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UPSC ESE 2019

Main Exam Detailed Solutions

Civil Engineering

PAPER-I

EXAM DATE : 30-06-2019 | 09:00 AM to 12:00 AM

MADE EASY has taken due care in making solutions. If you find any discrepency/ error/typo or want to contest the solution given by us, kindly send your suggested answer with detailed explanations at info@madeeasy.in

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Civil Engineering Paper Analysis
ESE 2019 Main Examination

ESE Mains Paper-I

Sl.	Subjects	Marks	Percentage Weightage
1.	Building Materials and Construction	84	17.5
2.	Strength of Materials	104	21.66
3.	Structural Analysis	52	10.83
4.	Steel Structures	44	9.17
5.	RCC	104	21.67
6.	CTPM and Equipments	92	19.17
Total		**480**	**100.00**

Scroll down for detailed solutions

SECTION 'A'

1\. (a) (i) Explain briefly the various tests conducted on Bricks mentioning the relevant codal provisions. [8 Marks]

Solution:

1. **Absorption**
   - A brick is taken and it is weighed dry. It is then immersed in water for a period of 16 hours.
   - Then weigh again and the difference in weights should not, in any case, exceed
     (a) 20 per cent of weight of dry brick for first class bricks.
     (b) 22.5 per cent for second class bricks.
     (c) 25 per cent for third class bricks.

2. **Crushing Strength**
   - Minimum crushing strength for first class bricks $\nless$ 10 N/mm^2 and for second class bricks $\nless$ 7.5 N/mm^2.

3. **Hardness**
   - In this test, a scratch is made on brick surface with the help of a finger or nail. If no impression is left on the surface, brick is treated to be sufficiently hard.

4. **Presence of Soluble Salts**
   - Soluble salts, if present in bricks, will cause efflorescence on the surface of bricks.
   - It is immersed in water for 24 hours. It is then taken out and allowed to dry in shade. Absence of grey or white deposits on its surface indicates absence of soluble salts.
   - If the white deposits cover about 10% surface, the efflorescence is said to be slight.
   - When white deposit cover about 50% of surface then it is said to be moderate.
   - If grey or white deposits are found on more than 50% of surface, the efflorescence becomes heavy and it is treated as serious.

5. **Shape and Size**
   - Its shape should be truly rectangular with sharp edges.
   - 20 bricks are randomly selected of standard size (19 × 9 × 9 cm). For good quality bricks, the results should be within the following permissible limits:

Length	–	368 cm to 392 cm
Width	–	174 cm to 186 cm
Height	–	174 to 186 cm

6. **Soundness**
   - In this test, two bricks are taken and they are struck with each other.
   - Bricks should not break and a clear ringing sound should be produced.

7. Structure

- It should be homogeneous, compact and free from any defects such as holes, lumps, etc.
- High duty fireclays can resist temperature range of 1482°C to 1648°C; medium duty fireclays can resist temperature range of 1315°C to 1482°C and low duty fireclays can resist temperature up to 870°C only.

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.3(a) (ii) of Test-12
- **Theory Book:** Construction Material (Page No. 85)
- **Conventional Practice Question Book:** (Q.35, Page 26)
- **MADE EASY Classnotes**

* Qualities of good brick or testing of brick.
① Bricks must be atleast table moulded, free from voids and posses uniform bright colour.
② Bricks must be of uniform shape and size. (dimension test)
③ Bricks must produce clear ringing sound when struck against each other. (soundness test)
④ Brick must posses uniform structure across any section free from voids (structure test)
⑤ Bricks must not absorb more than 20% of water by weight when immersed in water for 24 hours in case of first class brick and not more than 22% in case of second class brick. (absorption test)

⑥ Brick must not show any sign of stains over the surface when immersed in water for 24 hours. (alkali test)
⑦ Brick must posses min comp strength of 3.5 N/mm². (strength test)
⑧ Brick must not break into pieces when dropped over the hard surface from height of 1meter. (toughness test)
⑨ Brick must not show any sign of indentation mark when scratched with finger nail. (Hardness test)

End of Solution

1. (a) (ii) Explain the products of hydration of C_3S and C_2S (Bogues compounds) giving the relevant equations involving the reactions. [4 Marks]

Solution:

- The chemical reactions that take place between cement and water is referred to as hydration of cement.
- The hydration of cement can be visualized in two ways viz. "through solution" and "solid state" type of mechanisms

- The reaction of cement with water is exothermic i.e. it liberates a considerable quantity of heat and this liberated heat is called as **heat of hydration**.
- The hydration process is not an instantaneous one. The reaction is faster in the early periods and continues indefinitely at a decreasing rate.
- During hydration, C_3S and C_2S react with water and calcium silicate hydrate (C-S-H) is formed along with calcium hydroxide $[Ca(OH)_2]$.
- Calcium silicate hydrate is the most important product of hydration and it determines the good properties of concrete.
- $2(3CaO \cdot SiO_2) + 6H_2O \rightarrow 3CaO \cdot 2SiO_2 \cdot 2H_2O + 3Ca(OH)_2$

  or it can be written as:

$$2C_3S + 6H \rightarrow C_2S_2H_3 + 3Ca(OH)_2$$
$$100 + 24 \rightarrow 75 + 49$$

The corresponding weights involved are

Similarly,

$$2(2CaO \cdot SiO_2) + 4H_2O \rightarrow 3CaO \cdot 2SiO_2 \cdot 3H_2O + Ca(OH)_2$$

or it can be written as:

$$2C_3S + 6H \rightarrow C_3S_2H_3 + Ca(OH)_2$$
$$100 + 21 \rightarrow 99 + 22$$

The corresponding weights involved are:

$$2C_3S + 6H \longrightarrow C_3S_2H_3 + 3Ca(OH)_2$$
$$2C_2S + 4H \longrightarrow C_3S_2H_3 + Ca(OH)_2$$

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.3(a) of Test-10
- **Theory Book:** Construction Material (Page No. 9)

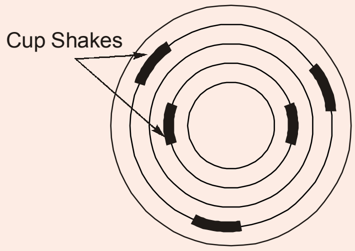
End of Solution

1. (b) (i) Explain the following defects in timber with neat sketches:
(A) Shakes (B) Knots [4 Marks]

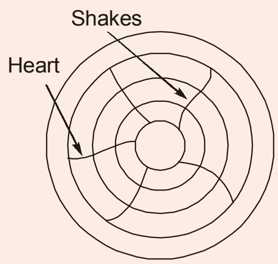
Solution:

Shakes: These are cracks which partly or completely separate the fibres of wood. Following are the different varieties of shakes:

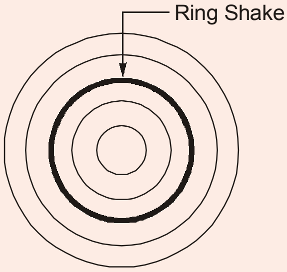
(a) **Cup Shakes:** These are caused by the rupture of tissue in a circular direction as shown in **Fig.** It is a curved crack and it separates partly one annual ring from the other. It develops due to non-uniform growth. It may not prove to be harmful, if it covers only a portion of ring.



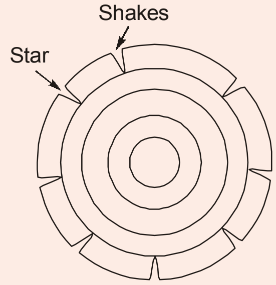
(b) **Heart Shakes:** These cracks occur in the centre of cross-section of tree and they extend from pith to sap wood in the direction of medullary rays as shown in **Fig.** These cracks occur due to shrinkage of interior part of tree which is approaching maturity. Heart shakes divide the tree cross-section into two to four parts.



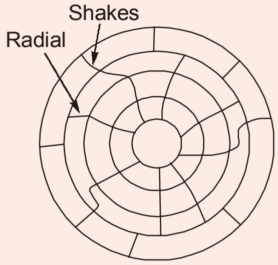
(c) **Ring Shakes:** When cup shakes cover the entire ring, they are known as ring shakes as shown in **Fig.**



(d) **Star Shakes:** These are cracks which extend from bark towards the sap wood. They are usually confined up to the plane of sap wood. They are usually formed due to extreme heat or frost.

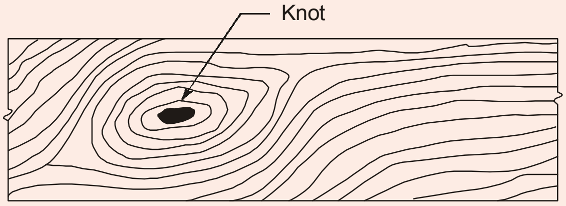


(e) **Radial Shakes:** These are similar to star shakes. But they are fine, irregular and numerous. They usually occur when tree is exposed to sun for seasoning after being felled down.



Knots: These are base of branches or limbs which are broken or cut off from the tree.

- The portion from which the branch is removed receives nourishment from the stem for a pretty long time and it ultimately results in the formation of dark hard rings which are known as the knots.



- These are caused by wood limb encased by wood of tree trunk.
- Caused when branch base embedded in timber by natural growth.

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.2(c) of Test-9
- **Theory Book:** Construction Material (Page No. 102)
- **MADE EASY Classnotes**

② Shakes

Development of cracks in the timber s/c which partially or completely seperate the fibres in it are termed as shakes.

③ Knot

• If the branch of the tree falls out, the portion from which it falls continues to receive the nourishment upto an extent a dark rot is formed termed as knot which reduces the strength of the section by reducing the availability of the area to transfer the load.

End of Solution

1. (b) (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 $\bar{E}$ for steel as 2×10^5 N/mm^2 and for brass as 1×10^5 N/mm^2.

[8 Marks]

Solution:

For **steel tube**

Given, Internal diameter, $D_{is} = 150$ mm

External diameter, $D_{os} = 170$ mm

For **brass tube**

Internal diameter, $D_{iB} = 170$ mm

External diameter, $D_{oB} = 190$ mm

Also, $L_B \Rightarrow L_S = 140$ mm

Axial load, $P = 1000$ kN

$E_{steel} = 2 \times 10^5$ N/mm^2

and $E_{brass} = 1 \times 10^5$ N/mm^2

Let us assume

The load carried by steel tube = P_s

And the load carried by brass tube = P_b

Using compatibility equation,

$$\Delta_{steel} = \Delta_{brass}$$

$$\frac{P_s L_s}{A_s E_s} = \frac{P_b \times L_b}{A_b E_b} \qquad (\because L_S = LB)$$

$$\frac{P_s \times 1000}{\frac{\pi}{4}(170^2 - 150^2) \times 2 \times 10^5} = \frac{P_b \times 1000}{\frac{\pi}{4}(190^2 - 170^2) \times 10 \times 10^5}$$

$$\frac{P_b}{(190^2 - 170^2)} = \frac{P_s}{(170^2 - 150^2) \times 2}$$

$$P_s = 1.778\, P_b$$

Also,

$$P_s + P_b = 1000 \text{ kN}$$

$$1.778\, P_b + P_b = 1000$$

$$P_b = 360 \text{ kN}$$

$$P_s = 640 \text{ kN}$$

So,

$$\sigma_s = \frac{640 \times 1000}{\frac{\pi}{4}(170^2 - 150^2)} = 127.3239 \simeq 127.32 \text{ N/mm}^2$$

$$\sigma_b = \frac{360 \times 1000}{\frac{\pi}{4}(190^2 - 170^2)} = 63.66 \text{ N/mm}^2$$

Shortening, i.e.,

$$\Delta_b = \Delta_s = \frac{P_s L_s}{A_s E_s}$$

$$= \frac{640 \times 1000 \times 140}{\frac{\pi}{4}(170^2 - 150^2) \times 2 \times 10^5}$$

$$= 0.089 \text{ mm}$$

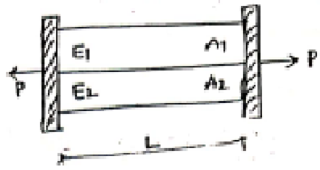
MADE EASY Source

- **Theory Book:** Strengthof Materials (Example 2.26, Page No. 47)
- **MADE EASY Classnotes**

Axial Deflection in composite section:

$P = P_1 + P_2$ — (1)

$\Delta_1 = \Delta_2$ [Composite section]



$$\frac{P_1 L}{A_1 E_1} = \frac{P_2 L}{A_2 E_2}$$

$$P_2 = P_1 \frac{A_2 E_2}{A_1 E_1}$$

$$P = P_1 + P_2 \Rightarrow P_1 + P_1 \frac{A_2 E_2}{A_1 E_1} = P_1\left[1 + \frac{A_2 E_2}{A_1 E_1}\right] = P_1\left[\frac{A_1 E_1 + A_2 E_2}{A_1 E_1}\right]$$

$$\boxed{P_1 = \frac{P\, A_1 E_1}{A_1 E_1 + A_2 E_2}} \qquad \boxed{P_2 = \frac{P\, A_2 E_2}{A_1 E_1 + A_2 E_2}}$$

$$\Delta = \Delta_1 = \frac{P_1 L}{A_1 E_1} \Rightarrow \frac{(P\, A_1 E_1)}{(A_1 E_1 + A_2 E_2)} \times \frac{L}{A_1 E_1}$$

$$\boxed{\Delta = \frac{PL}{A_1 E_1 + A_2 E_2}}$$

If this composite section replaced by one material having same length, load, area and deflection, then calculate Eequivalent.

$$\Delta' = \frac{PL}{(A_1 + A_2) E_{eqi}}$$

$\Delta' = \Delta$

$$\frac{PL}{(A_1 E_1 + A_2 E_2)} = \frac{PL}{(A_1 + A_2) E_{eq}}$$

$$\boxed{E_{eq} = \frac{E_1 A_1 + A_2 E_2}{A_1 + A_2}}$$

End of Solution

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1. (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 Marks]

Solution:

Diameter of bar, $d = 15$ mm

Gauge length, $l = 150$ mm

Let E be the Young's modulus of elasticity.

In Tensile test,

$$\text{Elongation} = \frac{PL}{AE}$$

$$0.063 = \frac{15 \times 1000 \times 150}{\frac{\pi}{4} \times 15^2 \times E}$$

$$E = 202101.515 \text{ MPa}$$

In Torsion test,

Let G be the modulus of rigidity

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

$$\frac{0.15°}{360} \times 2\pi = \frac{6.94 \times 1000 \times 150}{G \times \frac{\pi}{32} \times 15^4}$$

$$G = 80005 \text{ MPa}$$

Now as we know, if μ is Poisson's ratio then

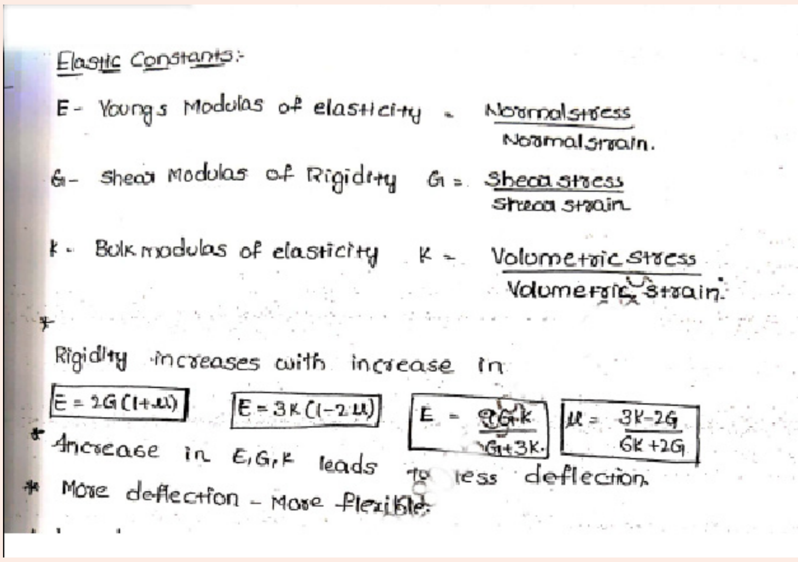
$$E = 2G(1 + \mu)$$

$$202101.515 = 2 \times 80005 (1 + \mu)$$

$$\mu = 0.263$$

MADE EASY Source

- **MADE EASY Classnotes**

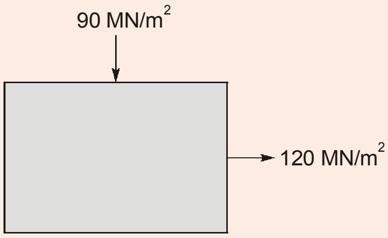


End of Solution

1. (c) (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 Marks]

Solution:



$$\sigma_1 = 120 \text{ MN/m}^2$$
$$\sigma_2 = -90 \text{ MN/m}^2$$

If $\quad \sigma_{P_1} \ngtr 150 \text{ MN/m}^2$

Then, $\quad \tau_{xy} = ?$

$$\sigma_{P_1} = \frac{\sigma_1+\sigma_2}{2} \pm \sqrt{\left(\frac{\sigma_1-\sigma_2}{2}\right)^2 + \tau_{xy}^2}$$

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

$$150 = 15 + \sqrt{105^2 + \tau_{xy}^2}$$

$$135^2 = 105^2 + \tau_{xy}^2$$

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

Other principle stress, $\quad \sigma_{P_2} = \frac{\sigma_1+\sigma_2}{2} - \sqrt{\left(\frac{\sigma_1-\sigma_2}{2}\right)^2 + \tau_{xy}^2}$

$$\sigma_{P_2} = \frac{120-90}{2} - \sqrt{\left(\frac{120+90}{2}\right)^2 + 84.85^2}$$

$$\sigma_{P_2} = 15 - \sqrt{105^2 + 84.85^2}$$
$$= -119.998 \text{ MN/m}^2 \simeq -120 \text{ MN/m}^2$$

Maximum shear stress $= \dfrac{\sigma_{P_1} - \sigma_2}{2}$

$$\tau_{max} = \frac{150-(-120)}{2} = 135 \text{ MN/m}^2$$

Orientation with plane,

$$\tan 2\theta_{P_1} = \frac{2\tau_{xy}}{\sigma_1 - \sigma_2} = \frac{2 \times 84.85}{120-(-90)}$$

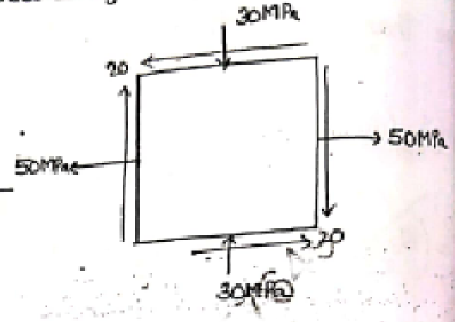
$\Rightarrow \quad 2\theta_{P_1} = 38.94°$

$$\theta_{P_1} = 19.47°$$

MADE EASY Source

- **Theory Book:** Strength of Materials (Examle 7.9, Page No. 282)
- **MADE EASY Classnotes**

A stress element shown, determine magnitude and location of principal plane, show all stresses by complete sketch



$\sigma_x = 50 \quad \sigma_y = -30 \quad \tau_{xy} = -20$

$\sigma_{P_1}/\sigma_{P_2} = \frac{\sigma_x+\sigma_y}{2} \pm \frac{1}{2}\sqrt{(\sigma_y-\sigma_x)^2+4(-20)^2}$

$= \frac{50-30}{2} \pm \frac{1}{2}\sqrt{(-30-50)^2+4(-20)^2}$

$\Rightarrow \sigma_{P_1} = 54.72$ MPa

$\sigma_{P_2} = -34.72$ MPa

$\sigma_{P_3} = 0$

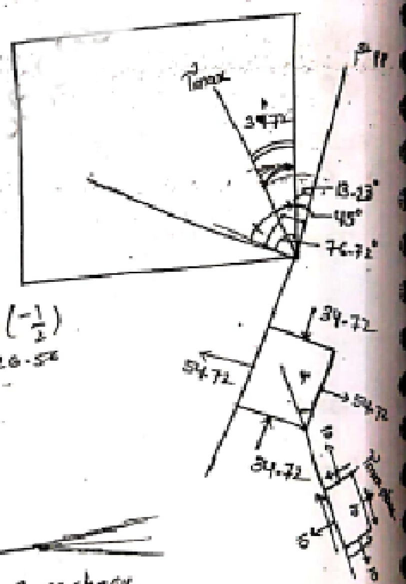
$\tan 2\theta_{P_1} = \frac{2(-20)}{\sigma_x-\sigma_y} = \frac{-40}{+80}$

$2\theta_{P_1} = \tan^{-1}\left(-\frac{1}{2}\right)$

$= 26.56$

$\theta_{P_1} = -13°28'$

$\theta_{P_2} = 90-13°28' = 76.72°$



$(\tau_{max})_{ab} = \frac{54.72-(-34.72)}{2}$

$= 44.72$ MPa

$\tan 2\theta_{\tau_1} = \frac{-1}{\frac{2(-20)}{50-(-30)}}$

$\theta_{\tau_1} = 31.72$

Cross check

$\theta = -13°28'$

$\sigma_x' = \sigma_x\cos^2\theta + \sigma_y\sin^2\theta + \tau_{xy}\sin 2\theta$

$\sigma_x' = 54.72$ MPa

$\theta_1 = 31.72 \quad \theta_2 = 121.72$

$(\tau_{x'y'})_{\theta_1} = -44.72 \quad (\tau_{x'y'})_{121.72} = +44.72$

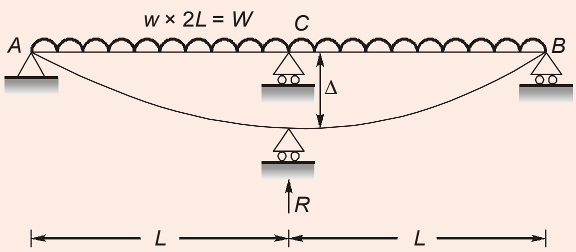
$\tau_{x'y'} = (\sigma_y - \sigma_x)\sin\theta\cos\theta + \tau_{xy}$ (calculating)

End of Solution

1. (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 Marks]

Solution:



Prop at C sinks by amount Δ

If reaction developed at C is R then as per question

$$\Delta = \lambda R$$

$$w = \frac{W}{2L}$$

Now using compatibility condition

$$\frac{5}{384}\frac{w(2l)^4}{EI} - \frac{R(2l)^3}{48EI} = \Delta$$

$$\frac{5}{384}\frac{(W/2L)\times 16L^4}{EI} - \frac{8RL^3}{48EI} = \lambda R$$

$$\frac{5}{48}\frac{WL^3}{EI} = R\left[\lambda + \frac{L^3}{6EI}\right]$$

$$R = \frac{\frac{5}{48}\frac{WL^3}{EI}}{\left[\lambda + \frac{L^3}{6EI}\right]} = \frac{5WL^3}{8\left[L^3 + 6\lambda EI\right]}$$

End of Solution

1. (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 Marks]

Solution:

Given,

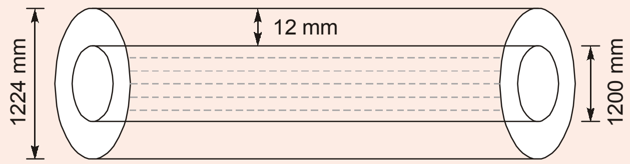
Internal diameter of water main = 1200 mm

Thickness, t = 12 mm

Given, bending stress is not to exceed 56 MPa

Unit weight of steel = 76.8 kN/m^3

Unit weight of water = 10 kN/m^3



Weight of water inside water main $= 10 \times \frac{\pi}{4} \times 1.2^2$

$= 11.3097$ kN/m $\simeq 11.31$ kN/m

Weight of steel tube $= 76.8 \times \frac{\pi}{4} \times (1.224^2 - 1.2^2)$

$= 3.509$ kN/m $\simeq 3.51$ kN/m

Total uniformly distributed load,

w = 11.31 + 3.51 = 14.82 kN/m

Max. BM, $M_{max} = \frac{wl^2}{8} = \frac{14.82 \times l^2}{8}$ kN/m $= 1.8525\, l^2$ kNm

Now, using bending equation,

$$\frac{M}{I} = \frac{\sigma}{y}$$

Here, $I = \frac{\pi}{64}(1224^4 - 1200^4) = 8.39 \times 10^9 \text{ mm}^4$

By putting the values, $\sigma = \frac{M}{I} \times y$

$$56 \text{ N/mm}^2 = \frac{1.8525 l^2 \times 10^6 \text{ Nmm} \times 606\text{mm}}{8.39 \times 10^9 \text{ mm}^4}$$

$l = 20.457$ m

So, the maximum possible length of water main should be 20.457 m so that the bending stress is not exceed 56 MPa.

