Scheme of valuation.

Scoring indicators.

Rev.(15) Course code – 6013.

Course title – Structural Design II

Qn No.	Scoring indicators	Split up Score	Sub total	Total
	PART – A. (max. marks : 10)			
I				
1.	A tension member is one which, is intended to resist axial tension.	2		2
2.	Rolled steel I sections, channel sections, angle sections, T sections steel bars, steel tubes etc	4×1/2		2
3.	compression members are those members in which axial forces act to cause compression	2		2
4.	A wall carrying a vertical load in addition to its own weight together with any lateral load	2		2
5.	compression flange is laterally supported by flooring. It is mainly subjected to bending & shear	2		2
	PART – B. (max. marks : 30)			
II				
1)	Advantages – Speedy construction is possible. – Dismantling and reuse will be possible – Occupy less space. – modification of sectional properties that of can be easily carried out. – Self wt is less.	any 3	3	
	Dis advantages – subjected to corrosion, periodic maintenance is required. – Tempt. stress will be high – low fire resistance. – Require skill labour for erection.	any 3	3	6

II 2) $t = 0.7 \times s = 0.7 \times 6 = 4.2\,m$. Design stress $f_{wd} = \frac{f_{wn}}{\gamma_{mw}} = \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$ 3

Ref. table no. 5 Page No. 30

$f_{wd} = \frac{410}{\sqrt{3} \times 1.25}$

design strength $= (L_w t) \frac{f_u}{\sqrt{3}\,\gamma_{mw}} = 240 \times 4.2 \times \frac{410}{\sqrt{3} \times 1.25}$ 3 6

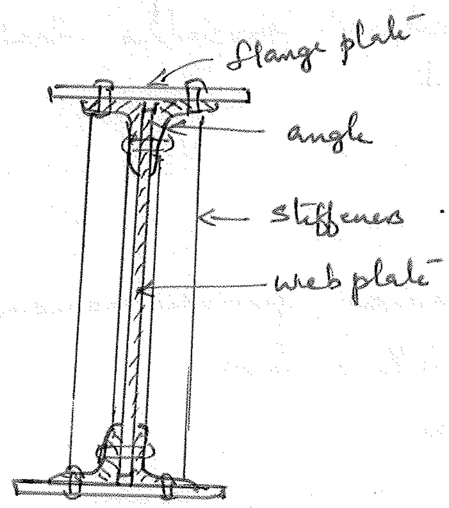
$= \underline{190.8\ kN}$.

II 3)
1) axial load
2) factored load = FOS × safe load.
3) Design comp. stress f_{cd}.
4) Area calculated $P_u = A f_{cd}$

$A_{cal} = \frac{P_u}{f_{cd}}$.

5) Providing section. → area morethan calculated area. 3
6) $\lambda = \frac{kL}{r_{min}}$
7) Design comp. stress based on λ and yield stress. obtained from table no. 9(c) page no. 42 (IS. 800) f_{cd} can be calculated
8) $\underline{P_d = A_{pro} \times f_{cd}}$. 3 6

II 4)



6 6

II 5. a) class 1 (plastic)
b) class 2 (Compact)
c) class 3 (Semi compact)
d) class 4 (slender)

Page no: 17 (3.7) IS 800 | 3×2 | 6

II 6. $\theta = 24^\circ$

Live load on roof $750 - (\theta - 10^\circ)\,20$.

$750 - (24^\circ - 10^\circ)\,20 = 470\ N/m^2$. | 4

Live load on truss $= \left(\frac{2}{3}\right) \cancel{750 \times}\ 470 = 313.33\ N/m^2$ | 2 | 6.

II 7. i) cavity wall :- A wall comprising of two leaves, each leaf being separated by a cavity and tied together with metal tie or bonding units to ensure that the two leaves ~~being either left~~ act as a one structural unit. | 3

ii) Faced wall :- A wall in which facing and backing of two different material are bonded together to ensure common action under load is called faced wall. | 3 | 6

PART – C (max. marks : 60)

UNIT – I

III a) Physical properties

Density – $7850\ kg/m^3$

modulus of elasticity – $2 \times 10^5\ N/mm^2$

Poisson's ratio $= 0.3$

Modulus of rigidity $= 0.769 \times 10^5\ N/m^2$

Coeff. of thermal expansion $= 12 \times 10^{-6}/^\circ c$. | 3

Mechanical properties

1. yield stress (fy)
2. ultimate stress (fu)
3. percentage of elongation
4. notch toughness. | 2 | 5

III b)

1. Size of weld $s = T - 1.5 = 8.5\,mm$. adopt 8 mm.

2. $t = 0.7 \times 8 = 5.6$.

