GATE WALLAH

ESE-2024

MAIN EXAM DETAILED SOLUTION

CIVIL ENGINEERING

PAPER-I

EXAM DATE - 23 JUNE 2024

9 : 00 AM TO 12 : 00 PM

SECTION-A

Q.1. (a) (i) Describe how stones are preserved.
(ii) What are the advantages and disadvantages of fibre reinforced concrete ?

Sol.

(i)

PRESERVATION OF STONES

Preservation of stone is essential to prevent its decay. Different types of stones require different treatments. But in general stones should be made dry with the help of blow lamp and then a coating of paraffin, linseed oil, light paint, etc. is applied over the surface. This makes a protective coating over the stone. However, this treatment is periodic and not permanent. When treatment is done with the linseed oil, it is boiled and applied in three coats over the stone. Thereafter, a coat of dilute ammonia in warm water is applied.

The structure to be preserved should be maintained by washing stones frequently with water and steam so that dirt and salts deposited are removed from time to time. However, the best way is to apply preservatives. Stones are washed with thin solution of silicate of soda or potash. Then, on drying a solution of $CaCl_2$ is applied over it. These two solutions called Szerelmy's liquid, combine to form silicate of lime which fills the pores in stones. The common salt formed in this process is washed afterwards. The silicate of lime forms an insoluble film which helps to protect the stones.

Sometimes lead paint is also used to preserve the stones, but the natural colour of the stone is spoilt. Painting stone with coal tar also helps in the preservation but it spoils the beauty of the stone. Use of chemicals should be avoided as far as possible, especially the caustic alkalis. Although cleaning is easy with chemicals, there is the risk of introducing salts which may subsequently cause damage to the stone.

In industrial towns, stones are preserved by application of solution of baryta, $Ba(OH)_2$ — Barium hydrate. The sulphur dioxide present in acid reacts on the calcium contents of stones to form calcium sulphate. Soot and dust present in the atmosphere adhere to the calcium sulphate and form a hard skin. In due course of time, the calcium sulphate so formed flakes off and exposes fresh stone surface for further attack. This is known as sulphate attack. Baryta reacts with calcium sulphate deposited on the stones and forms insoluble barium sulphate and calcium hydroxide. The calcium hydroxide absorbs carbon dioxide from the air to form calcium carbonate.

$$Ba(OH)_2 + CaSO_4 \longrightarrow \underset{\text{(Barium sulphate)}}{BaSO_4} + \underset{\text{(Calcium hydroxide)}}{Ca(OH)_2}$$

$$Ca(OH)_2 + CO_2 \longrightarrow \underset{\text{(Calcium carbonate)}}{CaCO_3} + H_2O$$

(ii)

Fibre Reinforced Concrete (FRC):

Fibre reinforced concrete is a type of concrete that incorporates fibres to improve its properties. Asbestos cement fibres so far have proved to be commercially successful.

For fibre reinforced concrete to be fully effective, each fibre needs to be fully embedded in the matrix, thus the cement paste requirement is more. For FRC, the cement paste required ranges between 35 to 45 percent as against 25 to 35 percent in conventional concrete.

Advantages of fibre Reinforced Concrete (FRC):

(i) **Strength:** Strength of concrete increases.

(ii) **Reduce Cracking:** Fibre used in FRC helps to reduce cracking and permit the use of thin concrete sections.

(iii) **Cohesive Mix:** The mix becomes cohesive and possibilites of bleeding and segregation are reduced.

(iv) **Ductility:** Ductility is improved in FRC.

(v) **Impact resistance:** Fibre absorbs impact energy, making FRC more resistant to damage from sudden load or blows. Bending and tensile strength improved.

(vi) **Lower Maintainance Cost:** Due to its enhanced crack resistance and durability, FRC may require less frequent maintainance compared to PCC.

Disadvantages of Fibre Reinforced Concrete (FRC):

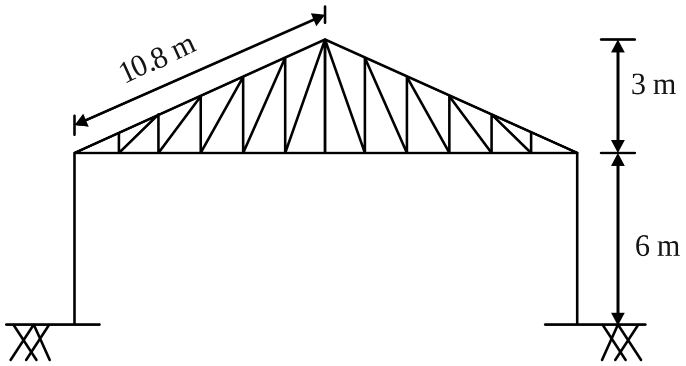
(i) **Reduce Workability:** Fibre reduce workability of a mix and may cause the entrainment of air.

(ii) **Forming of balls during mixing:** Steel fibres tends to intermes and form balls during the mixing of concrete.

(iii) **Increased Cost:** Adding fibres to the concrete mix increases the material cost compared to plain cement concrete.

