DETAILS EXPLANATIONS

[PART : A]

1. It is the strength below which not more than 5% test results are expected to fall.

2. For steels which do not show yield stress clearly, the 0.2% strain is taken as offset and corresponding stress is proof-stress.

3. Fe 250 - 0.53 d

Fe 415 - 0.48 d

Fe 500 - 0.46 d

Where d - effective depth of the section

4. Shear span is defined as the span where shear force is constant throughout the span.

5. Minimum shear reinforcement is provided to arrest the longitudinal cracks on side faces due to shrinkage and temperature variation.

6. (i) Pure Adhesion

(ii) Friction between concrete and Steel

(iii) Grade of concrete

(iv) Mechanical Resistance of slip

7. $$A_{SV} = \frac{T_u S_v}{b_1 d_1 (0.87 f_y)} + \frac{V_u S_v}{2.5 d_1 (0.87 f_y)}$$

8. Its the minimum of 3 times the effective depth of slab and 300 mm.

9. Pedestal is the member for which the ratio of length to least lateral dimension is less than 3.

10. The control of deflection of beam/member can be done by limiting 'span/depth' ratio.

11. Side face Reinforcement is provided when:

(i) Depth of web is greater than 750 mm.

(ii) Depth of web is greater than 450 mm and is subjected to torsion.

12. Shearing strength of joint is simply the sum of shearing strength of individual rivets.

13. The 'lug-angle' is a short length of an angle section used at a joint to connect the outstanding leg of a member by reducing the length of joint.

14. Battens are normally used for axially loaded columns and where the components are not far apart.

15. Thickness $t = 10\sqrt{\dfrac{90W}{16\sigma_{bs}} \times \dfrac{B}{(B-d_0)}}$

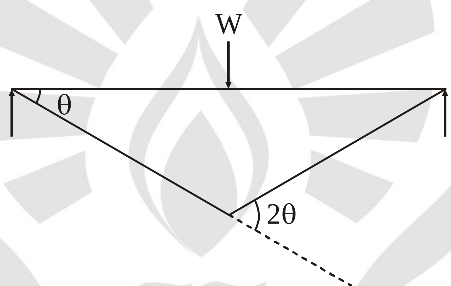
16. A steel beam is designed to resist maximum bending moment and is checked for shear stress and deflection, and also for web-crippling and web buckling.

17. Load bearing stiffners are used at the point of concentrated loads and at supports.

18. The load factor is the ratio of collapse load to service load.

19. Reserve-strength is the ratio of ultimate load 'W_u' to the load at first yield W_y of the structure.

20.



Internal Work = External Work

$$W \cdot \theta \cdot \frac{L}{2} = M_P 2\theta$$

$$W_u = \frac{4M_P}{L}$$

[PART : B]

21. **Grades of Cement :**

Grades of cement is based on crushing strength of a cement mortar cube of size 70.71 mm (surface area 50 cm^2) cured and tested at 28 days. They basically differ in terms of fineness of cement which in turn is expressed as specific surface area.

Specific surface is the surface area of the particles in 1 gram of cement (unit cm^2/gm). Chemically all the three grades of cement viz grade 33, 43 and grade 53 are almost similar.

22. Balanced-Section

The stresses in concrete and steel reach to maximum values at the same time.

Concrete and steel theoritically fail at the same time.

Practically, it is difficult to have a balanced section.

The failure is based on primary-compression.

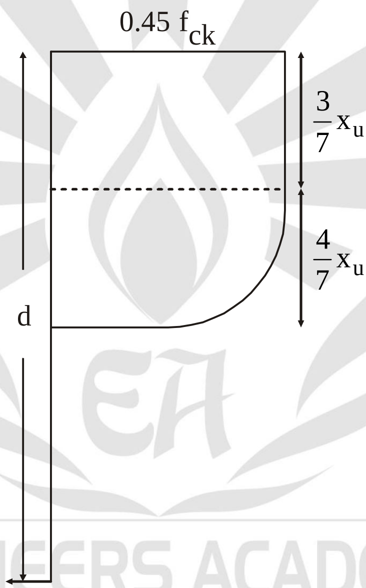
$$x_u = x_{u_{lim}}$$

23. M.R. = C × Lever-Arm = T × Lever Arm

Compressive - force

$$= \left\{\left(0.45f_{ck} \times \frac{3}{7}x_u\right) + \left(\frac{2}{3} \times 0.45f_{ck} \times \frac{4}{7}x_u\right)\right\}B$$