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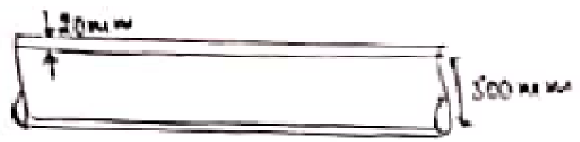
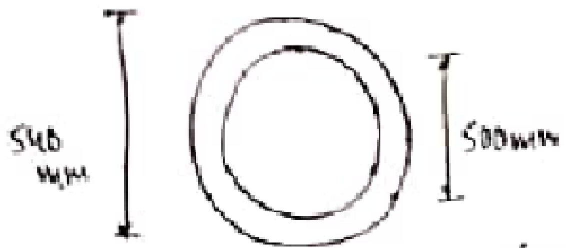
- **MADE EASY Classnotes**



span = 10m

$\rho_{ci} = 70.6\ kN/m^3$

$\rho_w = 9.8\ kN/m^3$



$W_{ci} = \rho_{ci} \cdot A_{ci} = 70.6 \times \left[\frac{\pi}{4}(540^2 - 500^2)\right] \times 10^{-6}$

$W_{ci} = 2.3044\ kN/m$

$W_w = 9.8 \times \frac{\pi}{4} \times 500^2$

$W_w = 1.924\ kN/m$

$w = W_w + W_{ci} = 4.23\ kN/m$

$M = \frac{wl^2}{8} = \sigma_{max} Z$

$\Rightarrow \frac{4.23 \times 10^2}{8} = \sigma_{max} \frac{\left[\frac{\pi}{64}(540^4 - 500^4)\right]}{\frac{540}{2}}$

$\Rightarrow \sigma_{max} = 12.91\ MPa$

End of Solution

2. (a) (i) How is the presence of surface oxide film responsible for excellent corrosion resistance of Aluminium?

[4 Marks]

Solution:

Resistance to Corrosion: Aluminium is inherently corrosion-resistant. Like stainless steel, aluminium alloys can also be rendered corrosion-resistant by the formation of a protective oxide film. However, in the case of aluminium it is the oxide of the base metal itself that has this characteristic. A transparent layer of aluminium oxide forms on the surface of aluminium almost immediately upon exposure to the atmosphere. Colour can be introduced to this oxide film by the anodising process, which can also be used to develop a thicker protective layer than one that would occur naturally.

Carbon steel, on the other hand, has a tendency to self-destruct over time by virtue of the continual conversion of the base metal to iron oxide, commonly known as rust. Thus, carbon steels require coatings or painting which also have to be maintained and periodically replaced.

Furthermore, aluminium is often used without any finish coating or painting. The cost of the initial painting alone may result in steel being more expensive than aluminium, depending on the quality of coating that is specified. In addition to the direct cost of painting, increasing environmental and worker safety concerns are associated with painting and paint preparation practices. The costs of maintaining steel, then, give aluminium a further advantage in life-cycle cost.

End of Solution

2. (a) (ii) What are the various factors that promote the Alkali Aggregate Reaction? How can this be controlled?

[8 Marks]

Solution:

Alkali aggregate reaction (AAR) is a chemical reaction of alkali in concrete and certain alkaline reactive minerals i.e., aggregate producing a hygroscopic gel which, when moisture present, absorbs water and expand. This gel expansion causes cracking in the concrete.

Factors promoting alkali-aggregate reaction are:

(i) **Reactive type aggregate:** The petrographic examination of thin rock sections may also immensely help to asses the potential reactivity of the aggregate. This test often requires to be supplemented by other tests.

(ii) **High alkali content in cement:** The high alkali content in cement is one of the most important factors contributing to the alkali-aggregate reaction.

(iii) **Availability of moisture:** Progress of chemical reactions involving alkali-aggregate reaction in concrete requires the presence of water. It has been seen in the field and laboratory that lack of water greatly reduces this kind of deterioration.

(iv) **Optimum temperature conditions:** The ideal temperature for the promotion of alkali-aggregate reaction is in the range of 10 to 38°C. If the temperature condition is more than or less than the above, it may not provide an ideal situation for the alkali-aggregate reaction.

MADE EASY Source

- **Conventional Practice Question Book:** (Q.47, Page 35)

End of Solution

2. (a) (iii) Describe the thermal and electrical properties of ceramics.

[8 Marks]

Solution:

Thermal properties of ceramic: Thermal capacity, conductivity and resistance to shocks need to be considered while using a ceramic.

The specific heat for refractories to be used in regenerator chambers should be more. The heat for refractories to be used in regenerator chamber should be more. The heat in ceramics is conducted by photon conductivity and by the interaction of lattice vibration. The ceramic materials do not have enough free electrons to bring out electronic thermal conductivity. At high temperatures, conduction takes place by transfer of radian energy. The thermal conductivity of refractories should be minimum for lining and maximum for crucibles and retorts.

Electrical Properties of ceramic: Ceramic materials have no free electrons so they have low electrical conductivity. However, at high temperatures the ionic diffusion is accelerated.

Clay displays a very high dielectric constant – a property of material related to its behaviour when located within an electric field between two electrodes – under static conditions. However, for alternating current, the dielectric constant in clay arises from ion and electron movement.

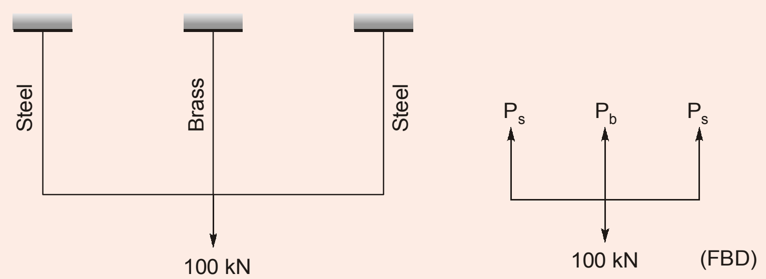
End of Solution

2. (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. $\alpha_S = 11 \times 10^{-6}/°C$; $\alpha_\beta = 18 \times 10^{-6}/°C$

[12 Marks]

Solution:



Given, the temperature rise = 60°C

The area of bars, $A_s = A_b = 500\ mm^2$

And $E_s = 2E_b = 210$ GPa

Assuming the length of the bars to be equal.

Also, $2P_s + P_b = 100$ kN ...(i)

Using compatibility equation,

$$\Delta_s = \Delta_b$$

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

$$\frac{P_s \times 1000}{500 \times 2.1 \times 10^5} - 11 \times 10^{-6} \times 60 = \frac{P_b \times 1000}{500 \times 1.05 \times 10^5} + 18 \times 10^{-6} \times 60 \quad [\because L_s = L_b]$$

After solving,

$$9.5238 \times 10^6\ P_s - 1.09047 \times 10^{-5}\ P_b = -0.42 \times 10^{-3}$$

$$9.5238\ P_s - 10.9047\ P_b = -420 \quad ...(ii)$$

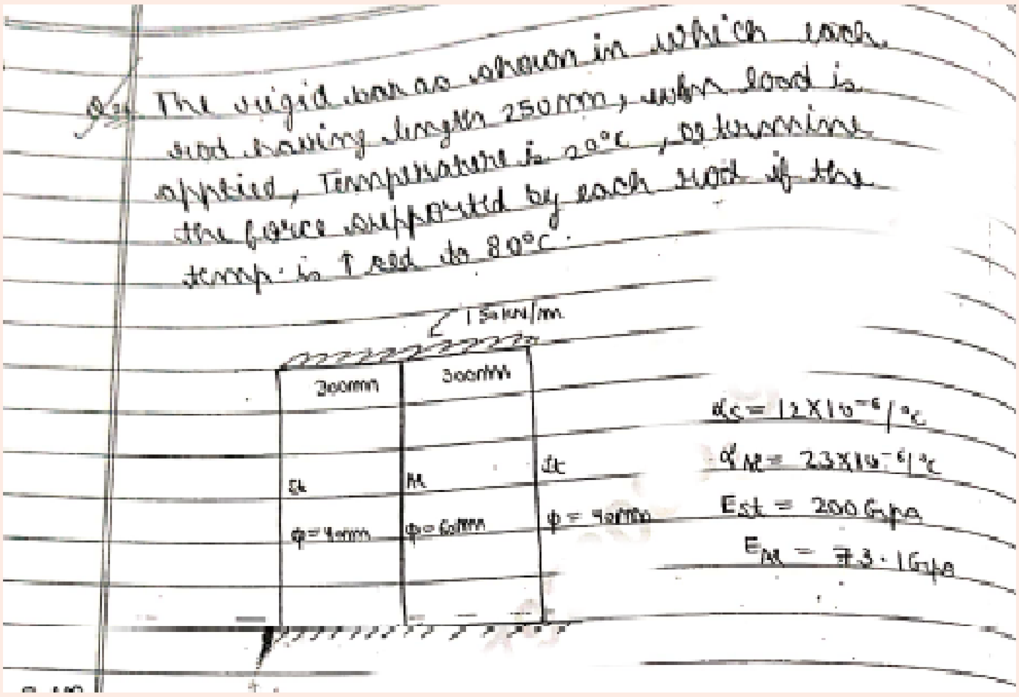
Solving equation (i) and (ii)

We get, $P_s = 48.82$ kN (Tension)

$P_b = -2.36$ kN (Tension)

MADE EASY Source

- **Theory Book:** Strength of Materials (Example 2.39, Page No. 64-65)
- **MADE EASY Classnotes**



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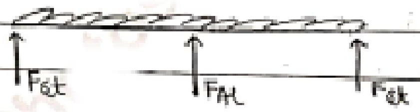


$$150 \times 0.6 = 2F_1 + F_2 \quad \text{—(1)}$$

$$\delta_{st} = \delta_{Al}$$

$$(L\alpha_s \Delta T)_s + \left[\frac{-F_1 L}{AE}\right] = (L\alpha_{Al} \Delta T) + \left[\frac{-F_2 L}{AE}\right]$$

$$\left[250 \times 12 \times 10^{-6} \times (80 - 20)\right] + \left[\frac{-F_1 \times 250}{\frac{\pi}{4} \times 40^2 \times E_{st}}\right] = \left[250 \times 23 \times 10^{-6} \times 60\right] + \left[-F_2 \times 250 \ldots\right]$$

$F_1 = -16.39$ kN → Tension (Steel) {as we assumed c}

$F_2 = 122.78$ kN → Aluminium

compression

End of Solution

2. (b) (ii) A solid steel shaft has to transmit 75 kW at 200 rpm. Taking allowable shear stress as 70 N/mm^2, find suitable diameter for the shaft, if the maximum torque transmitted at each revolution exceeds the mean by 30%.

[8 Marks]

Solution:

The power transmitted, $P = 75$ kW

Speed, $N = 200$ rpm

Allowable shear stress, $\tau = 70$ N/mm^2

Using equation, $$P = \frac{2\pi NT}{60}$$

$$75 = \frac{2\pi \times 200 \times T}{60}$$

$$T = 3.581 \text{ kNm}$$

If the maximum torque transmitted at each revolution exceeds the mean by 30%.

So, the modified torque, $T' = 1.3\ T$

$$T' = 1.3 \times 3.581$$

$$T' = 4.6553 \text{ kNm}$$

Using torque equation,

$$\frac{\tau}{R} = \frac{T}{I_P}$$

$$\tau = \frac{T}{I_P} \times R = \frac{T}{Z_p}$$

$$70 = \frac{4.6553 \times 10^6}{\frac{\pi D^3}{16}}$$

We get, $D = 69.706$ mm

So, the diameter of solid shaft,

$$D = 69.706 \text{ mm}$$

MADE EASY Source

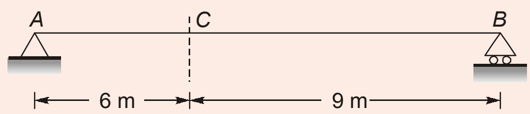
- **Theory Book:** Strength of Materials (Example 8.9, Page No. 348)

End of Solution

2. (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.

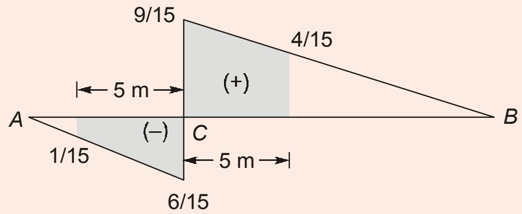
[20 Marks]

Solution:



ILD for shear force at C

Using Muller Breslau's principle, ILD for shear at C is given as



For max (+) SF at 'C', load should be placed at section CB

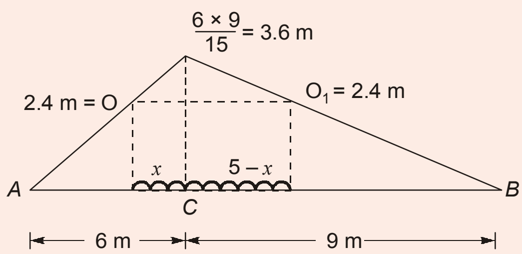
$$(+)\ SF_{max} = \left[\frac{\frac{9}{15}+\frac{4}{16}}{2}\right]\times 5\times 40 = 86.67\text{ kN}$$

For max (–) SF at 'C', load should be placed at section AC

$$(-)\ SF_{max} = \left[\frac{\frac{1}{15}+\frac{6}{16}}{2}\right]\times 5\times 40 = -46.67\text{ kN}$$

ILD for Bending moment at C

Using Muller Breslau's principle, ILD of BM is given as



For max BM, point C should divide load in same proportion as AC : CB

So, $\quad \frac{x}{5-x} = \frac{6}{9}$

$\Rightarrow \quad x = 2$ m

Ordinate at O and $O_1 = \frac{3.6}{6}\times 4 = 2.4$ m

So, $\quad \text{Max BM} = \left[2.4\times 5+(3.6-2.4)\times\frac{1}{2}\times 5\right]\times 40$

$BM_{max} = 600$ kNm

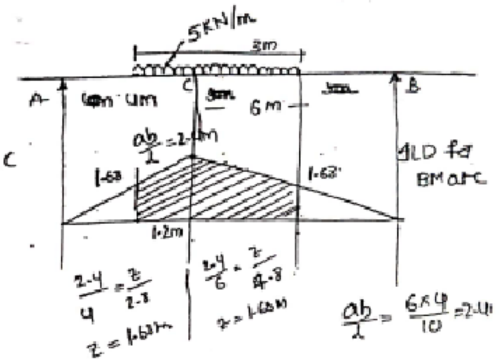
MADE EASY Source

- **Theory Book:** Structural Analysis (Example 2.9, Page No. 61-62)
- **MADE EASY Classnotes**

* A UDL of 5 kN/m and length 3 m is moving on a simply supported beam of span 10 m. Max BM at a distance of 4 m from left support is ____

To get Max BM at C.

Average load on AC = Average load on BC



$\frac{x}{a} = \frac{l-x}{b}$

$\Rightarrow \frac{x}{4} = \frac{3-x}{6} \Rightarrow x = 1.2\text{m}$

Max BM at C = Intensity of load × Area of ILD under load

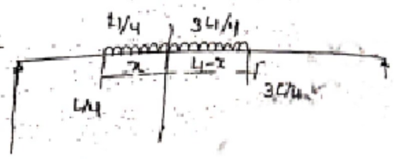
$= 5\left[(1.68\times3) + \frac{1}{2}\times3\times(2.4-1.68)\right]$

$= 30.36$ kNm

The Max BM at the left quarter point of simple beam due to crossing of UDL of length shorter than the span in crossing of left to right would occur after the load has crossed the section by.

Loaded length to get Max BM at C

Loaded length and corresponding span must be proportional



$\frac{x}{a} = \frac{L-x}{b} \Rightarrow \frac{x}{L/4} = \frac{L-x}{3L/4} \Rightarrow x(3L) = L\,L - xL$

$\Rightarrow 4Lx = L \Rightarrow x = \frac{L}{4}$

For Max BM at C, the $\frac{3}{4}$ of span of UDL must cross the quarter point of span.

End of Solution

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

[8 Marks]

Solution:

Testing of Lime

Visual inspection: (a) A sample of lime is examined for its colour and lumps. (b) Lumps of lime indicate quick lime of unburnt lime

Field Tests: The field tests usually performed on lime at site are as below:

- **Test for physical properties:** Hydraulic limes are brush grey, brown or dark coloured. Hydraulic limestones have a clayey teste and give out earthy smell. White colour of lime is an indication of pure variety of limestone. Shining particles on the surface of limestones indicate the presence of free salt.
- **Workability test:** A handful of mortar (lime sand mortar in the ratio 1 : 3) is thrown on the surface on which it is to be used. The area covered by the mortar and its quantity is recorded. These data indicate the workability of the lime mortar. It is a very crude field test performed with the actual mortar.
- **Impurity test:** A known weight of lime is mixed with water in a breaker. The solution is decanted. The residue is dried for 8 hours in the natural heat of sun and is then weighted. If the residue is less than 10 per cent of the weight of the lime taken initially it is considered to be of good quality. In case, if it is 10-20 percent, the lime is fat and if more than 20 per cent the lime is classed as poor.
- **Heat test:** Limestone is heated for four hours on open fire; CO_2 escapes and it loses weight. From this the carbonate percentage in the limestone can be worked out. Lumpy form indicate quick lime or unburnt limestone. A porous structure may indicate quick lime.
- **Hydraulic acid test:** The test is carried to know the classification and the carbonate content of lime.

  When a teaspoon full of lime is put in a test tube containing 10 ml of 50 per cent hydrochloric acid, effervescence takes place. Too much of effervescence indicates high percentage of calcium carbonate in limestone. The residue at the bottom of the tube indicates percentage of inert materials present in the limestone. Abundant of liberation of CO_2 indicates unburnt lime.

  In case of class-A lime, a good get is formed above the layer of inert material. A thick get indicate class-B lime and absence of get means class-C lime.

  For eminently hydraulic lime, the get formed is thick and does not flow. Absence of gel indicates non-hydraulic of fat lime. If get flows, it indicates feebly hydraulic lime.
- **Ball test:** Balls of stiff lime paste are made and left for six hours. They are placed in a basin of water. If expansion and disintegration of balls is observed, the lime is of type C. Little expansion and numerous cracks indicate it to be class-B lime. Class-A lime will have no adverse effect.

Physical Tests

- Sampling of testing lime should be done as quickly as possible so that the material does not deteriorate. From each lot, three test samples are taken for quick lime as well as for hydrated lime.

Fineness Test [IS : 6932 (Part IV)]

- The sieves are arranged one above the other with the coarser sieves at the top and the finer sieves at the bottom. Sieving is done with a gental wrist motion. 100 g of the hydrated lime is placed on the top sieve and is washed through the sieves with a moderate jet of water for not more than 30 minutes. The residue on each sieve is dried at 100 ± 10°C to constant mass and weighted. The result is expressed as a percentage of mass of hydrated lime taken.

Determination of Residue on Slaking of Quick Lime [IS : 6932 (Part III)]

- Sample of quick lime is sieved through 2.36 mm IS sieve and the residue, if any, is broken and sieved again until the whole quantity passes through the sieve. The quantity of water required for slaking is usually 4 times the mass of quick lime, however, it may be as high as 8 times for certain high calcium limes.

Workability Test [IS : 6932 (Part VIII)]

- The test is conducted on a standard flow table and a truncated conical mould. For testing hydrated lime, the lime putty is prepared by thoroughly mixing 500 g of hydrated lime with an equal mass of water at a temperature of 27 ± 2°C and kept for 24 hours. The soaked material is then thoroughly mixed and knocked up to produce a plastic putty, by passing the material twice through the mixer.

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. Here in this test lime putty is used instead of cement mortar.

Soundness Test [IS : 632 (Part IX)]

- The test is done to find the quality, i.e., the unsoundness or disintegration property of lime using the Le-chatelier apparatus.

Popping and Pitting Test [IS : 6932 (Part X)]

- To determine the soundness of fat lime, pats are prepared by mixing hydrated lime, Plaster of Paris and water. The pats are subjected to steam and then tested for disintegration, popping and pitting. If any of these occurs the lime is considered to be unsound.