(iv) **Selection of right fibre type:** Choosing the right type and amount of fibre for the specific application is crucial. Incorrect selection can reduce the intended benefits.

Q.1. (b) A factory shed of size 40 m × 18 m is to be constructed at New Delhi with roof trusses. Calculate the nodal wind force on a roof truss of span 18 m and central rise of 3 m. The spacing of roof truss is 4 m, basic wind speed = 50 m/s, $K_1 = 1$, $K_2 = 0{\cdot}9$ (up to 10 m ht and category-II), $K_3 = 1$, $K_4 = 1$, $K_a = 0{\cdot}9$, $K_c = 0{\cdot}9$, $K_d = 0{\cdot}9$. For wind angle 0°, take Cpe = – 1·2 (windward side), Cpe = – 0·4 (leeward side). Assume wall opening is less than 5%. Purlins are located at the node points. Show the forces in a sketch of truss for wind angle 0° only.



Sol.

Given

(40 m × 18 m)

l = 18 m

h = 3 m

b = 4 m

$V_b = 50$ m/s

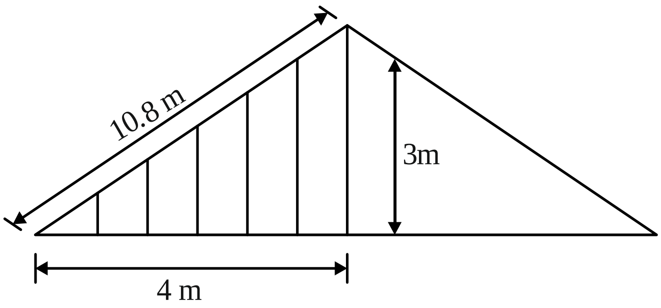
$k_1 = 1$

$k_2 = 0.9$

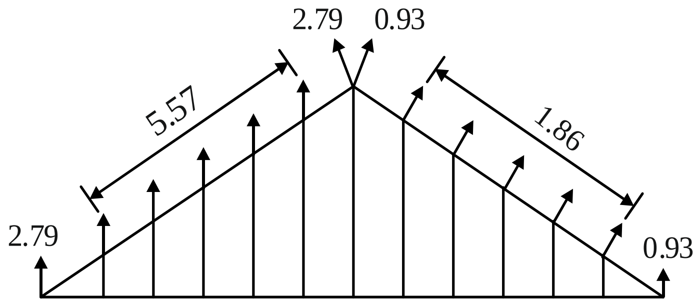
$k_4 = 1$

$k_a = 0.9$

$k_c = 0.9$
$k_d = 0.9$
$\theta = 0°$
$C_{pc} = -1.2$ windward side
$C_{pc} = -0.4$ leeward side
Design wind speed = $V_z = k_1k_2k_3k_4k_ak_ck_d \times V_b$
$= 1 \times 0.9 \times 1 \times 1 \times 0.9 \times 0.9 \times 0.9 \times 50$
$= 32.8$ m/s
Design wind pressure $= 0.6V_z^2$
$= 0.6\ (32.8)^2$
$= 645.5\ \text{N/m}^2 = 0.645\ \text{kN/m}^2$
The wind force is given by = $(C_{pc} - C_{pi})p_dA$



$a = \frac{10.8}{6} = 1.8m$ for area < 5%, $C_{pi} = 0$
b = 4 m
→ at intermediate point
at windward side $= C_{P_e} \times P_d \times A$
$= -1.2 \times 0.645 \times 4 \times 1.8$
$= -10.5$ kN (5.57 kN)
at leeward side $= C_{P_c} \times P_d \times A$
$= -0.4 \times 0.645 \times 4 \times 1.8$
$= -3.5$ kN (1.86 kN)
→ at end points
at windward side $= \frac{1}{2} \times C_{P_e} \times P_d \times A$
$= \frac{1}{2} \times 1.2 \times 0.645 \times 4 \times 1.8$
$= -5.25$ kN (2.79 kN)
at leeward side $= -\frac{1}{2} \times C_{pc} \times P_d \times A$
$= \frac{1}{2} \times 0.4 \times 0.645 \times 4 \times 1.8$
$= -1.75$ kN (0.93 kN)

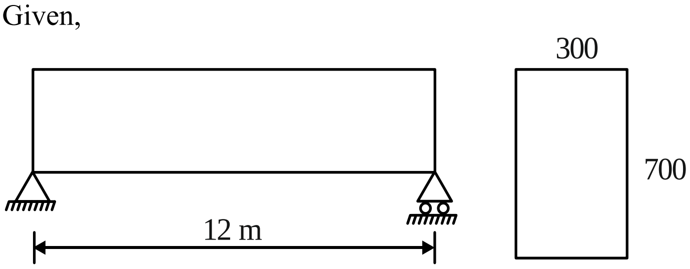


All in kN

Q.1. (c) A simply supported prestressed concrete beam of span 12 m and size 300 × 700 mm carries uniformly distributed load of 20 kN/m. Suggest a suitable cable profile and the prestressing force so that no tension is developed in the beam. Assume density of concrete to be 24 kN/m^3. Maximum eccentricity for the cable to be provided is 200 mm.

