WINTER-19 EXAMINATION
Model Answer

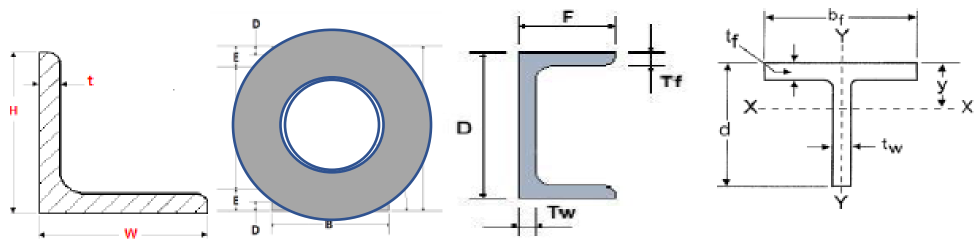
Subject Name: DESIGN OF STEEL STRUCTURES **Subject Code : 17505**

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
6) may vary and there may be some difference in the candidate's answers and model answer
7) In case of some questions credit may be given by judgement on part of examiner of relevant answer
8) For programming language papers, credit may be given to any other program based on equivalent concept.

QN	Sub Q	Answers	Marking scheme	Total Marks
Q1		**Attempt any THREE of the following**		**12M**
	a)	**Define dead, live, wind and snow load.** i. **Dead load: -** Dead load in steel structures is gravity loads and are relatively constant over the time. They are permanent known as permanent loads. They are the self-weight of the structural members or materials used for construction. These include weight of beam, slab, column etc. and elements such as weight of walls, partitions, floors and roofs. ii. **Live load: -** Live loads are also called as imposed loads or superimposed loads. Those are not permanent and may change in position and magnitude. The loads of furniture, equipment and occupants of the structure etc. are the examples of live load. Live loads on floors and roofs are given in IS:875-1987. iii. **Wind load: -** The wind load is more significant in case of tall structures. The wind pressure intensity at any height if structure depends upon basic wind speed, shape and height of structure, topography of surrounding ground surface and angle of wind attack. It is considered as per specifications given in IS:875-1987(Part 3) iv. **Snow load:-**In the areas of snow fall, an allowance for snow load is considered. It depends upon shape of the roof as well as the roofing material. It is variable load that may cover entire roof or part of it.	**1 M each**	

<table>
<tr><td></td><td>b)</td><td>Enlist any four limit states of collapse.
The limit sate of collapse includes the following: -
i) Strength
ii) Plastic collapse
iii) Fatigue (fluctuation of stresses)
iv) Stability against sway, overturning and sliding</td><td>1 M each</td><td></td></tr>
<tr><td></td><td>c)</td><td>Draw labeled sketches of Tee, Channel, angle, and hallow circular sections with meaning of each notation.



ANGLE SECTION   Hollow Circular section   CHANNEL SECTION   T- SECTION

<u>Angle section</u>
w & h – represent the size
t – thickness

<u>Hollow Circular</u>
D – External Diameter
d – Internal Diameter

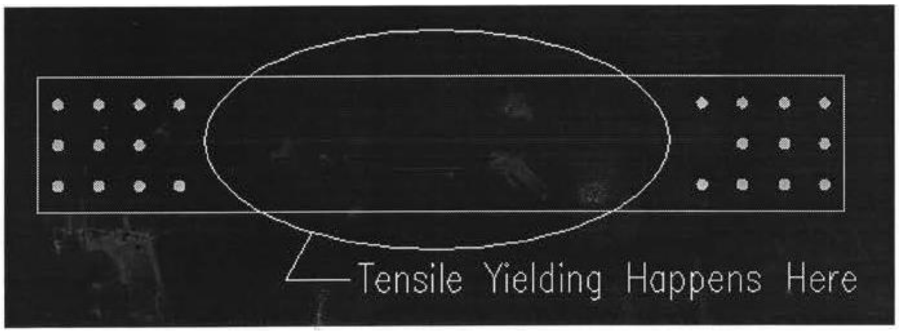
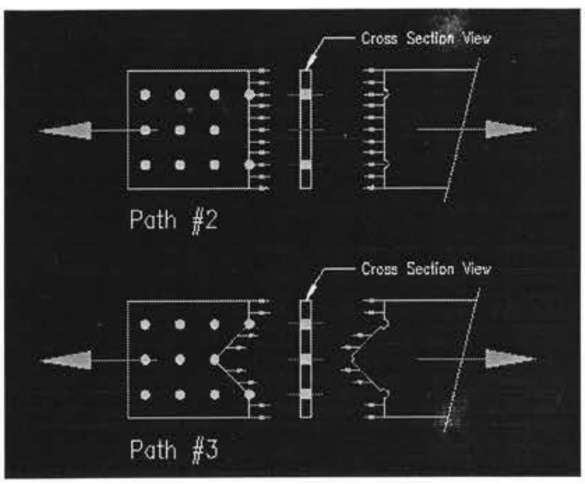
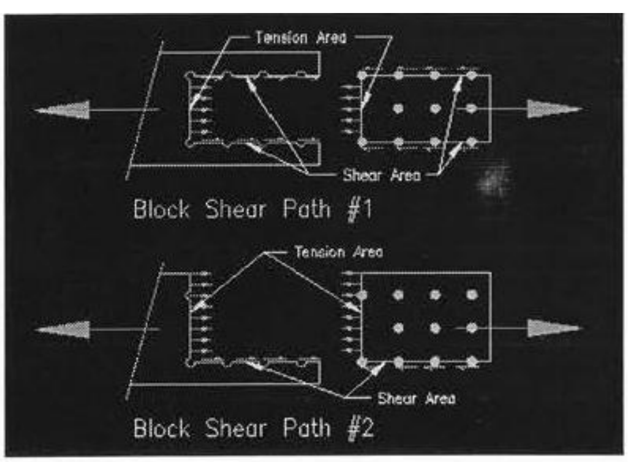
<u>Channel Section</u>
F – with of flange
D – Depth of section
T_W – thickness of web
T_f – thickness of flange

<u>T Section</u>
bf – width of flange
d – depth of section
tf – thickness of flange
tw – thickness of web</td><td>1 M each</td><td></td></tr>
<tr><td></td><td>d)</td><td>Define gross section yielding with sketch and write formula for it for tension member as recommended by IS 800 -2007
Gross Section Yielding: - When a tension member 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 exists 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.
$$T_{dg} = fy*A_g/\gamma_{mo}$$
Tdg – Gross section yielding strength
fy – yield stress of steel
Ag – gross area of the section
γ_{mo} - partial safety factor (whose value is 1.1)</td><td>2