End of Solution

3. (a) (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 Marks]

Solution:

Maturity, $M = \sum_{0}^{t}(T - T_0)\Delta t$

where, T = Average concrete temperature
T_0 = Datum temperature (–11°C)
Δt = Time interval

$$M = 7 \times 12\,[25 - (-11)] + 7 \times 12\,[15 - (-11)]$$
$$M = 5208\text{°C-hours}$$

Strength of maturity, $f = A + B\log_{10}(m \times 10^{-3})$

Where A and B are Plowman's coefficient

A = 32

and B = 54

$$f = 32 + 54\log_{10}\left[\frac{5208}{10^3}\right] = 70.7\%$$

Strength of fully matured concrete = 50 MPa

So strength of concrete after 7 day = 50 × 0.707 = 35.35 MPa

End of Solution

3. (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 Marks]

Solution:

According to maximum shear stress theory

$$\frac{\sigma_{P_1}}{2} \text{ or } \frac{\sigma_{P_2}}{2} \text{ or } \left(\frac{\sigma_{P_1} - \sigma_{P_2}}{2}\right) = \left(\frac{\sigma_y}{2\ \text{FOS}}\right) \quad \text{...(i)}$$

According to Distortion Energy theory

$$\frac{1}{2}\left[\left(\sigma_{P_1} - \sigma_{P_2}\right)^2 + \left(\sigma_{P_2} - \sigma_{P3}\right)^2 + \left(\sigma_{P_3} - \sigma_{P_1}\right)^2\right] = \left(\frac{\sigma_y}{\text{FOS}}\right)^2$$

For 2-dimensional system, $\sigma_{P_3} = 0$

$$\sigma_{P_1}{}^2 + \sigma_{P_2}{}^2 - \sigma_{P_1 P_2} = \left(\frac{\sigma_y}{FOS}\right)^2 \quad \text{...(ii)}$$

Now, put value of $\left(\frac{\sigma_y}{FOS}\right)$ from eq. (i)

$$\sigma_{P_1}^2 + \sigma_{P_2}^2 - \sigma_{P_1 P_2} = \left(\sigma_{P_1} - \sigma_{P_2}\right)^2$$

$$\sigma_{P_1}\sigma_{P_2} = 0 \qquad \text{(which is not possible)}$$

So taking another case in max shear stress theory

$$\left(\frac{\sigma_{P_1}}{2}\right) = \left(\frac{\sigma_y}{2FOS}\right) \qquad ...(iii)$$

From eq. (i) and (ii)

$$\sigma_{P_1}^2 + \sigma_{P_2}^2 - \sigma_{P_1}\sigma_{P_2} = \sigma_{P_1}^2$$

$$\sigma_{P_2}\left[\sigma_{P_2} - \sigma_{P_2}\right] = 0$$

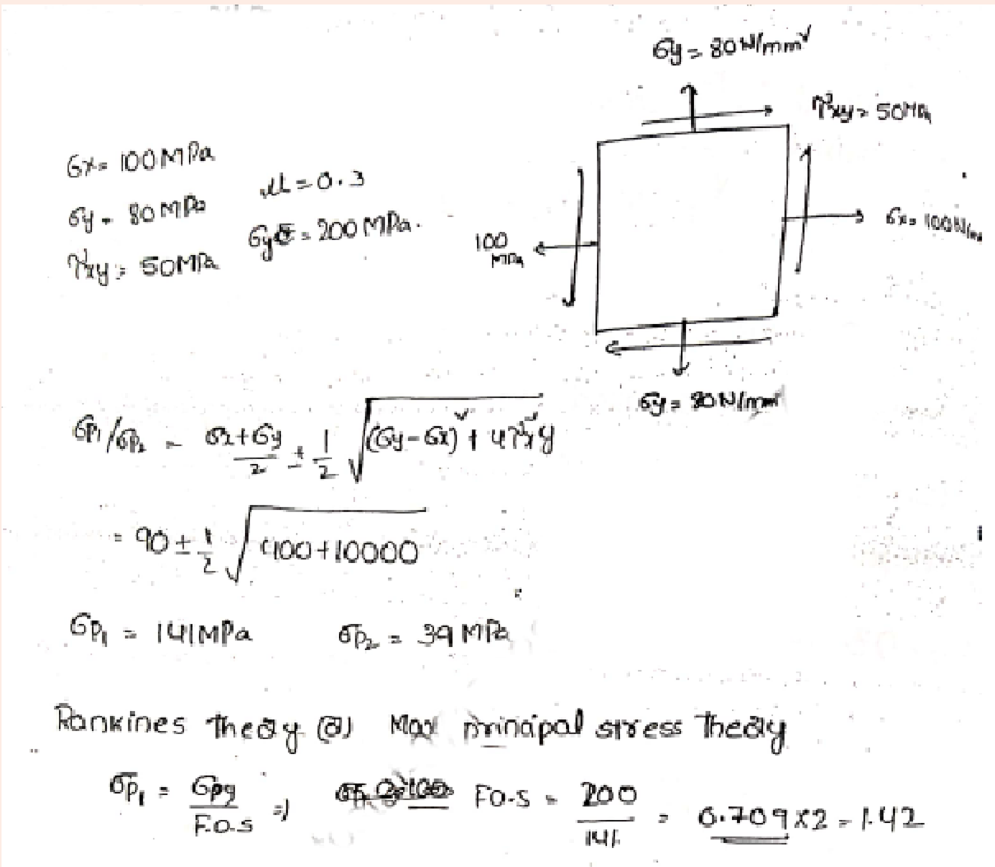
$$\sigma_{P_2} \neq 0$$

So, $\sigma_{P_2} = \sigma_{P_1}$

So both principle stresses must be equal.

MADE EASY Source

- **Theory Book:** Strength of Materials (Page No. 317)
- **MADE EASY Classnotes**


σx = 100 MPa
σy = 80 MPa
τxy = 50 MPa
μ = 0.3
σyt = 200 MPa
σy = 80 N/mm²
τxy = 50 MPa
σx = 100 N/mm²
100 MPa
σP1/σP2 = (σx+σy)/2 ± 1/2 √((σy−σx)² + 4τ²xy)
= 90 ± 1/2 √(400+10000)
σP1 = 141 MPa σP2 = 39 MPa
Rankines theory (or) Max principal stress theory
σP1 = σpy/F.o.S ⇒ F.o.S = 200/141 = 0.709×2 = 1.42

St-Venant's theory (or) Max. principal strain theory

$$\sigma_{P_1} - \mu \sigma_{P_2} = \frac{\sigma_y}{F.O.S}$$

$$\Rightarrow 141 - 0.3 \times 39 = \frac{200}{F.O.S} \quad \Rightarrow F.O.S = 1.546$$

Maximum shear stress theory or Tresca's theory

$$\frac{\sigma_{P_1} - \sigma_{P_3}}{2} = \frac{\sigma_y}{2\,F.O.S} \quad \Rightarrow \frac{141 - 0}{2} = \frac{\sigma_y}{2\,F.O.S}$$

$$\Rightarrow F.O.S = \underline{1.42}$$

Maximum strain energy theory. Haigh's theory

$$\left[\sigma_{P_1}^2 + \sigma_{P_2}^2 - 2\mu\,\sigma_{P_1}\sigma_{P_2}\right] = \left(\frac{\sigma_y}{F.O.S}\right)^2$$

$$141^2 + 39^2 - 2 \times 0.3 \times 141 \times 39 = \left(\frac{200}{F.O.S}\right)^2$$

$$F.O.S = \frac{200}{134.54} = \underline{1.486}$$

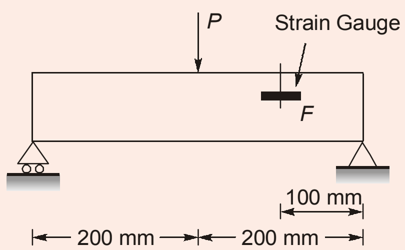
Maximum distortion theory (or) Von Moises theory
Distortion theory.

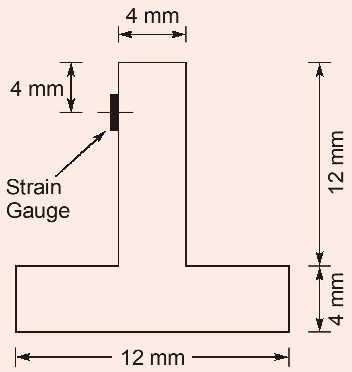
$$\left(\frac{\sigma_y}{F.O.S}\right)^2 = \left[\sigma_{P_1}^2 + \sigma_{P_2}^2 - \sigma_{P_1} \times \sigma_{P_2}\right]$$

$$F.O.S = \frac{200}{126} = \underline{1.586}$$

End of Solution

3. (b) (ii)

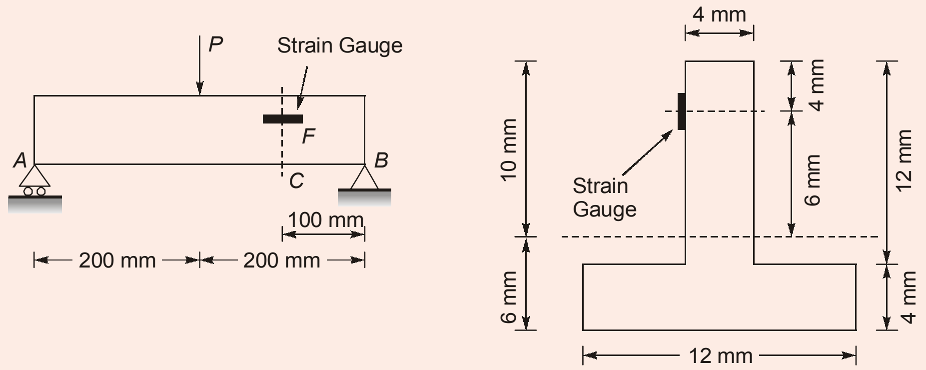




A small T-section is used in inverted position as a beam and is shown in figure over a span of 400 mm. If due to the application of forces shown, the longitudinal strain gauge at *F* registers a compressive strain of 1500 microstrains, determine the magnitude of *P*. Take E = 200 GPa.

[10 Marks]

Solution:



For CG of cross-section from bottom.

$$\bar{y} = \frac{12\times4\times2+12\times4\times10}{12\times4\times2} = 6 \text{ mm}$$

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

$$= 2176 \text{ mm}^4$$

Now as we know, $$\frac{M}{I} = \frac{E}{R}=\frac{\sigma}{y}$$

$$\text{Stain at F} = 1500\times10^{-6}$$

$$\text{Stress at F} = 1500\times10^{-6}\times E = 300 \text{ MPa}$$

$$\frac{M}{I} = \frac{\sigma}{y}$$

$\Rightarrow$ $$M = \frac{300\times2176}{6} = 108800 \text{ Nmm}$$

Now moment at cross-section 'C' will be

$$= \frac{P}{2}\times100 = 50P$$

$$50P = 108800$$

$$P = 2176 \text{ N}$$

$$P = 2.176 \text{ kN}$$

MADE EASY Source

- **Conventional Practice Question Book:** (Q.26, Page 83)

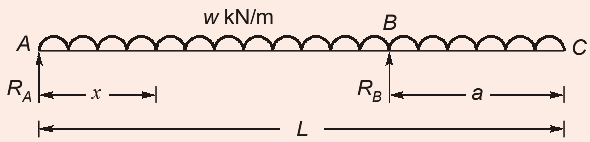
End of Solution

3. (c) A beam of span L carries a uniformly distributed load ω 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 Marks]

Solution:

For the given arrangement:



$$R_A + R_B = wL \quad \text{...(i)}$$

Taking moment about A,

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

$$R_B = \frac{wL^2}{2(L-1)}$$

From equation (i), $$R_A = \frac{wL(L-2a)}{2(L-a)}$$

Moment in span AB, $$M_x = \frac{wL(L-2a)}{2(L-a)} \times x - \frac{wx^2}{2}$$

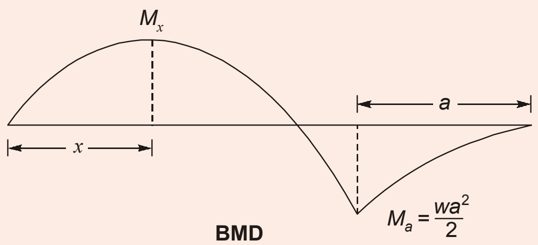
For maximum bending moment in span AB,

$$\frac{dM_x}{dx} = 0$$

$$\frac{wL(L-2a)}{2(L-a)} - \frac{2wx}{2} = 0$$

So, $$x = \frac{L(L-2a)}{2(L-a)}$$

The BMD of the beam will be



BMD

If the maximum bending moment for the beam is as small as possible.

So, the maximum +ve BM = Maximum –ve BM

i.e., $$M_x = M_a$$

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

After solving, $L^2 - 2aL = 2aL - 2a^2$

$a^2 - 2aL + 0.5\,L^2 = 0$

$a = 1.707\,L,\ 0.292\,L$

$a = 1.7072$ is not possible

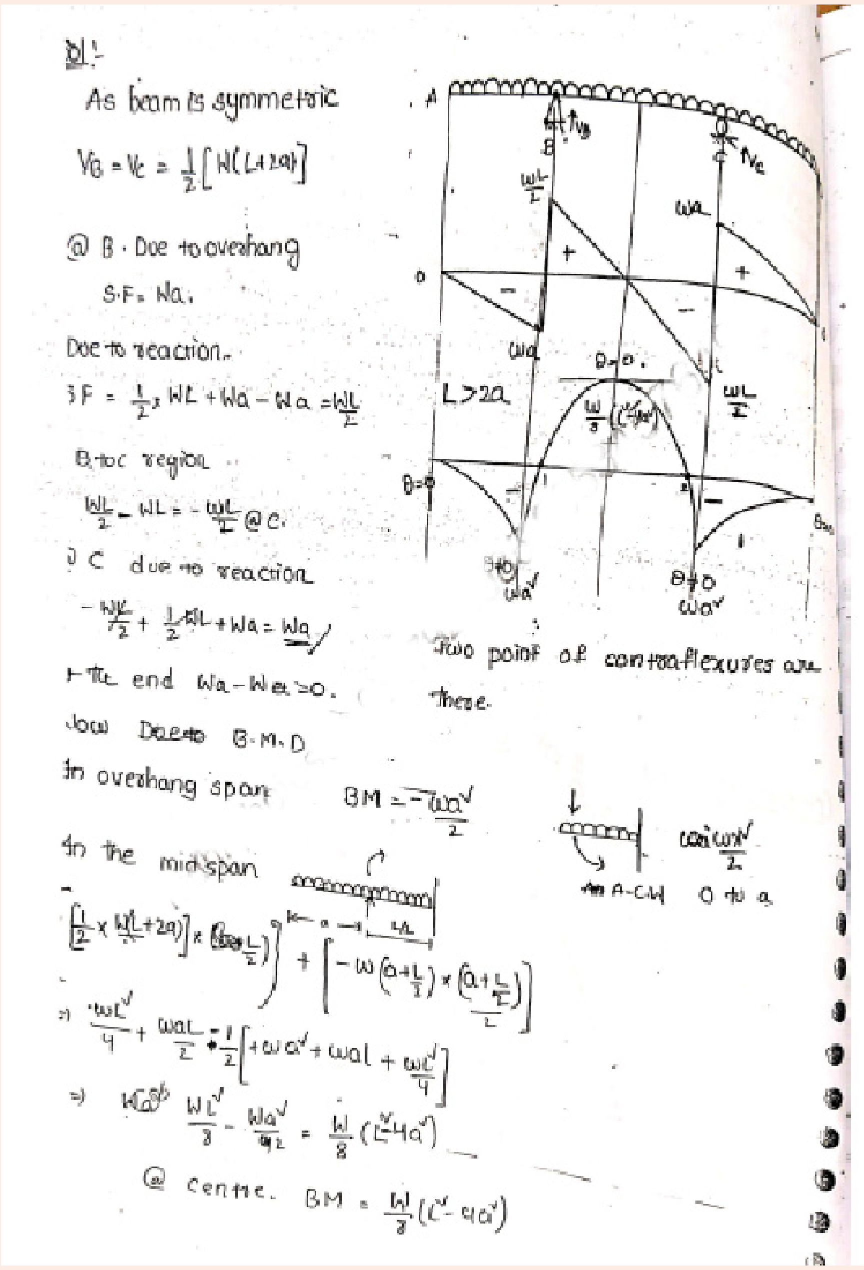
So, $a = 0.292\,L$

So, Maximum bending moment,

$$M_{max} = \frac{wa^2}{2} = \frac{w(0.292L)^2}{2} = 0.042\,wL^2$$

MADE EASY Source

- **MADE EASY Classnotes**



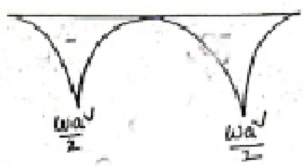
Take a section at C

$$BM = \left[\frac{1}{2} \times W(L+2a) \times L\right] - \left[\frac{W(a+L)(a+L)}{2}\right]$$

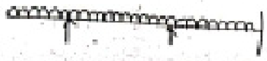
$$= \frac{WL^2}{2} + WaL - \left[\frac{Wa^2}{2} + \frac{2WaL}{2} + \frac{WL^2}{2}\right]$$

$$= -\frac{Wa^2}{2}$$

L = 2a



At end support D



BM =

$$\left[\left(W \times \frac{(L+2a)}{2}\right) \times (L+a) + \frac{W(L+2a)}{2} \times a\right]$$

$$- \left[\frac{W(L+2a) \times (L+2a)}{2}\right]$$

$$= 0$$

Left to right — Clockwise Positive, A.C.W −Ve → BMD ← Right to left — Clockwise −Ve, ACW +Ve

$$\% \text{ Overhang} = \frac{a}{L+2a} \times 100$$

≯ 25

If % overhang on each side 25% (L = 2a), B.M at centre will be zero.

When % overhang on each side < 25% [L > 2a], Two point of contraflexure is obtained.

When % overhang on each side more than 25% (L < 2a), B.M @ centre will be negative

End of Solution

4. (a) (i) Write briefly about the following:
(A) Air Entraining admixtures
(B) Role of Flyash as a part replacement of cement

[10 Marks]

Solution:

(A) **Air Entraining Admixture:** This cement is manufactured by mixing small quantity of air-entraining agent like alkali salts of natural wood resins, synthetic detergents of alkylaryl sulphate type, sodium salts of sulphonates, calcium lignosulphate, salts of fatty acids, etc., with ordinary Portland cement or Portland blast furnace slag cements. These agents in powder or in liquid forms are added to the extent of 0.025 to 0.100 per cent by weight of OPC cement clinker at the time of grinding. At the time of mixing concrete ingredients, these cements produce tiny, discrete noncoalescing air bubbles in the concrete mass which enhances workability and reduces tendency to segregation and bleeding. The air entrainment increases the frost and sulphate water resistance of concrete.

(B) **Role of Flyash as a part replacement of cement:** The flyash or Pulverised Fuel Ash (PFA) is the residue from the combustion of coal collected by the mechanical dust collectors or electrostatic preceptors or separators from the fuel gases of thermal power plants. Like Portland cement, flyash contains oxides of calcium, aluminium and silicon, but the amount of calcium oxide is considerably less. The properties of flyash depend on the type of coal burnt. In general, silicious flyash is pozzolanic, while calcareous flyash has latent hydraulic properties.

The pozzolanic activity is due to the presence of finely divided glassy silica and lime which produce calcium silicate hydrate as is produced in hydration of Portland cement. The carbon content is flyash should be as low as possible, whereas the silica content should be as high as possible.

The flyash obtained from Electrostatic Precipitators (ESP) is finer than the Portland cement. The most important mineral aspect of flyash is the presence of 60-90 per cent glassy spherical silicon dioxide (SiO_2). The flyash is generally used in the following three ways:

1. **As a part replacement of cement:** This simple replacement of Portland cement up to 60 per cent by mass reduces the strength at ages up to 3 months.
2. **As a simultaneous replacement of cement and fine aggregate:** This replacement enables the strength as a specified age to be equalled depending on the water content.
3. As a part of cement in the form of blended cement.

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.3(a) (i) of Test-12
- **Conventional Practice Question Book:** (Q.49, Page 38) & (Q.7, Page No. 7)
- **MADE EASY Classnotes**

(6) Air Entraining Admixture

- These are the type of admixtures which entraps millions of air bubbles in the voids of concrete which modifies property of the concrete in relation to its workability, durability, segregation, bleeding and frost action.
- commonly used air entraining admixture includes:
  1. Natural wood resin
  2. Plant and animal fatty acids
  3. Stearic acid
  4. Oelic acid
  5. Aluminium Powder
  6. Hydrogen Peroxide.

(all benefits except that strength decreases)

- These admixtures are now-a-days used as 5th ingredient for preparation of concrete.

End of Solution

4\. (a) (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 Marks]

Solution:

Volume of concrete = Vol. of air + Vol. of water + Vol. of solids of [Cement + Sand + Coarse aggregate]

$$1 = 0.02 + V_W + [V_{CS} + V_{SS} + V_{CAS}]$$

Assume volume of cement = V_C (dry volume)

So, Weight of cement = $V_C \times 1500$

$$\text{Volume of cement solids} = \frac{V_C \times 1500}{3.15 \times 1000}$$

Using same concept for sand of coarse aggregate

$$1 = 0.02 + \frac{V_C \times 1500 + 0.49}{1000} + \frac{V_C \times 1500}{3.15 \times 100}$$

$$+\frac{1.42 \times V_C \times 1700}{2.6 \times 100}+\frac{2.94 \times V_C \times 1600}{2.6 \times 1000}$$

$$V_C = 0.248 \text{ m}^3$$

$$V_{Sand} = 1.42 \times 0.248 = 0.352 \text{ m}^3$$

$$V_{CA} = 2.94 \times 0.248 = 0.729 \text{ m}^3$$

Mass of cement = 0.248 × 1500 = 372 kg

Mass of sand = 598.67 kg

Mass of coarse aggregate = 1166.59 kg

Mass of water = 372 × 0.49 = 182.28 kg

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.1(c) of Test-8
- **Theory Book:** Construction Material (Example 3.3, Page No. 72)
- **MADE EASY Classnotes**

Q The data of a material for making a cement concrete mix may be assume as follows.

water-cement ratio by mass = 0.46

Entrained air = 2%.

Serial No.	Material	Specific Gravity	Bulk density (kg/m³)	Proportion in the mix by dry volume
1	Cement	3.15	1500	1 part
2	Fine Agg.	2.61	1620	1.359 part
3	20 mm nominal size crushed coarse agg.	2.7	1530	2.79 part

Determine the following for cement concrete

(i) Absolute volume of fully compacted fresh concrete produced by 1 bag of cement (ignoring air content) of 50 kg.

(ii) Cement content per cubic meter of concrete (ignoring air content) produced by 1 bag of cement of 50 kg to make 1 cubic meter of concrete

(iii) Quantity of materials

(iii) C : S : G = 1 : 1.359 : 2.79 (By volume)

$$G = \frac{\rho_s}{\rho_w} = \frac{M_s}{V_s \rho_w} = \frac{M}{V_s \rho_w} = \frac{\rho V}{V_s \rho_w}$$

$$V_s = \frac{\rho V}{G \rho_w}$$

$V_{concrete} = V_{voids} + V_{solids}$

$$1 = \frac{2*1}{100} + \frac{M_w}{S_w} + \frac{S_c V_c}{G_c S_w} + \frac{S_s V_s}{G_s S_w} + \frac{S_a V_a}{G_a S_w}$$

$$0.98 = \frac{0.46 S_c V_c}{S_w} + \frac{S_c V_c}{G_c S_w} + \frac{S_s * 1.359 V_c}{G_s S_w} + \frac{S_a * 2.79 V_c}{G_a S_w}$$

$V_c = 0.2729\ m^3 \rightarrow M_c = 409.35\ kg\ (0.2729 \times 1500)$

$V_s = 0.3708\ m^3 \Rightarrow M_s = 600.7\ kg$

$V_a = 0.761\ m^3 \Rightarrow M_a = 1164.33\ kg$

$\Rightarrow M_w = 188.3\ kg$

(i) Volume of concrete for 409.35 kg of cement = 1 m³

" " " " " " (excluding air) = 0.98 m³.