Sol.

Given,



[dimension in mm]

$W_L = 20$ kN/m

W_D (Dead Load) = 0.3 × 0.7 × 24 = 5.04 kN/m

$e_{max} = 200$ mm

w = total load = $W_L + W_D = 20 + 5.04 = 25.04$ kN/m

Tendon Profile for udl load:-

For contract the flexure moment due to loading must be balanced by moment due to prestressing force.

Therefore,

$M = Pe_x$

$$\frac{wl^2}{8} = Pe_x$$

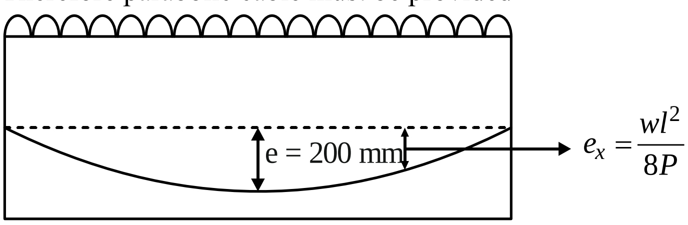
$$\frac{wl^2}{8} = Pe_x$$

$$e_x = \frac{wl^2}{8P}$$

Eccentricity of cable varies square of span

$e_x \propto l^2$

Therefore parabolic cable must be provided



At $x = \frac{L}{2} = \frac{12}{2} = 6\text{ m}$; $e_x = 200$ mm

$$e_x = \frac{wl^2}{8P}$$

$$200 \times 10^{-3} = \frac{25.04 \times 12^2}{8P}$$

P = 2253.6 kN

Max^m B.M at mid-span of beam,

$$M = \frac{wl^2}{8} = \frac{25.04 \times 12^2}{8} = 450.72\text{ kN} - \text{m}$$

The tension may develop at bottom under the action of self-weight and live load, for no tension in the beam at mid-span the required pre-stressing force is

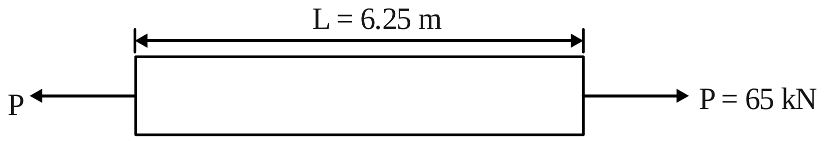
$$f_b = \frac{P}{A} + \frac{Pe}{z} - \frac{M}{z} = 0$$

$$\frac{P}{300 \times 700} + \frac{P(200)}{\frac{300 \times 700^2}{6}} = \frac{450.72 \times 10^6}{\frac{300 \times 700^2}{6}}$$

$P = 1423.48 \times 10^3$ N = 1423.48 kN

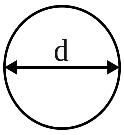
Q.1. (d) A metallic rod 6·25 m long and 35 mm in diameter is subjected to an axial tensile load of 65 kN. Determine the change in dimension and volume of the rod. Assume modulus of elasticity of metal as $E = 2{\cdot}10 \times 10^5$ N/mm^2 and Poisson's ratio = 0·26.

Sol.



Axial Tensile load = P = 65 kN

Length of rod = L = 6·25 m.



d = diameter of rod = 35 mm.

Area of rod $= A = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 35^2 = 962{\cdot}11\text{ mm}^2$.

Modulus of Elasticity = E = $2{\cdot}1 \times 10^5$ N/mm^2.

Poisson's ratio = μ = 0·26.

Axial Longitudinal Stress $= \sigma = \frac{P}{A} = \frac{65 \times 10^3}{962{\cdot}11} = 67{\cdot}56$ N/mm^2.

Axial Longitudinal Strain $= \varepsilon = \frac{\sigma}{E} = \frac{67{\cdot}56}{2{\cdot}1 \times 10^5} = 32{\cdot}17 \times 10^{-5}$

$$\varepsilon = \frac{\Delta L}{L}$$

$\Rightarrow \Delta L =$ Change in length = εL = $32{\cdot}17 \times 10^{-5} \times 6250 = 2.01$ mm

Lateral strain = – μ ε

$= -0{\cdot}26 \times 32{\cdot}17 \times 10^{-5}$

$= -8{\cdot}3642 \times 10^{-5}$

Lateral strain $= \dfrac{\Delta d}{d}$

$\Rightarrow \Delta d =$ Lateral strain × d = Change in diameter

$= -8{\cdot}3642 \times 35 \times 10^{-5}$

$= -2{\cdot}93 \times 10^{-3}$ mm

Volumetric strain $= \varepsilon_v = \dfrac{\sigma}{E}(1-2\mu)$

$= \dfrac{67{\cdot}56}{2{\cdot}1\times10^{5}}(1-2\times0{\cdot}26)$

$= 1{\cdot}544 \times 10^{-4}$

$\varepsilon_v = \dfrac{\Delta V}{V}$

$\Rightarrow \Delta V =$ Change in volume = $\varepsilon_v V = \varepsilon_v AL$

$= 1{\cdot}544 \times 10^{-4} \times 962{\cdot}11 \times 6250$

$= 928{\cdot}44$ mm^3

Change in

(i) Length = 2·01 mm

(ii) diameter = $-2{\cdot}93 \times 10^{-3}$ mm

(iii) Volume = 928·44 mm^3

Q.1. (e) (i) Explain the split tensile test of concrete. How is it related to compressive strength and flexure strength of concrete ?

