DETAILS EXPLANATIONS

CE : Paper-1 (Paper-3) [Full Syllabus]

[PART : A]

1. Euler's buckling load

$$P_{cr} = \frac{\pi^2 EI}{L_{eff}^2}$$

For one end fixed other end hinged

$$L_{eff} = 0.7\ L$$

$$P_{cr} = \frac{\pi^2 EI}{(0.7L)^2}$$

2. Unit Expansion

$$\frac{\Delta L}{L} = \frac{P}{AE} = \frac{70 \times 10^3}{100 \times 2 \times 10^5} = 3.5 \times 10^{-3}$$

3. Slendereness Ratio

$$\lambda = \frac{L}{r}$$

$$r = \frac{D}{4}$$

$$\because \quad \frac{L}{D} = \frac{180}{4} = 45$$

4. The centre point deflection of fixed beam carrying central load is one fourth of the central point deflection of simply supported carrying point load.

$$\delta = \frac{1}{4} \times \left(\frac{wL^3}{48EI}\right) = \frac{wL^3}{192EI}$$

5. Torsion formula is given as :

$$\frac{T}{J} = \frac{G\theta}{L} = \frac{\tau}{r}$$

$\Rightarrow$ Power = Torque × Rotational Speed = Tω

When diameter is doubled, the torque capacity becomes 8 times the orginal so power also increases 8 times.

6. In bone-dry all internal pores are dry but in Air dry samples some pores are dry and water is not sticking to the surface.

7. It is the strength gained by concrete with minimum water cement ratio with the help of admixtures.

8. Permissible punching stress in footing :

$$\left(\tau_{vp}\right)_{per} = 0.16K_s\sqrt{f_{ck}} \qquad \leftarrow \text{WSM}$$

$$\left(\tau_{vp}\right)_{per} = 0.25K_s\sqrt{f_{ck}} \qquad \leftarrow \text{LSM}$$

9. (i) Stress concept method

(ii) Strength concept method

(iii) Load balancing method.

10. ***Edge Distance :***

It is the distance between the edge of a member cover plate and the centre of the nearest rivet hole.

11. The loads to be considered in design are self weight, live-load, impact load, snow-load, earthquake load, etc. but the codal provisions are to be considered.

12. In order to resist large forces, built up compression members are necessary. Connection between the elements is provided by lacing, battens or perforated plates.

13. The maximum permissible slenderness ratio = 180

$$\lambda = \frac{l}{r} = 180$$

$$\frac{l}{20} = 180$$

$\therefore$ Unbraced/unsupported length

$$l = 180 \times 20 = 3600 \text{ mm} = 3.6 \text{ m}$$

14. Effective flange area in tension

$$A = A_F + \frac{A_w}{6} = 4500 + \frac{5000}{6}$$

$$4500 + 833.33 = 5333.33 \text{ mm}^2$$

15. Slope of ILD $= \frac{1}{L} = \frac{1}{10} = 0.1$

16. The compressibility of a soil will determine how much compression shall take place in that soil upon loading.

17. (i) Visual examination
(ii) Dilatancy test
(iii) Toughness test
(iv) Dry strength test
(v) Organic content and colour
(vi) Other identification tests

18. Net resultant pressure at depth Z :

$$p_z = \gamma z(K_P - K_A)$$
γ = Unit weight
K_p, K_A = Passive and active earth pressure coefficient.

19. A coffer dam is a temporary structure constructed usually in a river, lake etc. to keep the working area dry for construction of other structures.

20. $$\text{Void Ratio} = \frac{\text{Volume of voids}}{\text{Volume of Solids}}$$

$$e = \frac{7+8}{10} = \frac{15}{10} = 1.5$$

[PART : B]

21. There are four elastic-constants :

E = Young's modulus of elasticity.
G = Modulus of Rigidity.
K = Bulk modulus
μ = Possion's Ratio

The relation between these constants are :

(i) $G = \dfrac{E}{2(1+\mu)}$ (ii) $K = \dfrac{E}{3(1-2\mu)}$

(iii) $E = \dfrac{9KG}{3K+G}$ (iv) $\mu = \dfrac{3K-2G}{6K+2G}$

22. The elongation in the bar due to increase in temperature = $L \propto \Delta T$

Yield of support reduces strain in the bar by δ

$$\therefore \quad \frac{\sigma L}{E} = L \propto \Delta T - \delta$$

$$\sigma = E\left(\propto \Delta T - \frac{\delta}{L}\right) = 1\times10^6 \times\left(20\times10^{-6}\times100 - \frac{0.01}{25}\right)$$

$$\sigma = 1600 \text{ kg/cm}^2$$

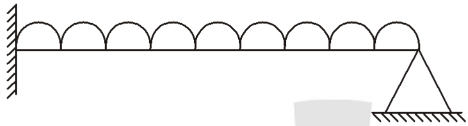
23. Normal strain $\in_n = 1.25$ mm/meter