$C = 0.36\ f_{ck}x_uB$



Lever Arm = $(d - \bar{y})$

where

$$\bar{y} = \frac{\left(0.45f_{ck} \times \frac{3}{7}x_u \times \frac{3}{14}x_u\right) + \left\{\left(\frac{2}{3} \times 0.45f_{ck} \times \frac{4}{7}x_u\right)\left(\frac{3}{7}x_u + \frac{3}{8} \times \frac{4}{7}x_u\right)\right\}}{\left(0.45f_{ck} \times \frac{3}{7}x_u\right) + \left(\frac{2}{3} \times 0.45f_{ck} \times \frac{4}{7}x_u\right)}$$

$\bar{y} = 0.42\ x_u$

L.A. = $d - 0.42\ x_u$

$MR_{lim} = 0.36\ f_{ck}Bx_{u_{lim}}(d - 0.42\ x_u)_{lim}$

24. ***Strength of column with helical reinforcement***

$$P_u = 1.05(0.40f_{ck}A_c + 0.67f_yA_{sc})$$

where A_c = Area of concrete = $A_g - A_{sc}$

A_{sc} = Area of steel

A_g = Gross Area

The above equation gives the strength if it satisfies the following condition:

$$\frac{V_h}{V_c} \nless 0.36\left(\frac{A_g}{A_c}-1\right)\frac{f_{ck}}{f_y}$$

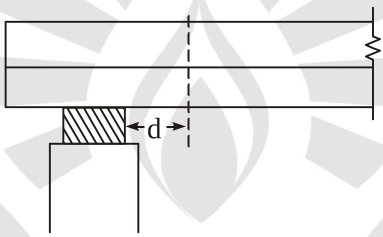
where V_h = Volume of helical reinforcement

V_c = Volume of core

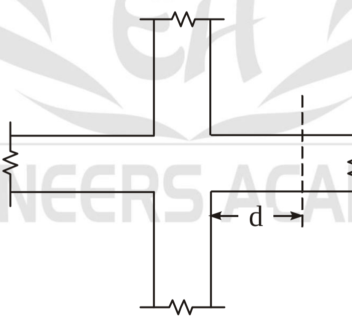
A_g = Gross Area

25. A section which is most prone to be failed in shear is called the critical section for shear.

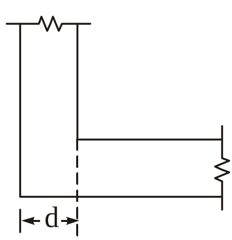
(i) For a simply supported beam



(ii) For Intermediate Beam



(iii) For water tanks etc.



26. For $C = T$

$$0.36 f_{ck} B x_{u\ lim} = 0.87\ f_y A_{st}$$

$$\Rightarrow 0.36 \times 25 \times B \times 0.48d = 0.87 \times 415 \times A_{st}$$

$$\Rightarrow \frac{A_{st}}{B.d} \times 100\% = \frac{0.36 \times 25 \times 0.48}{0.87 \times 415} \times 100\%$$

$$\Rightarrow \quad p_t = 1.196\%$$

$\because$ For Balanced section

$$x_{u\ lim} = k.d$$

$$x_{u\ lim} = 0.48d$$

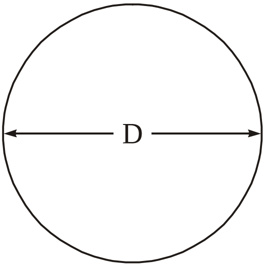
27. ***As per IS : 456 : 2000***

- Minimum diameter of bar should be 12 mm.
- Minimum number of bars in rectangular column = 4 and for Circular column = 6.
- Minimum Reinforcement = 0.8% of gross Area.
- Maximum reinforcement = 4% (when overlapping is done) and 6% (when no overlapping)
- Maximum spacing between the main bars = 300 mm.

