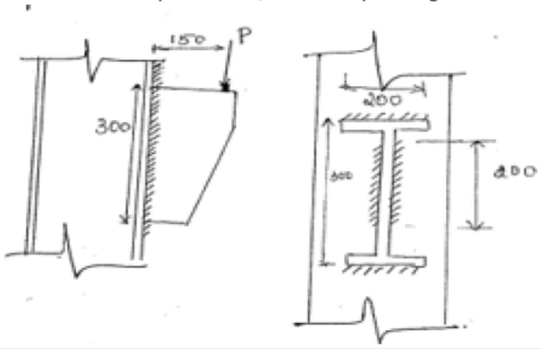
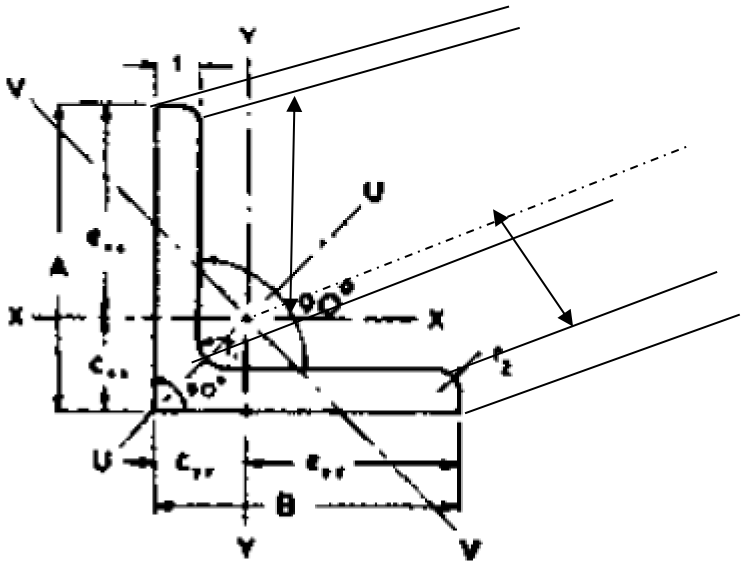
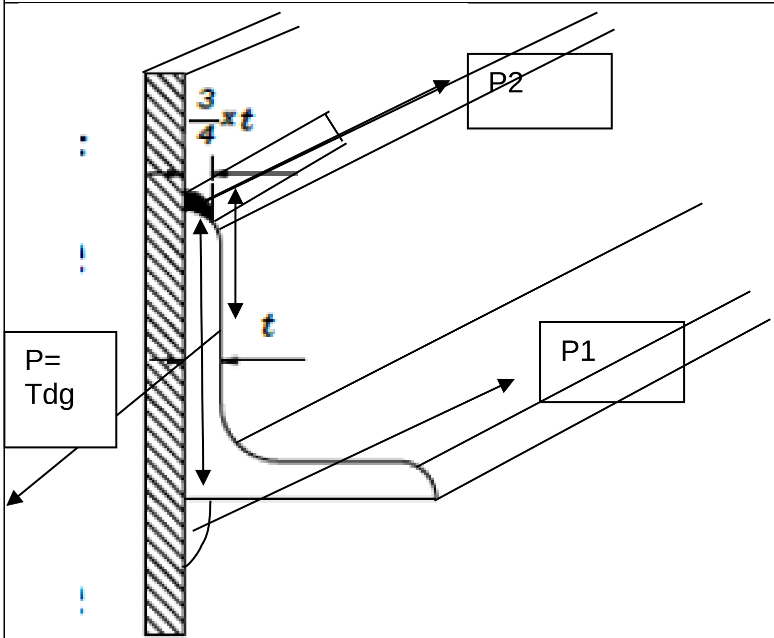
CMR Institute of Technology, Bangalore
DEPARTMENT OF CIVIL ENGINEERING
II - INTERNAL ASSESSMENT

Semester: 6-CBCS 2018
Subject: DESIGN OF STEEL STRUCTURAL ELEMENTS (18CV61)
Faculty: Ms Vibha N Dalawai

