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ESE – 2019
(MAINS)

Questions with Detailed Solutions

CIVIL ENGINEERING

PAPER - I

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CIVIL ENGINEERING

ESE _MAINS_2019_PAPER – I

PAPER REVIEW

Except few questions from RCC & Steel, remaining questions in the paper can be easily attempted. Particularly in this paper selection of questions plays a vital role in securing a good score. For example Section-A is relatively easy than Section-B, so choosing 3 questions from Section-A will fetch you a big advantage.

SUBJECT WISE REVIEW

SUBJECT(S) SECTION-A	LEVEL	Marks
Strength of Materials	Easy	116
Structural Analysis	Moderate	40
Building Materials & Concrete Technology	Easy	84
SECTION-B	**LEVEL**	**Marks**
R.C.C. & P.S.C	Hard	104
Steel Structures	Moderate	44
Construction Management & Equipment	Easy	92

Subject Experts,

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SECTION-A

01. (a)

(i) Explain briefly the various tests conducted on Bricks mentioning the relevant codal provisions.

(8 M)

Sol:

I. Compressive Strength Test: (IS 3495 (Part I) : 1992)

- A dry brick is taken and its frog is filled with 1:1 cement mortar and the surfaces are made even.
- Now the brick is completely immersed in cold water for 3 days.
- Now the brick is placed in the UTM/CTM with flat faces horizontal and mortar filled face facing upwards between two 3-ply plywood sheets each of 3 mm thick.
- Now compressive load is applied on the brick at a uniform rate of 14 N/mm^2 per minute till failure occurs.
- Compressive strength of the brick = Maximum load at failure / Average area of the bed faces.

II. Water Absorption Test: (IS 3495 (Part II) : 1992)

- A brick is taken and oven dried until it attains a substantially constant mass (M_1).
- Now this brick is taken and completely immersed in clean water at a temperature of 27 ± 2^oC for 24 hours.
- Now the mass of the brick is measured as M_2.
- Water Absorption = $(M_2 - M_1)/M_1 \times 100$.

III. Efflorescence Test: (IS 3495 (Part III) : 1992)

- A dry brick is taken and is placed in a dish of water, the depth of immersion in water being 25mm.
- The whore arrangement is placed in a warm and well ventilated room until all the water in the dish is absorbed by the brick and the surplus water evaporates.
- The dish is now covered with suitable glass cylinder so that excessive evaporation from the dish may not occur.

- When the water has been absorbed and the brick appear to be dry, place a similar quantity of water in the dish and allow it to evaporate as before.
- Now the brick is examined for efflorescence after second evaporation and the results are reported as Nil, Slight, Moderate, Heavy or Serious.

IV. Warping Test: (IS 3495 (Part IV) : 1992)

- A brick is taken and placed is placed on a flat surface of glass or steel.
- For Concave Warping: The greatest distance of the brick surface from the edge of straightness is measured by a steel rule or wedge.
- For Convex Warping: The distance from the flat surface to the four corners of the brick is measured and the maximum of the four measurements is taken.
- The higher of the above two measurements is reported as warpage of the brick.

(ii) Explain the products of hydration of C_3S and C_2S (Bogues compounds) giving the relevant equations involving the reactions. (4 M)

Sol:

- The hydration reaction of C_3S and C_2S are as follows

  $C_3S + H_2O \rightarrow C\text{-}S\text{-}H + 3Ca(OH)_2$

  $C_2S + H_2O \rightarrow C\text{-}S\text{-}H + Ca(OH)_2$
- The hydration products of C_3S and C_2S are Calcium Silicate Hydrate (C-S-H) and Calcium Hydroxide ($Ca(OH)_2$).
- Calcium Silicate Hydrate (C-S-H) is the ultimate desired hard compound which is responsible for the strength of cement.
- Depending on the other compounds present in the cement, the Calcium Hydroxide (($Ca(OH)_2$) produced in these reactions may take part in sulphate attack or may take part in pozzolanic action.

01.(b)

(i) Explain the following defects in timber with neat sketches:

(A) Shakes

(B) Knots (4 M)

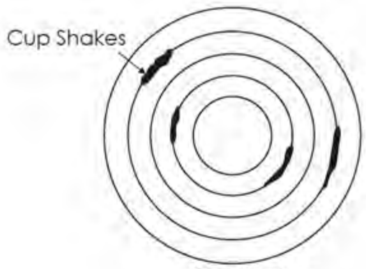
Sol: Shakes: Theses are cracks which partly or completely separate the fibers of wood

Types of Shakes:

Types of Shakes:

a) Cup shakes:

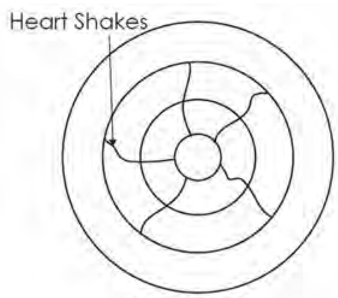
- Rupture of tissues in circular direction.
- Separate one annual ring from the other.
- Occur due to non – uniform growth or due to excessive bending during a cyclonic weather.



Cup Shakes

b) Heart Shakes:

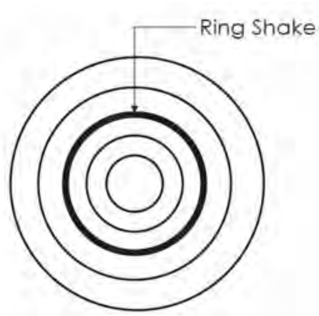
- Cracks extend from pith to sap wood in the direction of Medullary rays
- Due to shrinkage of interior part of tree.



Heart Shakes

c) Ring Shakes:

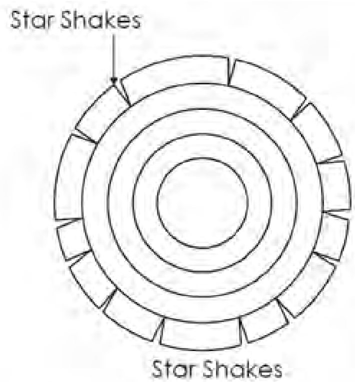
- Cup shake covering the entire ring.



Ring Shakes

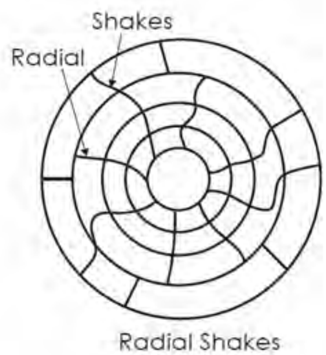
d) Star Shakes:

- Cracks extend from bark towards sap wood.
- Formed due to extreme heat or severe frost during growth of the tree.



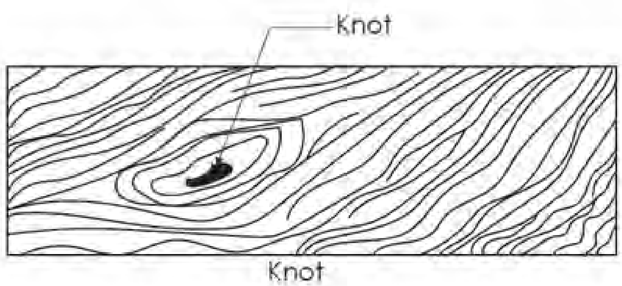
e) Radial Shakes:

- Similar to star shakes but fine irregular and numerous.
- Occurs when tree is exposed to sun for reasoning after being fell down.



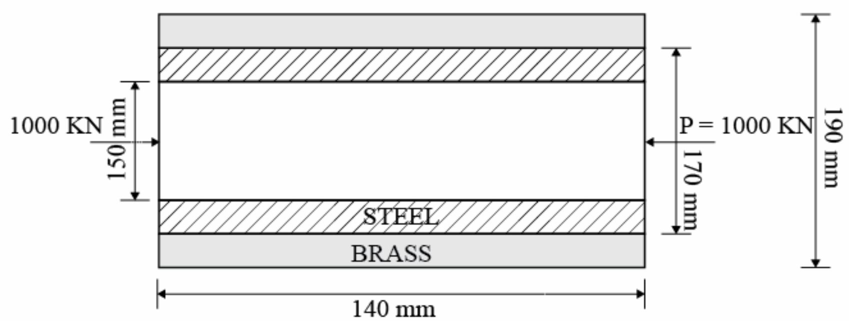
g) Knots:

- These are the bases of branches or limbs which are broken or cutoff from tree.
- Continuity of wood fibers is broken by knots, they form a source of weakness.



(ii) **A compound tube consists of a steel tube 150 mm internal diameter and 170 mm external diameter and a brass tube of 170 mm internal diameter and 190 mm external diameter. The two tubes are of the same length. The compound tube carries an axial load of 1000 kN. Find the stresses and the load carried by each tube and the amount it shortens. Length of each tube is 140 mm. Take $\overline{E}$ for steel as 2×10^5 N/mm^2 and for brass as 1×10^5 N/mm^2. (8 M)**

Sol:



Given: $E_s = 2 \times 10^5$ N/mm^2

$E_b = 1 \times 10^5$ N/mm^2

Since it is a compound tube the compressive loads will be shared by them.

Let loads on steel and brass tubes be P_s and P_b respectively.

$\therefore \quad P_s + P_b = P = 1000$ kN________ (1)

The deformation in the tubes must be equal

$$\therefore \quad \frac{P_s L}{A_S E_s} = \frac{P_b L}{A_b E_b}$$

$$\frac{P_s}{P_b} = \frac{A_s . E_s}{A_b E_b}$$

$$= \frac{\frac{\pi}{4}\left(170^2 - 150^2\right)}{\frac{\pi}{4}\left(190^2 - 170^2\right)} \times 2$$

$P_s = 1.78 P_b$ ______ (2)

Substituting in eqn (1)

$1.78 P_b + P_b = 1000$ kN

$P_b = 359.7$ kN

$P_s = 640.29$ kN

Stresses in the tubes:

Stress in steel tube:

$$\sigma_s = \frac{P_s}{A_s} = \frac{640.29\times10^3\,N}{\frac{\pi}{4}\left(170^2 - 150^2\right)mm^2}$$

$$= \frac{640.29\times10^3}{5024}$$

$$= 127.44 \text{ MPa}$$

Stress in brass tube:

$$\sigma_b = \frac{P_b}{A_b} = \frac{359.7\times10^3\,N}{\frac{\pi}{4}\left(190^2 - 170^2\right)}$$

$$= \frac{359.7\times10^3}{5652}$$

$$= 63.64 \text{ MPa}$$

Deformation in the tubes:

$$\delta L = \frac{P_s L}{A_s E_s} = \frac{P_b L}{A_b E_b}$$

$$= \frac{640.29\times10^3\times140}{5024\times2\times10^5} = 0.0892\,mm$$

Result:

(i) Stresses:

$\sigma_b = 63.64$ N/mm^2

$\sigma_s = 127.44$ N/mm^2

(ii) Loads:

$P_b = 359.7$ kN

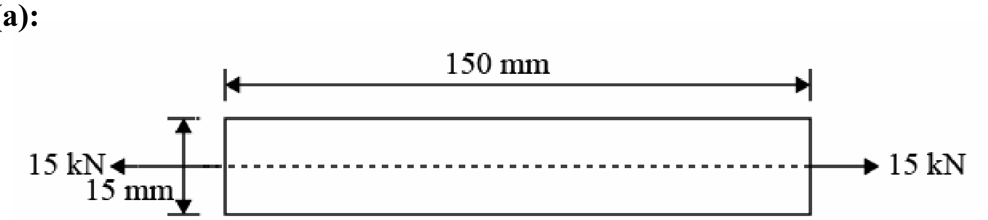
$P_s = 640.29$ kN

(iii) Decrease in length of tube = 0.0892 mm

01.(c)

(i) On a steel bar specimen of 15 mm diameter and 150 mm gauge length, when tested as a tensile test specimen, a force of 15 kN produces an extension of 0.063 mm. When the specimen of same diameter and same length is tested under torsion, a twisting moment of 6.94 Nm produces an angular twist of 0.15°. Determine the Poisson's ratio of the material of the bar (4 M)

Sol: Case (a):



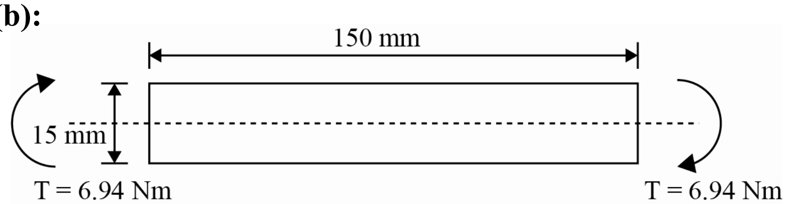
Deformation of extension in the bar due to tensile load = 0.063 mm

$$\Delta L = \frac{PL}{AE}$$

$$0.063 = \frac{15\times 10^3 \times 150}{\frac{\pi}{4}\times 15^2 \times E}$$

$$E = 2.02 \times 10^5 \text{ N/mm}^2$$

Case (b):



Angle of twist due to torque = 0.15°

$$\theta = \frac{TL}{GJ}$$

$$0.15\times\frac{\pi}{180} = \frac{6.94\times 10^3\,\text{N}-\text{mm}\times 150}{G\times\frac{\pi}{32}\times(15)^4}$$

$$G = 0.8 \times 10^5 \text{ N/mm}^2$$

Relation between E and G

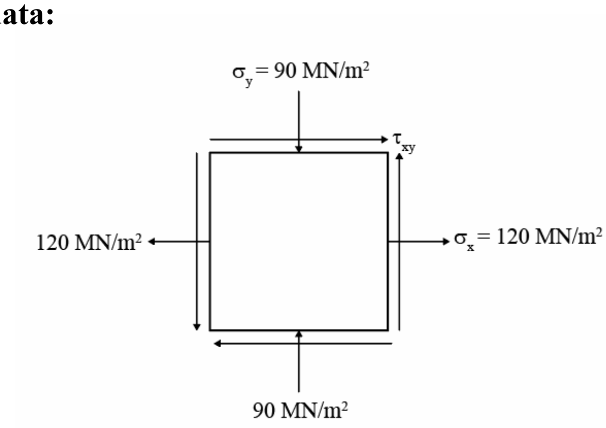
$$E = 2G(1+\mu)$$

$$2.02 \times 10^5 = 2 \times 0.8 \times 10^5 (1+\mu)$$

$$\Rightarrow \mu = 0.2625$$

(ii) Direct stresses of 120 MN/m^2 in tension and 90 MN/m^2 in compression are applied to an elastic material at a certain point on planes at right angles to each other. If the maximum principal stress is not to exceed 150 MN/m^2 in tension, to what shearing stress can the material be subjected? What is then the maximum resulting shearing stress in the material? Also find the magnitude of the other principal stress and its inclination to 120 MN/m^2 stress (8 M)

Sol: Given data:



$$\sigma_x = 120\frac{MN}{m^2} \qquad \sigma_y = -90\frac{MN}{m^2} \text{ (compressive)}$$

Max. principal stress = σ_1 = 150 MN/m^2

We know that algebraic sum of normal stresses on any two mutually perpendicular planes is constant

$\therefore \quad \sigma_x + \sigma_y = \sigma_1 + \sigma_2$

$120 - 90 = 150 + \sigma_2$

$\therefore \quad \sigma_2 = \ominus\ 120$ MN/m^2 (compressive)

$\therefore \quad$ Min. principal stress = σ_2 = 120 MN/m^2 (compressive)

And $\quad \sigma_1 = \dfrac{\sigma_x + \sigma_y}{2} \oplus \sqrt{\left(\dfrac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2}$

$$150 = \frac{120-90}{2} \oplus \sqrt{\left(\frac{120+90}{2}\right)^2 + \tau_{xy}^2}$$

$$150 = 15 \oplus \sqrt{11025 + \tau_{xy}^2}$$

$$\Rightarrow \quad 11025 + \tau_{xy}^2 = (150-15)^2 = 18225$$

$$\tau_{xy}^2 = 18225 - 11025 = 7200$$

$\Rightarrow \quad \tau_{xy} = 84.85\ \text{MN/m}^2$

And Max. shear stress $= \dfrac{\sigma_1 - \sigma_2}{2} = \dfrac{150+120}{2} = 135\ \text{MN/m}^2$

Position of principal plane

$$\tan 2\theta_p = \frac{2\tau}{\sigma_x - \sigma_y}$$

$$\tan 2\theta_p = \frac{2(84.85)}{120+90}$$

$$\tan 2\theta_p = 0.808$$

$$2\theta_p = 38.94$$

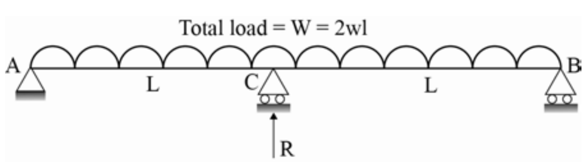
$$\theta_p = 19.47° \text{ or } 109.47°$$

Result:

1. Minor principal stress = σ_2 = 120 MN/m^2 (compressive)
2. Shear stress = τ_{xy} = 84.85 MN/m^2
3. Max. Shear stress = τ_{max} = 135 MN/m^2
4. $\theta_p = 19.47°$ or $109.47°$