(ii) What are the effects of air-entraining admixtures on the properties of concrete ?

Sol.

(i)

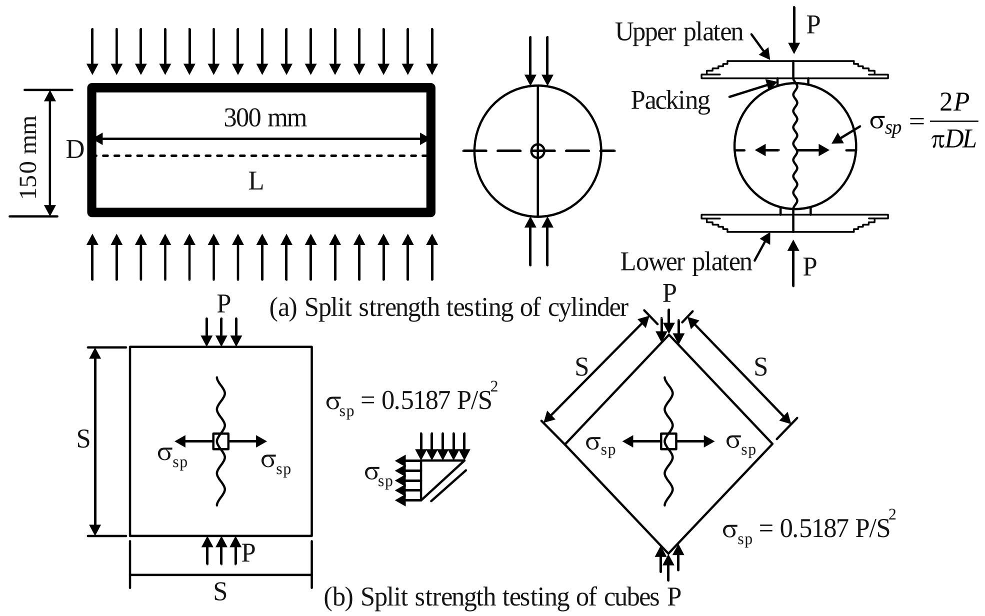
The splitting test are well-known indirect tests used for determining the tensile strength of concrete, sometimes referred as the splitting tensile strength of concrete. The test consists of applying compressive line loads along the opposite generators of a concrete cylinder placed with its axis horizontal between the platens. Due to the applied line loading a fairly uniform tensile stress is induced over nearly two-third of the loaded diameter as obtained from an elastic analysis. The magnitude of this tensile stress (acting in a direction perpendicular to the line of action of applied compression) is given by 2P/πDL = 0.637P/DL, where P is the applied load, and D and L are the diameter and length of the cylinder, respectively. Due to this tensile stress, the specimen fails finally by splitting along the loaded diameter and knowing P at failure, the tensile strength can be determined.

The test can also be performed on cubes by splitting either

(i) along its middle parallel to the edges by applying two opposite compressive forces through 15 mm square bars of sufficient length, or

(ii) along one of the diagonal planes by applying compressive forces along two opposite edges.

In the case of side-splitting of the cubes, the tensile strength is determined from $0.642\ P/S^2$ and in diagonal splitting it is determined from $0.5187\ P/S^2$, where P is the load at failure and S is the side of the cube.

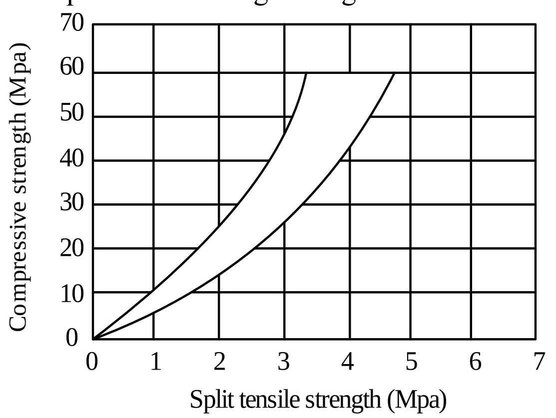


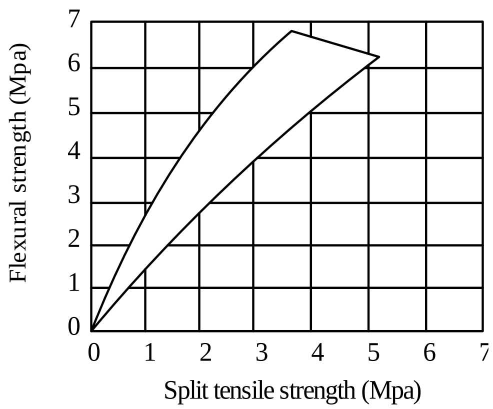
(a) Split strength testing of cylinder

(b) Split strength testing of cubes

Loading arrangement for split strength determination

The relationships between compressive strength and split tensile strength; and flexural strength and split tensile strength are given below:





Advantages of the splitting test for determining the tensile strength are as follows:

1. The test is simple to perform and gives more uniform results than other tension tests.
2. The strength determined is closer to the actual tensile strength of the concrete than that given by the modulus of rupture test.
3. The same moulds can be used for casting specimens for both compression and tension tests.