Date: 22 Jun 2021
Time: 09:00 AM - 10:30 AM
Max Marks: 50

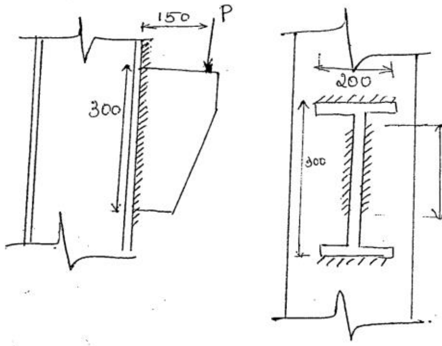
Answer All Questions

Q.No		Marks	CO	PO	BT/CL
1	A tie member of a truss consisting of an angle section ISA 90 x 90 x 6 of Fe 410 grade is welded to an 8 mm gusset plate. Design a weld to transmit a load equal to the full strength of the member. Assume shop welding	10	CO2,CO4	PO2,PO3	L5
2	A bracket of I section is welded to a a steel column by flange weld of 12mm and web weld of 6mm as shown in fig. determine the safe load carried by connection, Assume shop welding 	10	CO2	PO2,PO4	L5
3	An angle section ISA 90 x 90 x 6 mm is used as a tension member with its longer leg connected 12mm dia bolts. calculate the strength, Assuming p=30mm and e= 25mm	15	CO4	PO2,PO3,PO4	L5
4	A bracket plate 12mm thick is to be bolted to the flange of column ISHB 350@710.2 N/m by means of close tolerance and turned bolts. M20 bolts of grade 4.6 are arranged in two vertical rows 100mm apart at a pitch of 70mm. Design a bracket connection if the bracket plate carries a load 120kN at a lever arm of 250mm	15	CO2	PO2,PO3,PO4	L5

Scheme and solution	
1. A tie member of a truss consisting of an angle section ISA 90 x 90 x 6 of Fe 410 grade is welded to an 8 mm gusset plate. Design a weld to transmit a load equal to the full strength of the member. Assume shop welding.	
SP 6	
Properties of ISA 90 x 90 x 6	
A = 10.47 cm^2 = 1047 mm^2 = Ag	
Cxx = 2.42 cm =24.2 mm, ezz = 65.8 mm	
	
Cl: 6.2, P- 32	
Tension capacity of the angle= $T_{dg} = \frac{A_g \times f_y}{\lambda_{mo}} = \frac{1047\times250}{1.1}$ = 237.95kN …………………..2 marks	
	

Size of the weld, D= ¾*t = ¾*6= 4.5mm say 4mm which is greater than 3mm	
Strength of bottom weld = p1= $0.707*D*l_1*\frac{410}{\sqrt{3}*1.25}$	2 Marks
$P1=0.707*4*l_1*\frac{410}{\sqrt{3}*1.25}$	
$P1=535.54l_1$ N……………………………	
Strength of top weld = p2= $0.707*D*l_2*\frac{410}{\sqrt{3}*1.25}$	2 Marks
$P2= 0.707*4*l_1*\frac{410}{\sqrt{3}*1.25}$	
$P2= 535.54l_2$ ………………………………………………..	
P=P1+P2	
$237.954*10^3= 535.54l_1+535.54l_2$	2 Marks
Distributing weld in such a way that c.g of the weld coincides with that of the angle section.	
Taking the moment wrt to one of the force , wrt P2	
P1*90 = P*65.8	
$535.54* l_1*90= 237.95* 65.8$	
$l_1= 237.95*65.8/535.54*90$	
$l_1= 324.84mm = 325mm$	
on substituting the l_1 in P1 ………………………………………………	
$P1= 535.54*325= 174.050*10^3$ N	2 Marks
P1+P2= P	
$174.05*10^3+P2 = 237.95*10^3$	
$P2= 63.9*10^3$ N	
Wkt $535.54l_2$= P2	
Therefore = $l_2= 63.9*10^3/535.54$ =119.31mm	

9) A bracket of I-Section is welded to a steel column by using flange weld of 12mm and web weld of 6mm as shown in fig. Determine the safe load carried by connection, Assume shop weld.



Size of fillet weld.

Provide normal fillet weld joint.

Thickness of web weld = 6mm.

Thickness of flange weld = 12mm.

Throat thickness of flange weld $= 0.7 \times 12 = 8.4$ mm.

Throat thickness of web weld $= 0.7 \times 6 = 4.2$ mm.

Total throat thickness of weld group $= 2 \times 200 \times 8.4 + 2 \times 200 \times 4.2$
$= 5040 \text{ mm}^2$

Stress in weld $= \dfrac{f_u}{\sqrt{3} \times \gamma_{mw}} = 189.37 \text{ N/mm}^2$

Maximum Shear stress q_a in flange weld.

$$q_a = \frac{P \times 1000}{5040} = 0.198P \text{ N/mm}^2$$

$$I_{xx} = 2 \times 200 \times 8.4 \times 150^2 + \left(\frac{2 \times 4.2 \times 200^3}{12}\right)$$

$= 81200000 \text{ mm}^4$.

$M = P \times e = P \times 150 =$ kN-mm.