01.(d) A beam of uniform cross-section and of length 2L is simply supported by rigid supports at its ends and by an elastic prop at its centre. If the prop deflects by an amount λ times the load it carries and if the beam carries a total uniformly distributed load of W find the load carried by the prop if EI is constant throughout the length of beam. (12 M)

Sol:



Downward deflection @c due to UDL $\frac{5}{384}W\frac{(2\ell)^3}{EI} = \frac{5W\ell^3}{48EI}$

Upward deflection @ c due to prop reaction $= \frac{R(2\ell)^3}{48EI} = \frac{R\ell^3}{6EI}$

$\therefore$ Net deflection @c = $(\downarrow y_c)_{UDL} - (\uparrow y_c)_R = \lambda R \rightarrow$ compatibility condition

$$\frac{5}{48}\frac{W\ell^3}{EI} - \frac{R\ell^3}{6EI} = \lambda R$$

$$\Rightarrow \quad R\left(\lambda + \frac{\ell^3}{6EI}\right) = \frac{5W\ell^3}{48EI}$$

$$R = \frac{\frac{5W\ell^3}{48EI}}{\left(\lambda + \frac{\ell^3}{6EI}\right)}$$

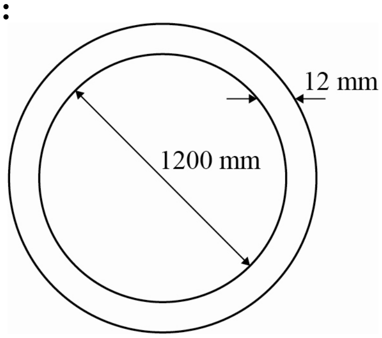
$$\Rightarrow \quad R = \frac{5W}{8\left(1 + \frac{6EI\lambda}{\ell^3}\right)}$$

Result:

$$R = \frac{5W}{8\left(1 + \frac{6EI\lambda}{\ell^3}\right)}$$

01.(e) A water main of 1200 mm internal diameter and 12 mm thick is running full. If the bending stress is not to exceed 56 MPa, find the longest span on which the pipe may be freely supported. Steel and water weigh 76.8 kN/m^3 and 10 kN/m^3 respectively. (12 M)

Sol: Given data:



$\rho_w = 10\ kN/m^3 = 10000\ N/m^3$

$\rho_{pipe} = 76800\ N/m^3$

$d = d_i = 1200\ mm = 1.20\ m$

$t = 12\ mm$

$D = d_o = d_i + 2t = 1224\ mm = 1.224\ m$

$f_{max} = 56\ MPa$

Consider 1 m run of main

$\therefore$ Area of pipe section $A_p = \frac{\pi}{4}\left(D^2 - d^2\right)$

$$= \frac{\pi}{4}\left(1.224^2 - 1.2^2\right) = 0.0457\,m^2$$

And Area of water section $A_w = \frac{\pi}{4}d^2 = \frac{\pi}{4}\left(1.2^2\right) = 1.131\,m^2$

Weight of pipe for 1 m run = $\rho_p \times A \times l$

$= 76800 \times 0.0457 \times 1 = 3509.76\ N$

Weight of water for 1m run of pipe = $\rho_w \times A \times l$

$= 10000 \times 1.131 \times 1$

$= 11310\ N$

$\therefore$ Total load on pipe for 1m run = 3509.76 + 11310

$= 14819.76\ N/m$

Let maximum span = l metre

$\therefore$ max. B.M = $M_{max} = \frac{w\ell^2}{8} = \frac{14819.76\ell^2}{8}$ N-m

$$= \frac{14819.76\ell^2 \times 10^3}{8} = 1852.47\ell^2 \times 10^3\ \text{N-mm}$$

MOI of pipe section about N.A = $I_{N.A} = \frac{\pi}{64}(D^4 - d^4)$

$\Rightarrow$ $\frac{\pi}{64}\left(1224^4 - 1200^4\right) = 8.386 \times 10^9\ mm^4$

And $y_{max} = \frac{D}{2} = \frac{1224}{2} = 612\,mm$

$\therefore$ From Bending equation, $\frac{M_{max}}{I} = \frac{f_{max}}{y_{max}}$

$$\frac{1852.47\ell^2 \times 10^3}{8.386 \times 10^9} = \frac{56}{612}$$

$\Rightarrow$ $l^2 = 414.22$ m

$\therefore$ $l = 20.36$ m

Result: Span length = l = 20.36 m

02.(a)

(i) How is the presence of surface oxide film responsible for excellent corrosion resistance of Aluminium? (4 M)

Ans: The resistance of aluminium to weathering is because of aluminium oxide. Since metallic aluminium is highly reactive in nature, it immediately reacts with atmospheric oxygen and a thin layer of aluminium oxide is formed on all the exposed aluminium surfaces. This surface oxide layer protects the metal from further oxidation and thus making aluminium highly corrosion resistant. The thickness and properties of this oxide layer can be enhanced using a process called anodising.

(ii) What are the various factors that promote the Alkali Aggregate Reaction? How can this be controlled? (8 M)

Ans: Alkali aggregate reaction referrers to the reaction which occurs over time in concrete between the highly alkaline cement paste and non-crystalline silicon dioxide, which is found in many common aggregates. This reaction can cause expansion of the altered aggregate, leading to spalling and loss of strength of the concrete. In general, aggregates are more or less chemically inert, but aggregates containing certain forms of silica will react with alkali hydroxide in concrete to form a gel that swells as it adsorbs water from the surrounding cement paste or the environment.

Factors which promote Alkali Aggregate reaction are

1. High presence of alkalis in cement.
2. Use of aggregates which have reactive silicates on their surfaces.
3. High water cement ratio in the concrete.
4. Availability of water in the surroundings.

The follows methods can be employed to control alkali aggregate reaction.

1. Limiting the presence of alkalies in the cement.
2. Using concrete mix with low water cement ratio.
3. Using air entraining admixtures in the preparation of concrete.
4. Using low cement concrete.

(iii) Describe the thermal and electrical properties of ceramics. (8 M)

Ans: Thermal Properties of Ceramics:

Ceramics have very good thermal insulating property. The heat in ceramics is conducted by photon conductivity and by the interaction of lattice vibration. Ceramics don't have enough free electrons to bring out electronic thermal conductivity. At high temperatures, conduction takes place by transfer of radiant energy.

Electrical Properties of Ceramics:

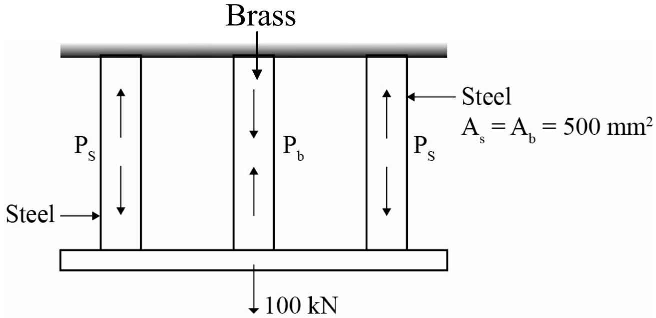
Since ceramics have no free electrons, they have low electrical conductivity. However, at high temperature the ionic diffusion is accelerated and conductivity increases. Clay displays a very high dielectric constant under static conditions. However, for alternating current, the dielectric constant in clay arises from ion and electron movement.

02.(b)

(i) Three vertical rods carry a tensile load of 100 kN. Area of cross-section of each rod is 500 mm^2. Their temperature is raised by 60°C and the load is now so adjusted that they extend equally. Determine the load shared by each. The outer two rods are of steel and the middle one is of brass.

$E_S = 2\ E_B = 210$ GPa. $\alpha_S = 11 \times 10^{-6}/^oC$; $\alpha_\beta = 18 \times 10^{-6}/^oC$. **(12 M)**

Sol:



$E_s = 2E_b = 210$ GPa

$\alpha_b > \alpha_s$

Since brass tends to expand more compared to steel due to rise in temperature

∴ The stresses induced due to temperature rise in brass are compressive in nature and the stresses induced in steel are tensile in nature.

Elongation in steel and brass rods is equal

$$\alpha_s L_s \Delta T + \frac{P_s L_s}{A_s E_s} = \alpha_b L_b \Delta T - \frac{P_b L_b}{A_b E_b}$$

$$\left(11\times10^{-6}\times60\right)+\frac{P_s}{500\times210\times10^3}=\left(18\times10^{-6}\times60\right)-\frac{P_b}{500\times105\times10^3}$$

$$660\times10^{-6}+\frac{P_s}{105}\times10^{-6}=1080\times10^{-6}-\frac{P_b}{52.5}\times10^{-6}$$

$$9.52\times10^{-3}P_s + 0.019\ P_b = 1080 - 660$$

$$9.52\times10^{-3}\ P_s + 0.019\ P_b = 420 \qquad \text{\_\_\_\_\_\_\_ (1)}$$

Considering equilibrium of forces

$$2P_s - P_b = 100\times10^3 \qquad \text{\_\_\_\_\_\_\_\_\_ (2)}$$

Solving for equation (1) and (2)

We get

$P_s = 48821.55$ N (tensile load)

$P_b = 2356.9$ N (compressive load)

(ii) A solid steel shaft has to transmit 75 kW at 200 rpm. Taking allowable shear stress as 70 N/mm², find suitable diameter for the shaft, if the maximum torque transmitted at each revolution exceeds the mean by 30% (8 M)

Sol: Given: power transmitted

$$P = 75 \text{ kW}$$

Speed, N = 200 rpm

Maximum torque = 1.3 × Mean torque

Power transmitted is given by:

$$P = \frac{2\pi N T_{mean}}{60}$$

$$= \frac{75\times10^3\times60}{2\pi\times200} = T_{mean}$$

$\Rightarrow$ $T_{mean} = 3582.8$ N-m

$\therefore$ Maximum torque = 1.3 × 3582.8

= 4657.64 N-m

Allowable shear stress is given by

$$\tau_{max} = \frac{16T_{max}}{\pi d^3}$$

$$70\text{N}/\text{mm}^2 = \frac{16\times4657.64\times10^3\,\text{N}-\text{mm}}{\pi d^3}$$

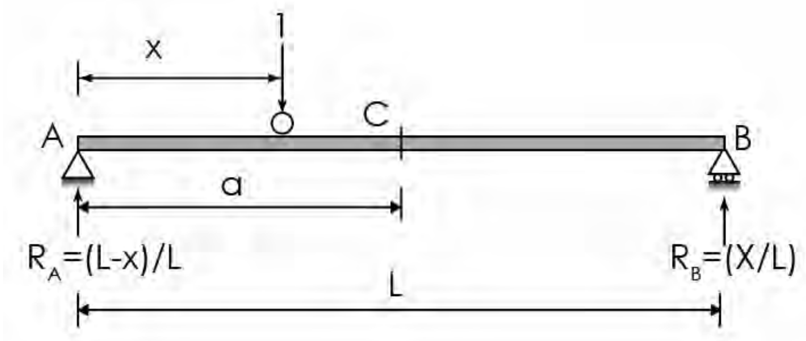
d = 69.73 mm

Result: Shaft diameter d $\simeq$ 70 mm

02.(c) A uniformly distributed load of 40 kN/m and 5 m long crosses a simply supported beam of span 15 from left to right. Draw the influence line diagram for shear force and bending moment at a section 6 m from left end. Use these diagrams to get the maximum shear force and bending moment at this section. (20 M)

Sol: **ILD for shear force at 'C':**

Let 'C' be the section at a distance 'a' from support 'A' as shown in figure below.



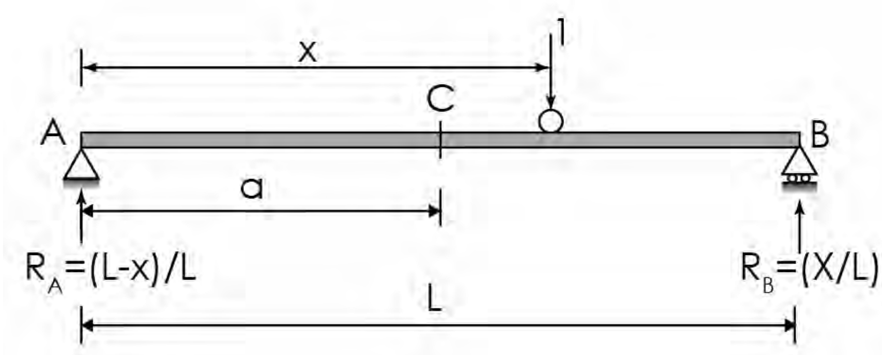
When the unit load in portion AC

$SF_C = -R_b = -\dfrac{X}{L}$ (linear variation with 'x')

When x = 0; $SF_C = 0$

When x = a; $SF_C = -a/L$

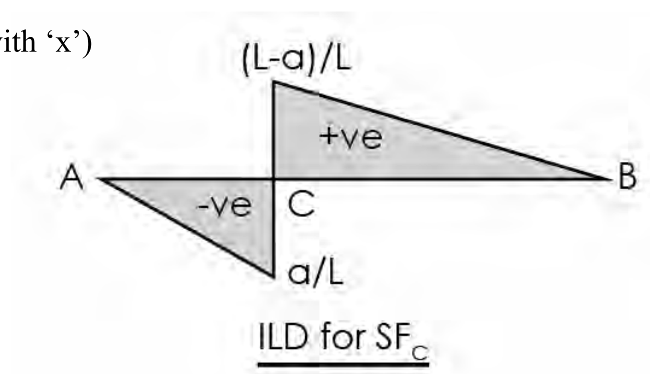
When the unit load in portion CB



$SF_C = R_A = \dfrac{(L-x)}{L}$ (Linear variation with 'x')

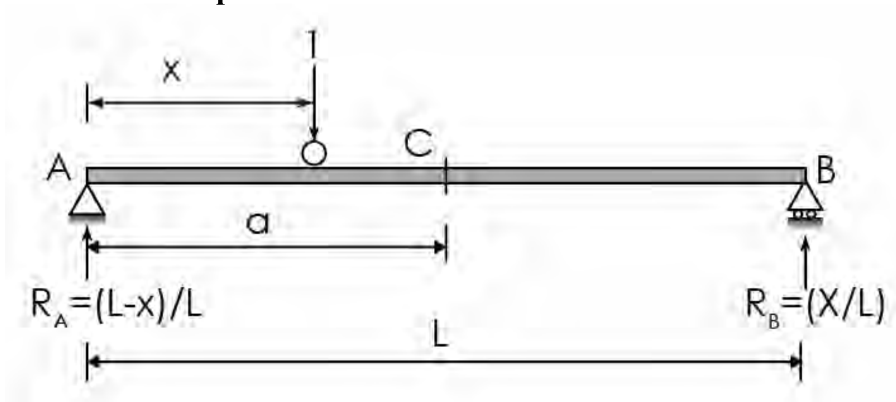
When x = a; $SF_C = \dfrac{L-a}{L}$

When x = L; $SF_C = 0$



ILD for Bending moment at 'C':

When the unit load portion AC



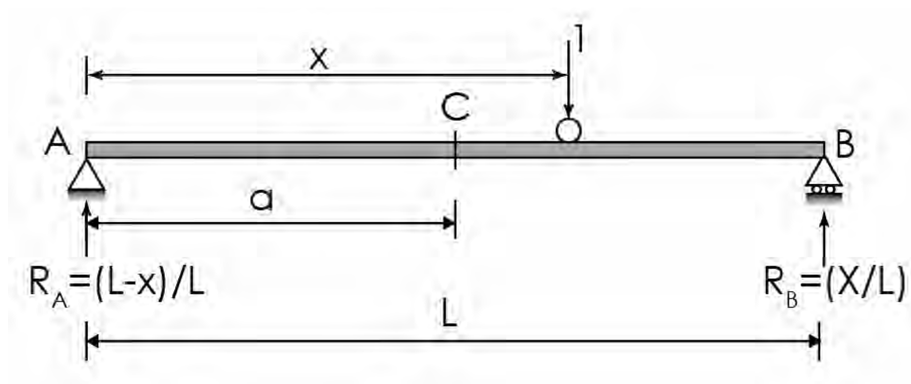
$$BM_C = R_B\,(L - a)$$

$$= \frac{X}{L}(L - a) \text{ (linear variation with 'x')}$$

When x = 0; $BM_C = 0$

When x = a; $BM_C = \frac{a}{L}(L - a)$

When the unit load in portion CB

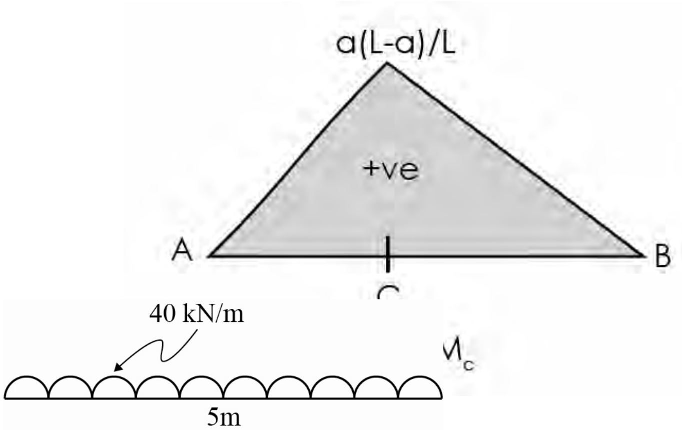


$$BM_C = R_A \times a$$

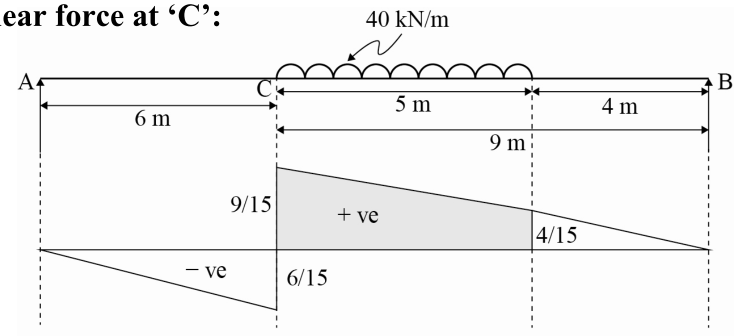
$$BM_C = \frac{(L - x)}{L} \times a \text{ (linear variation with x)}$$

