ACE
Engineering Academy
Leading Institute for ESE/GATE/PSUs

HYDERABAD | DELHI | PUNE | BANGALORE | LUCKNOW | CHENNAI | VISAKHAPATNAM | VIJAYAWADA | TIRUPATHI | KOLKATA | AHMEDABAD

ESE-2020

(MAINS)

QUESTIONS WITH DETAILED SOLUTIONS

CIVIL ENGINEERING

PAPER-I

ACE Engineering Academy has taken utmost care in preparing the ESE-2020 MAINS Examination solutions. Discrepancies, if any, may please be brought to our notice. ACE Engineering Academy do not owe any responsibility for any damage or loss to any person on account of error or omission in these solutions. ACE Engineering Academy is always in the fore front of serving the students, irrespective of the examination type (GATE/ESE/PSUs/PSC/GENCO/TRANSCO etc.,).

All Queries related to ESE - 2020 MAINS Solutions are to be sent to the following email address hyderabad@aceenggacademy.com
Contact Us : 040-23234418,19,20

CIVIL ENGINEERING

ESE _MAINS_2020_PAPER – I

Questions with Detailed Solutions

SUBJECT WISE WEIGHTAGE

S.No.	NAME OF THE SUBJECT	Marks
1	Strength of Materials	84
2	Structural Analysis	104
3	Building Materials	52
4	Design of Concrete and Masonry Structures	104
5	Steel Structures	84
6	Construction Management & Equipment	52

01. (a)

(i) How are aggregates classified based on particle size? What is bulking in sand? (6)

Sol: According to size, aggregates can be classified as coarse aggregates and fine aggregates.

(i) Coarse Aggregates:

The aggregates which pass through the 80 mm sieve and are retained on the 4.75 mm sieve are called as coarse aggregates.

(ii) Fine Aggregates:

All the aggregates which pass through the 4.75 mm sieve are called as fine aggregates. On the basis of particle size distribution, the fine aggregates are classified into four zones (i.e.) Zone I to Zone IV. The grading zones are progressively finer from grading Zone I to grading Zone IV.

Bulking of Sand:

The increase in volume of sand due to the presence of free moisture in it is called as bulking of sand. This free moisture forms a thin film over the sand particles which keeps the neighboring particles away due to the effects of surface tension. Thus, resulting in bulking of the volume of the sand. The significance of surface tension forces and consequently how far the sand particles are pushed away will depend on the percentage of moisture present and also the size of the fine aggregate particles. Generally, this phenomenon occurs in all sizes of aggregates, but it is significant mostly in fine sands and coarse silts. It is also to be noted that the effects of bulking increases with increase in moisture up to a certain limit and further increase in moisture content leads to decrease in the bulking effects. No bulking can be observed when the sand is completely saturated.

(ii) How is workability of concrete defined as per Indian Standard Specification IS 1199-1959? Briefly explain the method of measurement of workability through compaction factor test. (6)

Sol: As per Indian Standard Specification IS 1199-1959, workability is defined as the property of concrete which determines the amount of useful internal work necessary to produce complete compaction.

Compaction Factor Test:

This test is more suitable for concrete mixes of medium and low workability.

This test is primarily designed for laboratory, but can also be used at site of work.

The sample of concrete is placed in the top hopper and levelled.

The Trap-door of the top hopper is opened to allow the concrete to fall into the lower hopper.

Now the Trap-door of the lower hopper is opened to allow the concrete to fall into the cylinder.

The concrete over the top of the cylinder is removed.

The weight of the concrete in the cylinder is measured and is known as the weight of the partially compacted concrete.

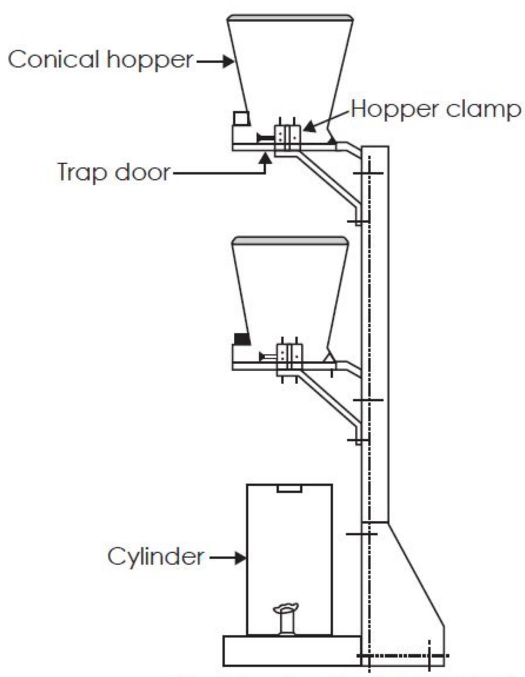
The cylinder is refilled with concrete from the same sample in layers of 50 mm.

Each layer is rammed heavily or preferably vibrated to get 100% compaction.

The top surface of the fully compacted concrete is carefully struck off level with the top of the cylinder.

The mass of the concrete in this cylinder is measured and is called as the mass of the fully compacted concrete.

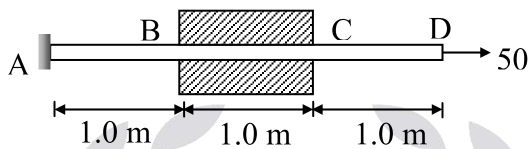
$$\text{Compaction Factor (CF)} = \frac{\text{Weight of practially compacted concrete}}{\text{Weight of fully compacted concrete}}$$



Compacting Factor Apparatus

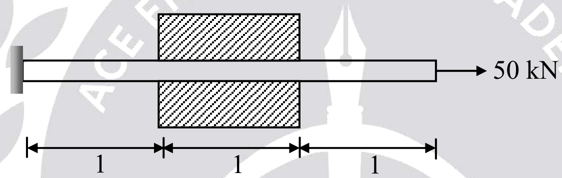
(b) A semi-composite steel bar as shown in figure is loaded at free end with an axial load of 50 kN. Determine the axial stiffness of the system and extension of the free end. Diameter of steel is 40 mm, outer diameter of the concrete portion is 200 mm. Modulus of elasticity of steel = 200 GPa, Modulus of elasticity of concrete = 20 GPa. Central portion of the bar is embedded with concrete.

(12)

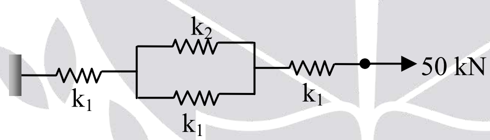


Sol:

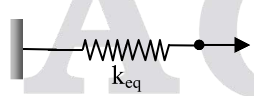
Given $E_S = 200$ GPa, $d_S = 40$ mm, $E_C = 20$ GPa, $d_{iC} = 40$ mm, $d_{OC} = 200$ mm



Step – I: Actual stiffness diagram



Step – II: Equivalent stiff diagram is



Find $k_{eq} = ?$ $\quad \delta_{end} = \dfrac{50\,\text{kN}}{k_{eq}} = ?$

Step -III: As bars are in series we can write

$$\frac{1}{k_{eq}} = \frac{1}{k_1} + \frac{1}{k_1 + k_2} + \frac{1}{k_1} \quad \text{———— (1)}$$

$$k_1 = \frac{A_1E_1}{L_1} \qquad k_2 = \frac{A_2E_2}{L_2}$$

$$k_1 = \frac{\frac{\pi}{4}\times 40^2 \times 200\times 10^3}{1000\,\text{mm}} = 251.327\,\frac{\text{kN}}{\text{mm}}$$

$$k_2 = \frac{\frac{\pi}{4}\left[200^2 - 40^2\right]\times 20\times 10^3}{1000} = 603.185\,\frac{\text{kN}}{\text{mm}}$$

Sub k_1, k_2 in equation (1)

$$\frac{1}{k_{eq}} = \frac{1}{251.327} + \frac{1}{251.327+603.185} + \frac{1}{251.327}$$

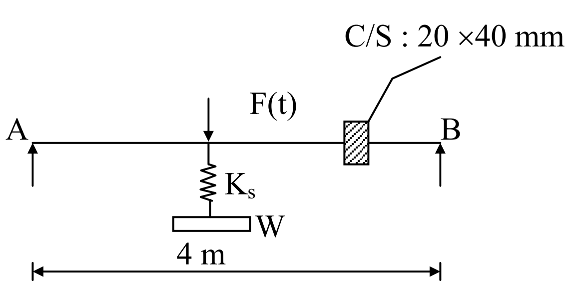
$$k_{eq} = 109.552\,\text{kN/mm} \approx 110\,\text{kN/mm}$$

$\rightarrow$ extension of freed end is

$$\delta_{\text{force end}} = \frac{P}{k_{eq}} = \frac{50\,\text{kN}}{110\,\text{kN/mm}} = 0.45\,\text{mm}$$

(c) A machine is mounted at the centre of a simple supported beam that can exert a harmonic load F(t) = 20 sin (0.12 t) kN in vertical direction. The length of beam is 4 m and its cross-section is uniform throughout. Cross-section of beam: width 20 mm and depth 40 mm. A weight W = 200 N is suspended from the centre of the beam by a spring of spring constant K_s = 40 N/m. Determine the natural frequency of the weight W. Neglect mass of the beam and weight of machine. E = 2 × 10^5 MPa. (12)

Sol: From the given data, schematic diagram of the system given can be shown as.



W=200N

Spring constant K_s=40 N/m

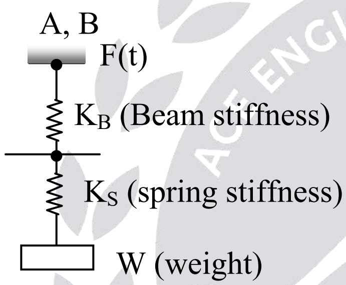
F(t) = Applied load = 20 Sin(0.12t)kN

Beam's E = 2×10^5MPa

Find Natural frequency of the bean.

1. Since Mass of Bean is neglected, Simply supported bean with central load can be treated as a linear spring with stiffness K_b = Bean Stiffness = $\dfrac{45EI}{L^3}$

2. Given Bean model B now.



$$\text{Now } K_B = \frac{48EI}{L^3} = \frac{48\times 2\times 10^5 \times 10^6 \times I}{(4)^3}$$

$$I = bd^3 = (0.02)(0.04)^3 = 1.0667\times 10^{-7}$$

K_B = 1.6×10 N/m

K_S = 40 N/m

From the model, both springs K_B and K_S are in series. (Subjected to same load but deflection is sum of both springs)

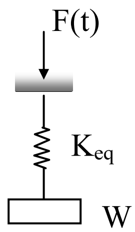
$\therefore$ K_{eq} = Equivalent Stiffness

$$= \frac{K_B K_S}{KB+KS}\text{(For Springs in Ser)}$$

$$= \frac{1.6\times10^4\times 40}{1.6\times10^4+40} \approx 39.900\,N/m$$

$$\approx 40\text{ N/m}$$

As per new reduced model



$\therefore$ Natural frequency in rad/s $W_n \sqrt{\dfrac{K_{eq}}{m}} = \sqrt{\dfrac{K_{eq}}{W/g}}$

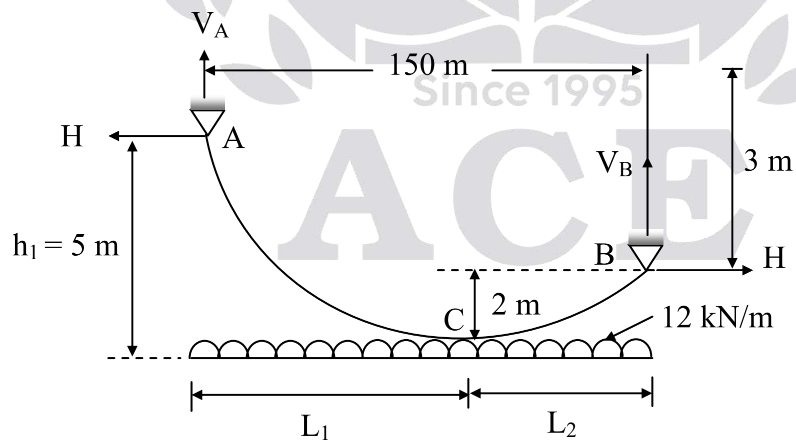
$\therefore W_n = \sqrt{\dfrac{40 \times 9.81}{200}}$ rad/s = 1.399 rad/s

$\therefore W_n = 1.399$ rad/s

Note: For natural frequency applied load has no influence.

(d) A cable of uniform cross-section hangs between two points A and B, which are 150 m apart. The end 'A' of the cable is 3 m above the other end of the cable. The sag of the cable measured from 'B' is 2 m. If the cable carries a UDL of 12 kN/m, determine the maximum tension in the cable. Also find the horizontal pull. (12)

Sol:



$L_1 + L_2 = 150$

Since ACB is parabolic we have $\dfrac{x^2}{y} = \text{constant}$ with $\hat{c}$ as origin

$$\frac{L_1^2}{h_1} = \frac{L_2^2}{h_2}$$

$$\frac{L_1}{\sqrt{h_1}} = \frac{L_2}{\sqrt{h_2}} = \frac{L_1 + L_2}{\sqrt{h_1} + \sqrt{h_2}} = \frac{L}{\sqrt{h_1 + h_2}}$$

$$L_1 = \frac{L\sqrt{h_1}}{\sqrt{h_1} + \sqrt{h_2}} \quad \text{and} \quad L_2 = \frac{L\sqrt{h_2}}{\sqrt{h_1} + \sqrt{h_2}}$$

$$L_1 = \frac{150\sqrt{5}}{\sqrt{5} + \sqrt{2}} \qquad L_2 = \frac{150\sqrt{2}}{\sqrt{5} + \sqrt{2}}$$

$L_1 = 91.88$ m $\qquad L_2 = 58.11$ m

$\Sigma V = 0 \; \overset{\downarrow}{\underset{\uparrow}{=}}$

$V_A + V_B = 12 \times 150$ ------ (1)

$BM_c = 0$ ↻ -ve ↺ +ve [left part]

$$+V_A \times 91.88 - H \times 5 - 12 \times 91.88 \times \frac{91.88}{2} = 0$$

$$V_A = \frac{5H + 50651.60}{91.88} \text{ ------ (2)}$$

$BM_c = 0$ ↻ -ve ↺ +ve [Right part]

$$-V_B \times 58.11 + H \times 2 + 12 \times 58.11 \times \frac{58.11}{2} = 0$$

$$V_B = \frac{2H + 20260.63}{58.11} \text{ ------- (3)}$$

$$\left[\frac{5H + 50651.60}{91.88}\right] + \left[\frac{2H + 20260.63}{58.11}\right] = 12 \times 150$$

$0.089\ H \pm 900$

$H = 10112.35$ kN

$$V_A = \frac{5 \times 10112.35 + 50651.60}{91.88}$$

$V_A = 1101.58$ kN

Maximum tension in the cable

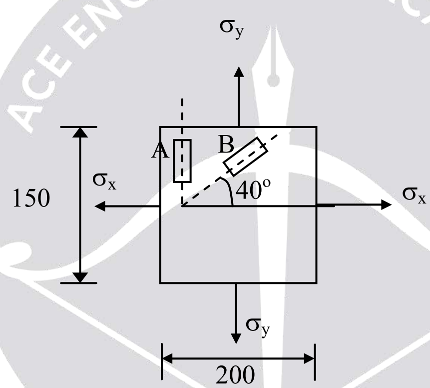
$$T_{max} = \sqrt{V_A^2 + H^2}$$

$$= \sqrt{1101.58^2 + 10112.35^2}$$

$T_{max} = 10172.172$ kN

$H = 10112.35$ kN

(e) A 200 mm × 150 mm × 10 mm aluminium plate is subjected to uniform bi-axial stresses σ_x and σ_y. Two strain gauges A and B are attached to the surface of the plate as shown in the figure. If readings in strain gauges are $\varepsilon_A = 200 \times 10^{-6}$ and $\varepsilon_B = 285 \times 10^{-6}$, what are the values of σ_x and σ_y ? What is the reduction in thickness of the plate as a result of stresses? Take Young's modulus E = 75 GPa and Poisson's ratio $\nu = 0.33$. (12)

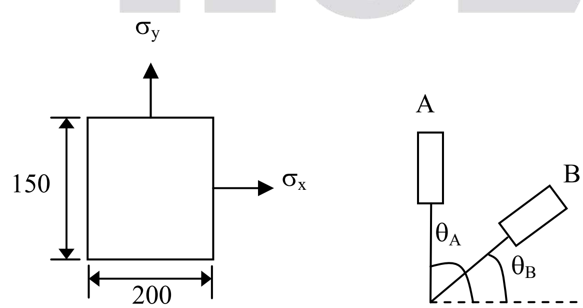


Sol: Given $\in_A = 200 \times 10^{-6}$

$\in_B = 285 \times 10^{-6}$, $L \times B \times t = 200 \times 150 \times 10$

Find $\sigma_x, \sigma_y = ?$

$\delta t = ?$ (Change in thickness = ?)