28. ***General Requirements of lacings:***

- Radius of gyration about the axis perpendicular to the plane of lacing and radius of gyration in the plane of lacing.
- The lacing system should not be varied through the length of strut as far as practicable.
- The single-laced system on opposite sides on the main components should preferably in the same direction so that one can be the shadow of the other.

29. $$\text{Shape-factor} = \frac{\text{Plastic Section Modulus}}{\text{Elastic Section Modulus}}$$



$$\text{Shape-factor} = \frac{Z_p}{Z_e}$$

$$\text{Plastic-modulus } (Z_p) = \frac{A}{2}\left(\bar{y}_1 + \bar{y}_2\right)$$

$$Z_P = \frac{\pi}{8} D^2 \left(\frac{2D}{3\pi} + \frac{2D}{3\pi}\right)$$

$$Z_P = \frac{D^3}{6}$$

Elastic Section modulus

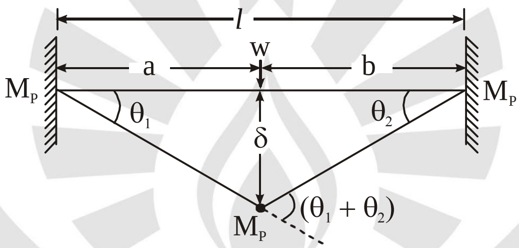
$$Z_e = \frac{\pi}{32}D^3$$

$\therefore$ Shape-factor

$$\frac{Z_p}{Z_e} = \frac{(D^3/6)}{(\pi D^3/32)}$$

$$\text{Shape-factor} = \frac{32}{6\pi} = 1.7$$

30. By external-work = Internal Work



$$M_p\theta_1 + M_p(\theta_1 + \theta_2) + M_p\theta_2 = w.\delta$$

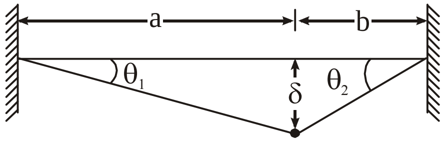
$$2M_p(\theta_1 + \theta_2) = w.\delta$$

$$2M_P\left(\frac{b}{a}\theta_2 + \theta_2\right) = w.b\theta_2$$

$$2M_P\theta_2\left(\frac{b+a}{a}\right) = w.b\theta_2$$

$$\text{Collapse load } (w_c) = \frac{2M_P l}{ab}$$

$\because$



$$\delta = b\theta_2 = a\theta_1$$

$$\theta_1 = \frac{b}{a}\theta_2$$

$$l = a + b$$

31. ***Black Bolts :***

(i) Hexagonal black bolts are generally used in steel works.

(ii) They are made from low or medium carbon steels.

(iii) They are designated as black bolts M × d × l

Where, d = diameter and l = length of bolt.

Precision and Semi Precision Bolts :

(i) They are also known as close tolerance bolts.

(ii) Sometimes to prevent excessive slip, close tolerance bolts are provided in holes of 0.15 to 0.2 mm oversize this may cause difficulty in alignment and delay in the progress of work.

32. In the welded connection the load is taken by weld through it's shear strength.

Shear strength of the weld

$$P_s = p_s l t$$

where, p_s = Permissible shear strength

$= 108 \simeq 110 \text{ N/mm}^2$

l = Length of the weld

t = Throat thickness

In the case where load is eccentric, the resultant force is compared with the shear strength.

[PART : C]

33. The maximum shear force

$$V_u = 1.5 \times 40 \times \frac{4.5}{2} = 135 \text{ kN}$$

$$\text{Shear Stress} = \tau_v = \frac{135 \times 10^3}{230 \times 560} = 1.08 \text{ N/mm}^2$$

For grade M20 concrete, maximum shear stress

$$\tau_{c_{max}} = 2.8 \text{ N/mm}^2$$

$$\tau_v < \tau_{c_{max}}$$

Now, Since the beam is normal Tee-Beam i.e., the beam ends are confined by compressive-reaction. The critical section for shear is at distance 'd' from the face of the support.