When x = a; $BM_C = \frac{(L - a)}{L} \times a$

When x = L; $BM_C = 0$



Maximum shear force at 'C':



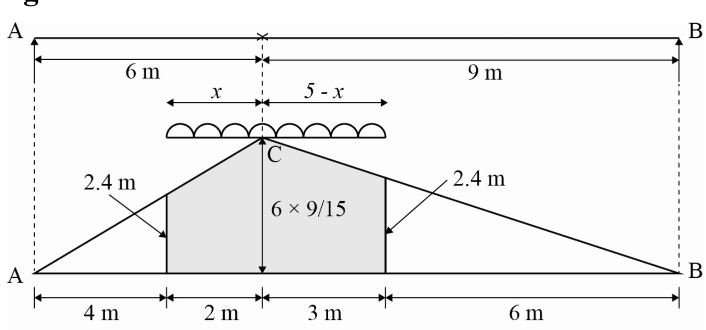
ILD for SF at C

Maximum shear force at 'C'

= intensity of udl × area of shaded ILD

$$=40\left[\frac{\frac{9}{15}+\frac{4}{15}}{2}\right]5=86.67\text{ kN}$$

Maximum bending moment at 'C':



ILD for BM at C

Average load on AC = Average load on BC

$$\frac{x}{6} = \frac{5-x}{9}$$

$$x = 2 \text{ m}$$

Max BM_c = Intensity of ud$l$ × Area of shaded ILD

$$\text{Max } BM_c = 40\left[\frac{2.4+3.6}{2}\right]2 + 40\left[\frac{2.4+3.6}{2}\right]3$$

$$= 600 \text{ kN-m}$$

03.(a)

(i) Describe the various tests performed to assess the suitability of Lime as a cementing material. (8 M)

Sol: The various tests performed to assess the suitability of lime as cementing material are as follows:

1. Fineness Test (IS 6932 (Part IV)): The fineness of lime is determined by doing sieve analysis.
2. Determination of Residue on Slaking of Quick Lime (IS 6932 (part III)): This is determined by measuring the residues on 850 micron IS sieve and 300 micron IS sieve.
3. Workability Test (IS 6932 (Part VIII)): This test is conducted on a standard flow table and a truncated conical mould.
4. Setting Time Test: The initial and final setting times of hydrated lime are determined using vicat's apparatus in the same way as that for Portland cement.
5. Soundness Test: This test is done to find the quality, i.e., the unsoundness or disintegration property of lime using the Le-chatelier apparatus.
6. Popping and Pitting Test (IS 6932 (Part X)): This test is performed to determine the soundness of fat lime.
7. Transverse Strength Test (IS 6932 (Part VII)): Test specimens of size 25x25x100 mm are prepared from standard lime-sand mortar (1:3) and tested and modulus of rupture if determined.
8. Compressive Strength Test (IS 6932 (Part VII)): Cubes of sides 50mm are prepared from standard lime-sand mortar (1:3). These cubes are loaded until failure in UTM/CTM to determine the compressive strength of lime.

(ii) The strength of a sample of fully matured concrete is found to be 50 MPa. Find the strength of identical concrete at the age of 7 days when cured at an average temperature of 25°C during day time and 15°C during the night time. Take constants A and B as 32 and 54 respectively. These are the Plowman's Coefficients for Maturity Equation. (12 M)

Sol: Given strength of fully matured concrete = 50 N/mm^2

Maturity of concrete at the age of 7 days = Σ time × Temperature

$= 7 \times 12 \times (25 - (-11)) + 7 \times 12 \times (15 - (-11))$

$= 7 \times 12 \times 36 + 7 \times 12 \times 26$

$= 5208\ ^{o}\text{C hours}$

Given A = 32 & B = 54

% of Strength of maturity 5208°C hr

$$= A + B\log_{10}\frac{5208}{1000}$$

$$= 32 + 54\log_{0}\frac{5208}{1000}$$

= 70.7%

∴ Strength of concrete at 7 days

$$= 50 \times \frac{70.7}{100}$$

= 35.35 N/mm^2

03.(b)

(i) What combination of Principal stresses will give the same factor of safety for failure by yielding according to the maximum shear stress theory and distortion energy theory. Consider only a two dimensional case. (10 M)

Sol: **Maximum shear stress theory (for 2D):**

$$\sigma_1 - \sigma_2 = \frac{f_y}{FS}$$

Maximum distortion energy theory:

For 3D:

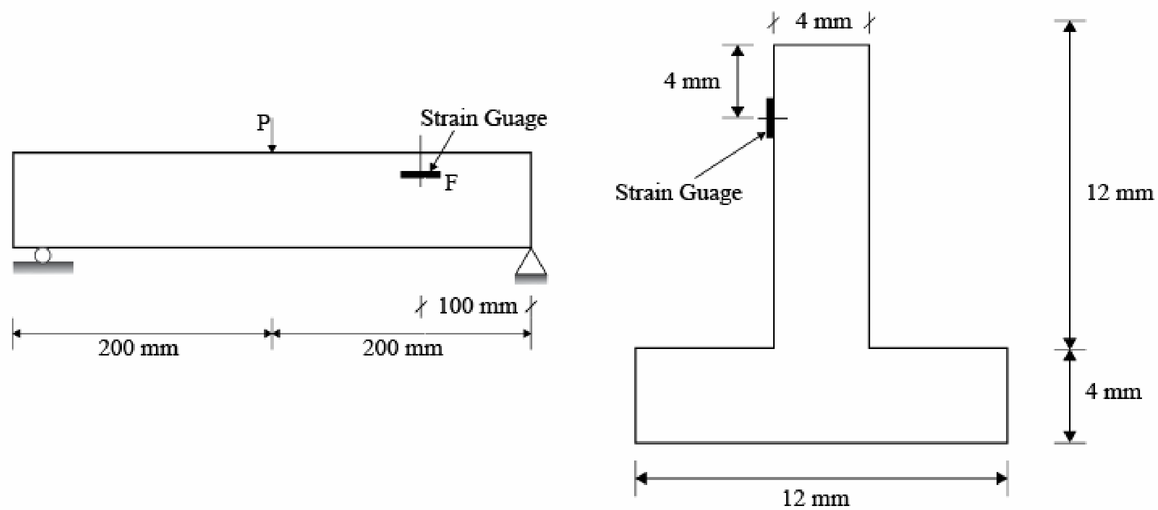
$$(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2 = 2\left(\frac{f_y}{FS}\right)^2$$

for 2D ($\sigma_3 = 0$):

$$(\sigma_1-\sigma_2)^2+(\sigma_1)^2+(\sigma_2)^2=2\left(\frac{f_y}{FS}\right)^2$$

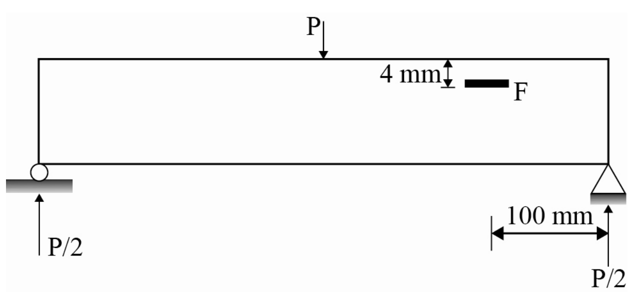
If a member is subjected to uniaxial stress system where $\sigma_1 = \sigma$ and $\sigma_2 = 0$. The factor of safety in "maximum shear stress theory" is equal to "maximum distortion energy theory"

(ii)



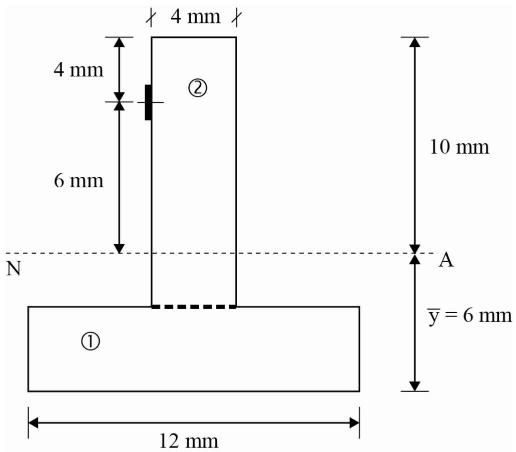
A small T-section is used in inverted position as a beam and is shown in figure over a span of 400 mm. If due to the application of forces shown, the longitudinal strain gauge at F registers a compressive strain of 1500 microstrains, determine the magnitude of P. Take E = 200 GPa. (10 M)

Sol:



Bending moment @ F

$$M_F=\frac{P}{2}(100)$$



Distance of NA from base

$$\bar{y} = \frac{(4\times12)(10)+12\times4\times2}{4\times12+12\times4}$$

$= 6$ mm from base

MI about NA

$$I_{NA} = \frac{4\times12^3}{12}+4\times12\times4^2+\frac{12\times4^3}{12}+12\times4\times4^2$$

$= 2176\ mm^4$

Bending stress of strain gauge, $f_F = (\varepsilon_F)\ (E)$

$$f_F = (1500\times10^{-6})\ (200\times10^3)$$

$= 300$ MPa

From bending equation

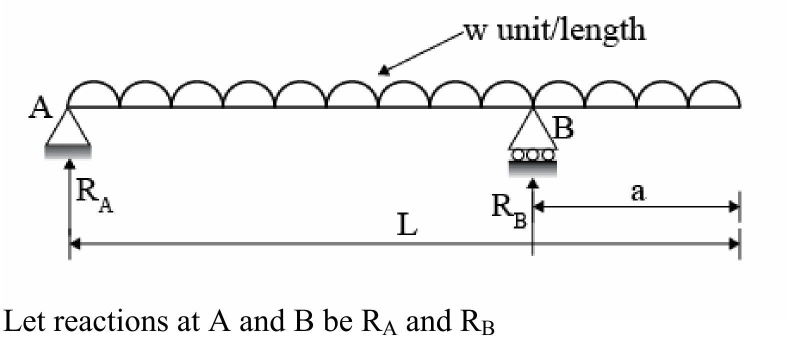
$$\frac{M_F}{I_F} = \frac{f_F}{y_F}$$

$$\frac{\left(\frac{P}{2}\right)(100)}{2176} = \frac{300}{6}$$

$$P = 2176\ N$$

03.(c) A beam of span L carries a uniformly distributed load w per unit length on its whole span. It has one simple support at its left end and other support is at a distance of 'a' from the other end. Find the value of 'a' so that the maximum bending moment for the beam is as small as possible. Find also the maximum bending moment for this condition. (20 M)

Sol:



Let reactions at A and B be R_A and R_B

$$R_A + R_B = wL \quad \_\_\_\_ (1)$$

$$\Sigma M_A = 0$$

$$R_B \times (L-a) - w \times L \times \frac{L}{2} = 0$$

$$R_B = \frac{wL^2}{2(L-a)}$$

$$\therefore \quad R_A = wL - \frac{wL^2}{2(L-a)}$$

$$= \frac{wL(2L-2a-L)}{2(L-a)}$$

$$R_A = \frac{wL(L-2a)}{2(L-a)}$$

For maximum bending moment to be a minimum as possible.

Maximum sagging moment = maximum Hogging moment

$$\frac{wL(L-2a)}{2(L-a)} \times \frac{(L-a)}{2} - \frac{w(L-a)^2}{8} = \frac{wa^2}{2}$$

$$\frac{wL(L-2a)}{4} - \frac{w(L-a)^2}{8} = \frac{wa^2}{2}$$

$$\frac{L(L-2a)}{2} - \frac{(L-a)^2}{4} = a^2$$

$$2L^2 - 4aL - L^2 + 2aL - a^2 = 4a^2$$

$$L^2 - 2aL = 5a^2$$

$$5a^2 + 2aL - L^2 = 0$$

$$a = \frac{-2L \pm \sqrt{(2L)^2 - 4(5)(-L)^2}}{2(5)}$$

$$= \frac{-2L \pm \sqrt{4L^2 + 20L^2}}{10}$$

$$= \frac{-2L \pm \sqrt{24L^2}}{10}$$

$$a = 0.289L$$

Maximum Bending moment $= \dfrac{wa^2}{2} = \dfrac{w \times (0.289L)^2}{2}$

$= 0.0417\ wL^2$

04.(a)

(i) Write briefly about the following

(A) Air Entraining admixtures

(B) Role of Flyash as a part replacement of cement **(10 M)**

Sol:

(A) Air Entraining Admixtures:

- These are the chemical admixtures which are used to improve the workability of concrete.
- These admixtures create tiny entrapped air bubbles which improve the workability of concrete.
- These admixtures also improve resistance of concrete against freezing and thawing.
- Aluminium powder, resins, vegetable oils and fats are some of the commonly used air entraining admixtures.

(B) Role of Flyash as a part replacement of cement:

The use of fly ash as part replacement of cement has many benefits and improves concrete performance in both the fresh and hardened state.

- **Improved Workability:** The spherical shaped particles of fly ash act as miniature ball bearings within the concrete mix, thus providing a lubricant effect.

- **Decreased water demand:** The replacement of cement by fly ash reduces the water demand for a given slump. When fly ash is used at about 20 percent of the total cementitious, water demand is reduced by approximately 10 percent. Higher fly ash contents will yield higher water reductions.
- **Reduced heat of hydration:** Replacing cement with the same amount of fly ash can reduce the heat of hydration of concrete. This reduction in the heat of hydration does not sacrifice long-term strength gain or durability. The reduced heat of hydration lessens heat rise problems in mass concrete placements.
- **Increased ultimate strength:** Because of the pozzolanic action of flyash with available lime, the strength of concrete keeps on increasing over time, thus giving higher ultimate strength.

(ii) Calculate the quantities of ingredients required to produce one cubic metre of structural concrete. The mix is to be used in proportions of 1 part of cement to 1.42 parts of sand to 2.94 parts of 20 mm nominal size crushed coarse aggregate by dry volumes with a w/c ratio of 0.49 (by mass). Assume the bulk densities of cement, sand and coarse aggregate to be 1500, 1700 and 1600 kg/m^3 respectively. The percentage of entrained air is 2.0. Take specific gravity of cement, sand and coarse aggregate as 3.15, 2.6 and 2.6 respectively. (10 M)

Sol: Given mix proportion 1:1.42:2.94 (by volume)

w/c = 0.49 $\Rightarrow V_{FA} = 1.42\ V_c$

% Entrained air = 2% $V_{CA} = 2.94\ V_c$

$$\Rightarrow \text{Net volume of concrete } = 1 - \frac{2}{100} = 0.98 \text{ m}^3$$

$$\Rightarrow 0.98 = \frac{W_w}{1000} + \frac{W_c}{3.15\times 1000} + \frac{W_{FA}}{2.6\times 100} + \frac{W_{CA}}{2.6\times 1000}$$

$$= \frac{0.49 w_c}{1000} + \frac{1500V_c}{3.15\times 1000} + \frac{1700V_{FA}}{2.6\times 1000} + \frac{1600V_{CA}}{2.6\times 1000}$$

$$= \frac{0.49\times 1500V_c}{1000} + \frac{1500V_c}{3.15\times 1000} + \frac{1700\times 1.42V_c}{2.6\times 1000} + \frac{1600\times 2.94V_c}{2.6\times 1000}$$

$\Rightarrow V_c = 0.248\ \text{m}^3$

$\Rightarrow w_c = 1500 \times 0.248$ $V_{CA} = 2.94\ V_c$

$= 372$ kg $= 2.94 \times 0.248 = 0.729\ \text{m}^3$

$w_W = 0.49 w_c$ $\quad\quad$ $w_{CA} = 1600\ V_{CA}$

$= 0.49 \times 0.372$ $\quad\quad$ $= 1600 \times 0.729$

$= 182$ kg $\quad\quad$ $= 1166$ kg

$V_{FA} = 1.42\ V_c$ $\quad\quad$ $w_{FA} = 1700\ V_{FA}$

$= 1.42 \times 0.248$ $\quad\quad$ $= 1700 \times 0.352$

$= 0.352\ m^3$ $\quad\quad$ $= 598$ kg

04.(b)

(i) Explain briefly with an example the Acceptance Criteria for Concrete as per IS 456-2000.