Lateral strain $\in_L = -\mu \cdot \in_n = -0.3 \times 1.25 = -0.375$ mm/m

Volumeteric Strain :

$$\in_v = \in_1 + \in_2 + \in_3$$
$$= 1.25 - 0.375 - 0.375 = 0.5 \text{ mm/meter}$$
$$= \frac{0.5}{1000} = 5 \times 10^{-4}$$

24. (i)



Internal indeterminacy = 0

External indeterminancy

$$E = R - r$$
$$E = R - r$$
$$R = 2 + 1 = 3$$
$$r = 2$$
$$E = 3 - 2 = 1$$

Total indeterminacy = I + E = 1

(ii)

Internal indeterminancy I = 0

External indeterminancy E = R – r = (3 + 3) – 3 = 3

Total Indeterminancy T = I + E = 3

25.

By applying compatibility condition at B

$$\frac{Rl^3}{3EI} = \frac{Ml^2}{2EI}$$
$$R = \frac{3M}{2l}$$

At joint A, $\Sigma M = 0$

$$M_A + M - Rl = 0$$
$$M_A = \frac{M}{2}$$

26. For an under Reinforced section :

(i) $x_u < x_{u\ lim}$ or x_c

(ii) Steel reaches to ultimate/permissible strain/stress before concrete reaching the same.

(iii) $M_u < M_{u\ lim}$

(iv) Lever arm, = (d – 0.42 x_u) will be more as compared to the lever arm of a balanced section.

27. The limiting principal tensile strength/sress in an uncracked prestressed concrete member is given by

$$f_t = 0.24\sqrt{f_c}$$

For M25 grade concrete

$$f_t = 0.24\sqrt{25} = 1.2 \text{ MPa}$$

28. $$(\text{Factor of Safety})_A = \frac{\text{Yield Stress}}{\text{Allowable Stress}} = 1.5$$

But in this case, allowable stress is increased by 20%.

$$\text{Then factor of safety} = \frac{\text{Yield Stress}}{1.20 \times \text{allowable Stress}}$$

$$(\text{FOS})_B = \frac{1.5}{1.2} = 1.25$$

Load factor = Shape factor × Factor of safety

$$= 1.12 \times (\text{FOS})_B = 1.12 \times 1.25$$

Load-factor = 1.40

29. $$\text{Plastic section modulus } Z_P = \frac{BD}{2}\left(\frac{D}{4}+\frac{D}{4}\right)$$

$$Z_P = \frac{BD^2}{4} = \frac{300 \times 600^2}{4} = 27 \times 10^6 \text{ mm}^3$$

30.

Clay Mineral	*Property*
• Kaolinite	Hydrogen bond → Strongest Bond eg. china clay.
• Illite	Ionic-bond → medium change due to moisture change.
• Mont-morillonite	Water Bond → Weakest Bond Maximum change in volume due to moisture change. eg. → Black Soils, Bentonite Soils.

31. Time factor

$$T_v = \frac{C_v . t}{d^2}$$

$$\frac{C_v}{T_v} = \frac{d^2}{t} = \frac{(0.02)^2}{30} \times 60 = 0.0008 \text{ m}^2/\text{hr}$$

For sandwiching between sand layers $d = \frac{H}{2} = \frac{4.8}{2} = 2.4$ m

$$T_v = \frac{C_v t}{d^2}$$

$$t = \frac{T_v}{C_v} . d^2 = \frac{1}{0.008} \times 2.4^2 = 7200 \text{ hr} = 300 \text{ days}$$

32. The permeability of soil $K = 100(D_{10})^2$ in cm/sec

$$K = 100 \times \left(\frac{0.1}{10}\right)^2 = 0.01 \text{ cm/s} = 10^{-4} \text{ m/sec}$$

