IAT- 3 → Department of Civil Engg.

USN

SOLUTION OF IA-3

SOLUTION OF Internal Assessment Test3 – May 2018

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Sub:	Design of steel structures	Sub Code:	15CV62	Branch:	Civil
Date: 21/05/2018	Duration: 90 min's / Max Marks: 50	Sem / Sec:	6 A and 6B		OBE

Note: Use of IS 800:2007 is permitted and Assume missing data. **PART A : Answer ALL Questions**		MARKS	CO	RBT
1 (a)	Design a a beam of effective span 6m. Subjected to an UDL 10kN/m. Along with 150kN load. The beam is laterally supported. The thickness of wall is 230mm	[15]	CO3	L4
2 (a)	Determine the compressive strength of angle strut ISA 100X65 X 8mm with length 3m when connected by i) with single bolt. ii) More than two bolts iii) welded connection	[15]	CO4	L4
PART B: Answer any ONE full question.				
3 (a)	Design a compression member using double channel section " Face to Face " to carry a factored load of 2000kN. The length of column is 5m. With one end fixed and one end hinged. Also design " single lacing system ".	20	CO4	L4
4 (a)	Design a "column splice" for ISHB @72.4 kg/m subjected to 600kN load, 50kN-m moment and 150kN horizontal shear. Use M20 property class 5.6 bolts. Design both flange splice and web splice.	20	CO4	L4

Q.2. Design a beam of effective span 6m. Subjected to an UDL 10kN/m. along with 100kN load. The beam is laterally supported. The thickness of wall is 230mm.

(a) load calculation.

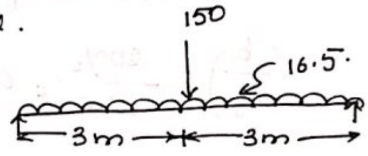
UDL on beam = 10 kN/m.

Assume self wt. = 1 kN/m.

11 kN/m.

ultimate load = UDL = 11 × 1.5 = 16.5 kN/m.

& ultimate point load = 1.5×100 = 150 kN.



$$V_u = \frac{16.5 \times 6}{2} + \frac{150}{2} = 124.5 \text{ kN}.$$

$$M_u = \frac{16.5 \times 6^2}{8} + \frac{150 \times 6}{4} = 299.2 \text{ kN-m}.$$

Plastic Modulus required $= Z_p = \frac{M_d \gamma_{mo}}{\beta \times f_y}$ mm². → pg 53.

$$Z_p = \frac{299.2 \times 10^6 \times 1.10}{1 \times 250} = 1316.5 \times 10^3 \text{ mm}^3 \simeq 1316.5 \text{ cm}^3$$

let us increase value by 20% approximately.

$$= 1.20 \times 1316.5 = 1580 \text{ cm}^3.$$

from IS-800 page 138 Try. ISWB -450 @ 79.4 kg/m.

$Z_p = 1760.59$ cm³

$Z_e = 1558.1$ cm³ $= Z_{xx}$.

$I_{xx} = 35057.6$ cm⁴ → SP 6 steel table.

(b) Check for deflection.

Permissible $= \frac{\text{Span}}{250} = \frac{6000}{250} = 24$ mm.

$$\delta = \frac{5}{384}\frac{wl^4}{EI} + \frac{wl^3}{48EI}$$

$$= \frac{5}{384} \times \frac{11 \times 6000^4 \times 10^3}{2 \times 10^5 \times 3.505 \times 10^8} + \frac{100 \times 10^3 \times 6000}{48 \times 2 \times 10^5 \times 35057.6 \times 10^4}$$

$\delta = 9.06 < 24$mm Safe.

(c) Check for Shear. → 59 pg NO

(f)

$$V_d = 0.6\left[\frac{f_y}{\sqrt{3}\times \lambda_{mo}} \times A_v\right] > V_u.$$

$$= 0.6\left[\frac{250}{\sqrt{3}\times 1.1} \times 450 \times 9.2\right] = 325.94\ KN > V_u \quad \text{Safe.}$$

So

(d) Check for Moment of Resistance:

Section classification based on table 2 @ page 18.

$$\left(\frac{b}{t_f}\right) = \frac{200/2}{15.4} = 6.49 < 9.4.$$

$$\left(\frac{d}{t_w}\right) = \frac{(h-2t_f)}{t_w} = \frac{450-2\times15.4}{9.2} = 45.56 < 84.$$