(8 M)

Sol: Acceptance Criteria:

Compressive Strength:

The concrete shall be deemed to comply with the strength requirements when both the following conditions met

(a) The mean strength determined from any group of four consecutive test results complies with the appropriate limits in below table

(b) The individual test result complies with the appropriate limit in below table

S. No	Grade of Concrete	Mean of the group of 4 non overlapping consecutive test results in N/mm^2	Individual test result in N/mm^2
1	M15	$\left.\begin{array}{l}\geq f_{ck} + 0.825\sigma \\ \text{(or)} \\ \geq f_{ck} + 3\end{array}\right\}$ Max	$\geq f_{ck} - 3$
2	M20 and above	$\left.\begin{array}{l}\geq f_{ck} + 0.825\sigma \\ \text{(or)} \\ \geq f_{ck} + 4\end{array}\right\}$ Max	$\geq f_{ck} - 4$

Ex: The following four non overlapping consecutive test results in N/mm^2 for batch of M20 concrete have been obtained: 29, 17, 25 and 27 N/mm^2

Check whether concrete satisfies the complete requirements assumed standard deviation of 4 MPa

Individual test result $\geq f_{ck} - 4$

$$\geq 20 - 4 = 16 \text{ N/mm}^2$$

In the present case minimum individual strength is 17 N/mm^2 > 16 N/mm^2 $\therefore$ O.K

Now mean strength = (29 + 17 + 25 + 27)/4 = 24.5 N/mm^2

As per above table, mean strength shall be greater of

(i) $f_{ck} + 0.825\,\sigma = 20 + 0.825 \times 4 = 23.3$ MPa

(ii) $f_{ck} + 4 = 20 + 4 = 24$ MPa

The actual strength = 24.5 N/mm^2 > 24 N/mm^2

$\therefore$ The concrete satisfies the strength requirement.

(ii) Calculate the modulus of rigidity and bulk modulus of a cylindrical bar of diameter 30 mm and of length 2.0 m if the longitudinal strain in a bar during a tensile stress is six times the lateral strain. Find the change in the volume, when the bar is subjected to a hydrostatic pressure of 120 N/mm^2. Take E = 1 × 10^5 N/mm^2 (12 M)

Sol: Given data;

d = 30 mm; l = 2 m = 2000 mm

Longitudinal strain = 6 times lateral strain

$$E = 1 \times 10^5 \text{ N/mm}^2$$

Hydrostatic pressure = P = 120 N/mm^2

We have Poisson's ratio $= \mu = \dfrac{\text{lateral strain}}{\text{longitudinal strain}} = \dfrac{1}{6} = 0.167$

$$E = 2G(1 + \mu)$$

$$\Rightarrow \quad G = \frac{E}{2(1+\mu)} = \frac{1\times 10^5}{2(1+0.167)} = 0.428\times 10^5 \text{ N/mm}^2$$

And E = 3k (1 – 2μ)

$$k = \frac{E}{3(1-2\mu)} = \frac{1\times 10^5}{3(1-2\times 0.167)} = 0.50 \times 10^5 \text{ N/mm}^2$$

$$\text{Volumetric strain} = \varepsilon_v = \frac{p}{k} = \frac{120}{0.50\times 10^5} = 0.0024$$

∴ Decrease in volume = $\delta v = \varepsilon_v . v$

$$= 0.0024 \times \frac{\pi}{4} d^2 \times \ell$$

$$= 0.0024 \times \frac{\pi}{4} \times 30^2 \times 2000$$

$$= 3392.92 \text{ mm}^3$$

Result:

1. $G = 0.428 \times 10^5$ MPa
2. $K = 0.50 \times 10^5$ MPa
3. $\delta V = 3392.92 \text{ mm}^3$

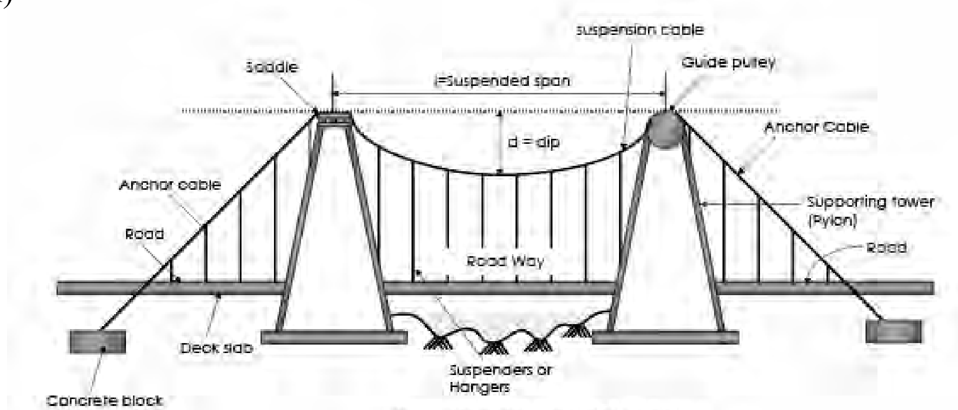
04.(c) A suspension cable of 160 m span and 16 m central dip carries a load of $\frac{1}{2}$ kN per linear horizontal metre. Calculate the maximum and minimum tension in the cable. Also find horizontal and vertical forces in each pier under the following alternate conditions:

(i) If the cable passes over the frictionless pulley on the top of the piers.

(ii) If the cable is firmly clamped to saddles carried on frictionless roller on the top of the piers

In each case the backstay is inclined at 30° to the horizontal (20 M)

Sol: (i)



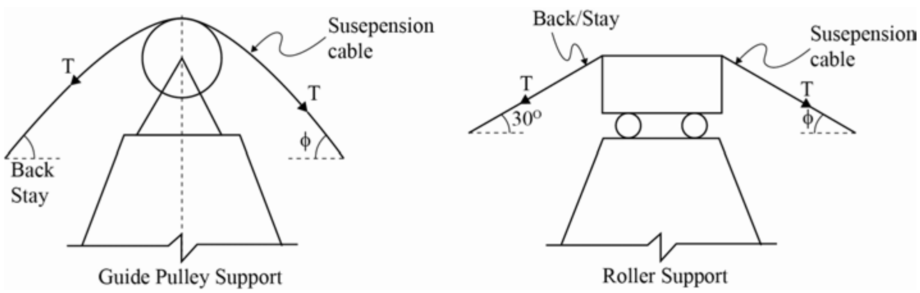
Suspension bridge with a stiffening girder

Consider the suspension cable between the supports vertical reaction at each end of the cable

$$V = \frac{wL}{2} = \frac{\frac{1}{2}\times 160}{2} = 40\,\text{kN}$$

Horizontal reaction at each end of the suspension cable

$$H = \frac{wL^2}{8h} = \frac{1/2\times 160^2}{8\times 16} = 100\,\text{kN}$$



The maximum tension (T) in the cable is always at its ends while the minimum tension in the cable is at its lowest point and is equal to H.

$$T_{min} = H = \frac{wL^2}{8h} = \frac{\frac{1}{2}(160)^2}{8\times 16} = 100\,\text{kN}$$

$$T = T_{max} = H\sqrt{1+16\frac{h^2}{L^2}}$$

$$= 100\sqrt{1+\frac{16\times 16^2}{160^2}} = 107.703\,\text{kN} \simeq 108\text{ kN}$$

Forces in the Pier:

When the cable passes over the frictionless pulley, the tension in the back stay is equal to the tension in the cable. Let the inclination of the cable be ϕ with horizontal.

The load on pier = T sin ϕ + T sin30° = V + T sin30°

= 40 + 108 sin30°

= 94 kN

Net horizontal load transmitted to the pier

= T cosϕ – T cos30°

= H – T cos30°

= 100 – 108 cos30° = 6.47 kN

(ii) When the cable is clamped to a saddle with rollers resting on the pier:

Let T_s = Tension in the back stay

$\therefore$ $T_s \cos 30° = T \cos\phi = H = 100$

$$T_s = \frac{100}{\cos 30°} = 115.5\,\text{kN}$$

$\therefore$ Total compression on pier

$= T_s \sin 30° + T \sin\phi = T_s \sin 30° + V$

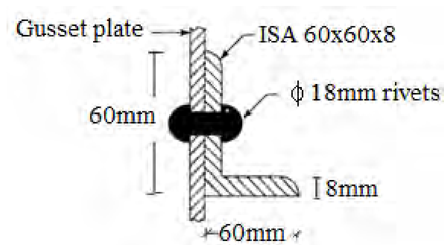
$$= \frac{115.5}{2} + 40$$

$= 97.8$ kN

SECTION-B

05.(a) In a roof truss a diagonal consists of an ISA 60 mm × 60 mm × 8 mm (ISA 6060 @ 0.07 kN/m) and it is connected to gusset plate by one leg only by 18 mm diameter rivets in one chain line along the length of the member. Determine tensile strength of the member, if yield stress for steel in 250 MPa. (12 M)

Sol: Yield stress of steel $f_y = 250$ Mpa



Permissible axial tensile stress $\sigma_{at} = 0.6\times f_y$

$\sigma_{at} = 0.6\times f_y = 0.6\times 250 = 150$ MPa

Nominal diameter of rivet $\phi = 18$ mm

Gross diameter of rivet d = 18 + 1.5 = 19.5 mm

Tensile strength of the tie member $P_t = A_{net} \times \sigma_{at}$

Net effective sectional area of single angle $A_{net} = A_1 + A_2K_1$

Where Reduction factor $K_1 = \dfrac{3A_1}{3A_1 + A_2}$

A_1 = Net sectional area of connected leg

$= (60 - 19.5 - 8/2) \times 8 = 292\ \text{mm}^2$

A_2 = Gross sectional area of outstanding leg

$= 2\times(60 - 8/2) \times 8 = 448\ \text{mm}^2$

$$K_1 = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 292}{3 \times 292 + 448} = 0.66$$

Net effective sectional area of double angle

$$A_{net} = A_1 + A_2K_1 = 292 + 448 \times 0.66$$
$$= 587.68 \text{ mm}^2$$

The safe tensile force of tie $P_t = A_{net} \times \sigma_{at}$

$$= 587.68 \times 150$$
$$= 88.15\times 10^3 \text{ N} = 88.15 \text{ kN}$$

05.(b) Check the adequacy of a HB 450@ 0.872 kN/m rolled steel beam section for a column to carry an axial load of 1100 kN. The column is 4 m long and restrained in position but not in direction at both ends. Allowable axial stress in compression is 105 MPa. The sectional properties of the given section are as follows:

$A = 11114$ mm^2, $r_{xx} = 187.8$ mm, $r_{yy} = 51.8$ mm (12 M)

Sol: Axial compressive load P = 1100 kN

Allowable axial stress in compression σ_{ac} = 105 Mpa

Unsupported length of column L = 4 m

Effective length of column KL= 1.0 L = 4 m = 4000 mm

A_e=11114 mm^2, r_{xx} = 187.8 mm and r_{yy}=51.8mm

Safe compressive strength of the column $P_c = \sigma_{ac}\times A_e$

$$= 105\times 11114$$
$$= 1166.97\times 10^3 \text{ N} = 1166.97 \text{ kN} \geq P = 1100 \text{ kN}$$

Hence the column HB450 is safe

Check for slenderness ratio of the column

Effective slenderness ratio of the column $= \dfrac{KL}{r_{min}}$

$$= \frac{4000}{51.8} = 77.22 \leq 180$$

(Maximum slenderness ratio as per IS800)

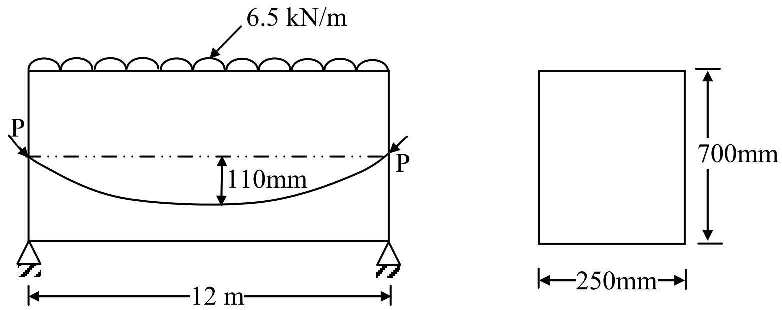
Hence the column HB450 is safe and adequate

05.(c) A prestressed concrete beam supports an imposed load of 6.5 kN/m over an effective span of 12 m. The beam has a rectangular section of width 250 mm and depth of 700 mm. Find the effective prestressing force in the cable if it is parabolic with an eccentricity of 110 mm at the centre and zero at the ends, for the following conditions:

(i) if the bending effect of the prestressing force is nullified by the imposed load for the mid-span section (neglecting self weight of the beam)

(ii) if the resultant stress due to self-weight, imposed load and prestressing force is zero at the soffit of the beam for the mid-span section. Assume the density of concrete is 24 kN/m^3 (12 M)

Sol:



(i) The external load is balanced by cable force

Hence equate moment due to prestressing to moment due to external load

$M_P = m_E$

$$Ph = \frac{w\ell^2}{8}$$

$$\frac{P(110)}{1000} = \frac{6.5 \times 12^2}{8}$$

P = 1063.6 kN

(ii) Stress at soffit at midspan is zero

i.e. $f_b = 0$ (stress at bottom fibre)

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

Self weight of the beam, $w_D = \gamma bD$

$= 24 \times 0.25 \times 0.7$

$= 4.2$ kN/m

Moment due to self weight, $m_D = \frac{w_D L^2}{8}$

$$= \frac{4.2 \times 12^2}{8} = 75.6 \text{ kN}$$

Moment due to live load, $m_L = \frac{w_L L^2}{8}$

$$= \frac{6.5 \times 12^2}{8} = 117 \text{ kN}$$

Total moment, $m = m_D + m_L = 75.6 + 117 = 192.6$ kN-m

Bending stress, $\frac{m}{Z} = \frac{192.6 \times 10^6}{\frac{1}{6} \times 250 \times 700^2} = 9.43 \text{ N/mm}^2$

$\Rightarrow$ Stress at bottom fibre is zero

$$p_b = \frac{P}{A} + \frac{Pe}{Z} - \frac{m_D + m_L}{Z} = 0$$

$$\Rightarrow \frac{P}{250 \times 700} + \frac{P(110)}{\frac{1}{6} \times 250 \times 700^2} = 9.43$$

P = 849.4 kN

05.(d) Define the terms activity, event and Net work. (12 M)

Sol: **Activity:** It can be defined as an identifiable, quantifiable, measurable, cost able, assignable and controllable, lowest level, element of work, which must be performed during the course of a project for achieving the project mission. It is the actual performance of a task. Activity requires time and resources such as manpower, material, space etc., for its completion and it has definite start and finish time.

Event: It is a state of commencement of completion of an activity. It is a instantaneous stage in the project. It is used to connect the activity. The commencement or completion of an activity is called *an event*.

In a network diagram it is a junction of arrows representing activities.

(i) It is either the start or completion of an activity.

(ii) It represents a note worthy, significant and recognizable point in the project.

Events act as control points in a project. It is an accomplishment occurring at a instantaneous point in time, *but requiring no time or resource itself.*

Network: It is the pictorial representation of the inter relationships of all required events and activities comprising a project.

Basic elements of Project Network are

(i) Activity. (ii) Event .

For construction planning two kinds of networks can be used

(a) Activity-On-Arrow diagram (AOA diagram)

(b) Activity-On-Node diagram (AON diagram)

05.(e) Find the moment of resistance of a beam 300 × 600 mm deep if it is reinforced with 3 Nos. of 20 mm dia. Bars in compression and tension, each at an effective cover of 40mm. Use M20 grade concrete and steel grade Fe 415.

Points on stress-strain curve for Fe 415 steel **(12 M)**

Stress level	Fe 415 grade	
	Strain	Stress (N/mm^2)
0.80 fy	0.00144	288.7
0.85 fy	0.00163	306.7
0.90 fy	0.00192	324.8
0.95 fy	0.00241	342.8
0.975 fy	0.00276	351.8
1.00 fy	0.00380	360.9

Sol: Given: b = 300 mm; d = 600 mm

f_{ck} = 20 MPa; f_y = 415 MPa

Area of tensile steel = $3\times\frac{\pi}{4}\times20^2 = 942.477\,\text{mm}^2$

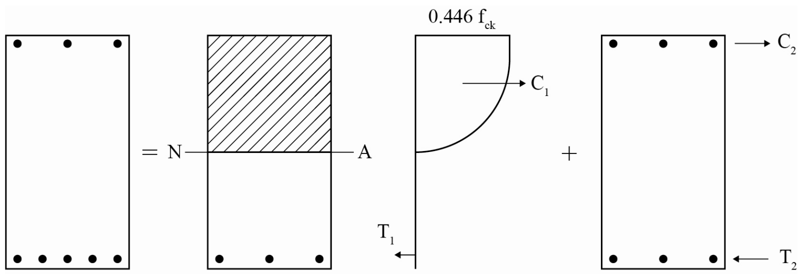
Area of compressive steel = $3\times\frac{\pi}{4}\times20^2 = 942.477\,\text{mm}^2$

Effective cover = 40 mm; effective depth 'd' = 600 – 40 = 560 mm

In doubly reinforced beams.

$$\text{MOR} = \begin{pmatrix}\text{moment of resistance of}\\ \text{compression concrete and}\\ \text{corresponding tensile steel}\\ \text{to balance it}\end{pmatrix} + \begin{bmatrix}\text{moment of resistance of}\\ \text{compression steel and remaining}\\ \text{tensile steel to balance it}\end{bmatrix}$$

i.e



In the given question, area of compression steel is equal to tension steel. This shows that contribution from concrete is to very less. To analyse this type of problems, steel beam theory is used.