3. Design strength, $f_{wd} = \dfrac{f_u}{\sqrt{3}\,\gamma_{mw}} = 189.37\ mm^2$

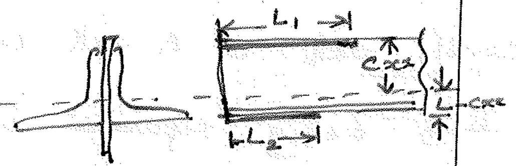
$$= (L_w t)\, f_{wd} = \cancel{189}\, L_w \times 5.6 \times 189.37$$ 3

4. Factored full design strength $= \dfrac{1.5\, A_g f_y}{\gamma_{m0}} = 716.5\ kN$

5. $L_w \rightarrow (L_w t)(f_{wd}) = \dfrac{1.5\, A_g f_y}{\gamma_{m0}} \quad \therefore L_w = 675.7\,mm.$ 3

6. Length of weld in top & bottom position

$L_w = L_1 + L_2 = 675.7\,mm.$



$L_1\, c_{xx} = L_2\,(L - c_{xx})$

$L_1 = 465.97\ mm \qquad L_2 = 209.7\ mm$ 4 10

(OR)

IV a)

- Shearing failure
- Tensile failure
- Bearing failure. 5

V b) Lap joint. $V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}}$, $V_{nsb} = \dfrac{f_u}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb})$

$n_n = 1$, $n_s = 0$. $V_{nsb} = 58\ kN$

$V_{dsb} = \dfrac{V_{nsb}}{\gamma_{mb}} = 46.40\ kN$ 5

Single cover butt joint.

$t = 10\,mm$, $V_{dsb} = 46.40\ kN$.

$V_{dpb} = \dfrac{2.5\, k_b\, d t f_u}{\gamma_{mb}} = \dfrac{2.5 \times 0.5 \times 20 \times 10 \times 410}{1.25}$

$= 82\ kN$.

The strength bolt is min. of shear & bearing $= 46.40\ kN$. 5 10

V a) i) Gross area :– The total area of cross section of the member without deducting the area of holes in it is called gross area of the member. 2

ii) Net area :– The area of c/s of the member after deducting the area of holes. 2

iii) Net effective area :– The equivalent area of an imaginary axially loaded member of equal load carrying capacity. ~~called~~ 2 6

V b) Design strength due to yielding of gross section.

$$T_{dg} = \frac{A_g f_y}{\gamma_{mo}} \qquad A_g = 4278\,mm^2. \qquad T_{dg} = \underline{1166.7}\ KN$$ 2

Design strength due to rupture

$$T_{dn} = \frac{0.9\,A_{nc}\,f_u}{\gamma_{m1}} + \frac{\beta\,A_{go}\,f_y}{\gamma_{mo}}$$

$$\beta = 1.4 - 0.076\left(\frac{w}{t}\right)\left(\frac{f_y}{f_u}\right)\frac{b_s}{L_c}$$

$$= 1.4 - 0.076\left(\frac{100}{15}\right)\left(\frac{300}{440}\right)\left(\frac{100}{240}\right) = 1.256$$ 3

$$A_{nc} = \left(L - \frac{t}{2}\right)t = \left(200 - \frac{15}{2}\right)15 = 2887.5\,mm^2.$$

$$A_{go} = \left(b - \frac{t}{2}\right)t = 1387.5\,mm^2.$$

$$T_{dn} = \frac{(0.9)(2887.5)(440)}{1.25} + \frac{(1.256)(1387.5)(300)}{1.1}$$

$$= 914.76 + 475.28 = \underline{1390.04}\ kN$$ 4 9

$$=$$

(OR)

VI a) 1) Angle 2) Double angle (3) I section

4) I section with cover plate 5) T section.

6) channel section. 6×1 6

VI b)

both ends are fixed $(kl) = \frac{l}{2} = 2.5\,m$.

