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ESE-2021
(MAINS)

QUESTIONS WITH DETAILED SOLUTIONS

CIVIL ENGINEERING

PAPER-I

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CIVIL ENGINEERING

ESE _MAINS_2021_PAPER – I

Questions with Detailed Solutions

SUBJECT WISE WEIGHTAGE

S.No.	NAME OF THE SUBJECT	Marks
1	Strength of Materials	96
2	Structural Analysis	104
3	Building Materials	52
4	Design of Concrete and Masonry Structures	124
5	Steel Structures	52
6	Construction Management & Equipment	52

SECTION A

01.

(a). (i). What are the functions of cement mortar in brick masonry work? **(6 M)**

Sol: Following are the functions of cement mortar in masonry work.

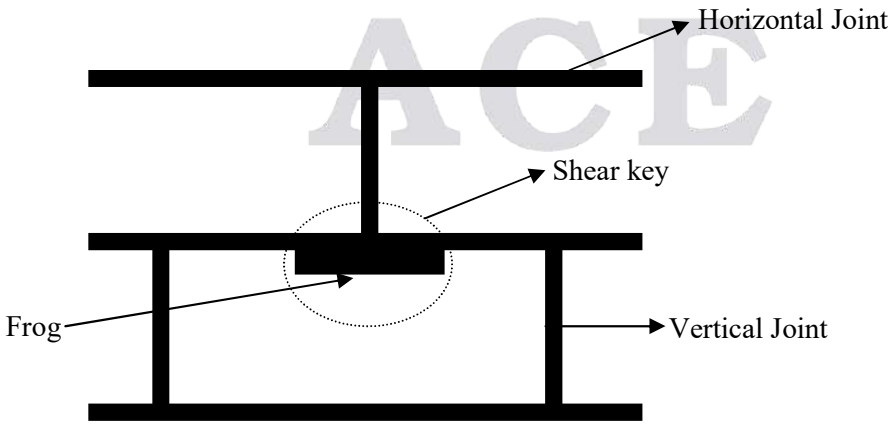
1. In masonry work, mortar is filled between the joints of the building blocks (bricks / stones) which makes these building blocks bond together and act as a solid mass.
2. Mortar is used to carry out pointing and plastering works on exposed surfaces of masonry to protect the masonry from dampness and to give the masonry a good appearance.
3. Mortar is used for ornamental works on masonry like coping, cornice and coving works.

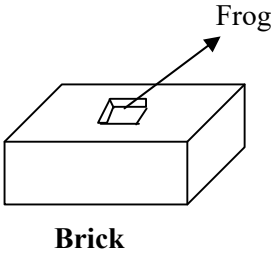
(ii). What are the purposes of making frog in bricks during moulding? Explain with the help of neat sketches. **(6 M)**

Sol: A small indentation made on the surface of bricks during moulding is called as a frog.

Application of frog are as follows:

1. **Shear key:** When mortar is filled while laying the bricks, frog helps in the formation of a shear key which improves the strength of masonry against horizontal loads.
2. **Source:** Generally, the initials of the manufacturer are impressioned in the frog, thus helping in identification of the manufacturer.





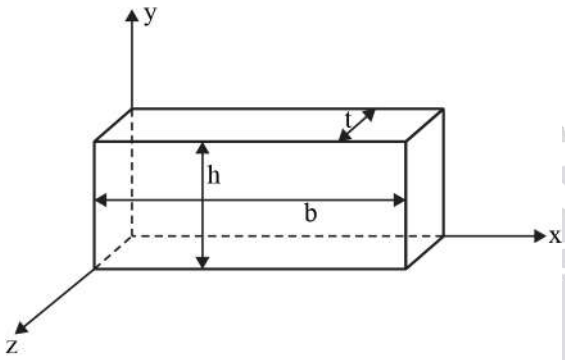
Brick

(b). A rectangular plate in biaxial stress in subjected to normal stress $\sigma_x = 65$ MPa and $\sigma_y = -20$ MPa. The plate (b × h × t) has dimensions 200 × 300 × 15 and is made of aluminimum with E = 75 GPa and $\nu = 0.33$. All dimensions are in mm. Determine

(i) the maximum in-plane shear strain (γ_{max}) in the plate,

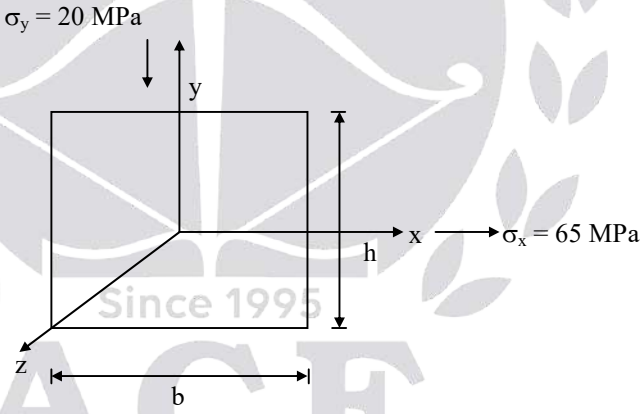
(ii) the change in the thickness of the plate (Δt), and

(iii) the change in volume of the plate (ΔV).



(12 M)

Sol:



b = 200 mm

h = 300 mm

t = 15 mm

$E = 75 \times 10^3$ MPa

$\mu = 0.33$

$\Rightarrow$ As there is no shear stress acting on x & y planes.

So, $\sigma_1 = \sigma_x = 65\,\text{MPa}$

$\sigma_3 = \sigma_y = -20\,\text{MPa}$

$$\varepsilon_1 = \frac{\sigma_1}{E} - \mu\frac{\sigma_3}{E} \Rightarrow \frac{65}{E} - 0.33 \times \frac{(-20)}{E}$$

$$= 9.546 \times 10^{-4}$$

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$$\varepsilon_3 = \frac{\sigma_3}{E} - \mu\frac{\sigma_1}{E} = \frac{-20}{E} - 0.33\times\frac{65}{E}$$

$$= -5.526 \times 10^{-4}$$

$$\varepsilon_2 = -\mu\frac{\sigma_1}{E} - \mu\frac{\sigma_3}{E}$$

$$= \frac{-\mu}{E}(\sigma_1 + \sigma_3) = \frac{-0.33}{75\times10^3}\times(65-20)$$

$$= -1.98 \times 10^{-4}$$

(i) $\frac{\phi_{max}}{2} = \frac{\varepsilon_1 - \varepsilon_3}{2}$

$\phi_{max} = (9.54 + 5.526) \times 10^{-4}$

$= 15.066 \times 10^{-4}$

$\therefore$ Max in plane shear strain $= 15.066 \times 10^{-4}$

(ii) $\frac{\Delta t}{t} = \varepsilon_2$

$\Rightarrow \frac{\Delta t}{15} = -1.98 \times 10^{-4}$

$\boxed{\Delta t = -29.7 \times 10^{-4} \text{ mm}}$

$\therefore$ Thickness decrease by 2.97×10^{-3} mm

(iii) $\frac{\Delta V}{V} = \varepsilon_1 + \varepsilon_2 + \varepsilon_3$

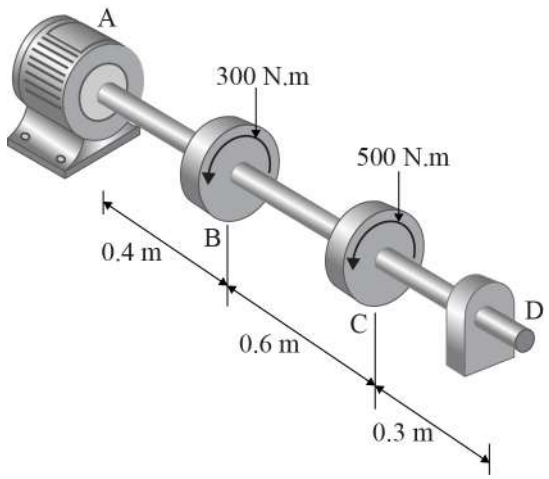
$= 2.034 \times 10^{-4}$

$\Delta V = 2.304 \times 10^{-4} \times 200 \times 300 \times 15$

$= 183.06 \text{ mm}^3$

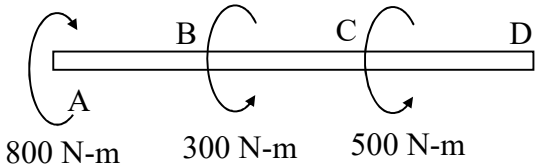
$\therefore$ Volume increases by 183.06 mm^3

(c). The electric motor exerts a torque of 800 N on the steel shaft ABCD when it is rotating at constant speed. Design specifications require that the diameter of the shaft be uniform from A to D and that the angle of twist between A and D not exceed 1.5°. Knowing that $\tau_{max} \leq 60$ MPa and G = 77 GPa, determine the diameter of the shaft that may be used.



(12 M)

Sol:



Maximum torque (in AD) = 800 N-m

i. Maximum shear stress criteria

$$\frac{\tau_{max}}{r} = \frac{T}{J}$$

$$(Z_P)\,\tau_{max} = T$$

$$\frac{\pi}{16}\left(D^3\right)(60) = 800\times10^3\,(N-mm)$$

d = 40.8 mm ..... (1)

ii. Maximum angle of twist criteria

$$\theta_{AD} = \frac{1.5^0 \times \pi}{180} = \theta_{AB} + \theta_{BC} + \theta_{CD}$$

$$\frac{1.5 \times \pi}{180} = \frac{(800 \times 10^3)(400)}{77 \times 10^3 \left(\frac{\pi}{32.} d^4\right)} + \frac{(500 \times 10^3)(600)}{(77 \times 10^3)\left(\frac{\pi}{32} d^4\right)} + 0$$

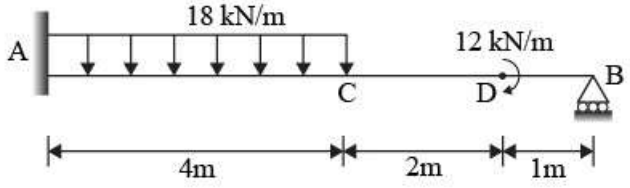
$[\because \theta_{CD} = 0]$

$$1.5 \times 180 (d^4) = \frac{32}{77 \times 10^3} [800 \times 10^3 \times 400 + 500 \times 10^3 \times 600]$$

d = 42.07 mm…….. (2)

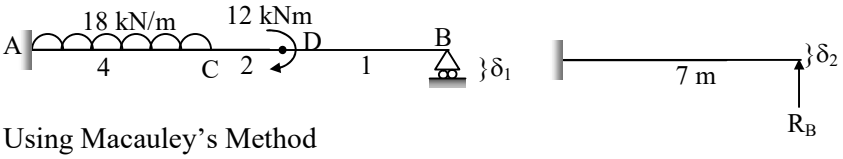
$\therefore$ Dia of shaft = max of 1 & 2 = 42.07 mm ≈ 43 mm.

(d). Analyse the propped cantilever beams shown in the figure. During loading the prop B sinks by 6 mm. E = 2 × 10^5 MPa. I = 86.04 × 10^6 mm^4. Draw Shear force and Bending moment diagrams. Find the maximum BM value and its location from either of the supports.



(12 M)

Sol:



Using Macauley's Method

$$EI\frac{d^2y}{dx^2} = R_B(x) - 12(x-1)^0 - \frac{18}{2}(x-3)^2$$

$$EI\frac{dy}{dx} = R_B\left(\frac{x^2}{2}\right) - 12(x-1)^1 - 3(x-3)^3 + c_1$$

$$EI(y) = R_B\left(\frac{x^3}{6}\right) - 6(x-1)^2 - \frac{3}{4}(x-3)^4 + c_1 x + c_2$$

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Boundary conditions

@ B: x= 0: y = 6mm = 6/1000 (m) EI = 17208 kN-m

C_2=103.248

@ A ; x = 7m y = 0

$$0 = R_B(57.16) - 6(6)^2 - \frac{3}{4}(7-3)^4 + C_1(7) + 103.248$$

$$57.16\,R_B - 304 + 7\,C_1 = 0 \rightarrow (1)$$

@ A : x = 7m dy/dx = 0

$$0 = R_B(24.5) - 72 - 3(4^3) + C_1$$

$$24.5R_B - 264 + C_1 = 0 \rightarrow (2)$$

$$57..16R_B - 304 + 7C_1 = 0$$

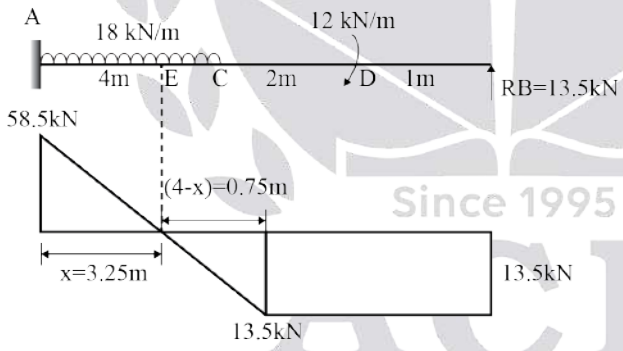
$$7 \times 24.5R_B - 264 \times 7 + 7C_1 = 0$$

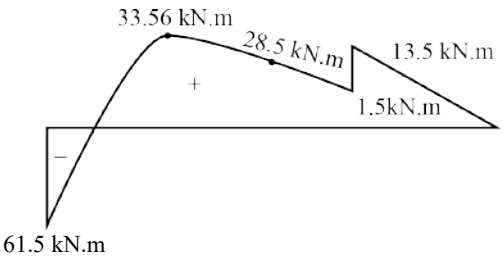
$$(-) \quad (+) \quad (-)$$

$$-114.34\,R_B\ 1544 = 0$$

$$R_B = 13.5\text{kN}$$

From (2) C_1= - 66.8





Max sagging BM = 33.56 kN – m [@E, 3.25 m from A]

Max hogging BM = 61.5 kN-m [@A]

Alternate method

By deflection method

Deflection at B due to u.d.l & moment = deflection due to reaction at B

$$\frac{W_{AC}(L_{AC})^4}{8EI}+\frac{W_{AC}(L_{AC})^3}{6EI}\times L_{CB}+\frac{M_D\times(L_{AD})^2}{2EI}+\frac{M_D L_{AD}\times L_{DB}}{EI}+6mm=\frac{R_B\times(L_{AB})^3}{3EI}$$

$$\Rightarrow \frac{18\times(4\times10^3)^4}{8\times2\times10^5\times86.04\times10^6}+\frac{18\times(4\times10^3)^3\times(3\times10^3)}{6\times2\times10^5\times86.04\times10^6}$$

$$+\frac{12\times10^6\times(6\times10^3)^2}{2\times2\times10^5\times8604\times10^6}+\frac{12\times10^6\times6\times10^3\times1\times10^3}{2\times10^5\times86.84\times10^6}+6$$

$$=\frac{R_B\times(7\times10^3)^3}{3\times2\times10^5\times86.04\times10^6}$$

$$\Rightarrow 33.47+33.47+12.55+4.18+6=R_B\times6.64\times10^{-3}$$

$$R_B=13.50\text{ kN}$$

$$\Sigma V=0\Rightarrow R_A=18\times4-R_B$$

$$=18\times4-13.50$$

$$=58.50\text{ kN}$$

$$\Sigma M_A=0\Rightarrow M_A+13.50\times7=18\times4\times2+12$$

$$M_A=61.50\text{ kN.m}$$

(e). A simply supported rectangular beam (cross-section 300 mm × 400 mm) with effective span of 6 metres is carrying the following characteristic load:

(i) Characteristic Dead Load (including self-weight) = 15 kN/m

(ii) Characteristic Imposed Load (not fixed) = 10 kN/m

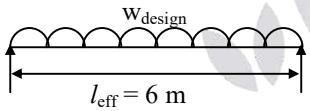
(iii) Characteristic Equivalent Wind Load (acting downward) = 5 kN/m

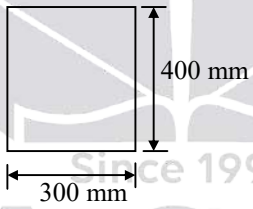
Calculate the Design Bending Moment and Design Shear Force for most critical load combination for limit state of collapse and limit state of serviceability.

Partial Safety Factor (γ_f) for loads						
Load Combination	**Limit State of Collapse**			**Limit State of Serviceability**		
	Dead Load	**Imposed Load**	**Wind Load**	**Dead Load**	**Imposed Load**	**Wind Load**
Dead Load + Imposed Load	1.5	1.5	-	1.0	1.0	-
Dead Load + Wind Load	0.9	-	1.5	1.0	-	1.0
Dead Load + Imposed Load + Wind Load	1.2	1.2	1.2	1.0	0.8	0.8

(12 M)

Sol:





D.L. = 15 kN/m

L.L. = 10 kN/m

W.L = 5 kN/m

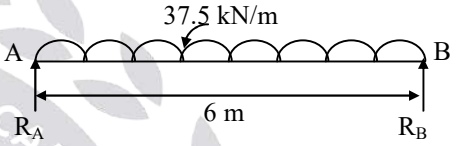
Calculation of Design Load (Using Limit State of Collapse):

$$w_{design} = \max\begin{cases}1.5[D.L + L.L]\\ 0.9D.L + 1.5W.L\\ 1.2[D.L + l.L + W.L]\end{cases}$$

$$w_{design} = \max\begin{cases}1.5[15+10] = 37.5\\ 0.9\times 15 + 1.5\times 5 = 21\\ 1.2[15+10+5] = 36\end{cases}$$

$\Rightarrow w_{design} = 37.5$ kN/m

Calculation of Design BM and S.F:



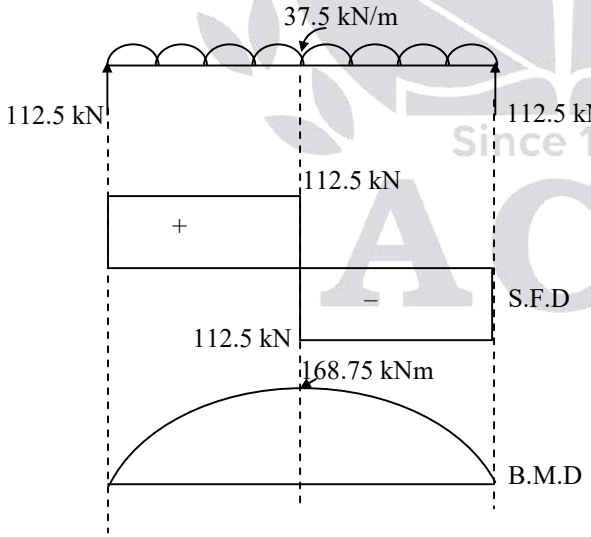
$$R_A = \frac{37.5\times 6}{2} = 112.5\,\text{kN}$$

$$R_B = 112.5\,\text{kN}$$

Design S.F = Maximum S.F = R_A = 112.5 kN

Design B.M = Maximum (at centre) $= \frac{w\ell^2}{8}$

$$= \frac{37.5\times 36}{8} = 168.75\ \text{kNm}$$

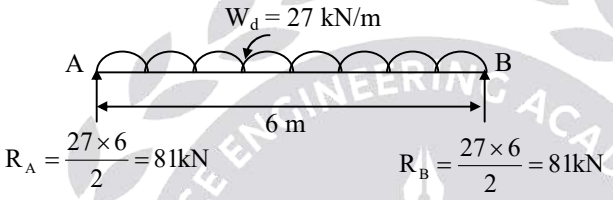


Calculation of Design Load (Using Limit State of Serviceability):

$$w_d = \max\begin{cases}1.(D.L + L.L)\\ 1.(D.L + W.L)\\ 1.D.L + 0.8(L.L + W.L)\end{cases}$$

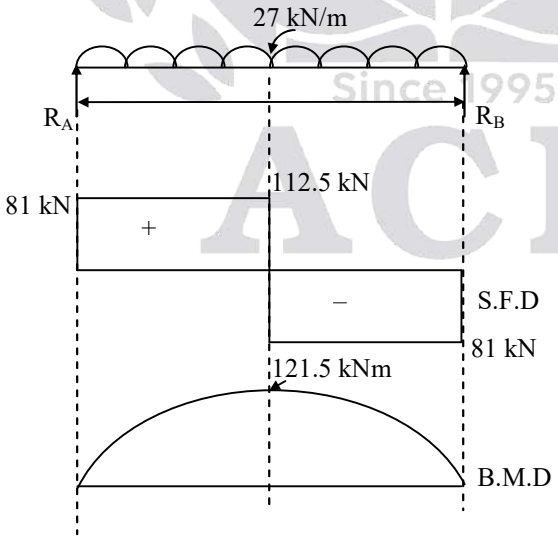
$$= \max\begin{cases}1.(15+10) = 25\ kN/m\\ 1.(15+5) = 20\ kN/m\\ 1\times15 + 0.8\times15 = 27\ kN/m\end{cases}$$

W_{design}= 27 kN/m



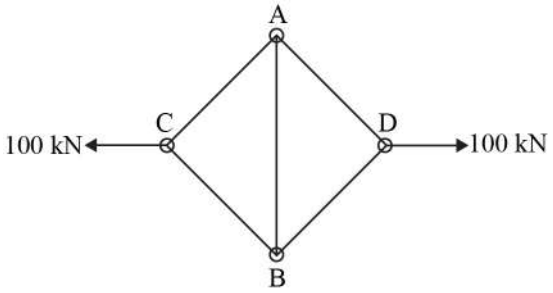
Design S.F = 81 kN (at support)

Design B.M= $\frac{w\ell^2}{8} = \frac{27\times36}{8} = 121.5$ kNm



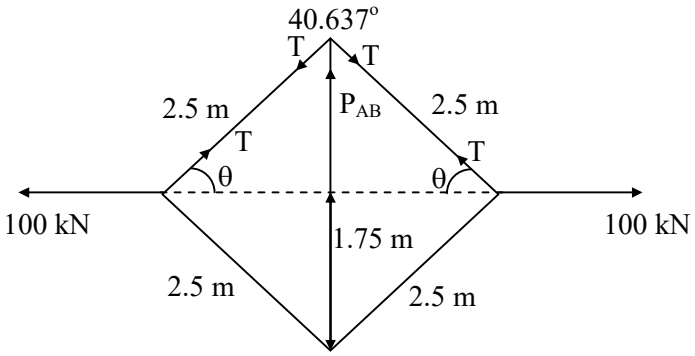
02.

