	
	$t_p \simeq 22mm$.	
	~~Step 7~~:- Provide thickness of base plate as 22 mm.	(1m).
	Step 7:- Provide ; 2ISA 100×100×10 and 4, 20 mm ϕ bolts one at each corner for connection of base plate to pedestal.	(1m)
	[Note:- Students can assume any other value of ISA and Cleat angle, it is not mandatory to assume the above given value]	

Q.NO	SOLUTION	MARKS

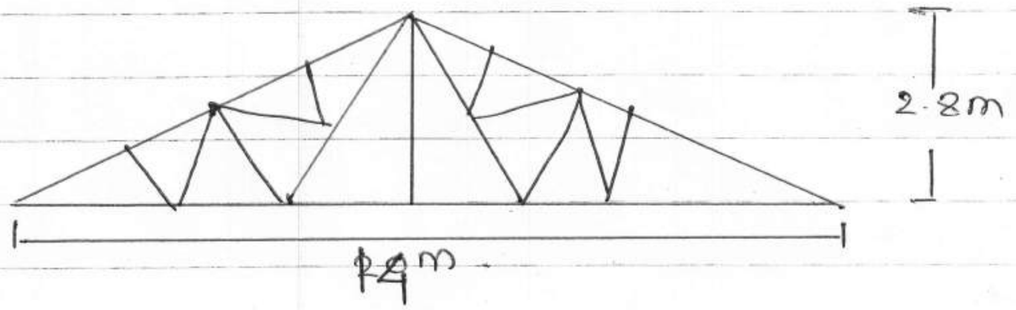


Slab base.

[Note①:-The above diagram is mandatory for the students to draw in the end; the examiner can reduce 1 mark from the overall answer if the above diagram is not drawn by the Students]

[Note 2:- In the design problem, marks should be given on the basis of design steps and evaluation of the question should not be done on the basis of final answer]

[The above note is applicable for all the design problems]

Q.NO	SOLUTION	MARKS
Q.5] b.	Given data	
	$L = 14m$.	
	Spacing = 4m c/c.	
	$\frac{Rise}{span} = \frac{1}{5}$	
	$\frac{Rise}{14} = \frac{1}{5}$	
	Rise = 2.8m.	(1m)
	∴ No of panels as 5 no's. length of each panel is 2.8m.	
		(1m)
Step ② Calculate plan area		
	Plan area = span × spacing $= 14 \times 4$ $= 56 m^2$	
Step ③ :- Calculation of dead load		
ⓐ	Self wt $= \left(\frac{L}{3}+5\right)\times 10 = \left(\frac{14}{3}+5\right)\times 10 = 96 N/m^2$	
ⓑ	wt of roof covering $= 165 N/m^2$ (given).	(2m)
ⓒ	self wt of purlins $= 100 N/m^2$	
ⓓ	wt of bracings $= 15 N/m^2$ (assume)	
	∴ Total dead load $= 376 N/m^2$	

Q.NO	SOLUTION	MARKS
	Actual dead load $= 376 \times$ Plan area	
	$= 376 \times 56$	
	$= 21.05$ KN.	
	Total dead load on each panel $= \frac{21.05}{5} = 4.21$ KN.	(1m)
	Dead load on end panels $= \frac{4.21}{2} = 2.105$ KN	(1m).
	Step (4) :- Live load/m^2 of plan area.	
	Live load $= [750 - (\theta - 10) \times 20]$	(1m)
	live load $= [750 - (\theta - 10) \times 20]$	
	$\tan\theta = \left(\frac{2.8}{7}\right)$	
	$\therefore \theta = 21.80^\circ$.	
	$\therefore$ live load $= [750 - (21.80 - 10) \times 20]$	
	live load $= 513.97$ N/m^2.	
	live load for truss $= \frac{2}{3} \times 513.97$	
	$= 342.64$ N/m^2.	
	step (5) :- Live load per truss	(1m).
	live load per truss $= 342.64 \times 56$	
	$= 19.18$ KN.	
	live load per panel $= \frac{19.18}{5} = 3.836$ KN.	
	live load at end panel $= \frac{3.836}{2} = 2.873$ KN.	