Critical shear

$$V_u = V_{max} - d.w$$

Critical shear force;

$$V_u = 135 - (0.56 \times 60) = 101.34 \text{ kN}$$

Nominal shear stress

$$\tau_v = \frac{V_u}{b.d} = \frac{101.34 \times 10^3}{230 \times 560}$$

$$= 0.787 \text{ N/mm}^2$$

Design Shear Strength of the Section (τ_c)

$$p_t = \frac{A_{st}}{B.d} \times 100\%$$

$$p_t = \frac{4 \times (\pi/4) \times 20^2}{230 \times 560} \times 100\% = 0.975$$

P_t	$\tau_c (N/mm^2)$ M20
0.75	0.56
1.00	0.62

By Interpolation;

$$\therefore \quad \tau_c = \frac{0.62 - 0.56}{1.00 - 0.75}(0.975 - 0.75) + 0.56$$

$$\tau_c = 0.605 \text{ N/mm}^2$$

$$\because \quad \tau_v > \tau_c$$

Shear Design is necessary

Design vertical shear Reinforcement :

Design shear force

$$V_s = (\tau_v - \tau_c)B.d$$

$$V_s = (0.787 - 0.605) \times 230 \times 560 \times 10^{-3}$$

$$V_s = 23.44 \text{ kN}$$

Let, us take 2-legged 8 mm ϕ stirrups

So, $A_{sv} = 2 \times \frac{\pi}{4} \times 8^2$

$= 100.53 \text{ mm}^2$

$\therefore$ Spacing $S_v = \frac{0.87 f_y A_{sv} d}{V_s}$

$$S_v = \frac{0.87 \times 415 \times 100.53 \times 560}{23440}$$

$$S_v = 867.15 \text{ mm}$$

Now, Check for maximum spacing :

(i) 0.75 d = 0.75 × 560 = 420 mm

(ii) 300 mm

(iii) $S_v \geq \frac{0.87 f_y A_{sv}}{0.4B}$

$= \frac{0.87 \times 415 \times 100.53}{0.4 \times 230}$

$= 394 \text{ mm}$

So, provide maximum spacing = 390 mm

34. p_t = Percentage of tension reinforcement

$$p_t = \frac{4 \times (\pi/4) \times 25^2}{300 \times 500} \times 100\% = 1.309\%$$

For M20 concrete

p_t	$\tau_c (N/mm^2)$
1.25	0.67
1.50	0.72

$$\tau_c = \frac{0.72 - 0.67}{1.50 - 1.25}(1.309 - 1.25) + 0.67$$

$$\tau_c = 0.6818 \text{ N/mm}^2$$

and $\tau_{c_{max}} = 2.8 \text{ N/mm}^2$

Nominal shear-stress

$$\tau_v = \frac{V_u}{B.d} = \frac{250 \times 10^3}{300 \times 500} = 1.67 \text{ N/mm}^2$$

$$\tau_v < \tau_{c_{max}}$$

$$\tau_v > \tau_c$$

Design Shear force

$$V_s = (\tau_v - \tau_c)B.d$$

$$V_s = (1.67 - 0.68) \times 300 \times 500$$

$$V_s = 148000 \text{ N}$$

Shear Resistance for a series of bent-up bars at different cross sections.

$$V_{sb} = 0.87 f_y A_{sb} \sin\alpha$$

$$= 0.87 f_y A_{sb} \sin\alpha$$

$$V_{sb} = 0.87 \times 250 \times 2 \times \frac{\pi}{4} \times 20^2 \sin 45^o$$

$$V_{sb} = 96632.92 \text{ N}$$

But the shear-resistance of the bentup bars cannot exceed 0.5 times the design shear force

$$= 0.5 \times 148000 = 74000 \text{ N.}$$

So, Providing $\rightarrow$ 6 mm-2 legged stirrups.

$$A_{sv} = 2\times\frac{\pi}{4} \times 6^2 = 56.55 \text{ mm}^2$$

$$S_v = \frac{0.87 f_y A_{sv} d}{V_s}$$

$$= \frac{0.87\times250\times56.55\times500}{74000}$$

$$S_v = 83.10 \text{ mm}$$

So, Spacing = 83.10 mm

Check for maximum spacing :

(i) 300 mm

(ii) 0.75 d = 0.75 × 500 = 375 mm

(iii) $S_v \geq \dfrac{0.87 f_y A_{sv}}{0.4b} = \dfrac{0.87\times250\times56.55}{0.4\times300}$

$$= 102.40\text{mm}$$

So, provide spacing 80 mm.