Hence the Section is Plastic $\therefore \beta = 1$

9) A

$$M_d = \frac{\beta_b Z_p f_y}{\lambda_{mo}} > M_u.$$

$$= \frac{1\times1760.59\times10^3\times250}{1.1}$$

$$= 400.13\ KN\text{-}m > M_u = 299.2 \quad \text{SAFE}$$

But $M_d < 1.2 \frac{Z_e f_y}{\lambda_{mo}}$.

$$= \frac{1.2\times1558.1\times10^3\times250}{1.1}$$

$M_d < 424.93\ KN\text{-}m$ SAFE.

(e) Check for ~~deflection~~ web crippling:

$$F_w = (b_1+n_2)t_w \frac{f_{yw}}{\lambda_{mo}}$$

Assume $b_1 =$ ~~200~~ 230mm

$$= (\overset{230}{\cancel{200}} + 70.5)9.2 \times \frac{250}{1.1}$$

$n_2 = 2.5\,(t_f + r_1)$

$= 2.5\,(14.2 + 14)$

$= 70.5\,mm$

$F_w \qquad > V_u$

Hence safe.

Pro

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(f) Check for web Buckling:

$F_{wb} = (b_1 + n_1)\, t_w\, f_c$

$=$

$\lambda = 2.5 \frac{d}{t_w}$

$= 2.5 \times \frac{(h - 2t_f)}{t_w}$

$= 2.5 \frac{(450 - 2 \times 15.4}{9.2}$

$=$

So find fcd from table 9c for $\lambda =$

$\therefore f_{cd} =$

$F_{wd} = (200 + \quad) 9.2 \times$

$= \quad > V_u$

$n_1 = h/2 = 450/2 =$

Hence safe.

$\therefore$ The Section Decided is ISWB 450 @ 79.4 kg/m. is safe to take loads coming on 6m Span Beam.

Problem

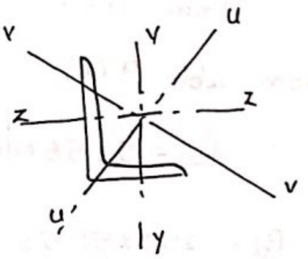
Type I

Q1. Determine the compressive strength of angle strut ISA 100X65 X 8mm with length 3mts when connected by

(i) with single bolt. (ii) More than two bolts, (iii) welded connection.

Take $f_y = 250$ Mpa, $l = 3m$.

(i) Angle with single bolt:



$l_e = l = 3m = 3000mm$

From steel table ISA 100X65X8mm.

c/s area = $12.57cm^2$

Radius of gyration.

$r_{xx} = 3.16cm$ $\quad r_u = 3.38 cm$

$r_{yy} = 1.83cm$ $\quad r_v = 1.39cm.$

$\therefore r_{min} = 13.9mm$

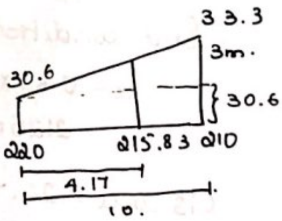
$\therefore \lambda = \frac{l_e}{r_{min}} = \frac{3000}{13.9} = 215.831$

According to table (10) Pag 44. of IS 800:2007

Single Angle → Buckling class = C

∴ Refer table 9(c) from pg-42, IS 800:2007.

by interpolation.



$\frac{x}{4.17} = \frac{3}{10}$.

$f_{cd} = 31.85 N/mm^2$

∴ Design strength = $P_d = A_e f_{cd}$

$= 1257 \times 31.85$

$P_d = 39.87$ KN

(ii) "More bolts" are used:

$l_e = 0.85 l$

$= 0.85 \times 3$

$l_e = 2550mm$

$\lambda = \frac{l_e}{r_{min}} = \frac{2550}{13.9} = 183.45$

from table 9(c) — $f_{cd} = 42.5 N/mm^2$

$P_d = A_e f_{cd}$

$= 1257 \times 42.5$

$P_d = 53.10$ KN

(iii) **Welded Connection:**

$l_e = 0.7 l$

$= 0.7 \times 3000$

$l_e = 2100\ mm$

$\lambda = \frac{l_e}{r_{min}} = \frac{2100}{13.9} = 151.07$

from table 9 (c)