In steel beam theory the assumption are

(i) Compression is resisted only by compression steel i.e. concrete is neglected.

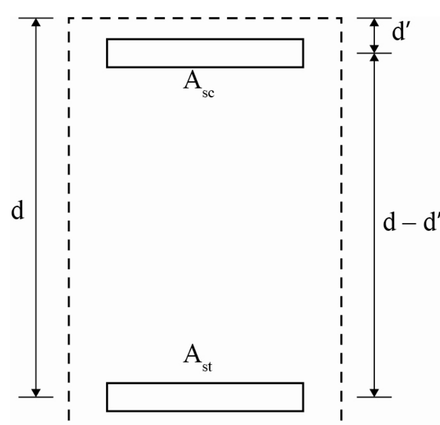
(ii) Tension is resisted only by tension steel

(iii) Stress in compression steel = stress in tension steel

(iv) Concrete serves only as web of an 'I' beam whole flanges are represented by compression and tension.

This is applicable when % of tensile reinforcement <3%

For the given case: $\frac{A_{st}}{bd}\times 100 = \frac{942.477}{300\times 600}\times 100 = 0.52\%$



Hence steel beam theory can be used.

∴ The transformed beam as per steel beam theory

Moment of resistance = $\sigma_{st}A_{st}$ (d – d′)

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

$= 0.87 \times 415 \times 942.477\ (560 - 40)$

$= 176.946 \times 10^6$ N-mm

$= 176.946$ kNm

Note: In this method, quality of concrete is considered. Also, as permissible stress in compressive steel and tensile steel is assumed to be same, the stress in concrete will be more than the allowable stress. Hence, this theory is generally not recommended. However, it can be used to find the approximate value of moment of resistance of section when area of compression steel is equal to tensile steel (and these areas are less than 3% of cross sectional area).

06.(a)

(i) What are the various modes of failure for a steel beam? (6 M)

Sol: Various modes of failures of steel beam are

1. Excessive bending triggering collapse

   This is the basic failure mode provided (1) the beam is prevented from buckling laterally,(2) the component elements are at least compact, so that they do not buckle locally. Such "stocky" beams will collapse by plastic hinge formation.

2. Lateral torsional buckling of long beams

   Which are not suitably braced in the lateral direction.(i.e. "un restrained" beams) Failure occurs by a combination of lateral deflection and twist. The proportions of the beam, support conditions and the way the load is applied are all factors, which affect failure by lateral torsional buckling.

3. Failure by local buckling of a flange in compression or web due to shear or web under compression due to concentrated loads

   Unlikely for hot rolled sections, which are generally stocky. Fabricated box sections may require flange stiffening to prevent premature collapse. Web stiffening may be required for plate girders to prevent shear buckling. Load bearing stiffeners are sometimes needed under point loads to resist web buckling.

4. Local failure by (1) shear yield of web (2) local crushing of web (3) buckling of thin flanges.

(ii) A pitched roof is to be provided for a workshop of effective span 18m. the trusses are spaced at 4m centre to centre and purlins at 1.6m centre to centre. The pitch of the roof is 28°, weight of the roofing material is 0.162 kN/m, normal wind pressure is 1.2 kN/m^2 and permissible bending stress is 165 MPa. Check the suitability of ISLB 12575 @ 0.119 kN/m section for purlins, if I_{xx} = 406.8 cm^4 and I_{yy} = 43.4 cm^4 for given section (14 M)

Sol: Effective span of purlin L = 4 m

$\theta = 28^0$

$I_{XX} = 406.8\ cm^4 = 406.8 \times 10^4\ mm^4$

$I_{YY} = 43.4\ cm^4 = 43.4 \times 10^4\ mm^4$

Spacing of purling (centre to centre distance of purling) = 1.6m

Normal wind pressure = 1.2 kN/m^2 = 1.200 N/m^2

Wind load per meter length

$H_1 = 1200 \times 1.6 = 1920$ N/m

Weight of roofing material = 0.162 kN/m = 162 N/m

Self weight of purlin = 0.119 kN/m = 119 N/m

Total gravity (or) vertical load on purlin per m length $P_1 = 162+119 = 281$ N/m.

Load along minor axis of purlin $P = P_1 \cos\theta + H_1$

$= 281 \cos 28° + 1920 = 2168.108$ N/m

Load along major axis of purlin $H = P_1 \sin\theta$

$= 281 \sin 28° = 131.921$ N/m

Maximum bending moment about major axis

$$M_{XX} = \frac{PL^2}{10} = \frac{2168.108 \times 4^2}{10}$$

$= 3468.97$ N-m $= 3.46 \times 10^6$ N – mm

Maximum bending moment about minor axis

$$M_{YY} = \frac{HL^2}{10} = \frac{131.921 \times 4^2}{10}$$

$= 211.07$ N-m

Calculated maximum bending stresses in prulin

$$\sigma_{bc\ cal} = \sigma_{bt\ cal} = \frac{M_{XX}}{I_{XX}}.y + \frac{M_{YY}}{I_{YY}}.x$$

$$= \frac{3.46 \times 10^6}{406.8 \times 10^4} \times \left(\frac{125}{2}\right) + \frac{211.07 \times 10^3}{43.4 \times 10^4} \times \left(\frac{75}{2}\right)$$

$= 71.36$ N/mm^2 $\leq$ Permissible bending stresses $\sigma_{bc} = \sigma_{bt} = 165$ N/mm^2

Hence the purlin ISLB125 is adequate

06.(b) Design a two way slab for an office room 5.8 m × 4.2 m clear in size if the superimposed load is 4 kN/m². Use M25 grade of concrete and steel grade Fe 415. The bending moment coefficients for two-way slabs simply supported on four sides is given below:

l_y/l_x	1.0	1.1	1.2	1.3	1.4	1.5	1.75	2.0
α_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118
α_y	0.062	0.061	0.059	0.055	0.051	0.046	0.037	0.029

Assume the edges simply supported and the corners not held down. Assume the shape factor for shear k = 1.3.

Design shear strength of concrete of M25 grade. (20 M)

100 Ast/bd	τ_c N/mm²
0.25	0.36
0.50	0.49
0.75	0.57
1.00	0.64

Sol: Given: Clear dimensions = 5.8 m × 4.2 m

Superimposed load = w_{LL} = 4 kN/m²

f_{ck} = 25 MPa; f_y = 415 MPa

Assume a total depth of 150 mm with effective cover of 30 mm.

∴ Effective depth 'd' = 150 – 30 = 120 mm

Step 1: Effective length:

Assuming support of 300 mm wide:

$$\ell_x = \left.\begin{matrix}\ell_{xo} + d \\ \ell_{xo} + b\end{matrix}\right\} \text{minimum}$$

$$= \left.\begin{matrix}4.2 + 0.12 = 4.32\text{m} \\ 4.2 + 0.3 = 4.5\text{m}\end{matrix}\right\} \text{minimum}$$

$$= 4.32 \text{ m}$$

Similarly,

$$\left.\begin{aligned}\ell_y &= 5.8 + 0.3 = 6.1 \text{ m}\\ &5.8 + 0.12 = 5.92 \text{ m}\end{aligned}\right\}\text{minimum}$$

$$= 5.92 \text{ m}$$

$$\frac{\ell_y}{\ell_x} = \frac{5.92}{4.32} = 1.37$$

Step II: Load calculation:

Self weight = $\gamma_c \times D$

Assuming $\gamma_c = 25$ kN/m^3

$\therefore\ w_{DL} = 25 \times 0.15$

$= 3.75$ kN/m^2

$W_{LL} = 4$ kN/m^2

Total load = 7.75 kN/m^2

Total factored load = 1.5×7.75

$= 11.625$ kN/m^2

Step III: Moment Coefficient:

From given table: using linear interpolation for $\frac{\ell_y}{\ell_x} = 1.37$

Short span coefficient $\alpha_x = \left[\left(\frac{1.37 - 1.3}{1.4 - 1.3}\right) \times (0.099 - 0.093)\right] + 0.093$

$= 0.0972$

Long span coefficient $\alpha_y = 0.051 + \left[\left(\frac{1.4 - 1.37}{1.4 - 1.3}\right) \times (0.055 - 0.051)\right]$

$= 0.0522$

Step IV: Bending Moment:

$M_x = \alpha_x w l_x^2$

$= 0.0972 \times 11.625 \times 4.32^2$

$= 21.087$ kNm

$M_y = \alpha_y\, w l_x^2$

$= 0.0522 \times 11.625 \times 4.32^2$

$= 11.325$ kNm

Step V: Check for depth:

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$$

$$= \sqrt{\frac{21.087 \times 10^6}{0.138 \times 25 \times 1000}}$$

$= 78.18$ mm

Provided effective depth = 120 mm

Hence ok.

Step VI: Area of steel required:

a) Along short direction:

$$A_{st} = 0.5 \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right) bd$$

$$= 0.5 \times \frac{25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 21.087 \times 10^6}{25 \times 1000 \times 120^2}}\right] \times 1000 \times 120$$

$= 525.09\ \text{mm}^2$

Minimum reinforcement required = 0.12% bD = $\frac{0.12}{100} \times 1000 \times 150 = 180\ \text{mm}^2$

$\therefore$ Provide $A_{st} = 525.09\ \text{mm}^2$

Using 10 mm bars spacing = $\dfrac{1000 \times \frac{\pi}{4} \times 10^2}{525.09} = 149.57$ mm

Maximum permissible spacing

$$\left.\begin{matrix} 3d = 360\ \text{mm} \\ 300\ \text{mm} \end{matrix}\right\} \text{minimum} = 300\ \text{mm}$$

$\therefore$ Provide 10 mm ϕ bars @ 140 mm c/c.

Along Y – direction:-

$$A_{st} = \frac{0.5\times 25}{415}\left[1-\sqrt{1-\frac{4.6\times 11.325\times 10^6}{25\times 1000\times 120^2}}\right]\times 1000\times 120$$

$= 271.76\ mm^2$ ($> A_{st,\ min} = 180\ mm^2$)

Using 10 mm ϕ bars, spacing = $\dfrac{1000\times \frac{\pi}{4}\times 10^2}{271.76} = 289.03$ mm

Maximum permissible spacing = 300 mm

∴ Provide 10 mm ϕ bars @ 280 mm c/c.

Since corners are not held down, torsion reinforcement is not required.

Step VII: Check for shear:

Maximum shear force = $\dfrac{w\ell_x}{2}$

$$V_u = 11.625\times \frac{4.32}{2} = 25.11\ kN$$

Nominal shear stress $\tau_V = \dfrac{V_u}{bd}$

$$= \frac{25.11\times 1000}{1000\times 120} = 0.209\ MPa$$

% A_{st} provided = $\dfrac{A_{st}}{bd}\times 100 = \dfrac{\left(\dfrac{1000\times \frac{\pi}{4}\times 10^2}{140}\right)\times 100}{1000\times 120}$

$= 0.467\%$

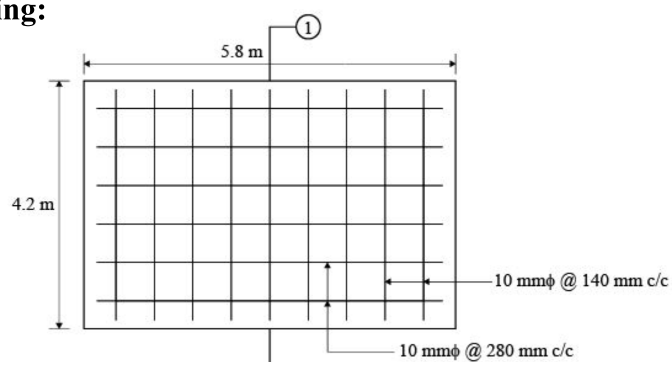
From given table $\tau_c = 0.36 + \left(\dfrac{0.5-0.467}{0.5-0.25}\right)(0.49-0.36)$

$= 0.376$ MPa

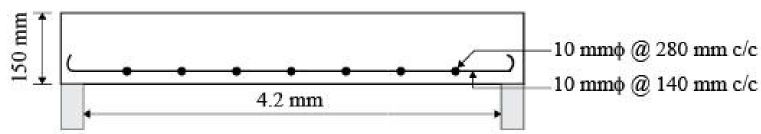
Design shear strength = $k\tau_c$ = 1.3 × 0.376 = 0.49 MPa > 0.209 MPa

Hence safe in shear.

Detailing:



Along (1) – (1)



Check for development length:

$$L_d = \frac{\phi 0.87 f_y}{4\tau_{bd}} = \frac{10 \times 0.87 \times 415}{4 \times 1.2 \times 1.6} = 470.11 \text{ mm}$$

Curtailing alternate bars at 0.15 l_x = 0.15 × 4.32 = 0.648 m

Moment of resistance at support = 0.87 f_y A_{st} (d – 0.42x)

$$x = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times \frac{\pi}{4} \times 10^2 \times \frac{1000}{2 \times 140}}{0.36 \times 25 \times 1000} = 11.25 \text{ mm}$$

$$M = 0.87 \times 415 \times \frac{\pi}{4} \times 10^2 \times \frac{1000}{2 \times 140}(120 - 0.42 \times 11.25)$$

= 11.67 × 10^6 Nmm

= 11.67 kNm

V = 25.11 kN

$$L_d \le 1.3 \frac{M_1}{V} + L_o$$

$$\left.\begin{aligned} L_o &= 12\phi = 120 \text{ mm} \\ d &= 120 \text{ mm} \end{aligned}\right\} \text{maximum} = 120 \text{ mm}$$

$$\Rightarrow 470.11 \le 1.3 \times \frac{11.67 \times 10^3}{25.11} + 120$$

$$\le 724.40 \text{ mm}$$, Hence safe.

06.(c) Briefly explain at least five different types of vibrators used in cement concrete making industry (20 M)

Sol:

Concrete vibrators are used in concrete compaction for different construction and structural requirements. Concrete contains different size particles, the compaction is necessary and can be achieved by different vibrations with different speeds of vibration.

Different types of vibrations are

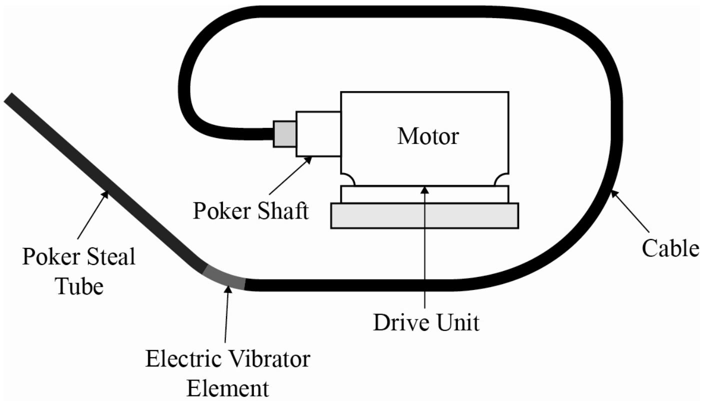
1. Needle vibrator (or) Immersion vibrator
2. Shutter vibrator (or) External Vibrator
3. Surface vibrator
4. Vibrating table
5. Rebar shaker

1. **Needle Vibrator:** it is also known as immersion vibrator. It consists a steel tube with end closed and rounded, having an eccentric vibrating element inside the tube. The steel tube is named as Poker is connected to an electric motor (or) any prime mover (like oil engine) through a flexible tube. The standard diameter size of pokers varies from 40 mm to 100 mm. The size of the poker selected based on the spacing between the reinforcing bars in the form work. The normal action of radius of an immersion vibrator varies from 500 mm to 1000 mm. The preferable to immerse the vibrator into concrete at intervals of not more than 600 mm (or) 8 to 10 times the poker tube diameter.

The frequency of vibration varies upto 15000 rpm. The suggested range is 3000 to 6000 rpm the acceleration of vibrator i.e. varies from 4g to 10 g.

The time period of vibration required may be of the order of 30 seconds to 2 minutes.

The concrete should be placed in layers limited to 600 mm maximum.