(a). A steel bar AB of length 3.5 m and diameter 25 mm is connected by four inextensible cables of length 2.5 m each, forming a rhombus with AB as diagonal. A 100 kN force acts at the points C and D. Determine the decrease in length of the member AB and increase in the length between points C and D. Take modulus of steel as 201 GPa.



(20 M)

Sol:



$$\theta = \sin^{-1}\left(\frac{1.75}{2.5}\right) = 44.43°$$

$2T\cos\theta = 100$

$$T = \frac{100}{2\cos 44.43°} \Rightarrow 70.18 \text{ kN}$$

Compressive force on AB (P_{AB}) = 2T sin 44.43

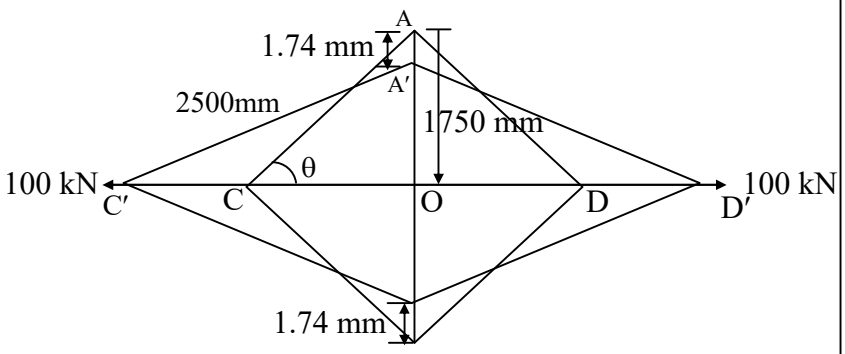
= 98.26 kN

Displacement of AB (δ)

$$\delta = \frac{P_{AB}L_{Ab}}{AE}$$

$$= \frac{98.26\times10^3\times3.5\times10^3}{\frac{\pi}{4}\times25^2\times201\times10^3}$$

$$= -3.48 \text{ mm}$$



As cables are inextensible, there wont be any change in length. ∴ AC = A' C'

$OA = 1750\text{mm}$

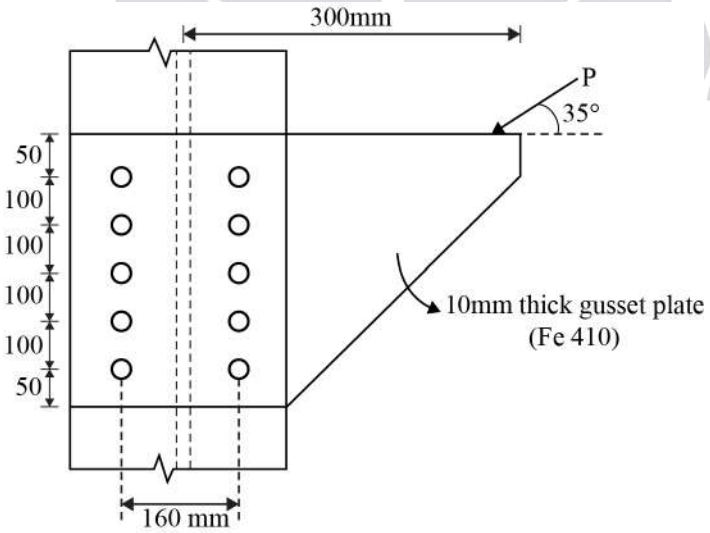
$OA' = 175 - 1.74 = 1748.26\text{mm}$

$$OC = \sqrt{CA^2 - OA^2} = \sqrt{2500^2 - 1750^2} = 1785.36\text{mm}$$

$$OC' = \sqrt{(C'A')^2 - (OA')^2} = \sqrt{2500^2 - 1748.26^2} = 1786.96\text{mm}$$

Increase in length between C & D = 2(1786.96-1785.36) =3.21 mm

(b). Determine the maximum load P the joint can carry if bolts of M 20 grade 4.6 are used.



Design shear strength of bolt $V_{nsb} = \frac{f_{ub}}{\sqrt{3}} . \frac{A_{nb}}{\gamma_{mb}}$

Bearing strength of bolt = $\frac{1}{\gamma_{mb}} 2.5 k_b .d.t.f_u$

Where k_b = Least of $\left[\frac{e}{3d_o}, \left(\frac{p}{3d_o} - 0.25 \right), \frac{f_{ub}}{f_u}, 1.0 \right]$

(20 M)

Sol: Given, M20 bolts, i.e. nominal diameter of bolt = 20 mm

Grade of the bolt = 4.6

i.e. f_{ub} = 400 MPa

f_{yb} = 240 MPa

thickness of gusset plate = 10 mm

Step I: Calculating Bolt value of the given connection as per IS 800:2007

Bolt value minimum $\{V_{dsb}, V_{dpb}\}$

V_{dsb} = design shear strength of the bolt

V_{dpb} = design bearing strength of the bolt

Now,

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}} \frac{A_{ub}}{\gamma_{mb}}$$

$$= \frac{400}{\sqrt{3}} \times \frac{0.78 \times \frac{\pi}{4} \times 20^2}{1.25}$$

$\{\because A_{ub}$ = tensile net area of the bolt = $0.78 \times \frac{\pi}{4} \times \phi^2, \because \gamma_{mb} = 1.25\}$

V_{dsb} = 45.272 kN ……(A)

Now,

$$V_{dpb} = \frac{2.5 k_b dt f_u}{\gamma_{mb}} \qquad \{\text{here } f_u = 410 \text{ MPa}\}$$

Here,

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f_u = ultimate strength of plate = 410 MPa (given)

d = Nominal diameter of bolt = 20 mm (given)

t = thickness of gusset plate = 10 mm (given)

γ_{mb} = partial safety factor for bolt = 1.25

$$k_b = \min\left[\frac{e}{3d_o}, \frac{P}{3d_o} - 0.25, \frac{f_{ub}}{f_u}, 1.0\right]$$

Now, d_o = diameter of bolt hole

= d + 2 mm

= (20 + 2) mm = 22 mm

So,

$$k_b = \min\left\{\frac{100}{3\times 22} - 0.25, \frac{50}{3\times 22}, \frac{400}{410}, 1.0\right\}$$

= minimum {0.75, 1.265, 0.9757, 1.0}

$k_b = 0.75$

now,

$$V_{dpb} = \frac{2.5k_b dtf_u}{\gamma_{mb}} = \frac{2.5\times 0.75\times 20\times 10\times 410}{1.25}$$

V_{dpd} = 123 kN ……(B)

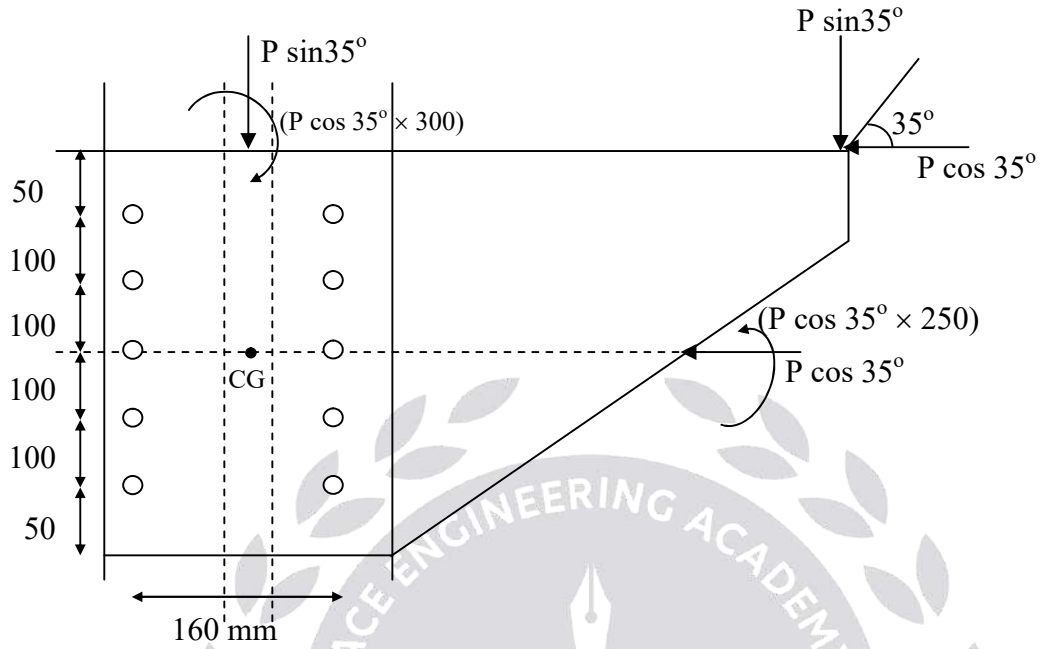
Now,

Strength of one bolt, bolt value (B_v) = minimum {V_{dsb}, V_{dpb}}

= minimum {45.272 kN, 123 kN}

Bolt value = 45.272 kN

Step II: Finding out the most critical bolts in the connection

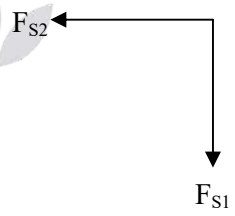


Now, there are two sets of loading, both are producing a direct load and a twisting moment.

Let two vertical load ($P\sin35^o$) and ($P\sin35^o \times 300$) be set (A)

Let two horizontal load ($P\cos35^o$) and ($P\cos35^o \times 300$) be set (B)

First consider ($P\sin35^o$) and ($P\cos35^o$) only for Bolt (A)



Two direct S,F will be there

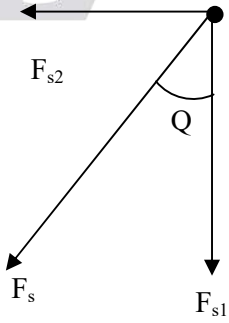
F_{S1} → due to ($P\sin35^o$)

F_{S2} → due to ($P\cos35^o$)

$$F_1 = \frac{P\sin35^o}{n} = \frac{P\sin35^o}{10}$$

$$F'_1 = \frac{P\cos35^o}{n} = \frac{P\cos35^o}{10}$$

Resultant direct shear force in one bolt (F_s)



$$F_S = \sqrt{F_1^2 + F_1'^2}$$

$$= \sqrt{\left(\frac{P\sin 35}{10}\right)^2 + \left(\frac{P\cos 35}{10}\right)^2} = \sqrt{\left(\frac{P}{10}\right)^2 (\sin^2 35 + \cos^2 35)}$$

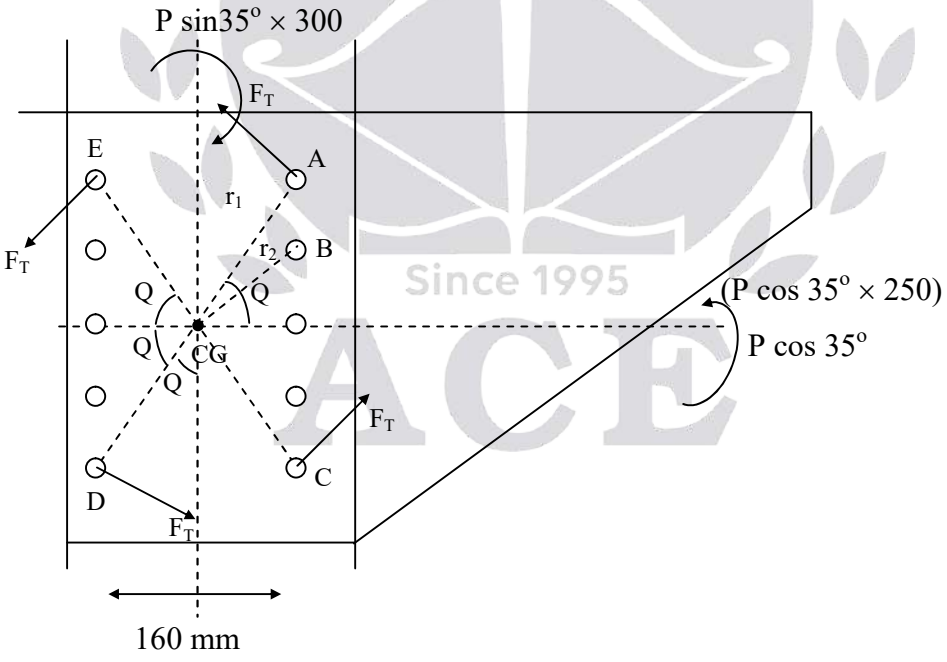
$$F_s = \frac{P}{10}$$

Direction of $F_S = \tan^{-1}\left(\frac{F_1'}{F_1}\right)$

$$= \tan^{-1}\left(\frac{P\cos 35^o}{P\sin 35^o}\right) = 55^o$$

Note: you can see that he direction of 'f_s' is same as that of applied load P on the bracket with respect to horizontal.

Now consider two moments



Net moment about (torsion) CG = psin35° × 300 – Pcos35° × 250

` = 32.715 P (↺)

$r_1 = \sqrt{200^2 \times 80^2} = 215.4$ mm

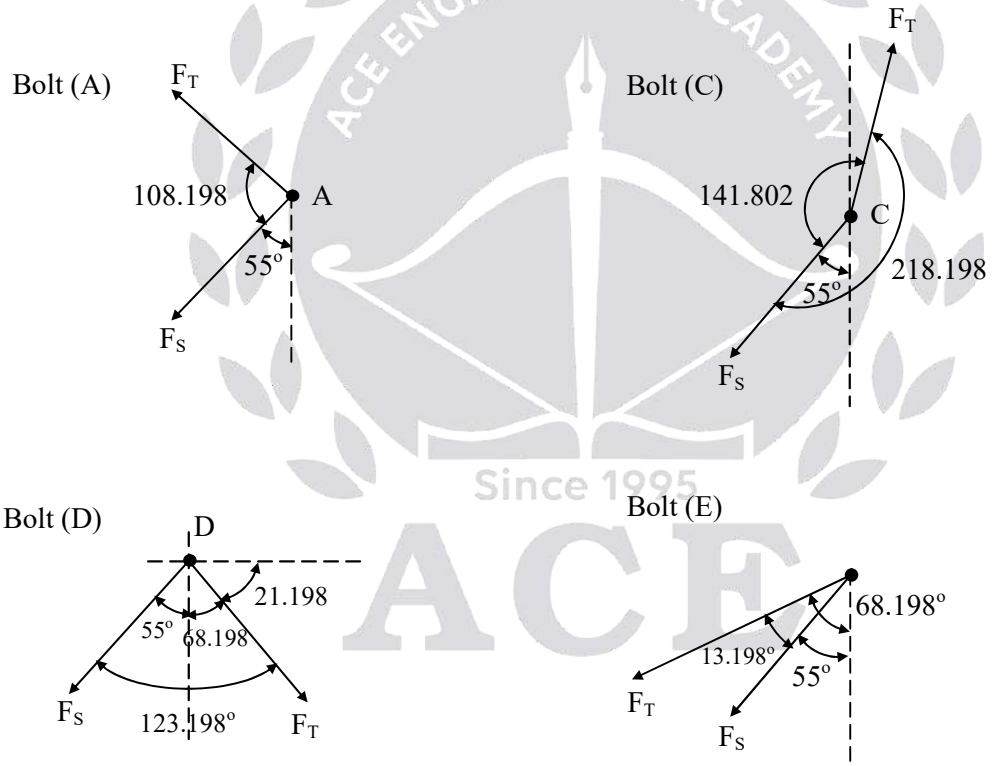
$r_2 = \sqrt{100^2 \times 80^2} = 128.06$ mm

For bolt (A)

$$F_T \text{ (shear force due to net torsion)} = \frac{(32.715P) \times 215.4}{\Sigma\left(4 \times 215.4^2 \times 4 + 128.06^2 P + 2 \times 80^2\right)}$$

$$= 0.0266\ P$$

Choosing the bolts with maximum radial is false.



Q is minimum for Bolt (E), Hence most critical bolt (E)

Step III: Resultant Force for Bolt (E)

$$F_R = \sqrt{F_S^2 + F_T^2 + 2F_S F_T \cos Q}$$

$$= \sqrt{(0.1P)^2 + (0.0266P)^2 + 2(0.1P)(0.0266P)\cos 13.198}$$

$F_R = 0.126\ P$

Now $F_R \le B_V$

$0.126\ P \le 45.272$ kN

$P \le 359.3$ kN

So $P_{max} = 359.3$ kN

(c). (i). How do super plasticizers increase the mobility and made the cement concrete flow? What is the advantage of using fly ash in concrete for massive dam construction work?

(10 M)

Sol: Super plasticizers:

Super plasticizers are high range water reducing chemical admixtures used in the preparation of High Strength Concrete, High Performance Concrete and Self Compacting concrete.

Super plasticizers are water soluble macromolecules (many times larger than water molecules) when mixed during the preparation of concrete are adsorbed by C_3A. Because of this adsorption entrapped water gets released; thus increasing the available water in the concrete which in turn makes concrete flow more easily.

Advantages of using fly ash in concrete for massive dam construction work:

When fly ash is mixed during the preparation of concrete, it improves workability and reduces the heat of hydration of concrete. Because of better workability, concrete can be handled easily and thicker placements can be done without the risk of improper compaction. Because of the reduced heat of hydration, formation of shrinkage cracks during setting and hardening is reduced.

(ii). What are the disadvantages of destructive methods of testing concrete and advantages of non-destructive testing of concrete?

(10 M)

Sol: Destructive Vs Non-destructive Testing of concrete:

Disadvantages of Destructive testing of concrete are as follows:

1. Strength of in-service structures cannot be determined without causing any damage.
2. Costly equipment (UTM / CTM) which are not very easy to handle are needed to determine the strength of concrete.
3. Internal defects like bubbles, cracks, pores, etc. cannot be determined.

Advantages of Non-destructive testing of concrete are as follows:

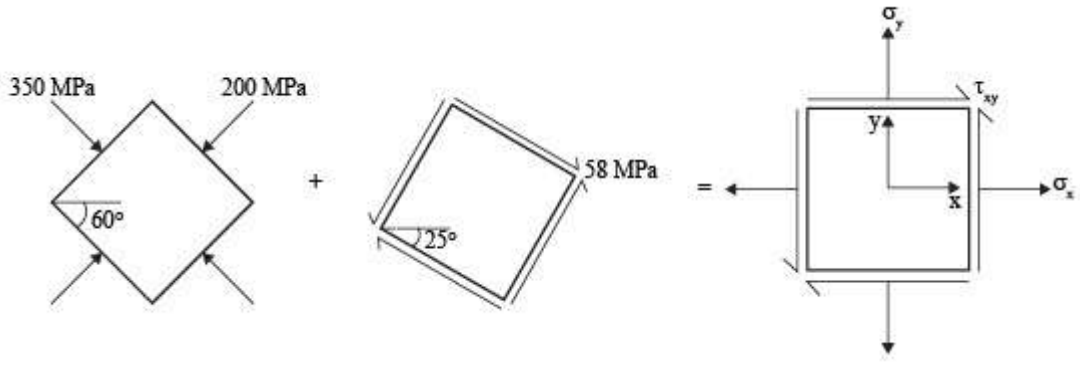
1. Equipment used for non-destructive testing of concrete are easy to handle.
2. Strength of in-service structures can be determined without causing them any damage.
3. These tests can be used to determine the internal defects in concrete.
4. These tests can be done quickly and easily.