1

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

Q1	B)	**Attempt any ONE of the following**		**6M**
	a)	**100mm x 10mm and 100mm x 8mm plats are connected by using 16mm diameters bolt of grade 4.6 and grade & steel plate in 410N/mm^2 provide single row of bolts, find strength of joint. Find no of bolts required to form joint Assume kb=0.49** Dia of bolt = 16mm Dia of bolt hold = 16 + 2 = 18mm Grade of bolt = 4.6 fub = 410N/mm^2 Grade of plate = 410N/ mm^2		
		Strength of bolt in shear Vdsb = Vnsb / γ_{mb} Vnsb = $\frac{\text{Vnsb}}{\sqrt{3}}$ (n_n*Anb + n_s*Asb) Asb =Ag = (Π x 16^2/4) = 201.06 Anb = 0.78 x Asb = 0.78 x 201.06 = 156.83 Vnsb = $\frac{400}{\sqrt{3}}$ (1*156.83) = 36.22 x 10^3 N **Vdsb = 36.22 / 1.25 = 28.97 kN**	1	
		Strength of bolt bearing Vdpb = Vnpb / γ_{mb} Vnpb = 2.5 x kb x d . tp . fu = 2.5x0.49x16x8x41O=64288 N **Vdpb = 64.288/ 1.25 = 51.43 KN**	1	
		Bolt value is least of Vdsb and Vdpb which is **28.97kN**	1	
		Full strength = fy x Ag/γ_{mo} = 250 x (100 x 8)/1.1 = 181.82 x 10^3 N	1	
		Full strength = 181.82 kN	1	
		No of bolts = Full Strength of plate / Bolt value **= 181.82/28.97 = 6.27 ~ 7 No's**	1	

	b)	**Draw the sketches of three different modes of failure in tension members of steel structures** **The different modes of failures in tension members are** i) Gross section yield ii) Net section rupture iii) Block shear **i) Gross section yield**  **ii) Net section rupture**  **iii) Block shear**  	**2M each**	**6M**

Q2		**Attempt Any THREE of the following**		**16 M**

a) **ISA 80 x 50 x 8 nun is to be connected to 10mm thick gusset plate using 6mm fillet weld. The angle is subjected to 150 kN tensile load, assume shop welding. Provide welds at extremities of the longer legs. For ISA 80 x 50 x 8 take Cxx = 27.3mm**

Given size of angle = 80 x 50 x 8mm
Size of weld = 6mm
Tensile load = 150 kN
Welding type = shop welding = γw = 1.25

i) Factored Tensile load = 150 x 1.5 = 225 kN — **1**

ii) Design stress of shop weld (fwd) — **1**

fwd = fu/($\sqrt{3}$ *γ_{mb})= 410/($\sqrt{3}$ x 1.25)= 189.4N/mm^2

iii) Throat Thickness = 0.7 x 6 = 4.2mm — **1**

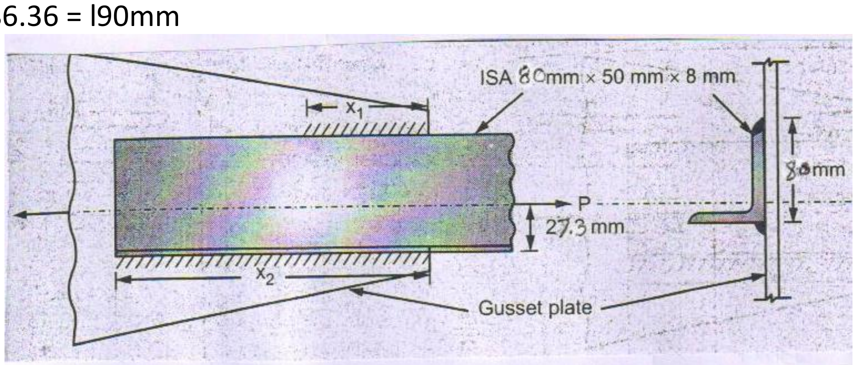
iv) Design strength per mm length of weld

Pq = fwd x t_t x 1 = 189.4 x 4.2 x 1 = 795.48N/mm — **1**

v) Effective length of weld required — **1**

L= P/Pq =225 x 10^3 /795.48= 282.89 mm ~290 mm

Let x1 let x2 be the length longitudinal at upper and edge of angle
x1 + x2 = 282.89 — **1**
Taking moment about the bottom weld
795.48 x 1 x 80 = 225 X 10^3 x 27.3
X1 = 96.52 = 100 mm — **1**
x2 = 186.36 = l90mm



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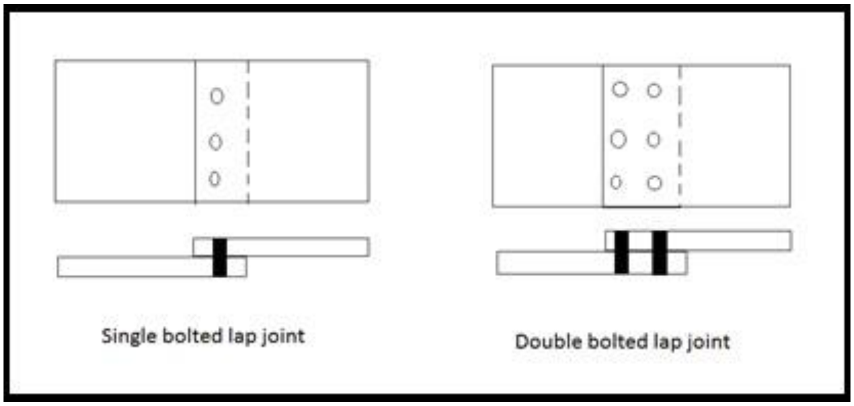
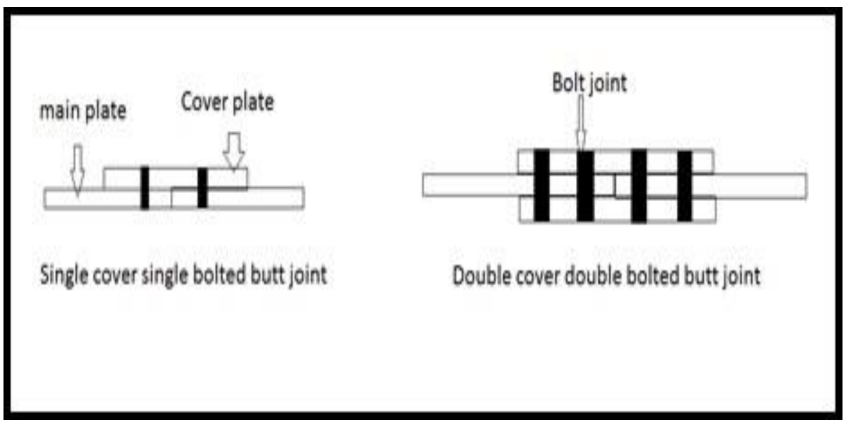
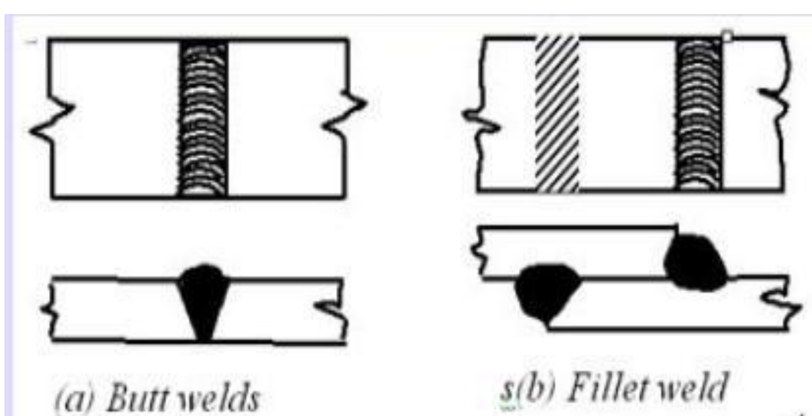
b) **A double angle discontinuous strut carry of factored load of 140KN. The length of strut in 4m between intersection. The two angles are placed back to back on opposite side of 12mm thick gusset plate take both are provide, design the section. Assume Fy = *250N/mm²* let kL = 0.85L**