Strain transformation equations are

$$\in_a = \frac{\in_x + \in_y}{2} + \frac{\in_x - \in_y}{2}\cos 2\theta_A$$

$$\in_b = \frac{\in_x + \in_y}{2} + \frac{\in_x - \in_y}{2}\cos 2\theta_B$$

Given $\theta_A = 90^o, \theta_B = 40^o$

$$\in_a = \in_y = 200 \times 10^{-6}$$

$$\in_b = 285 \times 10^{-6} = \frac{\in_x + 200\mu}{2} + \left[\frac{\in_x - 200\,\mu}{2}\right]\cos 80^0$$

$$\in_x = 344.84\,\mu$$

Now

$$\left.\begin{aligned}\in_x &= \frac{1}{E}\left[\sigma_x - \nu\sigma_y\right]\\ \in_y &= \frac{1}{E}\left[\sigma_y - \nu\sigma_x\right]\end{aligned}\right\} \text{———— Equation (1)}$$

$$\in_z = \frac{1}{E}\left[-\nu\left[\sigma_x + \sigma_y\right]\right]$$

Solve Equation (1)

$$345\,\mu = \frac{1}{75\times 10^3}\left[\sigma_x - 0.33\times\sigma_y\right]$$

$$200\mu = \frac{1}{75\times 10^3}\left[\sigma_y - 0.33\times\sigma_x\right]$$

(OR)

$$\sigma_x = \frac{\in}{1-\nu^2}\left[\in_x + \nu \in_y\right] = 34.59 Mpa$$

$$\sigma_y = \frac{\in}{1-\nu^2}\left(\in_y + \nu \in_x\right) = 26.4\,MPa$$

$$\in_z = \frac{\delta t}{t} = \frac{1}{E}\left[0.33\left(\sigma_x + \sigma_y\right)\right]$$

$$\delta t = 2.687\times 10^3 \text{ mm}$$

$$\delta t = 2.68\,\mu$$

EXCLUSIVE

ONLINE LIVE CLASSES

ENGLISH

ESE | GATE | PSUs – 2022

COURSE DETAILS

- **For ESE+GATE+PSUs Students**
  1. Online Live Classes – Technical Subjects Only.
  2. Recorded Classes - General Studies Subjects (on ACE Deep Learn Platform)
- Recorded version of the online live class will be made available through out the course (with 3 times view).
- Doubt clearing sessions and tests to be conducted regularly.
- 3 to 4 hours of live lectures per day in week days (Timing 5 pm to 9 pm) On Sundays 5-6 Hours Live Online Lectures (6 days a week).
- Access the lectures from any where.

BATCH DATE

7th NOVEMBER 2020

DISCOUNTS

- Rs. 5,000 OFF for ACE Old Students
- Pay Full fee & Get 5% Additional Discount.
- 20% off for IIT / NIT, Students.
- 15% off for IIIT / Govt. College students.

FEE

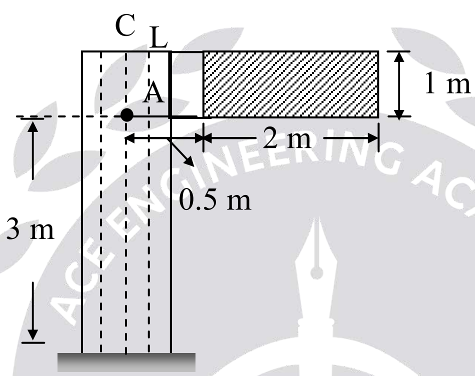
ESE + GATE + PSUs : Rs. 70,000/-

GATE + PSUs : Rs. 55,000/-

(Fee can be paid in two installments)

02.

(a) **A steel tube is to be used as a post for a road sign board as shown in figure. The maximum wind pressure on the sign board is 1960 N/m^2. The angle of rotation of the tube at the bottom of the sign board marked as A must not exceed 4^o and the maximum shear stress (due to torsion only) must not be greater than 38 MPa. Determine the mean diameter of the tube if the wall thickness is 4.2 mm. Take G = 70 GPa. Assume wind is transmitting only over the sign board portion.** **(20)**

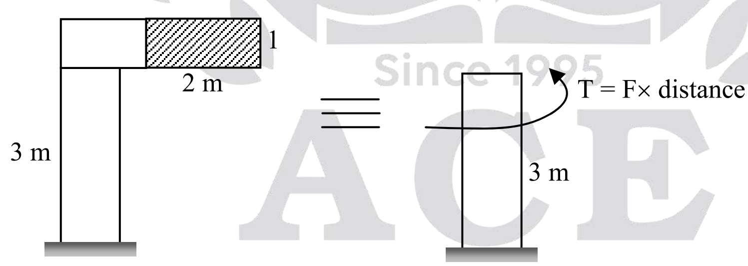


Sol:

Given P = 1960 N/m^2, t = 4.2 mm

$\theta_{max} = 4^o$, $\tau_{max} = 38$ MPa

Find d_{mean} = ?



(only Torsion Considered)

Force = P × A = $1960 \times (2\times1) = 3920\,N$

Torque acting on Tube = $T \times$ Perpendicular Distance

$= 3920 \times (1 + 0.5) = 5880$ N-m

Case (1): Strength based design:

$$\tau_{max} = 38\,MPa = \frac{T}{2\pi r^2 t} \quad \text{(Shear stress in tube)}$$

$$38 = \frac{5880\times10^3}{2\pi r^2 \times 4.2}$$

$$r = 76.5\,\text{mm}$$

d_{mean} = 153.14

Case (2): stiffness based design:

$$\theta = 4^o \times \frac{\pi}{180} = \frac{TL}{GJ} = \frac{5880\times3\times10^6}{70\times10^3\times2\pi r^3\times t}$$

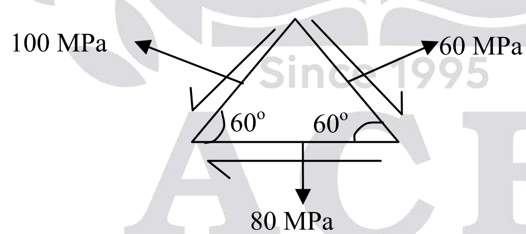
r = 51.5 mm

d_{mean} = 103 mm

Safest mean diameter = max (153, 103)

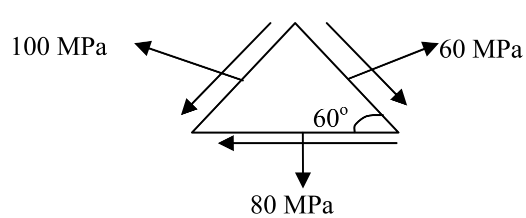
d_{safe} = 153 mm

(b) In a strained body the normal stresses on three planes inclined as shown in figure are 60 MPa (Tensile) and 80 MPa (Tensile) and 100 MPa. Determine the shear stresses acting on these planes. Also find the principal stresses. (20)

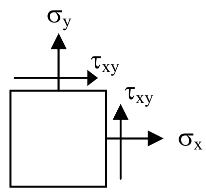


Sol:

Given elements

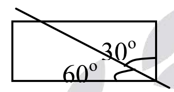


General “2D” element representation



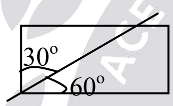
So $\sigma_x = ?$, $\tau_{xy} = ?$, $\sigma_y = 80\,\text{MPa}$

and given



$\sigma_{30^\circ} =$ 60 MPa

$\tau_{30^\circ} = ?$



$\sigma_{-30^\circ} =$ 100 MPa

$\tau_{-30^\circ} = ?$

Also find $\sigma_1, \sigma_2 = ?$

Now

Stress transformation equations are

$$\sigma = \left[\frac{\sigma_x + \sigma_y}{2}\right] + \frac{\sigma_x - \sigma_y}{2}\cos 2\theta + \tau_{xy}\sin 2\theta \qquad \text{——— (1)}$$

$$T_\theta = -\left[\frac{\sigma_x - \sigma_y}{2}\right]\sin 2\theta + \tau_{xy}\cos 2\theta \qquad \text{——— (2)}$$

Now substitute

$\sigma_{\theta\ = 30^\circ} = 60\,\text{MPa}$

$\sigma_{\theta\ -30^\circ} = 100\,\text{MPa}$

$$60 = \frac{\sigma_z + 80}{2} + \left[\frac{\sigma_x - 80}{2}\right] \times \cos 60 + \tau_{xy}\sin 60 \qquad \text{——— (3)}$$

$$100 = \frac{\sigma_x + 80}{2} + \left[\frac{\sigma_x - 80}{2}\right]\cos -60 + \tau_{xy}\sin(-60) \qquad \text{——— (4)}$$

(3) + (4)

$$160 = \sigma_x + 80 + (\sigma_x - 80)\cos 60 + 0$$

$\sigma_x = 80 \rightarrow$ Substitute in Equation (3)

$$\tau_{xy} = -23\,\text{MPa}$$

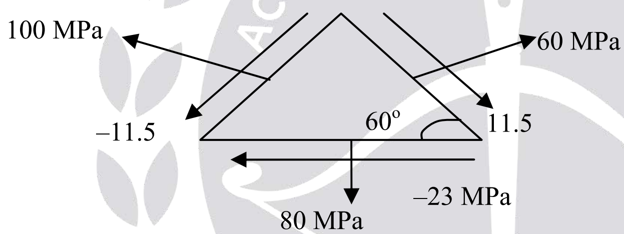
Now

$$\tau_{30^\circ} = -\left[\frac{80-80}{2}\right]\sin 60 - 23\times\cos 60$$

$$\tau_{30^\circ} = -11.5\,\text{MPa}$$

$$\tau_{-30^\circ} = -11.5\,\text{MPa}$$

Given state of stress element is



Principle stress are

$$\sigma_{1,2} = \frac{\sigma_x + \sigma_y}{2} \pm \sqrt{\left[\frac{\sigma_x - \sigma_y}{2}\right]^2 + \tau_{xy}^2}$$

$$\sigma_{1,2} = \frac{80+80}{2} \pm \sqrt{0^2 + 23^2}$$

$$\sigma_1 = 103\,\text{MPa}$$

$$\sigma_2 = 57\,\text{MPa}$$

(c)

(i) Describe how the compounds of clinker affect the properties of cement.

(12)

Sol: The compounds of cement clinker and how they affect the properties of cement are briefly discussed below:

Tricalcium Silicate (C_3S): This compound is supposed to have the best cementing properties among the different compounds of cement. It has high rate of hydrolysis and is responsible for 7-day hardness and strength of cement. However, high C_3S content leads to high heat of hydration. It has a heat of hydration of about 500 J/gm.

Dicalcium Silicate (C_2S): This compound hydrates and hardens slowly and takes longer time to contribute to the strength compared to C_3S. But in a period of one year, its contribution to the strength and hardness is proportionately equal to C_3S. It also imparts resistance to chemical attack, as the hydration of this compound releases less $Ca(OH)_2$ compared to C_3S. However, high C_2S content reduces the rate of early strength gain, decreases resistance to freezing and thawing and decreases the heat of hydration. It has a heat of hydration of about 260 J/gm.

Tricalcium Aluminate (C_3A): This compound rapidly reacts with water and may lead to immediate stiffening of the cement paste, known as flash set. This action is regulated by the addition of 2 – 3 % gypsum at the time of grinding of clinkers. It is responsible for the initial set, a greater tendency to volume changes causing cracking and has high heat of hydration. Raising the C_3A content reduces the setting time, weakens resistance to sulphate attack and lowers the ultimate strength. It has a heat of hydration of about 865 J/gm.

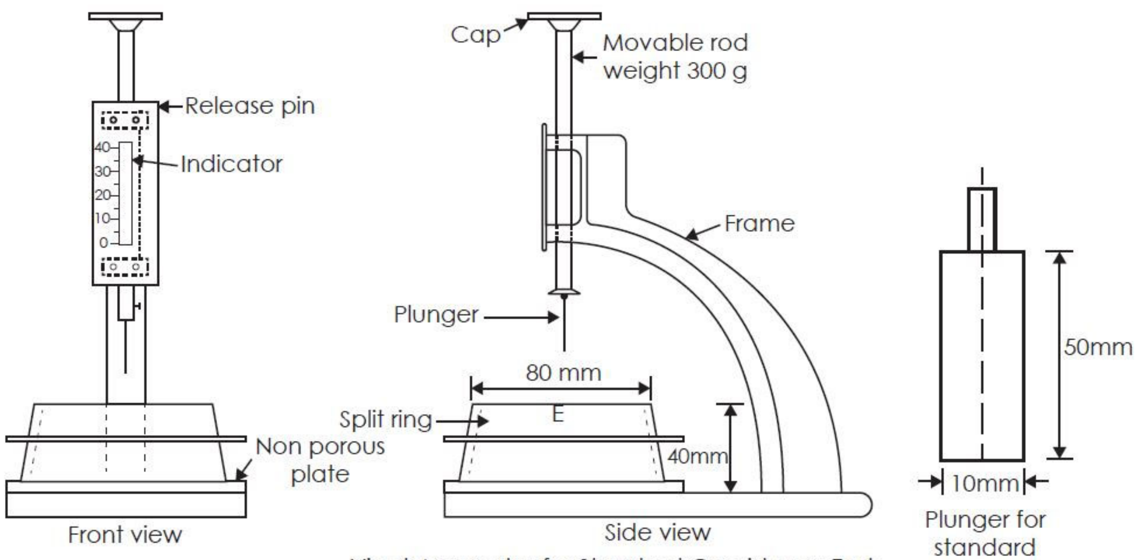
Tetracalcium Alumino Ferrite (C_4AF): This compound has poor cementing value and is also responsible for flash set, but to a lesser extent compared to C_3A. It generates less heat of hydration compared to C_3A. It has a heat of hydration of about 420 J/gm.

(ii) What do you mean by normal consistency of cement? What is its significance? How is it tested? (8)

Sol: The Normal Consistency or Standard Consistency of a cement is defined as the percentage of water required to make a workable cement paste. It can also be defined as the percentage of water required to make a cement paste which will permit a Vicat plunger to penetrate a depth of 33 to 35 mm from the top (5 to 7 mm from the bottom) of the mould of the Vicat Apparatus.

Test Apparatus:

The apparatus used for performing this test is the Vicat apparatus assembly with of a plunger having 10 mm diameter and 50 mm length and a mould which is 40 mm in height and 80 mm in diameter.



Vicat Apparatus for Standard Consistency Test

Test Procedure:

A cement paste is prepared by adding water 24% by weight of cement for the first trial.

The time of mixing should not be less than 3 minutes and should not be more than 5 minutes.

Now the paste is filled in the Vicat mould and the top is levelled off.

The mould is now placed under the plunger and is brought down to touch the surface of the paste.

Now the plunger is suddenly released and allowed to sink into the cement paste by its own weight.

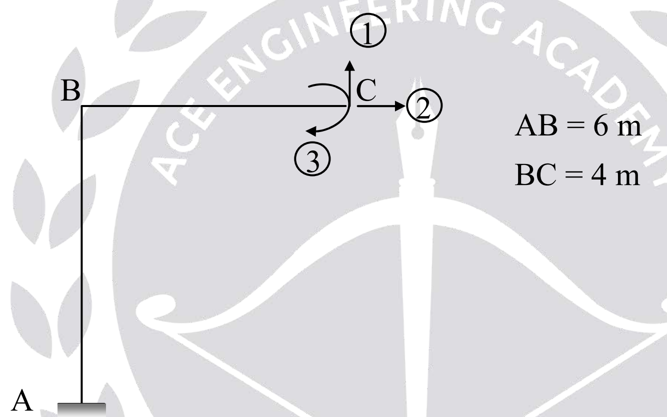
The depth of penetration of the plunger is noted down.

The whole experiment is repeated with increments of water cement ratio until such a time when the plunger penetrated to a depth of 33 to 35 mm from the top, or 5 to 7 mm from the bottom, of the mould.

The percentage of water at which the plunger penetrated to a depth of 33 to 35 mm from the top, or 5 to 7 mm from the bottom, of the mould is known as the Normal Consistency or Standard Consistency of the cement, which is denoted as P.

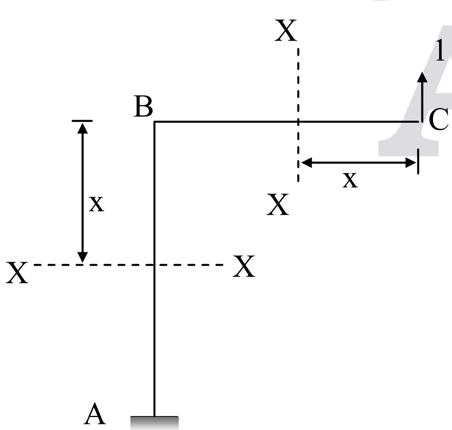
03.