03.

(a). A point on a thin plate is subjected to the two successive states of stress as shown in the figure below. Using Mohr's circle method, determine

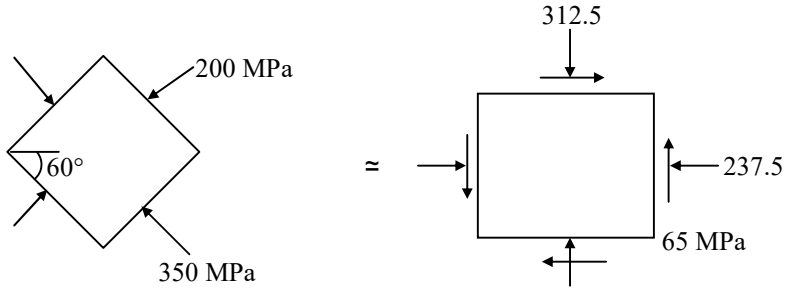
(i). the resultant state of stress represented on the element oriented as shown on the right.

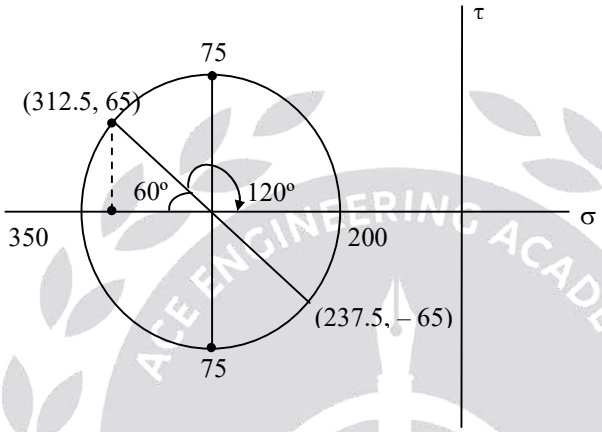
(ii). the principal stresses and principal planes caused by the superposition of these two stress states.



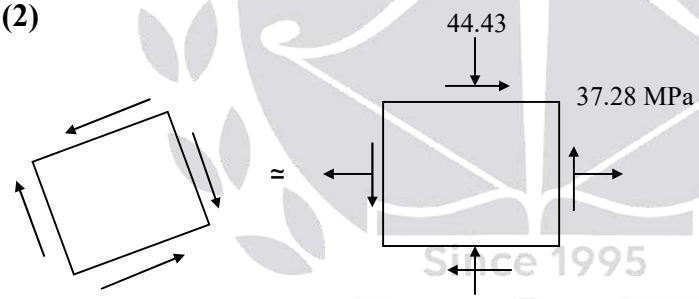
(20 M)

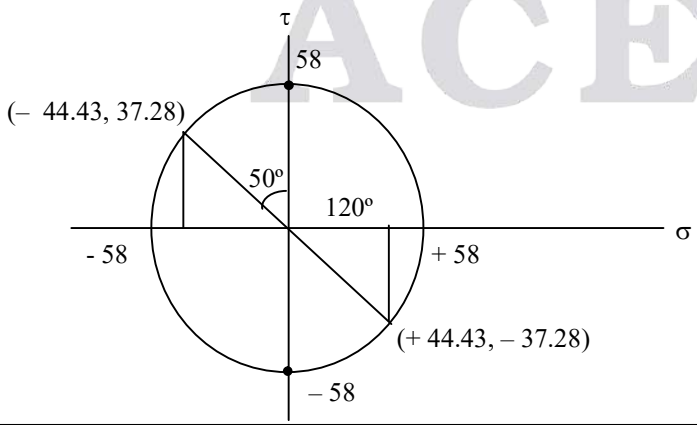
Sol: Case (1)



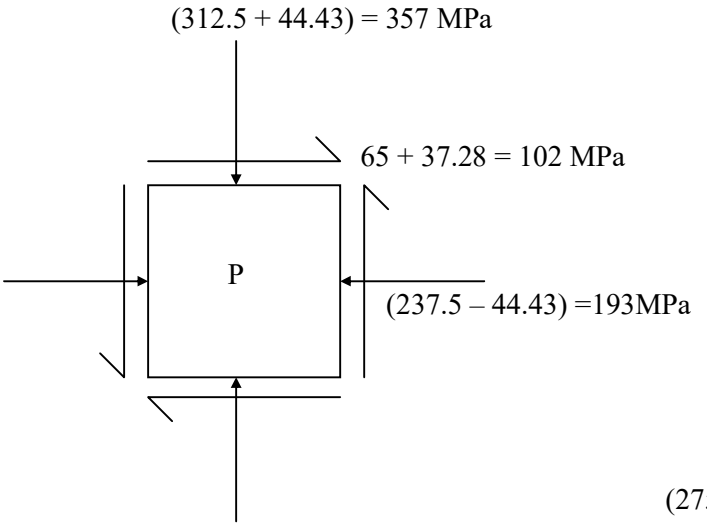


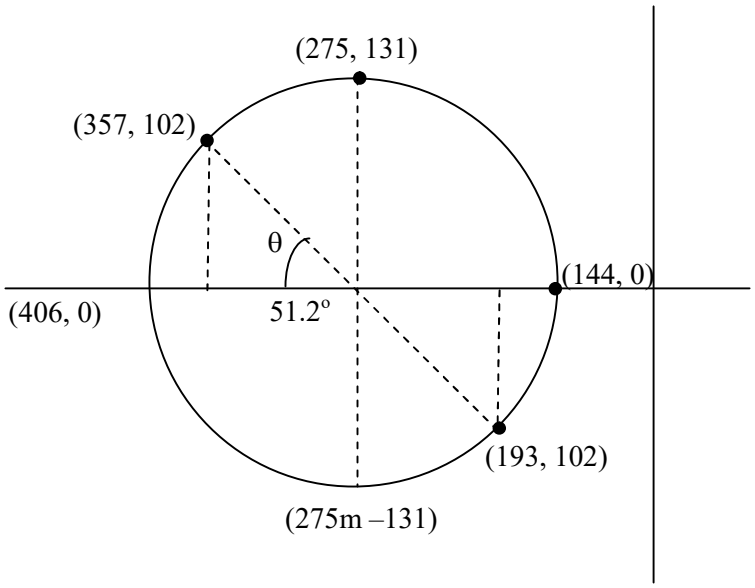
Case (2)





Resultant:





From Mohr circle

Principal stress :

σ_1 = 406 MPa (comp)

σ_2 = 144 MPa (comp)

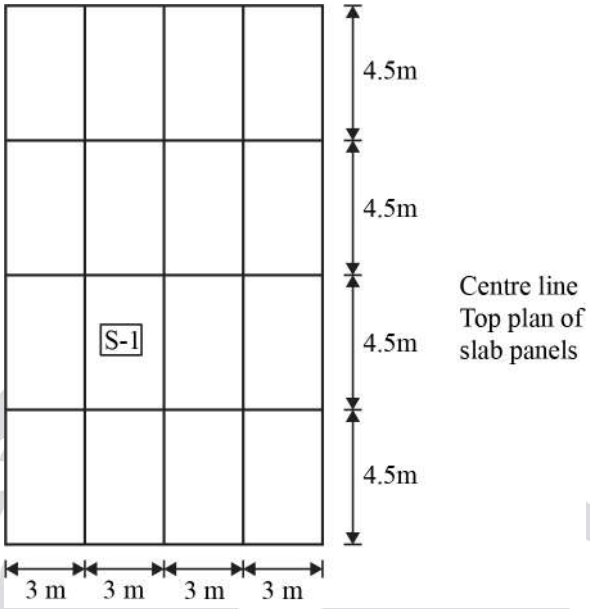
Maximum shear stress, τ_{max} = ± |3| MPa (in plane)

Normal stress on τ_{max} plane, σ' = 275 MPa

Angle of major principal plane = $\frac{51.2^o}{2}$

= 25.6°

(b). A solid RCC slab is having effect span of L_x = 3.0 m and L_y = 4.5 m. Position of slab S-1 is shown in the figure below.



Slab carrying total design load of 20 kN/m^2. In slab S-1, calculate and provide the spacing of all main reinforcement only, by limit state method of design, for 8 mm diameter high strength deformed bars. Check these spacings must not exceed the standard guidelines of IS 456:2000.

Effective thickness of slab = 120 mm

Grade of concrete M20

Grade of reinforcement Fe 415

$\frac{M}{bd^2}$	0.30	0.40	0.512	0.60	0.65	0.662
p_t	0.085	0.114	0.143	0.172	0.187	0.203

p_t is the percentage of reinforcement

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Table: Bending Moment Coefficients for Rectangular Panels Supported on Four Sides with Provision for Torsion at Corners (Clauses D.1.1 and 24.4.1)

Case No.	Type of Panel and Moments Considered	Short Span Coefficients α_x (Values of l_y/l_x								Long Span Coefficients α_x for all Values of l_y/l_x
		1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	**Interior Panels:** Negative moment at continuous edge	0.032	0.037	0.043	0.047	0.051	0.053	0.060	0.065	0.032
	Positive moment at mid-span	0.024	0.028	0.032	0.036	0.039	0.041	0.045	0.049	0.024
9	**Four Edges Discontinuous:** Positive moment at mid-span	0.056	0.064	0.072	0.079	0.085	0.089	0.100	0.107	0.056

(20 M)

Sol: For panel of slab S_1

$$\text{Aspect ratio} = \frac{L_y}{L_x} = \frac{4.5}{3.0} = 1.5 < 2$$

$\therefore$ it is a two way slab

Given effective thickness of slab d = 120 mm

Assume effective cover = 30 mm

Overall thickness of slab D = 120+30 = 150 mm.

Given that Total design load

$\omega_u = 20\ KN/m^2$

Moment co-efficients

$-\alpha_x = 0.053$ $-\alpha_y = 0.032$

$+\alpha_x = 0.041$ $+\alpha_y = 0.024$

Design moments

Support moments:

$M_{ux(-)} = (-)\ \alpha_x\ \omega_u L_x^2 = 0.053 \times 20 \times 3^2 = -9.54$ kNm

$M_{uy(-)} = (-)\ \alpha_y\ \omega_u L_x^2 = 0.032 * 20 * 3^2 = -5.76$ kNm

Span moments

$M_{ux(+)} = (+)\ \alpha_x\ \omega_u L_x^2 = 0.041 * 20 * 3^2 = +7.38$ kNm

$M_{uy(+)} = (+)\ \alpha_x\ \omega_u L_x^2 = 0.024 * 20 * 3^2 = +4.32$ kNm

Design of reinforcement

Minimum reinforcement required = 0.12%

$$= \frac{0.12}{100} \times bD = \frac{0.12}{100} \times 1000 \times 150 = 180\,mm^2$$

Negative moment reinforcement:

$$\frac{M_{ux}}{bd^2}(-) = \frac{9.54 \times 10^6}{1000 * 120^2} = 0.662 \rightarrow p_t = 0.203$$

$$p_t = \frac{100 A_{stx}(-)}{bd} = 0.203 \rightarrow Ast_{x(-)} = 243.6mm^2 \qquad \therefore \text{ Hence provide } A_{st} = 243.6mm^2$$

$$\frac{M_{uy}}{bd^2}(-) = \frac{5.76 \times 10^6}{1000 \times 120^2} = 0.4 \rightarrow p_t = 0.114$$

$$P_t = \frac{100 A_{sty}(-)}{bd} = 0.114 \rightarrow A_{sty}(-) = 136.8mm^2 \qquad \therefore \text{ Hence provide } A_{st} = A_{st,min} = 180mm^2$$

Positive moment reinforcement

$$\frac{M_{ux}(+)}{bd^2} = \frac{7.38 * 10^6}{1000 * 120^2} = 0.512 \rightarrow p_t = 0.143$$

$$P_t = \frac{100 A_{stx}(+)}{bd} = 0.143 \rightarrow A_{stx}(+) = 171.6mm^2 \qquad \therefore \text{ Hence provide } A_{st} = A_{st,min} = 180mm^2$$

$$\frac{M_{uy}(+)}{bd^2}=\frac{4.32\times10^6}{1000\times120^2}=0.3\rightarrow p_t=0.085$$

$$P_t=\frac{100A_{sty}(+)}{bd}=0.085\rightarrow A_{sty}(+)=102mm^2$$

Spacing of negative moment reinforcement

$$S_x(-)=1000\frac{a_{st}}{A_{stx}(-)}=1000\times\frac{\frac{\pi}{4}\times8^2}{243.6}=206mm$$

$$S_y(-)=1000\frac{a_{st}}{A_{sty}(-)}=1000\times\frac{\frac{\pi}{4}\times8^2}{136.8}=367mm$$

Spacing of positive moment reinforcement

$$S_{x(+)}=1000\frac{a_{st}}{A_{stx}(+)}=1000\times\frac{\frac{\pi}{4}\times8^2}{171.6}=293mm$$

$$S_{y(+)}=1000\frac{a_{st}}{A_{sty}(+)}=1000\times\frac{\frac{\pi}{4}\times8^2}{102}=492mm$$

Max spacing of main reinforcement in a slab

$$\left.\begin{matrix}(i)\,3d=3*120=360mm\\(ii)\,300mm\end{matrix}\right\}\text{small}$$

$S_x(-)$ = 206 mm < 300 mm ∴ ok

$S_y(-)$ = 367 mm > 300 mm ∴ Not ok

$S_x(+)$ = 293 mm < 300 mm ∴ ok

$S_y(+)$ = 492 mm > 300 mm ∴ Not ok

Minimum reinforcement = $0.12\% = \frac{0.12\%}{100}\times b\times D$

$$=\frac{0.12}{100}\times1000\times150=180\ mm^2$$

Hence this is the reinforcement to be adopted for positive moment along short span, long span and negative moment along long span. (M_{uy}).

$$\text{spacing} = \frac{1000 \times \frac{\pi}{4} \times 8^2}{180}$$

$$= 279 \text{ mm}$$

Adopt spacing of 275 mm

Adopt:

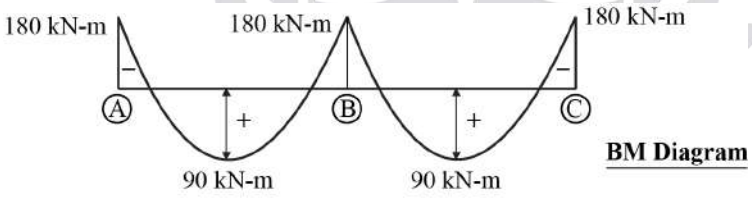
$Ast_x(-)$: 8 mm ϕ @ 200 mm c/c

$Ast_y(-)$: 8 mm ϕ @ 275 mm c/c

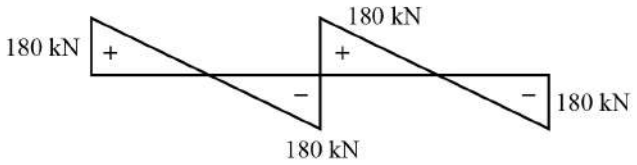
$Ast_x(+)$: 8 mm ϕ @ 275 mm c/c

$Ast_y(+)$: 8 mm ϕ @ 275 mm c/c

(c). Design Bending moment and Shear force diagram have been given below for a two span continuous beam. Effective span of beam is 6.0 m each. Design a rectangular, singly reinforced RCC beam section at support 'B' only by LSM of design.



BM Diagram



SF Diagram

The following parameters may be used for design,. All notations are as per IS 456:2000.

1. Grade of concrete M20)
2. Grade of reinforcement Fe 415
3. Nominal cover to reinforcement – 25 mm
4. (Effective depth/width) ratio – 2
5. Diameter of flexural bar – 20 mm
6. Diameter of shear reinforcement – 8 mm (CS Area 50 mm^2)
7. Design shear strength of concrete τ_c= 0.88 N/mm^2
8. Maximum shear stress $\tau_{c\ max}$ = 2.8 N/mm^2
9. Round off effective (calculated) to next higher multiple of 100
10. $M_{u,\ lim}/f_{ck}\ bd^2 = 0.138$
11. $(x/d) = 1.2 - \left[(1.2)^2 - \dfrac{6.68M_u}{f_{ck}bd^2}\right]^{1/2}$
12. Lever arm $z = d\left(1 - 0.42\dfrac{x}{d}\right)$
13. $A_{st} = \dfrac{M_u}{0.87f_y.z}$
14. $V_{us} = \dfrac{0.87f_y.A_{sv}.d}{S_v}$
15. Minimum shear reinforcement = $\dfrac{A_{sv}}{b.S_v} = \dfrac{0.40}{0.87f_y}$

Check all spacing of reinforcement as per standard guidelines.

(20 M)

Sol: Design BM, M_u = 180 kNm (Hogging moment

Given that $\dfrac{d}{b} = 2$

Dimensions of the beam

$M_u = M_{u\ limit}$

$180 \times 10^6 = 0.138\ f_{ck}\ bd^2$

$180 \times 10^6 = 0.138 \times 20 \times \left(\frac{d}{2}\right)\left(d^2\right)$ (For $M_{20} \rightarrow f_{ck} = 20\ N/mm^2$)

$d = 507\ mm \approx 600\ mm$

overall depth of beam, $D = d + 25 + 8 + \frac{20}{2}$

$= 600 + 25 + 8 + \frac{20}{2}$

$= 643\ mm \approx 650\ mm$

Width of beam, $b = \frac{d}{2} = \frac{600}{2} = 300\ mm$

Adopt D = 650 mm

d = 600 mm > 507 mm ∴ U.R.S

b = 300 mm

Area of tension reinforcement required

$$A_{st} = \frac{0.5\ f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6M_u}{f_{ck}bd^2}}\right]bd$$

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 180 \times 10^6}{20 \times 300 \times 600^2}}\right]300 \times 600$$

$A_{st} = 931.3\ mm^2$

Area of minimum tension reinforcement, $\frac{A_s}{bd} = \frac{0.85}{f_y}$

$$\frac{A_s}{300 \times 600} = \frac{0.85}{415}$$

$A_s = 368.67\ mm^2 < 931.3\ mm^2$ o.k

Area of maximum tension reinforcement

$0.04bD = 0.04 \times 300 \times 650 = 7800\ mm^2 > 931.3\ mm^2$ ∴ o.k

Number of 20 mm ϕ bars required

$$n = \frac{A_{st}}{a_{st}} = \frac{931.3}{\frac{\pi}{4} \times 20^2} = 2.96 \approx 3$$

Provide 3 – 20 mm ϕ of Fe – 415

Design for shear force

$V_u = 180 + 180 = 360$ kN

Nominal shear stress

$$\tau_v = \frac{V_u}{bd} = \frac{360 \times 10^3}{300 \times 600} = 2 \text{ N/mm}^2$$

$\tau_v < \tau_{cmax}$ $\therefore$ o.k

$\tau_v < \tau_c$ $\therefore$ not safe in shear

Hence design the shear reinforcement

Design shear force for stirrups

$V_{us} = V_u - \tau_c bd = 360 \times 10^3 - 0.88 \times 300 \times 600$

$= 201.6$ kN

Assume 2 legged stirrups

Spacing required for vertical stirrups

$$V_{us} = 0.87 f_y A_{sv} \frac{d}{s_v}$$

$$201.6 \times 10^3 = 0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 8^2 \times \frac{600}{s_v}$$

$s_v = 108$ mm

spacing required for shear reinforcement

$$\frac{A_{sv}}{b.s_r} \geq \frac{0.4}{0.87 f_y}$$

$$\frac{2 \times \frac{\pi}{4} \times 8^2}{300 \times s_v} \geq \frac{0.4}{0.87 \times 415}$$

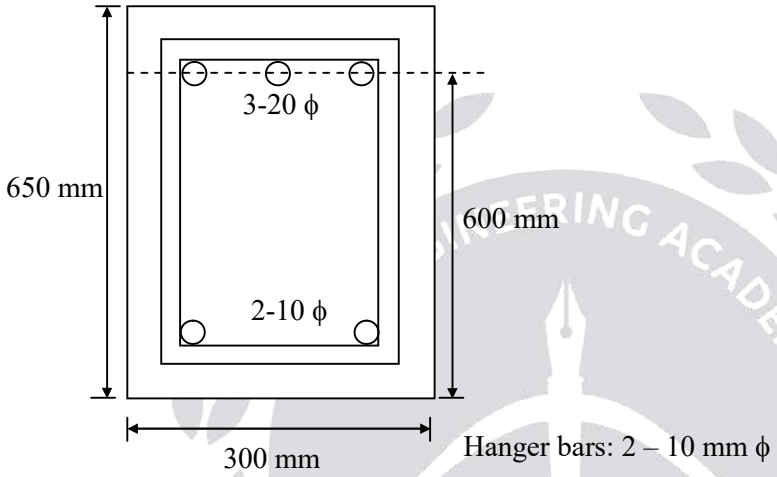
$s_v \leq 302$ mm > 108 mm $\therefore$ o.k

Maximum spacing for vertical stirrups.

$$\left.\begin{matrix} 0.75\,d = 0.75 \times 600 = 450\,mm \\ 300\,mm \end{matrix}\right\} \text{smaller}$$

$s_v = 108\ mm < 300\ mm$

adopt 2 legged 8 mm ϕ @ 100 mm c/c



04.