Actual Bending stress in the flange

$$q_b = \frac{M \times y}{I_{xx}} = \frac{P \times 150 \times 1000 \times 150}{81200000} = 0.2770P \text{ Mpa}.$$

$q = \sqrt{q_a^2 + q_b^2} = \sqrt{(P \times 0.918)^2 + (0.2770P)^2}$

$q = 0.9592\,P$

Equating this to stress in weld.

$0.9592\,P = 189.37$

$P = 197.42$ kN.

Safe load $= \dfrac{P}{1.25} = 157.93$ kN

3) An angle section ISA 90×90×6mm is used as tension member with its longer leg connected 12mm dia bolts. calculate the strength, Assuming p=30mm and e= 25mm

- Strength shear (*assume fully threaded bolts*)

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

$$= \frac{400}{\sqrt{3}}(1 * 0.78 * pi * 12 * 12/4 + 0)\ /1.25 = 16.28\text{ kN}$$

- Bolts in Bearing

$V_{nsb} = (2.5 K_b\, d\, t\, f_u)/\gamma_{mb}$

K_b *is least of the following* e/3d$_o$, p/3d$_o$- 0.25 , f_{ub}/f_u , 1

Assuming p=30mm and e= 25mm

25/ 3*13= 0.64

30/3*13-0.25= 0.52

400/410= 0.97

1

V_{nsb} = (2.5*0.52*12*6*400)/1.25*1000= 29.95kN

No. of bolts = load /bolt value = 237.95/16.28= 14.61= 15no's

1. **Design Yielding Strength T_{dg}- (6.2)**

$T_{dg} = f_y \times A_g / \gamma_{m0}$ (γ_{m0} = 1.10 from table 5)

= 250×1047/1.1*1000= 237.95 kN

A_g = Gross cross-sectional area 1047 mm^{2}steel table pg=12

2. **Design Rupture Strength of Net Area T_{dn}- (6.3.3)** *since it is an* ***angle (since it is affected by shear lag)***

$$\mathbf{T_{dn} = 0.9 \times A_{nc} \times f_u / \gamma_{m1} + \beta \times A_{go} \times f_y / \gamma_{m0}}$$

A_{nc} = ((50-6/2)*6)-(13*6)=204 mm^2

A_{go} = Gross c/s area of the **unconnected** leg= (30-6/2)*6= 162mm^2

$$\beta = 1.4 - 0.076\left(\frac{W}{t}\right)\left(\frac{f_y}{f_u}\right)\left(\frac{b_s}{L_c}\right) \le \left(\frac{f_u}{f_y}\right)\left(\frac{\gamma m0}{\gamma m1}\right) \ge 0.7$$ ……….pg 33

=1.4-0.076*5*0.609*0.144

$b_s = w + w_1 - t$ = 90+50-6 =134mm

L_c= 30*14= 420mm

$$\beta = 1.4 - 0.076\left(\frac{30}{6}\right)\left(\frac{250}{410}\right)\left(\frac{134}{420}\right) \le \left(\frac{410}{250}\right)\left(\frac{1.1}{1.25}\right) \ge 0.7$$

=1.17 ≤1.44≥0.7

T_{dn} = 0.9 ×204 × 410 / 1.25 +1.17×162×250 / 1.1

=60.22+50.44=269.84kN

3. **Design Block Shear Strength T_{db} (6.4.1)**

$T_{db1} = A_{vg} \times (f_y/\sqrt{3}) / \gamma_{m0} + 0.9 \times A_{tn} \times f_u / \gamma_{m1}$……………………………pg 33, 6.4.1