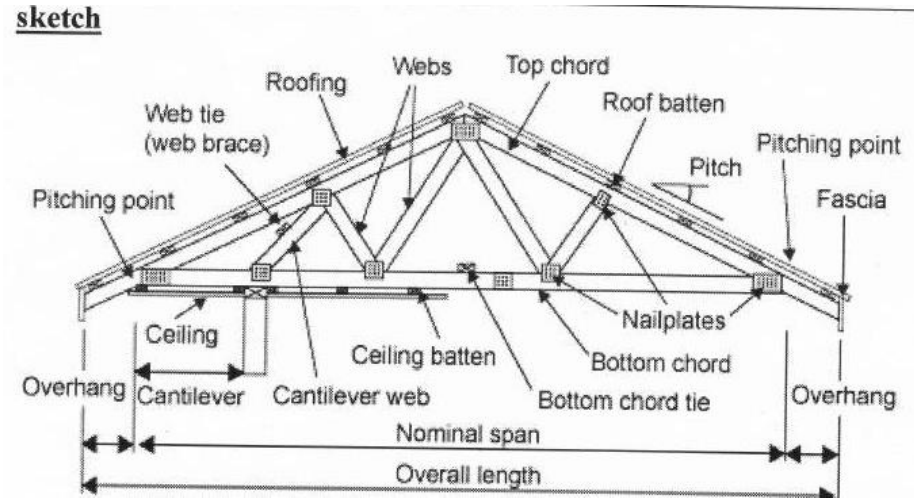
1) Effective length = 0.85 x 4 = 3.4m

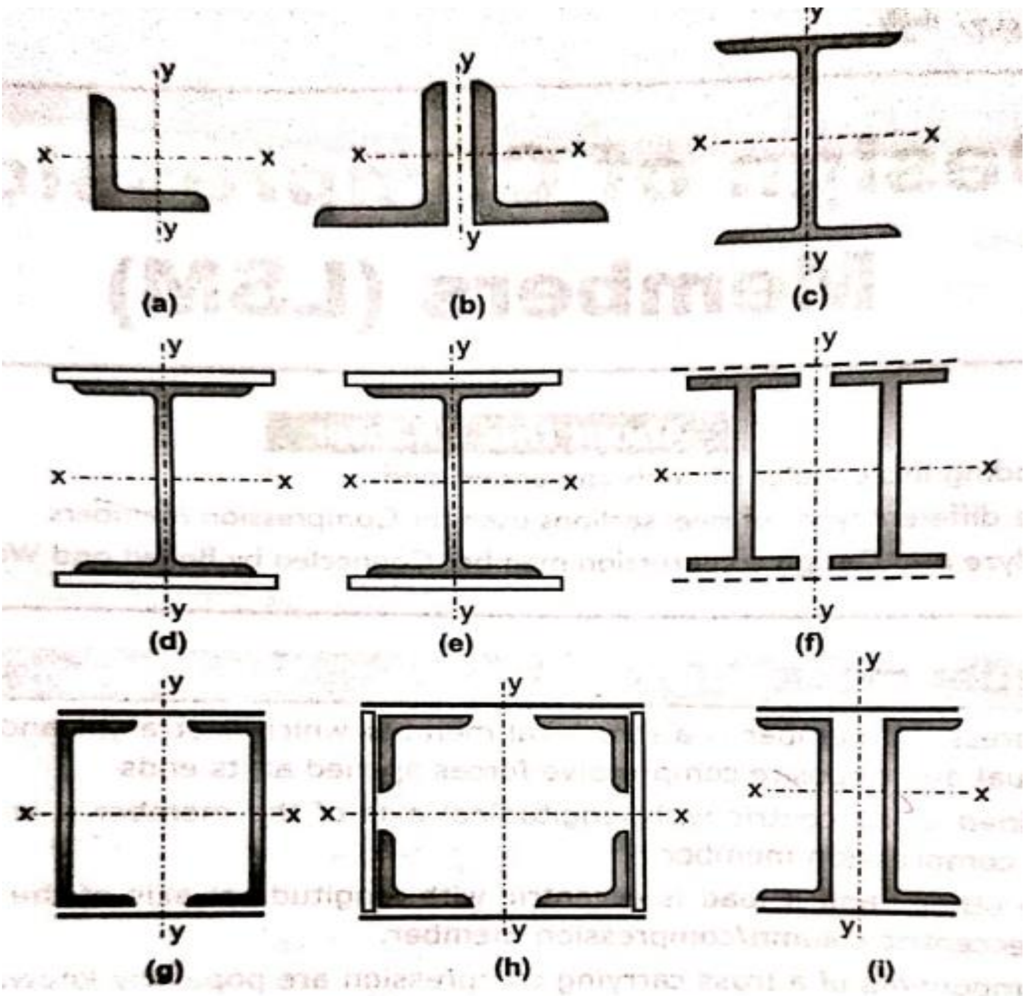
2) Required area is got by assuming initial $\mathbf{f_{cd}}$ Value as **90 N/mm^2** — **1**

		Area required Ag = load / fcd $=140 \times 10^3/90 = 1555.56mm^2$ As its 2ISA to be provide, hence area of one angle = *1555.56/2* $=777.78mm^2$ hence choosing ISA 70 x 70 x 6 from table :- As r_{min} is given as 21.4mm **Slenderness Ratio λ** = kL/ r_{min} = (0.85 x 4000)/21.4 = 158.88 Therefore fcd from interpolation fcd = 53.3 + [(59.2 – 53.3)/(160 – 15.88) / (160- 150)] **fcd = 53.96N/mm²** **Load carrying capacity** = fcd x total area =53.96 x (806 x 2) =86.984kN < 140 **hence section is not sufficient** **Choose the next given section ISA 90 x 90 x 6** **Slenderness Ratio λ** = 0.85 x 4000/27.7 = 122.74 fcd = 74.3 + [(83.7-74.3)/(130 – 120) x (130 – 122.74) Load carrying capacity = fcd x total area = 81.12 x (1047 x 2) = 169874N = 169.87kN > 140 hence okay **NOTE: Student can choose any section , marks to be given accordingly**	1 1 1 1 1 1 1	
	c)	**A beam 5m effective span, carrying a factored UDL of 45 kN on entire span (excluding self weight) concentrated factored load of 15 kN at mid span. Design a laterally support beam.** Given load UDL 45kN Point load = 15kN **1) Factored bending movement** = $wl^2/8 + wl/4$ 45 x 52/8 + 15 x 5/4 = 159.37 kNm **2) Plastic modulus required(Zp)** = (Md x ɣmo)/fy = $(159.37 \times 10^6 \times 1.1)/250$ $= 701228\ mm^3$ **Elastic section modulus (Ze)**= Zp / shape factor Shape factor for I section is assumed to be 1.14 (Ze)(reqd) = $701228/1.14 = 615112\ mm^3$ **3) Trying the section ISMB 350**. Properties of which are h = 350, t_f = 14.2, tw = 8.1, r1 = 14 Zxx 778.9 x 103, Ixx = 1360.3×10^4, Zp = 889.57×10^3	1 1 1 1	