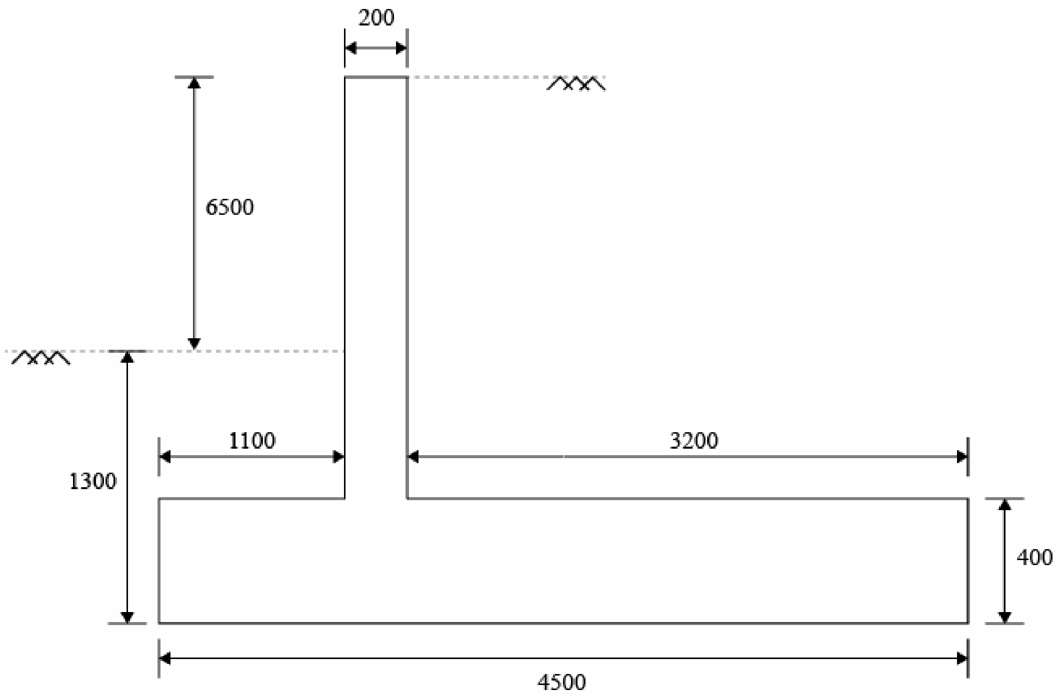
2. **Shutter Vibrator:** It is also known as external vibrator. It is clamped rigidly to the form work at predetermined points so that the form and concrete are vibrated. It can compact upto 450 mm from the face, is to be moved from one place to another as concrete progresses. It operate at a frequency of 3000 rpm to 9000 rpm at an acceleration of 4g to 5 g. It is more often used for pre-casting of thin in-situ sections of such shape and thickness as cannot be compacted by internal vibrator. It consume more power for given compaction than internal vibrator.

3. **Surface Vibrator:** It is placed directly on the concrete mass. The operating frequency is about 4000 rpm at an acceleration of 4g to 9g. It is mainly found in compaction of small slabs, not exceeding 150 mm in thick and patching and repair work of pavement slabs. It is commonly used are pan vibrator and vibrating screeds. Dry mixes can be most effectively compacted with this type of vibrator. This vibrator is best suited for compaction of shallow elements and it should not be used when depth of concrete to be vibrated is more than 250 mm.

4. **Vibrating Table Vibrator:** It consists a vibrating table in rigid from built by steel plot form mounted on flexible springs and is being driven by an electric motor.
The recommended frequency of vibration is 4000 rpm at an acceleration of 4g to 7g. It is very efficient in compacting stiff and harsh concrete mixes required for making precast elements in the factories and test specimens in laboratories.

5. **Rebar Shaker:** This device is slipped over the top of the reinforcing bar and shakes (or) transmits the vibration into the concrete. It can be found in different diameter sizes and can result in great savings in man hours and reduce clean-up activities. This type vibrator tool will shorten the time it takes to pouring concrete into a cell (or) in a very tight space.

07.(a) Design the counterforts of a retaining wall to retain earth for a height of 6.5m above the ground level. The unit weight of soil is 16 kN/m^3 and the angle of repose of soil is 30°. The safe bearing capacity of soil is 180 kN/m^2. Use M20 grade concrete and steel of grade Fe415. The cross-section of the retaining wall is given below. The spacing of counterfort is taken as 3.5m. Assume a cover of 40mm for counterforts.



All dimensions are in mm.

Assume the maximum pressure at toe end is 166.05 kN/m^2 and the minimum pressure at the heel end is 38.92 kN/m^2. Sketch the reinforcement details. (20 M)

Sol:

Given:

Height of earthfill above GL = 6.5m.

γ_{soil} = 16 kN/m^3; ϕ = 30;

SBC of soil = 180 kN/m^2

Spacing of counterfort = 3.5 m

Effective cover = 40 mm,

f_{ck} = 20 MPa, f_y = 415 MPa

Step-I:

Total height h = 6.5 + 1.3 = 7.8 m

Thickness of counterforts = 0.05 h

$= 0.05 \times 7.8 = 0.39$ m

$\therefore$ Assume thickness of counterfort as 500 mm.

c/c spacing between counterforts = 3.5 m

Step-II: Loads Calculation:

Counterfort is subjected to earth pressure and downward reaction from heel slab. Each counterfort receives earth pressure from a width of 3.5 m.

Intensity of earth pressure at base of stem

$$p_r = k_a \gamma h$$

$$h = 6.5 + 1.3 - 0.4 = 7.4 \text{ m}$$

$$k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$$

$$r = 16 \text{ kN/m}^3$$

$$\therefore\ P_a = \frac{1}{3}\times 16\times 7.4 = 39.467 \text{ kN/m}^2$$

Bending moment at base = $\left(\frac{1}{2}\times P_a \times h \times \frac{1}{3}\right)$ c/c spacing

$$= \frac{1}{2}\times 39.467\times 7.4\times\frac{7.4}{3}\times 3.5$$

$= 1260.71$ kNm

Shear force at base = $\frac{1}{2}\times P_a \times h$ c/c spacing $= \frac{1}{2}\times 39.467\times 7.4\times 3.5$

$= 511.09$ kN

Factored BM (M_u) = 1.5 × 1260.71 = 1891.06 kNm

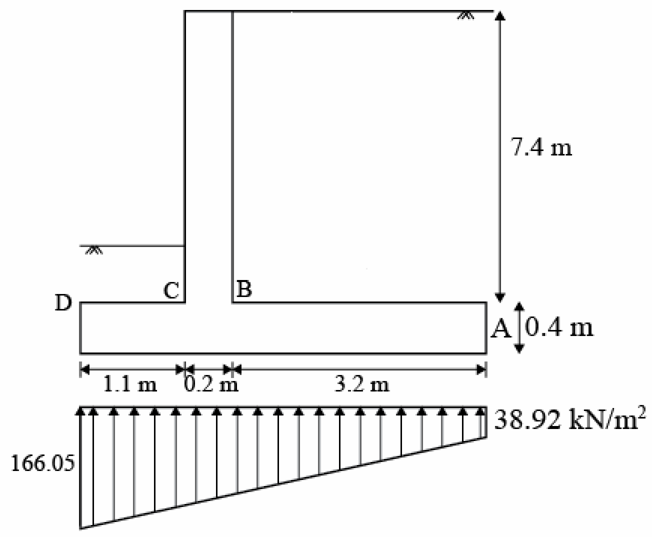
Factored SF (V_u) = 1.5 × 511.09 = 766.635 kNm

Net downward pressure on heel slab at 'A'

= weight due to earth fill + weight of heel slab – upward soil pressure

= 16 × 7.4 + 25 × 0.4 – 38.92 = 89.48 kN/m^2

Total downward force at A = 89.48 × 3.5 = 313.18 kN/m



Step-III: Counter fort acts as a T-beam

$\therefore$ effective flange width = min $\left(\frac{l_0}{6} + b_w + 6D_f, b_w + \text{clear span}\right)$

l_o = 7.4 m, b_w = 500 mm; D_f = 200 mm

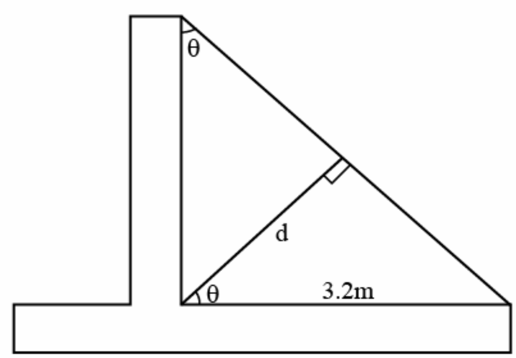
Clear span = 3.5 – 0.5 = 3 m

$\therefore \; b_f = \min\left(\frac{7400}{6} + 500 + 6(200), 3000 + 500\right)$

= min(2933.33, 3500)

= 2933.33 mm

$\simeq$ 2.93 m



Assuming $x_u = D_f$ = 5200 mm

moment of resistance = 0.36 f_{ck} b_{eff} x (d = 0.42 x)

MOR = 0.36 ×2.0×2933.33×200(D – 0.42 × 200)

$\tan\theta = \frac{3.2}{7.4} = 0.432$

$\Rightarrow \theta = 23^o 23' 6.8''$

d = 3.2 cosθ

= 2.937 m = 937 mm

$\therefore$ MOR = 12051.66 kNm > 1891.06 kNm

Hence neutral axis lies in flange

Step-IV: Area of steel required:

$$A_f = \frac{0.5f_{ck}}{f_y}\left(1-\sqrt{1-\frac{4.6M_u}{f_{ck}bd^2}}\right)bd$$

$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 1891.06\times 10^6}{20\times 2933.33\times 2937^2}}\right]\times 2933.33\times 2937$$

= 1791.96 mm^2

Min reinforcement:

$$\frac{A_{sf}}{bd} = \frac{0.85}{f_y}$$

$$A_{sf} = 0.85\times\frac{500\times 2937}{415} = 3007.77\text{mm}^2$$

$\therefore$ Provide A_{st} = 3007 mm^2

Using 25mmϕ bass, number of bars required = $\dfrac{3007.77}{\frac{\pi}{4}\times 25^2} = 6.12$

$\simeq$ 7 No′s

Step-V: Vertical ties: -

Due to net vertical forward force on base slab, it has tendency to separate from counterfort. Hence avoid this vertical ties are provided.

The maximum pull is at end of heel slab ‘A’ = 313.18 kN/m

Considering load factor of 1.5,

area steel required = $\dfrac{1.5\times 313.18\times 10^3}{0.87\times 415}\text{mm}^2/\text{m}$ = 1301.12 mm^2/m

Using 2-legged 10 mmϕ ties,

Spacing = $\dfrac{1000\times\frac{\pi}{4}\times 10^2\times 2}{1301.12} = 120.72\text{mm}$

$\therefore$ provide 10ϕ - 2legged vertical ties @120mm c/c.

Step-VI: Horizontal ties:

Due to horizontal earth pressure vertical stem has a tendency to separate out from counterfort. Here horizontal ties are provided.

Tension resisted by the reinforcement = lateral pressure of wall × area

$= p_a \times 3/\text{mtr}$

$= 39.467 \times 3/\text{m} = 118.401$ kN/m

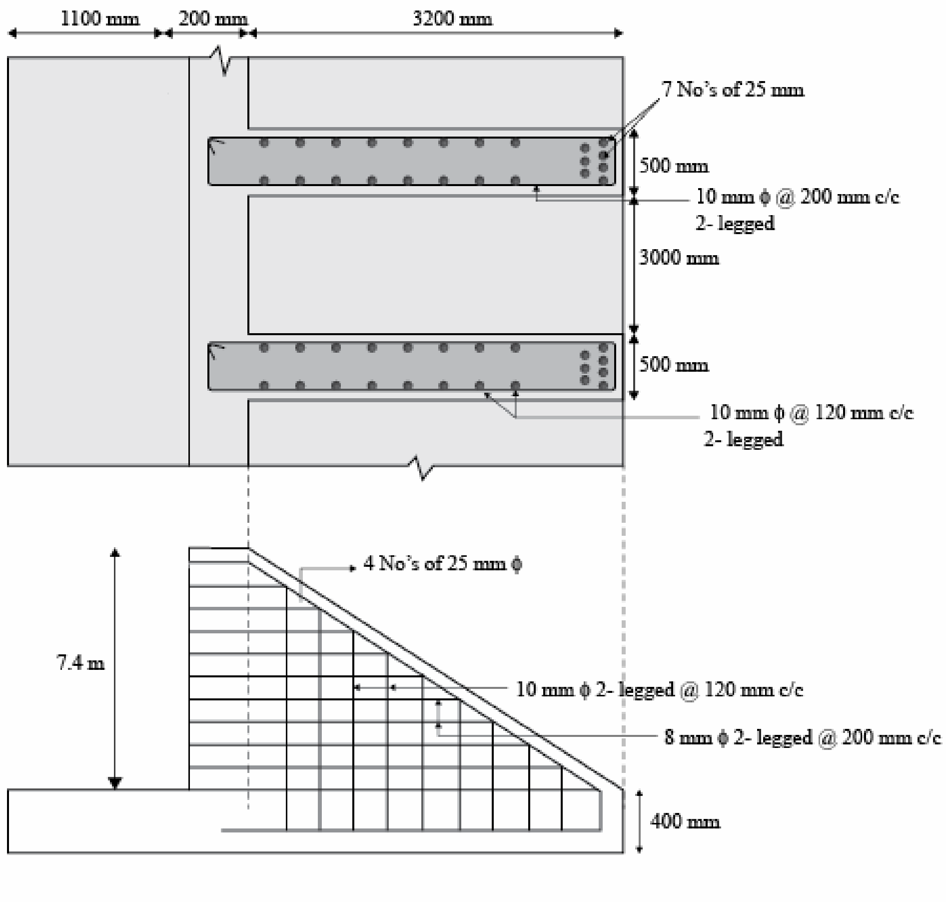
Area of steel required considering load factor is $= \dfrac{1.5 \times 118.401 \times 1000}{0.87 \times 415} = 491.90\ \text{mm}^2/\text{m}$

using 2-legged 8 mmϕ ties:

$$\text{Spacing} = 2 \times \frac{\pi}{4} \times 8^2 \times \frac{1000}{491.90} = 204.37\ \text{mm}$$

$\therefore$ Provide 2 – legged 8 mmϕ ties @ 200 mm c/c

Detailing:



07.(b) Design the side walls of an underground tank of size 12 m × 3 m × 3 m deep. The angle of repose of soil is 30°. The density of soil is taken as 17 kN/m^3. Assume the soil is saturated. Use M25 grade of concrete and Fe 415 grade of steel. Take Q = 1.156 N/mm^2 and J = 0.87

(20 M)

Sol: Given:

L = 12 m, B = 3 m, H = 3 m

$\phi = 30$, $\gamma_{soil} = 17$ kN/m^3

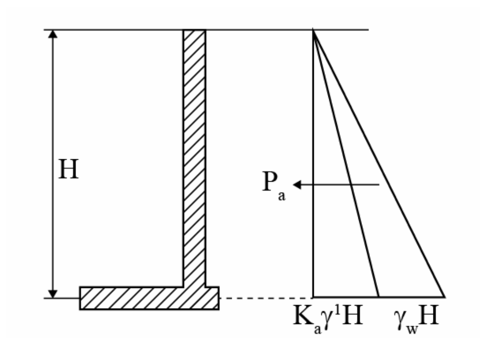
$f_{ck} = 25$ MPa, $f_y = 415$ mPa

Q = 1.156 N/mm^2, J = 0.87

Design of Long Wall:

Case I: Tank is empty and surrounded by saturated soil

Step I: Thickness Required:



$\therefore$ Pressure at bottom $P_a = K_a\gamma' H + \gamma_w H$

$$k_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$$

$\gamma' = 17 - 9.81 = 7.19$ kN/m^3

$\gamma_w = 9.81$ kNm^3

H = 3 m

$$\therefore p_a = \left(\frac{1}{3}\times 7.19\times 3\right) + (9.81\times 3) = 36.62 \text{ kN/m}^3$$

$$\text{BM at the base of wall} = \frac{1}{2}p_a H\times\frac{H}{3}$$

$$M = \frac{1}{2}\times 36.62\times 3\times\frac{3}{3} = 54.93 \text{ kNm}$$

thickness of wall required = $\sqrt{\dfrac{M}{Qb}}$

$$= \sqrt{\frac{54.93\times10^6}{1.156\times1000}} = 217.98 \text{ mm}$$

$\therefore$ Provide total depth = 260 mm; effective depth = 225 mm

Effective cover = 35 mm

Step II: Reinforcement Required:

a) Area of steel required $= \dfrac{M}{\sigma_{st}\, jd}$

$$= \frac{54.93\times10^6}{150\times0.87\times225} = 1870.75 \text{ mm}^2$$

Using 16 mm ϕ bars, Spacing $= \dfrac{1000\times\frac{\pi}{4}\times16^2}{1870.75} = 107.476$ mm

$\therefore$ Provide 16 mm ϕ bars @ 100 mm c/c on outer face of long wall

b) Distribution Steel:

Minimum % upto 100 mm thick wall = 0.3%

Minimum % upto 400 mm thick wall = 0.2%

for 260 mm thick wall

% distribution steel = $0.2 + \left(\dfrac{400-260}{400-100}\right)\times(0.3-0.2)$

= 0.2467%

$\therefore A_{st} = \dfrac{0.2467}{100}\times260\times1000 = 641.33 \text{ mm}^2$

this has to be provided in 2 layers as thickness > 225 mm

$\therefore$ Area in each face $= \dfrac{641.33}{2} = 320.67 \text{ mm}^2$

Using 8 mm ϕ bars, Spacing $= \dfrac{\frac{\pi}{4}\times8^2\times1000}{320.67} = 156.75$ mm