(a) Develop the flexibility matrix for the beam shown in figure, with respect to the generalized coordinates mentioned. EI is constant for all members. (20)

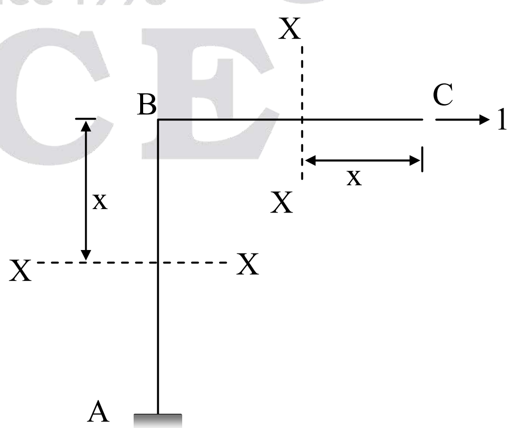


Sol:

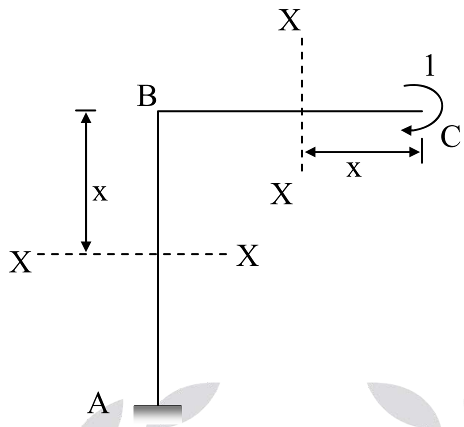
The displacements can be computed easily by applying the unit load method.



m_1 – values



m_2 – values



m_3 – values

Sagging + ve, hogging –ve

Member	Limits	m_1	m_2	m_3
CB	0 to 4	x	0	-1
BA	0 to 6	4	-x	-1

Displacement $\Delta = \int_0^L \frac{mm}{EI} dx$

$$f_{11} = \int_0^L \frac{m_1 m_1}{EI} dx = \int_0^4 \frac{x^2}{EI} dx + \int_0^6 \frac{16}{EI} dx = \frac{64}{3EI} + \frac{96}{EI} = \frac{352}{3EI}$$

$$f_{21} = \int_0^L \frac{m_2 m_1}{EI} dx = \int_0^4 0 + \int_0^6 \frac{(-x)(4)}{EI} dx = \frac{-72}{EI}$$

$$f_{31} = \int_0^L \frac{m_3 m_1}{EI} dx = \int_0^4 \frac{(-1)(x)}{EI} dx + \int_0^6 \frac{(-1)(4)}{EI} dx$$

$$= \frac{-8}{EI} + \frac{-24}{EI}$$

$$f_{31} = \frac{-32}{EI}$$

To generate the 2nd column of the flexibility matrix, we apply a unit force at coordinate '2' only and compute the displacement at all coordinates.

$$f_{12} = f_{21} = \frac{-72}{EI}$$

$$f_{22} = \int_0^L \frac{m_2 m_2}{EI} dx = \int_0^4 0 + \int_0^6 \frac{(-x)^2}{EI} dx = \frac{72}{EI}$$

$$f_{32} = \int_0^L \frac{m_3 m_2}{EI} dx = \int_0^4 \frac{(-1)(0)}{EI} + \int_0^6 \frac{(-1)(-x)}{EI} dx$$

$$= \frac{18}{EI}$$

To generate the 3rd column of the flexibility matrix, we apply a unit force at coordinate '3' only and compute the displacements at all coordinates.

$$f_{13} = f_{31} = \frac{-32}{EI}$$

$$f_{23} = f_{32} = \frac{18}{EI}$$

$$f_{33} = \int_0^L \frac{m_3 m_3}{EI} dx = \int_0^4 \frac{(-1)^2}{EI} dx + \int_0^6 \frac{(-1)^2}{EI} dx \text{ f}$$

$$= \frac{4}{EI} + \frac{6}{EI} \quad = \frac{10}{EI}$$

$$\text{Flexibility matrix 'f'} = \begin{bmatrix} \frac{352}{3EI} & \frac{-72}{EI} & \frac{-32}{EI} \\ \frac{-72}{EI} & \frac{72}{EI} & \frac{18}{EI} \\ \frac{-32}{EI} & \frac{18}{EI} & \frac{10}{EI} \end{bmatrix}$$

(b) A bar of length 1.2 m, diameter 40 mm is subjected to an axial tensile load of 130 kN and a twisting moment of 600 N.m. If the same material yielded at an axial stress of 200 N/mm^2, determine the safety factor associated with the bar, considering

(i) Principal stress failure theory

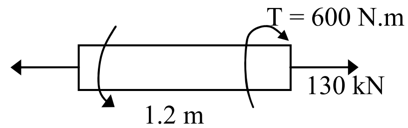
(ii) Maximum shear stress theory

(iii) Distortional strain energy theory

Take E = 200 GPa and μ = 0.25. (20)

Sol:

Given



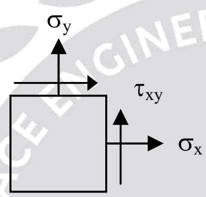
$\sigma_{yt} = 200\,\text{MPa}$

d = 40 mm, $\nu = 0.25, E = 200\,\text{GPa}$

Find, FOS = ?

Step – 1:

Find the $\sigma_1, \sigma_2, \sigma_3$



$$\sigma_x = \frac{130\times10^3}{\frac{\pi}{4}\times40^2} = 103.45\,\text{MPa}$$

$$\sigma_y = 0$$

$$\tau_{xy} = \frac{16T}{\pi d^3} = \frac{16\times600\times10^3}{\pi\times40^3} = 47.7\,\text{MPa}$$

$$\sigma_{1,2} = \frac{\sigma_x+\sigma_y}{2} \pm \sqrt{\left[\frac{\sigma_x-\sigma_y}{2}\right]^2 + \tau_{xy}^2} = 51.725 \pm \sqrt{4950}$$

$$= 51.725 \pm 70.38$$

$$\sigma_1 = 122\,\text{MPa}$$

$$\sigma_2 = -18.6\,\text{MPa}, \sigma_3 = 0$$

Case – (1)

Principle stress failure theory

$$\sigma_1 = \frac{\sigma_y t}{F.S}$$

$$F.S = \frac{200}{122} = 1.63$$

Case – 2:

Max shear stress theory

$$\tau_{max} = \max\left[\left[\frac{\sigma_1 - \sigma_2}{2}, \frac{\sigma_1}{2}, \frac{\sigma_2}{2}\right]\right] = \frac{\sigma_y t}{2 \times f.s}$$

$$122 + 18.6 = \frac{\sigma_y t}{F.S}$$

F.S = 1.42

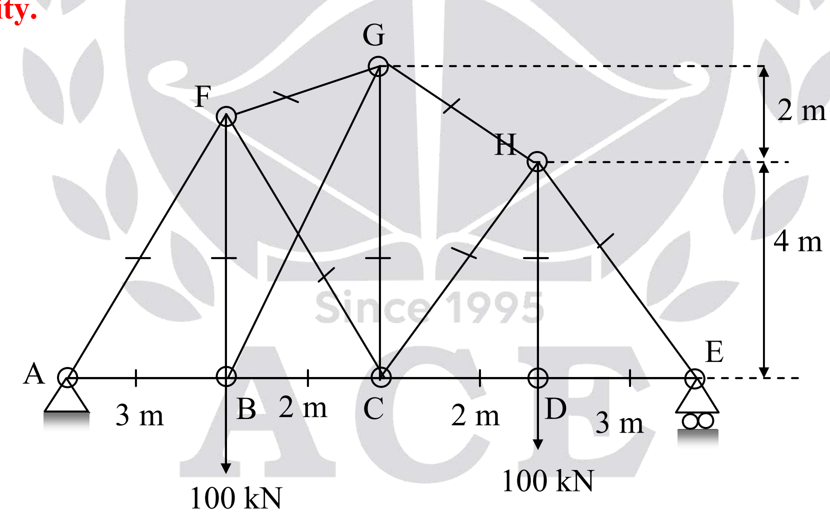
Case – 3:

Distortion energy theory

$$\sqrt{\sigma_1^2 - \sigma_1\sigma_2 + \sigma_2^2} = \frac{\sigma_y t}{F.S}$$

F.S = 1.51

(c) Determine the forces in the members of the truss shown in figure. All members have same axial rigidity. (20)



Sol: Note:

Insufficient data. Length of members FB is missing.

Assume length of member 'FB' is 4 m

$D_{se} = 0$

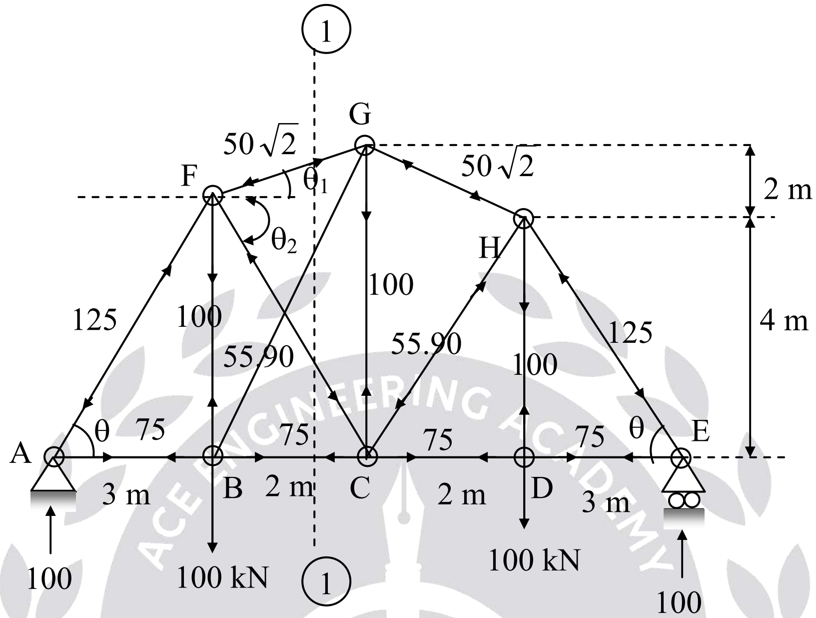
$D_{si} = m - (2j - 3)$

$= 14 - (2\times 8 - 3) = 1$

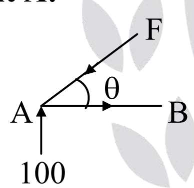
$D_s = 1$

Chosen redundant member is 'BG'

P- Values:



@ joint A:



$\cos\theta = 3/5$

$\sin\theta = 4/5$

$\Sigma V = 0$

$F_{AF} \sin\theta = 100$

$F_{AF} = 100 \times \dfrac{5}{4} = \dfrac{500}{4} = 125 \text{ kN (compression)}$

$\Sigma H = 0$

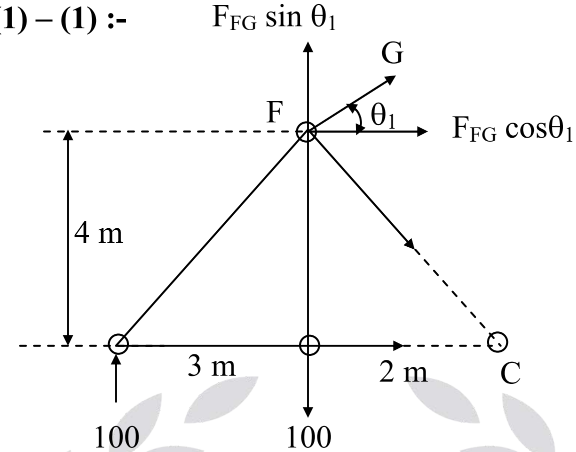
$F_{AB} = F_{AF} \cos\theta = 125 \times \dfrac{3}{5} = 75 \text{ kN (Tension)}$

$\theta_1 = 45^o$

$\tan\theta_2 = \dfrac{4}{2}$

$\theta_2 = 63.43^o$

Consider section (1) – (1) :-



$\Sigma M_C = 0$ ↻ –ve ↺ +ve [left part]

$+ F_{FG} \sin 45^o \times 2 + F_{FG} \cos 45^o \times 4 + 100 \times 5 - 100 \times 2 = 0$

$$3\sqrt{2}F_{FG} + 300 = 0$$

$$F_{FG} = -\frac{300}{3\sqrt{2}} = -50\sqrt{2}$$

$$F_{FG} = 50\sqrt{2}\text{kN}(\text{comp})$$

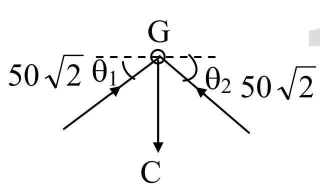
$\Sigma V = 0$ [left part]

$$F_{FC} \sin \theta_2 + 100 = 100 + 50\sqrt{2} \sin \theta_1$$

$$F_{FC} \sin 63.43^o = 50\sqrt{2} \times \frac{1}{\sqrt{2}}$$

$$F_{FC} = 55.90 \text{ kN (comp)}$$

@ joint 'G' :-

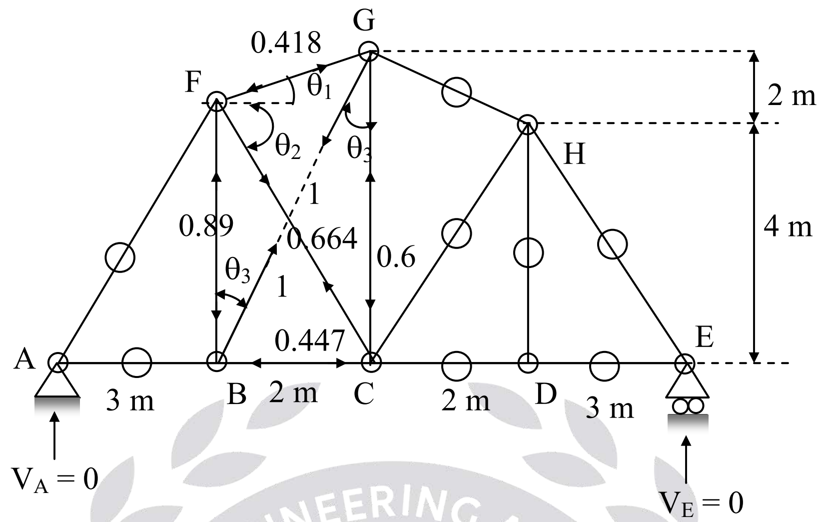


$\Sigma V = 0$

$$50\sqrt{2} \times \frac{1}{\sqrt{2}} + 50\sqrt{2} \times \frac{1}{\sqrt{2}} = F_{GC}$$

$F_{GC} = 100$ kN (Tension)

K – values:



$\text{Tan } \theta_3 = \dfrac{2}{4} \Rightarrow \theta_3 = 26.56^o$

$\Sigma V = 0$

$V_A + V_E + \cos\theta_3 = \cos\theta_3$

$V_A + V_E = 0$

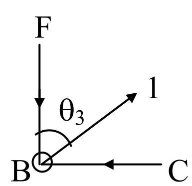
$\Sigma M_A = 0$ ↻ –ve ↺ +ve

$- V_E \times 10 + \cos\theta_3 \times 5 - \cos\theta_3 \times 3 - \sin\theta_3 \times 6 = 0$

$10\, V_E = 4.47 - 2.68 - 2.68$

$V_E = 0$

@ joint 'B' :-



$\Sigma H = 0$

$F_{BC} = \sin\theta_3$

$= \sin 26.56$

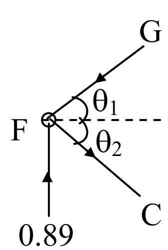
$F_{BC} = 0.447$ (compression)

$\Sigma V = 0$

$F_{FB} = \cos\theta_3 = \cos 26.56$

$F_{FB} = 0.89$ (comp)

@ joint F:-



$\theta_1 = 45^o$

$\theta_2 = 63.43^o$

$\Sigma H = 0$

$F_{FG} \cos 45^o = F_{FC} \cos 63.43^o$

$$F_{FG} = \frac{F_{FC} \cos 63.43}{\cos 45}$$

$F_{FG} = 0.63\ F_{FC}$

$\Sigma V = 0$

$F_{FG} \sin 45^o + F_{FC} \sin 63.43 = 0.89$

$0.63\ F_{FC}$

$1.34\ F_{FC} = 0.89$

$F_{FC} = 0.664$ (Tension)

$F_{FG} = 0.63 \times 0.664 = 0.418$ (comp)

@ joint 'C':-

$\Sigma V = 0$

$F_{CG} = 0.664 \sin \theta_2$

$= 0.664 \sin 63.43 = 0.6$ (comp)

Tension + ve

Compression -ve

Member	P	K	L	PKL	K^2L	S = P + KX
AB	75	0	3	0	0	75 (Tension)
BC	75	-0.447	2	-67.05	0.4	48.34 (Tension)
CD	75	0	2	0	0	75 (T)
DE	75	0	3	0	0	75 (T)
BF	100	-0.89	4	-356	3.16	47 (T)
GC	100	-0.6	6	-360	2.16	64.21 (T)
DH	100	0	4	0	0	100 (T)
AF	-125	0	5	0	0	-125 (C)
FG	$-50\sqrt{2}$	-0.418	$2\sqrt{2}$	83.6	0.5	-95.64 (C)
FC	-55.90	0.664	$2\sqrt{5}$	-166	1.97	-16.3 (C)

BG	0	1	$2\sqrt{10}$	0	6.32	59.64 (T)
CH	-55.90	0	$2\sqrt{5}$	0	0	-55.90 (C)
GH	$-50\sqrt{2}$	0	$2\sqrt{2}$	0	0	$-50\sqrt{2}$ (C)
EH	-125	0	5	0	0	-125 (C)
			Σ	-865.45	14.51	

Force in redundant member $'X' = \dfrac{\sum \frac{PKL}{AE}}{\sum \frac{K^2L}{AE}}$

$$X = -\frac{(-865.45)}{14.51}$$

$$X = 59.64$$

04. (a)

(i) Differentiate in brief between Thermoplastic and Thermosetting plastic. (8)

Sol: **Thermo-plastic:** The thermo-plastic or heat non-convertible group is the general term applied to the plastics which become soft when heated and hard when cooled. It is possible to shape and reshape this plastic by means of heat and pressure. One important advantage of this variety of plastics is that the scrap obtained from old and warn-out articles can be effectively used again.