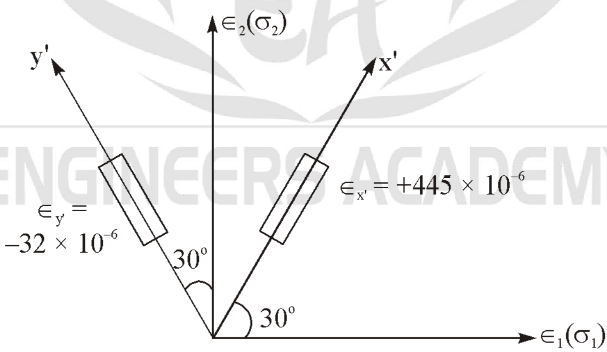
The discharge per unit length is

$$q = KH\, N_F/N_D = 10^{-4} \times 3 \times \frac{1}{2}$$

$$= 1.5 \times 10^{-4} \text{ m}^3/\text{sec/m length}$$

[PART : C]

33. $\in_{x'} = \frac{\in_1 + \in_2}{2} + \frac{\in_1 - \in_2}{2} \cos 2\theta$



$$\Rightarrow 445 \times 10^{-6} = \frac{\in_1 + \in_2}{2} + \frac{\in_1 - \in_2}{2} \cos(2 \times 30^o)$$

$$\Rightarrow \frac{\in_1 + \in_2}{2} + \frac{\in_1 - \in_2}{4} = 445 \times 10^{-6}$$

$$\Rightarrow 3\in_1 + \in_2 = 1780 \times 10^{-6} \quad ...(1)$$

Now we know that,

$$\in_{x'} + \in_{y'} = \in_1 + \in_2$$

$$\Rightarrow \in_1 + \in_2 = (+445 \times 10^{-6}) - (32 \times 10^{-6})$$

$$\Rightarrow \in_1 + \in_2 = 413 \times 10^{-6} \quad ...(2)$$

Solving equation (1) and (2)

$$\in_1 = 683.5 \times 10^{-6}$$

$$\in_2 = -270.5 \times 10^{-6}$$

Now we know that,

$$\sigma_1 = \frac{E}{1-\mu^2}(\in_1 + \mu \in_2)$$

and $$\sigma_2 = \frac{E}{1-\mu^2}(\in_2 + \mu \in_1)$$

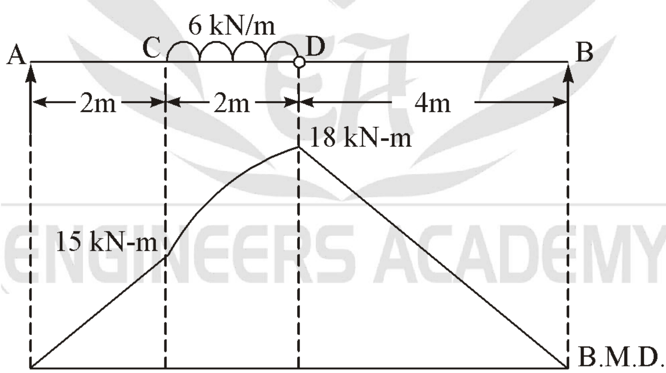
$$\sigma_1 = \frac{2.1\times10^5}{1-(0.3)^2}[683.5\times10^{-6} - 0.3\times270.5\times10^{-6}]$$

$$= 139 \text{ N/mm}^2$$

Thus $$\sigma_2 = \frac{2.1\times10^5}{1-(0.3)^2}[(-270.5\times10^{-6}) + (0.3\times683.5\times10^{-6})]$$

$$= -15.10 \text{ N/mm}^2$$

34. For bending moment diagram :



$$\Sigma M_A = 0$$

$$R_B \cdot 8 = 6 \times 2 \times 3$$

$$R_B = \frac{36}{8} = 4.5 \text{ kN}$$

$$R_A + R_B = (6 \times 2)$$

$$R_A = 12 - 4.5 = 7.5 \text{ kN}$$

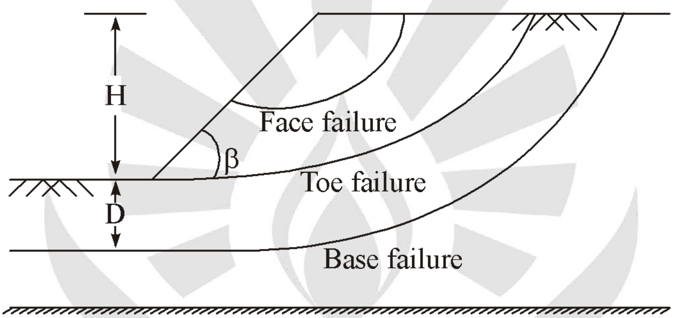
For deflection at mid span by moment area method,

$$\delta_D + \delta_B = \text{Area of } \frac{M}{EI} \text{ diagram}$$

$$\delta_D + \delta_B = \frac{\left(\frac{1}{2}\times 18\times 4\right)\times 10^6\times 10^3\times\left(\frac{2}{3}\times 4\times 10^3\right)}{2\times 10^5\times 8000\times 10^4}$$

$$\delta_D = 12 \text{ mm}$$

35. If the slope is of the finite extent, bounded by top and bottom surface then it is termed as finite slope. Failure of finite slope takes place due to rotation and failure surface is either circular or spiral.