(a). (i). Briefly explain the following with the help of neat sketches.

I. Structure of an exogenous tree

II. Heat shakes and star shakes defects in timber.

(10 M)

Sol: The horizontal section of a typical exogenous tree has the following parts:

Pith: This is the inner most part or core having soft tissues. It is found near about the centre of log or a timber. Its size varies from 1.5 mm to 10 mm in dia.

Heart Wood: The inner part of the tree surrounding the pith is called Heart Wood. It imparts rigidity to tree and provides strong and durable timber for various engineering purposes.

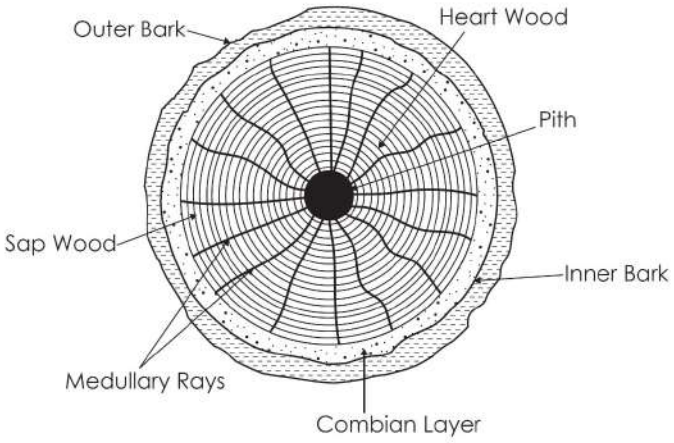
Sap Wood: It is the area between the heart wood and cambium layer. It contains living cells and takes active pan in growth of tree. It is Light in colour and weight.

Cambium Layer: It is the thin layer between sap wood and inner bark. It indicates sap which has yet to be converted into sap wood.

Inner Bark: It is the Inner skin or layer covering the cambium layer. It gives protection to the cambium layer.

Outer Bark: It is the outer most protective layer of the tree. It consists of cells of wood fiber and it is known as cortex.

Medullary Rays: These are thin radial fibers extending from pith to cambium layer. It holds together the annual rings of heart wood and sapwood. One ring is added every year, which decides the age of the tree.



Heart Shakes:

These are the cracks in timber which extend from pith to sapwood in the direction of medullary rays which occur due to shrinkage of interior parts of a tree.

Star Shakes:

These are the cracks in timber which extend from bark towards sap wood which occur due to extreme heat or severe frost during the growth of the trees.

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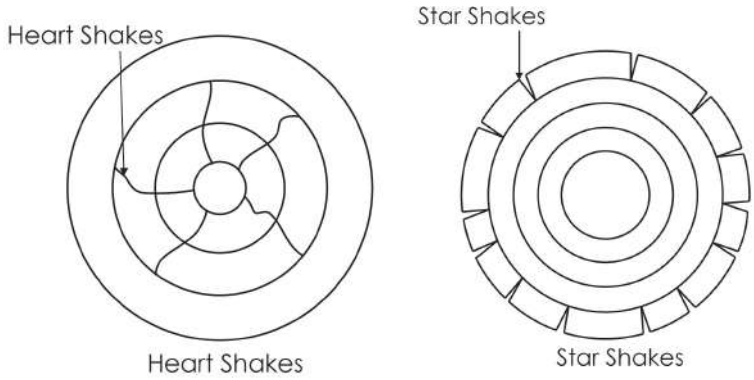


(ii). Determine the proportion of aggregates A (with Fineness Modulus FM = 7.83) and B (FM = 6.81) required to suitably combine to provide the following grading of such a combination.

IS sieve designation (mm)	80 mm	40 mm	20 mm	10 mm	4.75 mm	2.36 mm	1.18 mm	600 µm	300 µm	150 µm
Cumulative percentage retained on each sieve	0	6	45	72	95	100	100	100	100	100

(10 M)

Sol: By definition, Fineness Modulus (FM) = $\frac{\text{Sum of Cummulative percentage retained on each sieve}}{100}$

$$FM_{mix} = \frac{0+6+45+72+95+100+100+100+100}{100}$$

$$FM_{mix} = \frac{718}{100}$$

$$FM_{mix} = 7.18$$

Given, the fineness modulus of aggregate A is 7.83 and that of aggregate B is 6.81

Let the proportion of aggregate A in the mix = a

The proportion of aggregate B in the mix = b = 1-a

$FM_{mix} = FM_A \times a + FM_B \times b$

$7.18 = 7.83 \times a + 6.81 \times (1-a)$

$7.18 = 7.83a + 6.81 + 6.81a$

$1.02a = 0.37$

$a = 0.3627$ & $b = 1-a = 0.6373$

Therefore, the proportion of aggregate A in the mix is 36.27% and the proportion of aggregate B in the mix is 63.73%.

(b). A T-section beam is constructed by gluing two pieces of wood together as shown in the figure. The maximum stress in the glue joints is to be limited to 2 MPa in tension and the maximum shear stress is to be limited to 1.7 MPa

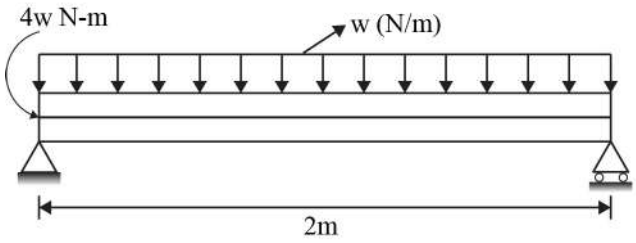
(i). Determine the stress components on element at point 'P'. Point 'P' is located at glued joint.

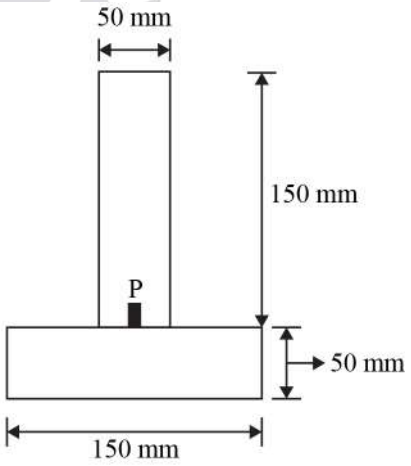
(ii). Determine the principal stresses at point 'P'.

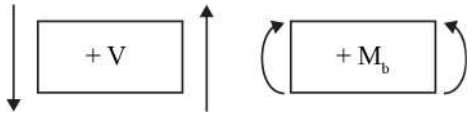
(iii). Show these stresses on properly oriented 2-D element.

(iv). Determine the maximum value for load w.

Sign convention for shear force and bending moment are as follows:

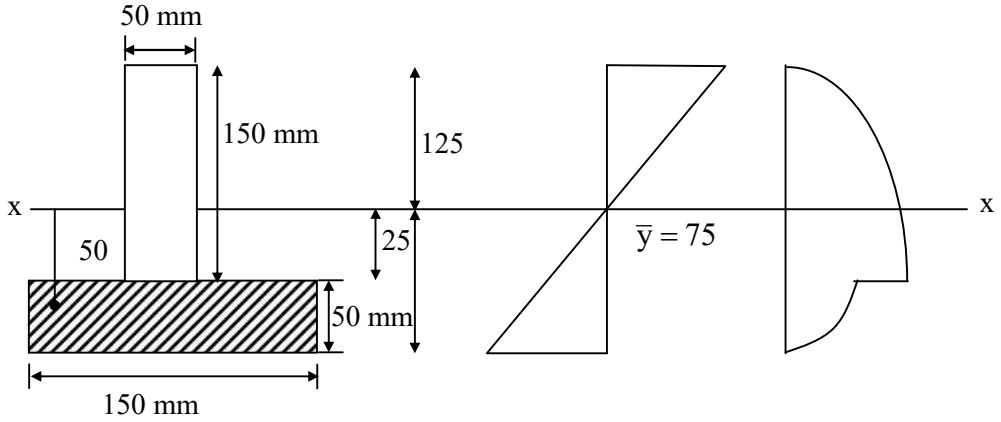


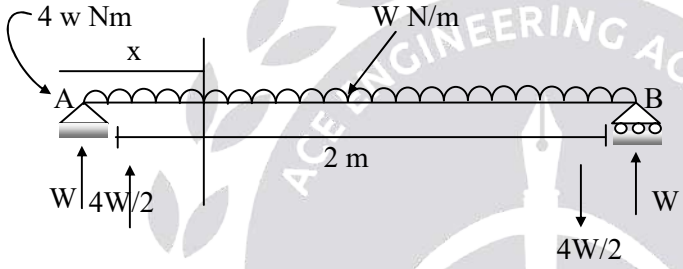




(20 M)

Sol:





$$R_A = \frac{w \times 2}{2} + 2w = 3w,$$

$$R_b = \frac{w \times 2}{2} - 2w = -w$$

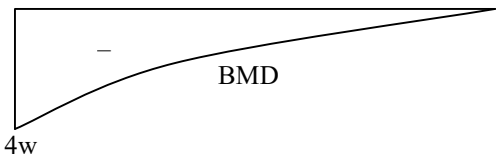
SFD

$$M_x = -4w + 3wx - \frac{wx^2}{2}$$

$M_x = 0$

$$7 - 4 + 3x\frac{-x^2}{2} = 0$$

$x = 2$ m



Hence, maximum SF = 3 wkN

Maximum BM = 4 w Knm

$$\bar{y} = \frac{150 \times 50 \times 25 + 50 \times 150 \times (50 + 75)}{50 \times 150 + 150 \times 50} = 75 \text{ mm}$$

$$I_{xx} = \frac{50 \times 150^3}{12} + 50 \times 150 \times 50^2 + \frac{150 \times 50^3}{12} + 150 \times 50 \times 50^2$$

$$= 53.125 \times 10^6 \text{ mm}^4$$

$(A)_{total} = 150 \times 50 + 150 \times 50 = 15000 \text{ mm}^2$

$$\tau_P = \frac{Va\,\bar{y}}{Ib} \Rightarrow \frac{3w \times 150 \times 50 \times 50}{53.12\delta \times 10^6 \times 50}$$

$\Rightarrow \tau_P \leq 1.7$

$$\Rightarrow \frac{3w \times 150 \times 50 \times 50}{53.125 \times 10^6 \times 50} \leq 1.7$$

$W \leq 4013$ N/m

$W \leq 4.013$ kN/m

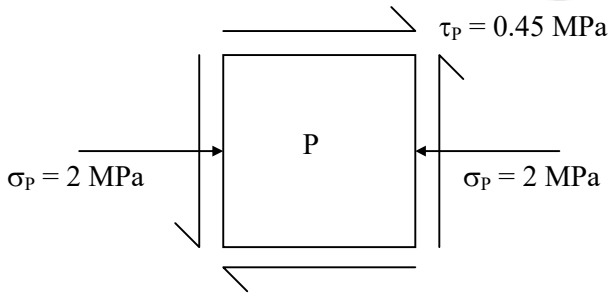
$$\sigma_P = \frac{M}{I} y_p$$

$$= \frac{4w \times 10^3 \times 25}{53.125 \times 10^6} \leq 2$$

$\Rightarrow w \leq 1.062$ kN/m

$\Rightarrow$ Therefore man value of w = 1.062 kN/m

I. Stress components on element at point P



$$\tau_p = \frac{3w \times 150 \times 50 \times 50}{53.125 \times 10^6 \times 50}$$

$= 0.45$ MPa

II. $\sigma_{max} / \sigma_{min} = \frac{\sigma_p}{2} \pm \sqrt{\left(\frac{\sigma_p}{2}\right)^2 + \tau_P^2}$

$$= \frac{2}{2} \pm \sqrt{\left(\frac{2}{2}\right)^2 + (0.45)^2}$$

$$= 1 \pm \sqrt{1 + 0.202\delta}$$

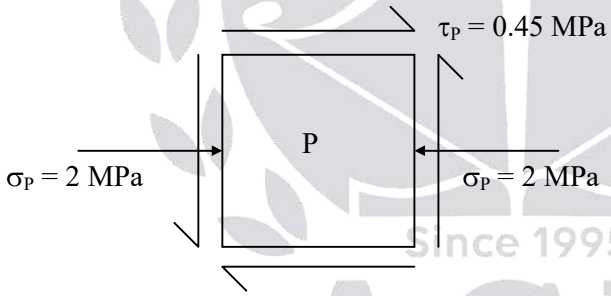
$= 1 \pm 1.0965$

$= 2.096$ MPa, 0.0965 MPa

$\sigma_{max} = 2.096$ MPa

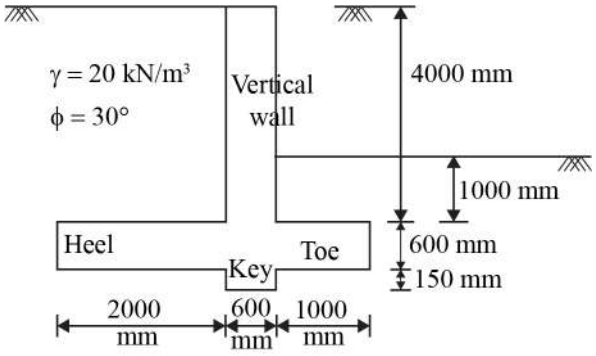
$\sigma_{min} = 0.0965$ MPa

III.



IV. $w = 1.062$ kN/m

(c). An RCC cantilever retaining wall is to be designed to support the soil as shown below. Design and sketch the reinforcement for vertical wall only. Also sketch the position of main reinforcement in Toe and Heel slab. (Do not design Toe and Heel slab). Neglect the effect of passive earth pressure and self-weight of vertical wall. Water table is not affecting the moisture condition of retained soil.



1. M20 grade of concrete
2. Fe 415 grade of reinforcement
3. Diameter of main and distribution reinforcement : 8 mm
4. Minimum effective thickness required: 400 mm
5. $K_a = \frac{1-\sin\phi}{1+\sin\phi}$

For M20 and Fe415

Percentage reinforcement P_t in %

$\frac{M}{bd^2}$	0.4	0.5	0.6	0.667
p_t	0.114	0.142	0.172	0.204

Design shear strength of concrete τ_c in N/mm^2

p_t	0.15	0.25	0.5	0.75
τ_c	0.28	0.36	0.48	0.56

(20 M)

Sol: $K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$

Moment at the base of stem

$$M = K_a \frac{\gamma H^3}{6} = \frac{1}{3} \times 20 \times \frac{4^3}{6} = 71.11 \text{ kN-m}$$

Design BM, $M_u = 1.5 \text{ m} = 1.5 \times 71.11 = 106.665$ kN-m

$$d = \frac{M_u}{R_{ub}} = \sqrt{\frac{106.665 \times 10^6}{0.138 \times 20 \times 1000}} = 196.58 \text{ mm}$$

Adept min effective thickness = 400 mm > 196.58 mm

$\therefore$ U.R.S

Area of main reinforcement – required

$$\frac{M_u}{bd^2} = \frac{106.665 \times 10^6}{1000 \times 400^2} = 0.667 \rightarrow P_t = 0.204$$

$$Pt = \frac{100 A_{st}}{bd} = 0.204$$

$$A_{st} = \frac{0.204 \times 1000 \times 400}{100} = 816 \text{ mm}^2$$

- Area of min tension reinforcement

  $$\frac{0.12}{100} bd = \frac{0.12}{100} \times 1000 \times 450 = 540 \text{ mm}^2 < 816 \text{ mm}^2$$

  $\therefore$ O.K

  (Assume effective cover = 50 mm)

- Area of max tension reinforcement

  $0.04bD = 0.04 \times 1000 \times 450 = 18000 \text{ mm}^2 > 816 \text{ mm}^2$

  $\therefore$ O.K

- Distribution reinforcement in a stem

  Let us provide min steel = 540 mm^2

  Spacing of main reinforcement in stem

$$S = 1000\ \frac{a_{st}}{A_{st}} = 1000 \times \frac{\frac{\pi}{4} \times 8^2}{816} = 61.6 \text{ mm}$$

Provide 8 mm ϕ @ 60 mm c/c

Spacing of distribution reinforcement in stem

$$S = 1000\ \frac{a_{st}}{A_{st}} = 1000 \times \frac{\frac{\pi}{4} \times 8^2}{540} = 93 \text{ mm}$$

Provide 8 mm & @ 90 mm c/c

Check for shear

$$\text{Shear force } V = Ka\ \frac{\gamma H^2}{2} = \frac{1}{3} \times 20 \times \frac{4^2}{2} = 53.33 \text{ kN}$$

Design SF $V_u = 1.5\ V = 1.5 \times 53.33 = 80$ kN

Nominal shear stress

$$\tau_V = \frac{V_u}{bd} = \frac{80 \times 10^3}{1000 \times 400} = 0.2 \text{ N/mm}^2$$

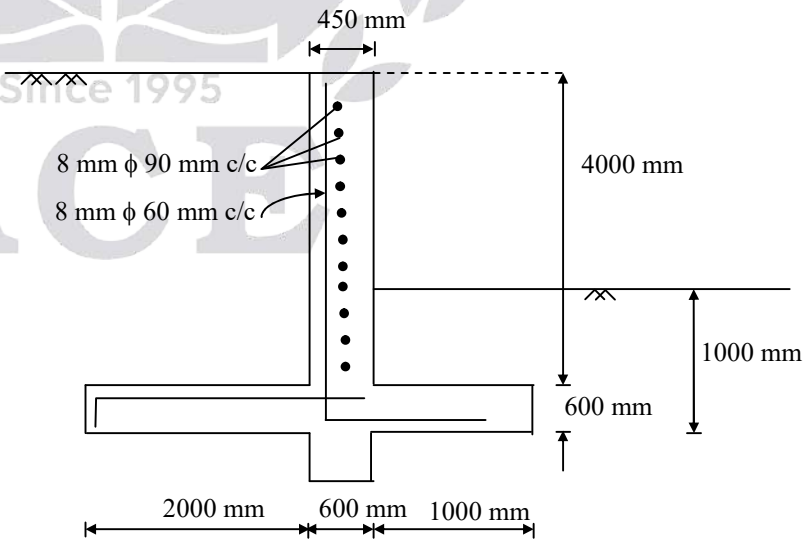
Shear resistance of concrete for $P_t = 204$