Q.NO	SOLUTION	MARKS

Q. No 5.

(C). Span = 16 m

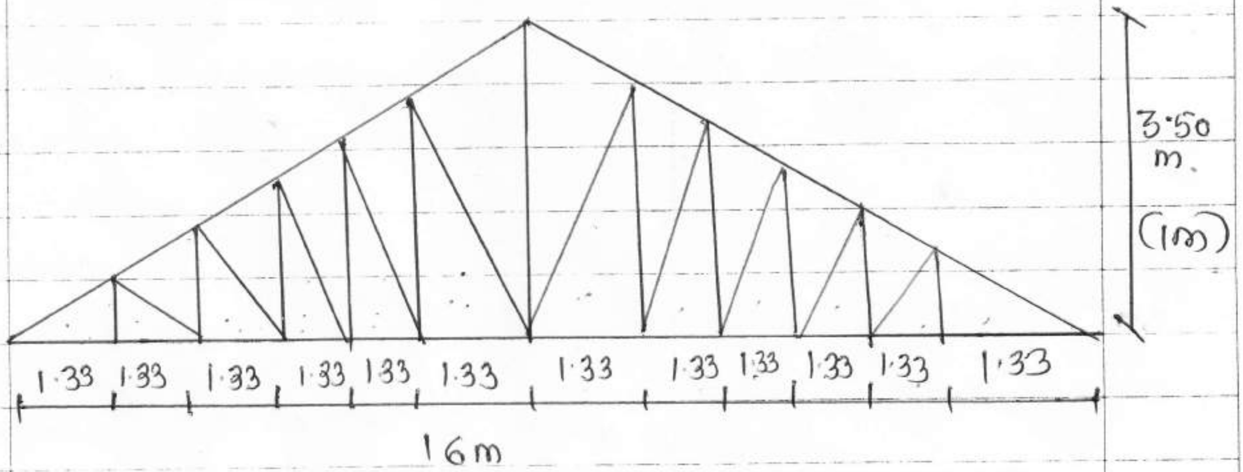
spacing = 4 m c/c.

Rise of truss = 3.50 m.

No. of panels = 12

$\theta = \tan^{-1}\left(\frac{3.5}{8}\right) = 23.62^\circ$.

Assume howe type of roof truss



(1m)

All dimensions are in meter.

Step (1) Calculate plan area.

plan area = span x spacing

= 16 x 4

plan area = 64 m²

(1m)

Step (2) Calculate Live load of truss /m²

$L.L = [750 - (\theta - 10) \times 20]$

$L.L = [750 - (23.62 - 10) \times 20]$

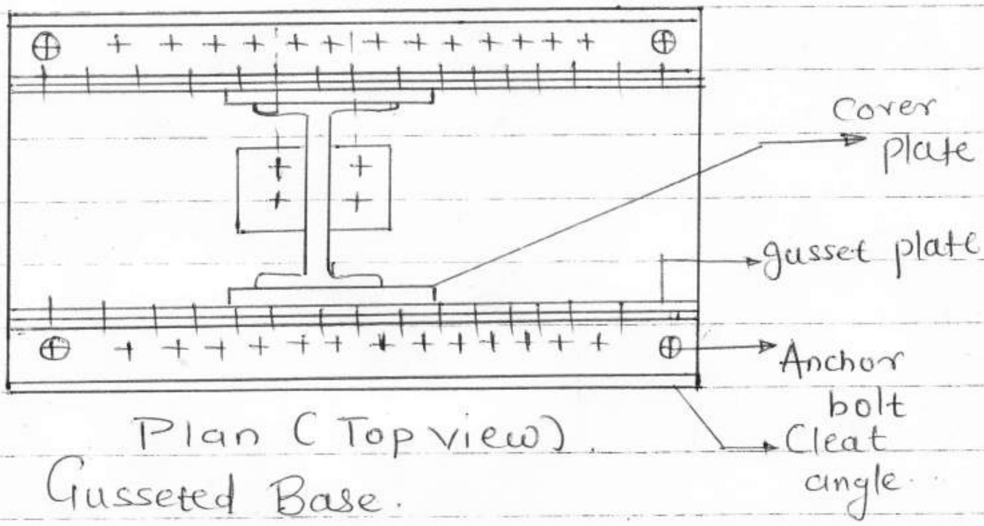
Liveload /m² = 477.6 N/m²

(2m)

live load for truss $= \frac{2}{3} \times 477.6 = 318.4$ N/m²

Q. NO	SOLUTION	MARKS
	Step No:-3	
	Live load on one truss = Plan area × L.L	
	= 64 × 318.4	
	= 20.3 KN.	
	live load on one panel = $\frac{20.3}{12}$ = 1.69 kN	(2m)
	live load on end panel = $\frac{1.69}{2}$ = 0.845 KN	
	Wind load calculations:-	
	Step No:-(4) Calculate sloping area	
	Sloping length or sloping side = $\sqrt{R^2 + \left(\frac{l}{2}\right)^2}$	
	∴ Sloping side = $\sqrt{3.50^2 + (8)^2}$	(1m)
	Sloping side = 8.73m.	
	Sloping area = 2×8.73×4	
	= 69.85 m^2.	

