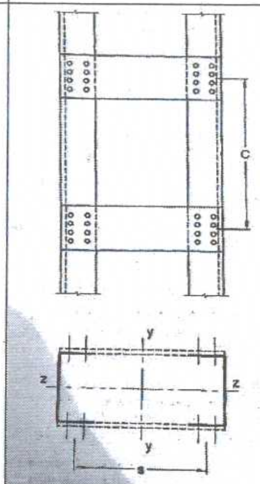
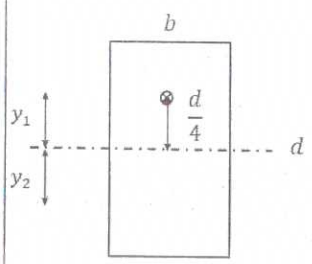
(1) (12) NOV-20-23

Scoring Indicators

COURSE NAME: DESIGN OF STEEL & RCC STRUCTURES

COURSE CODE: 5012 **QID:2109230005**

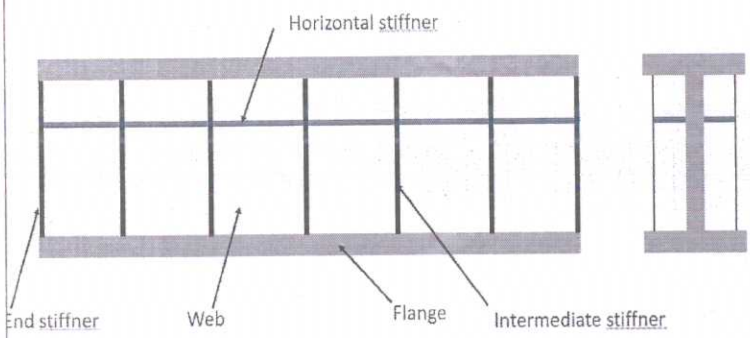
Q No	Scoring Indicators	Split score	Sub Total	Total score
	PART A			9
I. 1	$2x10^5\ MPa$	1	1	
I. 2	Effective length	1	1	
I. 3	Angle section, Channel section, I section	Any one -1	1	
I. 4	Equal	1	1	
I. 5	Flange splice	1	1	
I. 6	0.0035	1	1	
I. 7	One way slab & Two-way slab	½ + ½	1	
I. 8	Pedestal	1	1	
I. 9	Footing	1	1	
	PART B			24
II. 1		3	3	
II. 2	<u>Laterally unsupported beam</u> A beam in which the compression flange is not laterally supported. It is mainly subjected to bending, shear, web buckling and web crippling	3	3	

II. 3	 *Plastic moment carrying capacity,* M_p=$fy \ x \ Z_p$ $Z_p = \frac{A}{2}\ (y_1 + y_2)$ $= \frac{250x400}{2}\ \left(\frac{400}{4} + \frac{400}{4}\right)$ Z_p=10000000 mm3 $M_p = fy \ x \ Z_p \ = 250x10000000 = 2500 \ KN.m$		3	
II. 4	1. King post truss 2. Queen post truss 3. Howe truss 4. Fink truss 5. Fan truss 6. North light truss or saw tooth truss 7. Belgian truss	Any 3 3x1	3	
II. 5	Properties of ISLB 600 @995 N/m (IS800, table 46 ,page 138) h=600mm,b_f=210mm, t_f=15.5mm, t_w=10.5mm We have design shear strength of a beam $V_d = \frac{A_v f_{yw}}{\sqrt{3} \cdot \gamma_{m0}}$ $A_v = h \ x \ t_w \ = 600 \ x \ 10.5 = 6300 \ mm^2$ $V_d = \frac{6300 \ x \ 250}{\sqrt{3} \ x \ 1.1} = 826660.61N = 826.6 \ kN$		3	