35. ***Design steps of footing :***

- First calculate the area of the footing:

$$\text{Footing-Area} = \frac{\text{Load transferred to earth}}{\text{Bearing Capacity of earth}} = \frac{P + 10\% \text{ of } P}{q}$$

$$A = \frac{1.1P}{q}$$

$$A = L \times B \qquad ...(1)$$

$$\frac{L}{B} = \frac{a}{b} \qquad ...(2)$$

where,

L,B = Size of footing

a, b = Size of column

q = Bearing capacity of soil

P = Load transferred from column.

L, B are calculated from equation (1) and (2)

- Calculate depth of the footing:

$$BM_x = \frac{w}{8}(L-a)^2$$

$$BM_u = \frac{w}{8}(B-b)^2$$

where, w = Upward pressure = $\frac{P}{\text{Area}}$

Depth is calculated for more bending-moment.

Effective depth (d) = $\sqrt{\frac{BM}{QB}}$

Now check the footing depth for :

- On way shear:

The critical section for one-way shear is at a distance 'd' from the face of column.

So, shear force causing one way Shear :

$$V_{\text{one-way}} = w \times B\left\{\left(\frac{L-a}{2}\right) - d\right\}$$

and the sheared area

$$A = B \times d$$

So, Nominal one-way shear stress

$$\tau_{v_1} = \frac{V_{one-way}}{A}$$

$$\tau_{v_1} = \frac{wB\left\{\left(\frac{L-a}{2}\right)-d\right\}}{B\times d}$$

Since for Footing :

$$\tau_v \leq k.\tau_C$$

- For two-way shear/punching

The critical section for two way shear is at a distance '$\frac{d}{2}$' from the face of column.

Shear force causing punching

$$V_p = w[(L\times B)-\{(a+d).(b+d)\}]$$

and area punched

$$A = 2(a + d + b + d).d$$

So, punching-stress

$$\tau_{vp} = \frac{w[(L\times B)-(a+d).(b+d)]}{2\{(a+d)+(b+d)\}.d}$$

$$\tau_{vp} \leq \tau_{P\ permissible}$$

$$\tau_{p\ per} = K_s 0.16\sqrt{f_{ck}} \rightarrow \text{For W.S.M.}$$

$$= K_s 0.25\sqrt{f_{ck}} \rightarrow \text{for L.S.M.}$$

Where $K_s = 0.5 + \frac{\text{short side of column}}{\text{long side of column}}$

But not greater than 1.0

- Provide steel :

$$A_{st_x} = \frac{BM_x}{\sigma_{st}jd}$$

$$A_{st_y} = \frac{BM_y}{\sigma_{st}jd}$$

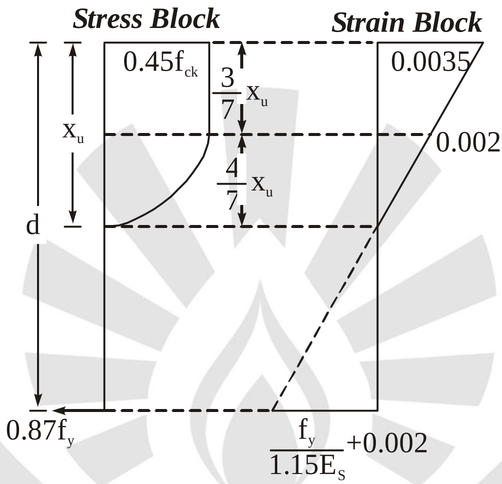
Where, jd = Lever-arm

- Steel in central band of width equal to shorter side of footing

$$A_{CT_{Central}} = A_{ST}\left(\frac{2}{1+\beta}\right)$$

$$\beta = \frac{\text{Longer side of footing}}{\text{Short side of footing}}$$

36. ***As Per IS : 456 - 2000 :***



Assumptions for limit state of collapse flexure

- Plane section normal to the axis remains plane after bending. i.e., Strain-diagram is linear.
- The maximum strain in concrete at the outermost compression fibre is taken as 0.0035 in bending.
- The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be rectangular, trapezoid, parabola or any other shape which results in prediction of strength.