(ii) design strength $P_d = A \cdot f_{cd}$.

$$f_{cd} = \frac{f_y / \gamma_{m0}}{\phi + (\phi^2 - \lambda^2)^{1/2}}$$

$$\frac{kl}{r_{min}} = \frac{2500}{28} = 89.28, \quad f_{cc} = \frac{(\pi^2 E)}{\left(\frac{kl}{r_{min}}\right)^2} = 247.67\ N/mm^2$$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} = 1$$

Imperfection factor (α)

first calculate the value of $\frac{h}{b_f} = 2 > 1.2 = 9.4\,mm \leq 40\,m$.

buckling about axis and buckling class will be z-z - a, y-y - b 3

a → 0.21
b → 0.34 (Page 35, Table 7, IS 800)

Calculate ϕ

for ϕ_z → α value can be 0.21

ϕ_y → α value can be 0.34

$$\phi_z = 0.5\left[1 + \alpha(\lambda - 0.2) + \lambda^2\right] = 1.084.$$

$$\phi_y = 1.136.$$

$$f_{cdz} = \frac{f_y / \gamma_{m0}}{\phi_z + (\phi_z^2 - \lambda^2)^{1/2}} = 151.31\ N/m^2.$$

$$f_{cdy} = \frac{f_y / \gamma_{m0}}{\phi_y + (\phi_y^2 - \lambda^2)^{1/2}} = 135.76\ N/m^2.$$ 4

Design capacity $P_d = A_e f_{cd}$

$= 4808 \times 135.76$

$= \underline{652.76\ kN}$. 2 9

UNIT – III

VII a) Area = bd , $\bar{y}_1 = \frac{d}{4} = \bar{y}_2$

$$Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2) = \frac{b}{2}\left(\frac{d}{4} + \frac{d}{4}\right) = \frac{bd^2}{4}$$ 3

$$Z = \frac{I}{y} = \frac{\frac{bd^3}{12}}{d/2} = \frac{bd^2}{6}, \quad S = \frac{Z_p}{Z} = \underline{1.5}$$ 3 6

VII b) $0.6\,V_d > V_u$

$w_1 = 20$ kN/m.

$w_2 = 995$ N/m

Total $w = w_1 + w_2 = 20.995$ kN/m.

$w_u = 1.5 \times 20.995 = 31.49$ kN/m. 2

$V_u = \frac{w_u l}{2} = 94.47$ kN. 2

Design shear $V_d = \frac{A_v f_y}{\sqrt{3}\,\gamma_{m0}}$

A_v = shear area = 600×10.5

$V_d = \frac{(h \times t_w) f_y}{\sqrt{3} \times \gamma_{m0}} = \underline{826.66}$ kN 3

60% of Vd = $0.6\,V_d$ = 496 kN.

$0.6\,V_d > V_u$ The beam is safe in shear

∴ 496 kN > $\underline{94.47}$ kN. 2 9

OR

VIII a) ~~1) King post truss~~

2) ~~Queen post truss~~

3) ~~Howe flat truss~~

4) ~~Fink roof truss~~

5) ~~Fan truss~~

6) ~~North light truss~~

7) ~~Saw tooth truss~~

8) ~~Monitor truss~~

Step ① ultimate design load w_u,

② BM & SF (M_u, & V_u) calculation

③ Plastic section modulus calculation (Z_p cal)

(4) providing of section from steel table

(5) section classification

• plastic • compact • semi compact

• slender. 6×1 6

(6) check for shear

(7) check for deflection

(8) check for design capacity.

VIII

b. 1) calculation of load (w) and span (l).

$V = \frac{wl}{2} = 80$.

$w = \frac{160}{l}$ —①

$M = \frac{wl^2}{8} = 100$

$w = \frac{800}{l^2}$ —②.

equate ① & ②

$\frac{160}{l} = \frac{800}{l^2}$.

$l = 5m$.

$w = \frac{160}{l} = \frac{160}{5} = 32\ kN/m$

2) Factored load $w_u = 1.5 \times w = 48\ kN/m$.

$V_u = 120\ kN$

$M_u = 150\ kNm$. 3

3) check for shear $(0.6 V_d > V_u)$

$$V_d = \frac{A_v f_y}{\sqrt{3}\,\gamma_{m0}} = \frac{(h \times t_w) f_y}{\sqrt{3} \times \gamma_{m0}} = \frac{400 \times 8.9 \times 250}{\sqrt{3} \times 1.1} = 467.12 \times 10^3\ N.$$

$0.6 V_d > V_u$ ∴ The beam is safe against shear. 3

4) check for deflection $(\delta_{max} < \delta_{per})$

$$\delta_{max} = \frac{5wl^4}{384EI} = \frac{5 \times 32 \times 5000^4}{384 \times 2 \times 10^5 \times 20458.3 \times 10^4} = 6.36mm$$

$\delta_{per} = \frac{l}{300} = 16.66\ mm$. $\delta_{max} < \delta_{per}$ ∴ Safe. 3 9

UNIT . IV

IX

a) 1. Dead load 2) Live load 3) wind load 4) Snowload
5) Earth quake load 6) Erection load. 3

Dead load
i) Self wt of sheeting
ii) self wt of purlin
iii) self wt of bracing
iv) self wt of truss.

