Scheme Of Evaluation
Internal Assessment Test 1 – March 2019

Sub:	Design of steel structures							**Code:**	15CV62
Date:	15/04/2019	Duration:	90mins	Max Marks:	50	**Sem:**	6	**Branch:**	CIVIL

Note: Answer ALL TWO Questions

Question		**Description**
1	a)	Explain the different forms of tension members commonly used in steel structures:
	b)	Design single unequal angle tension members to carry a factored load of 300 kN. The length of the member is 3.0m. The tension member is connected to a gusset plate of 16mm thick with one line of M20 bolts of grade 4.6. Take pitch=60mm and edge distance=40mm. Use steel of Fe 410.

1 a) Explain the different forms of tension members commonly used in steel structures:

- Wires and Cables. The wire **type tension members** are used for derricks, hoists, hangers for suspension bridges, rigging slings, and guy wires.
- Bars and Rods. The round and square bars are frequently used for the small **tension members**. ...
- Single Structural Plates and Shapes. ...
- Built-up Sections.

1 b) Design single unequal angle tension members to carry a factored load of 300 kN. The length of the member is 3.0m. The tension member is connected to a gusset plate of 16mm thick with one line of M20 bolts of grade 4.6. Take pitch=60mm and edge distance=40mm. Use steel of Fe 410.

Q.2. Given Single unequal Angle

Load = 300KN
l = 3.0m
t = 16mm
M_{20} = of 4.6 = d = 20,
f_{ub} = 400 Mpa.
f_{yb} = 250 Mpa.
f_u = 410 Mpa.
e = 40mm
p = 60mm.

$\lambda = \frac{l_{eff}}{r_{min}}$

$180 = \frac{3000}{r_{min}}$

r_{min} = 16.67mm. = 1.687cm.

let us take section ~~150×75×~~ 125×95×12

A_g = 24.98 cm² = 2498 mm²

Required A_g = , $T_{dg} = \frac{A_g f_y}{\lambda_{m0}}$, $\frac{300 \times 1.1}{250} = A_g = 1320 mm^2$

∴ Section 125×95×12 is fine. Since slenderness ratio satisfies.

Design strength due to rupture:

$T_{dn} = \frac{0.9 A_{nc} f_u}{\lambda_{m1}} + \beta \frac{A_{go} f_y}{\lambda_{m0}} = \frac{0.9 \times 1164 \times 410}{1.25} + \frac{0.88 \times 1068 \times 250}{1.1} = 537.79$ KN.

$A_{nc} = [125 - 22 - 12/2] \times 12 = 1164 mm^2$

$A_{go} = [95 - 12/2] \times 12 = 1068 mm^2$

$\beta = 1.4 - 0.076 \left(\frac{95}{12}\right)\left(\frac{410}{250}\right)\left(\frac{186}{360}\right) = 0.88$

★ No of bolts = $\frac{300}{\text{Bolt value}} = \frac{300}{45.27}$ = 7no

∴ Bolt value = $V_{dsb} = \frac{400}{\sqrt{3}}(01 \times 0.78 \times \pi/4 \times 20^2)/1.25$

= 45.27 ~~Mpa~~ KN

$V_{dpb} = 2.5 \times K_b \times 20 \times 12 \times 410 / 1.25$.

= 131.85 ~~Mpa~~ KN.

$K_b = \frac{40}{3 \times 20} = 0.67$ ✓

$= \frac{60}{3 \times 20} - 0.25 = 0.75$

$\frac{400}{410} = 0.97$

1 =

$T_{db} = \left[\frac{A_{vg} f_y}{\sqrt{3} \lambda_{m0}} + \frac{0.9 A_{tn} f_u}{\lambda_{m1}}\right]$

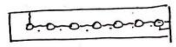
$= \left[\frac{4800 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 468 \times 410}{1.25}\right]$

= 767.99 KN

$T_{db_2} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3} \lambda_{m1}} + \frac{A_{tg} f_y}{\lambda_{m0}}\right]$

$= \left[\frac{0.9 \times 3084 \times 410}{\sqrt{3} \times 1.25} + \frac{600 \times 250}{1.1}\right]$

= 661.98 KN



A_{vg} = 01 [400×12] = 4800 mm²

A_{vn} = 4800 − 6.5×22×12 = 3084 mm²

A_{tg} = 50×12 = 600 mm²

A_{tn} = 600 − 0.5×22×12 = 468 mm²

∴ Design strength of tension member = 537 KN

2 a) Describe briefly the advantages and disadvantages of welded connections.