" " " " 50 kg of cement (excluding air) $= \frac{0.98}{409.35} \times 50 = 0.1197\ m^3$

(ii) Cement content (excluding air) $= \frac{\frac{50}{1500}}{0.1197} \times 100 = 27.84\ \%$

End of Solution

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

[8 Marks]

Solution:

Acceptance Criteria for Concrete

As per 16, 1(a) / IS 456 / 2000 Page 30

1. **For all concrete: M15 grade and above**

   The average strength of four (4) non overlapping consecutive test result shall be not less than

   For M15 or higher:

$$f_{average} \leq (f_{ck} + 0.825\ \sigma)\ \text{N/mm}^2$$

(Rounded off to 0.5 N/mm²)

or $(f_{ck} + 3)\ \text{N/mm}^2$

(whichever is more)

2. As per 16, 1(b) / IS 456 / 2000 Page 30

Individual test results of any sample

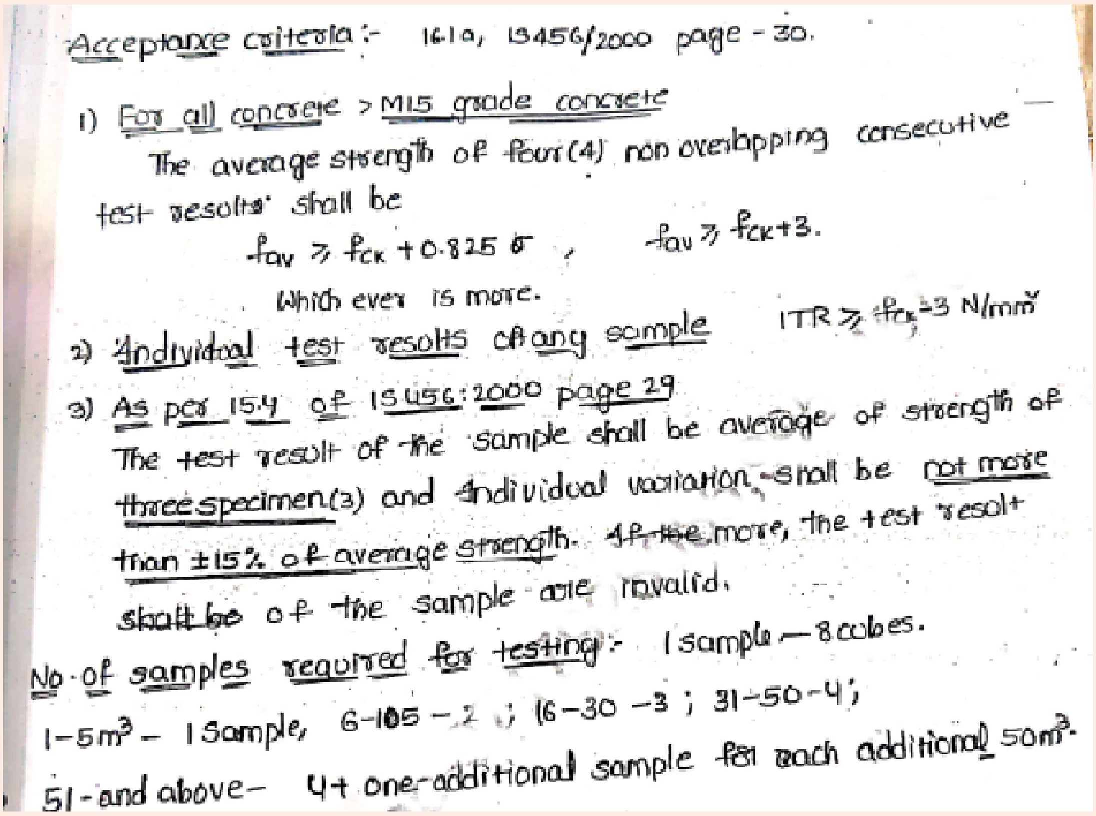
$$\text{ITR} \geq (f_{ck} - 3) \text{ N/mm}^2$$

3. As per 15, 4 of IS 456 / 2000 Page 29

The test result of the sample shall be average of the strength of three specimen and the individual variation shall be not more than ±15% of average strength. If more the test result of the sample are invalid.

MADE EASY Source

- **Theory Book:** Summary of IS 456 (Page No. 17)
- **MADE EASY Classnotes**



Acceptance criteria :- 16.1 a, IS456/2000 page - 30.

1) For all concrete > M15 grade concrete
The average strength of four (4) non overlapping consecutive test results shall be
$f_{av} \geq f_{ck} + 0.825\sigma$, $f_{av} \geq f_{ck} + 3$.
Which ever is more.

2) Individual test results of any sample ITR ≥ f_{ck} - 3 N/mm²

3) As per 15.4 of IS 456:2000 page 29
The test result of the sample shall be average of strength of three specimen(3) and individual variation shall be not more than ±15% of average strength. If the more, the test result shall be of the sample are invalid.

No. of samples required for testing :- 1 sample — 3 cubes.
1-5 m³ – 1 sample, 6-15 – 2 ; 16-30 – 3 ; 31-50 – 4 ;
51 and above – 4 + one additional sample for each additional 50 m³.

End of Solution

4. (b) (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/mm². Take E = 1 × 10^5 N/mm².

[12 Marks]

Solution:

For a cylindrical bar,

$$\epsilon_1 = \text{Longitudinal strain} = \frac{\sigma}{E}$$

$$\epsilon_2 = \text{Lateral strain} = -\frac{\mu\sigma}{E}$$

But,
$$|\epsilon_1| = 6|\epsilon_2|$$

$$\frac{\sigma}{E} = 6\frac{\mu\sigma}{E}$$

$$\mu = \frac{1}{6}$$

Now using relation,
$$E = 3K(1 - 2\mu)$$

$$1 \times 10^5 = 3K\left(1-\frac{2}{6}\right)$$

Bulk modulus,
$$K = 0.5 \times 10^5 \text{ MPa}$$

Modulus of rigidity,
$$G = \frac{E}{2(1+\mu)}$$

$$G = \frac{1\times 10^5}{2\left(1+\frac{1}{6}\right)} = \frac{3}{7}\times 10^5 \text{ MPa}$$

Now change in volume = ΔV.

We know,
$$K = \frac{\sigma}{\epsilon_v}$$

$$\epsilon_v = \frac{\sigma}{K} = \frac{120}{0.5\times 10^5}$$

$$\Delta V = \frac{120}{0.5\times 10^5}\times\frac{\pi}{4}\times 30^2\times 2000$$

Reduction in volume,
$$\Delta V = 3392.92 \text{ mm}^3$$

MADE EASY Source

- **Theory Book:** Strength of Materials (Example 2.14, Page No. 31)

End of Solution

4. (c) A suspension cable of 160 m span and 16 m central dip carries a load of 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.

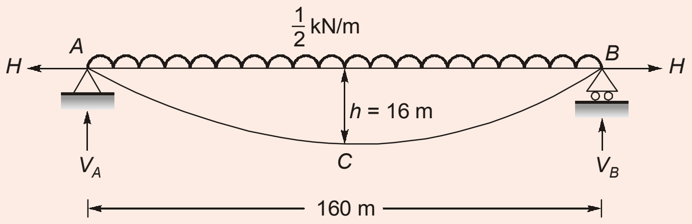
(ii) If the cable is firmly clamped to saddles carried on frictionless roller on the top of the piers.

In each case the backstay is inclined at 30° to the horizontal.

[20 Marks]

Solution:

Given,



The reactions, $$V_A = V_B = \frac{\frac{1}{2}\times 160}{2} = 40 \text{ kN}$$

The horizontal reaction, $$H = \frac{wl^2}{8h} = \frac{\frac{1}{2}\times 160^2}{8\times 16} = 100 \text{ kN}$$

The minimum reaction will occur at mid span, which is equal to the value of H.

So, $$T_{min} = H = 100 \text{ kN}$$

At support,

Maximum tension, $$T_{max} = \sqrt{100^2 + 40^2} = 107.703 \text{ kN}$$

Assuming the shape of cable to be parabola,

So, the dip, $$y = \frac{4h}{l^2}x(l-x)$$

$$\text{Slope of cable} = \frac{dy}{dx} = \frac{4h}{l^2}(L-2x)$$

At support, (x = 0)

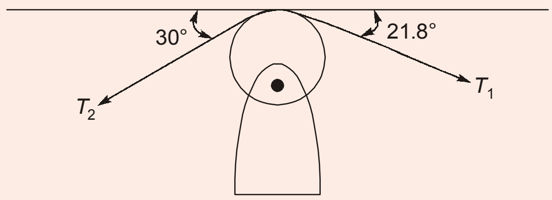
$$\text{Slope} = \frac{4h}{l} = \frac{4\times 16}{160} = 0.4$$

$$\tan\theta = 0.4$$

So, $$\theta = 21.80°$$

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

In this case, tension in cable on two sides will be equal



Here, $T_1 = T_2 = T_{max} = 107.703$ kN

So, vertical pressure on pier, $V = T_{max}$ (sin 21.8° + sin 30°)

= 107.703 (sin 21.8° + sin 30°)

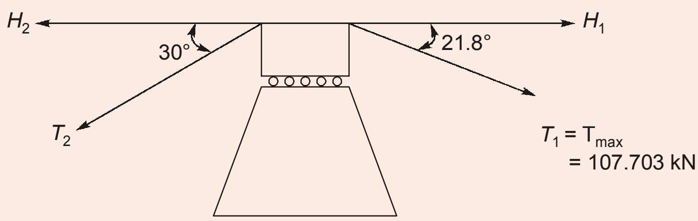
= 93.85 kN

Horizontal thrust on pier, $H = T_{max}$ (cos 21.8° – cos 30°)

= 107.703 (cos 21.8° – cos 30°)

= 6.73 kN (→)

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



Here, $H_1 = H_2$

$$T_1 \cos 21.8° = T_2 \cos 30°$$

$$T_2 = \frac{T_1 \cos 21.8°}{\cos 30°}$$

$$= \frac{107.703 \times \cos 21.8°}{\cos 30°}$$

= 115.47 kN

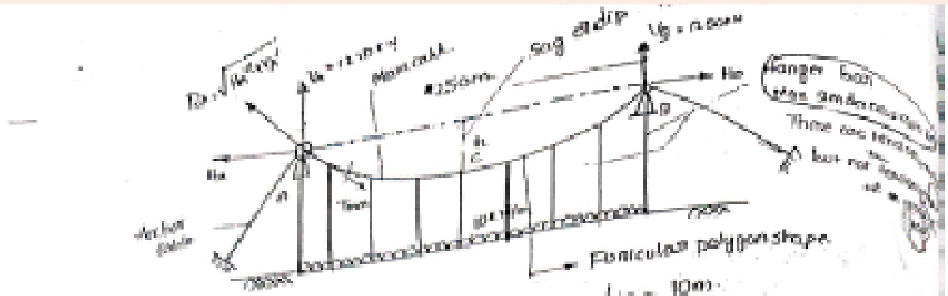
Vertical pressure on pier, $V = T_1 \sin 21.8° + T_2 \sin 30°$

= 107.703 sin 21.8° + 115.45 sin 30°

= 97.72 kN

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.2(a) of Test-14
- **Conventional Practice Question Book:** (Q.18, Page 155)
- **MADE EASY Classnotes**



Max sag = 1 in 25,
Min. sag = 1 in 150 → 10 in 150

Support Reactions:-

$V_A = V_B = \frac{wL}{2} = \frac{250\times 10}{2} = 1250$ kN.

B.M at C = 0 ⇒ stand at C and look left

$\Rightarrow (1250\times 125) - H_A\times 10m - 10\times\frac{125\times 125}{2} = 0$

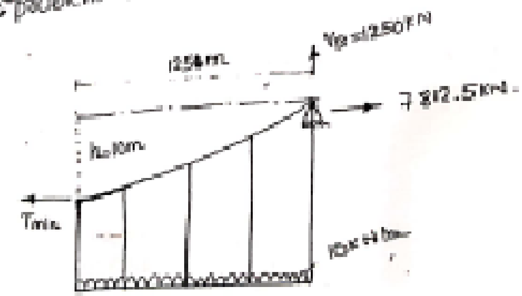
$H_A = 7812.5$ kN.

$R_A = T_{max} = \sqrt{H_A^2+V_A^2} = 7991.4$ kN.

Note:

1) Maximum tension is developed at highest point in the cable and minimum tension is developed at lowest point in the cable.

2) In the above problem minimum tension in the cable is.

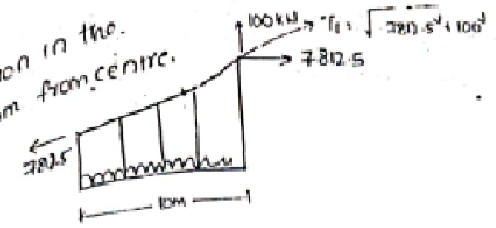


F.B.D of C.B

$\Sigma x = 0 \Rightarrow -T_{min} + 7812.5 = 0$

$T_{min} = 7812.5$ kN.

tension in the ... at 10m from centre.



If any cable sags due to its own weight along its curved length it takes the shape of a catenary. { A cosh hyperbolic function }

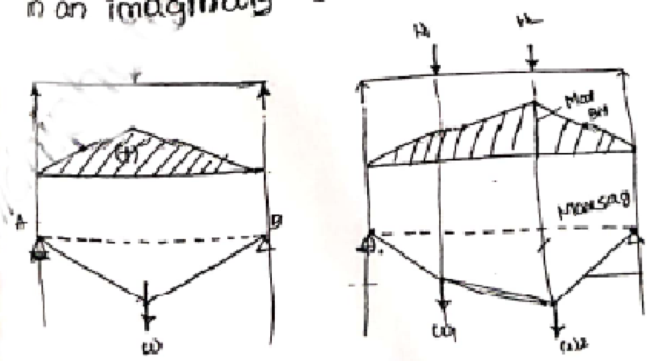
If the cable is subjected to horizontal UDL then it takes the shape of a parabola.

$$y = \frac{4h}{L^2}(Lx - x^2)$$

$$(y)_{x=50} = \frac{4 \times 100}{250^2}(250-50)\,50 = \ldots m.$$

Funicular polygon (or) string polygon:

- It is the shape taken by a simply supported cable or string when subjected to loads.
- It takes the shape of B.M.D for an imaginary S.S Beam, subjected to same loads.
- Max. sag in a cable occurs where bending moment is max in an imaginary S.S beam, subjected to same loads.



Funicular polygon.

End of Solution

SECTION 'B'

5. (a) If 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. Diameter tensile strength of the member, if yield stress for steel is 250 MPa.

[12 Marks]

Solution:

Diameter of rivet hole = 18 + 1.5 = 19.5 mm

Effective area, $A_{eff} = A_1 + kA_2$

A_1 = Net area of connected leg

A_2 = Area of outstanding leg

If angle connected by one leg only,

$$k = \frac{3A_1}{3A_1 + A_2} \text{ or } \beta = \text{shear lag factor} = 0.7$$

$$A_1 = \left(60 - \frac{8}{2} - 19.5\right) \times 8 = 292 \text{ mm}^2$$

$$A_2 = \left(60 - \frac{8}{2}\right) \times 8 = 448 \text{ mm}^2$$

$$k = \frac{3 \times 292}{3 \times 292 + 448} = 0.662 \text{ or } \beta = 0.7$$

$$A_{eff} = 292 + 0.662 \times 448 = 588.576 \text{ mm}^2$$

or $$A_{eff} = 292 + 0.7 \times 448 = 605.6 \text{ mm}^2$$

Allowable tensile stress, $f_a = 0.6 \times f_y \times 150$ MPa

Tensile strength = $150 \times 588.578 \times 10^{-3} = 88.29$ kN

or Tensile strength = $150 \times 605.6 \times 10^{-3} = 90.84$ kN

MADE EASY Source

- **MADE EASY Handbook:** Page No. 127

End of Solution

5. (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:

$$A = 11114 \text{ mm}^2,\ r_{xx} = 187.8 \text{ mm},\ r_{yy} = 51.8 \text{ mm}$$

[12 Marks]

Solution:

Applied load = 1100 kN

1. $$\lambda = \frac{kL}{r} = \frac{1 \times 4000}{51.8} = 77.22 < 180$$

2. $$\sigma_{ac} \propto \frac{1}{\lambda}$$

$$\sigma_{ac} = \frac{0.6 f_{cc} f_y}{\left(f_y^n + f_{cc}^n\right)^{1/n}} \quad \text{(where n = 1.4)}$$

$$f_{cc} = \frac{\pi^2 E}{\left(\frac{kL}{r}\right)^2}$$

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

$$f_{cc} = 331.03 \text{ N/mm}^2$$

$$\sigma_{ac} = \frac{0.6 \times 331.03 \times 250}{\left(250^{1.4} + 331.03^{1.4}\right)^{\frac{1}{1.4}}}$$

$$= 103.77 \text{ N/mm}^2 < 105 \text{ MPa}$$

3. $$P_{safe} = A_g \times \sigma_{ac}$$

$$= 11114 \times 103.77 = 1153.3 \text{ kN}$$

4. $$P_{safe} > P_{applied}$$

Hence design is safe.

MADE EASY Source

- **MADE EASY Theory book** (Design of Steel Structures)**:** Page No. 114 (5.9)

End of Solution

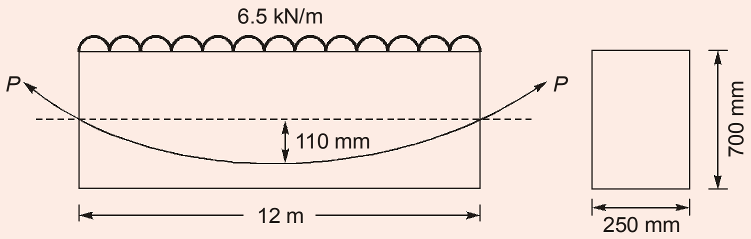
5. (c) A prestressed concrete beam supports an imposed load of 6.5 kN/m over an effective span of 12 m. The beam has a rectangular section of width 250 mm and depth of 700 mm. Find the effective prestressing force in the cable if it is parabolic with an eccentricity of 110 mm at the centre and zero at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

Solution:



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

Moment due to prestressing force = Moment due to imposed load

$$P \times e = \frac{wl^2}{8}$$

$$P = \frac{wl^2}{8e} = \frac{6.5 \times 12^2}{8 \times 0.11}$$

$$P = 1063.64 \text{ kN}$$

(ii) Stress due to imposed load, self-weight and prestressing force is zero at the soffit of the beam for the mid-span section.

Self weight of beam = 24 × 0.25 × 0.7 = 4.2 kN/m

Total load = 4.2 + 6.5 = 10.7 kN/m

$$\Rightarrow \quad \text{Moment} = \frac{wl^2}{8} = \frac{10.7 \times 12^2}{8} = 192.6 \text{ kNm}$$

Stress of soffit = 0

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

$$P\left(\frac{1}{A} + \frac{e}{Z}\right) = \frac{M}{Z}$$

$$P\left[\frac{1}{250 \times 700} + \frac{110 \times 6}{250 \times 700^2}\right] = \frac{192.6 \times 10^6 \times 6}{250 \times 700^2}$$

$$P = 849.7 \text{ kN}$$

MADE EASY Source

- **MADE EASY Theory book** (RCC)**:** Page No. 339, Ex. 14.3

End of Solution

5. (d) Define the terms activity, event and Network.

[12 Marks]

Solution:

Event

- An event is either start or completion of an activity.
- Events are significant points in a project which act as control points of the project.
- An event is an instant of time and it does not require time or resources.

Following are examples of an event:

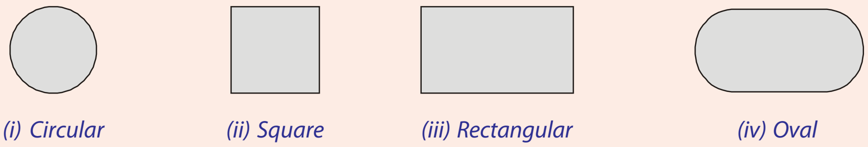
1. All parts assembled
2. A budget prepared
3. Construction completed

Following can not be events:

1. Prepare budget

2. Assemble parts
3. Excavate trench

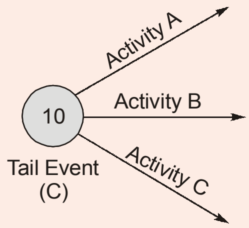
- Events are represented by nodes in a network. It may have any of the following shapes.



Different Shapes for Events

Most commonly adopted shape for events is circular shape.

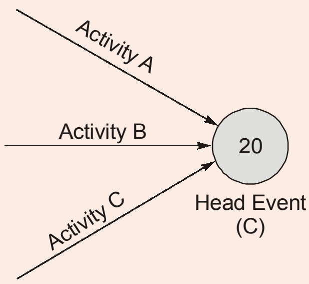
- **Tail event or the start event:**



Tail Event

It makes the beginning of an activity.
If it is the first event of project then known as "initial as start event".
It has only outgoing arrow.
e.g: Event 10 is a tail event. Arrows represent job or activity of the project.

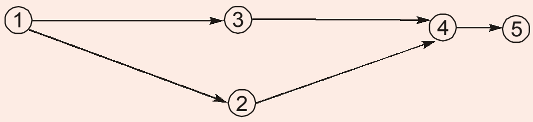
- **Head event or the final event :**



Head Event

The event which marks the completion of an activity is known as "head event".
If this event represents completion of entire project then it is called "Finish event".
It has only incoming arrows.
e.g.: Event 20 is a head event.

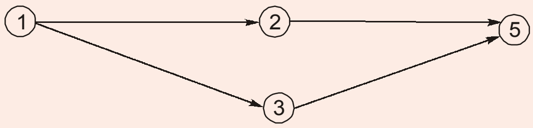
- **Dual role events** : All events except the first and the last event of a project are dual role events. They have both incoming and outgoing arrows.



Dual Role Events

e.g.: Events 2, 3 and 4, are dual role events.

- **Successor events :** The event or events that follow another event are called successor events to that event.



Successor Events

e.g.: Event 2 and 3 are successor events of event 1.

- **Predecessor events :** The event or events that occur before another event are called predecessor event to that event.

  In above figure, events 2, 3 are predecessor to event 5.