Thermo-setting Plastic: The thermo-setting or heat convertible group is the general term applied to the plastics which become rigid when moulded at suitable pressure and temperature. The thermosetting plastics are soluble in alcohol and certain organic solvents, when they are in thermo-plastic stage. This property is utilized for making paints and varnishes from these plastics. The thermo-setting plastics are durable, strong and hard. They are available in a variety of beautiful colours. They are mainly used in engineering application of plastics.

S.No	Thermoplastics	Thermosetting Plastics
1	These solidify when cooled below a particular temperature.	These solidify when heated above a particular temperature.
2	These have weak forces of interaction among chains.	The whole mass of polymers is well connected with strong covalent bonds.
3	Average molecular weight can be defined.	Average molecular weight cannot be defined.
4	Expensive	Cost-effective
5	Highly recyclable.	Cannot be recycles.
6	These melt when heated	More resistant to high temperatures

(ii) Discuss in brief the methods of preserving timber by water soluble preservatives. (12)

Sol: Water soluble preservatives are odourless organic or inorganic salts and are adopted for inside locations only. If applied over outside surfaces, the salts can be leached by rain water. Examples of leachable type of preservatives are zinc chloride, boric acid (borax), etc. Zinc chloride, sodium fluoride and sodium-penta-chloro-phenate are toxic to fungi. These are expensive and odourless (except for sodium-penta-chloro-phenate). Benzene-hexa-chloride is used as spray against borers. Boric acid is used against Lyctus borers and to protect plywood in tea chests.

Some of the other water-soluble preservatives are fixed type and are as follows:

Copper-chromate-arsenic composition:

Arsenic-pentaoxide ($As_2O_5.2H_2O$) – 1 part.

Copper Sulphate ($CuSO_4.5H_2O$) – 3 parts.

Sodium or potassium dichromate $(Na \text{ or } K)_2Cr_2CO_7$ – 4 parts.

Acid-cupric-chromate composition:

Chromic acid – 1.7 parts.

Copper sulphate - 50 parts.

Sodium dichromate – 48.3 parts.

Chromate-zinc chloride composition:

Zinc chloride – 1 part.

Sodium or potassium dichromate – 1 part.

Copper-chrome-boric composition:

Boric acid – 1.5 parts.

Copper sulphate – 3 parts.

Sodium or potassium dichromate – 4 parts.

Zinc-meta-arsenate composition:

Arsenious trioxide – 3 parts.

Zinc oxide – 2 parts.

Acetic acid – Just to keep the above in solution under operating conditions.

Zinc-chrome-boric composition:

Boric acid – 1 part.

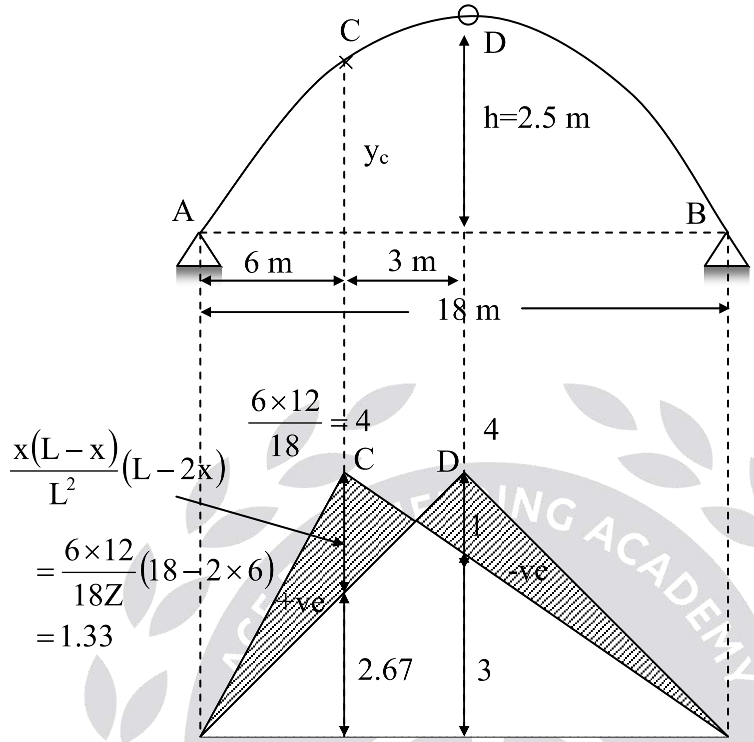
Zinc chloride – 3 parts.

Sodium dichromate – 4 parts.

Water – 100 parts.

(b) Sketch influence line diagram for the bending moment at a point 'c' located 6 m from one of the supports of a three hinged symmetrical parabolic arch having span of 18 m and central rise 2.5 m. Locate the point from where the moving load changes the sign of bending moment at c. (20)

Sol:



ILD for BMC

By inspection of influence line for BMD, it is clear that maximum positive BM occurs when load is on the section while maximum negative BM occurs when load is at the crown.

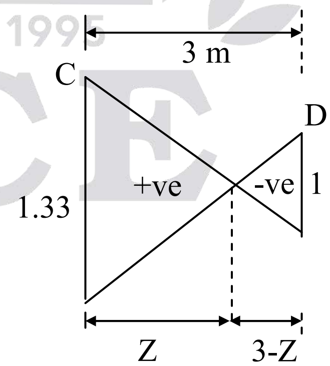
From similar triangle concept

$$\frac{Z}{1.33}=\frac{3-Z}{1}$$

$Z = 1.33(3 - Z)$

$2.33\ Z = 4$

$Z = 1.716$ from point 'C'

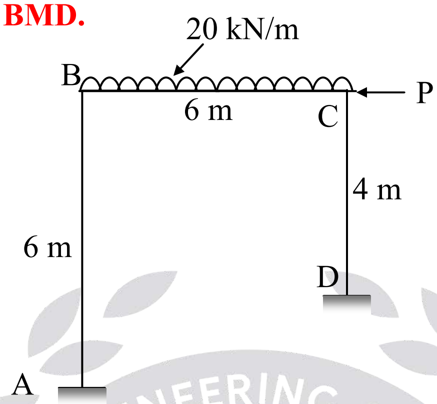


Location of point from where BM changes sign from point C is 1.716 m.

(or)

From support A is 6 + 1.716 = 7.716 m

(c) **Frame ABCD shown in figure, is acted upon by a UDL of intensity 20 kN/m on the horizontal span. What should be value of horizontal force 'P' applied at C, that will prevent sway of the frame? Draw BMD.** **(20)**



EI is same for all members

Sol: Distribution Factors (DF) :-

Joint	Member	Relative Stiffness (K)	Total relative stiffness IK	Distribution Factors DF $=\frac{K}{IK}$
B	BA	I/6	I/3	½
	BC	I/6		½
C	CB	I/6	$\frac{5I}{12}$	2/5
	CD	I/4		3/5

Non sway analysis :-

Fixed end moments:

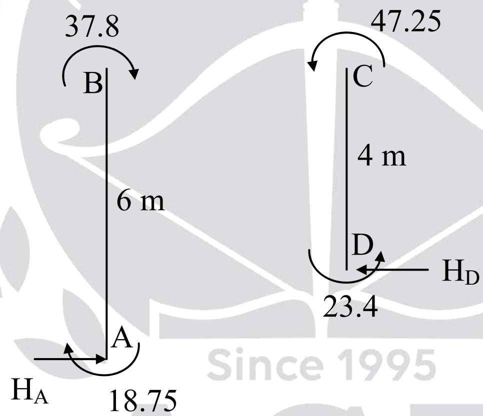
$$M_{FAB} = M_{FBA} = M_{FCD} = M_{FDC} = 0$$

$$M_{FBC} = \frac{-wL^2}{12} = \frac{-20\times 6^2}{12} = -60\ \text{kN}-\text{m}$$

$$M_{FCB} = \frac{+wL^2}{12} = 60\ \text{kN}-\text{m}$$

The moment distribution is carried out below:

	A		B		C		D
			1/2	1/2	2/5	3/5	
FEM	0	0	-60	60	0	0
Balance		30	30	-24	-36	
Carry Over	15		-12	15		-18
Balance		6	6	-6	-9	
Carry Over	3		-3	3		-4.5
Balance		1.5	1.5	-1.2	-1.8	
Carry Over	0.75		-0.6	0.75		-0.9
Balance		0.3	0.3	-0.3	-0.45	
Non-sway moments	18.75	37.8	-37.8	47.25	-47.25	-23.4



Horizontal reaction at A = $\dfrac{18.75+37.8}{6} = 9.425 \text{ kN} \rightarrow$

Horizontal reaction at D = $\dfrac{23.4+47.25}{4} = 17.66 \text{ kN} \leftarrow$

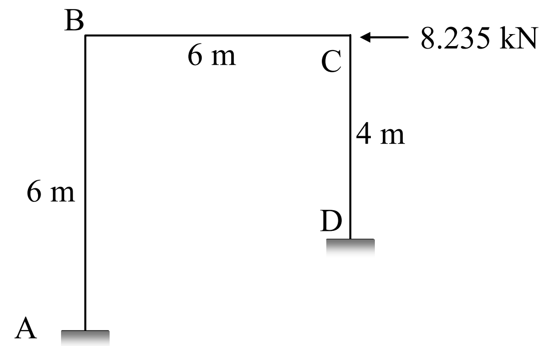
$\therefore$ Sway force (unbalanced horizontal force)

= 17.66 – 9.425

= 8.235 kN $\leftarrow$ $\qquad$ $\therefore$ Value of force 'P' preventing sway = 8.235 kN

Sway analysis:

The frame will now be analysed for a sway force of 8.235 kN ($\leftarrow$)



Since the force sways towards left the initial equivalent moments are positive. The ratio of the initial equivalent moments at the tops of the columns equals.

$$\frac{m_{BA}}{m_{CD}} = \frac{I/6^2}{I/4^2} = \frac{16}{36} = \frac{4}{9}$$

Let us take $m_{BA} = +4$ kN-m and $m_{CD} = +9$ kN-m

$m_{AB} = +4$ kN-m and $m_{DC} = +9$ kN-m

with those initial moments, the moment distribution is worked out below:

	A	B (1/2)	B (1/2)	C (2/5)	C (3/5)	D
Balance	4	4	0	0	9	9
		-2	-2	-3.6	-5.4	
Carry Over	-1		-1.8	-1		-2.7
Balance		0.9	0.9	0.4	0.6	
Carry Over	0.45		0.2	0.45		0.3
Balance		-0.1	-0.1	-0.18	-0.27	
Carry Over	-0.05		-0.09	-0.05		-0.135
Balance		0.045	0.045	0.02	0.03	
Moments	3.4	2.845	-2.845	-3.96	3.96	6.465

Horizontal reaction at A = $\frac{3.4 + 2.845}{6} = 1.04$ kN $\rightarrow$

Horizontal reaction at B = $\dfrac{3.96+6.465}{4} = 2.60\text{ kN} \rightarrow$

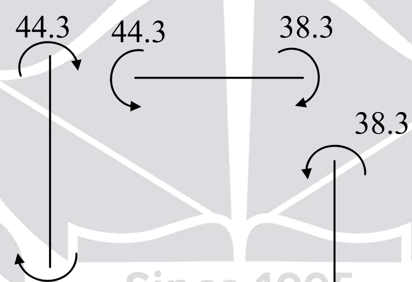
Resolving horizontally S = 1.04 + 2.60 = 3.64 kN

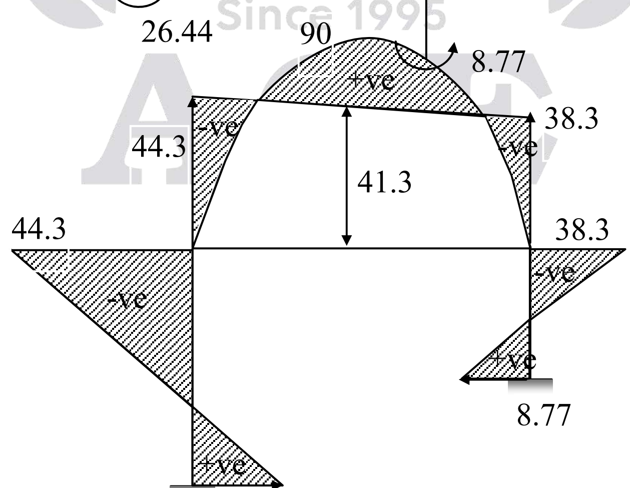
For the actual sway force of 8.235 kN, the actual sway moments will be

$$\frac{8.235}{3.64}\times \text{moments}$$

	A		B		C	D
Non-sway moments	18.75	37.8	-37.8	47.25	-47.25	-23.4
Actual sway moments $\frac{8.235}{3.64}\times$moments	7.69	6.50	-6.50	-8.95	8.95	14.626
Final moments	26.44	44.3	-44.3	38.3	-38.3	-8.77

Final moments:





BMD

05.

(a) A tie member of a truss consisting of an angle section ISA 65 × 65 × 6 is welded to a gusset plate. Design a fillet weld to transmit a load equal to full tensile strength of the plate. Assume shop weld. Take grade of steel E 250 (Fe 410). Also sketch the weld length. (12)

Properties of ISA 65 ×65 × 6

A = 744 mm^2

C_Z = 18.1 mm

Thickness of gusset plate is 10 mm.

Sol:

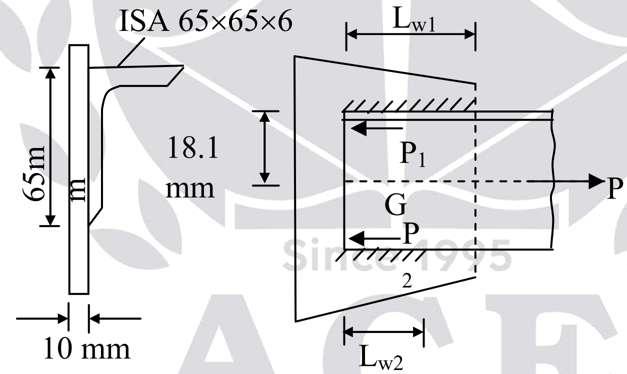
For Fe410 grade steel, $f_u = 410$ N/mm^2 & $f_y = 250$ N/mm^2

For shop welding = 1.25

Design axial tensile strength of an angle tie based on gross section yielding

$$P = T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{744 \times 250}{1.10} = 169.09 \times 10^3\ N$$

Let S and L_w be the size and effective length of fillet weld respectively.



Minimum size of fillet weld based on 10mm thicker gusset plate

S_{min} = 3 mm

Maximum size of fillet weld

$$S_{max} = \frac{3}{4} \times 6 = 4.5 \text{ mm}$$

Adopt size of fillet weld S = 4.5 mm

Effective threat thickness $t_t = K \times S = 0.7 \times 4.5 = 3.15$ mm

Equating the design axial tensile strength of an angle tie (P) = Design shear strength of fillet weld (P_{dw})

$$169.09 \times 10^3 = L_w \times t_t \times \frac{f_u}{\sqrt{3}\gamma_{mw}}$$

$$169.09 \times 10^3 = L_w \times 3.15 \times \left(\frac{410}{\sqrt{3} \times 1.25}\right)$$

$$L_w = 283.46 \text{ mm}$$

Assuming above effective weld length to be arranged top and bottom edges of angle along length only.