		4) Classification of beam d = h – 2 (tt + r1) = = 350-2(14.2 + 14) = 293.6mm bh/tt = 140/2/14.2 = 4.92 < 9.4 d/tw = 293.6/8.1 = 36.24 <64 **hence the section is plastic** **5) Calculation of actual load including self-weight** Actual wd1 = 45 + (514 x 1.5/1000) = 45.77kN/m Wd2 = 15kN Actual factored BM = 45.771 x 52 x 8 + 15 x 5/4 = 161.78kNm Actual factored SF = 45.77] x 52/2 + 15/2 = 121.92kN **6) Also check** Vd/Vdr = 121.92/371.99 = 0.327 <0.6 **hence section safe in shear** **7) Check for flexure** Mr = β_b .fy/ γ_{mo} = 1 x 889.57 X 103 x 250/1.1 = 202.175> (Md = 161.78) **Hence section safe flexure** **8) Check for defection** $\delta_{allowable} = \frac{L}{300} = \frac{5000}{300} = 16.67$ $\delta_{max} = \frac{5wL^4}{385EI} + \frac{wL^3}{48EI}$ $\frac{5 * 30.514 * 5000^4}{385 * 2 * 10^5 * 1360.3 * 10^4} + \frac{10 * 5000^3}{48 * 2 * 10^5 * 1360.3 * 10^4}$ = 91.27+ 0.00095 = 91.27mm > 16.67 hence **NOT** safe in defection **Note: Student may choose any other section marks to be allotted accordingly.** **Note: Value of Ixx for the section ISMB -350 is not correct in the question the given value in question is 1360.3 x 10^4** **But actual value is 13630.3 x 10^4 mm4, hence marks to be given accordingly**	**1** **1** **1** **1**	

Q3		**Attempt any FOUR of the following**		**16M**
	a)	**Define lap and butt bolted joint along with their sketches** **Lap Joint :** When a member is placed one above the other and both are connected by means of bolts/weld the joint is known as lap joint.  **Butt Joint:** When two members are placed butt against each other and are connected by means of a cover plate on one side or on either side, such a joint is known as butt joint  	**2 M each**	
	b)	**Draw detailed sketches of single V butt weld and fillet weld.**  	**2 M each**	

	c)	**Draw sketch of truss and show eight important components of truss** **sketch**  **Students should be given credit for any 8 components**	**½ mark for each component**	
	d)	**Define pitch, principal rafter main tie and spacing of truss** **Pitch:** It is defined as the ratio between the rise and span of the truss. **Principal Rafter :** It is the top chord member of the truss and is usually in compression . it supports the purlins and these purlins in-turn support the roofing sheet **Main Tie:** It is the bottom chord of the roof truss and is usually in tension. **Spacing of roof truss:** It is the center to center distance between the trusses.	**1 M each**	
	e)	**A pratt roof truss has spacing of 4.1 m panel length 2.1m pitch1/4 and span 16.4m. no access provided to roof, calculate panel point live load.** Rise = span / 4 = 16.4 / 4 = 4.1 m. $\theta = \tan^{-1}$(Rise/0.5 x span) = (4.1 / 0.5 x 16.4) = 26.56^0 **Live Load:** Live load = 750 – [(θ – 10) x 20] = 418.69 N/m^2. Live load intensity for truss = (2/3) X 418.69 = 279.13 N/m^2 Live load per panel point = 2.1 x 4.1 x 279.13 = 2403.3 N. Live load on end panel point = 2403.3/2 = 1201.65 N.	**1** **1** **1** **1**	

Q4		Attempt any THREE of the following		12M
	a)	**Attempt any THREE of the following:** **Draw labelled sketches of any four types of built up compression member.**  **Fig. 4.1: Various Forms of Compression Members**	1 mark each	

	b)	**Define slenderness ratio & radius of gyration and write formula also** **Slenderness ratio:** It is the ratio of effective length to least radius of gyration. SR = kL/r_{min} Where kl is the effective length of the member. r_{min} the least radius of gyration of the member. **Radius of gyration**: It is the property of a section and is equal to $r = \sqrt{\frac{I}{A}}$ Where I = Moment of inertia of the section A is the area of the section	**2M** **2M**	
	c)	**A single angle discontinuous strut, 100 x 100 x 10 mm has effective length of 3000 mm, end connections are made by 4 bolts of 20 mm diameter and 4.6 grade. Assume Fy = 250 *N/mrri*², k) = 0.2, ~ = 0.35, k3 = 20 and imperfection factor a. = 0.49 for ISA 100 x 100 x 10 mm, A = 1903 mrn-, 'Ymin = 19.4 mm. Find design compressive load.** Solution: ISA 100mm x 100mm x 10mm le = 3000mm 4 bolts of 20mm Fy = 250N/mm^2 K1 = 0.2 K2 = 0.35 K3 = 20 α = 0.49 A = 1903mm^2 ϒmin = 19.4mm E = yield stress ratio = $\sqrt{\frac{250}{fy}}$ $= \sqrt{\frac{250}{250}} = 1$ $\lambda_{vv} = \frac{\left(\frac{l}{r_{vv}}\right)}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$ $= \frac{\frac{3000}{19.4}}{1X\sqrt{\frac{3.14^2 x2x10^5}{250}}}$ $= 1.741$ $\lambda_{\varphi} = \frac{(b_1+b_2)/2t}{\varepsilon\sqrt{\frac{\pi^2 E}{250}}}$	 **1M**	

$$= \frac{100+100}{1\sqrt{\frac{3.14^2 x 2 x 10^5}{250}} x 2\ x\ 10}$$

1M

$= 0.1125$

Equivalent slenderness ratio

$$\lambda_e = \sqrt{k_1 + k_2 \lambda_{vv}^2 + k_3 \lambda_{\phi}^2}$$

$= \sqrt{0.2 + 0.35 \text{ x } 1.741^2 + 20 \text{ x } 0.1125^2}$

1M

$= 1.23$

Imperfection factor α = 0.49

$$\phi = 0.5\left[1 + \alpha\,(\lambda_e - 0.2) + \lambda_e^2\right]$$

$= 0.5[1+0.49(1.23-0.2)+1.23^2]$

$= 1.509$

Compressive stress

$$f_{cd} = \chi \frac{f_y}{\gamma_{m0}}$$

$$\overline{\phi + \sqrt{\phi^2 - \lambda_e^2}}$$

$$= \frac{\frac{250}{1.1}}{1.509+\sqrt{1.509^2 - 1.23^2}}$$

$= 95.42$ N/mm^2

Design Compressive Load

= fcd x Ag

= 95.42 x 1903

= 181.58 x 10^3N

1M

= 181.58 kN

d) **A strut 2ISA 100 x 100 x 6 mm, 2.8 m long connected to 10 mm thick gusset plate on either side by two bolts at each end. Determine compressive load carrying capacity of angle strut. For ISA 100 x 100 x 6 mm,**
A = 1167 mm-

$$I_{xx} = I_{yy} = 111.3 \times 10^4 \text{ mm}^4,\ C_{xx} = C_{yy} = 26.76 \text{ mm}$$