Network

- It is the flow of diagram consisting of activities and events connected logically and sequentially.
- Network diagram are of two types:
  (i) Activity-on-Arrow Network (A-O-A)
  (ii) Activity-on-Node Network (A-O-N)
- It is an outcome of the improvements in the previous method (discussed further).
- They are called by various names such as PERT, CPM, UNETICS, LESS, TOPS and SCANS.
- However all these have emerged from the two major network systems viz.:
  1. PERT
  2. CPM

Advantages of network method over bar chart and milestone chart (discussed further):

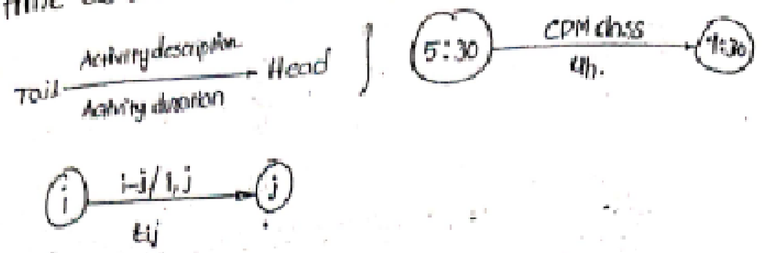
1. Inter-relationships between activities and events of a project are clearly shown.
2. The project can be treated as an integrated whole with all its sub-activities clearly related with each other. It helps in controlling the project.
3. Network method is useful for very complicated projects having large number of activities.
4. It indicates the time required in between two activities in which rescheduling of a project is possible.
5. Time uncertainty is accounted for and so it is also useful for research and development projects.

MADE EASY Source

- **Theory Book:** Construction Practice, Planning and Management (Page No. 5)
- **MADE EASY Classnotes**

Activity/Job/task.

It is the actual performance of an operation which consumes time as well as resources. It is represented by arrow.



Types of Event:

i) TAIL EVENT

It is the event which marks the starting of an activity. If a tail event marks starting of the project it is called as initial event.

NOTE: An activity cannot start until its tail has started.

2) HEAD EVENT

It is the event which marks the completion of an activity. If a head event marks completion of project, it is called Final event or finish event.

NOTE A head event occurs only when all the activities leading to it are complete

3) DUAL ROLE EVENT

It is the event which marks the starting of an activity and also the completion of some other activity.

NOTE Except initial and final events, all the intermediate events are dual role event

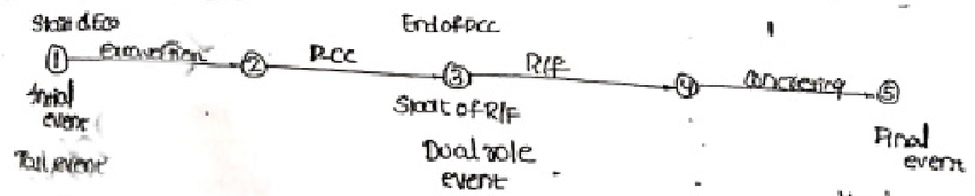
Types of Activities.

i) Serial Activities.
The activities which happen one after another are called as serial activities & dependent activities.

ii) Parallel Activities.
The activities which can happen independently of each other i.e. there is no influence of these activities on one another are called as parallel activities.

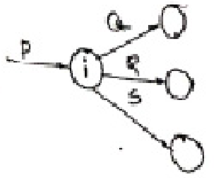
NOTE :-
The activities which terminate at the same node and originate afterwards from same node called as concurrent activities.
The activities which originate from the same node are called as Burst activities (concurrent)
The activities which terminate at the same node are called as merge activities.

eg
i) Linear Network.



* Excavation, P.C.C, R/F, concreting are serial.

Parts of Network.



As soon as P gets completed, Q, R, & S can get simultaneously started.

P, Q – Serial
P, R – Serial
P, S – Serial

Q, R, S – Parallel acts
concurrent acts
Burst acts.

End of Solution

5. (e) Find the moment of resistance of a beam 300 x 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 40 mm. Use M 20 grade concrete and steel grade Fe415.

Points on stress-strain curve for Fe415 steel.

Stress level	Fe415 grade	
	Strain	Stress (N/mm^2)
0.80 f_y	0.00144	288.7
0.85 f_y	0.00163	306.7
0.90 f_y	0.00192	324.8
0.85 f_y	0.00241	342.8
0.975 f_y	0.00276	351.8
1.00 f_y	0.00380	360.9

[12 Marks]

Solution:

$$B = 300 \text{ mm}, \quad D = 600 \text{ mm}$$

$$A_{sc} = 3 \times \frac{\pi}{4} \times 20^2 = 942.48 \text{ mm}^2$$

$$A_{st} = 3 \times \frac{\pi}{4} \times 20^2 = 942.48 \text{ mm}^2$$

Effective cover = 40 mm

f_{ck} = 20 N/mm^2

f_y = 415 N/mm^2

Let us assume trial depth of NA = 100 mm

Strain at the level of compression steel,

$$\varepsilon_{sc} = \frac{0.0035}{100}(100-40) = 0.0021$$

$$f_{sc} = 324.8 + \frac{342.8-324.8}{0.00241-0.00192}(0.0021-0.00192)$$

$$= 331.4 \text{ N/mm}^2$$

Depth of NA

$0.36\ f_{ck}Bx_u + (f_{sc} - 0.45f_{ck}) \times A_{sc} = 0.87\ f_yA_{st}$

$0.36 \times 20 \times 300 \times x_u + (331.4 - 0.45 \times 20) \times 942.48$

$= 0.87 \times 415 \times 942.48$

$x_u = 16.86 \text{ mm} \neq (x_u)$ assumed.

Assuming new depth of NA,

$x_u = 50 \text{ mm}$

Now, $\varepsilon_{sc} = \dfrac{0.0035}{50} \times (50-40) = 0.0007$

Assuming, linear variation upto strain 0.00144

So, $f_{sc} = 0.007 \times E_s$

$= 0.007 \times 2.1 \times 10^5$

$= 147 \text{ N/mm}^2$

Now, new depth of NA

$0.36\ f_{ck}\ Bx_u + (f_{sc} - 0.45\ f_{ck})\ A_{sc} = 0.87\ f_y\ A_{st}$

$0.36 \times 20 \times 300 \times x_u + (147 - 0.45 \times 20) \times 942.48$

$= 0.87 \times 415 \times 942.48$

$x_u = 97.32 \text{ mm} \neq (x_u)$ assumed.

Now, assuming, $x_u = 60$ mm

$\varepsilon_{sc} = \dfrac{0.0035}{60}(60-40) = 0.001167$

So, $f_{sc} = 0.001167 \times E_s$

$= 0.001167 \times 2.1 \times 10^5$

$= 245 \text{ N/mm}^2$

$0.36 \times 20 \times 300 \times x_u + (245 - 0.45 \times 20) \times 942.48$

$= 0.87 \times 415 \times 942.478$

$x_u = 54.56 \text{ mm} \simeq 60 \text{ mm}$

So, taking average, $x_u = \dfrac{54.56+60}{2} = 57.28$ mm

$M_u = 0.36\ f_{ck} Bx_u(d - 0.42x_u) + (f_{sc} - f_{ck})A_{sc}(d - d')$

$= 0.36 \times 20 \times 300 \times 57.28 + (560 - 0.42 \times 57.28)$

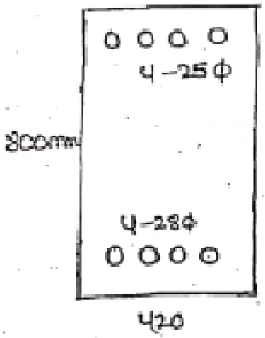
$+(245 - 0.45 \times 20) \times 942.48 \times (560 - 40)$

$M_u = 181.971$ kNm

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.7(b) of Test-12
- **Conventional Practice Question Book:** (Q.15, Page 216)
- **MADE EASY Classnotes**

Calculate maximum moment that can be applied over a dobly reinforced beam as section follows, M30, Fe500



① $x_{ulim} = 0.46d = 345\text{mm}$

② Actual depth of neutral axis x_u

$$0.36 f_{ck} B x_u + (f_{sc} - 0.45 f_{ck}) A_{sc} = 0.87 f_y A_{st}$$

$$\Rightarrow 0.36 \times 30 \times 420 \times x_u + (f_{sc} - 0.45 \times 30) \times 1964 = 0.87 \times 500 \times 2463$$

$$\Rightarrow 4536 x_u + 1964 f_{sc} = 1097919$$

$$\Rightarrow x_u = \frac{1097919 - 1964 f_{sc}}{4536}$$

Trail ① ⇒ Assume $x_u = 345\text{mm}$

$$\varepsilon_{sc} = \frac{x_u - d_c}{x_u} \times 0.0035 = \frac{345 - 50}{345} \times 0.0035$$

$$= 0.00299$$

$$f_{sc} = 413 + \frac{423.9 - 413.0}{0.00312 - 0.00277}(0.00299 - 0.00277)$$

$$= 419 \text{ N/mm}^2$$

$x_u = \frac{(107919 - 1964 \times 413)}{4536} = 60.6 \text{ mm}$ fail

Trail 2 : Assume $x_u = 200$ mm.

$200 A = 107919$

$\varepsilon_{sc} = \frac{x_u - d_c}{x_u} \times 0.0035$

$= \frac{200 - 50}{200} \times 0.0035 = 0.002625$

0.002626	391.3
0.00277	413.0

$f_{sc} = 391.3 + \frac{413.0 - 391.3}{0.00277 - 0.002626} \times (0.002625 - 0.002626)$

$= 406.83 \text{ N/mm}^2$

$\therefore x_u = \frac{107919 - 1964 \times 406}{4536} = 151 \text{ mm}$ (fail)

Trail-3 : Assume $x_u = 96$ mm

$\varepsilon_{sc} = \frac{x_u - d_c}{x_u} \times 0.0035 = 0.00167$

$f_{sc} = 0.00167 \times 2 \times 10^5 = 335.42 \text{ N/mm}^2$

$x_u = \frac{107919 - 1964 \times 335}{4536} = 96.8 \text{ mm}$

Trail-4 : $x_u = 96.4$ mm

$\varepsilon_{sc} = 0.00168$

$f_{sc} = 336 \text{ N/mm}^2$

$x_u = 96.5$ mm

$x_u = 96.4$ mm

$f_{sc} = 336$ N/mm²

$x_u < x_{ulim}$

Hence design an under reinforced section.

$M_u = 0.36 f_{ck} B x_u (d - 0.42 x_u) + (f_{sc} - 0.45 f_{ck}) A_{sc} (d - d_c)$.

$= 0.36 \times 30 \times 420 \times 96.4 (750 - 0.42 \times 96.4)$

$+ (336 - 0.45 \times 30) \times 1964 \times (750 - 450)$

$= 753.6$ kNm.

Very important table:

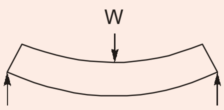
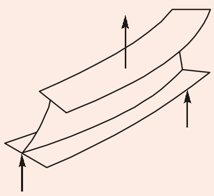
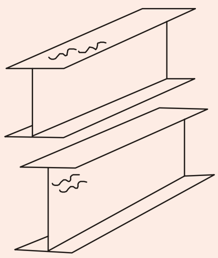
	Fe 250	Fe 415	Fe 500
f_{sc} =	217.5 N/mm²	350	420

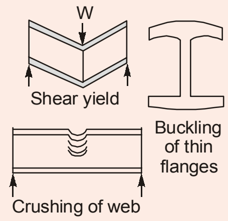
End of Solution

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

[6 Marks]

Solution:

Failure mode	Description	Illustration	Comments
Excessive bending	Beam is adequately braced in the lateral plane. Plate components are not too Components are not too thin (compact section) Beam fails due to excessive deformation in the plane of loading.		This is the mode of failure if all other modes of failure are prevented.
Lateral torsional buckling	In this case failure occurs due to lateral deflection as well as twist. The load at which this failure occurs depends upon the proportion of the beam, manner of loading and support conditions.		By providing suitable lateral bracing this can be prevented.
Local buckling	Flange may buckle due to compression. Web may buckle due to shear or due to combined effect of shear and bending or due to direct vertical compression under a concentrated load.		This failure is unlikely for hot rolled sections since their proportioning are made suitably. However in plate girders web stiffening may be necessary. Bearing stiffeners are provided at supports and under point loads.

Local Failure	This may occur due to the following: (i) Web may yield due to shear (ii) Web may suffer local crushing (iii) Local failure around any openings if present in the web		These are likely in short span or deep beam. Suitable web stiffening should be done. Regions surrounding web holes may be strengthened by local reinforcement.

MADE EASY Source

- **MADE EASY Theory book** (Design of Steel Structures)**:** Page No. 7

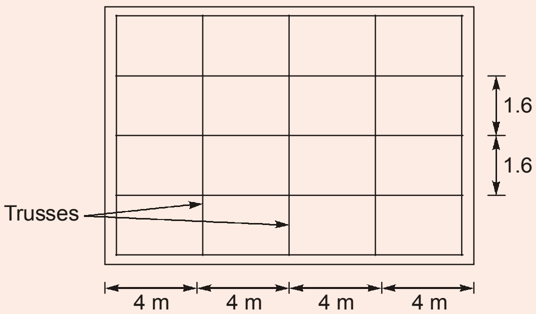
End of Solution

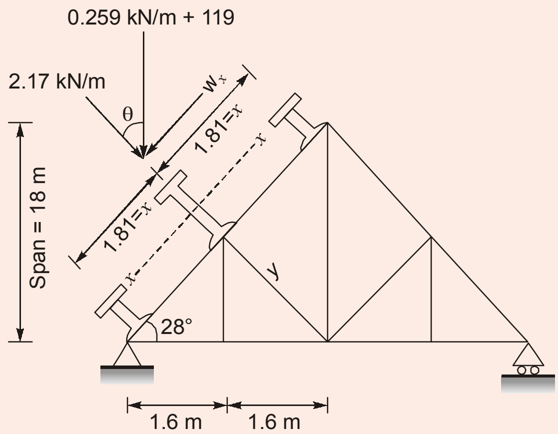
6. (a) (ii) A pitched roof is to be provided for a workshop of effective span 18 m. The trusses are spaced at 4 m centre to centre and purlins at 1.6 m centre to centre. The pitch of the roof is 28°, weight of the roofing material is 0.162 kN/m, normal wind pressure is 1.2 kN/m^2 and permissible bending 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 Marks]

Solution:

Part-1: Load analysis





Wind load

Wind process acts normal to surface of roof.

$$\text{Wind load} = 120 \text{ kg/m}^2 = 120 \text{ N/m}^2 = 1.2 \text{ kN/m}^2$$

$$\text{Wind load/m width} = 1.2 \times 1.81 \text{ m} = 2.17 \text{ kN/m} \quad \text{[If acts along y-axis]}$$

$$\text{Weight of roofing material} = 162 \text{ N/m}^2 \text{ of plan area}$$

$$= 0.162 \text{ kN/m}^2 \text{ of plan area}$$

$$\text{Weight of roofing material/m run} = 0.162 \times 1.6 \text{ m} = 0.259 \text{ kN/m}$$

[If acts vertically downwards]

$$\text{Self weight of purlin} = 11.9 \text{ kg/m} = 0.119 \text{ kN/m}$$

[If acts vertically downward]

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GS + Maths + Reasoning & Aptitude (10 Subjects)	300 Hrs.	₹20,000	₹14,500

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Total vertically downward load = 0.119 + 0.259 = 0.378 kN/m

Total UDL along y-axis = 2.17 + (0.378 × cos 28°)

w_y = 2.51 kN/m

Total UDL along x-axis = w_x = 0.378 × sin 28° = 0.178 kN/m

$$M_{xx} = \text{BM about } x\text{-axis} = \frac{w_y l^2}{10} = \frac{2.51 \times 4^2}{10} = 4 \text{ kNm}$$

$$M_{yy} = \text{BM about y-axis} = \frac{w_x l^2}{10} = \frac{0.178 \times 4^2}{10} = 0.28 \text{ kNm}$$

$$Z_{xx} = \frac{I_{xx}}{y} = \frac{406.8 \times 10^4}{(125/2)} \text{ mm}^4 = 65.1 \times 10^3 \text{ mm}^3$$

$$Z_{yy} = \frac{I_{yy}}{y} = \frac{43.4 \times 10^4}{(75/2)} = 11.57 \times 10^3 \text{ mm}^3$$

$$\sigma_{bc,\ cal} \text{ at A} = \frac{M_{xx}}{Z_{xx}} + \frac{M_{yy}}{Z_{yy}} \le \sigma_{bc} = 165 \text{ MPa}$$

$$= \frac{4 \times 10^6}{65.1 \times 10^3} + \frac{0.28 \times 10^6}{11.57 \times 10^3} = 85.6 \text{ N/mm}^2$$

$$< \sigma_{bc} = 165 \text{ MPa}$$

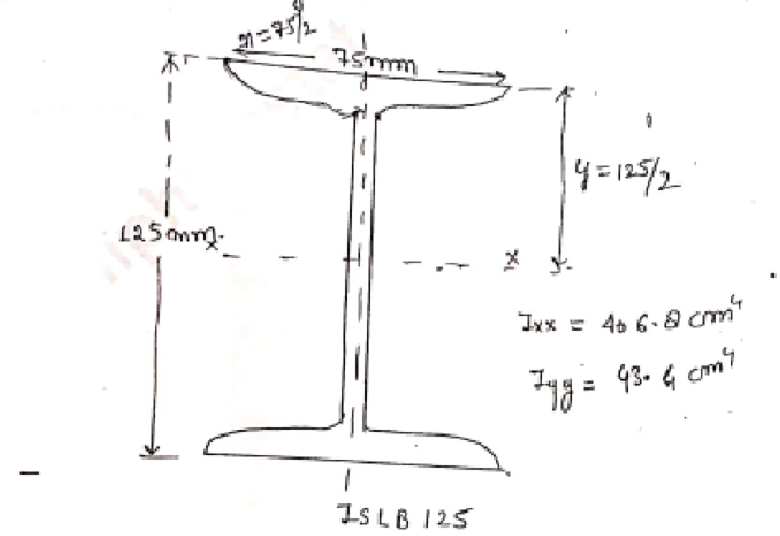
Hence, purlin is safe in bending

$$\cos 28° = \frac{1.6}{x} = x = 1.81 \text{ m}$$

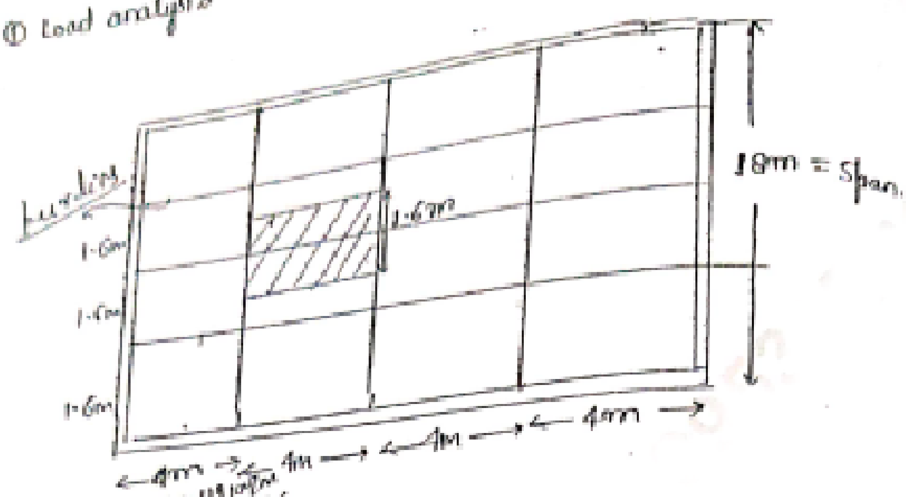
MADE EASY Source

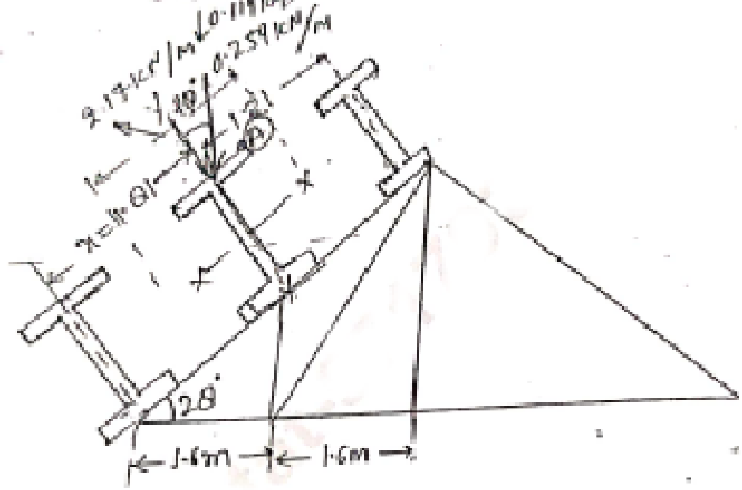
- **Theory Book:** Design of Steel Structures (Example 13.2, Page No. 342)
- **MADE EASY Classnotes**

(*) A pitched roof is to be provided for a workshop of effective span 18 m trusses are spaced at 4 m centre to centre and purlins at 1.6 m centre to centre. The slope of the roof is 30°. The wt of roofing material is 16.2 kg/m² and normal wind pressure is 120 kg/m² of ISLB 125 @ 11.9 kg/m is used for purlins, check the adequacy of the section, if permissible bending stress 165 MPa, the details of I-section are as shown in fig.



Sol. ① Load analysis





$\cos 28^\circ = \frac{1.6}{x}$ $\quad x = \frac{1.6}{\cos 28}$ $\quad x = 1.81\ m.$

wind load: wind pressure acts normal to the surface of roof

wind load $= 120\ kg/m^2 = 1.2\ kN/m^2$

wind load/m width $= 1.2 \times 1.81\,m = 2.17\ kN/m$

(It acts along y-axis)

Wt of roofing material = 162 N/m^2 plan area

$= 0.162\ kN/m^2$

Wt of roofing material/m run

$= 0.162 \times 1.6 = 0.259\ kN/m$

It acts vertically downward

Self wt of purlin = 11.9 kg/m = 0.119 kN/m

It acts vertically downward

Total vertically downward load = 0.119 + 0.259

$= 0.378\ kN/m$

Total udl along y-axis $= w_y = 2.17 + 0.378 \cos 28°$

$= 2.51\ kN/m$

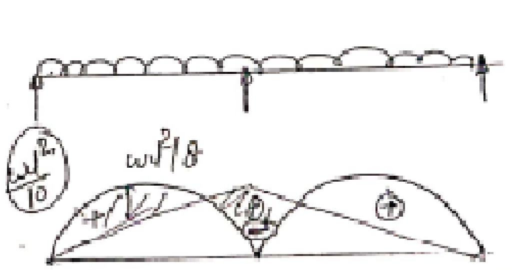
Total udl along x-axis $w_x = 0.378 \times \sin 28$

$= 0.178\ kN/m$

M_{xx} = B.M about x axis $= \frac{w_y l^2}{10} = \frac{2.51 \times 4^2}{10} = 4\ kN\,m$

(assume the purlin is continuous over more than 2 supports

max B.M $= \frac{wl}{10} = \frac{wl^2}{10}$



l = span of purlins = spacing of trusses

= 4m

$M_{yy} = \frac{w_x l^2}{10} = \frac{0.178 \times 4^2}{10} = 0.28\ kN\,m$

σ_{bc} cal at A $\frac{M_{xx}}{Z_{xx}} + \frac{M_{yy}}{Z_{yy}} \leq \sigma_{bc} = 0.66 f_y = 165$ MPa

$Z_{xx} = \frac{I_{xx}}{y} = \frac{406.8 \times 10^4}{125/2} = 65.1 \times 10^3 \text{ mm}^3$

$Z_{yy} = \frac{I_{yy}}{x} = \frac{43.4 \times 10^4}{75/2} = 11.57 \times 10^3 \text{ mm}^3$

σ_{bc} cal at (A)

$\frac{4 \times 10^6 \text{ Nmm}}{65.1 \times 10^3} + \frac{0.28 \times 10^6}{11.57 \times 10^3} = 85.6 \text{ N/mm}^2 < \sigma_{bc} = 165 \text{ MPa}$

Hence purlins is safe in bending.