Finite slope may have either of the following modes of failure.

(i) Slope failure :

(a) Face failure

Failure surface passes through slope above the toe. This type of failure takes place in case of steep slope, when soil mass near the toe is rigid and strongtr in comparison to soil mass above the tee.

(b) Toe failure

It is the most common mode of failure of finite slope. In which failure surface passes through toe this failure also occur in case of steep slopes.

When the soil mass is homogenous above and below the toe

(ii) Base Failure

Failure surface passes below the toe. This type of failure takes place in flat slopes when soil mass below the toe is soft and weak in comparison to soil mass above the toe.

If (H + D) is total depth of failure surface and H is height of slope then,

$$\text{depth-Factor } (D_f) = \frac{H+D}{H}$$

If $D_F > 1 \rightarrow$ base failure.

If $D_F = 1 \rightarrow$ toe failure.

Note: *For slope angle $\beta > 53^o$, Critical slip-circle passes through the toe (toe failure). This is true for any angle of friction(θ^o).*

The stability of finite slope is analysed in two stages :

(i) Immediately after construction

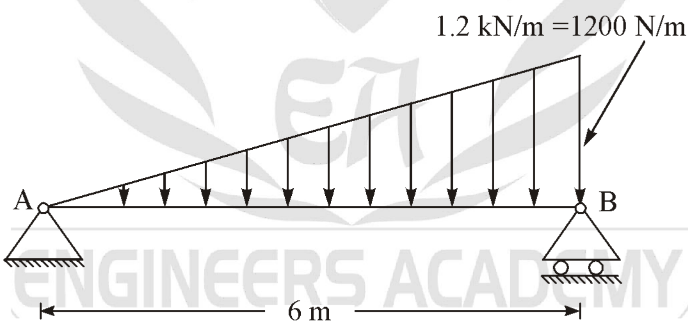
(ii) Long time after construction

For sand, effective stress analysis (Drained) is prefered for both stages.

For clay, total stress analysis (Undrained) is preferred immediately after construction and effective stress analysis (drained) is preferred long time after construction.

36. Load Intensity at support B = 1200 N/m

EI = 420 × 10^6 N-mm^2 = 420 N-m^2



Let R_A and R_B be the reactions at support A and B.

$$R_A + R_B = \frac{1}{2}\times 6\times 1200 = 3600 \text{ N}$$

$$\Sigma M_B = 0$$

$$R_A\cdot 6 - \frac{1}{2} \times 1200 \times 6 \times \left(\frac{1}{3}\times 6\right) = 0$$

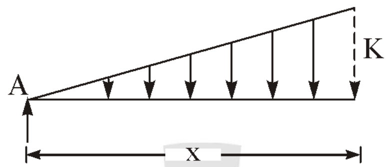
$$R_A = \frac{1200\times 6}{6} = 1200 \text{ N}$$

$$\therefore \quad R_B = 3600 - 1200 = 2400 \text{ N}$$

Consider a section at a distance 'x' from 'A' from similar triangle property.

$$\frac{1200}{6} = \frac{k}{x}$$

$\Rightarrow$ $K = 200\,x$



$$M_x = EI\frac{d^2y}{dx^2}$$

or, $EI\dfrac{d^2y}{dx^2} = R_A \cdot x - \dfrac{1}{2}\times(x)\cdot(200x)\left(\dfrac{x}{3}\right)$

$$= 1200x - 33.33x^3$$

On integration,

$$EI\frac{dy}{dx} = \frac{1200x^2}{2} - \frac{33.33x^4}{4} + C_1$$

Integrating again :

$$EI\cdot y = \frac{1200x^3}{6} - \frac{33.33x^5}{4\times 5} + C_1x + C_2$$

at $x = 0,\ y = 0;\ 0 = 0 - 0 + 0 + C_2$

$C_2 = 0$

at $x = 6;\ y = 0$

$\Rightarrow$ $0 = 200 \times 6^3 - \dfrac{33.33\times 6^5}{20} + (C_1\times 6)$

$C_1 = -5048.64$

at $x = 3,\ y = ?$

$EIy = 200 \times 3^3 - 1.66 \times 3^5 - 5048.6 \times 3$

$$y = \frac{5400 - 403.38 - 15145.92}{420} = -24.16 \text{ m}$$

Deflection at centre = 24.16 (downward)

37. In order to get the maximum advantage of a pre-stressed concrete member, it is necessary to use both high strength concrete and high tensile steel wires. High strength concrete is necessary for the following resous.