2ISA 100 x 100 x 6
Thickness of gusset plate = 10mm
L = 2.8m

A single angle = 1167mm^2
Ixx = Iyy = 111.3 x 10mm^4
Cxx = 26.76mm
Ixx = 2 x 111.3 x 10^4
= 2.226 x 10^{6}mm^4 1M
Iyy = 2 [Iy+Ah^2]
$= 2[111.3 \times 10^4 + 1167(26.76+\frac{10}{2})^2]$
= 4.58 x 10^6 mm^4
Imin = 2.226 x 10^{6}mm^4 1M

$$rmin = \frac{\sqrt{Imin}}{A}$$
$$= \frac{\sqrt{2.226 \times 10^6}}{2 \times 1167}$$
= 30.88mm

Slenderness ratio 1M

$$SR = \frac{kL}{rmin}$$
$$= \frac{0.85 \times 2800}{30.88}$$
= 77.07

By interpolation fcd = 140.69N/mm^2

Design Compressive Strength

Pq = fcd x Ag
= 140.69 x (2 x 1167)
= 328.37 x 10^{3}N 1M
= 328.37kN

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4 B (a) 6

An ISA 125 x 75 x 8 mm is connected to 10 mm thick gusset plate by 4 bolts, 18 mm diameter. Assume F$_y$ = 250 *N/mm2* and Fu = 410 Nzmm? for ISA 125 x 75 x 8 mm, Ag = 1538 mm-. Determine _ tensile strength of angle section.

Solution:

Design tensile strength governed by yielding of gross section

$$Tdg = \frac{Ag\ fy}{\gamma m0}$$

$$= \frac{1538 x 250}{1.10} = 349545\ N$$

= 349.54 kN

(i) Design tensile strength governed by net section rupture

1M

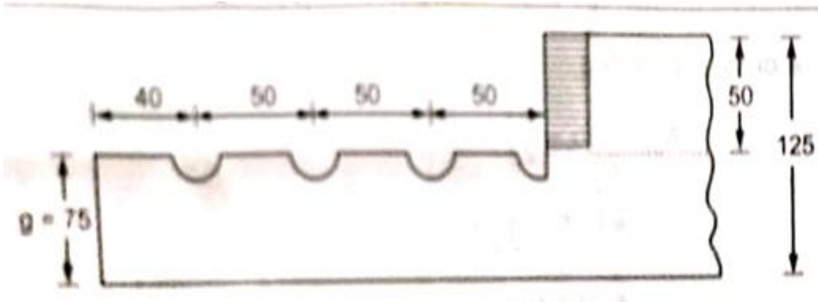


Fig. 3.17

Tdn = $0.9\,\frac{0.94nc}{Ym_1} + \frac{B\;Ago\;f_y}{Ym_0}$

Where B = $1.4 - 0.0776\frac{w}{t}$ x $\frac{fy}{fu}$ x $\frac{b_s}{Lc} \leq 0.9\,\frac{fu}{yu}\,\frac{ym_0}{ym_1}$

W = Outstanding lag width

= $75 - \frac{8}{2} = 71$ mm

B_s = w + g-t

= 71 + 75 – 8 = 128 mm

1M

L_c = Length of end connection

= Distance between outermost bolt holes

= 50 + 50 50= 150 mm

$\beta = 1.4 - 0.076\text{x}\,\frac{71}{8}\text{x}\,\frac{250}{410}\text{ x }\frac{138}{150} = 1.02$

$0.9\,\frac{f_u}{f_y}\,\frac{ym_0}{ym_1} = 0.9\text{x}\,\frac{410}{250}\text{ x }\frac{1.10}{1.25} = 1.29$

$\beta \leq 1.29 \geq 0.7$

β = 1.02 is acceptable

1M

Tdn = $\frac{0.9x\;808\;x\;410}{1.25} + \frac{1.02x568x250}{1.10}$

= 370194 N = 370.19 kN

(ii) Design tensile strength by block shear

A_{vg} = Minimum gross area in shear in along bolt line

= {40 + 3 x 50} x 8 = 1520 mm^2

A_{vg} = Minimum gross area in shear in along bolt line

= [40+ 50 – 3.5 x 20] x 8 = 960 mm^2

A_{vg} = Minimum gross area in tension from bolt hole to toe of angle perpendicular to the line of force

= 8 x 50 = 400 mm^2

A_{vg} = Minimum net area in tension from bolt hole to toe of angle perpendicular to the line of force

= (50 – 0.5 x 20) x 8 = 320 mm^2

1M

Design tensile strength governed by block shear

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

$$= \frac{1520\ x\ 250}{\sqrt{3}x\ 1.1} + \frac{0.9\ x\ 320\ x\ 410}{1.25}$$

= 293912 N

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

$$= \frac{400\ x\ 250}{1.1} + \frac{0.9\ x\ 960\ x\ 410}{\sqrt{3}x\ 1.25}$$

= 254525 N

1M

Tdb = Minimum of Tdb_1 and Tdb_2 = 254525 N = 254.525 kN

Tdb = Minimum of Tdb_1 and Tdb_2 = 254525 N = 254.525 kN

Design Tensile strength of angle

= Least of the strengths (i), (ii) a

= 254.525 kN

1M

(B) b — **An unequal angle section used as a tie member, carrying 125 ~ factored load, connected to 12 mm thick gusset plate by 20 mm diameter bolt. The design strength of 20 mm diameter bolt is 45.3 kN, assume edge distance as 2d, pitch as 2.5 dn and g = 60 mm.**

Design the tension member. d = 20 mm, diameter of bolt and dn diameter of bolt hole.

6

Given data
Td = 125 kN
d = 20 mm
do = 22 mm
Tb = 45.3 KN

Designation	Area in mm2	wt (kglm)
ISA 100 x 75 x 6	1014	8
ISA 100 x 75 x 12	1956	15.4

step I calculate the gross area required

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

γmo = 1.1

$= 1.1 \times \frac{1.25\, x\, 10^3}{250}$ **1M**

Ag, required = 550 mm^2
Select the section from table
ie ISA 150 x 75 x 6, Ag = 1014 mm^2

1M

2. find no. of bolts required (N)
N = $\frac{Td}{Tb}$ **=** $\frac{125}{45.3} = 2.759$ **say 3**
2. calculate the design strength due to yielding of gross section

$$T_{dg} = \frac{A_g f_y}{\gamma_{m0}},$$ **1M**

= 1014 x $\frac{250}{1.1}$
= 230.455 x 10^3 N
= 230.45 kN
3. calculate design strength due to rupture of net section

$$T_{dn} = \frac{\alpha A_n f_u}{\gamma_{m1}}$$

An =An1 +An2

		An1 = [a-do - $\frac{t}{2}$]t = [100-22-$\frac{6}{2}$]6 $= 450mm^2$ An2 = [b - $\frac{t}{2}$]t= [75-$\frac{6}{2}$]6 $= 432\ mm^2$ An = 450+432 =882 mm^2 (σ = 0.7 for N = 3 bolts) $Tdn = 0.7\ x\ 882\ x\ \frac{410}{1.25}$ Tdn = 202.50 x 10^3 **Tdn = 202.50kN** **Find design strength due to block shear** e = 2d = 2 x 20 =40 p =2.5 x 22 = 55 g = 60 from fig Lv = 40+55+55=150 Lt = a-g = 100-60=40 Avg = Lv.t = 150 x 6 = 900 Avg. = Lt.t =40 x 6 =240 Avh = [Lv – n.do]t = [150-2.5 x 22]6 = 570 Atn = [Lt – 0.5do]t = [40-0.5x 22]6 = 174 a. Find shear yield & tension fracture (SYTF) Tb1 = Avg. $\frac{fy}{\gamma mo\sqrt{3}} + 0.9\ Atn\ \frac{fy}{vm1}$ = 900 x $\frac{250}{1.1\sqrt{3}} + 0.9\ x\ 174\ x\ \frac{410}{1.25}$ = 118.094 x 10^3+51.36 x 10^3 **= 169.46 x 10^3N** **b. Find tension yield & shear failure** Tb2 = Afg $\frac{fy}{\gamma mo}$ + 0.9 Avn. $\frac{fu}{vm1\sqrt{3}}$ = 240 x $\frac{250}{1.1} + 0.9\ x\ 570\ x\ \frac{410}{1.25\sqrt{3}}$ = 54.54 x 10^3 + 97.147 x 10^3 = 151.687 x 10^3 Tb = Tb2 = 151.687 x 10^3 **Tb = 151.687 kN** Design strength = least of Tdg ,Tdn and Tb **Design strength= Td= 151.687 KN** **Check: Td = 151.687 $>$ 125 KN** **Hence O.K.**	**1M** **1M** **1M**	