End of Solution

6. (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/m^2. Use M25 grade of concrete and steel grade Fe415. 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
a_x	0.062	0.074	0.084	0.093	0.099	0.104	0.113	0.118
a_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.

$100A_{st}/bd$	τ_c N/mm^2
0.25	0.36
0.50	0.49
0.75	0.57
1.00	0.64

[20 Marks]

Solution:

Short span = 4.2 m

$$\text{So total depth} = \frac{4200}{20} = 210 \text{ mm}$$

$$\text{Effective depth} = 210 - 30 = 180 \text{ mm}$$

$$L_{ex} = 4.2 + 0.18 = 4.33 \text{ m}$$

$$L_{ey} = 5.8 + 0.18 = 5.98 \text{ m}$$

$$\gamma = \frac{L_y}{L_{ex}} = \frac{5.98}{4.38} = 1.37 \not> 2$$

$$\alpha_x = 0.093 + \frac{0.099 - 0.093}{1.4 - 1.3}(1.37 - 1.3) = 0.0972$$

$$\alpha_y = -\frac{0.055 - 0.051}{1.4 - 1.3}(1.4 - 1.37) = 0.055 = 0.0539$$

Load calculation:

$$\text{Self weight of slab} = 0.18 \times 25 = 4.5 \text{ kN/m}^2$$

$$\text{Live load} = 4 \text{ kN/m}^2$$

Total factored load, $w_u = 1.5\,[4.5+ 4] = 12.75 \text{ kN/m}^2$

$$M_x = \alpha_x w_u L_x^{\,2}$$

$$= 0.0972 \times 12.75 \times 4.38^2 = 23.78 \text{ kNm}$$

$$M_y = \alpha_y w_u L_x^{\,2}$$

$$= 0.0539 \times 12.75 \times 4.38^2 = 13.18 \text{ kNm}$$

Check for depth:

$$d = \sqrt{\frac{23.78 \times 10^6}{0.138 \times 25 \times 1000}} = 83.02 \not> 180 \text{ mm} \qquad \text{(OK)}$$

Steel calculation:

Along shorter span, $A_{st_x} = \dfrac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \dfrac{4.6\,M_{ux}}{f_{ck}\,Bd^2}}\right]Bd$

$$= \frac{0.5 \times 5}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 23.78 \times 10^6}{25 \times 1000 \times 180^2}}\right] \times 1000 \times 180$$

$$= 379.36 \text{ mm}^2$$

$$\text{Spacing of 8 mm bar} = \frac{1000 \times \frac{\pi}{4} \times 8^2}{379.36} = 132.5 \text{ mm c/c}$$

(So provide 130 mm c/c)

For longer span:

$$A_{st_y} = \frac{0.5 \times 25}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 13.18 \times 10^6}{25 \times 1000 \times 180^2}}\right]1000 \times 180$$

$$= 206.85 \text{ mm}^2$$

$$A_{st\ min} = \frac{0.12 \times 210 \times 100}{100} = 252 \text{ mm}^2$$

$$\text{Spacing of 8 mm bar} = \frac{1000 \times \frac{\pi}{4} \times 8^2}{252} = 199.36 \text{ mm}$$

(So provide 190 mm c/c)

Check of shear:

$$\text{Maximum shear force} = \frac{wL_x}{2} = \frac{12.75 \times 4.2}{2} = 26.78 \text{ kN}$$

$$\text{Nominal shear stress} = \frac{26.78 \times 10^3}{1000 \times 180} = 0.15 \text{ MPa}$$

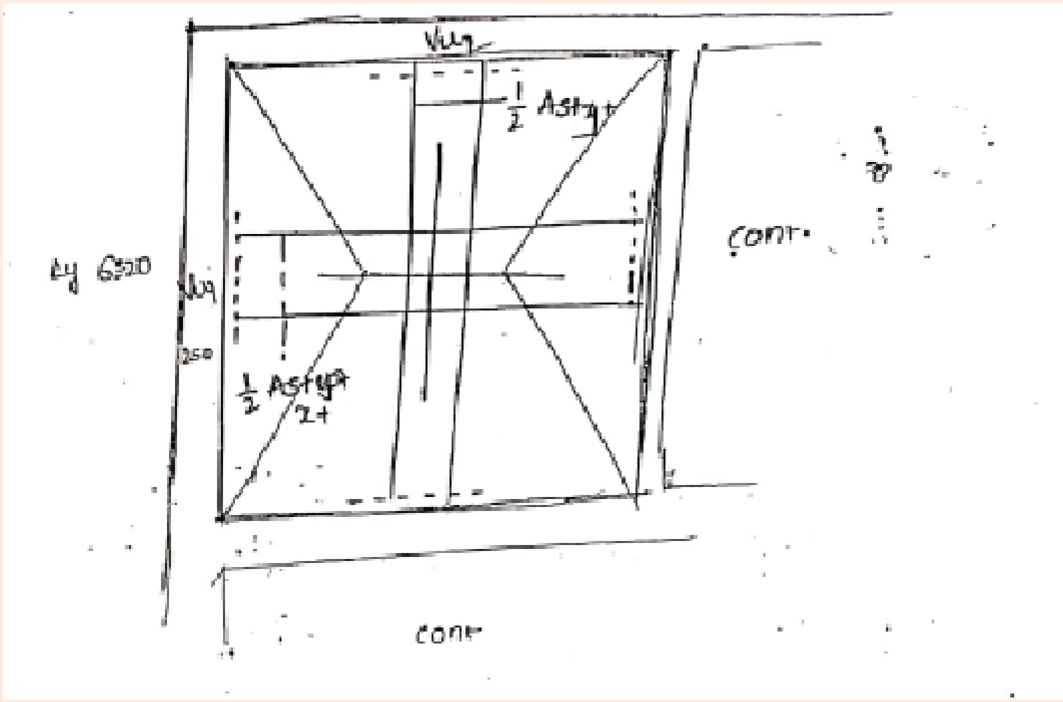
$$\text{Pt\%} = \frac{100 \times \frac{\pi}{4} \times 8^2 \times \frac{1000}{130}}{1000 \times 180} = 0.22\%$$

$$\tau_c = 0.3 \times 1.3 = 0.39 \text{ MPa} > 0.15 \text{ MPa}$$

(So safe in shear)

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.6(a) of Test-6
- **Theory Book:** RCC (Example 11.2 & 11.3, Page No. 233)
- **MADE EASY Classnotes**



1. L_x clear = 4.60 m

L_{ycl} = 6.20 m

$\frac{L_{ycl}}{L_{xcl}} = \frac{6.20}{4.60} = 1.35 < 2$

Hence two-way slab.

Panel - no. 4 of IS 456:2000

2. Depth of slab required using deflection criteria

Effective depth = $\frac{\text{Span}}{A \times MF} = \frac{4600}{26 \times 1.2}$ = 147.44 mm

Say d = 150 mm

③ Effective span

Width of support = 250 mm

$\frac{L_{cl}}{12}$ = 383.33 mm

$w < \frac{L_{cl}}{12}$ Hence take as S.S. slab

$L_{xef} = L_{xcl} + d$ or $= L_{xcl} + w$ } Min

L_{xef} = 4.75 m

L_{yef} = 6.35 m ($L_{ycl} + d$)

Span ratio = $\frac{L_{yef}}{L_{xef}} = \frac{6.35}{4.75} = 1.34$

(4) Loads :

$DL = 0.15 \times 1 \times 1 \times 25 = 4.5\ kN/m^2$

$FL = \qquad = 1.5\ kN/m^2$

$LL = 7 \times 1 \times 1 = 7\ kN/m^2$

$TL = 13\ kN/m^2$

Factored $W_u = 1.5 \times 13 = 19.5\ kN/m^2$

(5) Moment coefficients.

$\alpha_{x-} = 0.068 \qquad \alpha_{y-} = 0.047$

$\alpha_{x+} = 0.051 \qquad \alpha_{y+} = 0.035$

(6)

	α	Moment $\alpha W_u l_x^2$	d_{reqd}	A_{st} req	Spacing ϕ	Spacing	Spacing provided.
M_{ux-}	0.068	29.992	d_x 145 150	$A_{st\,x-}$ 604	10	130.3	10mm @ 130 mm c/c
M_{ux+}	0.051	~~44.96~~ 22.44	d_x 150	$A_{st\,x+}$ 442	10	177.7	10mm @ 170 mm c/c
M_{uy-}	0.047	20.68	d_y 140	$A_{st\,y-}$ 438	10	179.3	10mm @ 170 mm c/c
M_{uy+}	0.035	15.4	d_y 140	$A_{st\,y+}$ 320	10	245 mm	10mm @ 240 mm c/c.

Moment $= M_u = \alpha \cdot 19.5 \times 4.75^2$

$= 440\,\alpha$.

(7) Check depth required.

$$d = \sqrt{\frac{M_{u\,max}}{Q\,B}} = \sqrt{\frac{29.42 \times 10^6}{0.138 \times 20 \times 1000}} = 104\ mm < 150\ mm.$$

(8) Ast required

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

(9) Spacing $s = \frac{1000}{A_{st}} \times \frac{\pi}{4} \times 10^2$

(10) Distribution bars

$$A_{st} = \frac{0.12}{100} \times 1000 \times 180 = 216\ mm^2$$

Spacing of 8mm ϕ = $\frac{1000}{216} \times \frac{\pi}{4} 8^2 = 232\ mm$

Provide 8 mm ϕ @ 232mm c/c.

(11) Check for shear

$$V_{u1} = W_u L_x \left(\frac{\gamma}{2+\gamma}\right)$$

$$V_{u2} = \frac{W_u L_x}{3}$$

(2) Check for (1) $V_{u1} = 19.5 \times 4.60 \times \frac{1.35}{1.35+2}$

$= 36.15\ kN.$

(2) $\tau_v = \frac{V_{u1}}{Bd} = \frac{36.15 \times 1000}{1000 \times 150} = 0.241\%$

(3) $P_t\% = \frac{\frac{1}{2} A_{st,xt}}{Bd} \times 100$

$$= \frac{\frac{442}{2}}{1000 \times 150} \times 100 = 0.15\%$$

(4) $\tau_c = 0.28\ N/mm^2$

(5) $k = 1.24$

(6) $k \times \tau_c = 0.347 > \tau_v$

$\tau_v < k \cdot \tau_c$

Hence safe.

(9) (1) $V_{u2} = W_u L_y / 3 = \frac{19.3 \times 4.60}{3} = 29.9$ kN.

(2) $\tau_v = \frac{V_{uL}}{Bd} = \frac{29.9 \times 10^3}{Bd} = 0.214$ N/mm²

(3) $P_t \% = \frac{\frac{1}{2} A_{st\,y+}}{Bd} \times 100 = 0.11\%$

(4) $\tau_c = 0.28$ N/mm²

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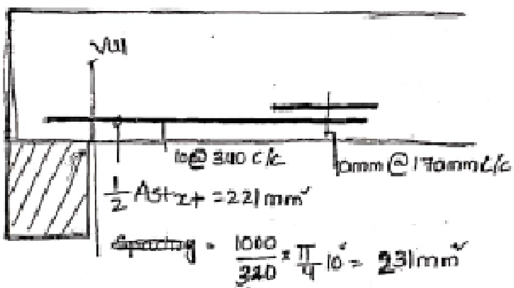
(5) $k = 1.29$

(6) $k \cdot \tau_c = 0.347$ N/mm² $> \tau_v$

Hence safe

(12) Check for development length (L_d)

(A) Simply supported edge where $A_{st(x)+}$ is available where shear force $= V_{u1}$



$V_{u1} = 36.15$ kN $\qquad L_d \leq 1.30 \frac{M_{u1}}{V_{u1}} + L_o$

$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = \frac{0.87 \times 415 \times 10}{4 \times 1.60 \times 1.20} = 471$ mm

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} B} = \frac{0.87 \times 415 \times 231}{0.36 \times 20 \times 1000}$$

$$= 11.58 \simeq 11.6 \text{ mm}$$

$$M_{u_1} = 0.87 f_y A_{st} (d - 0.42 x_u)$$

$$= 12.10 \text{ kN-m}$$

$$L_0 = \frac{L_s}{2} - x' = \frac{250}{2} - 30 = 95 \text{ mm}$$

$$\frac{M_{u1}}{V_u} = 334.72 \text{ mm}$$

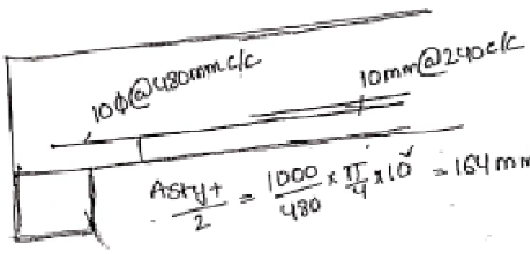
$$L_d \leq 1.30 \frac{M_{u1}}{V_u} + L_0 \Rightarrow 471 \leq \frac{1.30 \times 12.1 \times 10^6}{36.15 \times 10^3} + 95$$

$$L_d \leq 530$$

Hence safe

(B) At simply supported edge (for $A_{st}/2$) i.e. for V_{u2}

$$V_{u2} = 29.9 \text{ kN}$$



$$\frac{A_{st}}{2} = \frac{1000}{480} \times \frac{\pi}{4} \times 10^2 = 164 \text{ mm}^2$$

Check with curtailment:

$$L_d = \frac{0.87 f_y \phi}{4 \tau_{bd}} = 471 \text{ mm}$$

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} B} = \frac{0.87 \times 415 \times 164}{0.36 \times 20 \times 1000} = 8.2 \text{ mm}$$

$$M_{u2} = 0.87 f_y A_{st} (d - 0.42 x_u) = 0.87 \times 415 \times 164 \times (140 - 0.42$$

$$= 8.08 \text{ kN-m}$$

$L_o = 95\,mm$

$$1.30\frac{M_{u2}}{V_{u2}} + L_o = \frac{1.30 \times 8.08 \times 10^6}{29.9 \times 10^3} + 95$$

$$= 446\,mm < L_d$$

$$\left(L_d > \frac{1.30 M_{u2}}{V_{u2}} + L_o\right) \text{ — failed.}$$

Let us continue all reinforcement 100% upto support

Check without curtailment:-

$$A_{st} = \frac{1000}{240} \times \frac{\pi}{4} \times 10^2 = 327\,mm^2$$

$$x_u = \frac{0.87 f_y A_{st}}{0.36 f_{ck} B} = 16.4\,mm$$

$$M_{u2} = 0.87 f_y A_{st}(d - 0.42x_u)$$

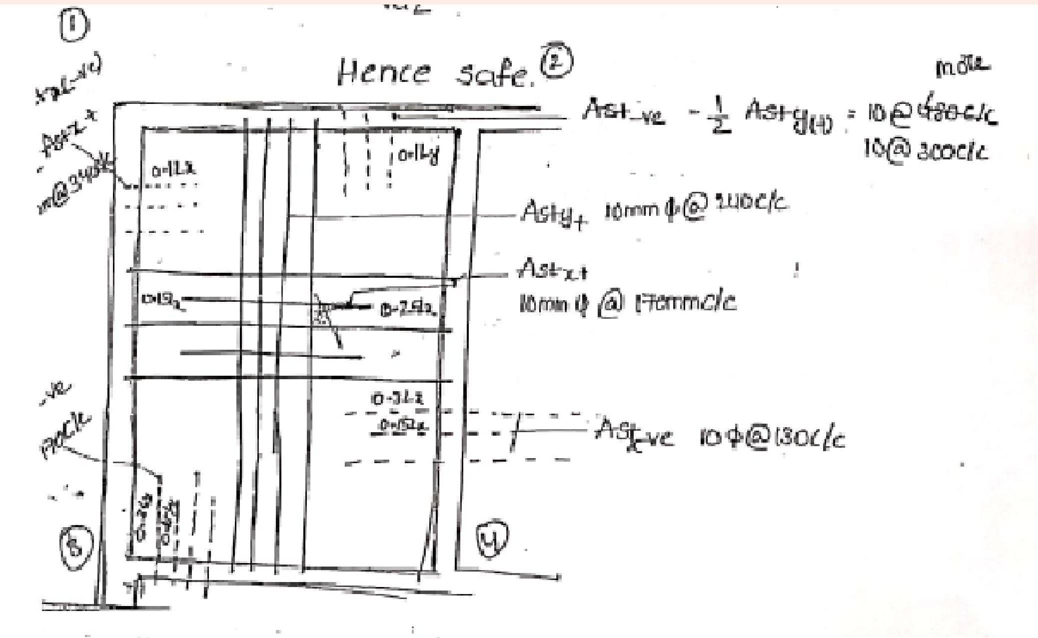
$$= 0.87 \times 415 \times 327 \times (140 - 0.42 \times 16.4)$$

$$= 15.7\,kN\text{-}m$$

$$\frac{1.3 M_{u2}}{V_{u2}} + L_o = \frac{1.3 \times 15.7 \times 10^6}{29.9 \times 10^3} + 95$$

$$= 777\,mm$$

$$L_d < \frac{1.3 M_{u2}}{V_{u2}} + L_o$$



(13) Check for torsion

(a) At corner (1)

→ Provided in 4 layers

→ Size of m/f = $\frac{L_x}{5} = \frac{4.75}{5}$

$= 0.95\text{m} = 950\text{mm}.$

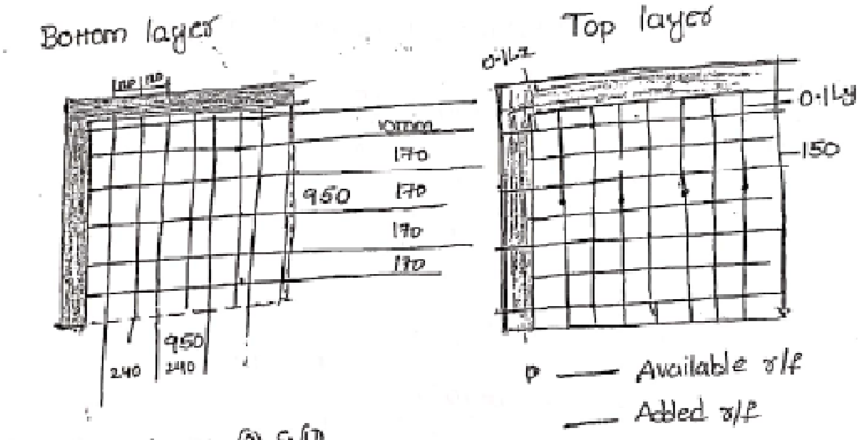
→ $Ast_{req} = \frac{3}{4} Ast_{x+}$

$= \frac{3}{4} \times 442 = 332\text{mm}^2$

→ Spacing required for 10mm φ

$s = \frac{950}{332} \times \frac{\pi}{4} \times 10^2 = 224\text{mm}$

Say 10mm φ @ 220mm c/c.



(b) At corner ③ & ④

→ Provided in 4 layers

→ Size of mesh = 950mm

→ Area of steel = $\frac{3}{8} \times \frac{3}{4} \times \frac{1}{2} \times 442 = 166\ mm^2$

→ Spacing = $\frac{950}{166} \times \frac{\pi}{4}\, 10^2 = 449 mm$

All spacing provided is less than 449 hence OK.

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End of Solution

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

[20 Marks]

Solution:

Vibrators

These are the mechanical devices which are used to compact concrete in the formwork. The advantages of vibrators over hand methods are as follows:

(i) It is possible by means of vibrators to make a harsh and stiff concrete mix, with a slump of about 40 m or less, workable.

(ii) The quality of concrete can be improved by use of vibrators as less water will be required or in other way, economy can be achieved by adopting a leaner mix when vibrators are used.

(iii) The use of vibrators results in the reduction of consolidation time. Hence the vibrators are used where the rapid progress of work is of great importance.

(iv) With the help of vibrators, it is possible to deposit concrete in small openings or places where it will be difficult to deposit concrete by hand methods.

Following are the four types of vibrators:

1. Internal or immersion vibrators
2. Surface vibrators
3. Form or shutter vibrators
4. Vibrating tables

1. **Internal or immersion vibrators:** These vibrators consist of a steel tube which is inserted in fresh concrete. This steel tube is called the poker and it is connected to an electric motor or a petrol engine through a flexible tube. They are available in sizes varying from 40 mm to 100 mm diameters and the size is decided by keeping in mind the spacing between reinforcing bars in concrete. The frequency of vibration is about 300 to 600 rpm.

   The poker vibrates while it is being inserted. The internal vibrators should be inserted and withdrawn slowly and they should be operated continuously while they are being withdrawn. Otherwise holes will be formed inside the concrete. The vibrator can be placed vertically or at a slight inclination not exceeding 10° to the vertical with a view to avoid flow of concrete due to vibration into the mould and consequent scope of segregation. Hence skilled and experienced men should handle internal vibrators. Theses vibrators are more efficient than other types of vibrators and hence they are most commonly used.

2. **Surface vibrators:** These vibrators are mounted on platform or screeds. They are used to finish concrete surfaces such as bridge floors, road slabs, station platform, etc. These vibrators are found to be more effective for compacting very dry concrete mixes because the vibration acts in the same direction of gravity and the concrete is compacted in a confined zone. These vibrators also cause movement of fine material to the top and it aids in finishing operations. However the movement of excess fine material at top will not be desirable for plastic mixes as the wearing resistance of such fine material is very low.

3. **Form or shutter vibrators:** These vibrators are attached to the formwork and external centering of walls, columns, etc. The vibrating action is conveyed to the concrete through the formwork during transmission of vibrations. Hence, they are not generally used. But they are very much helpful for concrete sections which are too thin for the use of internal vibrators.

   These vibrators require more power because of loss of some power in vibrating the rigid shutters. They are also heavy and hence they cannot be clamped at as many points as possible for uniform compaction of concrete. The compaction by these vibrators is found to be effective only upto a distance of about 450 mm form the face of the formwork.

4. **Vibrating tables:** These are in the form of a rigidly built steel platform mounted on flexible springs and they are operated by electromagnetic action or electric motors. They are found to be very effective in compacting stiff and harsh concrete mixes and hence they are invariably used in the preparation of pre-cast structural products in factories and test specimens in laboratories.