Advantages of welding

- ✓ The weight of the joint is minimum.
- ✓ In tension members - absence of holes improves the efficiency of the section.
- ✓ It involves less fabrication cost
  - ✓ Drilling, punching etc.
  - ✓ Ideal for oil storage tanks, ships
- ✓ Neat appearance and enable the connection of any shape
- ✓ More rigid compared to structures with riveted and bolted connections.
  stronger than the base metal

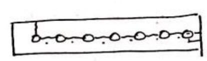
Disadvantages of welding

- ➢ It requires skilled manpower for welding as well as inspection.
- ➢ Also, non-destructive evaluation may have to be carried out to detect defects in welds.
- ➢ Welding in the field may be difficult due to the location or environment.
- ➢ Welded joints are highly prone to cracking under fatigue loading.
- ➢ Large residual stresses and distortion are developed in welded connections.

b A tension members consists of 2ISA 100×75×8 carries a factored tensile load of 300kN. The angles are connected to a 10mm thick gusset plate with longer leg placed back to back on the either side of gusset plate. Design the joint assuming shop welding.

$$T_{db} = \left[\frac{A_{vg} f_y}{\sqrt{3}\,\gamma_{m0}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}\right] = \left[\frac{4800 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 468 \times 410}{1.25}\right] = 767.99\ KN$$

$$T_{db_2} = \left[\frac{0.9 A_{vn} f_u}{\sqrt{3}\,\gamma_{m1}} + \frac{A_{tg} f_y}{\gamma_{m0}}\right] = \left[\frac{0.9 \times 3084 \times 410}{\sqrt{3} \times 1.25} + \frac{600 \times 250}{1.1}\right] = 661.98\ KN$$



$A_{vg} = 01[400 \times 12] = 4800\ mm^2$

$A_{vn} = 4800 - 6.5 \times 22 \times 12 = 3084\ mm^2$

$A_{tg} = 50 \times 12 = 600\ mm^2$

$A_{tn} = 600 - 0.5 \times 22 \times 12 = 468\ mm^2$

∴ Design strength of tension member = 537 KN

Q. 2(b) Max size of weld = $\frac{3}{4} \times 8 = 6\ mm$.

∴ Strength of weld @ bottom $[P_2] = 0.707 \times D \times l_2 \times \frac{f_u}{\sqrt{3}\,\gamma_{mw}}$

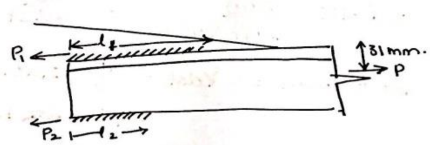
$= 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.5} \times l_2$

$= 670\, l_2\ N/m$

∴ Strength of weld @ top $[P_1] = 0.707 \times 6 \times \frac{410}{\sqrt{3} \times 1.5} \times l_1$

$= 670\, l_1\ N/m$.

$P_1 + P_2 = P$.



Taking Moment w.r.t. P_1,

$P_2 \times 75 = P \times 31$

$670\, l_2 \times 75 = 300 \times 31 \times 10^3$

$l_2 = 185\ mm$.

$670\, l_1 + 670 \times 185 = 300 \times 10^3$

$l_1 = 262.76\ mm$

3 a) Explain the failure modes in tension members with neat sketch

Tension yielding :

- This failure mode looks at yielding on the gross cross sectional area, A_g.
- Consequently, the critical area is located away from the connection as shown.
- Strength of the section = the *gross area*, A_g, times the *minimum yield stress*, F_y, of the member.