Q.5		Attempt any TWO of the following:		16
	a)	**A truss has following details:** **Type of truss: Howe, span = 14.4 m, Rise = 3.5 m, panel point = 8 Nos., Spacing of truss = 3.5 m *c/c,* Weight of roof covering = 150 *N/m2,* Weight of purl in = 80 *N/m2,* Weight of bracing = 22 *N/m2,* Coefficient of external. Wing pressure (Cpe) = -0.5 Pd, coefficient of internal wind pressure (Cpi) = ± 0.2 Pd Design wind pressure Pd = 1200 *N/m2* and Self-weight of truss** **truss = (L/3 + 5) x 10 *N/m2* ..** **Find dead, live and wind load per panel point.** Solution: Span of Truss = 14.4 Spacing 3.5m Panel point = 8 nos Weight of roof covering = 150N/m^2 Weight of purlin = 80 N/m^2 Weight of bracing = 22N/m^2 Cpe = -0.5pd Cpi = $\underline{+}$0.2 pd Pd = 1200 N/m^2 SW of Truss = $(\frac{L}{3} + 5)\ x\ 10\ N/m^2$ **Dead load calculation**	**1M**	
		1) Weight of roof covering =$\frac{150}{cos} = \frac{150}{\cos 25.92}$ = 166.78 N/m^2 2) Weight of purlin = 80N/m^2 3) Weight of bracing = 22N/m^2 4) Self weight of Truss = $(\frac{L}{3} + 5)\ x\ 10$ = $(\frac{14.4}{3} + 5)\ x\ 10 = 98\ N/m^2$	**1M**	
		Total = 366.78 N/m^2 Total Dl = 366.78 x 14.4 x 3.5 = 18485.71 N DL on each top panel = $\frac{18485.71}{8}$ = 2310.71 N = 2.31 kN	**1M**	
		Dl on each panel point = $\frac{2310.71}{2}$ = 1155.36N = 1.16 kN **Note: If a student has taken 7 panels, since 8 panel points are given, it should be considered** **<u>Live load calculation</u>**		

		LL intensity for purlins = 750 – (0-10) x 20 = 750-(25.92-10) x 20 = 431.6 N/m² > 400 N/m² ok LL for Trusses = $\frac{2}{3}\ x\ 431.6$ = 287.73 N/m² For the given Truss LL = 287.73 x 14.4 x 3.5 = 14501.59N LL per panel = $\frac{14501.59}{8} = 1812.7\ N$ = 1.81 kN LL per end panel = $\frac{1812.7}{2} = 906.35\ N$ = 0.91 kN **Note : If a student has taken 7 panels, since 8 panel points are given, it should be considered** **WL calculations** Design wind pressure = (cpe – cpi)p = (-0.5-0.2) 1200 = 840 N/m² Wind load per panel point = Design wind pressure x Inclined panel length x spacing of Truss = 840 x $\frac{1.8}{cos 25.92}\ x\ 3.5$ **or** $\frac{840\ x\ 14.4\ x\ 3.5}{\cos 25}$ = 47071.1N load per panel point = $\frac{47071.1}{8}$ = 585838N = 5883.89 N = 5.88 kN (uplift) Wind load per end panel = $\frac{5.88}{2} = 2.94\ kN$	1M 1M 1M 1M 1M	

b) **Design an unequal angle purlin for truss with span = 6m, Rise = 3m, Spacing of truss = 4 m c/c, panels = 6 Nos., Dead load = 150 N/m2, Live load = 350 N/m2 and wind load = 750 N/m2. Apply checks as per IS 800-2007.**

Designation	wtN/m	I_{xx} (mm")	C_{xx}in mm
ISA 100 X 65 X 6	74	96.7 X 10^4	31.9
ISA 100 X 75 X 6	78	100.9 X 10^4	30.1
ISA 90 X 90 X 6	'67	70.6 X 10^4	28.7

Solution:

Span 6m
Rise 3m
Spacing = 4 m c/c
Panels = 6 nos
DL = 150N/m^2
LL = 350N/m^2
WL = 750N/m^2

$\sin\theta = Tan^{-1}\frac{3}{3} = 45^0$

Inclined span of Truss = $\frac{6}{\cos 45}$ = 8.49m

Spacing of purlin = $\frac{8.49}{6} = 1.41m$

Load calculations (For 1.41m spacing of purlins)

a) Dl of GI sheet = 150 x 1.41 = 211.5 N/m
Self weight of purlin (assumed) = 100N/m
Total = 311.5 N/m

b) Live load = 750-20 (θ-10)
= 750-20 (45-10)
= 50 N/m^2 < 400 N/m^2
:. LL = 400 N/m^2
LL on purlin per meter = 400 x 1.41 cos 45
=398.81 N/m

c) Wind load = -750 x 1.41 = -1057.5 N/m (Normal to roof) **1M**

Components of load along xx axis (parallel to roof) and along yy axis (normal to roof)

a) DL = Wdy = 311.5 x cos 45 = 220.26 N/m
Wdx = 311.5 x sin 45 = 220.26 N/m

b) LL Wdy = 398.81 x cos 45 = 282 N/m
Wdx = 398.81 x sin 45 = 282 N/m

c) WL Wwy = -1057.5 N/m **1M**
d) Wwx = 0

Factored Loads due to combinations

a) Load combination 1 =1.5 (DL + LL)

Wy1 = 1.5 (220.26+282) = 753.39 N/m

Wx1 = 1.5 (220.26+282) = 753.39 N/m

b) Load combination 2 = 1.5 (DL + WL)

Wy2 = 1.5 (220.26 – 1057.5) = -1255.86 N/m

Wx2 = 1.2 (220.26-0) = 330.39 N/m

1M

c) Load combination 3 = 1.2 (DL + LL + WL)

Wy3 = 1.2(220.26+282-1057.5) = -666.29 N/m

Wx3 = 1.2(220.26+282-0) = 602.71 N/m

1M

Hence among the above three combinations

1.5 (DL+wL) is critical

Wy = -1255.86 N/m

Wx = 330.39 N/m

Biaxial BM **1M**

Mx = $\frac{wy\ xL^2}{10}$

$= \frac{1255.86\ x\ 4^2}{10} = 2009.38\ Nm$

= 2.0 kNm

My = $\frac{wx \text{ x } \mathrm{L}^2}{10}$

$= \frac{330.39 \text{ x } 4^2}{10} = 528.62\ Nm$

= 0.529Knm

Selection of Angle section

Width of angle parallel to the roof = $\frac{L}{60}$ **1M**

$= \frac{4000}{60} = 66.67\ mm$

Depth of angle normal to the roof = $\frac{L}{45}$

$= \frac{4000}{45} = 88.89\ mm$

Try ISA 100 x 75 x 6@ 78 N/m [unequal angle purlin]