   The tables are vibrated either mechanically or by placing the springs under the supports of tables. The frequency of vibrations varies form 3000 to 7200 vibrations per minute. The two parameters of vibrations are frequency and time and they are related as follows:

$$\text{Frequency} \propto \frac{1}{\text{Time}}$$

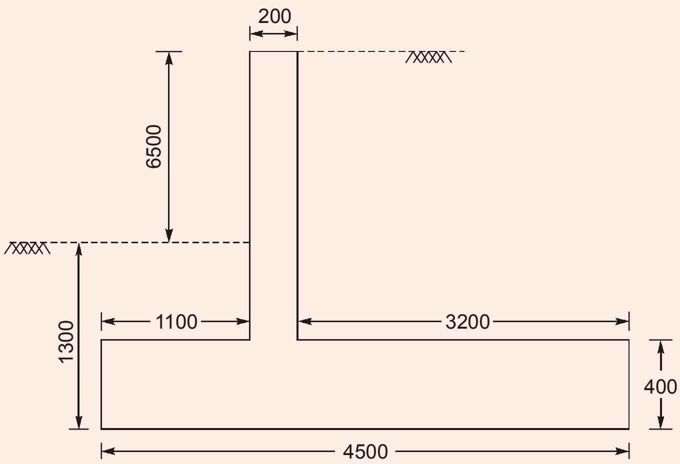
It means that the frequency is inversely proportional to the time of vibration. In other words, if frequency is more, the consolidation of concrete will be achieved in less time and vice-versa.

MADE EASY Source

- **Theory Book:** Construction Practice, Planning and Management (Page No. 116)

End of Solution

7. (a) Design the counterforts of a retaining wall to retain earth for a height of 6.5 m 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 3.5 m. Assume a cover of 40 mm for counterforts.



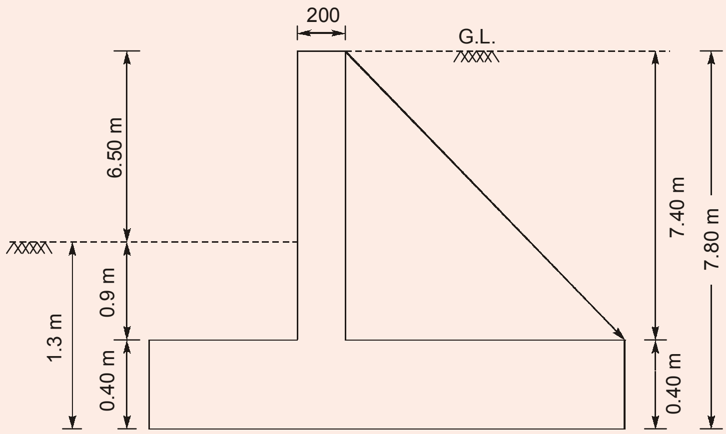
All dimension are in mm.

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 Marks]

Solution:

Design of counterfort



(a) Unit weight of soil, $\gamma = 16\ kN/m^3$

Angle of repose, $\phi = 30^\circ$

Safe bearing capacity of soil = $180\ kN/m^2$

M20, Fe415

Spacing of counterfort = 3.50 m

Cover = 40 mm

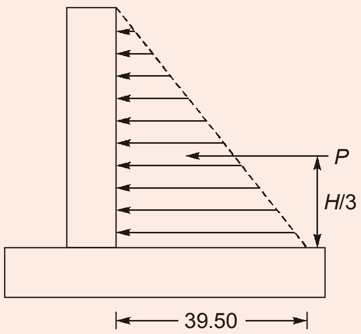
Each counterfort will receive a total pressure from 3.50 m width of soil.

Earth pressure, $$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = \frac{1}{3}$$

(b) Earth pressure at base of stem

$$= \frac{1}{3} \times 16 \times 7.40 = 39.50\ kN/m^2$$

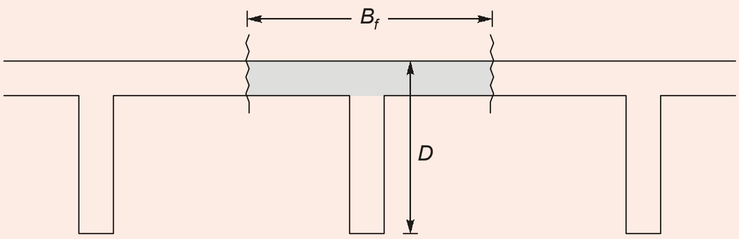
(c) Maximum bending moment at base of stem to be considered for each counterfort



$$BM = \frac{1}{2} \times 39.50 \times \frac{7.40^2}{3} = 360.503\ kNm$$

$$M_u = 1.50 \times 360.503 \times 3.50 = 1892.80\ kN/m$$

The counterforts are designed as T-beams with stem portion acting as slab and counterforts.



(d) Assume width of counterforts = 500 mm

Effective width of flange,

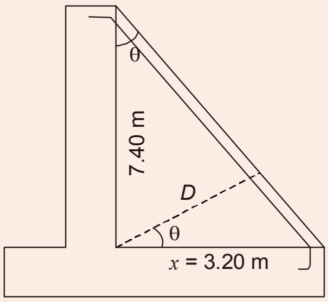
$$B_f = \frac{L_0}{6} + b_w + 6d_f = \frac{7400}{6} + 500 + 6 \times 200 = 2933\ mm$$

$B_{f\ max}$ = 3500 mm (Consider B_f = 2933 mm)

(c/c distance between counterforts)

(e) $$\tan\theta = \frac{3200}{7400} = 0.4324$$

$$\theta = 23.4°$$



Overall depth of section perpendicular to reinforced.

$$D = x\cos\theta$$
$$= 3200 \times \cos 23.4° = 2937 \text{ mm}$$

$$\text{Effective cover} = \frac{20}{2} + 40 = 50 \text{ mm}$$

$$d = 2937 - 50 = 2887 \text{ mm}$$

(g) **Depth of NA**

Assuming NA lies in flange portion of T-beam (Stem)

Equating

$$BM_u = 0.36 \times f_{ck} \times B_f \times x_u (d - 0.42\, x_u)$$
$$1892.8 \times 10^6 = 0.36 \times 20 \times 2933 \times x_u \times (2887 - 0.42\, x_u)$$
$$0.42x_u^{\,2} - 2887x_u + 89631.4 = 0$$
$$x_u = 31.2 \text{ mm}, < 200 \text{ mm}$$

Assumption is correct.

(h) **Area of steel**

$$A_{st} = \frac{BM_u}{0.87f_y(d - 0.42x_u)}$$
$$= \frac{1892.8 \times 10^6}{0.87 \times 415(2887 - 0.42 \times 31.2)}$$
$$= 1824.2 \text{ mm}^2$$

$$\text{Nos. of 28 mm } \phi \text{ bars} = \frac{2957}{\frac{\pi}{4} \times (28)^2} = 4.80$$

$$\left[\text{Minimum steel} = \frac{0.85}{415} \times 2887 \times 500 = 2957 \text{ mm}^2\right]$$

Say 5-28 mm ϕ bars.

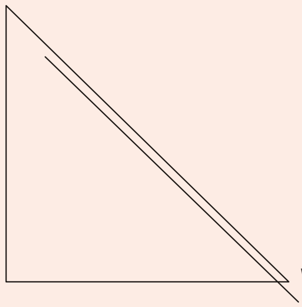
(i) **Check for shear**

Maximum shear force at base

$$V_u = 1.5 \text{ P}$$

$$= 1.5 \times \frac{1}{2} \times 39.50 \times 7.40 \times 3.50$$

$$= 767.30 \text{ kN}$$



The section is varying, (depth and moment are measuring in same direction)

$$V_{u_e} = V_u - \frac{M_u}{d} \tan\theta$$

$$= 767.30 - \frac{1892.80}{2.887} \times \tan 23.4°$$

$$= 483.6 \text{ kN}$$

Nominal shear stress, $\tau_v = \frac{V_{ue}}{Bd} = \frac{483.6 \times 1000}{2887 \times 500} = 0.34 \text{ N/mm}^2$

Percentage of steel $= \frac{5 \times \frac{\pi}{4} \times (28)^2}{2880 \times 500} \times 100 = 0.205\%$

$$\tau_c = 0.28 + \frac{(0.36 - 0.28)}{(0.25 - 0.15)} \times (0.205 - 0.15)$$

$$= 0.324 \text{ N/mm}^2$$

$(\tau_v - \tau_c) = 0.34 - 0.32 = 0.02$ (Very less)

So provide minimum shear reinforced as per

$$\frac{A_{sv}}{BS_v} \geq \frac{0.40}{0.87 f_y}$$

Spacing 2-legged 10 mm ϕ stirrups.

$$\frac{2 \times \frac{\pi}{4} \times (10)^2}{500 \times S_v} \geq \frac{0.40}{0.87 \times 415}$$

$$S_v \leq 283 \text{ mm}$$

Provide 2-legged 10 mm ϕ @ 280 mm c/c

(j) Tie connection

Tension force on bottom of counterfort

$$T = 39.50 \times 3.50 = 138.25 \text{ kN/m}$$

$$T_u = 1.5 \times 138.25 = 207.40 \text{ kN}$$

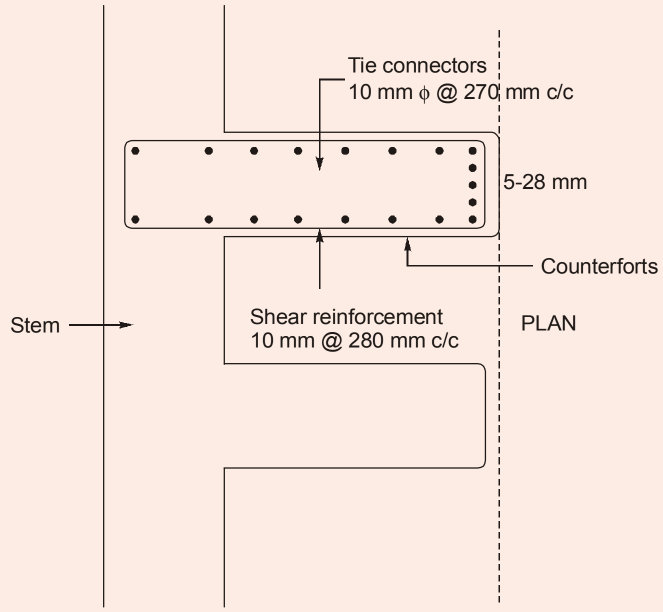
Area of steel required

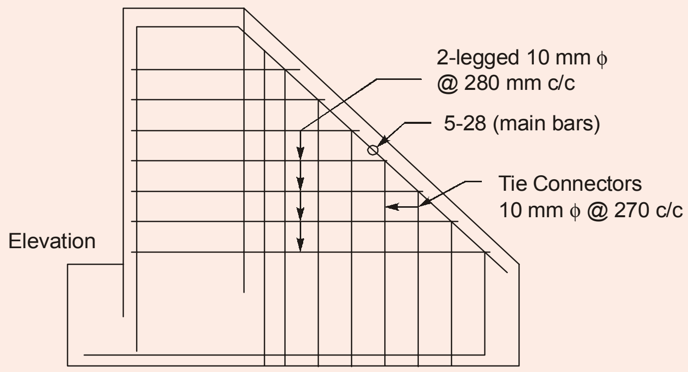
$$A_{st_v} = \frac{207.40 \times 100}{0.87 \times 415} = 574.4 \text{ mm}^2$$

Spacing of 10 mm ϕ bars in two layer.

$$= \frac{1000}{(A_{st}/2)} \times \frac{\pi}{4} \times (10)^2 = \frac{1000}{\left(\frac{574.4}{2}\right)} \times \frac{\pi}{4} \times (10)^2 = 273 \text{ mm}$$

Provide 10 mm ϕ @ 270 mm c/c.





End of Solution

7. (b) Design the side walls of an underground tank of size 12 m × 3 m × 3 m deep. The 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 Fe415 grade of steel. Take Q = 1.156 N/mm^2 and J = 0.87.

[20 Marks]

Solution:

Design of long wall

$$\frac{L}{B} = \frac{12}{3} = 4 > 2$$

So long walls will be designed as cantilever.

(a) Tank empty with pressure of saturated soil from outside

$$p_a = k_a\gamma' H + \gamma_w H$$

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

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

$$p_a = \frac{1}{3}\times 7193\times 3 + 9810\times 3 = 36620 \text{ N/m}^2$$

$$\text{BM at base of wall} = 36620\times\frac{3}{2}\times\frac{3}{3} = 54.93 \text{ kNm}$$

$$d = \sqrt{\frac{54.93\times 10^6}{1.156\times 1000}} = 218 \text{ mm}$$

Provide total depth, $T = 260$ mm so that

$$d = 260 - 35 = 225 \text{ m}$$

$$A_{st} = \frac{54.93\times 10^6}{Jd\sigma_{st}} = \frac{54.93\times 10^6}{130\times 0.84\times 225}$$

$$= 2158.56 \text{ mm}^2$$

$$\text{Spacing of 16 mm } \phi \text{ bar} = \frac{1000+201.1}{2158.56} = 93.1 \text{ mm c/c}$$

Distribution steel

$$\% \text{ distribution steel} = 0.3 - 0.1\times\frac{(260-100)}{450-100} = 0.254\%$$

$$A_{std} = \frac{0.254\times 260\times 1000}{100} = 660.4 \text{ mm}^2$$

$$\text{Area on each face} = 330.2 \text{ mm}^2$$

$$\text{Spacing of 8 mm } \phi = \frac{1000\times 50.26}{330.2} = 152.2 \text{ mm}$$

So provide 8 mm @ 150 mm c/c on each face.

(b) Tank full with water and no earth fill outside

$$p = \gamma_w H = 9810 \times 3 = 29430 \text{ N/m}^2$$

$$m = P\times\frac{H}{2}\times\frac{H}{3} = 29430\times\frac{3}{2}\times\frac{3}{3} = 44.145 \text{ kNm}$$

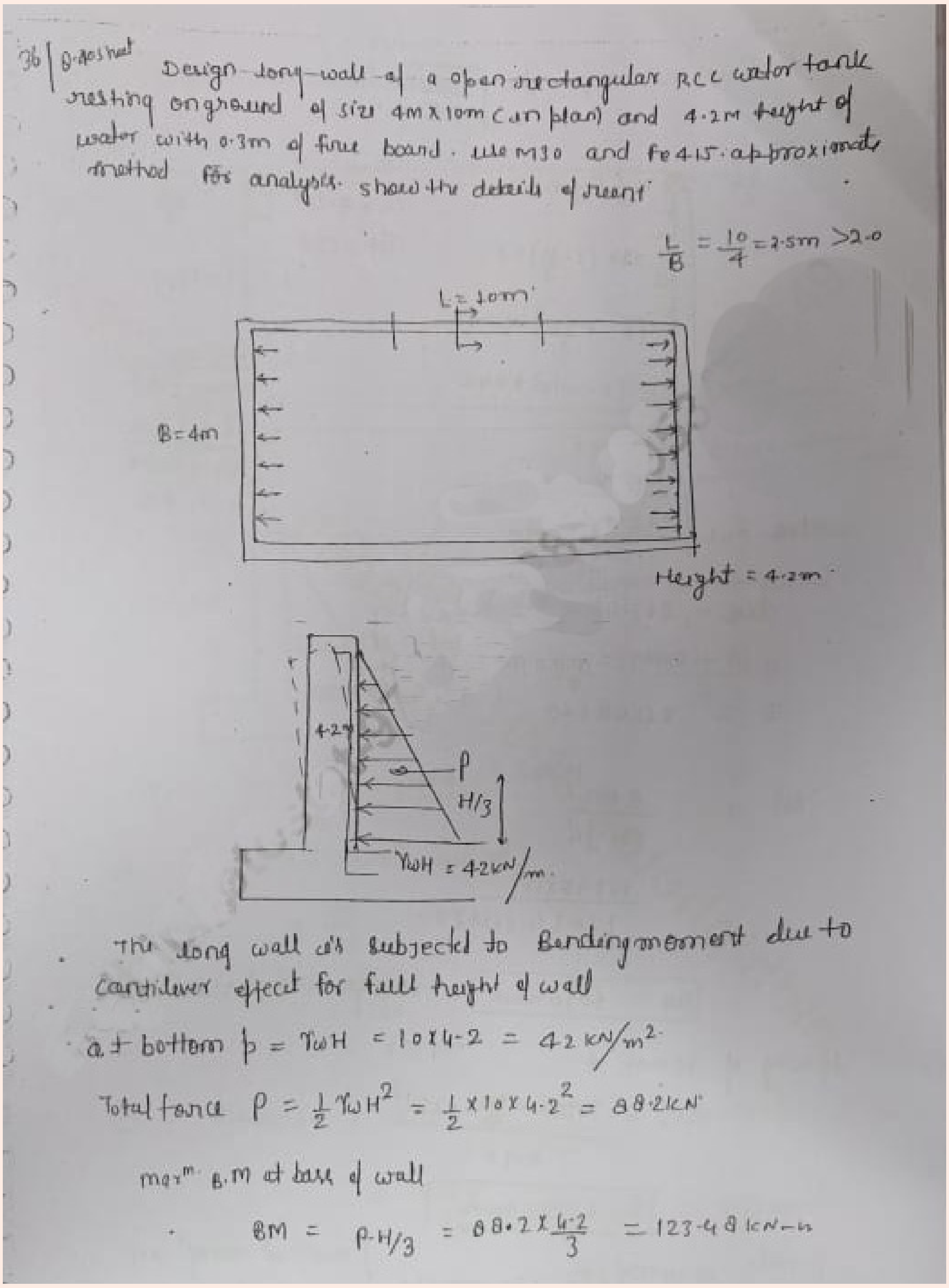
$$A_{st} = \frac{44.145\times10^6}{130\times0.87\times225} = 1734.75\ \text{mm}^2$$

$$\text{Spacing of 16 mm } \phi \text{ bar} = \frac{1000\times201.1}{1734.75} = 115.92\ \text{mm}$$

Provide 110 mm c/c at inside face.

MADE EASY Source

- **MADE EASY Classnotes.**



36 | Q.Posted Design long wall of a open rectangular RCC water tank resting on ground of size 4m x 10m (in plan) and 4.2m height of water with 0.3m of free board. use M30 and Fe415. approximate method for analysis. show the details of reinf.

$\frac{L}{B} = \frac{10}{4} = 2.5m > 2.0$

The long wall is subjected to Bending moment due to cantilever effect for full height of wall

at bottom $p = \gamma_w H = 10\times4.2 = 42\ kN/m^2$

Total force $P = \frac{1}{2}\gamma_w H^2 = \frac{1}{2}\times10\times4.2^2 = 88.2\ kN$

max^m B.M at base of wall

$BM = P\cdot H/3 = 88.2\times\frac{4.2}{3} = 123.48\ kN\text{-}m$

min^m thickness req. $d = \sqrt{\frac{BM}{Q \cdot B}}$

$\sigma_{cbc} = 10$
$m = 9.33$
$\sigma_{st} = 130$

$Q = \frac{1}{2} C \cdot j \cdot k$

$= \frac{1}{2} \times 10 \left(1 - \frac{k}{3}\right) \times k$

$j = (1 - k/3)$

$= 5 \times \left(1 - \frac{0.42}{3}\right) \times 0.42$

$K = \frac{m \times \sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}}$

$= \frac{9.33 \times 10}{93.3 + 130}$

$= 0.42$

$= 5\,(1 - 0.14) \times 0.42$

$Q = 1.806$

$d_{req} = \sqrt{\frac{123.48 \times 10^6}{1.806 \times 1000}}$

$d_{req} \approx 261.48$ mm $\simeq 270$

cover $= +60$ mm

$270 + 60 = 330$ mm

$D = 261.48 + 60$

$A_{st} = \frac{BM}{\sigma_{st} \cdot j d}$

$= \frac{123.48 \times 10^6}{130 \times (0.86) \times 270}$

$A_{st} = 4090$ mm^2 (Design vertical reinf^n)

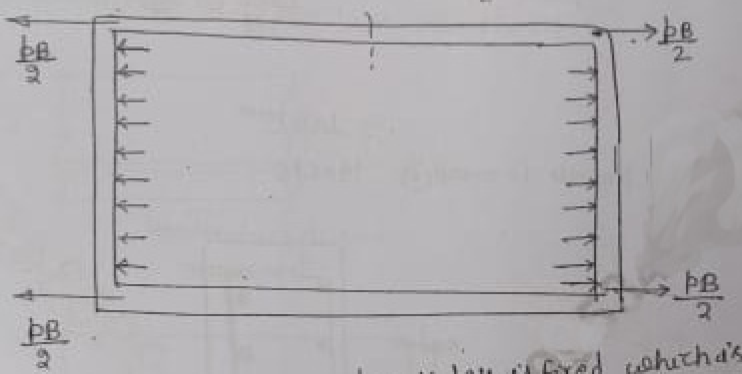
Spacing of 25mm ϕ

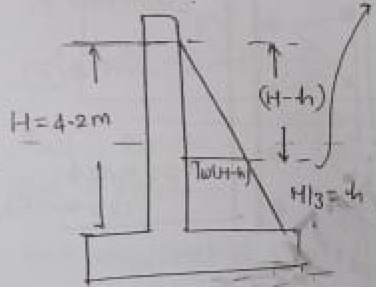
$S = \frac{1000 \times \frac{\pi}{4} \times (25)^2}{4090}$

$S = 120$ mm

provide 25mm ϕ @ 120 mm c/c {vertical reinf^n on water face}

(B) Design of Horizontal reinf^n





because base is fixed which is not more Direct pull on long wall for 1m height (in horizontal dir^n

$T_1 = \frac{pB}{2} = \frac{\gamma_w (H-h)}{2} B$

$T_1 = \frac{10 \times (4.2 - 1.40) \times 4}{2} = 56\text{kN}$

(i) Horizontal reinf^n required.

$= \frac{T_1}{f_{st}}$

$= \frac{56000}{130}$

$= 431\text{mm}^2$

(ii) $Ast_{min}\% = 0.3 - \frac{(0.3 - 0.2)}{(450 - 100)} \times (330 - 100)$

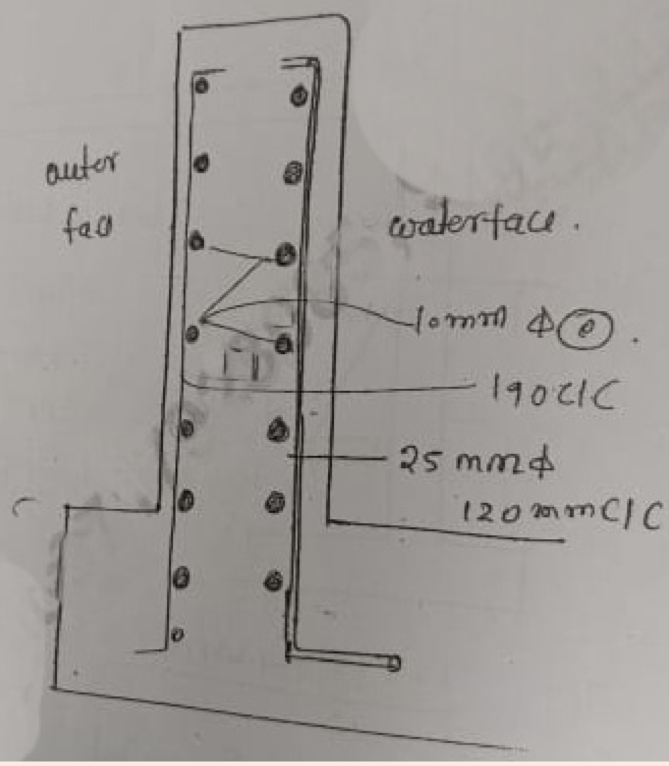
$= \frac{0.24}{100} \times 1000 \times 330 = 792\text{mm}^2$

Area of steel on each $= \frac{792}{2} = 396\ mm^2$.