<table>
<tr><td>II. 6</td><td>Limit state is defined as the acceptable limit of safety and serviceability requirements before failure. The aim of this method is that the structure should be able to withstand safely all the load that are liable to act on it throughout its life and it should also satisfy the serviceability requirements of limiting deflection and cracking. The most important limit states which are considered in design are as follows
A) Limit state of collapse
B) Limit state of serviceability</td><td>3</td><td>3</td><td></td></tr>
<tr><td>II. 7</td><td>Unsupported length – L
<table>
<tr><th>End condition</th><th>Effective length</th></tr>
<tr><td>Both ends fixed</td><td>0.65L</td></tr>
<tr><td>Both ends hinged</td><td>1.0L</td></tr>
<tr><td>One end fixed and other end hinged</td><td>0.8L</td></tr>
<tr><td>One end fixed and other end free</td><td>2.0L</td></tr>
<tr><td>Restrained against translation and rotation at one end and restrained against rotation and not restrained against translation at the other end</td><td>1.2L</td></tr>
</table></td><td>Any 3
3x1</td><td>3</td><td></td></tr>
<tr><td>II. 8</td><td>Short column - The column is considered as short when the slenderness ratio of column that is ratio of effective length to its least lateral dimension is less than or equal to 12. Such columns generally fail by crushing
Long column - If the slenderness ratio of the column is greater than 12, it is called as long or slender columns. Long columns generally fail by buckling</td><td>1
1+1</td><td>3</td><td></td></tr>
<tr><td>II.9</td><td>The pitch of transverse reinforcement shall not be more than the least of the following distances
i. The least lateral dimension of the compression member
ii. Sixteen times the smallest diameter of the longitudinal reinforcement bar to be tied and
iii. 300 mm</td><td>3</td><td>3</td><td></td></tr>
</table>

II.10	Foundations 1. Shallow foundation a) Spread footing b) Wall/strip footing c) Combined footing d) Cantilever/strap footing e) Mat/raft footing 2. Deep foundation a) Pile foundation b) Well foundation c) Pier foundation	Any 3 3x1	3	
	PART C			42
III.	Given data f_y = 250 N/mm^2 For ISA 100 x 65 x 10 mm from steel table (Page No. 14) A_g= 15.51 cm^2 = 1551 mm^2 We have $T_{dg} = A_g \frac{f_y}{\gamma_{mo}}$ $= \frac{1551 \, x \, 250}{1.1}$ =352.5kN		7	7
IV.	Length of strut is 3m, ISA 70 x 70 x 6mm Area of cross section $A_e = 8.06 \, cm^2 = 806 \, mm^2$ radius of gyration, $r_{xx} = r_{yy}$=21.4 mm, so $r_{min} =$ 21.4mm From table 10, we know that all angle sections belong to buckling class 'c' Design compressive stress (f_{cd}) $\frac{KL}{r}$, For weld, K= 0.7 $\frac{KL}{r} = \frac{0.7 \, x \, 3200}{21.4} = 104.67$ For $\frac{KL}{r} = 104.67$ & $f_y = 250 \, N/mm^2$ From table 9 (c) We will get $f_{cd} = 101.21 \, N/mm^2$ Load carrying capacity with respect to buckling class c $P_d = A_e f_{cd}$ = 806 x 101.21 =81575.26N =81.58kN		7	7