$\therefore f_{cd} = 58.56\ N/mm^2$

$P_d = 1257 \times 58.56$

$P_d = 73.61\ kN$

Q.3 Design a compression member using double channel section "Face to Face" to carry a factored load of 1600 KN. The length of column is 5m. with one end fixed & one end hinged. Also design "single lacing system"

Design of column

$l_e = KL$

$= 0.8 \times 5000$

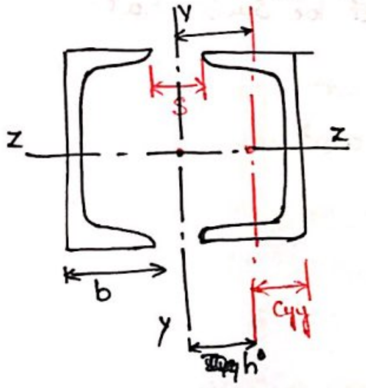
$l_e = 4000mm$

Assume $f_{cd} = 200 N/mm^2$

$\text{Area} = \frac{\text{load}}{\text{Stress}} = \frac{1600 \times 10^3}{200} = 8000mm^2 = 80.0cm^2$

area of one channel $= \frac{80}{2} = 40cm^2$

From steel table Try 2ISLC -300 @ 33.1 kg/m



Properties of one channel

area = 4211 mm^2

$I_{xx} = 6047.9 \times 10^4 mm^4$

$I_{yy} = 346.0 \times 10^4 mm^4$

$C_{yy} = 25.5mm$

b = 100 mm

To make any structure safe $I_{xx} = I_{yy}$.

$\therefore I_{xx} = I_{xx} + ah^2$

$= 2\left[6047.9 \times 10^4 + 4211 \times 0^2\right]$

$I_{xx} = 120.958 \times 10^6 mm^4$

$(a+b)^2$

$I_{yy} = 2\left[346.0 \times 10^4 + 4211 \times \left((b - c_{yy}) + \frac{S}{2}\right)^2\right]$

$= 2\left[346.0 \times 10^4 + 4211 \times \left[(100 - 25.5) + \frac{S}{2}\right]^2\right]$

$\therefore$ on equating $I_{xx} = I_{yy}$

$\therefore \quad 120.958 \times 10^6 = 2\left[346.0 \times 10^4 + 4211 \times 100 - 25.5 + \frac{S}{2}\right]$

$S = 83.73 \simeq 80mm.$

$I_{min} = 120.95 \times 10^4 \ mm^4.$

$$r_{min} = \sqrt{\frac{I_{min}}{A.}} = \sqrt{\frac{120.95 \times 10^4}{2 \times 4211}} = \sqrt{\frac{120.95 \times 10^4}{2 \times 4211}}$$

$r_{min} = 119.84mm.$

∴ from table 9(c) = Buckling class (c)

$f_{cd} = 206.61 \ N/mm^2$

Compression load = $P_d = A_e f_{cd}$

$= 2 \times 4211 \times 206.61$

$= 1740.0 \times 10^3 N > 1600 \times 10^3 N.$

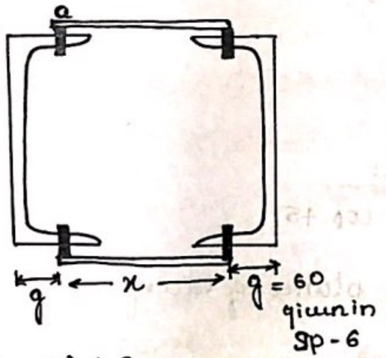
Hence SAFE

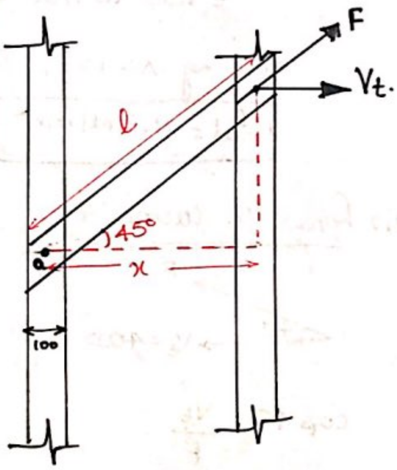
★ Design of lacing: Pg NO - 48, 49, 50.