Let L_{w1} and L_{w2} are length of weld on top and bottom weld and L_{w1} and L_{w2} are to be arranged in such a way that load must pass through C.G of an angle.

$$L_{w1} + L_{w2} = 283.46 \text{ mm} \ldots\ldots\ldots (1)$$

Taking moments of weld strengths and loads about top edge of an angle

$$P_2 \times 65 + P_1 \times 0 = P \times 18.1$$

$$\left(L_{w2} \times 3.15 \times \frac{410}{\sqrt{3} \times 1.25}\right) \times 65 = 169.09 \times 10^3 \times 18.1$$

$$L_{w2} = 78.93 \text{ mm} \approx 79 \text{ mm}$$

$$L_{w1} = 283.43 - 78.93 = 204.5 \text{ mm} \approx 205 \text{ mm}$$

(b) Find the web buckling and web crippling strength of a beam (ISLB 350) simply supported at both ends. Assume the stiff bearing length 100 mm and grade of steel E 250 . (12)

Setion properties of ISLB 350 : t_w = 7.4 mm

t_f = 11.4 mm

R = 16 mm

R = Radius of root

Given: Desing compressive stress f_{cd}, N/mm²

KL/r	F_{cd}
90	121
100	107
110	94.6

Sol:

For E250 grade of steel $f_y = 250\ N/mm^2$

Stiff bearing length, b = 100mm

For ISLB350, h=350mm, t_w=7.4mm, t_f=11.4mm, R = 16mm

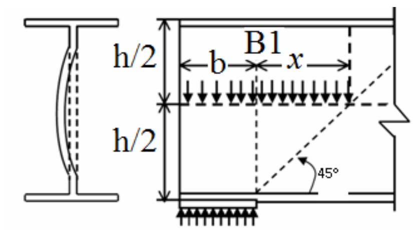
Depth of web $d = h - 2\times(t_f+R_1) = 350-2\times(11.4+16) = 295.2mm$

$$\varepsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1.0$$

$$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.89 \le 67\varepsilon$$

Hence web is free from shear buckling

Web buckling strength at support $F_w = B_1 \times t_w \times f_{cd}$



$B_1 = (b + x) = 100 + 350/2 = 275mm$

Effective length of web, KL = 0.7d = 0.7×295.2=206.64mm

Moment of inertia of web $I_{eff} = \frac{b\times t_w^3}{12} = \frac{100\times 7.4^3}{12} = 3376.86\ mm^4$

Effective area of web, $A_{eff} = b \times t_w = 100\times7.4 = 704\ mm^2$

$$r = \sqrt{\frac{I_{eff}}{A_{eff}}} = \sqrt{\frac{3376.86}{704}} = 2.13mm$$

Effective slenderness ratio of web KL/r = 206.64/2.136=96.74

For KL/r = 96.74 , Design compressive stress f_{cd} from table

$$f_{cd} = 121 - \frac{(121-107)}{(100-90)}\times(96.74-90) = 111.56\ N/mm^2$$

Web buckling strength at support $F_w = B_1 \times t_w \times f_{cd}$

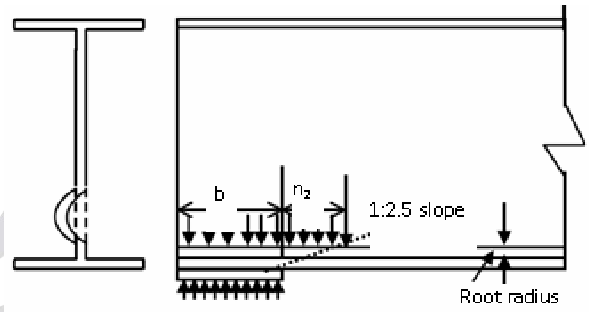
$= 275\times7.4\times111.56 = 227.02\times10^3\ N = 227.02\ kN$

Web crippling strength at support $F_w = B_1 \times t_w \times f_{cd}$

The design web bearing strength at support $F_w = \dfrac{A_e \times f_y}{\gamma_{mo}}$

f_y = yield design strength of the web

A_e = Effective area of the web = $b_1 \times t_w$



The angle of dispersion of the load is assumed to be 1: 2.5

Bearing length $b_1 = b + n_1$ (under reaction at support)

$n_1 = 2.5\,(t_f + R_1) = 2.5\times(11.4+16) = 68.5\text{mm}$

Bearing length $b_1 = b + n_1 = 100 + 68.5 = 168.5\text{mm}$

The design web bearing strength

$$F_w = \frac{A_e \times f_y}{\gamma_{mo}} = \frac{168.5\times7.4\times250}{1.1} = 283.38\times10^3\,\text{N} = 283.38\text{kN}$$

(c) A symmetrical reinforced concrete frame building 25 m × 25 m in plan is located in seismic zone IV on hard soil. The height of the building is 30 m. Determine the base shear due to earthquake.

Given:

Z = 0.24, I = 1.5

Total dead load = 1,50,000 kN

Total live load (effective) = 50,000 kN

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

$$\frac{Sa}{g} = \begin{cases} 1+15T & 0 \le T \le 0.10 \\ 2.5 & 0.10 \le T \le 0.40 \\ 1/T & 0.40 \le T \le 4.0 \end{cases}$$ **(12)**

Sol: Given data

Plan dimensions: 25 m × 25 m

Seismic zone : IV

Building height : 30 m

Z = 0.24, I = 1.5

Total dead load = 1,50,000 kN

Total effective load = 50,000 kN

Base shear?

Fundamental time period

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

$$= 0.09\frac{30}{\sqrt{25}}$$

$$= 0.54$$

$$\frac{Sa}{g} = \frac{1}{T} = \frac{1}{0.54} = 1.85$$

Total load W = 150,000 + 50,000 = 2,00,000

For hard soil D = 1

(D : Damping factor)

Response reduction factor

R = 4 (Assumed because not mentioned in the questions clearly)

Base shear due to earthquake

$$V_b = \left(\frac{Z}{2}\right)\left(\frac{I}{R}\right)\left(\frac{Sa}{g}\right)WD$$

$$= \left(\frac{0.24}{2}\right)\left(\frac{1.5}{4}\right)(1.85) \times 200000 \times 1$$

$$= 16650 \text{ kN}$$

(d) A floor of an old building consists of 150 mm thick RC slab monolithic with the beam of width 300 mm and total depth 400 mm. The beams are spaced 3.5 m c/c and their effective span (simply supported) is 7 m. The beams are reinforced with 5 No. 28 φ bars as tension reinforcement. Determine the moment carrying capacity of the beams. Use M 25 and Fe 500. Adopt limit state method of design. Nominal cover = 30 mm. Diameter of the stirrups = 8 mm. (12)

Sol: Given data

Slab thickness, $D_f = 150$ mm

Beam width , $b_w = 300$ mm

Total depth, D = 400 mm

c/c distance = 3.5 m

Effective span L = 7 m

Area of tension steel, $A_{st} = 5 - 28$m mm ϕ

Material grades : M 25 and Fe-500

Stirrup diameter = 8 mm

Moment carrying capacity of the beam ?

Nominal cover = 30 mm

For M 25 $\Rightarrow f_{ck} = 25$ N/mm^2

For Fe - 500 $\Rightarrow f_y = 500$ N/mm^2

Effective depth of the beam

$$d = 400 + 150 - \left(30 + 8 + \frac{28}{2}\right) = 498 \text{ mm}$$

Distance b/n zero moments $l_o = L = 7$ m

(for simply supported beam)

Effective width of flange (Assume it is T –beam)

$$b_f = \frac{\ell_o}{6} + b_w + 6D_f \ngtr c/c$$

$$= \frac{7}{6} + 0.3 + 6 \times 0.15$$

$$= 2.37 \text{ m} < 3.5 \text{ m} \quad \therefore \text{ O.K}$$

Limiting depth of Neutral axis

$X_{u_{max}} = 0.46d \quad \rightarrow$ For Fe-500

$= 0.46 \times 498 = 229.08$ mm

Actual depth of neutral axis

Case (i) : Assume N.A lies with in the flange

Total compression force = Total tension force

$$C = T$$

$$0.36\, f_{ck}\, b_f\, X_u = 0.87\, f_y\, A_{st}$$

$$0.36 \times 25 \times 2370 \times X_u = 0.87 \times 500 \times 5 \times \frac{\pi}{4} \times 28^2$$

$$X_u = 62.8 \text{ mm} < D_f$$

$\therefore$ Assumption is correct

So the above beam section will behave like rectangular cross section hence analysis and design is similar to rectangular c/s.

$X_u < X_{u\,max}$ $\quad \therefore$ It is an under reinforced section

Moment of resistance of the beam

$$MR = TZ$$

$$= 0.87\, f_y\, A_{st}[d - 0.42 \times x_u]$$

$$= 0.87 \times 500 \times 5 \times \frac{\pi}{4} \times 28^2\,[498 - 0.42 \times 62.8]$$

$$= 631.63 \times 10^6 \text{ N - mm}$$

$$= 631.63 \text{ kN - m}$$

(e) What is Work breakdown structure (WBS) with respect to construction planning and Management? How it WBS classified into different levels? (12)

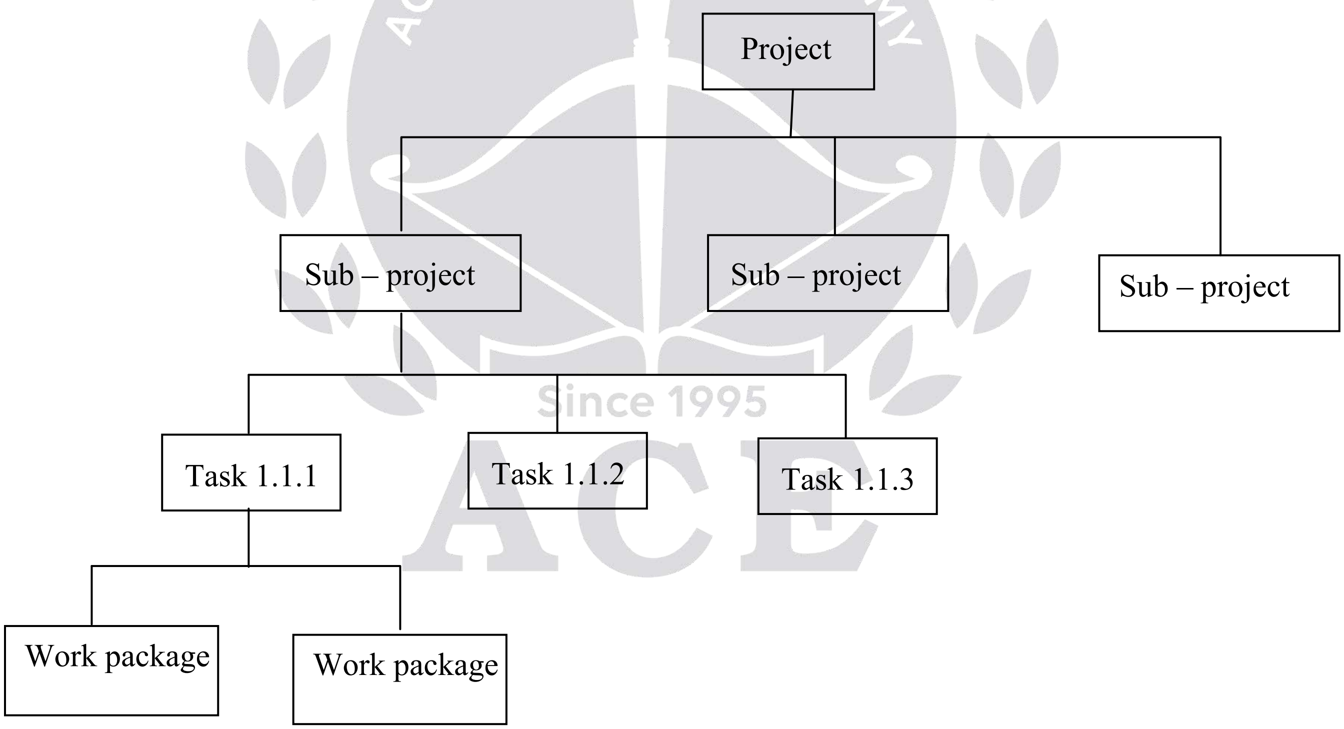
Sol:

Work breakdown structure (WBS) is a “deliverable – focused hierarchical grouping” of a project. It defines the total work scope of project.

Deliverables are tangible, measurable parts of the project, Non-deliverable items of work such as the designing, resource procurement and financing etc., are not include in the WBS.

The project work can be broken down into management parts arranged in a hierarchical order into levels of sub-projects, tasks, work packages and activities.

Level	Description	Criteria
1	Sub-Project level	An independent, deliverable end product requiring processing of multi-task having large volume of work
2	Task level	An identifiable and deliverable major work containing one (or) more work packages
3	Work package level	A sizeable, identifiable measurable, cost-able and controllable work item/package of activities
4	Activity level	Identifiable lower-level Job, operation (Or) process, which consumes time and possible resources



Work Break down Structure

WBS is core upon which construction project management processes are built. A good quality WBS should meet the core requirements for which it is created.

Each descending level of WBS signifies an increasingly detailed description of the elements of the preceding project level.

WBS can be represented in three familiar forms such as (i) hierarchical tree structure (ii) numbered levels table (iii) task matrix format

06.

(a) A simply supported reinforced concrete beam of size 300 mm × 500 mm is reinforced with 5 No.s 16 ϕ bars as tension reinforcement. Two bars are curtailed at quarter span from both ends. Find out the load carrying capacity (UDL) of the beam having effective span of 6 m. Also design the beam against shear force. Use M 25 and Fe 415. Nominal cover = 30 mm. Use limit state method of design. Show the reinforcement detail (cross-section) also. Use 2 No.s 12 ϕ bars as hanger bars.

$\frac{M_u}{bd^2}$	2	2.5	2.75	3	3.25	3.5
P_t	0.51	0.61	0.74	0.83	0.91	-

P_t	0.25	0.5	0.75	1.0	1.25
τ_c, MPa	0.36	0.49	0.57	0.64	0.7

(20)

Sol:

Given data:

Type of beam: Simply supported

Beam size: 300 mm × 500 mm (b × D)

Area of tension steel, A_{st} : 5 – 16 mm ϕ

Effective span, L : 6 m

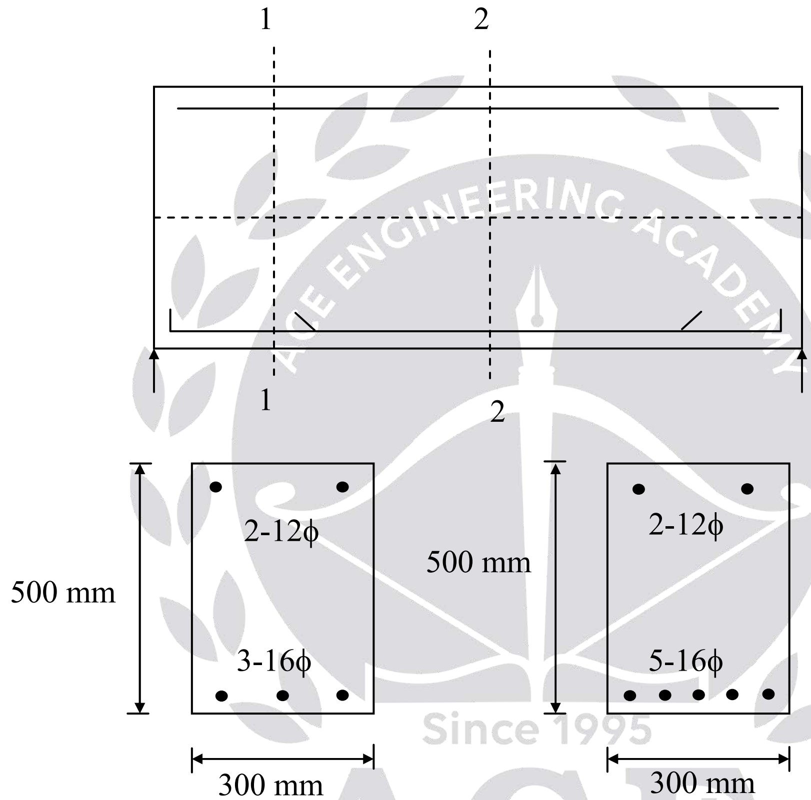
Grades of materials : M 25 and Fe – 415

Nominal cover : 30 mm

Hanger bars : 2 – 12 mm ϕ

(i) Load carrying capacity of the beam

(ii) Design for shear



C.S at 1-1 (At support)