$\therefore$ Provide 8 mm ϕ bars @ 150 mm c/c on each face

Total $A_{st} = 2\times\frac{\pi}{4}\times 8^2\times\frac{1000}{150} = 670.206$ mm

Direct compression in long wall due to earth pressure on short walls at h = above base and short wall

$$h = \max\left(1, \frac{H}{4}\right)$$

$$h = \max\left(1, \frac{3}{4}\right)$$

h = 1m

p at 'h' = $k_a\,\gamma'\,(H–h) + \gamma_w\,(H – h)$

$$= \frac{1}{3}\times 7.19(3-1) + 9.81(3-1)$$

$= 24.41$ kN/m^2

Direct compression P = $p\frac{B}{2} = 24.41\times\frac{3}{2} = 36.62$ kN

$$A_{st} = \frac{C}{\sigma_{st}} = \frac{36.62}{150}\times 10^3 = 244.13 \text{ mm}^2$$

Hence distribution steel and wall section will be sufficient to resist this

Case II: Tank full with water; and no soil outside:

Pressure at base = $p_1 = \gamma_w H = 9.81\times 3 = 29.43$ kN/m^2

Bending moment at base = $\frac{1}{2}p_1\times H\times\frac{H}{3}$

$$= \frac{1}{2}\times 29.43\times 3\times\frac{3}{3} = 44.145 \text{ kNm}$$

Area of steel required = $\frac{M}{\sigma_{st}jd} = \frac{44.145}{150\times 0.87\times 225} = 1503.45$ mm^2

Using 16 mm ϕ bars, Spacing = $\dfrac{1000\times\frac{\pi}{4}\times 16^2}{1503.45} = 133.73$ mm

$\therefore$ Provide 16 mm ϕ @ 130 mm c/c on the inner face

Since the top portion of short wall acts as slab supported on long walls, water pressure acting on short walls will cause tension in long wall

$$P_L = p \times \frac{B}{2}$$

'p' is at height of $h = \max\left(1, \frac{H}{4}\right) = 1$ m

$p = \gamma_w (H - h) = 9.81 (3 - 1) = 19.6$ kN/m^2

$\therefore P_L = 19.6 \times \frac{3}{2} = 29.4$ kN

Area of steel required = $\frac{29.4 \times 10^3}{150} = 196 \text{ mm}^2 < \frac{670}{2} \text{mm}^2$

$\therefore$ Distribution steel provide will be suffiicnet

Design of Short Wall:

Case 1: Tank is empty and surrounded by soil from outside:

Step I: Top Portion:

Bottom $h(1\text{m} > \frac{H}{4})$ acts as cantilever, and the remaining 2 m acts as slab supported on long wall

At h, pressure, $p = k_a\gamma(H - h) + \gamma_w(H - h)$

$= 24.41$ kN/m^2

BM at supports = $\frac{pb^2}{12} = 24.41 \times \frac{3^2}{12} = 18.307$ kNm

(Produces tension on outer face)

BM at centre $= \frac{pb^2}{8} - \frac{pb^2}{12} = \frac{pb^2}{24}$

$$= 24.4 \times \frac{3^2}{24} = 9.153 \text{kNm}$$

(Causing tension on inner face)

Effective thickness available

$$= 225 \times \frac{16}{2} - 8 = 209 \text{ mm}$$

Area of steel required at supports = $\dfrac{18.307\times10^{6}}{150\times0.807\times209}$ = 671.21 mm^2

Using 10 mm ϕ bars, Spacing = $\dfrac{1000\times\frac{\pi}{4}\times10^{2}}{671.21}$ = 117.01 mm

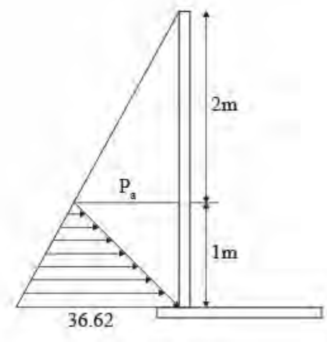
∴ Provide 10 mm ϕ @ 110 mm c/c at support outer face

At centre: $A_{st}=\dfrac{671.21}{2}$ = 335.605 mm^2

Min A_{st} = 641.33 mm^2

∴ Provide A_{st} = 641.33 mm^2

∴ Using 10 mm ϕ @ 120 mm c/c at inner face at midspan



Step II: Bottom Portion:

Bottom 1 m bends as cantilever:

Earth pressure at bottom = 36.62 kN/m^3 (from long wall design)

Bending moment at bottom $=\left(\dfrac{1}{2}\times36.62\times1\right)\times\dfrac{1}{3}$

= 6.1 kNm (Tension on outer face)

$A_{st}=\dfrac{M}{\sigma_{st}jd}=\dfrac{6.1\times10^{6}}{150\times0.87\times209}$ = 223.77 mm^2

Minimum A_{st} = 641.33 mm^2

∴ Provide 10 mm ϕ @ 120 mm c/c at outer face for bottom 1 m height in vertical direction

Step III: Direct compression in short wall:

Direct compression due to end one meter

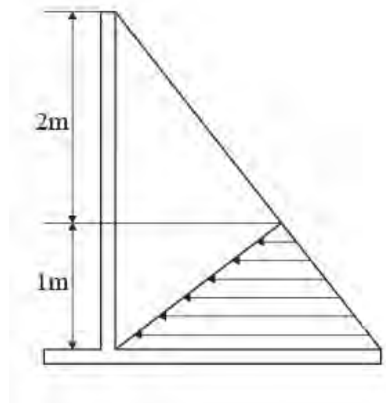
Width of long wall = 24.41 × 1 = 24.41 kN

$A_{st}=\dfrac{24.41\times10^{3}}{150}$ = 162 mm^2

Distribution steel provided will be sufficient

Case II: Tank full width water and no soil outside:

Step I: Top Portion:



At h = 1 m, $P = \gamma_w(H - h)$

$= 9.81\ (3 - 1) = 19.62\ \text{kN/m}^2$

BM at supports $= \dfrac{pB^2}{12} = 19.62 \times \dfrac{3^2}{12} = 14.715$ kNm

(Produces tension inner face)

BM at centre (M_c) $= \dfrac{pB^2}{24} = 7.35$ kNm

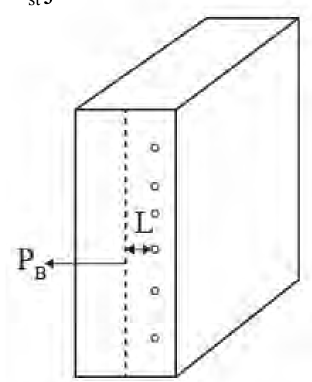
(produces tension on outer face)

Direct tension due to end one metre width of long wall = T = 19.62 × 1 = 19.62 kN

$\therefore$ Design BM = M – Tx

at inner face, $x = d - \dfrac{T}{2} = 209 - \dfrac{260}{2} = 79$ mm

$\therefore$ Area of steel required at ends on inner face at supports = $\dfrac{M - Tx}{\sigma_{st}\, jd}$



$$A_{st_1} = \frac{(14.715 - 19.62(0.079))}{150 \times 0.87 \times 209} \times 10^6 = 482.68\ \text{mm}^2$$

$$A_{st_2} = \frac{T}{\sigma_{st}} = \frac{19.62 \times 10^3}{150} = 130.8\ \text{mm}^2$$

$\therefore A_{st} = A_{st_1} + A_{st_2} = 613.48\ \text{mm}^2$

Min. $A_{st} = 641.33\ \text{mm}^2$

$\therefore$ Provide 10 mm ϕ 120 mm c/c at inner face of supports

Area of steel required at outside face at centre:

$$A_{st_1} = \frac{M_c - Tx}{\sigma_{st}\, jd}$$

$$= \frac{(7.35 - 19.62(0.079)) \times 10^6}{150 \times 0.87 \times 209} = 212.65\ \text{mm}^2$$

$$A_{st_2} = \frac{T}{\sigma_{st}} = 130.8\ \text{mm}^2$$

$\therefore$ Total $A_{st} = A_{st_1} + A_{st_2} = 343.45\ \text{mm}^2 < A_{st}$, min

$\therefore$ Provide $A_{st,\,min}$ = 641.33 mm^2

i.e. provide 10 mm ϕ @ 120 mm c/c at outer face

Step II: Bottom Portion:

Pressure at bottom = $9.81 \times 3 = 29.43$ kN/m^2

Bending moment at base $\frac{1}{2} \times 29.43 \times 1 \times \frac{1}{3} = 4.905$ kNm

(Producing tension on inner face)

$$A_{st} = \frac{4.905 \times 10^6}{150 \times 0.87 \times 209} = 179.83 \text{ mm}^2 < A_{st,\min}$$

$\therefore$ Provide $A_{st,min}$ = 641.33 mm^2

i.e. provide 10 mm ϕ @ 120 mm c/c at inner face

Summary:

For long Wall:

Provide 16 mm ϕ @ 100 mm c/c vertically on outer face

Provide 16 mm ϕ @ 130 mm c/c vertically on inner face

Provide 8 mm ϕ @ 150 mm c/c horizontally on each face

For Short Wall:

For top 2 m

At support:

Provide 10 mm ϕ @ 110 mm c/c at outer face horizontally

Provide 10 mm ϕ @ 120 mm c/c at inner face horizontally

At midspan:

Provide 10 mm ϕ @ 110 mm c/c at inner face horizontally

Provide 10 mm ϕ @ 120 mm c/c at outer face horizontally

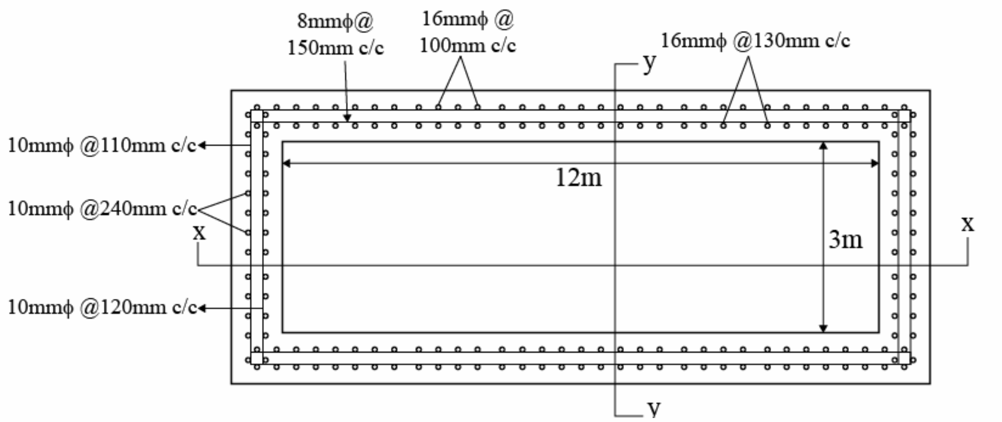
For bottom 1 m:

Provide 10 mm ϕ @ 120 mm c/c at outer face vertically

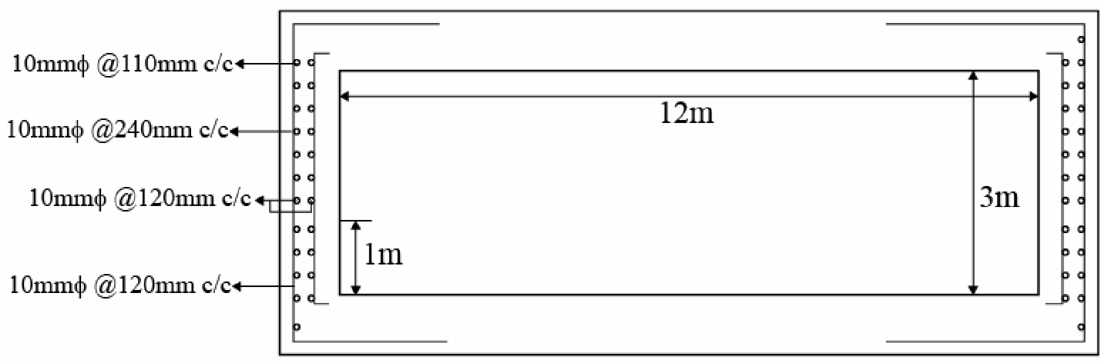
Provide 10 mm ϕ @ 120 mm c/c at inner face vertically

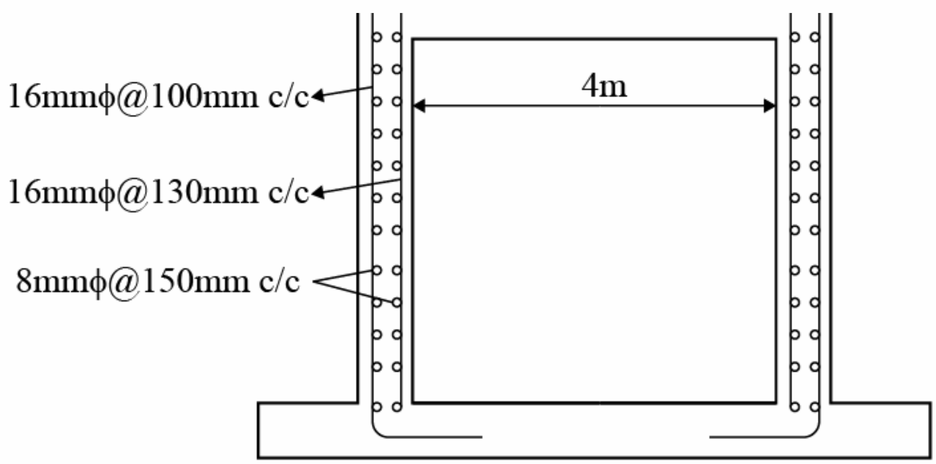
This may be continued till top and spacing can be doubled for top 2 m

Detailing:



Section x-x:





07.(c) Explain the different types of contracts adopted in construction **(20 M)**

Sol: Categories of contract (or) Types of Contracts

Construction Activities

- Engineering
  - → Process Finalization
  - → Design and Structural Analysis
  - → Technical Issues Rested to equipment design and selection etc.
- Procurement
  - → Raw materials
  - → Tools and equipment
  - → Electrical & Plumbing accessories
  - → Other procurements
- Construction
  - → Construction
  - → Form work
  - → Supporting structure
  - → Test run

Categories of Contract

- Separated
  - Lumpsum
  - Measurement
    - Item rate
    - Percentage
  - Cost Plus Percentage
- Management
  - Build only
  - Design only
  - Design and Build
- Integrated
  - Design-Build construction
  - Turnkey
  - Build-Operate Transfer
- Discretionary
  - Partnering
  - Joint Venture

⇒Separated contract:

Traditional system

Sequential process

Clear division between design and construction

Sufficient time is needed, it may need several months

Suitable for simple, small-to-medium sized projects

Time consuming aspects of development process.

(i) Lump sum:

Project drawings and other details are provided by the client.

Contractor quote a single lumpsum figure which is the total contract value.

Note: Contractor will use his experience /Expertize while quoting the amount.

From the client point of view:

Biggest advantage regarding the funds

For a lump-sum contract to be successful it should be ensured that

(a) The quantities of the different items involved are measurable at the stage of tendering.

(b) The nature of work to be done must be reasonably measurable

(c) The contractor must be given all the facilities to which he is contractually entitled.

(ii) Measurement contracts:

Item rate contract:

- The tender document contains detailed bill of quantities (BOQ), Where an estimated quantity of the work for each item in the particular work is listed, along with a detailed description.

  Total contract value is found out by multiplying the quantity of each item by the quoted rate of the contractor and adding the cost of all items.

Percentage rate contract:

The tender documents contain the analysed schedule of rate for each item, in addition to the detailed estimated quantities expected in the execution of works.

This method requires a detailed analysis of rates to be carried out by client organization and usually Govt departments (or) large organization adopt this system.

From the point of view of a client, the method results in tendering that are easily to evaluate and removes lot of problems such as "Front loading".

Note: It is important that the rates used are frequently updated list there are anomalies in the escalation clause (or) the percentage quoted become too high.

(iii) Cost Plus percentage:

In this kind of contract, the client agrees to pay the contractor a certain percentage of the cost incurred by the contract while completing a job, in addition to the cost itself. Thus, the tenderer only quotes this percentage.

This contract is useful for emergency works, when time may not be available to draw up an estimate and workout details of items involved.
This is also suitable for small works where the traditional contract may not be justified.

In certain cases, the cost of material brought to site is directly paid for (against appropriate bills) and any material left over after completion is retained by the client/owner.

⇒ **Management contract**

In management contract, the client has to deal with a single contractor besides a designer.
The principal contractor provides planning, management and co-ordination service to the client.
The design services are provided by the designer, who is separately appointment by the client.

Responsibilities assigned to the management contractor

Preparation of overall construction schedule.
Preparation of work package schedule
Coordinating with the designer to steer through the design stage
Sub-contractors selection
Co-ordinating among different sub-contractor

Note: Principal contractor can contribute some of resources such as form work, cranes etc., to the sub contractors.

Construction management contract:

Construction manager is appointed by the client at an early stage to provide planning, management and coordination.