<table>
<tr><td></td><td>V.</td><td>Given data
ISA 200x100x12
$$A_g = 34.59\ cm^2 = 3459\ mm^2$$
L =length of connected leg = 200mm
b = length of outstanding leg = 100 mm
Size of weld S= 4mm
Length of weld L_w = 260mm
<u>Design strength due to net section rupture</u>
$T_{dn} = \frac{0.9A_{nc}f_u}{\gamma_{ml}} + \frac{\beta A_{go}f_y}{\gamma_{mo}}$
$A_{nc} = \left(L - \frac{t}{2}\right)t = \left(200 - \frac{12}{2}\right)12 = 2328\ mm^2$
$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right)$
w =100mm, t =12mm,
$b_s = W$ =100mm
L_c =length of weld=260mm
$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) = 1.4 - 0.076\left(\frac{100}{12}\right)\left(\frac{260}{440}\right)\left(\frac{100}{300}\right) = 1.275$
$\beta \quad \leq 0.9\left(\frac{f_u\gamma_{mo}}{f_y\gamma_{ml}}\right) = 0.9\left(\frac{440*1.1}{300*1.25}\right) =$
1.161
≥ 0.7
So β = 1.162
$A_{go} = \left[b - \frac{t}{2}\right]t = \left[100 - \frac{12}{2}\right]10 = 940\ mm^2$
Design strength due to net section rupture is
$T_{dn} = \frac{0.9A_{nc}f_u}{\gamma_{ml}} + \frac{\beta A_{go}f_y}{\gamma_{mo}}$
$= \frac{0.9*2328*440}{1.25} + \frac{1.162*940*300}{1.1} = 1035.4kN$</td><td></td><td>7</td><td>7</td></tr>
<tr><td></td><td>VI.</td><td><u>Design of compression member</u>
1) Find the factored load (1.5xworking load)
2) Assume a suitable value of design compressive stress (f_{cd}) to determine the area of cross section
3) Calculate the effective sectional area required
$$A_e = \frac{P_d}{f_{cd}}$$</td><td>7</td><td>7</td><td>7</td></tr>
</table>

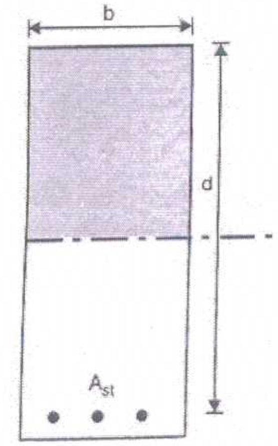
	4) Select a section having gross area greater than the calculated area 5) Calculate slenderness ratio ($\frac{KL}{r}$) 4) Find the value of design compressive stress(f_{cd}) 5) Calculate the load carrying capacity of the selected section $$P_d = A_e f_{cd}$$ 6) If it is greater than factored load the design is safe, otherwise redesign the section by considering another section of area greater than required area			
VII.	Plate girder  		7	7
VIII.	Properties of ISWB 550@ 1125N/m (SP6, page 4) $I_{xx} = 74906.1 \, x \, 10^4 \, mm^4$ Total load w = 24 kN/m = 24 N/mm (For serviceability condition partial safety factor is 1) Effective span l =6000mm Maximum deflection for SS beam with udl $\delta_{max} = \frac{5wl^4}{384EI} = \frac{5 \, x \, 24 \, x \, 6000^4}{384 \, x \, 2 \, x \, 10^5 \, x \, 74906.1 \, x \, 10^4}$ = 2.70 mm Permissible limit of deflection for a simply supported beam $\delta_{per} = \frac{span}{240} = \frac{6000}{240} = 25mm$ $\delta_{max} < \delta_{per}$ hence the beam is safe against deflection		7	7

IX.