C.S at 2-2 (At mid span)

Effective depth of the beam

d = D-(Nominal Cover + Dia of stirrup(Assumed) + half of dia of main steel)

$$= 500 - (30 + 12 + \frac{16}{2})$$

$$= 450\text{mm}$$

Percentage of area of tension steel

$$p_t = \frac{100Ast}{bd}$$

$$= \frac{100 \times 5 \times \frac{\pi}{4} \times 16^2}{300 \times 450} = 0.74\%$$

(i) Load Carrying capacity of the beam

From the table

$$P_t = 0.74 \quad \rightarrow \frac{m_u}{bd^2} = 2.75$$

$$\Rightarrow \quad m_u = 2.75\ bd^2$$

$$= 2.75 \times 300 \times 450^2$$

$$= 167.06 \times 10^6\ \text{N-mm}$$

$$= 167.06\ \text{kN-m}$$

(Max B.M at midspan is $\frac{W_u L^2}{8}$)

$$\frac{W_u L^2}{8} = 167.06$$

$$W_u = \frac{167.06 \times 8}{6^2}$$

$$= 37.12\ \text{kN/m}$$

Working load on the beam is

$$W = \frac{37.12}{1.5} = 24.75\ \text{kN/m}$$

(Load factor is 1.5)

Percentage of tension steel at

$$\text{Support } P_t = \frac{100Ast}{300 \times 450}$$

$$= \frac{100 \times 3 \times \frac{\pi}{4} \times 16^2}{300 \times 450} = 0.45\%$$

From the table for $P_t = 0.45\%$

P_t	τ_c(mPa)
0.25	0.36
0.50	0.49

$$\tau_c = 0.36 + \frac{0.49 - 0.36}{0.50 - 0.25}(0.45 - 0.25)$$

$$= 0.46\ \text{N/mm}^2$$

$\tau_v > \tau_c \therefore$ Not O.K

So design requires for shear Let us assume vertical stirrups of 2 legged 12mm diameter of grade Fe-250

Design shear force for stirrups

$$V_{us} = V_u - \tau_c bd$$

$$= 111.36\times10^3 - 0.46\times300\times450$$

$$= 49.26\times10^3 \text{N}$$

Spacing required for vertical stirrups

$$V_{us} = 0.87\ f_y\ Asv\ \frac{d}{S_v}$$

$$49.26\times10^3 = 0.87\times250\times2\times\frac{\pi}{4}\times12^2\times\frac{450}{S_v}$$

$S_v \approx 450$ mm > 300 mm $\therefore$ Not O.K

Spacing required for minimum shear reinforcement

$$\frac{A_{SV}}{bS_v} \geq \frac{0.4}{0.87\ f_y}$$

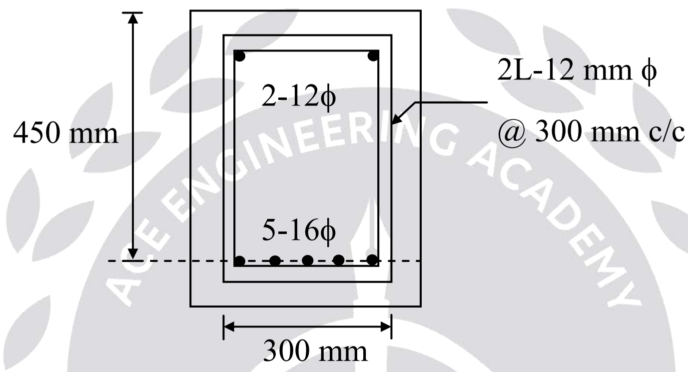
$$\frac{2\times\frac{\pi}{4}\times12^2}{300\times S_v} \geq \frac{0.4}{0.87\times250}$$

$S_v \leq 410$ mm

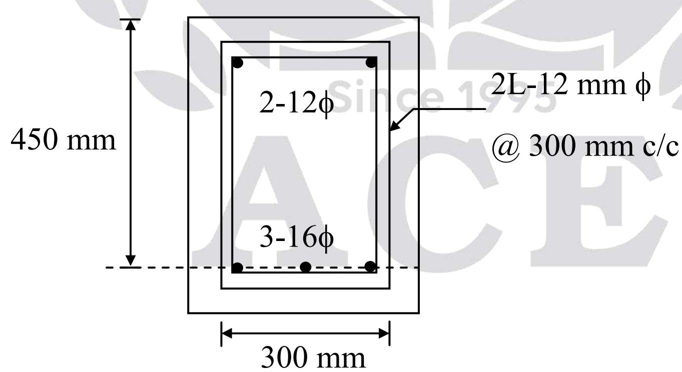
Check:

$$\left.\begin{array}{l}\text{(i) } 0.75d = 0.75 \times 450 = 337.5 \text{ mm} \\ \text{(ii) Cal } S_v = 450 \text{ mm} \\ \text{(iii) } S_{V_{min}} = 410 \text{ mm} \\ \text{(iv) } 300 \text{ mm}\end{array}\right\} \text{smaller}$$

Provide 2 legged – 12 mm ϕ @ 300 mm c/c with grade Fe-250



(At mid span)



(At supports)

(b) A bracket plate is connected to a flange of ISMB 500 as shown in figure. Find the safe load P carried by the joint. M 16 bolts of grade 4.6 are provided at a pitch of 50 mm and end distance 30 mm. (20)

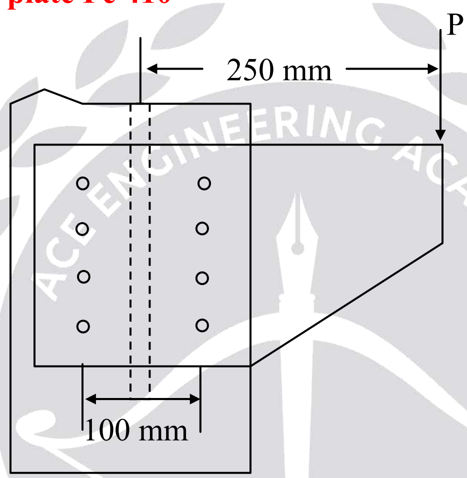
Given:

Thickness of bracket plate = 10 mm

Width of flange b_f = 180 mm

Thickness of flange t_f = 17.2 mm

Steel grade of bracket plate Fe 410



Sol:

For grade Fe 410 (E250) steel, $f_u = 410\ N/mm^2$ & $f_y = 250\ N/mm^2$

For grade M 16 bolt of 4.6 $f_{ub} = 400\ N/mm^2$ & $f_{yb} = 240\ N/mm^2$

Partial safety factors, $\gamma_{mb} = 1.25$, $\gamma_{mo} = 1.10$ & $\gamma_{ml} = 1.25$

Shank diameter of bolt d = 16 mm;

Diameter of bolt hole d_o = 18 mm;

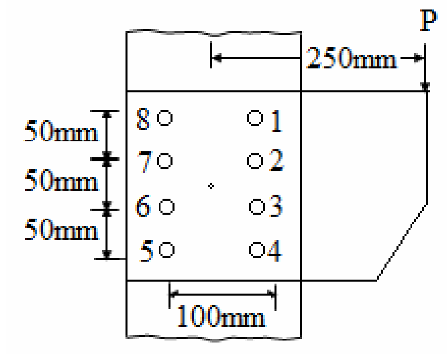
Pitch p = 50 mm & End distance e = 30 mm

Thickness of bracket plate t =10mm

Thickness of column flange t_f =17.2 mm

Let P be the factored eccentric load in kN

Twisting moment M = P×e = 250 P kN-mm



The maximum forced bolt is one, which is farthest from C.G. of bolt group [i.e. r $\rightarrow$ maximum (1, 4, 5, & 8)] and which are close to the applied load line.

[$\theta \rightarrow$ minimum (i.e. 1 and 4)]

Hence critical bolts are bolt no. 1 and bolt no. 4

Vertical shear force in any rivet due to P is F_a

$$F_a = \frac{P}{n} = \frac{P}{8} = 0.125P\,kN$$

Shear force in critical bolt due to M is $F_m = F_{m1} = F_{m4} = \dfrac{Mr_1}{\Sigma r^2}$

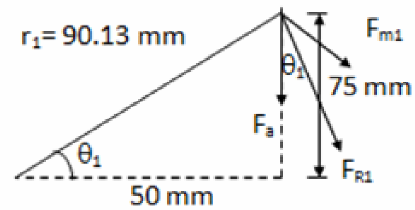
$$r_1 = r_4 = r_5 = r_8 = \sqrt{\left(\frac{100}{2}\right)^2 + 75^2} = 90.13 \text{ mm}$$

$$r_2 = r_3 = r_6 = r_7 = \sqrt{\left(\frac{100}{2}\right)^2 + 25^2} = 55.90 \text{ mm}$$

$$\Sigma r^2 = 4(r_1^2 + r_4^2 + r_5^2 + r_8^2) + 4(r_2^2 + r_3^2 + r_5^2 + r_6^2)$$

$$= 4(90.13)^2 + 4(55.90)^2 = 45000 \text{ mm}^2$$

$$F_m = F_{m1} = F_{m5} = \frac{Mr_1}{\Sigma r^2} = \frac{250P \times (90.13)}{45000} = 0.50 \text{ P}$$



$$\cos\theta_1 = \frac{50}{90.13} = 0.55$$

Maximum resultant shear force in critical bolt $F_{Rmax} = F_{R1} = F_{R4}$

$$F_{R\,max} = \sqrt{F_a^2 + F_{m1}^2 + 2F_aF_{m1}\cos\theta_1} = \sqrt{(0.125P)^2 + (0.5P^2) + 2(0.1P)(0.5P)0.55}$$

$F_{Rmax} = 0.578$ P

Design strength of one bolt V_{db}

V_{db} = Minimum of V_{dsb} and V_{dpb}

Design shear strength of one bolt (V_{dsb}), assuming the thread intercept the shear plane

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3}\,\gamma_{mb}}(n_n A_{nb} + n_s A_{sb}) = \frac{400}{\sqrt{3}\times 1.25}\left(1\times 0.78\times\frac{\pi}{4}(16)^2 + 0\right)$$

$= 28.97 \times 10^3$ N = 28.97 kN

Design bearing strength of one bolt (V_{dpb})

$$V_{dpb} = \frac{2.5\times k_b\times d\times t\times f_u}{\gamma_{mb}}$$

k_b is a bearing factor is lesser of

- $\frac{e}{3d_o} = \frac{30}{3\times 18} = 0.55$
- $\frac{p}{3d_o} - 0.25 = \frac{50}{3\times 18} - 0.25 = 0.675$
- $\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.97$
- 1.0

Hence bearing factor $k_b = 0.55$

$$V_{dpb} = \frac{2.5\times k_b\times d\times t\times f_u}{\gamma_{mb}} = 2.5\times 0.55\times 16\times 10\times\frac{410}{1.25}$$

$= 72.16 \times 10^3$ N = 72.16 kN

Design strength of one bolt V_{db} = 28.97 kN

For safety of bolt group as per LSD

Maximum resultant shear force in critical bolt $F_{Rmax} \leq$ Design strength of one bolt (V_{db})

Equating $\quad F_{Rmax} \leq V_{db}$

$$0.578\,P = 28.97 \Rightarrow P = \frac{28.97}{0.578} = 50.12\,kN$$

factored/design load on bracket plate P = 50.12 kN

$$\text{Safe load on bracket plate } P_s = \frac{P}{\gamma_f} = \frac{50.12}{1.5} = 33.41\,kN$$

(c) What is crane? How is it used in the construction industry? Briefly explain three different types of cranes that are being used in construction works. (20)

Sol:

A crane is a type of construction machine, equipped with boom, hoist ropes etc, can be used to lift and lower loads of materials and to move them horizontally by providing mechanical advantage.

Crane is essentially used in the construction industry for lifting heavy matters and shifting them to other places within the scope of constructers site locations. Crane replaces human efforts to create mechanical advantage and move loads beyond the capacity of human beings work at site. Cranes are employed for the loading and unloading of goods at site, movement of materials, erection of steel frames at over head elevation works and assembly works by steel structures etc.

The different type of cranes, used in construction Industry.

(1) Tower Crane (2) Gantry crane

Tower Crane: Is a modern form of balance crane that consists of the same basic parts like most, Jib, Hoist winch, operating cabin, swing unit, and counter Jib etc.

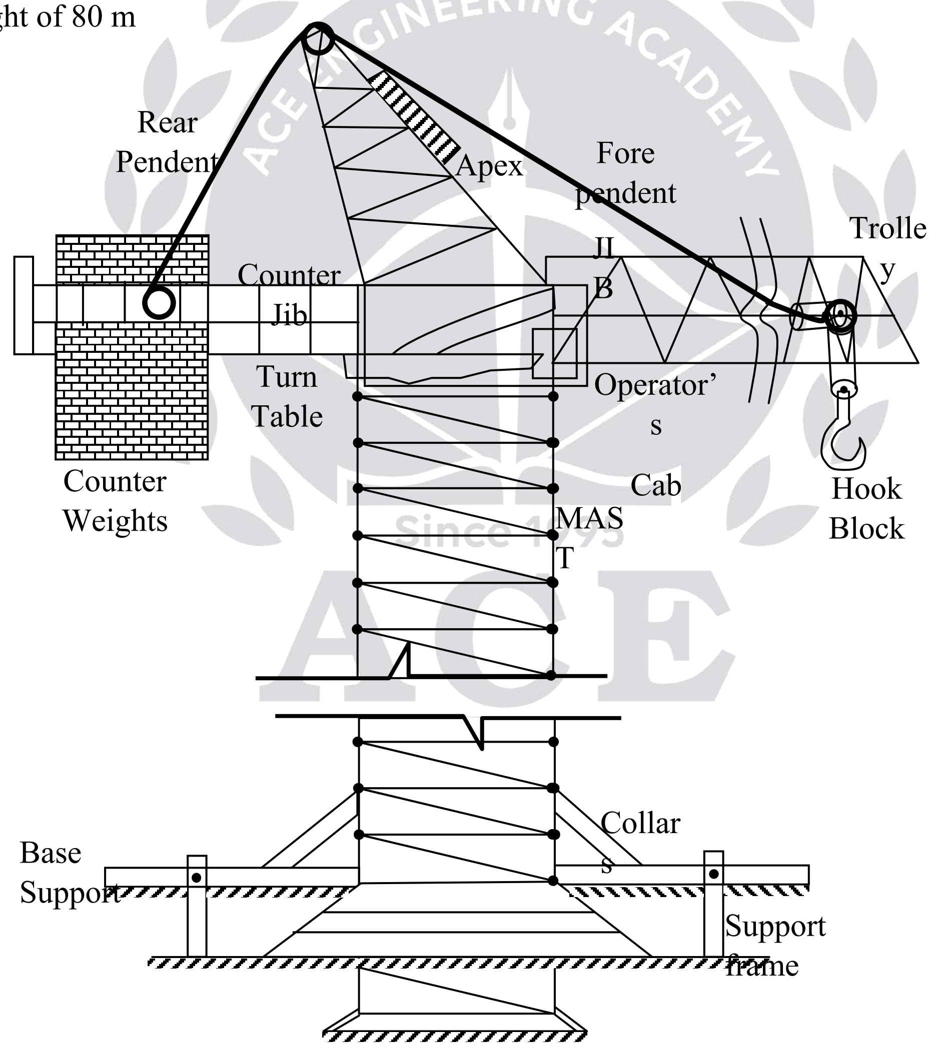
A tower crane is usually erected with a telescopic jib of greater reach, risen to constructing tall sky scapers, can be operated by remote control (to Dominates crane operator)

Tower crane is fixed to ground pm a concrete slab (or) strong frames, the provide combination of height and lifting capacity at tall building construction site, where floor area limited to use of construction works. The base is attached to most which gives the crane its height; Further the most is attached to the swing unit (Motor & Gear box) that allows the crane to rotate. On the top a swing unit 3 main parts: Long horizontal jib (Work operators's Cabin. Long horizontal jib carries the loads. The counter jib carries a cofounders weight. (usually concrete blocks), while jib is suspended the load to and from the center of the crane. Operator cab provided turntable. The lifting hook is operated using electric motors to manage wire rope cables through sheaves. Tower cranes can achieve a height under hook of over 100 mm.