Spacing of 10 mm $\phi = \frac{1000}{396} \times \frac{\pi}{4} \times (10)^2$

$= 198\ mm$

provide 10 mmϕ @ 190 c/c



End of Solution

7. (c) Explain the different types of contracts adopted in construction.

[20 Marks]

Solution:

Project Contract

- A contract is a legal agreement. It is an exchange of promises by two or more persons/organisations.

- Section 2 *(b)* of the Indian Contract Act, 1872 defines contract as "an agreement enforceable by law". The definition given by the Indian Contract Act has two distinct parts, *viz.,*
  (a) a contract is an agreement
  (b) the agreement must be enforceable by law.
- Though contract is necessarily an agreement, but all agreements are not contracts. Only those agreements that are enforceable by law are contracts. For an agreement to become enforceable by law, it must satisfy certain essential elements called "essentials" of valid contract".
- The contractor is a person, a firm or a company who undertakes contract.

Type of Contracts

Broadly contracts can be divided into two types. They are:

(a) Turnkey contracts

(b) Non-turnkey contracts

(a) Turnkey Contracts

- In a turnkey contract, the entire responsibility of project execution is entrusted to the contractor. It is as if the owner comes into the picture only when the project is completed and he turns the key of the plant to start production. Till such time the project reaches a 'ready-to-start' stage, the contractor takes care of all aspects of project execution.
- Since the contractor of a turnkey project is expected to do everything right from scratch, the scope of contract covers all areas of the project, *viz.,* design, engineering, construction, structural work, supply and erection of plant and machinery, supply of spares, testing and commissioning. Since the entire work is done by a single contractor, turnkey contract agreements invariably have a clause on performance guarantee.
- Turnkey contracts may be for the whole project or even for a sub-unit of a main project. Turnkey contract for a sub-unit is undertaken when such sub-unit is a separately identifiable component of the total project and whose performance can be assessed independent of other units of the project.

(b) Non-turnkey Contracts

- Non-turnkey contracts are preferred when the projects are small sized, the know-how for the project is available with the promoters and when there is a strong, competent and capable project team available with the organisation.
- In non-turnkey arrangement, the total project is divided into suitable work packages. The packages that need to be given to contractors are identified. A contractor may be entrusted with one or more work packages. However, the responsibility for overall supervision and performance of the project will rest with the in-house project department.

The following are some of the non-turnkey contracts:

1. **Lumpsum contract (Fixed price):** This is a single fixed price contact. In this contact, contractor agrees to perform specified job for fixed sum. The owner

provides the contractor exact specification of the work. In this contract following are the advantages of the fixed price contract.

(a) Owner in aware of the cost of the project before the project construction starts.
(b) It avoids a lot of details and accounting by both owner and contractor
(c) Contractor gets free hand to execute the work
(d) In this contract is used with design contract method of delivery, contractor gets opportunity to use value engineering.

In this contract following are the disadvantages of the fixed price contract.

(a) It is very difficult to accommodate any change and specification.
(b) This contract is as good as the accuracy of the contract document. If errors exist in the contract document, the contract need to be renegotiated and hence more risk in involved from the owner side.
(c) In the case of unforeseen hazard during the construction, contractor may be put in adverse situation.

Payment by the owner can be carried out in lump-sum contract as total amount at time or percentage of total cost after finishing certain amount of work. For example suppose a water tank was awarded as lump-sum contract. It can be said that 15% of amount will be paid after construction of foundation 50% of total amount will be paid after construction of staging and 80% of the total amount will be after the construction of the tank. Rest will be paid only when the tank becomes operational. Lump-sum contract in often used in sub-contracting for labour contract. In our country many labourers for excavation, plastering work with this method.

2. **Unit price contract:** In this type of contract, the price is paid per unit of the work carried out. There are different variations of this type of contract. Some of them are mentioned below.

Bill of quantities contract: In this type of contract owner provide the drawing, quantities of work to be done and specification. The contractor bid based on the unit cost of the items of construction. The contractor overhead, profit and other expanses can be included in the unit cost of the item of work. Sometimes contractor quotes the unit price of the work and lump-sum amount separately as profit overhead. The estimated quantities of the work to be done called Bill of the quantities is fixed.

This type of construction is usually followed in government sector for large infrastructure construction. This type of contract provides owner a competitive bid. Disadvantages of the methods are:

(a) Owner needs to measure the quantity of work done in the field, hence requires owner presence at the site.
(b) Final price of the construction is not known precisely until last price of work is completed. If there is significant difference between the estimated quantities and the reality of the situation, owner is put in adverse situation. Mistaken quantities is called unbalanced bid. Significant unbalanced bid now considered as unethical.

Schedule of rate contract: Many a time, the quantity of work to be executed is not known before. Contract is signed based on the unit cost of the item of work. Generally more items are inserted in the contract than to be executed because it becomes sometimes difficult to exactly specify all the items. There is no guarantee that all the items mentioned will be used in the construction. This type of contact are widely used in underground work, flood control and road constructions. Advantages and disadvantages of this type of contract in the same as the bill qualities contract. There are other variations of schedule of rate contract where unit price plus profit is charged as the cost.

(c) **Cost plus contract:** In this contract, the payment is made based on the work carried out plus the free which includes overhead, profit etc. Sometimes a cap is put on the type of contract by provided maximum and minimum cost limit such as guaranteed maximum cost contract. If project cost exceed this limit, contractor is responsible for that.

Sometimes incentive clause is also included if the contractor bring the project before certain specified limit.

The advantages of this type of contract is that considerable overlap is provide between design and construction. Hence the project can be executed in the fast-tract basis. This contract is suitable for the work where it is difficult to define the task to be done before the awarding the contract.

3. **Item rate contract:** Item rate contract is also known as unit price contract or schedule contract. A contractor undertakes the execution of work on an item rate basis. He is required to quote rate for individual item of work on the basis of schedule of quantities (i.e., bill of quantities) furnished by the department. The amount to be received by the contractor, depends upon the quantities of work actually performed. The payment to the contractor is made on the basis of the detailed measurements of different items of work actually executed by him.

Suitability: The item rate contract is most commonly used for all types of engineering works of the government undertakings including railway department. It is suitable for works which can be distinctly split into various items and quantities under each item can be estimated accurately.

MADE EASY Source

- **ESE 2019 Mains Test Series:** Q.6(b) (i) of Test-10
- **Theory Book:** Construction Practice, Planning and Management (Page No. 43)
- **Theory Book:** Basics of Project Management (Page No. 145)

End of Solution

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

[20 Marks]

Solution:

Planning is the most important technique of management . In a construction project plan includes the estimates, the budget and time schedule and sequences of completion of each part of the project, manpower planning and the plant and equipments.

Following steps are involved in effective planning:

(i) **Crystalizing the opportunity or problem:** The first step in planning would be to find out the problem or identify the opportunity to be seized, this is necessary to be able to formulate practical and realistic objectives.

(ii) **Securing and analyzing necessary information:** Adequate information is required on course of action possible. It is necessary to determine the nature of the information required and where this information will be available. This information must be analysed to establish the relationship and tabulate them for adequate interpretation.

(iii) **Establishing planning premises and constraints:** An analysis of the data so collected will result in the formulation of certain assumptions on the basis of which the plan will be made through a process of forecasting. Constraints such as government control will also exist. Planning will be in the backdrop of such premises and constraints which must be watched to detect changes and their effect on the plans.

(iv) **Ascertaining alternative course of action or plan:** Based on the above analysis, possible alternative course of action will be identified and examined. Generally, every situation will have more than one course of action. Exploitation of the right course will depend to a large content on experience, ingenuity and imagination of the planner.

(v) **Selecting optimum plan:** An evaluation of the above alternate course of action can be carried out either by judgement alone or with the help of quantitative techniques and staff assistant, to best suit the interest of the organisation.

(vi) **Determining derivative plan:** The above selected plan will form the basic plan from which other plans will develop to support it. For example, basic marketing plan may have been evolved which may result in other derivative plans such as the advertising plan.

(vii) **Fixing the timing of introduction:** The question of timing-who will do, what will have to be decided and an appropriate time schedule drawn up with the details of construction work for communication.

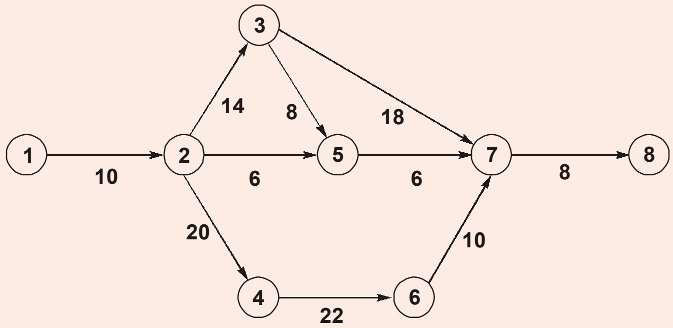
(viii) **Arranging future evaluation of effectiveness of the plan:** Since the ultimate aim of the plan to achieve the objective, result or goal, an evaluation at the earliest possible opportunity necessary to evaluate the adequacy of cost and time and determining whether the planned objectives are reached as desired.

MADE EASY Source

- **Theory Book:** Basics of Project Management (Page No. 77)

End of Solution

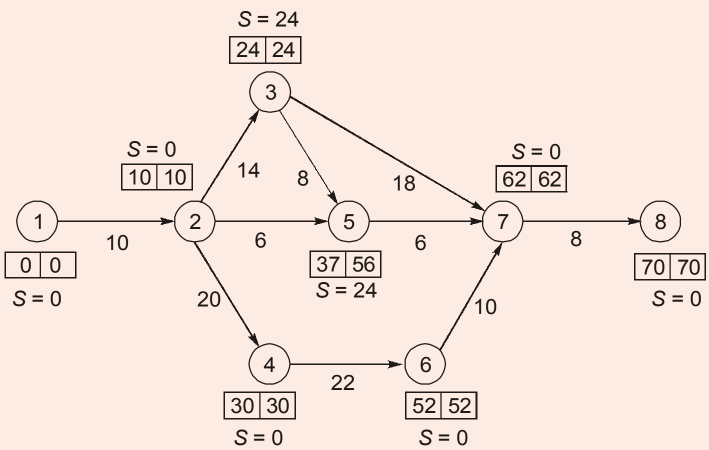
8. (b)



Identify the critical path in the network as shown in figure and determine the project completion time. The duration are in weeks.

[20 Marks]

Solution:



Notation

T_E	T_L

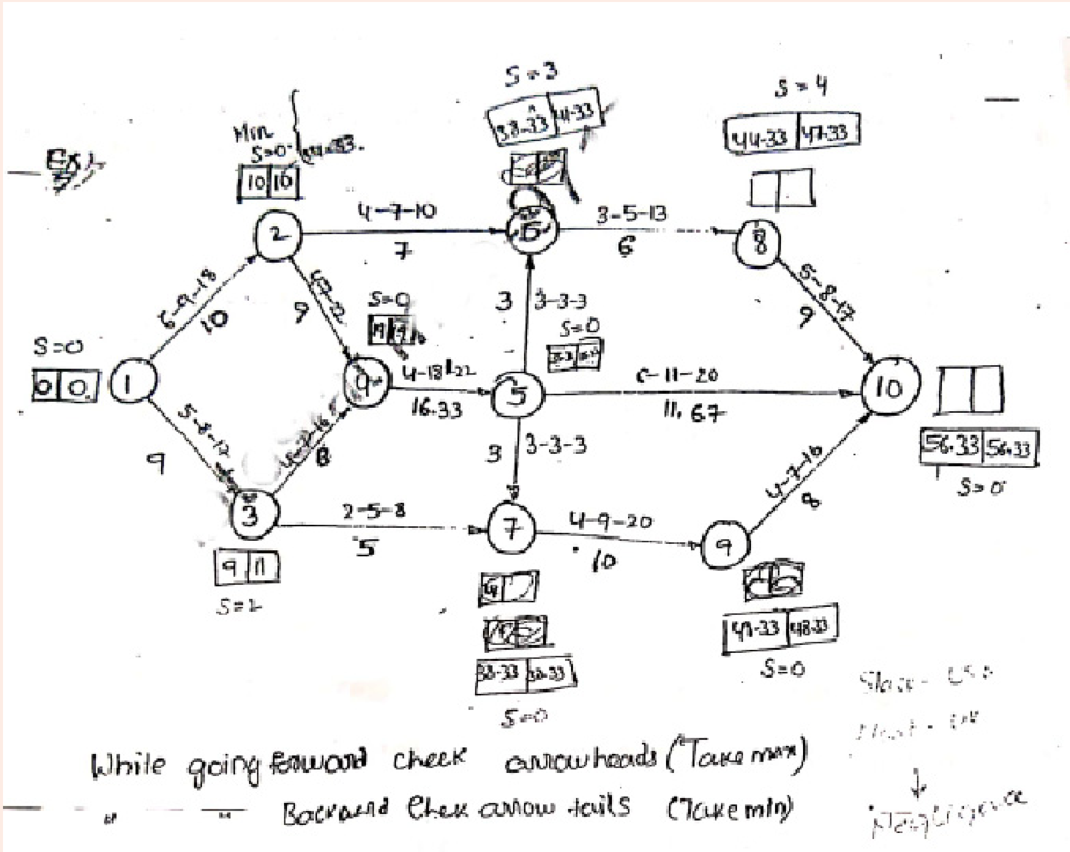
Slack = S = TL − F_E

Project duration = 70 weeks

Critical path = 1–2–4–6–7–8

MADE EASY Source

- **Theory Book:** Construction Practice, Planning and Management (Ex. 2.6, Page No. 22)
- **MADE EASY Classnotes**



End of Solution

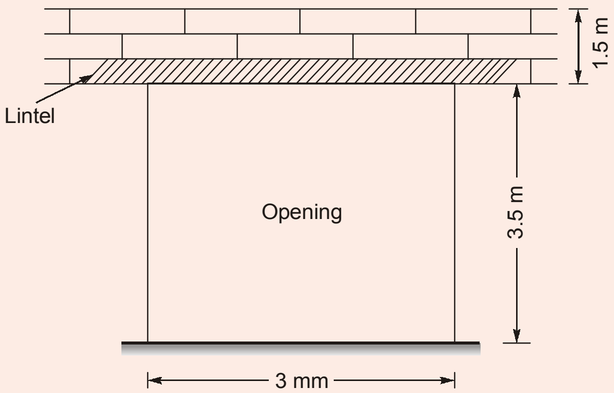
8. (c) The opening of a masonry building is 3 m and 3.5 m high. The ceiling of the roof is 4.5 m above the floor. The space between top of lintel and bottom of roof is filled with brick masonry. The roof transmits a total load of 25 kN/m run to the lintel. Design the lintel supported on brick walls of 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.

$\frac{100 A_s}{bd}$	τ_c N/mm^2 M 20
≤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/mm² for M20 grade of concrete

[20 Marks]

Solution:



Dead weight of brick = 1 × 0.3 × 20 = 6 kN/m

$$\text{Effective depth of lintel} = \frac{3000}{10} = 300 \text{ mm}$$

Total depth = 300 + 30 = 330 mm

Dead weight of lintel = 0.33 × 0.3 × 25 = 2.475 kN/m

Total load = 25 + 6 + 2.475 = 33.475 kN/m

w_u = 33.475 × 1.5 = 50.21 kN/m

Effective length = 3 + 0.3 = 3.3 m

$$M_u = \frac{w_u L^2}{8} = \frac{50.21 \times 3.3^2}{8} = 68.35 \text{ kN/m}$$

Check depth:

$$d = \sqrt{\frac{68.35 \times 10^6}{0.138 \times 20 \times 300}} = 284.32 < 300 \text{ mm} \qquad \text{(OK)}$$

$$A_{st} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 68.35 \times 10^6}{20 \times 300 \times 300^2}}\right] \times 300 \times 300$$

$$= 766.97 \text{ mm}^2$$

Provide 4 bars of 16 mm ϕ.

Check for shear

$$SF = \frac{wL}{2} = \frac{50.21 \times 3}{2} = 76.82 \text{ kN}$$

$$\tau_V = \frac{76.82 \times 10^3}{300 \times 300} = 0.854 \text{ N/mm}^2$$

$$P_t = \frac{100 \times \frac{\pi}{4} \times 16^2 \times 4}{300 \times 300} = 0.9\%$$

$$\tau_c = 0.56 + \frac{0.62 - 0.56}{1 - 0.75}[0.92 - 0.75] = 0.60 \text{ MPa}$$

$$\tau_v - \tau_c = 0.854 - 0.6 = 0.254 < 0.4 \text{ MPa}$$

So minimum shear reinforcement must be provided.

Spacing of 8 mm 2 legged bar

$$S_v = \frac{0.87 f_y A_{sv} d}{0.4 bd} = 300.875 \text{ mm}$$

$$= 0.75 \times 300 = 225 \text{ mm}$$

So provide 2L–8 mm ϕ @ 220 mm c/c

Check for bond:

$$\frac{1.3 M_u}{V_1} \geq L_d$$

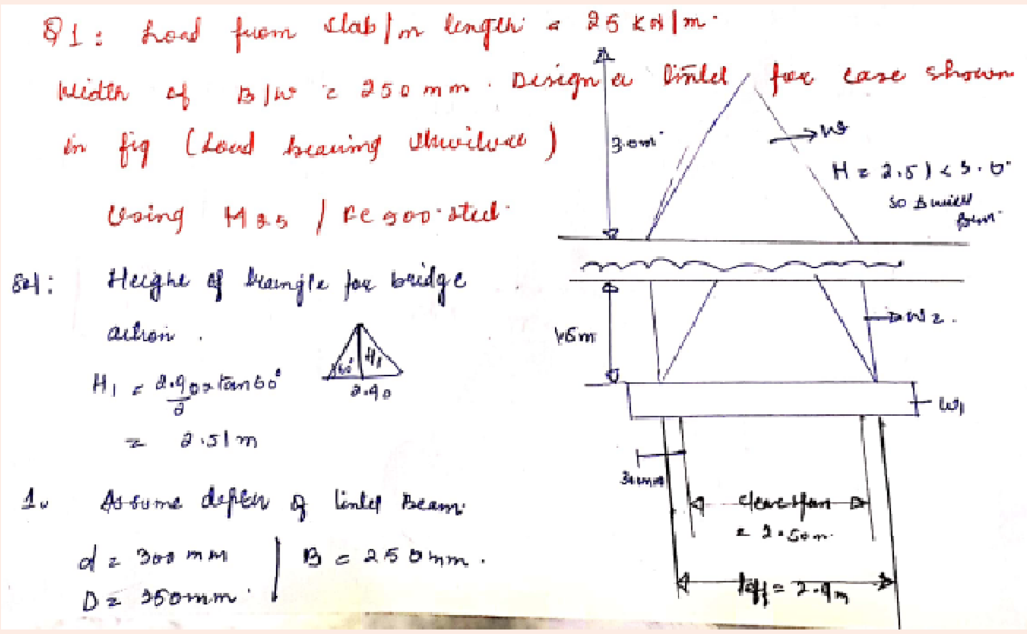
$$\frac{1.3 \times 68.35 \times 1000}{76.82} \geq \frac{0.87 \times 415 \times 16}{4 \times 1.6 \times 1.2}$$

$$11.56.66 \text{ mm} \geq 752.19 \text{ mm}$$ (OK)

Safe in bond.

MADE EASY Source

- **Conventional Practice Question Book:** (Q.21, Page 222)
- **MADE EASY Classnotes**


Q1: Load from slab/m length = 25 kN/m. Width of B/W = 250 mm. Design a lintel for case shown in fig (Load bearing structure). Using M25 / Fe 500 steel.

Sol: Height of triangle for bridge action.

$H_1 = \frac{2.90 \times \tan 60°}{2}$

$= 2.51$ m

1. Assume depth of lintel beam.

d = 300 mm | B = 250 mm.

D = 350 mm.

1. DL (lintel) $= 0.23 \times 0.35 \times 1 \times 25$

$= 2.20$ kN/m $= w_1$.

2. B/W load $= .25 \times 1.6 \times 1 \times 20$

$= 8.0$ kN/m $= w_2$.

3. Load from slab $= 25$ kN/m.

4. Load of sular portion

$= W = \frac{1}{2} \times 2.90 \times 2.51 \times 0.25 \times 20$

$= 18.20$ kN.

2. Effective span $L_{eff} = 2.6 + .3 = 2.90$ m.

3. Max BM

$$BM_u = 1.5 \times \left[(w_1 + w_2 + w_3) \frac{l_e^2}{8} + \frac{W \cdot L_e}{6} \right].$$

$$= 1.5 \times \left[(2.20 + 8 + 25) \times \frac{2.90^2}{8} + \frac{18.20 \times 2.90}{6} \right]$$

$= 68.7$ kN-m.

3. Effective depth reqd. (Moment).

$$d = \sqrt{\frac{BM_u}{Q \cdot B}} = \sqrt{\frac{68.7 \times 10^6}{0.133 \times 25 \times 250}}$$

$= 287$.

4. Effective depth 'd' (deflection) $= \frac{\text{Span}}{A} = \frac{2900}{20} = 145$ mm.

Provide $d = 300$ mm. $D = 350$ mm.

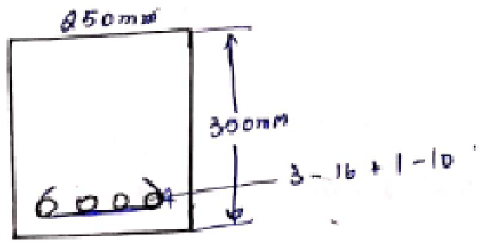
5. Area of steel

$$A_{st} = \frac{0.5 \times 25}{500} \times \left[1 - \sqrt{1 - \frac{4.6 \times 68.7 \times 10^6}{25 \times 250 \times 300^2}} \right] \times 250 \times 300.$$

$= 654$ mm².

no. of 16 mm ϕ = $\dfrac{634}{\frac{\pi}{4}(16)^2}$

Provide
3 - 16 = 603
1 - 10 = 78
681 mm^2



End of Solution

■■■■■