The splitting tests have also been performed on prisms, i.e., on one-half of the specimen left after performing the modulus of rupture test. Splitting-type tests have also been done on the ring specimens to determine tensile strength. Mortar and concrete rings have been tested by subjecting them to internal pressure. The double punch test is another test performed on concrete cylinders to determine the tensile strength.

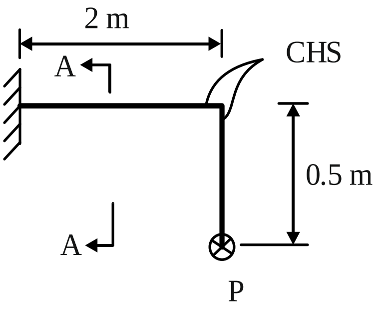
(ii)

Air-Entraining admixture: Air entraining admixture is an admixture used for concrete and mortar which causes air to be incorporated in the form of minute bubbles in the concrete or mortar during mixing. It is usually done to increase workability and resistance to freezing and thawing and disruptive action of de-icing salts.

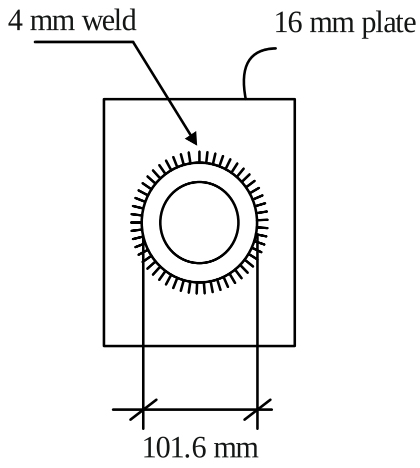
The effect of air entraining admixtures on property of concrete:

(1) Air entraining admixture increase the resistance against freezing and thauring in concrete.
(2) Workability of concrete is improved by using air entraining admixtures.
(3) Air entraining admixtures reduce the unit weight of concrete.
(4) Air entraining admixtures reduce the alkali-aggregate reaction.
(5) Air entraining admixture decrease the permeability of concrete.
(6) They improve placeability and early finish of concrete.
(7) They permits reduction in water content.
(8) They increase resistance to chemical attack.
(9) They reduce the cement content, and cost.
(10) They reduce the heat of hydration.
(11) They permits the reduction in sand content.
(12) Strength of concrete is reduced by using air entraining admixture.
(13) Air entraining admixture reduce the tendency of segregation of concrete ingradients.
(14) They also reduce the bleeding and laitance in concrete.

Q.2. (a) A circular hollow section of outside diameter 101·6 mm and thickness 4·05 mm is connected to a plate of thickness 16 mm as shown in the figure, by welding of weld size 4 mm. Determine the maximum load 'P' that can be applied for the weld. The hollow section is safe. Assume shop weld to be made. Apply limit state method. Assume E 250 grade of steel. Given $f_e = \sqrt{f_a + 3q^2}$. Assume partial load factor as 1·5.



PLAN



View A - A

Sol.

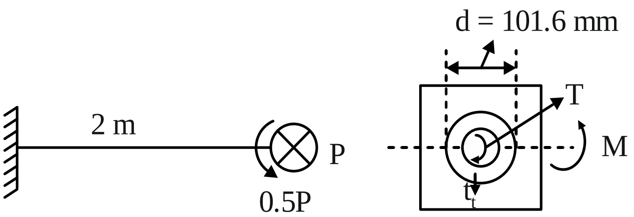
For E250 grade steel: $f_u = 410$ MPa

For shop welding: Partial safety factor for $\gamma_{mw} = 1.25$

$$\text{Permissible shear stress} = \frac{f_u}{\sqrt{3}\gamma_{mw}} = \frac{410}{\sqrt{3} \times 1.25} = 189.37 \text{ MPa}$$

Partial load factor $(\gamma_f) = 1.5$

$\therefore$ Factored direct load = 1.5P N



Factored twisting moment (T) = 1.5 (0.5 × P)
= 0.75 P Nm
Factored Bending Moment (M) = 1.5P × 2
= 3P Nm
Size of weld (S) = 4 mm
⇒ Throat thickness (t_t) = 0.7 × 4 = 2.8 mm
Polar moment of inertia of weld (J) $= 2\pi r^3 \times t_t$

$$= 2\pi \times \left(\frac{101.6}{2}\right)^3 \times 2.8$$

$= 2306370.298\ \text{mm}^4$

$q_1 \longrightarrow$ direct shear stress

$$q_1 = \frac{1.5P}{2\pi r t_t} = \frac{1.5P}{2\pi \times \left(\frac{101.6}{2}\right) \times 2.8} = 1.678P \times 10^{-3} MPa$$