(i) Since large prestressing forces are applied to the member by tendons, high bearing stresses are developed at the ends by anchering devices.

(ii) Bursting stresses liable to develop at the ends of the beam can not be satisfactorily resisted by low strength concrete.

(iii) When a stress transfer to concrete has to take place by bond action, the concrete should have a high bond stress which can be offered only by high strength concrete.

(iv) Shrinkage cracks will be very little when high strength concrete is used.

(v) Due to high modulus of elasticity of high strength concrete, the elastic and creep strains are very small resulting in smalles loss of prestress in steel reinforcement.

(v) By using high strength concrete the cross-sectional areas required for members will be reduced resulting in considerable reduction of dead-load.

The mild steel used in ordinary reinforced concrete has a yield point of 200 MPa to 300 MPa. If such steel is used and if even it is subjected to a stress say 200 MPA at the stage of tensioning, we find that due to creep and shrinkage of concrete the net tensile stress left over will be extremely low. In the design of a prestress concrete member, the estimated loss of prestress due to shrinkage and creep on concrete and steel is of the order of nearly 200 MPa. But high tension steel has an ultimate strength of 2100 MPa.

The advantage of prestressed concrete over reinforced concrete are :

(i) In a RCC-beam, the concrete in the compression side of the neutral axis alone is effective. The concrete in the tension side of the neutral side is ineffective, but in the prestressed concrete beam, the entire section is effective.

(ii) RCC sections are generally heavy but prestressed concrete members are lighter.

(iii) Prestressed concrete beams are suitable for heavy loads and long spans because for same load and span RCC beams are more heavy than PSC beams.

(iv) Cracks do not occur under working loads in PSC members. Even if minute cracks occur when overloaded, such cracks get closed when the overload is removed.

(v) In RCC beams, there is no way of testing the steel and concrete. In PSC members, testing of steel and concrete can be done while prestressing.

38. Calculation of load : (Section Size = 300 × 400)

Self weight = 0.3 × 0.4 × 1 × 25 = 3 kN/m

Live-load = 24 kN/m

Total load = 27 kN/m = w

Span = 5 m

$$\text{Bending moment} = \frac{wl^2}{8} = \frac{27 \times 5^2}{8} = 84.375 \text{ kN-m}$$

∵ Ultimate bending-moment

$(BM_u) = 1.5\ M$

$(BM)_u = 1.5 \times 84.375 = 126.5625$ kN-m

For balanced section,

Limiting moment of resistance,

$M_{u\ lim} = 0.36 f_{ck}\ B\chi_{u\ lim}(d - 0.42\chi_{u\ lim})$

$M_{u\ lim} = 0.36 \times 20 \times 300 \times (0.48 \times 350)$

$[350 - (0.42 \times 0.48 \times 350)] = 101352384$ N-mm

$M_{u\ lim} = 101.35$ kN-m

Since $(BM)_u > M_{ulim}$

So, the section is over reinforced

⇒ Design doubly reinformced beam

$$A_{st\ 1} = \frac{M_1 \equiv (M_{u\,lim})}{0.87 f_y (d - 0.42 x_{u\,lim})}$$

$$A_{st\ 1} = \frac{101.35 \times 10^6}{0.87 \times 415 \times (350 - 0.42 \times 0.48 \times 350)}$$

$A_{st\ 1} = 1005.04\ mm^2$

Remaining Bending – Moment

$M_2 = BM_u - M_1$

$M_2 = 126.5625 - 101.3523$

$M_2 = 25.21$ kN-m

$M_2 = 0.87 f_y A_{st\ 2}(d - d')$

$$A_{st\ 2} = \frac{25.21\times10^6}{0.87\times415\times(350-40)} = 225.24\ mm^2$$

Let effective cover to compression reinforcement = 40 mm.

$$A_{st} = A_{st\ 1} + A_{st\ 2} = 1005.04 + 225.24$$

$$A_{st} = 1229.28$$

$$A_{sc} = \frac{M_2}{(f_{sc}-0.45f_{ck})(d-d')}$$

$$A_{sc} = \frac{25.21\times10^6}{(0.87\times415-0.45\times20)\times(350-40)} = 230.99\ mm^2$$

Provide 4 - 20 mm dia bars in tension

and 3 - 12 mm dia bars in compression.