Q.NO	SOLUTION	MARKS
	(A) Wind load Calculations:-	
	$W.L = [-0.7-(+0.2)] \times 1200 = -1080\ N/m^2$	
	$W.L = (-0.7-(+0.2)] \times 1200 = -600\ N/m^2$.	
	Design wind load $= -1080\ N/m^2$.	(1m)
	windload on intermediate panel point	
	$= 1080 \times 0.73 \times 4 = 3.15\ kN$.	
	W. Load on end panel point $= \frac{3.15}{2} = \underline{1.57\ kN}$.	

Q.NO	SOLUTION	MARKS
Q.6	Attempt any four.	
Q a)	Draw plan of gusseted base showing all components.	
Ans.	 Plan (Top view). Gusseted Base.	1m for each component (4m).

Q.NO	SOLUTION	MARKS
Q.6 (b)	State four classification of C/S of beam based on moment - rotation behaviour as per IS-800 2007.	
Ans:-	Classification of beam are as follows.	
	Class 1 :- Plastic Section :-	(1m)
	This section is specified as plastic section as plastic hinge formation complets and plastic hinge has rotational capacity required for failure of structure	
	$Mp = Zp \times fy$	
	Class 2 :- Compact Section :- In this section though, extreme fibre has yielded fy is not reached in all fibres throught the depth. It can resist further moment	(1m)
	$Md = Ze \times fy$.	
	Class 3 :- Semi compact section.	(1m)
	In this section, extreme fibre in compression can reach yield stress. No plastic hinge formation occurs at this section.	
	$Md = Ze \times fy$.	

Q.NO	SOLUTION	MARKS
	Class 4 :- Slender Beam :-	(1m)
	The stress developed in extreme fibre of beam due to resisting moment is less than yield stress i.e slender beam is susceptible to buckle locally even before reaching f_y :-	
	[Note:- Full marks can be given to students, who have just mentioned or listed the above four classes; explaination to all these cases is not mandatory].	
Q.		

Q.NO 6	SOLUTION	MARKS
Q.c	Given data.	
	ISMB - 450	
	span - 4m	
	$V_u = 20 \frac{KN}{m}$	
	$\therefore V_u = 20 \times 4 = 80 KN$	
	$\therefore$ We have eqn.	
	$V_d = \frac{f_y (t_w \times h)}{\sqrt{3} \times \gamma_{mo}}$	1m.
	$V_d = \frac{250 (9.4 \times 450)}{\sqrt{3} \times 1.1}$	
	$V_d = \frac{1.057 \times 10^6}{1.90}$	2m
	$V_d = 555.04 KN > V_u = 80 KN$	
	Hence section is safe in shear.	1m.

Q. NO	SOLUTION	MARKS
Q.6		
(d)	Why beams are laterally restrained? Statemethods of providing lateral restrainment.	
Ans:-		
(a)	Thin projecting flange is susceptible to buckling under compression.	
(b)	In laterally supported beam, flange is restrained from buckling.	(1 m)
(c)	We can support compression flange in many ways as follows	
	(a) Connection of compression flange to the floor	(1m)
	(b) Connection of compression flange to the floor with the help of shear connectors	(1m)
	(c) Use of cross beam by braces for connection of compression flange	(1m).

Q.NO	SOLUTION	MARKS
Q.6 (e)	State the necessity of column bases. Also state the function of Cleat angle and anchor bolts in slab base.	
	Ans:- Column bases are necessary for the following cases:-	
(i)	To spread load from column on large area of concrete foundation.	(2m)
(ii)	To sustain bearing pressure below soil bending moment and shear force too	
	Cleat angles are used to provide stability to I-section and to secure col^m section	(1m)
	Anchor bots are used to provide support to cleat angles and to connect concrete pedestal and base plate anchor bolt is used.	(1m)