(i) Transverse shear = 2.5% of column load. → 7.6.6.1

$V_t = \frac{2.5}{100} \times 1600 = 40KN.$

(ii) lacing inclination = $\theta = 45^\circ$ → 7.6.4





$x = 2(b - g) + S$

$= 2(100 - 60) + 80$

$= 160mm$

$\cos 45^\circ = \frac{x}{l} = \frac{160}{l}$

$∴ \ l = \frac{160}{\cos 45^\circ} = 226.27mm$

∴ length of lacing = 226.27 mm

for single lacing = $l_e = l = 226.27$ mm

for Double lacing = $l_e = 0.7L = 0.7 \times 226.2$

(iv) Lacing dimension: (b & t) [Clause no. 7.6.2]

- Width = 3 × dia of bolt.

  = 3×18. (Assuming 18 mm d)

  b = 54 mm ≅ 55 mm.

- Thickness: $t = \frac{l_e}{40}$ → for single lacing. [Clause no: 7.6.3]

  $t = \frac{l_e}{60}$ → for double lacing.

∴ for single lacing. $t = \frac{l_e}{40} = \frac{226.7}{40} = 5.66 \simeq 6$ mm.

∴ **Lacing b×t = 55mm × 6mm**

(v) Check for slenderness ratio: [Clause no. 7.6.6.3]

$\lambda = \frac{l_e\sqrt{12}}{t} \not> 145$

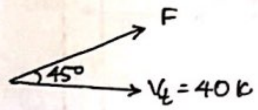
$= \frac{226.7\sqrt{12}}{6}$

= 130 is not greater than 145 → Safe.

Using λ = 130, calculating from table 9c

$f_{cd} = 71.3$ N/mm²

(vi) Forces in lacing:



$\cos 45 = \frac{V_t}{F}$

∴ $F = \frac{V_t}{n \cdot \cos 45}$

n = no. of planes of lacing

$F = \frac{40\times10^3}{2\times\cos 45°}$

F = 28.28 kN

(vii) Check for strength:

- Compression strength = Area × Stress

  $= b \times t \times f_{cd}$

  $= 55 \times 6 \times 71.3$

  $= 23.5 \times 10^3$ N.

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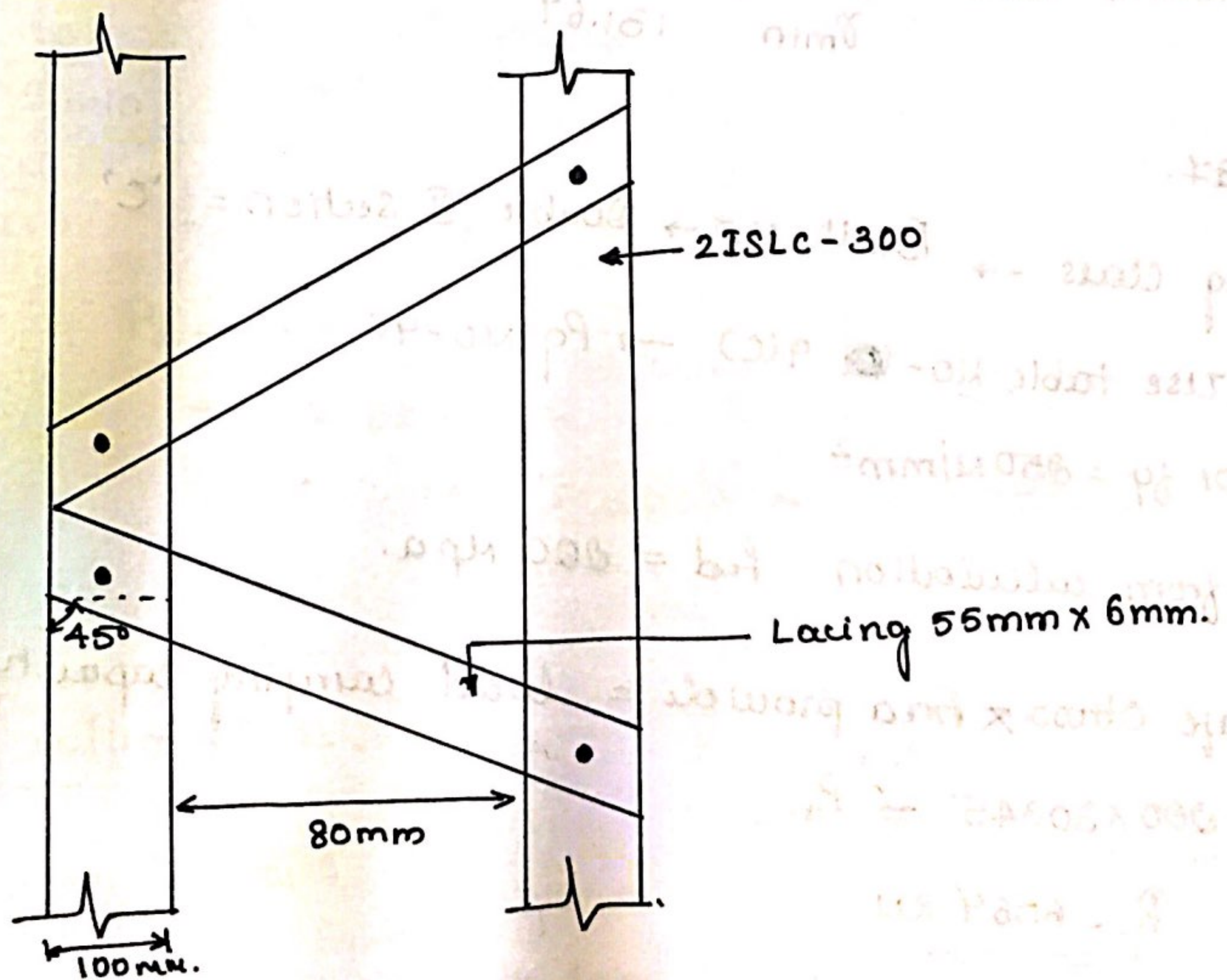
- Tensile Strength $= 0.9(b - d_0)t \times f_{cd}$