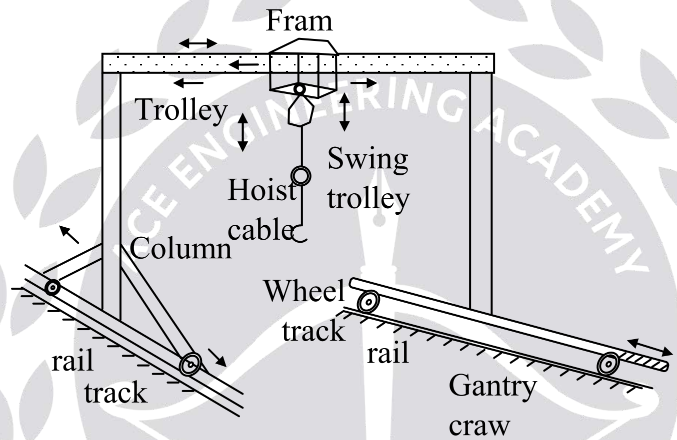
Components of tower crane:

(i) Mast	(ii) Swing	(iii) Operating cabin
(iv) JIB	(v) Counter JIB	(vi) Hoist winch
(vii) Hook	(viii) Counter weights	

Operation: Assembled tower crane at site powered with motor / Diesel engine. Maxi supposed Weight of 80 m

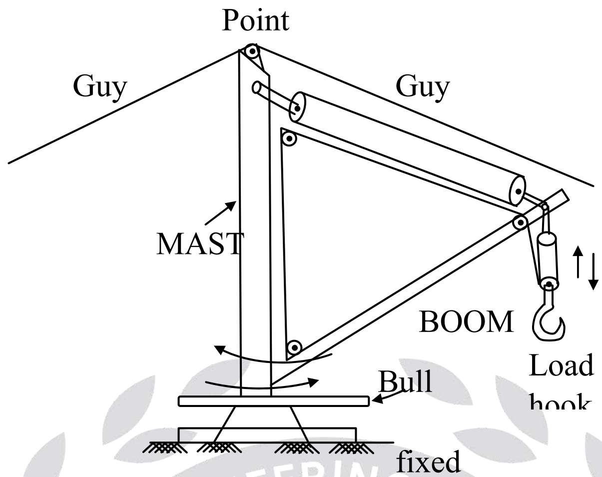


(II) Gantry crane: It is a overhead bridge – like structure supporting equipment such a crane, signals and provisions of handle load in fixed pre-determined paths. Over girder configuration supported by free standing legs that move on wheels (or) along the track (or) rail system. Gantry crane is used for an outdoor applications, eliminating permanent beams and support columns. Gantry consists 2 legs ride on rail embedded with surface of floor, run straight line in work Area with motorized equipment like motors, Cabs, hooks etc., Gantry crane is designed to handle volume of matter lifts and provide lifting and moving heavy loads through a construction yard, fabrication shop, ware houses etc.



(III) Derrick Crane: It is a lifting machine composed with guyed mast, self support tower and a boom. Mast is connected by 3 or 4 wire ropes to top of the mast, which allows laterally movement. It has turn table provision. Derrick allows loads raise and lower. It is a fixed crane usually. It assembled and dis assembled the location where it is used. It is used in location where it is used. It is used in locations where items need to be moved from common initial point to single nearby location, like on shipping flat forms and construction sites.

Hook can group down frequently



07.

(a) Design a square column of height 3 m subjected to an axial load of 1500 kN under dead and live load condition. Use limit state method of design. Assume effective length factor = 1.2. Size of the column is fixed at 400 mm × 400 mm. Show the reinforcement detail (cross-section). Use M 25 and Fe 500. (20)

Sol: Given data

Height of the column, $l = 3$ m

Axial load (DL + LL), P = 1500 kN

Effective length factor = 1.2

Column size = 400 mm × 400 mm

Grade of materials used M 25 and Fe – 500

Design the reinforcement in column

Effective length of a column

$L = 1.2 \times 3 = 3.6$ m

Slenderness ratio

$$\lambda = \frac{\text{Effective length}}{\text{Least lateral dimension}}$$

$$= \frac{3600}{400} = 9 < 12$$

$\therefore$ It is designed as short column

Minimum eccentricity

$$e_{min} = \frac{\text{unsupported length of a column}}{500} + \frac{\text{Least lateral dimension}}{30}$$

(or)

subjected to minimum of 20 mm

$$= \frac{3000}{500} + \frac{400}{30}$$

= 19.33 mm < 20 mm

Hence take e_{min} = 20 mm

$\Rightarrow$ 0.05 times least lateral dimension

$0.05 \times 400 = 20$ mm

$\Rightarrow$ $e_{min} = 0.05\ b$

$\therefore$ It is an axially loaded column and formula for columns given in IS : 456 – 2000 is valid

Factored load carrying capacity of column

$P_u = 1.5\ P$

$= 1.5 \times 1500$

$= 2250$ kN

Area of longitudinal reinforcement required in a column

$P_u = 0.4\ f_{ck}\ A_c + 0.67\ f_y\ A_{sc}$

$2250 \times 10^3 = 0.4 \times 25 \times (400 \times 400 - A_{sc}) + 0.67 \times 500 \times A_{sc}$

$A_{sc} = 2000\ mm^2$

$\Rightarrow$ Minimum area of longitudinal steel as per IS : 456 – 2000

0.8 % of gross C/S area

$$= \frac{0.8}{100} \times 400 \times 400$$

$= 1280\ mm^2 < 2000\ mm^2$

$\therefore$ O.K

$\Rightarrow$ Maximum area of longitudinal reinforcement in practice

4% of gross C/S area

$$= \frac{4}{100} \times 400 \times 400$$

$= 6400\ mm^2 > 2000\ mm^2 \quad \therefore$ O.K

(As per IS: 456 – 2000, max steel will in columns will be 6% of Ag)

$\Rightarrow$ No. of bars provided

$4 - 20$ mm dia $\Rightarrow 4 \times \frac{\pi}{4} \times 20^2 = 1257\ mm^2$

$4 - 16$ mm dia $\Rightarrow 4 \times \frac{\pi}{4} \times 16^2 = 804\ mm^2$

Total provided area is

$A = 1257 + 804 = 2061\ mm^2 > 2000\ mm^2$

$\therefore$ O.K.

Transverse steel

Since it is non circular column, so design lateral ties

Diameter of tie bar

$$\left.\begin{array}{l}\text{(i) } \frac{1}{4}\phi_{LL} = \frac{1}{4} \times 20 = 5\ mm \\ \text{(ii) } 6\ mm\end{array}\right\} \text{maximum}$$

ϕ_{LL} : Largest longitudinal bar diameter

Pitch:

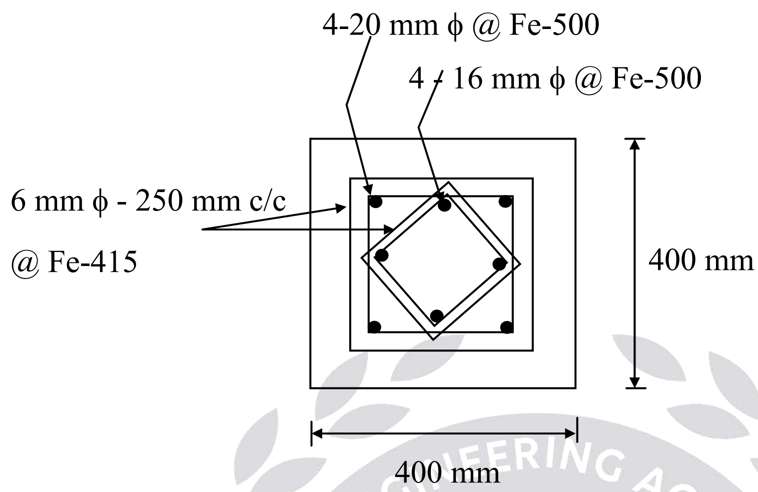
$$\left.\begin{array}{l}\text{(i) LLD } = 400\ mm \\ \text{(ii) } 16\ \phi_{SL} = 16 \times 16 = 256\ mm \\ \text{(iii) } 300\ mm\end{array}\right\} \text{smaller}$$

LLD : Least lateral dimension of column

ϕ_{SL} : Smallest longitudinal bar diameter

Provide 6 mm dia @ 250 mm c/c.

Sketch:



(b) In an industrial shed, it is proposed to provide a hot rolled section ISMB 500 to carry a two-wheeled system crab on it. The crab can move over the flange of the beam from one end to another end and each wheel of the crab is capable to carry a maximum vertical load of 60 kN (including self weight of wheel).

The centre to concrete distance between two supporting ends of the beam is 6 m and the end of the beams are restrained against torsion.

The space between two wheels = 2.4 m. Take impact factor for vertical load as 25%.

Verify the capability of the beam to carry the bending moment developed due to vertical load only. Assume the section is plastic. Grade of steel E 250. (20)

Given data:

Properties of ISMB 500 :

$b_f = 180$ mm, $t_f = 17.2$ mm, $t_w = 10.2$ mm

$r_z = 202.1$ mm, $r_y = 35.2$ mm

$z_{ez} = 1808.7\ cm^3$, $Z_{pz} = 2074.67\ cm^3$

Critical stress, $f_{cr,\ b}$ (MPa)

$\frac{KL}{r}$	h/t_f	
	25	**30**
170	**136.7**	**121.3**
180	**127.1**	**112.2**

Design bending compressive stress to lateral buckling f_{bd}, for f_y = 250 MPa

$f_{cr,\,b}$	f_{bd} (MPa)
150	**106.8**
100	**77.3**

Sol: Maximum vertical static wheel load (including self weight)

W = 60 kN

Impact load = Impact factor × Static wheel load

= 0.25 × 60

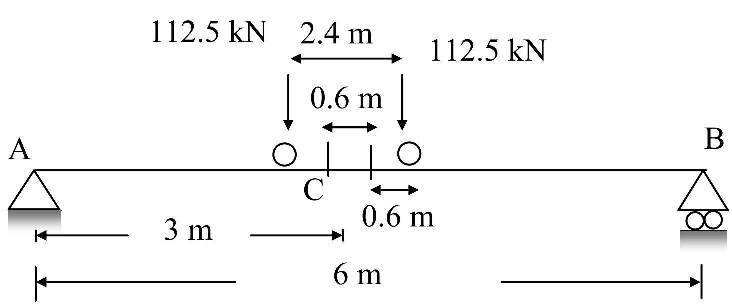
= 15 kN

Total load = static load + Impact load

⇒ 75 kN

Total factored load = 150 × 75 = 112.5 kN

Design Bending moment & Shear force



The wheel load bending moment is maximum when the two are in such a position that the centre of gravity of the wheel loads and one of the wheel load are equidistant from the centre of the gantry girder as shown in figure.

Taking moment about A = 0

$112.5 \times 1.2 + 112.5 \times 3.6 = R_B \times 6$

$R_B = 90$ kN

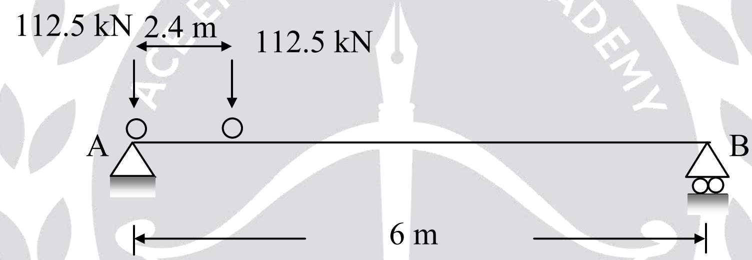
$\therefore \quad R_A = 112.5 \times 2 - 90 = 135$ kN

Design bending moment

$M_{umax} = R_B \times 2.4$ m

$= 216$ kN-m

Shear force: The shear due to wheel load is maximum when one of the wheel is at the support.



Taking moments about B

$R_A \times 6 = 112.5 \times 6 + 112.5\,(6 - 2.4)$

$R_A = 180$ kN

For ISMB 500

Design bending strength for laterally unsupported condition

$M_d = \beta_b.\ Z_p.\ F_{bd}$

Where, $\beta_b = 1$ (For plastic section)

Z_P = Plastic section modulus

F_{bd} = Design bending compression strength

For prismatic member made of rolled I-section

$$f_{cr,b}=\frac{1.1\pi^2E}{\left(\dfrac{L_{LT}}{r_g}\right)^2}\left[1+\frac{1}{20}\left(\frac{L_{LT}/r_y}{\dfrac{h}{t_p}}\right)^2\right]^{0.5}$$

Where L_{LT} = L When warping not restrained in both flange

$$f_{cr,b}=\frac{1.1\times\pi^2\times2\times10^5}{\left(\dfrac{6000}{35.2}\right)^2}\left[1+\frac{1}{20}\left(\frac{\dfrac{6000}{35.2}}{\dfrac{500}{17.2}}\right)^2\right]^{0.5}$$

$= 123.24\ \text{N/mm}^2$

From given table

$f_{cr,b}$	f_{bd}
150	106.8
123.23	x
100	77.3

Using interpolation

$$\frac{150-123.23}{123.23-100}=\frac{106.8-x}{x-77.3}$$

∴ x = 91.06 MPa

Design bending strength

$M_{dz} = \beta_b\, Z_{PZ}.\, F_{bd}$

$= 1 \times 2074.67 \times 10^3 \times 91.06 \times 10^{-6}$

= 188.92 kN-m

As design bending moment acting (216 kN-m) is greater than design bending strength (188.92 kN-m)

The given section is not capable to carry design bending moment.

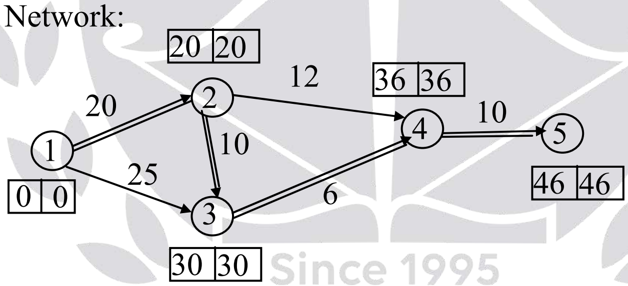
(c) The following table gives the activities in a construction project and other relevant information.

Activity	Duration (Days)
1 – 2	20
1 – 3	25
2 – 3	10
2 – 4	12
3 – 4	6
4 – 5	10

(i) Draw the network for the project.
(ii) Find the critical path
(iii) Find free, total and independent floats for each activity. **(20)**

Sol:

(i) Project Network:



(ii)

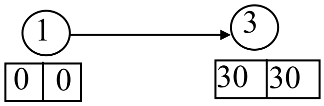
Path	Duration
1 – 2 – 4 – 5	42
1 – 2 – 3 – 4 – 5	46 (Critical path)
1 – 3 – 4 – 5	41

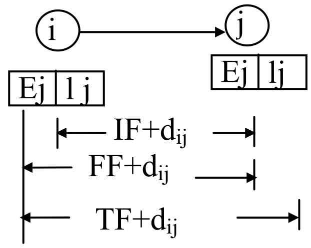
(ii) Critical activities:

1 – 2, 2 – 3, 3 – 4, and 4 – 5

All the three floats are zero values for critical activities.

Activity 1 – 3:





FF = TF – Slack_j

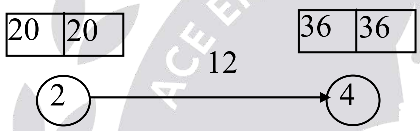
IF = FF - Slack_i

TF = $L_j - E_j - d_{ij}$

= (30 – 0) – 25 = 5

FF = 5 – (30 – 30) = 5; IF = 5 – (0 – 0) = 5

Activity 2 – 4:



TF = (35 – 20) – 12 = 4

FF = 4 – (36 – 36) = 4

IF = 4 – (20 – 20) = 4

Activity	Total Float	Free Float	Independent Float
1 – 2	0	0	0
1 – 3	5	5	5
2 – 3	0	0	0
2 – 4	4	4	4
3 – 4	0	0	0
4 – 5	0	0	0

08.