  For design purposes, the compressive-strength of concerete in the structure shall be assumed to be 0.67 times the characterstic strength. The partial safety factor γ_m = 1.5 shall be applied in addition to this.

  So, design strength $= \dfrac{0.67f_{ck}}{1.5}$

  $= 0.4467\ f_{ck} \simeq 0.45\ f_{ck}$

Note: *Area of stress block = $0.36f_{ck}X_u$*

Depth of centre of compressive force = $0.42X_u$ from extreme compression fibre.

- The tensile strength of concrete is ignored.
- For design purposes the partial safety factor for steel is taken as $\gamma_m = 1.15$.

  So, design strength of steel = $\dfrac{f_y}{1.15}$ = 0.87 f_y.
- The maximum strain in the tension reinforcement in the section at failure shall not be less than $\dfrac{f_y}{1.15E_s} + 0.002$.

37. ***As per IS : 800 - 1984:***

Unless the outer edge of each stiffner is continously stiffened, the outstand of all stiffners from the web should not be more than

$\dfrac{256t}{\sqrt{f_y}}$.

Where vertical stiffners are required, they should be provided throughout the length of the grider at a distance not greater than 1.5 d_1 and not less than 0.33 d_1.

When horizontal stiffners are provided d_1 be should be taken as clear distance between the horizontal stiffner and tension flange ignoring fillets.

The moment of inertia(I) of a pair of vertical-stiffeners about the center of web or a single stiffner about the face of the web should be,

$$I \geq \frac{1.5d_1^3t^3}{C^2}$$

where, t = Minimum required thickness of web

C = The maximum permitted clear distance between vertical stiffner for thickness 't'.

Sometimes vertical stiffners are subjected to external-forces and therefore the moment of inertia of the stiffner should be increased as described below.

(a) Bending moment on stiffner due to eccentricity of vertical loading with respect to vertical axis of web :

Iincrease of

$$I = \frac{150MD^2}{Et_w} cm^4$$

(b) Lateral loading on stiffner

$$I = \frac{0.3VD^3}{Et_w} cm^4$$

For first horizontal stiffner at $\left(\frac{2}{5}\right)^{th}$ of the distance between compression flange and neutral axis for the compression flange.

$$I \geq 4\ C\ t^3$$

where, t = Minimum thickness of web required.

C = Actual distance between vertical stiffner.

I = Moment of inertia of horizontal stiffner pair.

For second horizontal stiffner at neutral axis:

$$I \geq d_2 t^3$$

Stiffners are not connected to web to withstand a shearing force not less than $\frac{125t_w^2}{h} kN/m$

where, h = Outstand of stiffner in mm.

38. For solid column let diameter be 'D_s' and external diameter of hollow column b 'D_e'.

Since both columns have same cross-sectional area.

$$\frac{\pi}{4}\left[D_e^2 - \left(\frac{2}{3}D_e\right)^2\right] = \frac{\pi}{4}D_S^2$$

$$\frac{\sqrt{5}}{3}D_e = D_S$$

Buckling strength means Euler's critical load required.

So, Buckling-Strength of solid column

$$P_1 = \left(\frac{\pi^2 EI}{L_{eff}^2}\right)_1 = \frac{\pi^2 E(\pi D_S^4/64)}{(0.5L)^2}$$

$$P_1 = \frac{4\pi^2 E D_S^4}{64L^2}$$

$$P_1 = \frac{\pi^2 E D_S^4}{16}$$

and Buckling strength of hollow column

$$P_2 = \left(\frac{\pi^2 EI}{L_{eff}^2}\right)_2 = \frac{\pi^2 E(\pi/64)(D_e^4 - D_i^4)}{(0.5L)^2}$$

$$P_2 = \frac{4\pi^2 E}{64L^2}\left[D_e^4 - \left(\frac{2}{3}D_e\right)^4\right]$$

$$P_2 = \frac{4\pi^2 E}{64}\left[\left(\frac{3D_s}{\sqrt{5}}\right)^4 - \left(\frac{2}{3}\times\frac{3}{\sqrt{5}}D_S\right)^4\right]$$