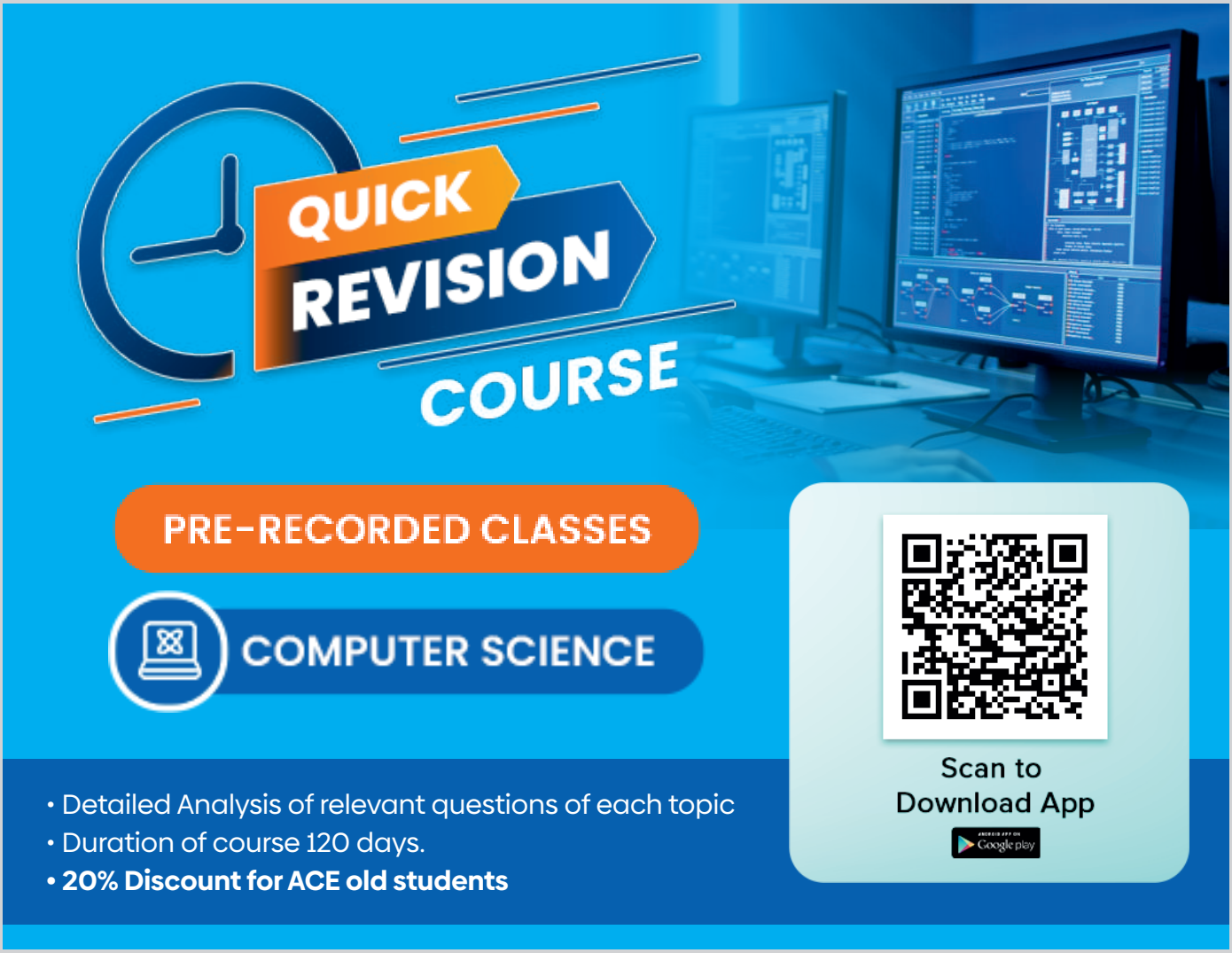
P_t	τ_c, N-mm^2
0.15	0.28
0.25	0.36

$$\tau_c = 0.28 + \frac{0.36 - 0.28}{0.25 - 0.15}\ (0.204 - 0.15)$$

$$= 0.32 \text{ N/mm}^2$$

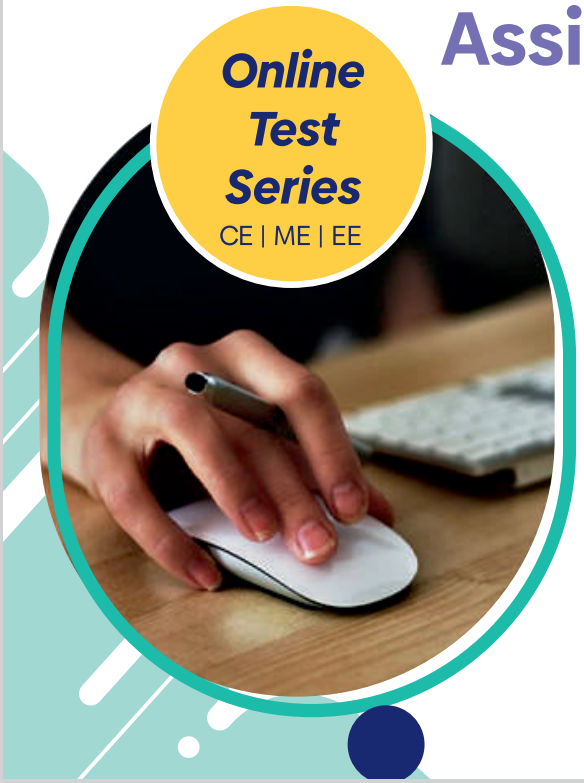
$\tau_V - \tau_C\ \therefore$ safe





UTTAR PRADESH PUBLIC SERVICE COMMISSION

Assistant Engineer-2021



Starts from: 1st Dec-2021

No. of Tests: **16**

Subject Wise: **12**

Mock Tests: **4**

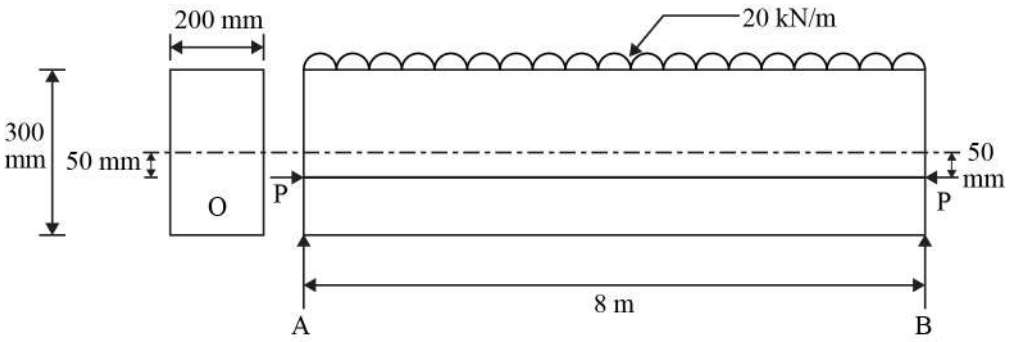
All tests will be available till UPPSC AE Examination

SECTION-B

05.

(a). A rectangular pre-stress concrete beam has a cross-section of 200 mm × 300 mm . Its effective span is 8 metres. This beam is pre-stressed by a straight cable, 50 mm below the central longitudinal axis. This beam supports an imposed load of 20 kN/m.

Find the magnitude of pre-stressing force which can balance the stresses due to dead load and imposed load at bottom fibre of mid-span. Unit weight of concrete : 25 kN/m^3.



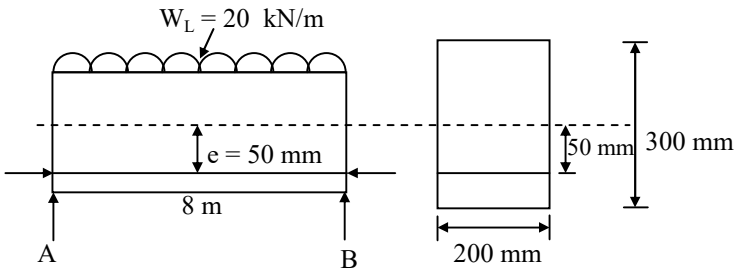
(12 M)

Sol: $w_d = 25 \times 0.2 \times 0.3 = 1.5$ kN/m

$w_L = 20$ kN/m

$w = 20 + 1.5 = 21.5$ kN/m

Now,



$$\sigma_B = \frac{P}{A} + \frac{Pe}{z} - \frac{M}{z}$$

$A = 200 \times 300\ mm^2$

$$z = \frac{200 \times 300^2}{6}\ mm^2$$

$$M = \frac{w\ell^2}{8} \text{ (at centre)}$$

$$= \frac{21.5 \times 8^2}{8} = 172 \text{ kNm}$$

Now , to balance the stress at bottom fibre

$\sigma_B = 0$

$$\Rightarrow \frac{P}{200 \times 300} + \frac{P \times 50 \times 6}{200 \times 300^2} = \frac{172 \times 10^6 \times 6}{200 \times 300^2}$$

$$\Rightarrow P + \frac{P \times 50 \times 6}{300} = \frac{172 \times 10^6 \times 6}{300}$$

$2P = 3440$ kN

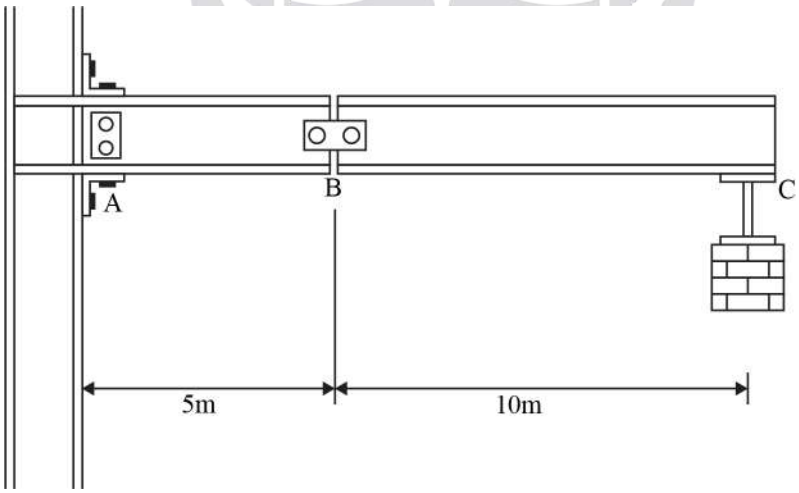
$P = 1720$ kN

(b) The compound beam is subjected to a uniform dead load of 1.5 kN/m and a single live load of 10 kN. Determine

i. The maximum negative moment created by these loads at 'A', and

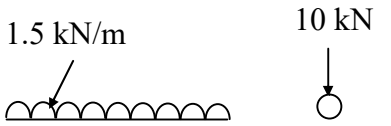
ii. Maximum positive shear at 'B'.

Assume A is a fixed support, B is a pin and C is a roller.

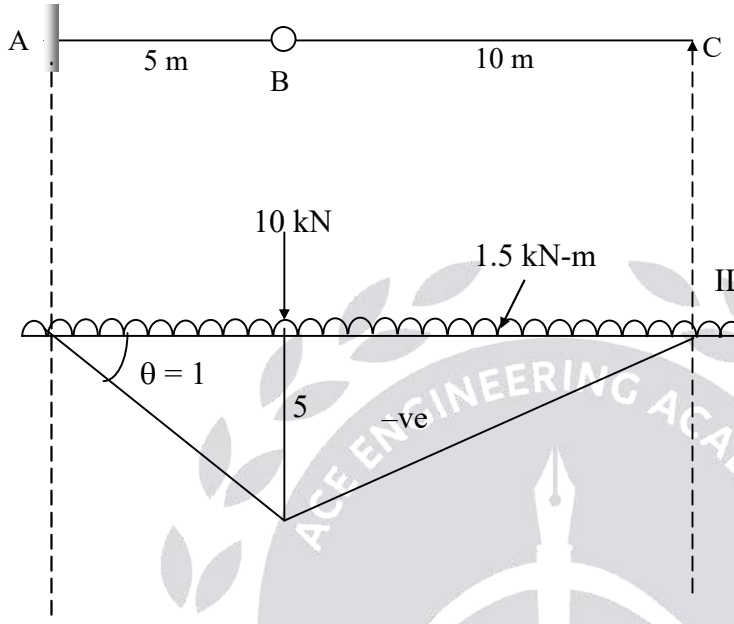


(12 M)

Sol:



(i)



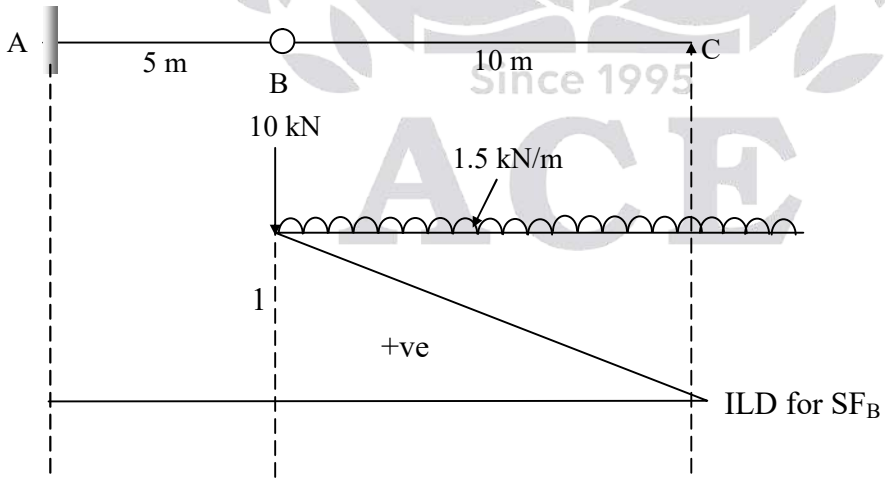
Maximum negative BM at A

$$= -1.5\left[\frac{1}{2}\times 5\times 15\right] - 10\times 5$$

$$= -56.25 - 50$$

$$= -106.25 \text{ kN m}$$

(ii)



Maximum +ve SF at B'

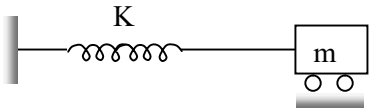
$$= +1.5\left[\frac{1}{2}\times 10\times 1\right] + 10\times 1$$

$$= 17.5 \text{ kN}$$

(c). Drive the expression for displacement of an un-damped free vibration of motion for a single degree of freedom system from first principles. Plot the un-damped free vibration response.

(12 M)

Sol: Consider a mass 'm' on a free surface attached to a wall through a spring.



Where the system is disturbed by a force (say F) to start the vibrations, the spring generates force (F_K) that tends to pull the system to equilibrium.

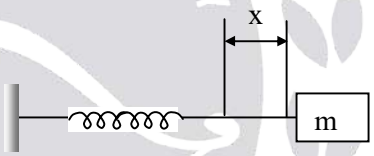
$F_K = Kx$

$F = ma$

$= m\ddot{x}$

To be in equilibrium

$F_k + F = 0$



$\Rightarrow m\ddot{x} + kx = 0$

$\Rightarrow \ddot{x} + \frac{k}{m}x = 0$

But $\sqrt{\frac{K}{m}} = \omega_n$ (natural frequency)

$\therefore \quad \ddot{x} + \omega_n^2 x = 0$

Solution of the differential equation

$x = A \sin \omega_n t + B \cos \omega_n t$

Let at time $t = 0$, $x = x_0$

$V = V_0$

$\therefore \quad x(0) = A\sin(0) + B\cos(0) = x_0$

$\Rightarrow B = x_0$

$\dot{x}(0) = A\,\omega_n \cos(\omega_n t) - B\,\omega_n \sin \omega_n t = V_0$

$= A\,\omega_n\cos(0) - B\,\omega_n\sin(0) = V_0$

$\Rightarrow A = \dfrac{V_0}{\omega_n}$

$\therefore \qquad x(t) = \dfrac{V_0}{\omega_n}\sin\omega_n t + x_0\cos\omega_n t$

Let $\sin\phi = x_0$; $\cos\phi = \dfrac{V_0}{\omega_n}$

i.e. $\tan\phi = \dfrac{x_0\omega_n}{V_0}$

$C = \sqrt{x_0^2 + \left(\dfrac{V_0}{\omega_n}\right)^2}$

$\Rightarrow x(t) = C[\sin\omega_n t\cos\phi + \cos\omega_n t\sin\phi]$

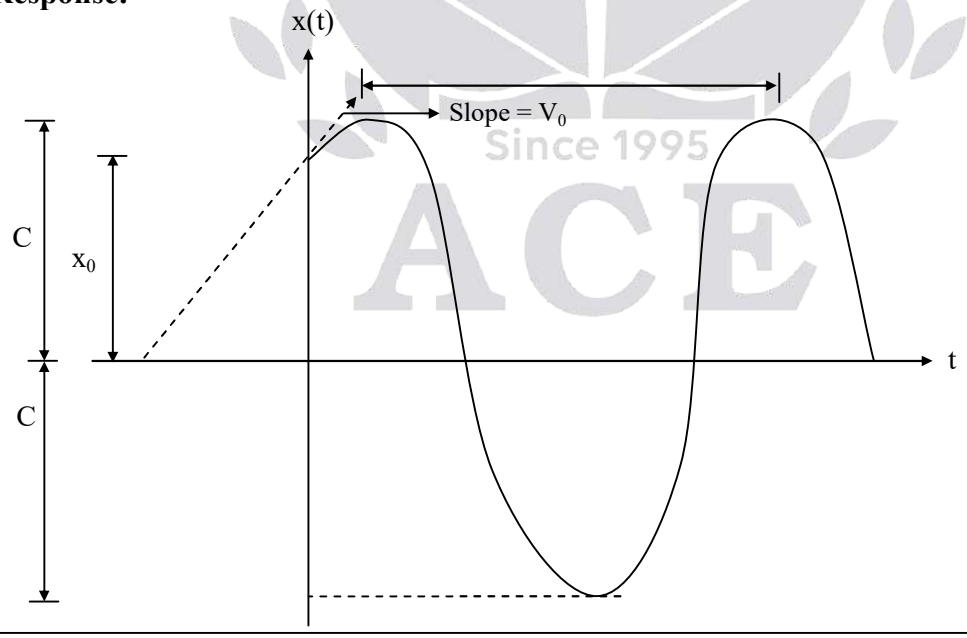
$= (\sin(\omega_n t + \phi))$

Where ω_n is the natural frequency

C is amplitude

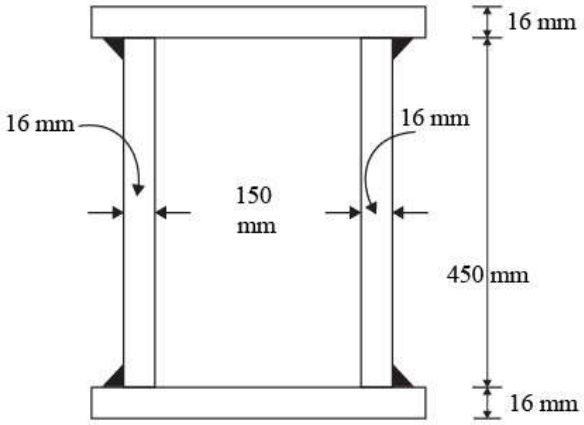
ϕ is the phase angle

Response:



(d). Calculate the safe load of a compression member made up to a welded box section as shown in the figure. The effective lengths along major axis and minor axis are 5 m and 2m respectively. The yield stress of the material is 340 MPa.

$\frac{KL}{r}$	Design compressive Stress f_{cd} (MPa)
20	299
30	278
40	256



(12 M)

Sol: Given:

Effective length along major axis = 5 m

Effective length along minor axis = 2m

Yield stress of material = 340 MPa

Now,

I_{zz} = MOI about z-z axis

$$= \frac{1}{12}\times 182\times 482^3 - \frac{1}{12}\times 150\times 450^3$$

$$= 559.303 \times 10^6 \text{ mm}^4$$

I_{yy} = MOI about y-y axis

$$= \frac{1}{12}\times 482\times 182^3 - \frac{1}{12}\times 450\times 150^3$$

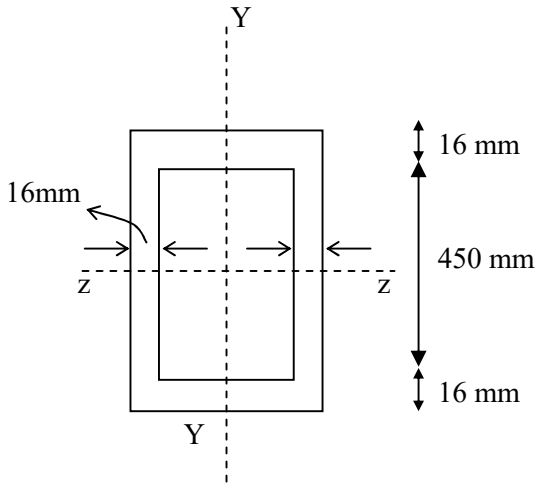
$I_{yy} = 115.584 \times 10^6 \text{ mm}^4$

Hence, z-z is major axis

y-y is minor axis

A_g = gross area = A

$= 482 \times 182 - 450 \times 150 = 20224 \text{ mm}^2$



Now, calculating radius of gyration about major and minor axis

$$r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{115.564\times10^6}{20224}} = 75.59 \text{ mm}$$

$$r_{zz} = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{559.303\times10^6}{20224}} = 116.3 \text{ mm}$$

Calculating slenderness ratio

$$\left(\frac{KL}{r}\right)_{zz} = \frac{5000}{166.3} = 30$$

$$\left(\frac{KL}{r}\right)_{yy} = \frac{2000}{75.59} = 26.458$$

Choosing the maximum of the above two values i.e. $\frac{KL}{r} = 30$, hence $f_{cd} = 278$ MPa (Given)

Factored load = $f_{cd} \times A_g$

$= 278 \times 20224 = 5622.272$ kN

Safe/working load = $\frac{5622.272}{1.5} = 3748.181$ kN

(e). (i) . What information is generally needed to be provided in tender documents? **(6 M)**

Sol: The following documents/details need to be provided in Tender documents

i. Name of the authority inviting the bids/tenders.
ii. Name of the project and brief details of the project
iii. Conditions for eligibility of contracting agencies to submit a bid.
iv. Estimated cost and time of completion of the project.
v. Technical specifications and drawings.
vi. Earnest money to be deposited with the completed tender.
vii. The date and time by which the bids are to be submitted and the place of submission.
viii. Terms and conditions like general conditions of contract (GCC) and special conditions of contract.