Tensile rupture

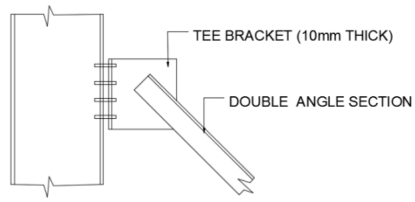
- In this case we have two potential failure paths that see the full force of the member.
- It is common to have multiple potential failure paths.

- Tensile rupture is complicated by the need to get the forces out of the flanges, through the web, and into the bolts.
- This means that we need to account for the stress concentrated in and around the bolts.
- The capacity of each failure path = the *effective net area*, A_n, times the *tensile stress*, F_u, of the member

Block shear occurs when a "block" of the member is "torn" out.

- Block shear is characterized by a failure that includes both tension (i.e. normal to the force) and shear (i.e. parallel to the force) failure planes.
- Like tensile rupture, there are frequently multiple valid failure paths that must be investigated.
- Each tension area capacity = the tension area (either gross or net) times a tensile stress (yield or ultimate).
- Each shear area capacity = the shear area (either gross or net) times a shear stress (yield or ultimate).

b) Determine whether the joint shown in the fig. 3.b is safe or not. 8-16 dia bolts of 4.6 grades used to make a connection. Neglect the action of prying. Also find the number of 16mm dia bolts of grade 4.6 to connect the double angle section (8mm thick each) member with web of Tee bracket.



Data: $d = 16mm$

$d_0 = 18mm$

grade = 4.6

$f_{ub} = 400$ Mpa.

$f_u = 410$ Mpa.

Factored tensile force: $T = 200$ KN.

→ Horizontal component $= T_h = 200\cos 45° = 141.42$ KN.

Tension in each bolt $= T_t = \frac{141.42}{8} = 17.68$ KN.

→ Vertical component: $T_v = 200\sin 45° = 141.42$ KN.

Shear in bolt $= \frac{141.42}{8} = 17.68$ KN.

Strength of bolt in single shear: (V_{dsb})

$$= \frac{f_{ub}}{\sqrt{3}}(n_n A_{nb})/\gamma_{mb} = \frac{\frac{400}{\sqrt{3}}(1 \times 0.78 \times \pi/4 \times 16^2)}{1.25} = 29.0 \text{ KN}.$$

		Strength of bolt in tension. $T_{db} = \frac{T_{nb}}{\gamma_{mb}}$. $T_{nb} = 0.9 f_{ub} A_{nb} \not> f_{yb}\left(\frac{\gamma_{mb}}{\gamma_{m0}}\right)$ $= 0.9 \times 410 \times 0.78 \times \frac{\pi}{4} \times 16^2 \not> (400 \times 0.6)\left(\frac{1.25}{1.10}\right)$ $= 56.52 \not> 54.82$ ✓ $\therefore$ $T_{nb} = \frac{54.82}{1.25} = 43.85$ kN. Check $\left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 \leq 1$ $= \left(\frac{17.68}{29}\right)^2 + \left(\frac{17.68}{43.85}\right)^2 \leq 1$ $0.53 \leq 1$ Hence the joint @ the section 1-1 is safe. The member is composed of double angle section with each leg of 8mm thickness. The angles are placed on opposite side of web of T-bracket. The bolt will be in double shear $\therefore$ Strength of bolt in double shear. $V_{dsb} = 2 \times A_{nb} \times \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}} = 2 \times 157 \times \frac{400}{\sqrt{3} \times 1.25} = 58.0$ kN ✓ (lesser of 10mm & 2×8mm) $V_{dpb} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}} = 2.5 \times 0.67 \times 16 \times 10 \times \frac{410}{1.25} = 87.90$ kN. $\therefore$ Strength of bolt = 58.0 kN no. of bolts required = $\frac{200}{58.0} = 3.44 \simeq 4$ no. $\therefore$ provide 16mm ϕ bolts of 4 no.