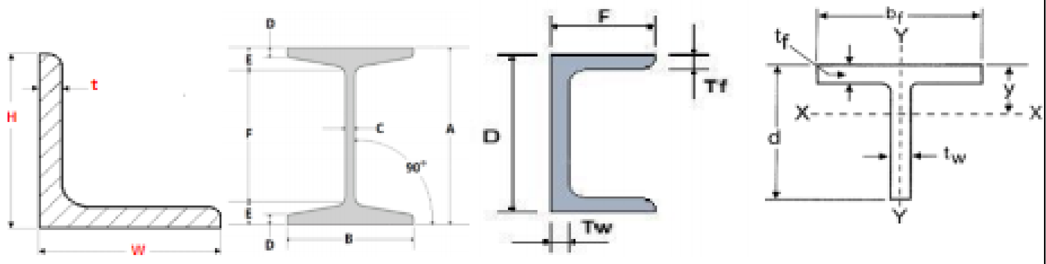
$\frac{b}{t} = \frac{75}{6} = 12.5 < 15.7\ e$

		$\frac{d}{t} = \frac{100}{6} = 16.67 < 84$ The section is semi – compact **Calculation of design moment** For bending @ xx axis Ze = $Zxx = \frac{Ixx}{ymax} = \frac{100.9\ x\ 10^4}{(100-30.1)}$ = 14.43×10^3 mm^3 Mdx = Ze $\frac{fy}{v_{mo}}$ = $14.43 \times 10^3 \times \frac{250}{1.1}$ = 3.28×10^6 Nmm = 3.28 kNm Mdy can not be calculated since Iyy is not given :- Mdy = 0 **Check for Biaxial bending** $\frac{Mx}{Mdx} + \frac{My}{Mdy} \leq 1$ $\frac{2}{3.28} + 0$ = 0.61 < 1 safe	1M 1 M 1 M	

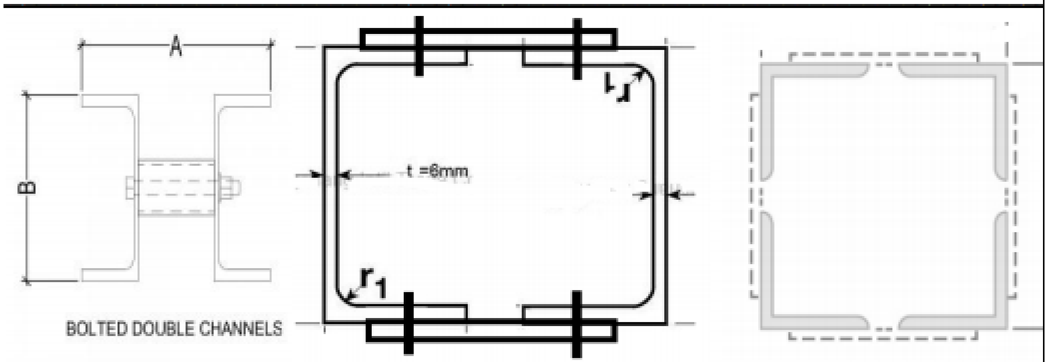
Q.5	**c)**	**A column ISHB 350 @ 724 *NIm* carrying a factored load of 1505 kN. The column rests on slab base and slab base rest on concrete pedestal.** **Assume** **Fy = 250 *N/mm2, Fu* = 410 *N/inm²*, Fck = 20 *N/mm2* (M20), ϒmo= 1.1 and SBC (Safe Bearing Capacity) = 150 *kN/m2.*For ISMB500 section bf = 250 mm and tf = 11.6mm.**		
		Solution: ISHB 350@ 724 N/M P = 1505 kN Fy = 250 N/mm^2 fu = 410 N/mm^2 sck = 20 N/mm^2 rmo = 1.1 SBC = 150kN/m^2	**1M**	
		NOTE : Since No data is given for ISHB 350, the student can assume the data or use The data given for ISHB 500 - : bf = 250mm - : tf = 11.6mm	**1M**	
		Bearing area of base plate $A = \frac{Pu}{0.6fck}$ $= \frac{1505x10^3}{0.6X20}$ = 125 416.67 mm^2	**1M**	
		Size of base plate Lp = $\frac{D-B}{2} + \sqrt{[(\frac{D-B}{2})^2 + A]}$ $= \frac{350-250}{2} + \sqrt{[(\frac{350-250}{2})^2 + 125416.67]}$ = 407.65mm = 410 mm Bp = $\frac{A}{Lp} = \frac{125\ 416.67}{410}$ = 305.89 =310 mm Large projection a = $(\frac{Lp-D}{2}) = \frac{410-350}{2}$ = 30 mm Smaller projection b = $(\frac{Bp-B}{2}) = \frac{310-250}{2}$ = 30 mm	**1M**	
		Area of base plate provided Ap = Lp x Bp = 410x 310 = 127 100 mm^2 > A	**1M**	
		Ultimate pressure from below on the slab base W = $\frac{Pu}{Lp \times Bp}$ $= \frac{1505x10^3}{410x\ 310}$ = 11.84 N /mm^2	**1M**	

		Thickness of base plate tp =SQRT[$\frac{2.5w(a^2-0.3b^2)\ \gamma_{mo}}{f_y}$] = $\sqrt{\frac{2.5x\ 11.84\ (30^2-0.3x30^2)x1.1}{250}}$ = 9.06mm = 10mm .< tf = 11.6 mm Provide thickness of 12mm **Size of concrete block** Af = $\frac{Pu.\gamma_{mo}}{SBC \text{ x } \gamma f}$ = $\frac{1505\ x\ 1.1}{150\ x1.5}$ $7.36\ m^2$ For equal projection Lp = $\frac{Lp-Bp}{2}+\sqrt{\left(\frac{Lp-Bp}{2}\right)2}+Af = \frac{0.41-0.31}{2}+\sqrt{\left(\frac{0.41-0.31}{2}\right)2}+7.36$ - 2.772 m - 2.80 or 2.85m Bf=$\frac{Af}{Lf}=\frac{7.36}{2.85}$ = 2.58 = 2.60m Provided M20 concrete Pedestal of size 2.85 m x 2.60m Actual projection a1 = $\frac{Lf-Lp}{2}=\frac{2850-410}{2}=1220mm$ b1 = $\frac{Bf-Bp}{2}=\frac{2600-310}{2}=1145mm$ Considering 45^0 angle of description ,provide depth of concrete block Df = 1220mm	1M 1 M	

Q.6		**Attempt any FOUR of the following:**		**16**
	a)	**State the different sections used for steel beam as per IS 800 code along with their two features of each.**	**1 M each**	