Bid documents are to be prepared in the following volumes

Volume		Bid Documents
I	:	Notice Inviting Tender (NIT)
	:	Instructions to Bidders (ITB)
	:	General Conditions of Contract (GCC)
	:	Special Conditions of Contract (SCC)
II	:	Technical Specifications
III	:	Forms of Bid
	:	Bill of Quantities
	:	Bid Security, Performance Security Forms
	:	Schedules for Supplementary Information
	:	Sample Forms for updating qualification data
IV	:	Drawings
V	:	Documents to be furnished by the bidder

(ii). Differentiate between 'Lump sum contract' and 'Unit price contract'. **(6 M)**

Sol:

Lump-sum Contract	Unit Price (or) Item rate Contract
1. The lump-sum amount refers to the total sum of money for which the contractor agrees to build the required facility with all possible risks.	1. In this contract, the tender document contains a detailed bill of quantities (BOQ) and the contractor has to specify the unit price (or) item rate against each item
2. The owner/client knows before hand exactly what the work will cost.	2. The contractor agrees to carryout a unit quantity of a particular work. Hence, the total cost can be arrived after completion of the work.

3.	Detailed measurements of the work done are not required.	3.	Detailed measurements of the work done need to be considered.
4.	Easy to prepare a comparative statement.	4.	Difficult to prepare a comparative statement.
5.	Not suitable when the work has lot of additions or alterations.	5.	Suitable for all kinds of works, however 'BOQ' is required.
6.	Technical specifications and drawings are provided before the execution	6.	Tight inspection and supervision is required when the quantities are increased or decreased.

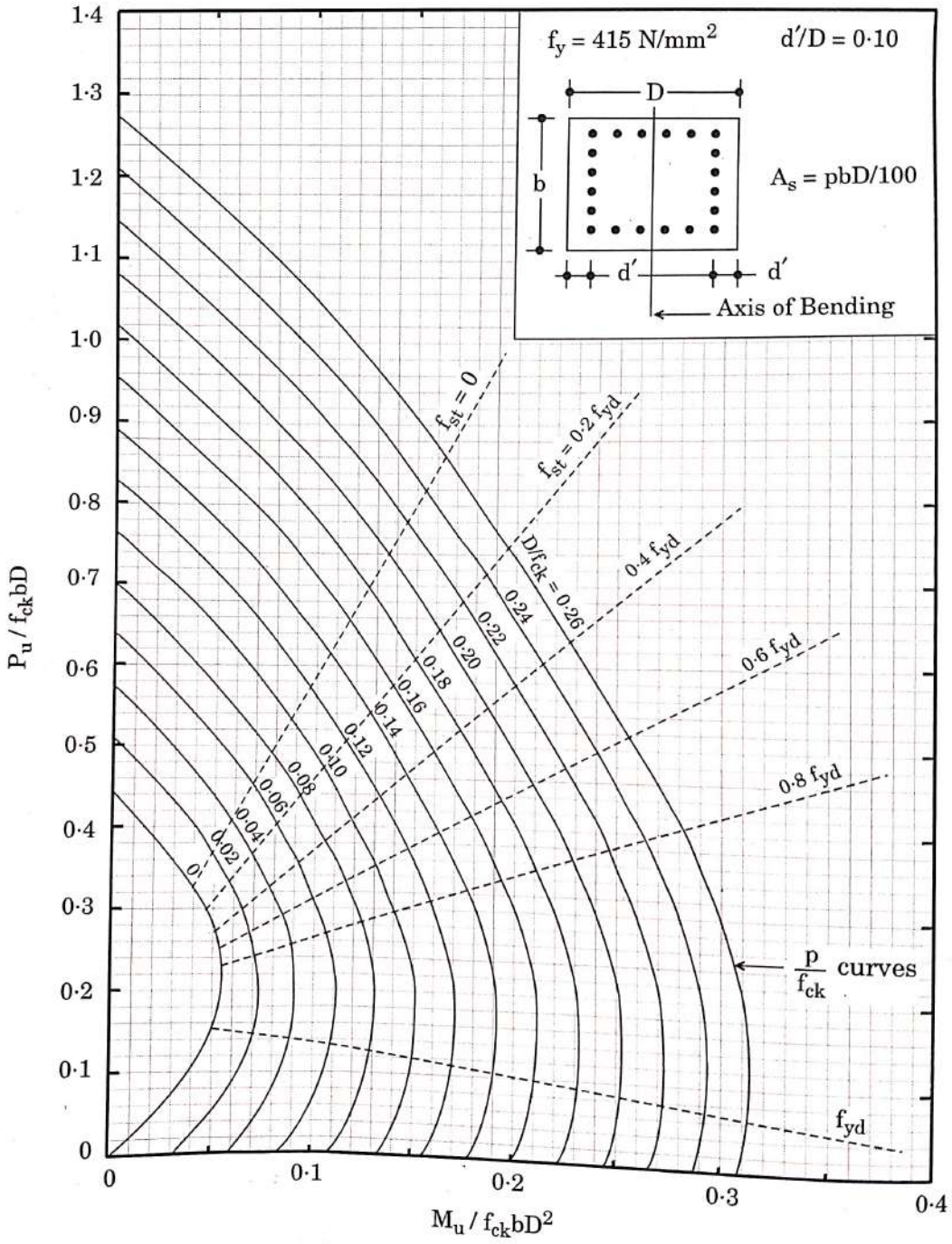
06.

(a). Design an RCC square column with the following data. Provide the main reinforcement and stirrups. Check the standard guidelines of IS 456 : 2000.

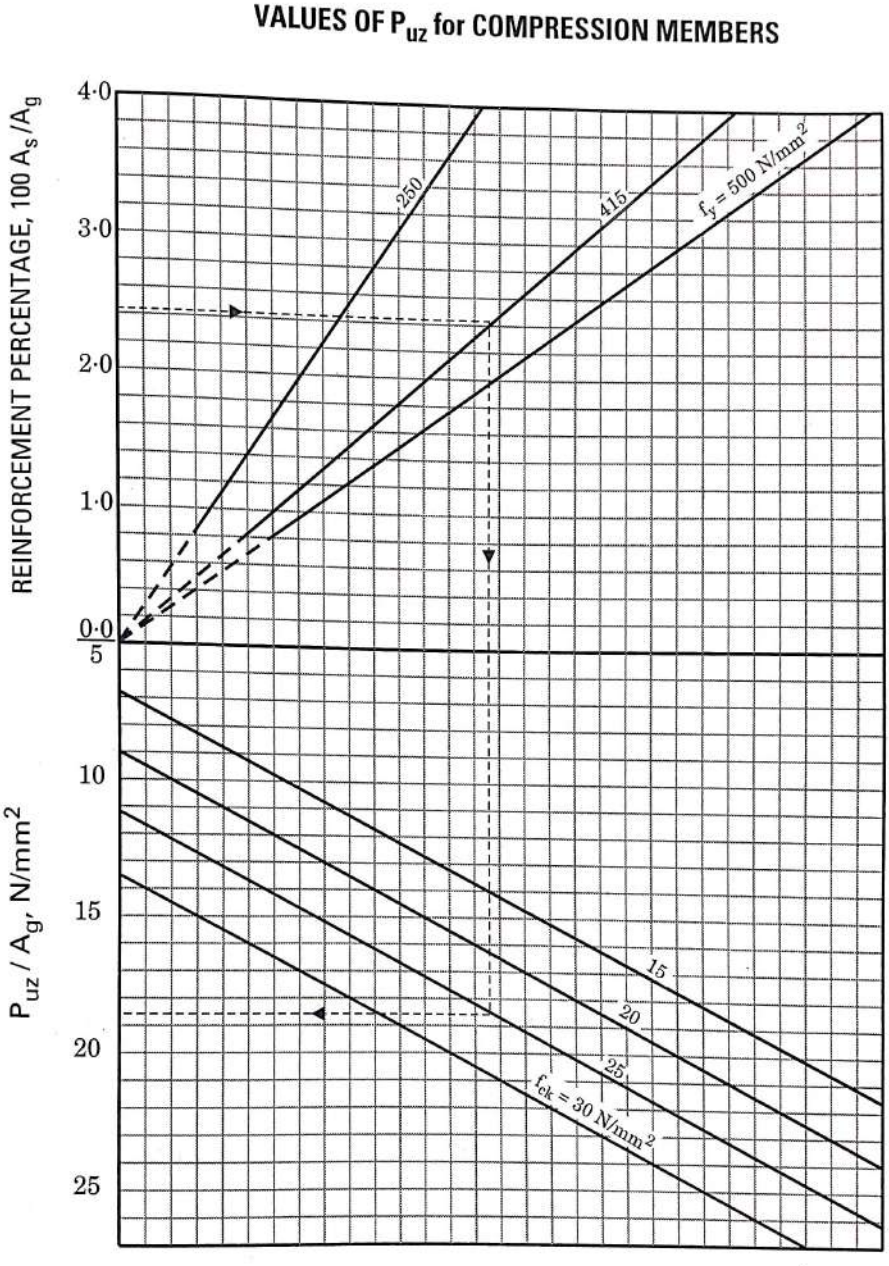
1. Column size : 500mm × 500 mm
2. Concrete grade M 20
3. Steel reinforcement Fe 415
4. Design load : 1600 kN
5. Design moment $M_{ux} = M_{uy} = 100$ kN
6. Eccentricities $e_x = e_y = 20$ mm
7. Minimum reinforcement : 1.2%
8. Check $\left[\frac{M_{ux}}{M_{ux_1}}\right]^{\alpha_n} + \left[\frac{M_{uy}}{M_{uy_1}}\right]^{\alpha_n} \le 1.5$, where $\alpha_n = \frac{P_u}{P_{uz}}$
9. Diameter of longitudinal reinforcement : 20 mm
10. Diameter of lateral ties : 8 mm
11. Nominal cover to reinforcement : 40 mm

Compression with Bending – Rectangular Section – Reinforcement Distributed equally on four sides.

COMPRESSION WITH BENDING – Rectangular Section – Reinforcement Distributed Equally on Four Sides



f_y
250
415
500
f_{ck}
15
20
25
30



(20 M)

Sol: effective cover $d' = 40 + 8 + \frac{20}{2} = 58\text{mm}$

$$\frac{d'}{D} = \frac{58}{500} = 0.11 \simeq 0.1$$

Given minimum percentage of steel p = 1.2%, Let us take 1.4%

$$\frac{p}{f_{ck}} = \frac{1.4}{20} = 0.07$$

$$\frac{p_u}{f_{ck} bD} = \frac{1600 \times 10^3}{20 \times 500 \times 500} = 0.32$$

From the chart

For $\frac{p}{f_{ck}} = 0.07$ and $\frac{p_u}{f_{ck} bD} = 0.32$

$$\frac{M_u}{f_{ck} bD^2} = 0.12$$

$M_{ux1} = 0.12\ f_{ck} bD^2 = 0.12 \times 20 \times 500 \times 500^2 = 300$ kNm

$M_{uy1} = M_{ux1}$ (b = D)

$P_{uz} = 0.45\ f_{ck}\ Ac + 0.75\ f_y\ Asc$

$$A_S = \frac{1.4}{100} \times 500 \times 500 = 3500\,\text{mm}^2$$

$Ac = (500 \times 500 - 3500) = 246500\ \text{mm}^2$

$P_{uz} = 0.45 \times 20 \times 246500 + 0.75 \times 415 \times 3500$

$= 3307.875$ kN

$$\frac{p_u}{P_{uz}} = \frac{1600}{3307.875} = 0.48$$

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The value of $\alpha_n = \frac{2}{3}\times\left(1+\frac{5}{2}\times 0.48\right) = 1.47$

$$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha n} + \left(\frac{Mu_y}{Mu_{y1}}\right)^{\alpha n} = \left(\frac{100}{300}\right)^{1.47} + \left(\frac{100}{300}\right)^{1.47} = 0.398 < 1 \therefore \text{ ok}$$

Hence the percentage of steel assumed is satisfactory

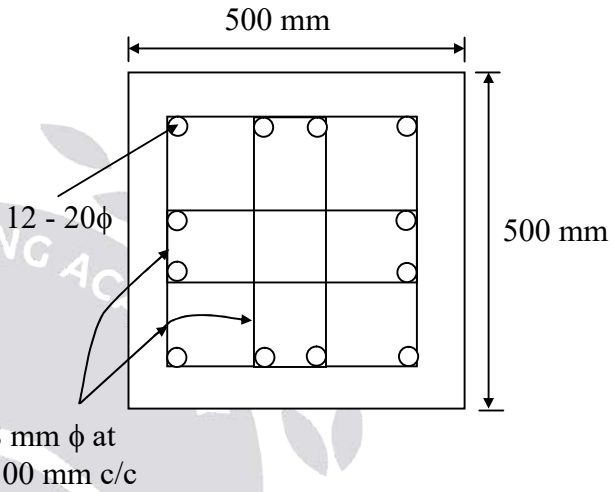
$$A_{SC} = \frac{1.4}{100}\times 500\times 500 = 3500 mm^2$$

No. of 20 mm ϕ bars required

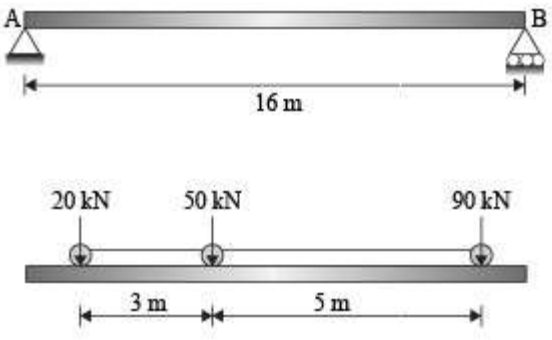
$$n = \frac{A_{SC}}{a_{sc}} = \frac{3500}{\frac{\pi}{4}\times 20^2} = 11.14 \approx 12 \simeq 10$$

Provide 12 – 20 mmϕ

Lateral ties 8 mmϕ @ 300 mm c/c

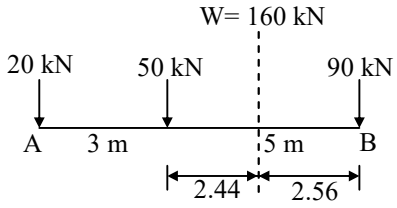


(b). Determine the absolute bending moment in a 16 m long beam AB due to the truck loading shown in the figure.



(20 M)

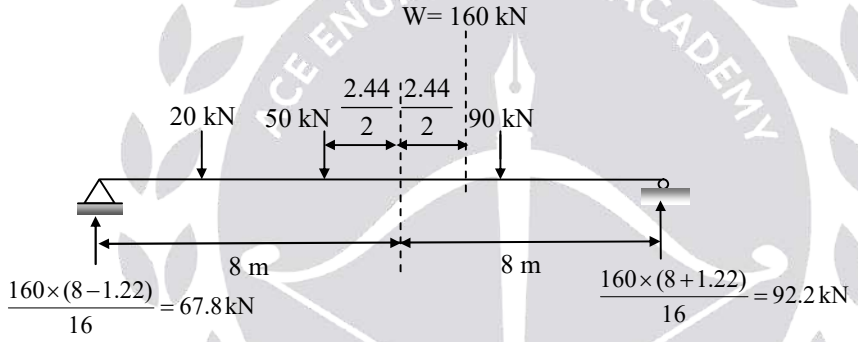
Sol:



$$\text{Centroid of Load from (A)} = \frac{(20)(0)+(50)(3)+(90)(8)}{20+50+90} = 5.4375 \text{ m} = 5.44$$

Total load (w) = 20 + 50 + 90 = 160 kN

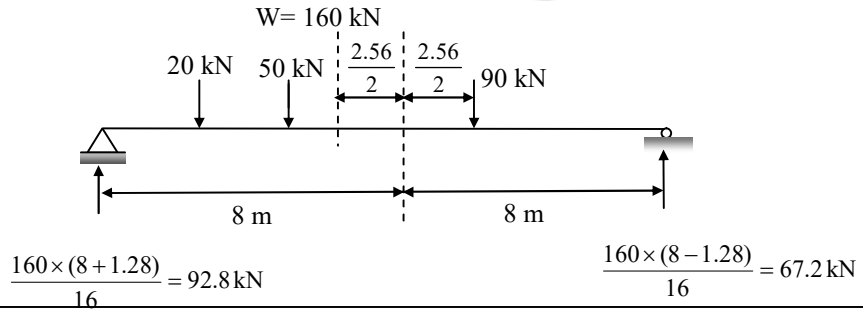
For absolute bending moment under 50 kN



$\therefore$ BM under 50 kN (from left support) = 67.8 (8 – 1.22) – 20 × 3

= 399.684 kNm

For absolute bending moment under 90 kN



BM under 90 kN (from right support) = 67.2 (8 – 1.28)

= 451.58 kNm

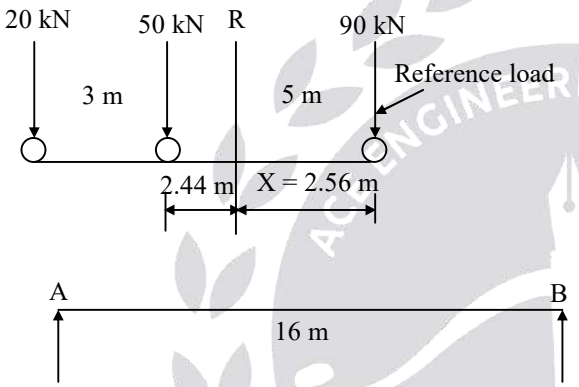
∴ Absolute bending moment = maximum (399.68, 451.58)

= 451.58 kNm

(or)

06.

(b)



Note: In this problem the section of the beam is not specified. We have to choose the location of maximum BM. Maximum BM occurs near center.

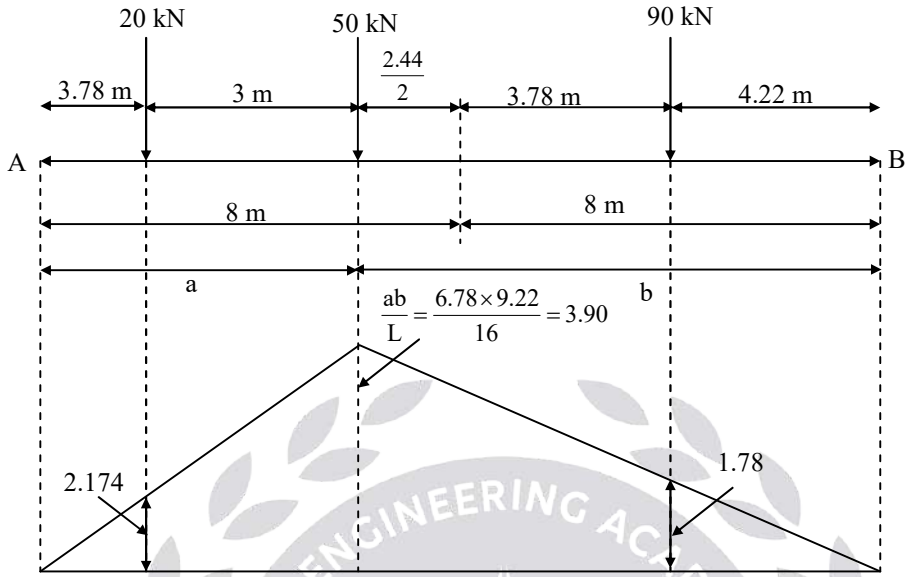
To get maximum BM under a chosen wheel load, the resultant of load system & the chosen load, must be at equal distance from centre. Then maximum BM occurs under the chosen load.