$$= (55 - 20)\,6 \times 71.3$$

$$= 14.97 \times 10^3\ N.$$

- Provide single bolt @ each end.



Q. Design a "column splice" for ISHB 350 @ 72.4 kg/m subjected to 600 kN load, 50 kN-m moment and 150 kN horizontal shear. Use M20 property class 5.6 bolts. Design both flange splice & web splice.

ISHB - 350 @ 72.4 kg/m

$h = 350$

$b = 250$

$t_f = 11.6$

$t_w = 10.1$

load = 600 kN }
Moment = 50 kN-m } → Flange splice

Shear = 150 kN → web splice

(a) Design of "flange splice"

$$\text{Area} = \frac{\text{load}}{\text{Stress}}$$

* Assume column faces are machined (or) milled (or) grinded.

∴ load on flange $= \frac{600}{2} = 300\text{ kN}$.

load on each flange $= \frac{300}{2} = 150\text{ kN}$.

Convert Moment into additional load $= \frac{M}{h}$

$$= \frac{50}{0.35} = 142.86\text{ kN}$$

Total load on flange splice = 292.86 kN.

$$\text{Area} = b \times t = \frac{292.86 \times 10^3}{250/1.1} = 1288.58\text{ mm}^2$$

Provide splice width = flange width = 250 mm

$$\therefore t = \frac{1288.58}{250} = 5.15\text{ mm} \simeq 6\text{ mm}$$

∴ **Flange splice → 250 mm x 6 mm.**

(b) Design of "web splice"

Horizontal Shear = 150 kN.

$$\text{Area} = \frac{\text{load}}{\text{stress}} = \frac{150\times10^3}{250/1.1} = 660\,mm^2$$

using 6mm thick plate

$$\therefore b = \frac{660}{6} = 110\,mm.$$

web splice → 110 × 6 mm.

(c) Connection

M_{20} – grade 5.6.

(i) $$V_{dsb} = \frac{\frac{500}{\sqrt{3}}\left(1\times0.78\times\pi/4\times20^2\right)}{1.25} = 56.6$$

(ii) $$V_{dpb} = \frac{2.5\times0.507\times20\times6\times410}{1.25}$$

e = 40mm
p = 50mm

$V_{dpb} = 49.88$ kN.

∴ No. of bolts for flange splice $= \frac{292.86}{49.88} = \boxed{6}$

∴ No. of bolts for web splice $= \frac{150}{49.88} = \boxed{3}$