$q_2 \longrightarrow$ Torsional shear stress

$$q_2 = \frac{Tr}{J} = \frac{0.75P \times 10^3 \times \frac{101.6}{2}}{2306370.298} = 16.519P \times 10^{-3} MPa$$

Resultant shear stress $(q)_2 = \sqrt{q_1^2 + q_2^2}$

$$\Rightarrow q = 16.60P \times 10^{-3} MPa$$

$f_a \longrightarrow$ Normal stress due to bending

$$f_a = \frac{My}{I_{zz}} \left(I_{zz} = \frac{J}{2}, y = \frac{d}{2} \right)$$

$$f_a = \frac{3P \times 10^3 \times \frac{101.6}{2}}{\frac{2306370.298}{2}} = 132.156P \times 10^{-3} MPa$$

$f_c \longrightarrow$ equivalent stress

$$f_c = \sqrt{f_a^2 + 3q^2} = 135.248P \times 10^{-3} MPa$$

for safety,

$$f_c \le \frac{f_y}{\sqrt{3}\gamma_{mw}}$$

$$\Rightarrow 135.248P \times 10^{-3} \le 189.37$$

$$P = 1400.168N \simeq \boxed{1.4kN}$$

Q.2. (b) Discuss the mechanisms that are used to enhance the performance characteristics of the concrete.

Sol.

Enhancing the performance characteristics of concrete involves various mechanisms aimed at improving its strength, durability, workability, and overall quality. Here are some key mechanisms and techniques used to enhance concrete performance:

1. Admixtures

a. Chemical Admixtures:

- Water Reducers (Plasticizers): Improve workability without adding extra water, enhancing the strength and durability of concrete.
- Superplasticizers: Highly effective water reducers that can significantly increase the fluidity of concrete, allowing for lower water-cement ratios and higher strengths.
- Accelerators: Speed up the hydration process of cement, which is useful in cold weather concreting and when early strength gain is required.
- Retarders: Slow down the hydration process, helping in hot weather concreting and extending the workability time.
- Air-Entraining Agents: Introduce microscopic air bubbles into the concrete, improving its resistance to freeze-thaw cycles and scaling.

b. Mineral Admixtures:

- Fly Ash: Improves workability, reduces heat of hydration, and enhances the long-term strength and durability of concrete.
- Silica Fume: Increases the compressive strength and durability by filling in the microscopic voids in the cement paste.
- Slag Cement: Enhances the durability, strength, and resistance to chemical attack.

2. Supplementary Cementitious Materials (SCMs)

- Pozzolans: Natural or artificial materials that, when finely divided, react with calcium hydroxide to form compounds possessing cementitious properties. Examples include volcanic ash, calcined clay, and metakaolin.
- Ground Granulated Blast Furnace Slag (GGBFS): A by-product of iron and steel making that, when used in concrete, increases its strength, reduces permeability, and enhances durability.

3. Aggregate Quality and Grading

- High-Quality Aggregates: Use of clean, hard, and well-graded aggregates improves the strength and durability of concrete.
- Optimal Aggregate Grading: Proper gradation of aggregates ensures good packing density, reducing the void content and improving the concrete's workability and strength.

4. Fiber Reinforcement

- Steel Fibers: Enhance tensile strength, impact resistance, and toughness.
- Polypropylene Fibers: Reduce plastic shrinkage cracking and improve durability.
- Glass Fibers: Provide high tensile strength and are resistant to corrosion.
- Carbon Fibers: Increase strength and durability, though they are more expensive.

5. Concrete Mix Design

- Low Water-Cement Ratio: Reducing the water-cement ratio increases the strength and reduces permeability, enhancing durability.
- Proper Proportioning: Ensuring the right balance of cement, aggregates, water, and admixtures for desired properties.

6. Curing Techniques

- Wet Curing: Keeping the concrete moist for an extended period ensures proper hydration and strength development.

- Membrane Curing: Applying curing compounds to prevent moisture loss from the concrete surface.
- Steam Curing: Accelerates the curing process, especially useful for precast concrete elements.

7. Advanced Techniques

- High-Performance Concrete (HPC): Designed to have superior mechanical and durability properties using high-quality materials and optimized mix designs.
- Self-Consolidating Concrete (SCC): Highly flowable concrete that can fill formwork without mechanical vibration, improving surface finish and reducing labor.
- Ultra-High-Performance Concrete (UHPC): Extremely high compressive and tensile strength with enhanced durability, often incorporating fibers and specialized admixtures.

8. Environmental Controls

- Temperature Control: Using chilled water, ice, or heating elements to control the temperature during mixing and curing can prevent thermal cracking and ensure proper hydration.
- Moisture Control: Managing the moisture content in the aggregates and during curing to avoid issues related to drying shrinkage and poor hydration.

9. Nanotechnology

- Nano-materials: Incorporating nano-silica or other nano-materials can significantly enhance the mechanical properties and durability by improving the microstructure of the cement paste.