To locate the resultant (R), use varignon's theorem

$$\bar{x} = \frac{90\times 0 + 50\times 5 + 20\times 8}{90+50+20} = 2.56 \text{ m}$$

In this problem, wheel load is not chosen. We have to choose the wheel load under which maximum BM may happen. In this case 50 kN is nearer to the resultant but greater load is 90 kN is slightly away from the resultant. So we have to check the possibility of getting max BM under 50 kN and 90 kN

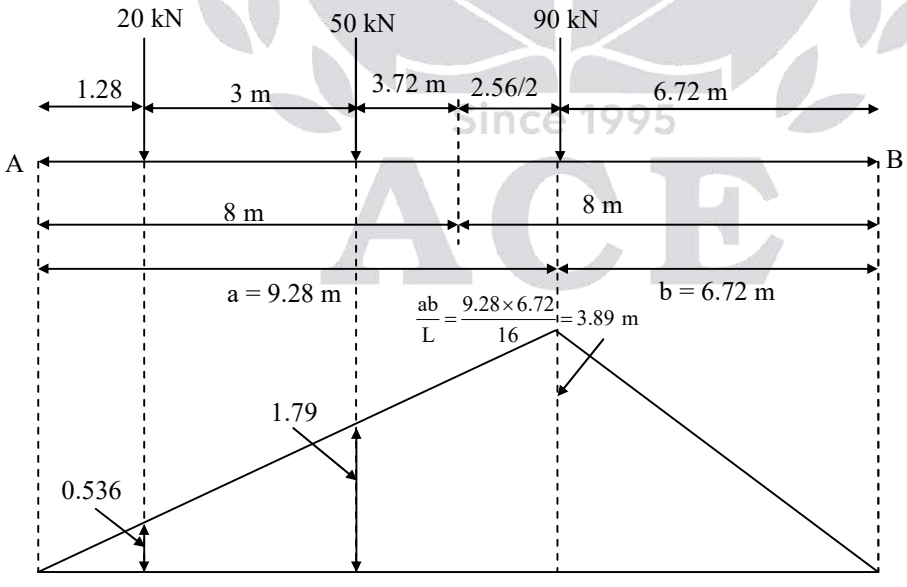
1st possibility: *BM under 50 kN load.*



Maximum BM under 50 kN $= 20 \times 2.174 + 50 \times 3.90 + 90 \times 1.78$

$= 398.68$ kN-m

2nd possibility: *BM under 90 kN load*



Maximum BM under 90 kN $= 20 \times 0.536 + 50 \times 1.79 + 90 \times 3.89$

$= 450.32$ kN-m

Maximum BM occur under 90 kN when it is at distance of 9.28 m from left support and magnitude of maximum BM is 450.32 kN-m

(c). (i). Briefly discuss various factors affecting the output of power shovel to excavate earth.

(10 M)

Sol: Factors affecting the output of power shovel:

1. **Angle of swing:** The horizontal angle between the place of the dipper or bucket, when it is digging and the place when discharging the soil load.The output of the power shovel is inversely proportional to the cycle time i.e., the angle of swing.
2. **Job conditions:** Job conditions can be classified as excellent, good, fair and poor depending on the conditions of the work site and climatic conditions.
3. **Optimum Depth of Cut:** The optimum depth of a cut is the depth which produces the greatest output and at which the dipper comes up with a full load without undue-crowding. The optimum depth of cut varies with class of soil material and the size of dipper or bucket.
4. **Management Conditions:** Excellent management conditions achieve maximum excavation production while poor can minimum yield.The small size of the hauling unit demands a small shovel size.A skilled operator can increases the output, by his skill and experience of manipulation of excavator. The physical condition of the shovel increases the production. The bad components subject to wear and tear.
5. **Site Condition:** The output performance of shovel depends on site conditions also which includes the physical conditions of site such as topography and geology of the site, geotechnical conditions of ground or rocks etc. Excavation might be tough for site containing hard soil. Rate of excavation may vary with respect to climatic changes also like temperature, rain snow etc.

6. **Cycle Time:** Cycle time is defined as the amount of time taken by machine to perform a repetitive segment of an operation, typically measured as the time it takes the machine to return to the same position. It is the time taken to complete one entire excavating process of an excavator which includes excavation time, time to swing to dumping position, dumping time and time to return to the digging position. Greater angle of swing results in greater cycle time which may lead the work to delay.

7. **Bucket Size:** A bucket or dipper is a bulk material handling component provided at the end of the dipper stick arm of an excavating machine. Selection of bucket size depends on the material to be excavated. The size of this bucket determines its capacity to excavate the material in one particular cycle. Thus, maximum capacity of bucket can save number of cycle and time required for excavation.

8. **Repairs and Maintenance of Equipment:** A regular maintenance is one of the most important factors in terms of cost and performance. Good maintenance will minimize maintenance and repair costs and maximize production and profits, with greatest impact on profit. It helps to control costs and service intervals, lengthens equipment life, minimizes downtime and adds resale value.

9. **Operator's Skill:** An operator plays a major role in working of any equipment. Proper use of equipment is what we need to obtain best result from the machine. A skilled or trained operator can handle the equipment in an effective manner providing maximum output from machine. Improper use equipment can cause damages to it and injuries to the operator as well. Thus, operator's skill is an important parameter to be considered.

(ii). What are the precautions to be taken for labour safety during excavation works?

(10 M)

Sol: Excavation work is the first activity on any construction site. It starts from digging the pit for the structure for either shallow or for deep foundations. It is completed by filling excavated soil or soil brought from outside in the same pit.

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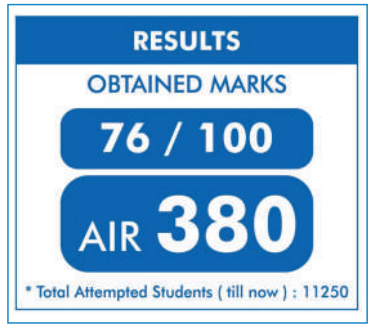
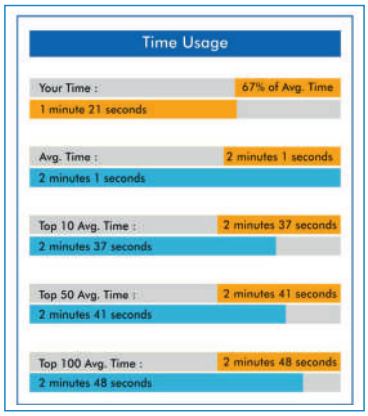
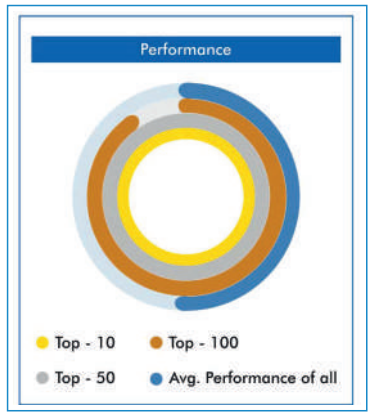
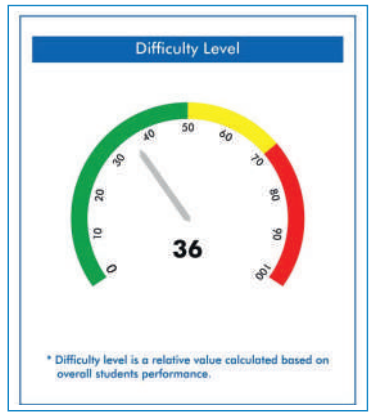


QUESTION WISE STATISTICS:







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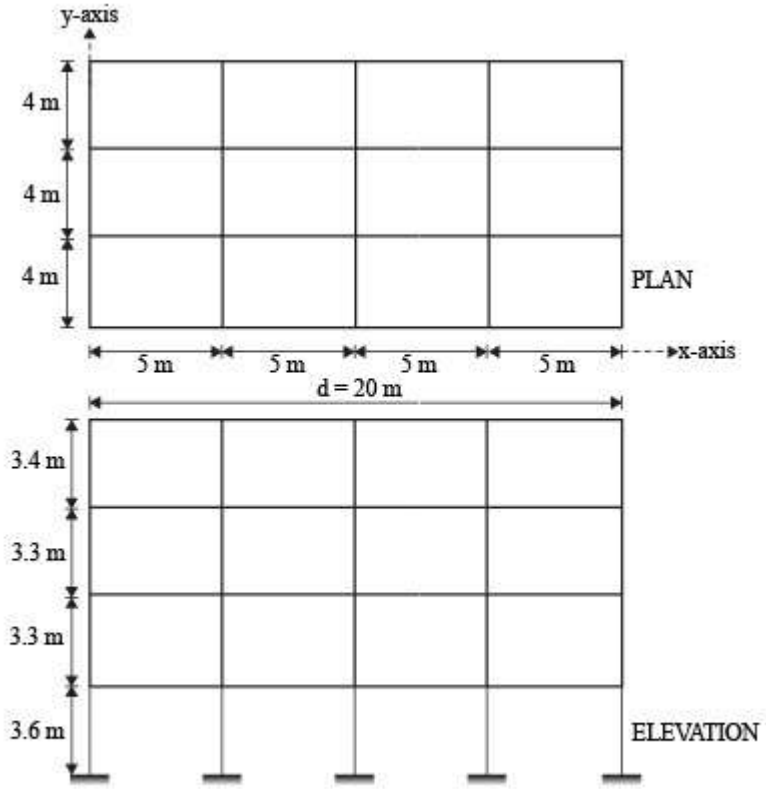
Safety Precaution for labour safety while Excavation Work:

- Prepare firm and broad approach roads.
- Avoid work in rainy season.
- The nail should not be lying around, after the completion of layout work.
- Barricade the site to restrict the entry of animal and the unauthorised person on site.
- Ensure that workers are wearing personal protective equipment.
- Safety belt, helmet, rubber hand gloves, goggles, facemask and rubber shoes are of ISI mark.
- Provide ladder to Excavation workers to climb in and out of the pit.
- First-Aid kits should be immediately available on site.
- Provide proper ventilation and electric lights at a time of work.
- Provide barricading to excavated pit. Check the shoring for adequate support.
- Check the material transporting machine for safe operation.
- The movement of the trucks should be away from the pit.
- Machine operator must have recognised agency's license.
- Do the sufficient pit size by providing extra working space on all sides.
- Cleaning of area and housekeeping of material is necessary.

07.

(a). A four-storey reinforced concrete (RC) office building located in seismic zone IV is shown in the figure. The RC frames are infilled with brick masonry. The lumped weights due to dead loads is 15 kN/m^2 on the floors and 12 kN/m^2 on the roof. The floors have to cater to a live load of 4 kN/m^2 on the floors and 1.5 kN/m^2 on the roof. Calculate the design seismic load on the structure at different floors using Linear Static (Equivalent Static) analysis, along x-axis.



Zone factor Z = 0.24

Importance factor I = 1.2

Response Reduction Factor R = 5

Fundamental translational natural period $T_a = \frac{0.09h}{\sqrt{d}}$

$$\frac{S_a}{g} = \begin{cases} 2.5 & 0 < T \le 0.55s \\ \frac{1.36}{T} & 0.55 < T \le 4.0s \\ 0.34 & T > 4.0s \end{cases}$$

(20 M)

Sol: Given,

At floor level:

Dead load = 15 kN/m^2

Live load = 4 KN/m^2

At roof level:

Dead load = 12 KN/m^2

Live load = 15 kN/m^2

Z= 0.4, I = 1.2; R = 5

Step 1: Seismic weight

Floor area = $20 \times 12 = 240 \text{m}^2$

As per IS 1893

As live load >4 kN/m^2 in floors, 50% is considered

At roof, live load need not be considered

$\therefore$ Seismic weight of floors = $240\ (15 + 0.5 \times 4)$

$\Rightarrow W_1 = W_2 = W_3 = 4080 \text{ kN}$

roof = $W_4 = 240(12)$

$= 2880 \text{ kN}$

Total seismic weight of structure

$= 3 \times 4080 + 2880$

$= 15120 \text{ KN}$

Step 2: Fundamental period and base shear

(a) Along x-direction

$$T = \frac{0.09h}{\sqrt{d}}$$

h =3.6+3.3+3.3+3.4 = 13.6 m

d = 20 m

$$\therefore T = \frac{0.09 \times 13.6}{\sqrt{20}}$$

$$= 0.2737 \text{ sec}$$

As T < 0.55 sec, $\frac{S_a}{g} = 2.5$

Horizontal acceleration $A_h = \frac{z}{2}\frac{I}{R}\frac{S_a}{g}$

$$= \frac{0.24}{2} \times \frac{1.2}{5} \times 2.5$$

$$= 0.072$$

Design Base shear = A_h W

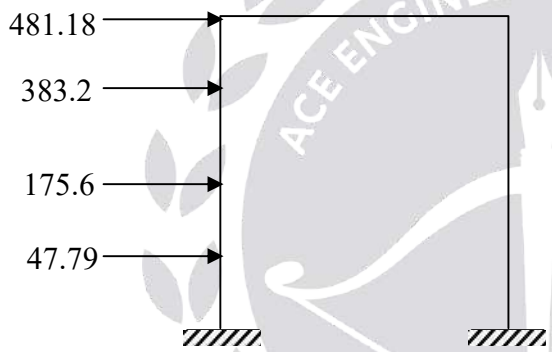
$$= 0.072 \times 15120$$

$$= 1088.64 \text{ kN}$$

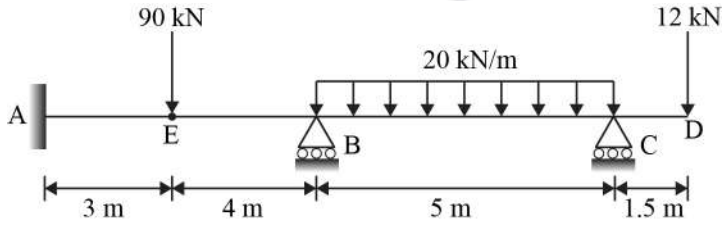
Step 3: Distribution of base shear

Base shear along height $= Q_l = V_B \frac{W_i h_i^2}{\Sigma W_i h_l^2}$

Storage level	h_i	W_i (kN)	$W_i h_i^2$ in $(x10^3)$	$\frac{W_i h_i^2}{\Sigma W_i h_i^2}$	$V_B \times \frac{W_i h_i^2}{\Sigma W_i h_i^2}$ (KN)
4	13.6m	2880	532.68	0.442	481.18
3	10.2m	4080	424.48	0.352	383.2
2	6.9m	4080	194.248	0.1613	175.6
1	3.6m	4080	52.876	0.0439	47.79
$\Sigma W_i h_i^2 = 1204.284 \times 10^3$					



(b). Analyse a continuous beam shown in the figure. During loading, the support B sinks by 12 mm. E = 210 GPa. 1 = 5131.6 × 10^4 mm^4. EI is constant. Draw BMD and Elastic Curve.



(20 M)

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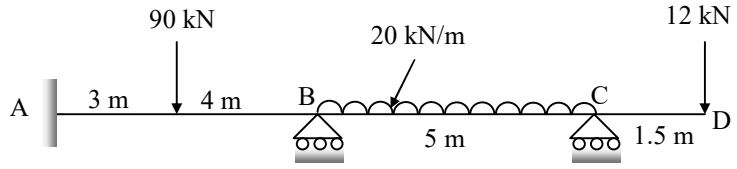
Sol:



$\delta_B = 12\ mm = 12 \times 10^{-3}\ m$

$E = 210\ GPa = 210 \times 10^9\ N/m^2 = 210 \times 10^6\ kN/m^2$

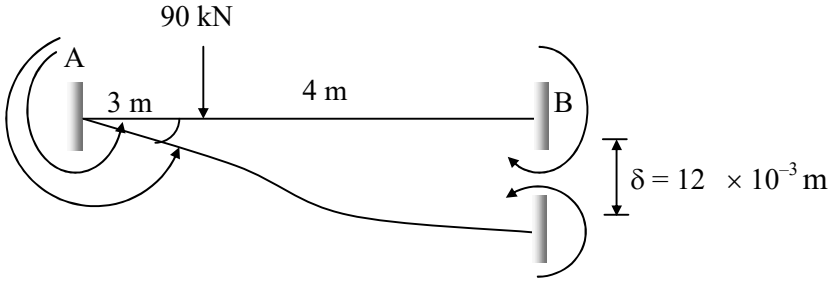
$I = 5131.6 \times 10^4\ mm^4 = 5.1316 \times 10^{-5}\ m^4$

Distribution factor $(DF) = \dfrac{K}{\Sigma K}$

Joint	Member	Relative Stiffness	Total stiffness (ΣK)	DF
B	BA	I/7	$\frac{I}{7}+\frac{3I}{20}=\frac{41I}{140}$	0.49
	BC	$\frac{3}{4}\times I/5$		0.51
C	CB	I/5	I/5	1
	CD	0		0

Fixed End moments (FEM):

Assume all supports are fixed/locked & find the fixed end moments for each span

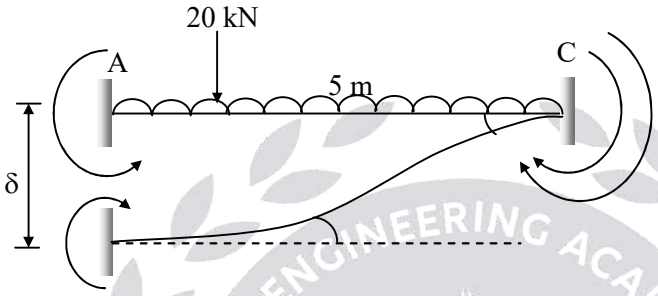


$$M_{FAB} = -\frac{Wab^2}{L^2} - \frac{6EI\delta}{L^2} = -\frac{90\times3\times4^2}{7^2} - \frac{6\times210\times10^6\times5.1316\times10^{-5}\times12\times10^{-3}}{7^2}$$

$$= -88.16 - 15.83 = -103.99 \text{ kN-m}$$

$$M_{FAB} = +\frac{wa^2b}{L^2} - \frac{6EI\delta}{L^2} = +\frac{90\times3^2\times4}{7^2} - \frac{6\times210\times10^6\times5.1316\times10^{-5}\times12\times10^{-3}}{7^2}$$

$$= 66.12 - 15.83 = 50.29 \text{ kN-m}$$

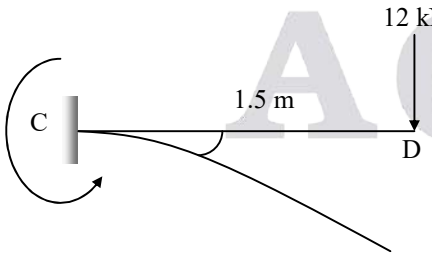


$$M_{FBC} = -\frac{wL^2}{12} + \frac{6EI\delta}{L^2}$$

$$= -\frac{20\times5^2}{12} + \frac{6\times210\times10^6\times5.1316\times10^{-5}\times12\times10^{-3}}{5^2}$$

$$= -41.67 + 31.03 = -10.64 \text{ kN-m}$$

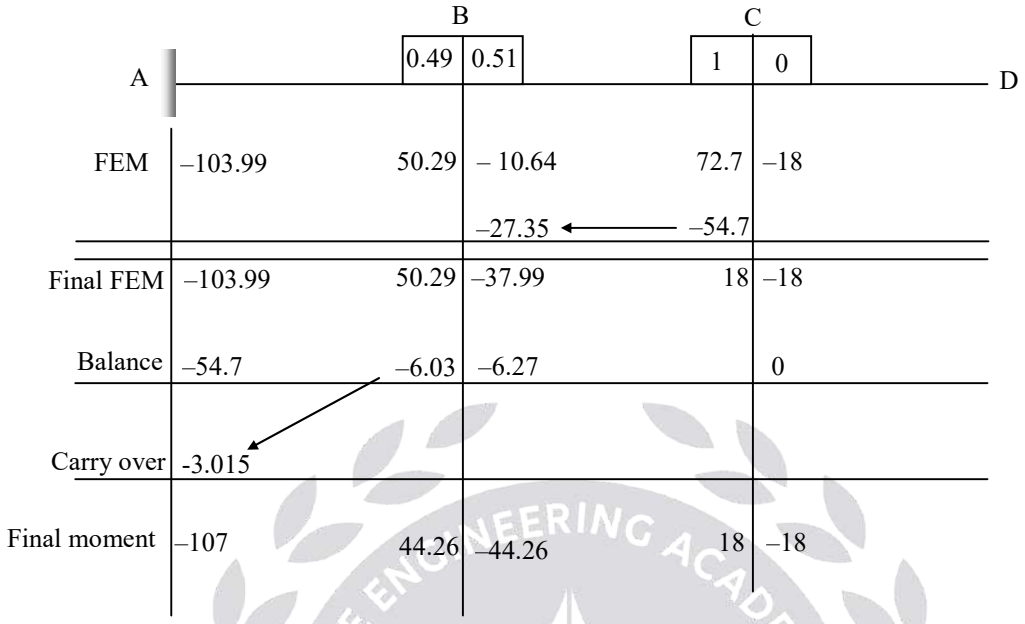
$$M_{FCB} = +\frac{wL^2}{12} + \frac{6EI\delta}{L^2} = +41.67 + 31.03 = 72.7 \text{ kN} - \text{m}$$

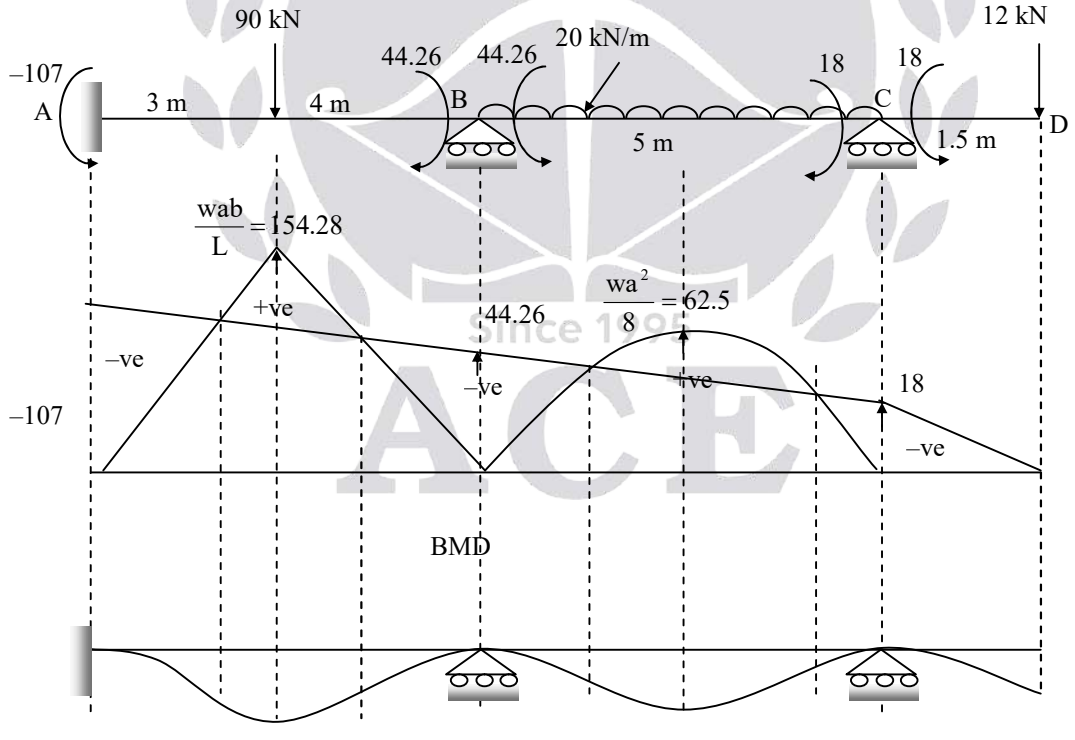


$$M_{FCD} = -12 \times 1.5$$

$$= -18 \text{ kN-m}$$

Moment Distribution





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(c). Differentiate between optimistic time estimate and pessimistic time estimate in a PERT network.