Responsibilities of Construction manager

Advising the designer
Advising on drawing suitable work package
Assignment in procurement
Managing the bidding process

Design management and construction contract

In this, the client appoints a single contractor to take care of design and construction

The basic design concepts may be provided by the client himself (or) through an independent agency.

⇒ **Integrated contract**

(i) Design-Build: This is a form of contract in which the contractor takes up the responsibility for both design and construction based on basic plans drawn up by the client.

Note: It is well suited when the client has no design / engineering division.

(ii) Turnkey contract:

Modern construction has become very complex and the client prefers to deal with a single organization rather than with a multiple specialist contractor each with his own contractual peculiarity.

Large contracting firms have both the technical and managerial skills to taken such works.

Ex: Engineers India LTD (EIL), L & T, GAMMON INDIA, Hindustan Construction Company etc.

The client prepares documents stating the requirements of the facility to be constructed and either selects the best proposal from those submitted by multiple bidders or designates a specific contractor from the beginning and enters into a contract when negotiations begin.

Turnkey project ⇒ Package deal contract

Note: If owner wants to "Turn the key" at completion to take over the facilities

Project consists of Civil, Electrical, Mechanical, Chemical and Mining works

Eg: Petro chemicals
Nuclear power plants

(iii) Build Operate-Transfer (BOT) contract:

The contractor is allowed to "Operate" the project/facility for an agreed period of time to recover the cost incurred in the design and construction of the facility.

This system is useful when the client does not want to invest directly in the project.

Highways and Airports are constructed on BOT basis

Since these contracts often involves long-term relationships and commitments it is crucial that the contractor carries out his own research into not only the economic and technical feasibility but also the social and administrative aspects of the project.

⇒ **Discretionary contract**

(i) **Partnering:**

Client and the contractor together form a project team based on mutual confidence and then work together to manage the project to successful conclusion, yielding a profit for both parties.

(ii) **Joint Venture:**

The companies usually sign on MOU and form a joint venture. The company providing the project leader (resident manager) is also specified in the MOU.

08.(a) Explain major activities involved in different stages of planning for a construction project (20 M)

Sol: Major activities involved in different stages of planning for a construction project:

➢ **Pre-Project Phase:**

Idea or initiation phase → Project concept phase → Feasibility phase

Initiation or idea phase: The pre-project phase aims to identify all possible project based on the examination of needs and the possible options.

Project concepts Phase: The initiation phase aims to sort out all the mentioned information to identify some project concepts. As many project concepts as possible are identified, and using some selection procedure (such as the benefits for the organization that intends to employ them) in line with the objectives of the organization, several project concepts are selected. The project concept phase of a new construction project is most important, since decisions taken in this phase tend to have a significant impact on the final cost. It is also the phase at which the greatest degree of uncertainty about the future is encountered.

Feasibility phase: This phase aims to analytically appraise project concepts in the context of the organization, taking into consideration factors such as the needs of the organization, the strategic charter of the organization, and the capabilities and know-how of the organization. With this information, the decision makers should be able to decide whether or not to go ahead with the project concept proposed.

The feasibility phase has sub-phases such as market feasibility analysis, technical feasibility, environmental analysis, financial feasibility analysis, It is only after the first three sub-phases are found to be positive that a financial feasibility analysis is performed.

The feasibility phase can be broadly characterized into the following.

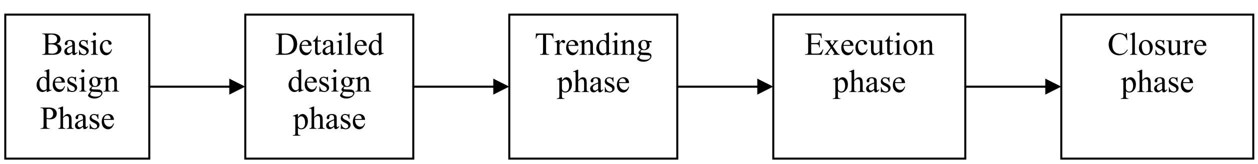
(i) **Conceptual:** For the selected project concepts, the preliminary process diagrams and layouts are prepared. Design basis or design briefs are also formulated.

(ii) **Project Strategy:** The strategy in terms of selection of an in-house design team or the contractor's design team is deliberated upon. The resources required and their availability is discussed.

(iii) **Estimate:** A preliminary estimate is prepared with reasonable accuracy by first breaking down the project into work packages/elements.

(iv) **Approval:** Approval consists of financial evaluation, identifying details of funding and their timing, capital/revenue, etc. besides evaluation of different options.

- **Project Phase:**



Basic design phase:

The activities in this phase are carried out by an engineering organization or an architect. During this phase, the documentation for tendering and contracting the physical construction or for procuring equipment is prepared. It involves performing basic design calculation, preparing tender drawings, preparing design and material specification, etc.

Detailed design phase: Detailed design may be carried out in-house or through contracting. In some cases, such as 'item rate' contract, it may be required to carry out the detailed design before starting the tendering process.

Tendering Phase: Tenders are issued if it is decided to execute the project through contracting. The preparation of clear and precise documents is essential to eliminate any dispute about scope of work at the contract stage. The tender preparation includes preparing the specifications and agreement condition, preparing bill of quantities and estimating the contract value.

Execution or construction phase: Immediately after the contract is awarded, construction phase begins. In cases where the detailed drawings and designs were not available as part of the tender document, the contractor proceeds with the preparation of detailed design and drawings, and follows it up with the construction.

Closure or completion phase: In this phase, the major equipment are tested and commissioned, and the constructed facility in totality is handed over to the client for use. Client issues approval of work and a completion certificate after all the work has been checked and found to be in order.

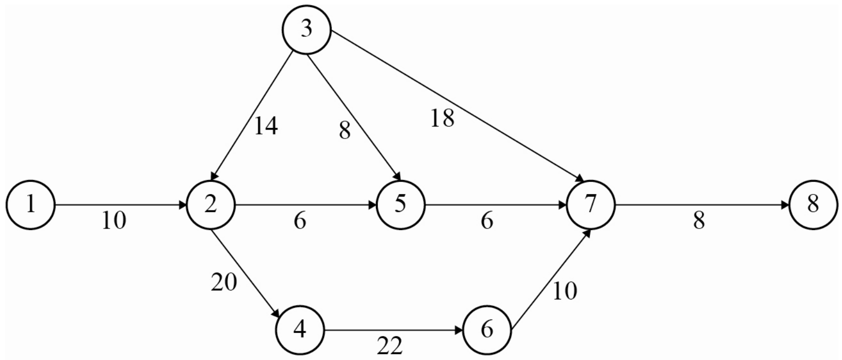
- **Post-Project Phase:**

Utilization phase → Close-down phase

Utilization phase: During this phase, the client or the end user makes use of the finished project. The performance of the constructed facility is monitored at regular intervals, and maintenance at regular intervals is performed.

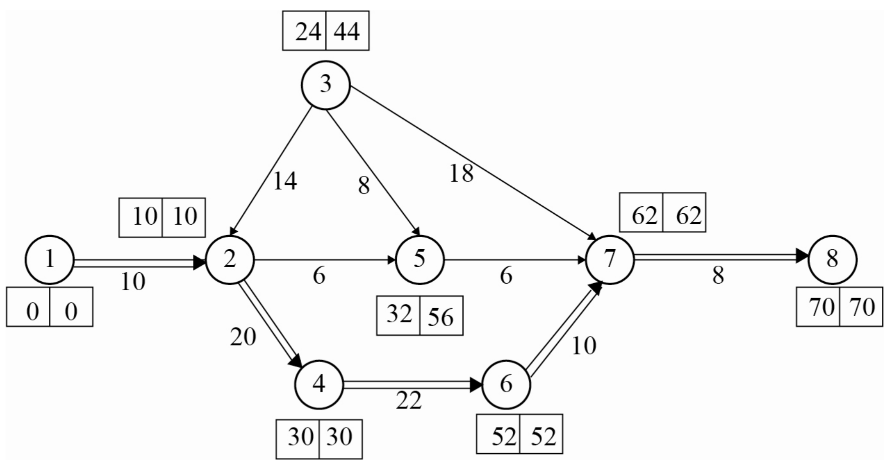
Close-down phase: Once the project has lived its intended life, it is dismantled and disposed of. The entire cycle explained under different phases is repeated.

08.(b)



Identify the critical path in the network as shown in figure and determine the project completion time. The duration are in weeks. **(20 M)**

Sol:



Path	**Duration**
1-2-3-7-8	10 + 14 + 18 + 8 = 50
1-2-5-7-8	10 + 6 + 6 + 8 = 30
1-2-4-6-7-8	10 + 20 + 22 + 10 + 8 = 70

Critical Path: 1-2-4-6-7-8

Project completion time = 70 weeks

08.(c) The opening of a masonry building is 3m and 3.5m high. The ceiling of the roof is 4.5 m above the floor. The space between top of lintel and bottom of roof is filled with brick masonry. The roof transmits a total load of 25 kN/m run to the lintel. Design the lintel supported on brick wall of width 300 mm. Use M20 grade concrete and steel grade of Fe415. Assume the unit weight of the brick masonry is 20 kN/m^3 and that of concrete is 25 kN/m^3. The design shear strength of concrete is given in Table.

$\frac{100A_s}{bd}$	τ_c N/mm^2 M20
= 0.15	0.28
0.25	0.36
0.50	0.48
0.75	0.56
1.0	0.62
1.25	0.67

The design bond stress for M_s bars is given by τ_{bd} = 1.2 N/mm^2 for M20 grade of concrete

(20 M)

Sol: Given: Opening width = 3 m; height = 3.5 m

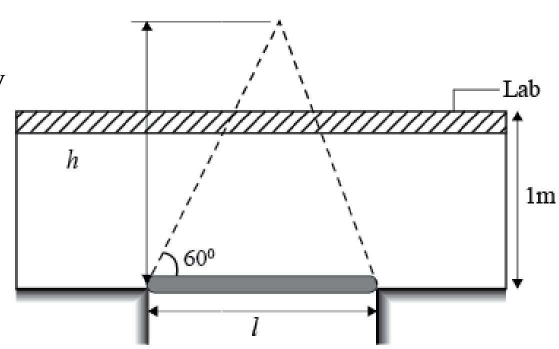
Distance between ceiling and floor = 4.5 m

Load transmitted by roof = 25 kN/m

Width of brick wall = 300 mm

f_{ck} = 120 MPa, f_y = 415 MPa

Step I:



Assuming arch action: for load dispersion of masonry

Effective height required above lintel = $l$sin 60

$= 3 \times \sin 60$

$= 2.59$ m

Available height = 4.5 – 3.5

$= 1$ m < 2.59 m

Also there is roof slab transmitting the load

Hence the load acting on lintel

(i) Self weight of lintel = W_1

(ii) Weight of masonry in rectangle above lintel = W_2

(iii) Load transferred by roof = W_3

Step II:

Let width of lintel = width of brick wall = 300 mm

Effective depth of lintel $= \dfrac{\text{Clear span}}{8}$

$$= \frac{3000}{8} = 375 \text{ mm}$$

$\therefore$ Provide effective depth = 400 mm

and total depth = 450 mm

Effective span = $\left.\begin{matrix} \ell_o + d \\ \ell_o + b \end{matrix}\right\} \min$

$$= \left.\begin{matrix} 3+0.4 \\ 3+0.3 \end{matrix}\right\} \min$$

= 3.3 m

Step III: Load Calculation:

W_1 = Self weight = 25 × 0.45 × 0.3 = 3.375 kN/m

W_2 = Weight of masonry = 20 × 1 × 0.3 = 6 kN/m

W_3 = Load transmitted from roof = 25 kN/m

$\therefore$ Total load = 34.375 kN/m

Factored load = 1.5 × 34.375 = 51.56 kN/m

Maximum Bending Moment = $\dfrac{w\ell^2}{8} = \dfrac{51.56 \times 3.3^2}{8}$ = 70.189 kNm

Step IV: Depth Required:

Effective depth required $= \sqrt{\dfrac{M_u}{0.138 f_{ck} b}}$ for Fe415 steel

$$= \sqrt{\frac{70.189 \times 10^6}{0.138 \times 20 \times 300}} = 291.15 \text{ mm}$$

Provided d = 400 mm

Hence OK

Step V: Area of steel required:

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

$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 70.189\times 10^6}{20\times 300\times 400^2}}\right]\times 300\times 400$$

$= 535.91\ mm^2$

$$\frac{A_{st,min}}{bd} = \frac{0.85}{f_y}$$

$$\Rightarrow A_{st,min} = \frac{0.85\times 300\times 400}{415} = 245.78\ mm^2$$

$\therefore$ Provide A_{st} = 535.91 mm^2

Using 16 mm ϕ bars, = no. of bars = $\frac{535.91}{\frac{\pi}{4}\times 16^2} = 2.66 \simeq 3$ Nos.

Step VI: Check for Shear:

$$V_u = \frac{w\ell}{2} = \frac{51.56\times 3.3}{2} = 85.07\ kN$$

Nominal shear stress $\tau_v = \frac{V_u}{bd} = \frac{85.07\times 10^3}{300\times 400}$

= 0.708 MPa

Area of steel provided = $A_{st} = 3\times\frac{\pi}{4}\times 16^2 = 603.18$

$$\%A_{st} = \frac{603.18}{300\times 400}\times 100 = 0.50\%$$

$\therefore$ From given table, shear strength of concrete = 0.48 MPa

$\tau_v > \tau_c$

$\therefore$ Shear reinforcement is to be designed

Shear force to be resisted by stirrups

$= (\tau_v - \tau_c)bd$

$= (0.708 - 0.48) \times 300 \times 400$

$= 27.36$

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

using 2 legged 8 mmϕ stirrups

$$\Rightarrow 27.36 \times 10^3 = 0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 8^2 \times \frac{400}{S_v}$$

$\Rightarrow S_v = 530.65$ mm

Minimum shear reinforcement

$$\frac{A_{sv}}{b.S_v} = \frac{0.4}{0.87 f_y}$$

$$\Rightarrow S_v = \frac{2 \times \frac{\pi}{4} \times 8^2 \times 0.87 \times 415}{300 \times 0.4} = 302 \text{ mm}$$

Maximum spacing of stirrups $\left.\begin{array}{l} = 0.75d = 300 \text{ mm} \\ = 300 \text{ mm} \end{array}\right\} \text{minimum}$

$= 300$ mm

$\therefore$ Provide 2-legged 8 mmϕ stirrups @ 300 mm c/c

Step VII: Check for Development Length:

$$L_d = \frac{\phi(0.87 f_y)}{4\tau_{bd}};$$ ($\therefore$ $\tau_{bd} = 1.2$ MPa for mild steel)

$$= \frac{12 \times 0.87 \times 415}{4 \times 1.6 \times 1.2} = 564.14 \text{ mm}$$

Moment at support $= 0.87 f_y A_{st} (d - 0.42x)$

$$x = \frac{0.87 f_y A_{st}}{0.36 f_{ck} b} = \frac{0.87 \times 415 \times 3 \times \frac{\pi}{4} \times 16^2}{0.36 \times 20 \times 300} = 100.82 \text{ mm}$$

$$\therefore M = 0.87\times 415\times 3\times \frac{\pi}{4}\times 16^2 (400 - 0.42\times 100.82)$$

$= 77.89 \times 10^6$ Nmm

$= 77.89$ kNm

$V = 85.07$ kN

$$L_d \le \frac{1.3M_1}{V} + L_o \qquad L_o = \left.\begin{aligned} &= 12\phi = 144 \text{ mm} \\ &= d = 400 \text{ mm}\end{aligned}\right\} \max$$

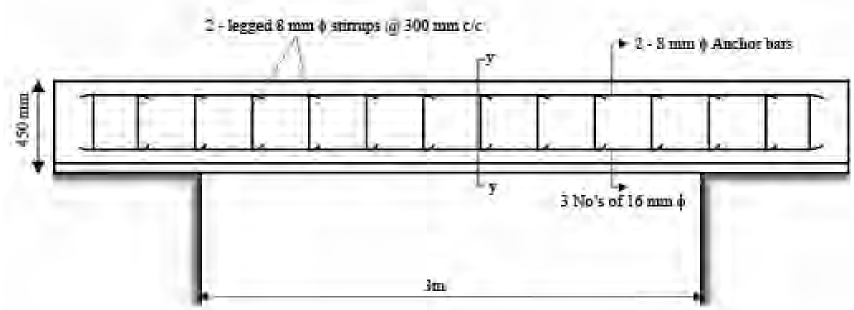
$= 400$ mm

$$\Rightarrow 564.14 \le \frac{1.3\times 77.89}{85.07} + 400$$

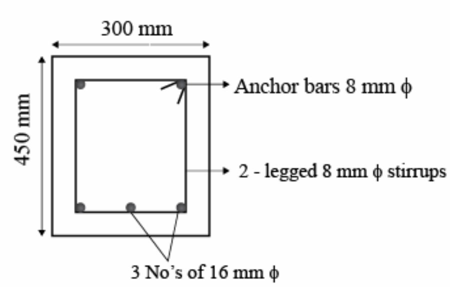
≤ 1590.27 mm

Hence Safe

Detailing:



Section 4-4:

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