=2670 × (250/√3) / 1.1 + 0.9 × 93×410/ 1.25

=264.05+27.45

$\underline{= 409.68\ kN}$	
T_{db2} $= 0.9\times A_{vn}\times (f_u/\sqrt{3}) / \lambda_{m1} + A_{tg} \times f_y / \lambda_{m0}$.................................pg 33, 6.4.1	
$=0.9\times 1539\times (410/\sqrt{3}) / 1.25 +132 \times 250/ 1.1$	
$=189.9+30$	
$\underline{=316.84\ kN}$	
$A_{vg}= L_v*t=(25+14*30)*6=2670\ mm^2$	
$A_{tn}= (L_t\text{-}nd_o)*t=(40\text{-}0.5*13)6=201mm^2$	
$A_{vn}= (L_v\text{-}nd_o)*t=(445\text{-}14.5*13)*6=1539\ mm^2$	
$A_{tg}= L_t*t =40*6= 240mm^2$	
DESIGN STRENGTH= least of T_{dg}, T_{dn}, T_{db1}, *and* T_{db2}	
Therefore, Design strength of the angle is 110.66 kN	
4) A bracket plate 12mm thick is to be bolted to the flange of column ISHB 350@710.2 N/m by means of close tolerance and turned bolts. M20 bolts of grade 4.6 are arranged in two vertical rows 100mm apart at a pitch of 70mm. Design a bracket connection if the bracket plate carries a load 120kN at a lever arm of 250mm	
From SP-6 Properties of ISHB 350= Weight=72.4 kg	
C/S area= 92.21cm^2	
depth of section= 350mm = width of web	
Width of flange = 250mm	
Thickness of flange = 11.6 mm	
No. of bolts calculation	
No. of bolts for regular connection = Load/ bolt value (B_v)	
Bolt value is the least of Shear and Bearing strength	
No. of bolts for bracket connection= $\sqrt{\frac{6M}{p\ m\ B_v}}$	
Assume dia of bolt as 20mm	
1. Design shear strength of the bolt:	
a. $V_{dsb} = V_{nsb}/ \gamma_{mf}$	
b. $V_{dsb} = \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb} + n_s A_{sb}) / \gamma_{mb}$ = $\frac{400}{\sqrt{3}}$(1*0.78*pi*20*20/4)/1.25 = 45.26kN	
As given in IS 800:2007, Pg no 75	
2. Design bearing strength of plate	
$V_{nsb} = (2.5K_b\ d\ t\ f_u)/ \gamma_{mb}$	
<u>K_b *is least of*</u>	
e/3d$_o$,	
p/3d$_o$- 0.25,	
fub/fu,	
1	
from the IS 800:2007 pg no 75	

Assumed e=1.7*d_0= 1.7*22= 37.5mm= 40mm, p= 70mm, d_o= 20+2=22mm	
40/ 3*22=0.61	
70/3*22-0.25= 0.81	
800/410= 1.95	
1	
V_{dpb} = (2.5*0.61*20*11.6*410*10^{-3})/ 1.25 =116.04kN	
B_v= bolt value = 45.26	
No of bolting rows given in question= m= 2	
M= ultimate moment = Ultimate load * eccentricity = (1.5*120)*250 = 45000kN-m	
1. No of bolts for bracket connection= $\sqrt{\frac{6M}{p\ m\ B_v}} = \sqrt{\frac{6*45000}{70*2*45.26}}$ = 6.85= 7no's	
$\sum r^2 = r_1^2 + r_2^2 + r_3^2$...............	
$\sum r^2$ =4(50^2+210^2) + 4(50^2+140^2) +4(50^2+70^2) + 2(50^2+0^2) = 309400mm^2	
Force on the extreme bolts F= Magnitude of Resultant force = $\sqrt{F_1^2 + F_2^2 + 2F_1F_2cos\theta}$ = $\sqrt{25.71^2 + 31.39^2 + 2 * 25.71 * 39.24cos76.6}$ = should be always less than bolt value = 44.95kN hence safe	
Direction of the force on the extreme bolt = θ= $\tan^{-1}\frac{210}{50} = 76.6$	
F_1= force due to direct load = $\frac{180}{7} = 25.71kN$	
F_2= force due to Moment acting =$\frac{M*r_n}{\sum r^2} = \frac{45000*215.87}{309400}$ = 31.39kN	
$r_n = distance\ between\ cg\ of\ the\ section\ to\ the\ center\ of\ the\ extreme\ bolt$	
$r_n = \sqrt{50^2 + 210^2}$ = 215.87mm	

Scheme of IAT 2		
Topis	**Solution**	**Marks**
Tension capacity of the angle	237.95kN	2
Size of the weld	4.5	1
Strength of bottom weld = p1	535.54l1 N	1.5
Strength of TOP weld = p2	535.54l2	1.5
Length of bottom weld	325mm	2
Length of top weld	119.31mm	2
Q1		10
Throat thickness of web	4.2	
total area of throat thickness	5040	1
Stress in the weld	189.4	2
Maximum shear stress in the flange weld , for the load P	0.198 P	2
Actual bending stress in the flange	0.277 P	2
Safe load	445.5kN	2
Q2		10
Strength shear (assume fully threaded bolts)	16.28kN	2
Bolts in Bearing	29.95 kN	2

No. of bolts	15	1
Design Yielding Strength Tdg-	237.95kN	2
Design Rupture Strength of Net Area Tdn- since it is an angle (since it is affected by shear lag	110.66 kN	2
Design Block Shear Strength Tdb1	377.80 kN	2
Design Block Shear Strength Tdb2	292.29 kN	2
Design strength of the angle	110.66 kN	2
Q3		
Strength shear (assume fully threaded bolts)	45.26 kN	2
Bolts in Bearing	116.04kN	2
No. of bolts	7	2
F1= force due to direct load	25.71 kN	2
F2= force due to Moment acting	31.39kN	2
Force on the extreme bolts F	44.95 kN	5