A construction company has an opportunity to submit a bid for the construction of a residential building and a commercial building. The 3 time estimates (in months) for completion of each building are as follows:

	Optimistic Time (in months)	Most likely time (in months)	Pessimistic Time (in months)
Residential Building	3	4	6
Commercial Building	4	6	8

Determine the expected time for completion of each building. Also analyse which building has more reliable time estimate.

(20 M)

Sol:

	Optimistic time estimate		Pessimistic time Estimate
1.	The estimate of minimum possible time in which an activity can complete under ideal condition is called optimistic time estimate	1.	The maximum time taken by an activity with delay is called pessimistic time estimate.
2.	This is the shortest possible time	2.	It is the longest time taken.
3.	It is a lower bound for the activity	3.	It is an upper bound for activity duration
4.	Probability of getting optimistic time is less	4.	Probability of getting pessimistic time is less
5.	It does not delay at any stage.	5.	If there is any delay at every stage is included.
6.	If the working conditions are extremely good then optimistic time is possible	6.	If the working conditions are extremely bad, then the activity will take pessimistic time.
7.	It is generally denoted by 't_o'	7.	It is denoted by 't_p'

Residential building

Given

$t_o = 3$ months $t_m = 4$ months, $t_p = 6$ months

$$\text{Expected time } (t_e) = \frac{t_o + 4t_m + t_p}{6}$$

$$= \frac{3+4(4)+6}{6}$$

$$= 4.167$$

$\therefore$ Expected time of completion of residential building $\cong$ 4.2 months.

$$\text{Variance } (V_t) \text{ for Residential building} = \left(\frac{t_p - t_o}{6}\right)^2 = \left(\frac{6-3}{6}\right)^2 = \left(\frac{3}{6}\right)^2 = (0.5)^2 = 0.25$$

Commercial building

Given

$t_o = 4$ months $t_m = 6$ months $t_p = 8$ months

$$\text{Expected time } (t_e) = \frac{4+4(6)+8}{6}$$

$$= 6 \text{ months}$$

$$\text{Variance } (V_t) \text{ for commercial building} = \left(\frac{t_p - t_o}{6}\right)^2 = \left(\frac{8-4}{6}\right) = \left(\frac{4}{6}\right)^2 = 0.44$$

V_t for residential building $< V_t$ for commercial building

$\therefore$ Residential building has more reliable time estimate.

08.

(a). Design an open cylindrical water tank of 350 m^3 capacity. This tank will rest on ground and have a free-flexible joint at base. Overall height of tank is 4.0 m, including the free board of 200 mm. Design the vertical cylindrical wall of tank and sketch the details.

Consider only maximum hoop tension for entire height.

The following parameters may be used for design, if required.

1. Permissible direct tensile stress of concrete $\sigma_{ct} = 1.2$ MPa

2. Permissible tensile stress of steel σ_{st} = 150 MPa up to 225 mm from water fall = 190 MPa beyond 225 mm from water fall.
3. Molecular ratio M = 13
4. Minimum thickness required = 180 mm
5. Minimum % of reinforcement upto 100 mm thickness 0.24 % linearly reduces to 0.16% for more than 450 mm thickness.
6. Use 16 mm diameter, high strength deformed bar for hoop reinforcement.
7. Use 8 mm diameter, high strength deformed bar for vertical distribution reinforcement.
8. Minimum diameter of tank required = 11.0 m
9. Unit weight of water = 10 kN/m^3.
10. Tensile stress in concrete $= \dfrac{T}{A_c - (m-1)A_s}$

(20 M)

Sol: Capacity of tank = 350 m^3

Height of tank = 4m

Height of water = 4 – 0.2 (Free board = 200mm)

= 3.8 m

Diameter of the tank

volume of tank = capacity of tank

$$\frac{\pi}{4}D^2H = 350$$

$$\frac{\pi}{4}D^2 \times 3.8 = 350$$

$$D = 10.8\ m \simeq 11m$$

$$\text{Hoop tension } T = \gamma H\frac{D}{2} = 10 \times 3.8 \times \frac{11}{2} = 209\,kN$$

Horizontal reinforcement to resist hoop tension

$$A_s = \frac{T}{\sigma_{st}} = \frac{2.09 \times 10^3}{150} = 1393\,mm^2$$

Using 16 mmϕ bars, spacing required

$$S = \frac{1000 \times \frac{\pi}{4} \times 16^2}{1393} = 144mm \simeq 140\ mm$$

Actual area, $A_S = \frac{\frac{\pi}{4} \times 16^2 \times 1000}{140} = 1436mm^2$

Thickness of wall required

$$\sigma_{ct} = \frac{T}{1000 + (m-1)As}$$

$$1.2 = \frac{209 \times 10^3}{1000t + (13-1)1436}$$

t = 157 mm

Provide min thickness = 180 mm

Vertical reinforcement

t = 100 mm → p = 0.24%

t = 450 mm → p = 0.16%

for t = 180 mm → $p = 0.24 - \frac{0.24 - 0.16}{450 - 100} \times (180 - 100) = 0.22$

$$A_S = \frac{0.22}{100} \times t \times 1000$$

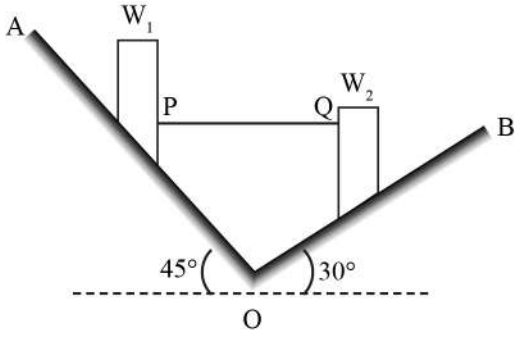
$$= \frac{0.22}{100} \times 180 \times 1000 = 396mm^2$$

Use 8 mmϕ, spacing required

$$S = 1000\frac{ast}{As} = 1000 \times \frac{\frac{\pi}{4}\times 8^2}{396} = 127\text{mm}$$

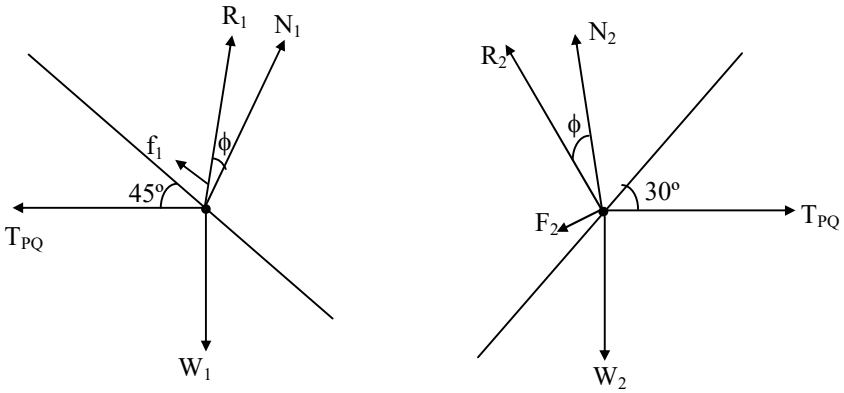
Provide 8 mmϕ @ 120 mm c/c

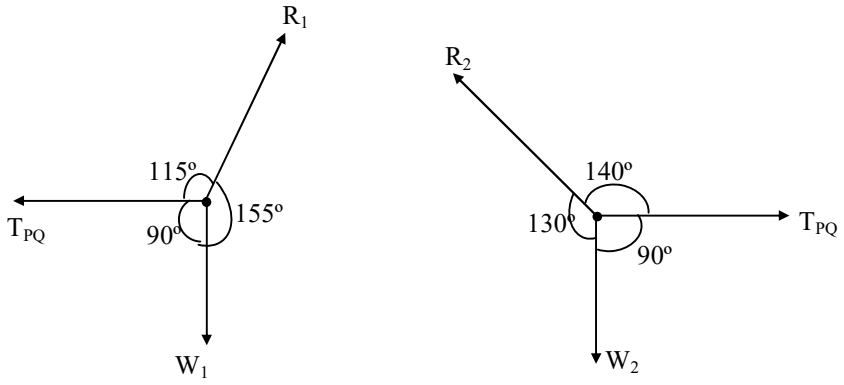
(b). Two loads W_1 (equal to 1.5 kN) and W_2, resting on two inclined rough planes OA and OB are connected by a horizontal link PQ as shown in the figure. Find the maximum and minimum values of W_2 for which the equilibrium can exist. Take angle of friction for both the planes as 20^o.



(20 M)

Sol: At minimum value of W_2, the block W_2 impends to move up & the block W1 impends to move down





$$\frac{W_1}{\sin 115^\circ} = \frac{T_{PQ}}{\sin 155^\circ}$$

$$\Rightarrow T_{PQ} = \frac{W_1 . \sin 155^\circ}{\sin 115^\circ}$$

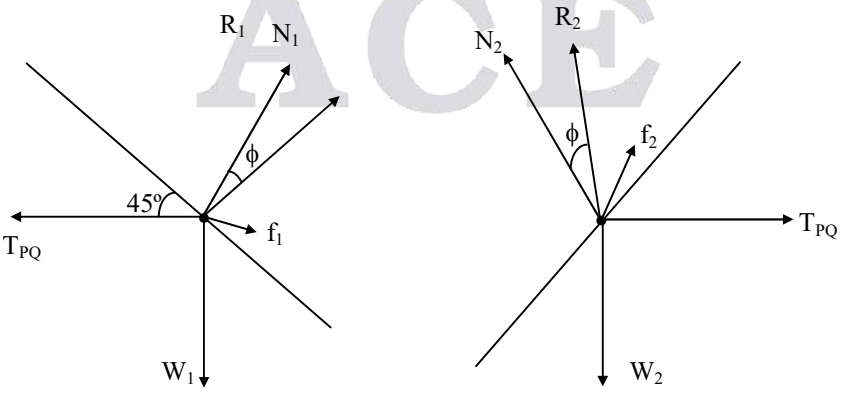
$$= 0.69946 \text{ kN}$$

$$\simeq 0.7 \text{ kN}$$

$$T_{PQ} = \frac{W_2}{\sin 140^\circ} \Rightarrow W_2 = \frac{T_{PQ} . \sin 140^\circ}{\sin 130^\circ}$$

$$(W_2)_{min} = 0.5869 \text{ kN}$$

At maximum value of W_2, the block W_2 impends to move down and the block W_1 impends to move up.



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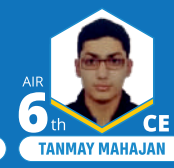
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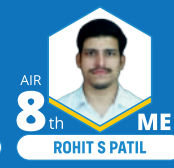
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D KUMAR PATIL

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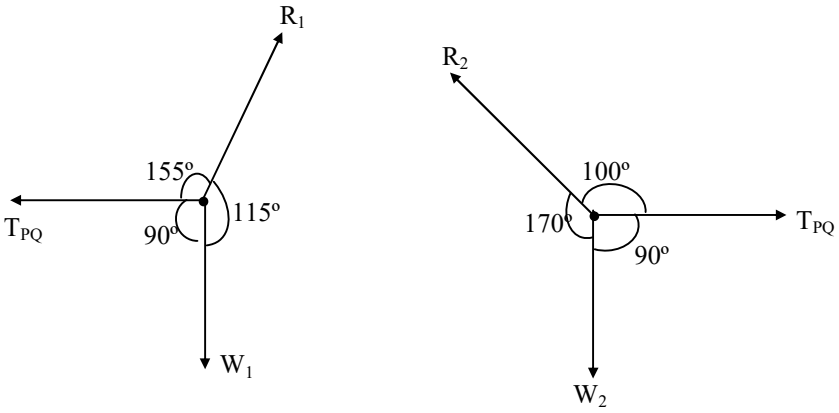
R SHIVAJI NALE

and many more...

Total 57 Ranks in Top 10

ME : 10	PI : 09	CE : 08	EE : 4	EC : 09	CS : 03	IN : 08	XE : 06



$$\frac{W_1}{\sin 155^\circ} = \frac{T_{PQ}}{\sin 115^\circ}$$

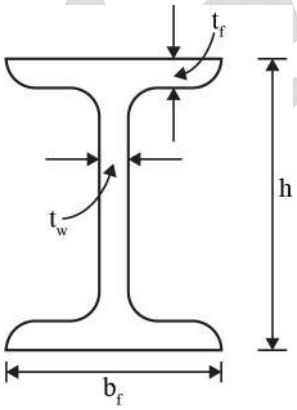
$$\Rightarrow T_{PQ} = \frac{W_1 . \sin .11^\circ}{\sin 115^\circ}$$

$$= 3.2167 \text{ kN}$$

$$\frac{T_{PQ}}{\sin 170^\circ} = \frac{W_2}{\sin 100^\circ} \Rightarrow W_2 = \frac{T_{PQ} . \sin 100^\circ}{\sin 170^\circ}$$

$$(W_2)_{max} = 18.243 \text{ kN}$$

(c). A rolled steel joist ISMB 450 is used as beam for the roof of a hall 7.5 m × 12 m. Thickness of RC slab is 125 mm. The rolled steel joists are spaced at 3 m centre to centre. The floor finishing load is 1.5 kN/m^2 and the roof slab has to support a live load of 4 kN/m^2. Assume the self-weight of the beam as 1 kN/m. Take the width of bearing for the beam as 300 mm. The limiting deflection of the beam is span/240. γ_{mo} = 1.1, f_y = 250 MPa. Check the adequacy of the section against any two modes of failure.



Properties of ISMB 450 :

Elastic Section Modulus = 30390.8×10^3 mm^3

Plastic Section Modulus $Z_P = 1533.36 \times 10^3$ mm^3

Depth of section h = 450 mm

Width of flange b_f = 150 mm

Thickness of flange t_f = 17.4 mm

Thickness of web t_w = 9.4 mm

Radium at root = 15 mm

Shear capacity $V_d = \dfrac{A_v.f_y}{\sqrt{3}\,\gamma_{mo}}$

Design bending strength $M_d = \dfrac{\beta_b.Z_p.f_y}{\gamma_{mo}}$

Slenderness ratio	Design compression Stress (f_{cb})
100	107
110	94.6
120	83.7

Buckling strength $F_b = (b_1 + n_1)\, t_w .f_{cd}$

Capacity of web at its connection to the flange $F_w = \dfrac{(b_1 + n_2)t_w.f_{yw}}{\gamma_{mo}}$

The section ISMB 450 may be assumed as plastic.

(20 M)

Sol: Given, f_y = 250 MPa

Dead load (due to slab) = w_c × thickness slab × c/c spacing

= 25 × 0.125 × 3 = 9.375 kN/m

Dead load (beam) = 1 kN/m

Live load = 4 kN/m^2 × c/c spacing

= 4 × 3 = 12 kN/m

Total load = (9.375 + 1 + 12) kN/m

= 22.375 kN/m

Factored load = 1.5 × 22.375 kN/m

= 33.5625 kN/m

Assuming provision of along 7.5 m clear span

Effective span = l = clear span + 2 × $\frac{\text{width of end bearing}}{2}$

$$= 7.5\text{m} + 2 \times \frac{0.3}{2} = 7.8 \text{ m}$$

Maximum bending moment = $= \frac{w\ell^2}{8} = \frac{33.5625 \times 7.8^2}{8} = 255.242$ kN-m

Maximum shear force, $V = \frac{w\ell}{2} = \frac{33.5625 \times 7.8}{2}$

= 130.89 kN

Now,

The section provided is ISMB 450, and is assumed as plastic

Check for Shear Capacity:

Design shear force = 130.89 kN

Design shear capacity = $V_d = \frac{A_v f_y}{\sqrt{3}\gamma_{mo}}$

$$= \frac{(ht_w)f_y}{\sqrt{3}\gamma_{mo}}$$

$$= \frac{(450 \times 9.4)250}{\sqrt{3} \times 1.1}$$

= 555.043 kN > 130.89 kN

(which is OK)

Check for High or Low Shear

$0.6\ V_d = 0.6 \times 555.043 = 333.026\ kN > 130.89\ kN$

Hence it is low shear case.

Check for Design Bending Strength

$$M_d = \frac{\beta_b z_p f_y}{\gamma_{mo}} \qquad \text{(for plastic } \beta_b = 1.0\text{)}$$

$$= \frac{1 \times 1533.36 \times 10^3 \times 250}{1.1}$$

$= 348.49\ kN\text{-}m > 255.242\ kN$

M_d should also be $\le 1.2\ \frac{z_e f_y}{\gamma_{mo}}$

So

$$\frac{1.2ze}{\gamma_{nw}} = \frac{1.2 \times 30390.8 \times 10^3 \times 250}{1.1}$$

$= 8288.4\ kN\text{-}m > 348.49\ kNm$ (OK)

Check for Deflection:

Service load = 22.375 kN/m

$$\text{Permissible deflection} = \frac{\text{span}}{240}$$

$$= \frac{7500}{240} = 31.25\ mm$$

$$\text{Maximum deflection} = \frac{5}{384} \times \frac{22.375 \times (7500)^4}{2 \times 10^5 \times 303908 \times 10^3} = 151.66 > 31.2 \quad \text{(Not OK)}$$

$$\text{Check for Bearing Strength} = \frac{(b_1 + h_1) t_w f_{y_w}}{\gamma_{mo}}$$

$h_1 = 2.5\ (t_y + R_1) = 2.5 \times (17.4 + 15) = 81\ mm$

$$f_w = \frac{(300 + 81) \times 9.4 \times 250}{1.1}$$

$= 813.954\ kN > 130.89\ kN$ (Hence OK)

Hearty Congratulations to our

ESE - 2020 TOP RANKERS

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AIR 10  HEMABH TRIVEDI ME

AIR 10  RAJAT DIXIT EE

AIR 10  L KUMARI JAISWAL E&T

AIR 10  AMIT SHARMA CE

and many more...

TOTAL 36 RANKS IN TOP 10

ME	EE	E&T	CE
10	09	10	07