Since $D_e = \frac{3}{\sqrt{5}}D_S$

$$P_2 = \frac{\pi^2 E}{16}\left[\frac{81D_S^4}{25} - \frac{16D_S^4}{25}\right]$$

$$P_2 = \frac{\pi^2 E}{16}\left[\frac{81-16}{25}\right]D_S^4 = \frac{\pi^2 E}{16}\left(\frac{65}{25}D_S^4\right)$$

$\therefore$ Ratio of buckling-strengths

$$\frac{P_1}{P_2} = \frac{25}{65} = \frac{5}{13}$$

39. For Fe410 grade of steel f_u = 410 N/mm^2

For bolts of grade 4.6; f_{ub} = 400 MPa

Partial safety factor for the material of bolt,

γ_{mb} = 1.25

A_{nb} = Stress area of bolt

$$= 0.78 \times \frac{\pi}{4} \times 20^2 = 245 \text{ mm}^2$$

Given, Diameter of bolt = 20 mm

Pitch = p = 80 mm

Edge distance = e = 40 mm

So, hole diameter = d_o = 20 + 2 = 22 mm

Strength of bolt in shear

$$V_{sb} = A_{nb}\frac{f_{ub}}{\sqrt{3}\gamma_{mb}} = 245\times\frac{400\times10^{-3}}{\sqrt{3}\times1.25} = 45.26 \text{ kN}$$

Bearing strength of bolt

$$V_{pb} = 2.5\ k_b.d.t.\frac{f_u}{\gamma_{mb}}$$

$$V_{pb} = 2.5k_b.20\times 9.1\times\frac{400\times10^{-3}}{1.25}$$

Where, f_u = least of f_u and f_{ub}

$$k_b = \text{Least of } \frac{e}{3d_o} = \frac{40}{3\times20} = 0.606$$

$$\frac{p}{3d_o}-0.25 = \frac{80}{3\times22} - 0.25 = 0.96$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.975 \text{ and } 1.0$$

So, $K_b = 0.606$

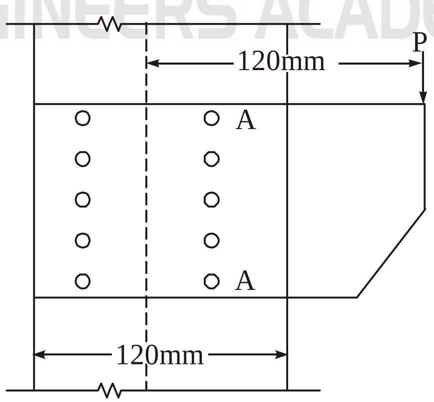
$$V_{pb} = 2.5\times0.606\times9.1\times\frac{400\times10^{-3}}{1.25}$$

$$V_{pb} = 88.23 \text{ kN}$$

So, strength of bolt = Minimum of shear and bearing strength of bolt = 45.26 kN

Let, P_1 be the factored load

Service load = $P = \dfrac{P_1}{1.50}$



The maximum stressed bolt is 'A'.

Total number of bolt in joint n = 10

The direct force = $\frac{P_1}{10}$ = F_1

The force due to torque

$$F_2 = \frac{Pe_o r_n}{\Sigma r^2}$$

$$r_n = \sqrt{(80+80)^2 + \left(\frac{120}{2}\right)^2}$$

$$= 170.88 \text{ mm}$$

$$\Sigma(r^2) = 4 \times [(160^2 + 60^2) + (80^2 + 60^2)] + 2 \times 60^2$$

$$= 164000 \text{ mm}^2$$

$$F_2 = \frac{P_1 \times 200 \times 170.88}{164000} = 0.20839\ P_1$$

$$\cos\theta = \frac{60}{\sqrt{60^2 + 160^2}} = 0.3511$$

Resultant force the bolt should be less than or equal to the strength of bolt.

The resultant force

$$= \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$$

$$F_R = \sqrt{\left(\frac{P_1}{10}\right)^2 + (0.20839P_1)^2 + 2\left(\frac{P_1}{10}\right)(0.20839P_1) \times 0.3511}$$

$F_R = 0.2608P_1$

$F_R \leq$ Strength of Bolt

$0.2608P_1 \leq 45.26$ kN

$P_1 = 173.48$ kN

❍❍❍