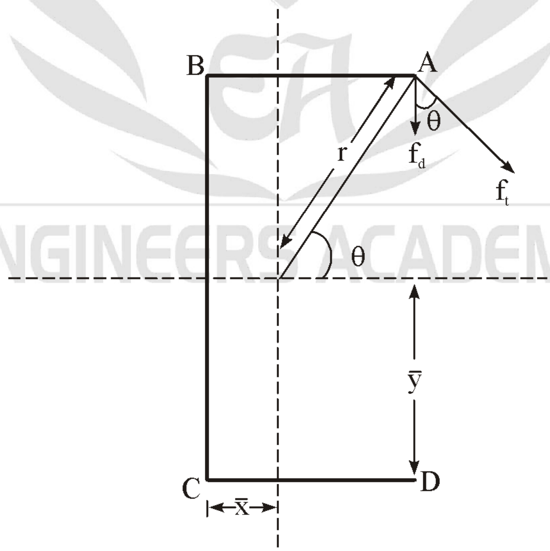
39. Let 't' be the throat thickness and 's' be size of the weld.

Shear stress due to direct load = $\frac{\text{Direct Load Area of}}{\text{Weld resisting direct load}}$

$$f_d = \frac{100\times1000}{(2\times150+250)t} = \frac{181.82}{t}$$

Shear stress due to torsional moment :

$$f_t = \frac{T}{J}\cdot y = \frac{(P.e)r}{J}$$



$$\bar{x} = \frac{2[(150\times t)\times75]}{2\times150\times t+250t} = 40.91\ mm$$

$$\bar{y} = \frac{(150\times t)\times 250+(250\times t)\times 125}{(2\times 150\times t)+250t} = 125 \text{ mm}$$

If θ is the angle between f_d and f_t, then

$$\tan\theta = \frac{150-\bar{x}}{125} = \frac{150-40.91}{125} = 0.873$$

If r is the distance of A from C.G of weld then

$$r = \sqrt{(125)^2+(150-40.91)^2} = 165.91 \text{ mm}$$

$$\cos\theta = \frac{150-40.91}{165.91} = 0.6575$$

$$e = 150 + (150 - 40.91) = 259.09 \text{ mm}$$

$$J = I_{xx} + I_{yy}$$

$$I_{xx} = \frac{t\times(250)^3}{12} + 2\left[\frac{150\times t^3}{12}+\left(150\times t\times 125^2\right)\right]$$

[t^3 term is negligible as compared to other terms]

$$= \frac{t\times 250^3}{12}+\left\{\left(2\times 150\times t\right)\times(125)^2\right\}$$

$$= 5.99t \times 10^6 \text{ mm}$$

$$I_{yy} = \left[\frac{250t^3}{12}+250\times t(40.91)^2\right]+2$$

$$\left[\frac{t\times 150^3}{12}+150\times t\times(75-40.91)^2\right]$$

$$I_{yy} = 1.33\ t \times 10^6 \text{ mm}^4$$

$$f_t = \frac{P.e.r}{I_{xx}+I_{yy}} = \frac{100\times 10^3\times 259.09\times 165.91}{(5.99t\times 10^6)+(1.33t\times 10^6)}$$

$$f_t = \frac{587.23}{t}$$

The resultant stress is given by

$$f_r = \sqrt{f_d^2 + f_t^2 + 2f_d.f_t \cos\theta}$$

$$f_r = \sqrt{\left(\frac{181.82}{t}\right)^2 + \left(\frac{587.23}{t}\right)^2 + 2\left(\frac{181.82}{t}\right)\left(\frac{587.23}{t}\right) \times 0.6575}$$

$$f_r = \frac{719.93}{t}$$

For safety of the connection

$$f_r \leq 110$$

$$\Rightarrow \frac{719.93}{t} \leq 110$$

$$t \geq \frac{719.93}{110} \geq 6.54 \text{ mm}$$

$\therefore$ Size of weld

$$s = \sqrt{2} \times t = \sqrt{2} \times 6.54 = 9.25 \text{ mm}$$

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