(a) A combined footing is to be provided for two columns (size 300 × 300) spaced at 3 m c/c. Axial load on each of the columns is 350 kN. The width of the footing is fixed at 1.4 m. A foundation beam of 400 mm × 800 mm is provided along the length. Design the foundation slab using M 25 and Fe 500. Assume the thickness of the slab varies from 250 mm to 150 mm. Also show the reinforcement detail (in cross-section) of the footing slab. Use limit state method of design. Baring capacity of the soil is 100 kN/m^2. (20)

Given

$\frac{M_u}{bd^2}$	0.3	0.35	0.4	0.45	0.5
P_t	0.070	0.082	0.094	0.106	0.118

P_t	<0.15	0.25	0.5	0.75	1.0
τ_c, MPa	0.29	0.36	0.49	0.57	0.64

Sol:

Total load on the two column (w) = 350 + 350 = 700 kN

Self weight of the footing (w′) = 70 kN

(10% of column load)

Total load transmitted to the soil = 700 + 70 = 770 kN , q_o = 100 kN/m^2

SBC of soil

Area of the footing required

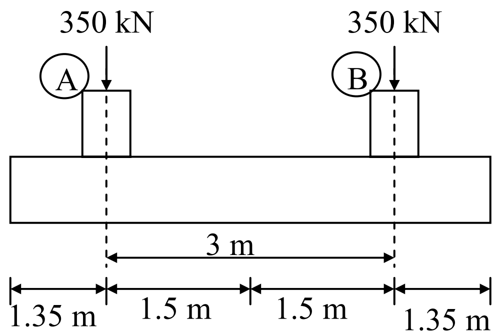
$$A = \frac{W + W'}{q_o} = \frac{770}{100} = 7.7 \text{ m}^2 \approx 8 \text{ m}^2$$

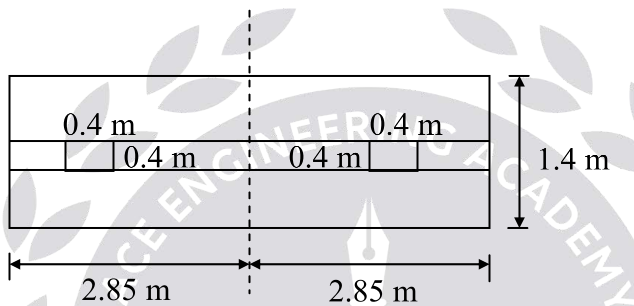
$A = B \times L = 1.4 \times L = 8 \text{ m}^2$

$\Rightarrow \quad L = 5.7$ m

Provided dimensions of footing = B × L = 1.4 × 5.7

Distance of the resultant column load form the axis of column A = $\frac{350 \times 3}{700} = 1.5$ m





Net upward soil pressure $P_o = \dfrac{W}{A_p} = \dfrac{700}{1.4 \times 5.7} \approx 88 \text{ kN/m}^2 < 100 \text{ kN}^2$

$\therefore$ O.K.

Design of the footing slab

Transverse projection of the footing slab = $\dfrac{1.4 - 0.4}{2} = 0.5$ m

Considering a 1 m wide strip of the footing slab

BM per m width = $\dfrac{88 \times 0.5^2}{2} = 11$ kN − m

Factored BM, $M_u = 1.5 \times 11 = 16.5$ kN-m

Effective depth of footing required

$$d = \sqrt{\frac{M_u}{R_u b}} - \sqrt{\frac{16.5 \times 10^6}{0.138 \times 20 \times 1000}} \approx 70 \text{ mm}$$

Overall depth required D = $70 + 50 + \dfrac{10}{2}$

$= 125$ mm < 250 mm

$\therefore$ Under reinforced section.

Area of tension steel required

$$\frac{M_u}{bd^2} = \frac{16.5\times10^6}{1000\times195^2} = 0.43 \approx 0.45 \rightarrow P_t = 0.106$$

$$\left(d = 250 - 50 - \frac{10}{2} = 195 \text{ mm}\right)$$

$P_t = 0.106$

$$\frac{100\ A_{st}}{bd} = 0.106$$

$$A_{st} = \frac{0.106\times1000\times195}{100} = 207 \text{ mm}^2$$

Minimum area of steel

$$\frac{0.12}{100}\times1000\times250 = 300 \text{ mm}^2 > 207 \text{ mm}^2$$

$\therefore$ Provide at least min steel spacing required per width

$$S = \frac{1000\times a_{st}}{A_{st}} = \frac{1000\times\frac{\pi}{4}\times10^2}{300} = 262 \text{ mm}$$

Provide 10 mm ϕ @ 250 mm c/c

Distribution steel

$$A_{st} = \frac{0.12}{100}\times1000\times250 = 300 \text{ mm}^2$$

Spacing

$$S = \frac{1000\times\frac{\pi}{4}\times8^2}{300} = 168 \text{ mm}$$

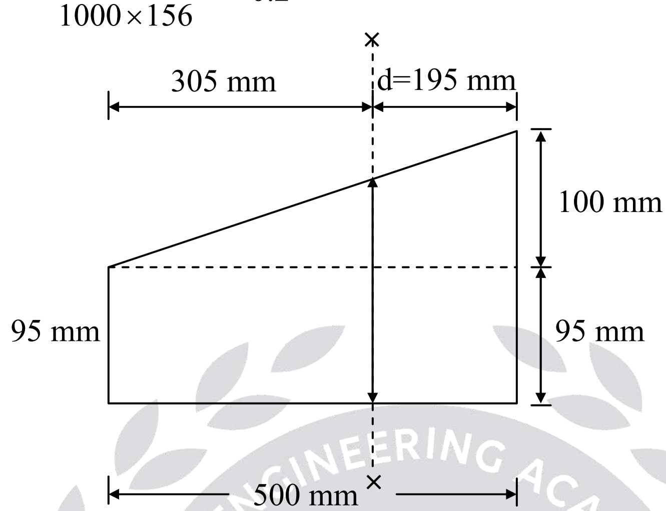
Provide 8 mm ϕ @ 160 mm c/c

Check for shear

Actual area of tension steel provided

$$A_{st} = \frac{\frac{\pi}{4}\times10^2\times1000}{250} = 314.159 \text{ mm}^2$$

$$P_t = \frac{100A_{st}}{bd} = \frac{100 \times 314.159}{1000 \times 156} = 0.2$$



(150 – 50 – 10 / 2 = 95 mm)

(250 – 50 – 10/2 = 195 mm)

At critical section

$$d = 95 + \frac{100}{500} \times 305$$

$= 156$ mm

500 mm $\rightarrow$ 100 mm

315 $\rightarrow$ y

From table (for $P_t = 0.2$)

$$\tau_c = 0.29 + \frac{0.36 - 0.29}{0.25 - 0.15}(0.2 - 0.15) = 0.325 \text{ N/mm}^2$$

Shear force at critical section

$V = 88\ (0.5 – 0.195) = 26.84$ kN

$V_u = 1.5 \times 26.84 = 40.26$ kN

Nominal shear stress

$$\tau_v = \frac{V_u}{bd} = \frac{40.26 \times 10^3}{1000 \times 156} = 0.26 \text{ N/mm}^2$$

$\tau_v < \tau_c$ $\therefore$ Safe

(b) A rafter member of a roof truss carries 40 kN compressive load (DL + LL) and 67 kN tensile load (DL + WL). The effective nodal legnth of the member is 2.1 m. A circular tube section of nominal bore diameter of 50 mm is used. Check the adequacy of the section. Grade of steel = E 250, Young's Modulus E = 200 GPa. (20)

Given:

Sectional properties of the section A = 523 mm^2 , r = 20.3 mm,

Outside diameter = 60.3 mm

Stress reduction factor $\chi = \dfrac{1}{\phi + \sqrt{\phi^2 - \lambda^2}}$

$\phi = 0.5\{1 + \alpha(\lambda - 2) + \lambda^2\{$

$$\lambda = \sqrt{\frac{f_y}{f_{cc}}} \qquad f_{cc} = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$$

Buckling class	a	b	c	d
α	0.21	0.34	0.49	0.76

Sol:

For grade (E250) steel, $f_y = 250$ N/mm^2 & E =200 GPa = 200 ×10^3 N/mm^2

Partial safety factors, $\gamma_{mo} = 1.10$

Compressive load in rafter due to DL+LL is $P_s = 40$ kN

Design compressive load in rafter $P = \gamma_f \times P_s = 1.5\times40 = 60$ kN

Tensile load in rafter due to DL+ WL is $T_s = 67$ kN

Design tensile load in rafter $T = \gamma_f \times T_s = 1.5\times67 = 100.5$ kN

For tube section , A=523mm^2 , r =20.3mm

Check for design compressive strength

Effective nodal length of strut KL = 2.1m

Effective slenderness ratio of rafter KL/r = 2100/20.3 = 103.44

Elastic critical stress in compression $f_{cc} = \dfrac{\pi^2 E}{\left(\frac{KL}{r_{min}}\right)^2} = \dfrac{\pi^2 \times 2\times10^5}{(103.44)^2} = 184.48$ N/mm^2

Non dimensional effective slenderness ratio $\lambda = \sqrt{\frac{f_y}{f_{cc}}} = \sqrt{\frac{250}{184.48}} = 1.164$

Tube sections as per IS800:2007 classed as a buckling class 'a' & $\alpha = 0.21$

$\phi = 0.5\,[1+\alpha(\lambda - 0.2)+\lambda^2] = 0.5\times[1+0.21\times(1.164-0.2)+(1.164)^2] = 1.278$

Design stress in axial compression $f_{cd} = \frac{f_y/\gamma_{mo}}{\phi + \left(\phi^2 - \lambda^2\right)^{0.5}}$ **hj**

$$= \frac{250/1.10}{1.278 + \left(1.287^2 - 1.164^2\right)^{0.5}} = 124.39\ \text{N/mm}^2$$

Design compressive strength of section $P_d = f_{cd}\times A_e = 124.39\times 523$

$= 65.05\times 10^3$ N $= 65.05$ kN $\geq P = 60$ kN

Hence section is adequate against design compressive load

Check for design tensile strength

Design tensile strength of rafter due to gross section yielding

$$T_{dg} = \frac{A_g \times f_y}{\gamma_{m0}} = \frac{523\times 250}{1.10} = 118.86\times 10^3\,N = 118.86\,kN \geq T = 100.5\,kN$$

Design tensile strength of rafter due to net section rupture

$$T_{dn} = \frac{0.9\times A_n \times f_u}{\gamma_{m1}} = \frac{523\times 250}{1.10} = 118.86\times 10^3\,N = 118.86\,kN \geq T = 100.5\,kN$$

Effective net area $A_n = A_g = A = 523$ mm^2

$$T_{dn} = \frac{0.9\times A_n \times f_u}{\gamma_{m1}} = \frac{0.9\times 523\times 410}{1.25} = 154.38\times 10^3\,N = 154.38\,kN \geq T = 100.5\,kN$$

Hence section is adequate against design tensile load

(c) A simply supported prestressed concrete beam of width 100 mm, depth 200 and span 10 m, carries a UDL of intensity ‘w’. If the member is prestressed with a parabolic cable having zero eccentricity at the ends and 60 mm eccentricity at midspan, determine the value ‘w’ for the following conditions, for effective prestressing force of 125 kN.

(i) Load Balancing Case

(ii) For no tensile stress condition at mid span

(iii) For cracking condition taking the tensile strength of concrete as 1.5 N/mm^2

For all cases neglect the weight of concrete. (20)

Sol:

Given Data

Width. b = 100mm

Depth, D = 200mm

Span, L = 10m

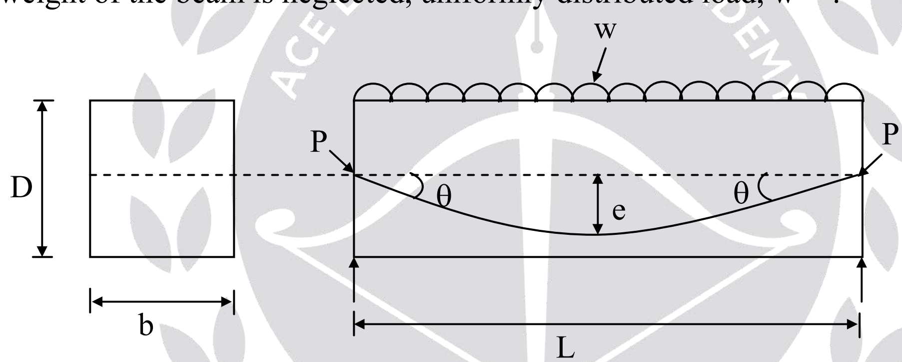
Eccentricity

e = 0 (At Supports)

e = 60mm (At Midspan)

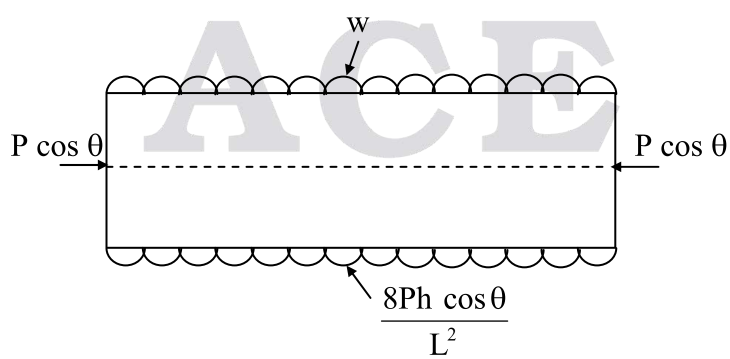
prestressing force. P = 125 KN

Self weight of the beam is neglected, uniformly distributed load, w = ?



(i) Load Balancing case

In this case gravity load (that is udl) is balanced by transverse component of cable force



As θ is very small hence we can write cosθ = 1 and sinθ = 0

⇒ Load acting on the beam is

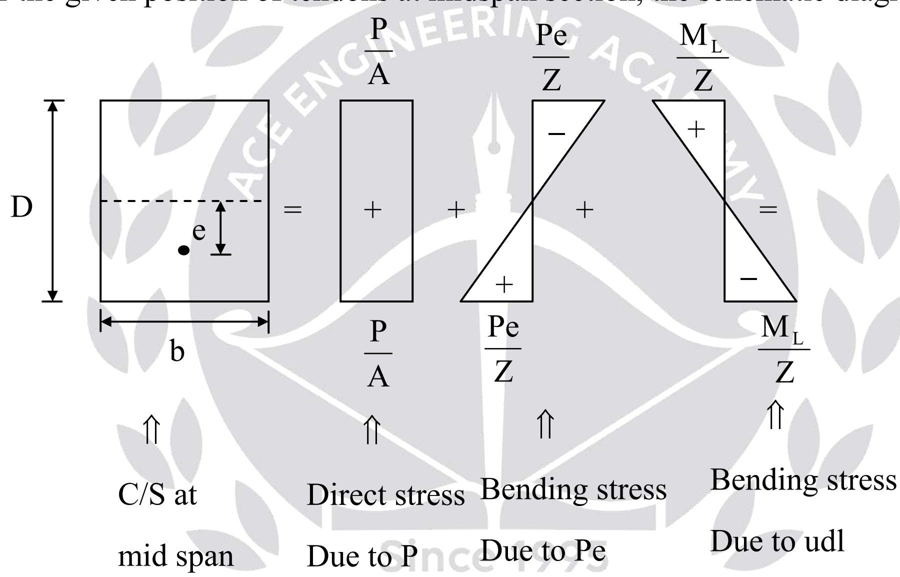
$$w = \frac{8ph}{L^2}$$

$$= \frac{8 \times 125 \times \left(\frac{60}{1000}\right)}{10^2}$$

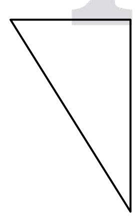
$$= 0.6\ KN/m$$

(ii) For no tensile Stress condition at midspan

Considering the effects of prestress and external loading the tensile stress with develop at bottom fibre, for the given position of tendons at midspan section, the schematic diagram shown below



$$f_t = \frac{P}{A} - \frac{Pe}{Z} + \frac{M_L}{Z}$$



$$f_b = \frac{P}{A} + \frac{Pe}{Z} - \frac{M}{Z} = 0$$

$\Uparrow$

Result stress at midspan

Stress at bottom fibre, at midspan is zero

$$\Rightarrow f_b = 0 = \frac{P}{A} + \frac{Pe}{Z} - \frac{M}{Z}$$

$$\Rightarrow \frac{P}{A} + \frac{Pe}{Z} = \frac{M}{Z}$$

$$\Rightarrow \frac{125\times10^3}{100\times200} + \frac{125\times10^3\times60}{\frac{1}{6}\times100\times2002} = \frac{M}{\frac{1}{6}\times100\times200^2}$$

$$\Rightarrow \quad M = 11.67\times10^6 \text{ N.mm} = 11.67 \text{ kN-m}$$

$$\Rightarrow \quad \frac{wL^2}{8} = 11.67$$

$$w = \frac{11.67\times8}{10^2} = 0.93 \text{ kN/m}$$

(Maximum BM at midspan of simply supported beam subjected to udl is $M = \frac{wL^2}{8}$)

(iii) For cracking condition

In tension zone, cracks will develop. Here in this case tension developing at bottom

Stress at bottom fibre

$$f_b = \frac{P}{A} + \frac{Pe}{Z} - \frac{M}{Z}$$

$$-1.5 = \frac{125\times10^3}{100\times200} + \frac{125\times10^3\times60}{\frac{1}{6}\times100\times200^2} - \frac{M}{\frac{1}{6}\times100\times200^2}$$

$$\Rightarrow -1.5 = 6.25 + 11.25 - \frac{M}{\frac{1}{6}\times100\times200^2}$$

$$M = 12.67\times10^6 \text{ N-mm} = 12.67 \text{ kNm}$$

$$\frac{wL^2}{8} = 12.67$$

$$w = \frac{12.67\times8}{10^2} = 1.01 \text{ kN/m}$$