7 7

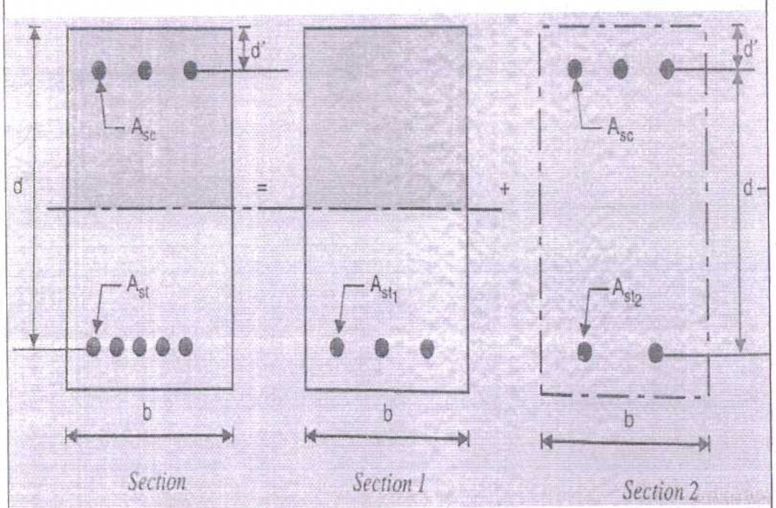
Singly reinforced beam

The beam that is longitudinally reinforced only in tension zone, it is known as singly reinforced beam. In such beams the tension is carried by the reinforcement while the compression is carried by the concrete.



Doubly reinforced beam

The beam that is reinforced with steel in the tension and compression zone is known as doubly reinforced beam. This type of beam is provided mainly when the depth of the beam is restricted.



X.	Adopt b=300mm, D=650mm Then d=650-50=600mm From Table D of SP16-1980 , for fy=415 N/mm² & fck=20 N/mm² $\frac{Mulim}{bd^2} = 2.76$ $Mulim = 298.08\ KN.m$ $Actual\ moment = \frac{wl^2}{8} = \frac{52.3125x8^2}{8} = 418.4KN.m$ Where w_u=1.5(30+25x0.3x0.65) =52.3125 KN/m Since Actual moment is greater than Mulim, the section is to be designed as a doubly reinforced section. $\frac{Mu}{bd^2} = \frac{418.4x10^6}{300x600^2} = 3.87\frac{N}{mm^2}$ $\frac{d'}{d} = \frac{40}{600} = 0.066$ Next higher value of d'/d = 0.1 will be used for referring tables Referring to table 50(SP16), Pt = 1.306% , Pc = 0.369% So, Ast = 2350 mm^2 and Asc = 664 mm^2		7	7
XI.	f_{ck} = 20 N/mm² f_y = 250 N/mm² b = 300 mm, d = 550 mm ,A_{st} = 1963 mm² Depth of Neutral axis (Xu): $\frac{X_u}{d} = \frac{0.87\, f_y\, A_{st}}{0.36\, f_{ck}\, bd}$ $\frac{X_u}{d} = \frac{0.87\ x\ 250\ x\ 1963}{0.36\ x\ 20\ x\ 300\ x\ 550}$ $\frac{X_u}{d} = 0.36$ $X_{u\ max}$ (limiting depth of Neutral axis) $\frac{X_{umax}}{d} = 0.53$ $\frac{X_u}{d} < \frac{X_{umax}}{d}$		7	7

	Hence, it is an under reinforced section. Ultimate Moment of Resistance (M_u): $Mu = 0.87 \times fy \times A_{st}\ d(1 - 0.42\frac{X_u}{d})$ $= 0.87 \times 250 \times 1963 \times 550\ (1 - 0.42 \times 0.36)$ $= 199318505.1$ N mm $= 199.32$kNm.			
XII.	Given that l_x= 3.5m, l_y= 7.5m , LL=5 KN/m^2 Grade of concrete =M20, Grade of steel =Fe 415 *Check for one way or two way* $\frac{l_y}{l_x} = \frac{7.5}{3.5} = 2.14 > 2$ Hence one-way slab *Calculation of depth* Assume $\frac{l}{d\ basic} = 20$ $\frac{l_x}{d} = 20,\ d = \frac{3500}{20} = 175$mm so, take d as 170mm D = 170+5+20 = 195 mm, so Take D=200mm (Assuming 10mm dia bars & 20mm clear cover) *Calculation of effective span* *(IS 456: 2000 PAGE 34)* l+d = 3.5+.17 = 3.67m c/c distance b/w supports = $3.5 + \frac{.23}{2} \times 2$ = 3.73 m so l_{eff} = 3.67m *Calculation of load* DL = Self weight + finishes =25 x 0.2 x 1 + 1 x 1 = 6 KN/m^2 LL = 5 x 1 = 5 KN/m^2 Total load = DL+LL =11 KN/m^2 Total factored load = 1.5 x 11 = 16.5 KN/m^2		7	7