ANGLE SECTION I-SECTION CHANNEL SECTION T- SECTION



CHANNEL SECTION FACE TO FACE FOUR ANGLE SECTION

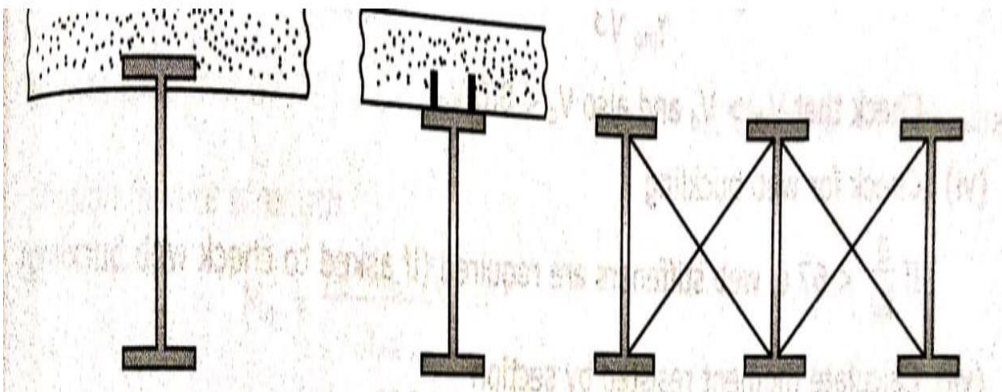
Features:

1)It can transmit direct axial pull applied at his ends.

2) It can sustain load upto ultimate load upto fracture.

Note : any four sections one mark each

b) **What is Laterally supported beam ? Draw two types of Laterally supported beam of each.**



a) Compression Flange Embedded in Slab

(b) Compression Flange Connected to Slab by Shear Connectors

(c) Beams Connected by Braces

Note : Student has to draw any two types of beams.

2 M Each for any 2

c) **An ISMB 250 is used as a simply supported beam of span 3 m to carry factored load of 30 kN/m, assume fy = 250 N/mm2. For ISMB 250, tw = 6.4 mm & h = 250 mm, check for shear only.**

ISMB 250

l = 3m

$w = 30kN/m$

fy = 250N/mm^2

tw = 6.4 mm

H = 250mm

Factored shear force Vu= $\frac{wL}{2}$

$= \frac{30\ x\ 3}{2}$ = 45 KN

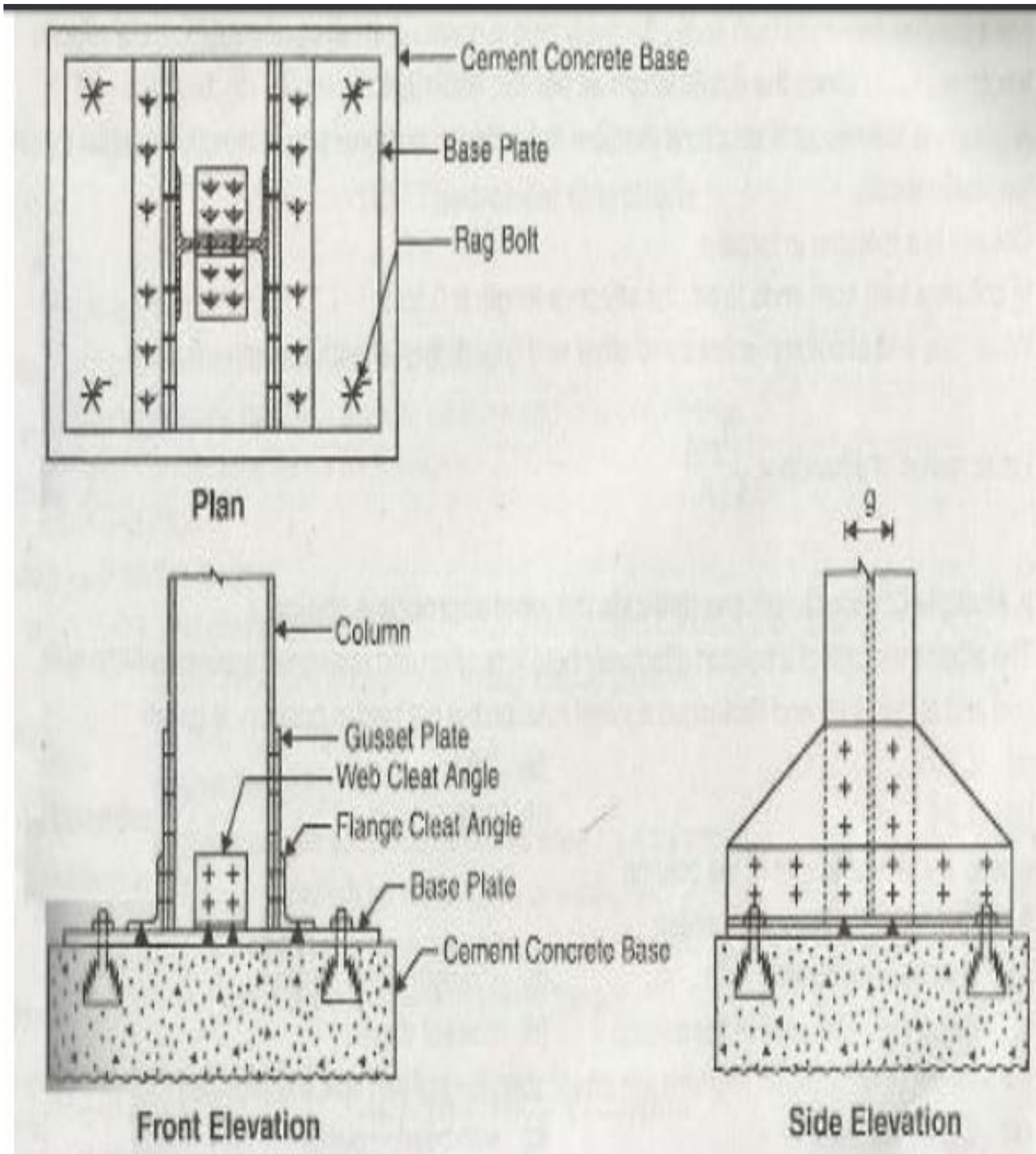
CHECK FOR SHEAR Vdr = $\frac{fy\ x\ tw\ x\ h}{\gamma mo\ x\ \sqrt{3}}$

$= \frac{250\ x\ 6.4\ x\ x250}{1.1\ x\ \sqrt{3}}$

= 209.95 x 10^{3}N

= 209.95 KN > Vu

2M

		Also $\frac{Vu}{Vdr}$ $= \frac{45}{209.95}$ $= 0.21 < 0.6$: Check for shear is satisfied	2M	
	c)	**Draw plan and sectional elevation of gusseted base by showing components of each.** 	4	

d **Write any four differences between slab base & gusseted base of column base.**

Sr. No.	Slab base	Gusseted base
1 .	The load on column is directly transferred to the base plate. thickness required for base plate is more.	The load on column is transferred through gusset plates and base plate together. Hence the thickness required is less than that of slab base.
2.	The cleat angles are used to fasten column section to base plate for width of column.	The cleat angles are used to faster gusset plate to base plate on more width, so that stiffness of joint is increased.
3.	The bearing surfaces may be (not machined). Hence the due to transit, unloading and may be caused.	All bearing surfaces are machined to ensure perfect control between them.
4.	The slab bases are simple in construction and fastening the elements speedily.	The gusseted base is complex in construction and more fastening joints are required. Hence low speed of joints.
5.	Economical as material required is less.	Expensive but stronger than slab base.

4