Live load — based on slope (θ)

Wind load consists of
i) basic wind speed (V_b)
ii) Design wind speed (V_z)
iii) Design wind pressure (P_d)
iv wind pressure on roof 2 5

1X

b) basic wind speed V_b for Delhi is 47 m/s.

Dead load.

1. Self wt of A.C sheeting = 200 N/m²
2. " purlin = 100 N/m²
3. " bracing = 20 N/m².
4. ~~truss~~ $= \underline{10\left(\frac{20}{3}+5\right)} \frac{4.5}{4} = 131.25$ N/m². 2

~~Total~~ DL = 451.25 N/m².

Live load

$\tan\theta = \frac{\text{Rise}}{\text{half span}}$ $\tan\theta = \frac{4}{10}$, $\theta = 21.80°$.

$LL = 750 - (\theta - 10°)20 = \underline{513.97 \text{ N/m}^2}$

Live load on truss $= \frac{2}{3}$ L.L $= 342.64$ N/m² 2

Internal pressure coeff. (C_{pe})

wind angle 0°		wind angle 90°	
wind ward side EF	Leeward side GH	wind ward side EG	Leeward side FH.
30 → 0 20 → 0.4 10 → +0.4 1 → 0.04	Cpe has no change b/w 20° & 30° ∴ Cpe = −0.4	Cpe has no change b/w 20° & 30° Cpe = −0.7	Cpe no change b/w 20° & 30°. Cpe = −0.6.

3

for 1.8° → 0.072. Cpe → = −0.328 Cpe is the min of above four values.

∴ Cpe = −0.7

wind load = $F = (C_{pe} - C_{pi}) A P_d$.

$P_d = 0.6 V_z^2$ $V_z = k_1 k_2 k_3 V_b = \underline{41.36}$ m/s.

$P_d = 0.6 \times 41.36^2 = \underline{1026.38}$

Assuming permeability can be medium then

$C_{pi} \pm 0.5$ $C_{pe} - C_{pi} = \cancel{+1.2}\ -1.2$ or -0.2. min $= \underline{-1.2}$

$C_{pe} = -0.7$

$\therefore F = (C_{pe} - C_{pi}) P_d = (-1.2) 1026.38$

$F = \underline{1231.65 \text{ N/m}^2}$ 3 10

9/10

OR

× a) Effective height :- The effective ht of the wall (h) to be used for calculating the slenderness ratio, is the function of actual ht (H) of the wall and the condition of lateral support. h varies from 0.75 H to 1.5H 2½

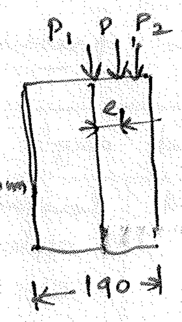
Effective length :- The eff. length of the wall (l) to be used in the calculation of 'λ' depends upon the length of wall from or centres of cross wall, piers or buttresses. 'l' varies from 0.8.L to 2L 2½ 5.

× b)

Taking moments about the centre of wall

$$P_1 \times 0 + P_2 \times 30 = P.\bar{e} \qquad \begin{pmatrix} P_1 = 50\ kN \\ P_2 = 30\ kN \end{pmatrix}$$

$$\therefore \bar{e} = \frac{P_2 \times 30}{P} = \frac{30 \times 30}{(30+50)} = \frac{90}{8}\ mm.$$



∴ stress in the masonry =

= Axial stress ± bending stress

$$f = \frac{P}{1000 \times t} \pm \frac{P \times \bar{e} \times 6}{1000 \times t^2}$$

$$= \frac{90 \times 10^3}{1000 \times 200} \pm 80 \times 10^3 \times \frac{90}{8} \times \frac{6}{1000 \times 200 \times 200}$$

$$= 0.450 \pm 0.135$$

$$f_1 = (0.450 + 0.135) = 0.585\ N/mm^2$$

$$f_2 = (0.450 - 0.135) = 0.315\ N/mm^2.$$

Both the stresses are compressive in nature.

10/10