Check whether section is over reinforced or under reinforced

(IS 456: 2000 PAGE 96

Max BM = $\frac{w\, l_{eff}^2}{8} = \frac{16.5 * 3.67^2}{8}$ = 27.77 kNm

$M_{u\,lim}$ = .36 x f_{ck} x $\frac{x_{u\,max}}{d}$ x {$1 - .42\ \frac{x_{u\,max}}{d}$} bd^2

= .36 x 20x .48x {$1 - .42$ x .48} x1000 x 170^2

= 79.74 x 10^6 N.mm = 79.74 kN.m

Max BM < $M_{u\,lim}$

Section is under reinforced, proceed with the design

-: Determination of area of steel

$M_{u\,lim}$ = .87 x f_y xA_{st} x d {$1 - \frac{f_y\, A_{st}}{b\, d\, f_{ck}}$}

27.77 x 10^6 = .87 x 415 x A_{st} x 170 {$1 - \frac{415\ A_{st}}{1000 * 170 * 20}$}

A_{st} = 480.63 mm^2 say 500 mm^2

Assuming 10mm dia bars

Number of bars = $\frac{500}{\frac{\pi}{4} x\ 10^2}$ =6.37 bars

Spacing = $\frac{1000}{6.37}$ = 156.98 mm say 150 mm

Provide 10mm dia bars at 150mm spacing as main reinforcement

Distribution steel (IS 456:2000 page 48)

Minimum area of steel

distribution steel = .0012 x 1000 x 200

= 240 mm^2

Assuming 8mm dia bars

Number of bars = $\frac{240}{\frac{\pi}{4} x\ 8^2}$ =4.775 bars

Spacing = $\frac{1000}{4.755}$ = 209.44 mm say 200 mm

Provide 8mm dia bars at 200 mm spacing as secondary reinforcement (Distribution steel)

XIII.	Given data b or D = 500mm P_u=3000kN f_{ck} =25 N/mm^2 f_y =415 N/mm^2 Since emin < 0.05b we can use the equation $P_u = 0.4f_{ck}A_c + 0.67f_yA_{sc}$ Gross area A_g = 500 x 500 =250000 mm^2 Area of concrete A_c=A_g-A_{sc} Substituting the values in the eqn $P_u = 0.4f_{ck}A_c + 0.67f_yA_{sc}$ 3000 x 10^3 = 0.4 x 25 x (250000-A_{sc}) + 0.67 x 415 x A_{sc} By solving we will get A_{sc}=1865.32 mm^2 A_{sc}=1865.32 mm^2 Assuming 20mm dia bars No. of bars =$\frac{1865.32}{\frac{\pi}{4} x\ 20^2}$ =5.93 say 6 bars **So, provide 6 No. of 20mm dia bars as longitudinal reinforcement** <u>Transverse reinforcements</u> Minimum dia of bars ($\frac{1}{4}$ of long bars) = $\frac{20}{4}$ =5 So, provide 8mm dia bars As per IS code spacing is least of the following 1. Least lateral dimension =500mm 2. 16 x 20 =320mm 3. 300 mm **Hence provide 8mm dia ~~square~~ lateral tie @300mm c/c**		7	7

XIV.	<u>Design steps – Isolated column footing</u> 1. Determination of size of footing Area of footing = $\frac{Load}{SBC\ of\ soil}$ Total load = Load transferred from column + weight of footing Usually, weight of footing is taken as 10% of load from column Area= $\frac{1.1W}{SBC}$ assume suitable ratio to length and breadth and find L&B (If rectangular) 2. Calculation of depth based on bending compression 3. Calculation of depth based on one way shear 4. Check for two-way shear 5. Determination of area of steel	7	7	7