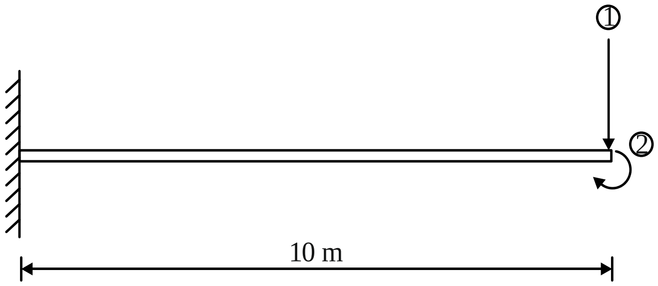
10. Quality Control

- Testing and Monitoring: Regular testing of raw materials, fresh and hardened concrete properties, and on-site monitoring ensures the consistency and quality of the concrete.
- Standards and Specifications: Adhering to industry standards and project specifications to meet the desired performance criteria.

By integrating these mechanisms and techniques, the performance characteristics of concrete can be significantly enhanced, leading to longer-lasting and more resilient structures.

Q.2. (c) (i) Two rectangular plates, one of steel and another of brass, each 50 mm wide and 8 mm deep are placed together to form a beam 50 mm wide and 16 mm deep, on two supports (simply supported) 1 m apart, the brass plate being on the top. Determine the maximum load which can be applied at the centre of the beam, if the plates are separate and can bend independently. Maximum allowable stress in steel = 120 N/mm^2 and in brass = 80 N/mm^2. Take $E_s = 2 \times 10^5$ N/mm^2 and $E_b = 8 \times 10^4$ N/mm^2.

(ii) For the cantilever beam shown in the figure with the co-ordinates, obtain the stiffness matrix. Take EI to be constant. Hence, find the flexibility matrix using their relationship.



Sol.

(i) Two plates one of steel and brass placed together with brass plate on top. Both plates are separated and bend independently. Each have width b = 50 mm and depth d = 8 mm

Maximum moment considering brass plate

$M = \sigma_b Z$

σ_b = Allowable stress in brass = 80 N/mm^2

$$M = 80 \times \frac{bd^2}{6} = 80 \times \frac{50 \times 8^2}{6 \times 10^6} = 0.0426 kNm = 42.67 Nm$$

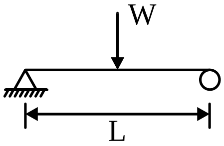
Maximum moment considering steel plate

$M = \sigma_s Z$

σ_s = Allowable stress in steel = 120 N/mm^2

$$M = 120 \times \frac{bd^2}{6} = \frac{120 \times 50 \times 8^2}{6 \times 10^6} = 64 Nm$$

Lower moment is considered.



L = 1 m

$$\text{Maximum Bending moment } = M = \frac{WL}{4} (\text{at mid span})$$

$$\Rightarrow 42.67 Nm = \frac{W \times 1m}{4}$$

$$\Rightarrow W = 170.66 N$$

(ii)

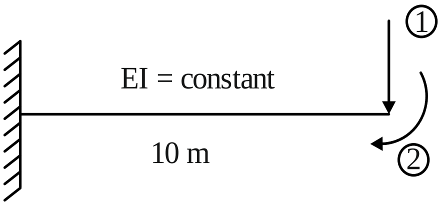
Given data

Cantilever beam with length = 10m

Flexural rigidity EI is constant

Step (1) <u>Formulate the stiffness matrix</u>

Let us consider the beam with co-ordinates shown in figure



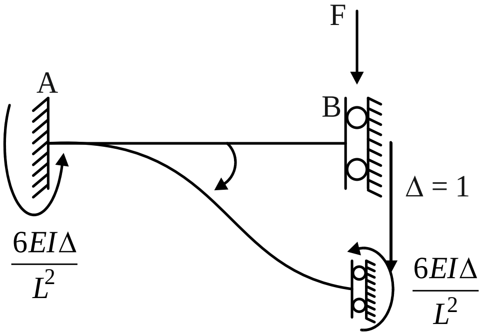
Two co-ordinate directions (1) and (2)

So, order of stiffness matrix is 2 × 2

$$K = \begin{vmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{vmatrix}$$
$$\quad Col(1) \quad Col(2)$$

Col (1) elements:

To find elements of 1st column of stiffness matrix give unit displacement in direction (1) only and restrain/lock other given co-ordinate direction (i.e. (2)) to find the force/reaction develop in direction (1) and direction (2).



$$\sum M_A = 0;\ F \times L - \frac{6EI\Delta}{L^2} - \frac{6EI\Delta}{L^2} = 0$$

$$F = \left(\frac{6EI\Delta}{L^2} + \frac{6EI\Delta}{L^2}\right) / L$$

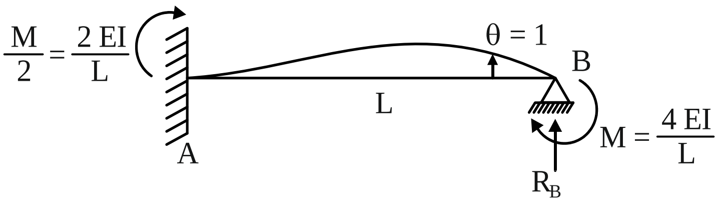
$$F = \frac{12EI\Delta}{L^3}$$

If $\Delta = 1$ $\boxed{F = K_{11} = \frac{+12EI}{L^3}}$

$$\boxed{K_{21} = \frac{-6EI}{L^2}}$$

Col (2) elements:

To find elements of 2nd column of stiffness matrix give unit rotation in direction (2) only and restrain/lock other given co-ordinate direction (i.e. (1)) and find force develop in direction (1) & (2).



$$\sum M_A = 0;\ -R_B \times L + \frac{4EI}{L} + \frac{2EI}{L} = 0$$

$$R_B = \frac{6EI}{L^2}$$

But the co-ordinate direction (1) is opposite to R_B which we have calculated in col (2) elements. So,

$$K_{12} = \frac{-6EI}{L^2}$$

$$\boxed{K_{22} = M = \frac{4EI}{L}}$$

$$K_{2\times 2} = \begin{vmatrix} \frac{12EI}{L^3} & \frac{-6EI}{L^2} \\ \frac{-6EI}{L^2} & \frac{4EI}{L} \end{vmatrix}$$

If L = 10 m (given), the stiffness matrix is K

$$K_{2\times 2} = \begin{vmatrix} \frac{3EI}{250} & \frac{-3EI}{50} \\ \frac{-3EI}{50} & \frac{2EI}{5} \end{vmatrix}$$

Step (2) Find the flexibility matrix

The flexibility matrix [F] is the inverse of the stiffness matrix [K]

To find the inverse $[F] = [K]^{-1}$, we use the formula for the inverse of a 2 × 2 matrix.

$$K^{-1} = \frac{Adj\ K}{|K|},\ Adj\ K = \begin{vmatrix} \frac{2EI}{5} & \frac{+3EI}{50} \\ \frac{3EI}{50} & \frac{3EI}{250} \end{vmatrix}$$

$$|K| = \frac{2EI}{5} \times \frac{3EI}{250} - \left(\frac{-3EI}{50}\right) \times \left(\frac{-3EI}{50}\right)$$

$$= \frac{3(EI)^2}{625} - \frac{9(EI)^2}{2500} = \frac{3(EI)^2}{2500}$$

$$K^{-1} = \frac{\begin{vmatrix} \frac{2EI}{5} & \frac{3EI}{50} \\ \frac{3EI}{50} & \frac{3EI}{250} \end{vmatrix}}{\frac{3(EI)^2}{2500}}$$

$$K^{-1} = \begin{vmatrix} \frac{1000}{3EI} & \frac{50}{EI} \\ \frac{50}{EI} & \frac{10}{EI} \end{vmatrix},\ F = \begin{vmatrix} f_{11} & f_{12} \\ f_{21} & f_{22} \end{vmatrix}$$

Flexibility matrix is- $F = K^{-1} = \begin{vmatrix} \frac{1000}{3EI} & \frac{50}{EI} \\ \frac{50}{EI} & \frac{10}{EI} \end{vmatrix}$

Q.3. (a) (i) What is quality management system in concrete construction ?

(ii) What is cement grout ? How does it differ from cement mortar ?

Sol.

(i) Quality Management System (QMS):

QMS is the management and control system document having three elements:

Quality Assurance (QA) plans, implementation of Quality Control (QC) process and Quality Audit (QA) system of tracking and documentation of quality assurance and quality control programmes. QMS ensures that the intended degree of excellence is attained. The owner or his representative formulates the policy, determines the scope of quality planning and quality management, establishes the relationship between the various participating agencies, and delegates responsibilities and authorities to them so that the quality objectives as set by owner are achieved. It must be understood that QMS cannot be developed in totality at the inception. QMS has to undergo stages of development as various project phases such as design, procurement of materials, construction, inspection, erection and commissioning are entered into with more and more agencies being involved and interfaces take place.

The various stages of development of QMS are given in table below:

Stages	QMS Elements
Planning	Owner formulates QA policy and develops QA plan.
Engineering	The consultant develops his own design QA programme and that of prospective vendors and contractors.
Procurement	Suppliers develop and submit their own QA programmes and QC methods.
Construction	Contractors develop and submit their QA programmes and QC methods.
Inspection	The testing agencies develop their QA programmes.

The introduction and implementation of Quality Management systems can be successful if the concrete industry directs its efforts towards increasing reliability, durability, economy, energy efficiency, versatility, capability, adaptability and aesthetics, as well as towards improvements in materials, material handling, quality control, education of users, construction methods, codes and specifications, disposal and recycling of waste and extension of the environment under which concrete can be used and placed. In addition, efforts should be directed to the development of accurate non-destructive testing procedures, continuous batching and new placing methods, immediate quality control tests, simplified forming methods, simplified reinforcing procedures, simplified methods of joining structural members, new design concepts, performance codes and improved cold and hot weather construction practices. The QMS need be updated to keep pace with advancement in concrete technology.

(ii) **Cement Grout:** Cement mortar of fluid consistency used to fill the voids and joints in masonry and to repair the cracks is known as cement grout.

- They are also used to increase the bearing capacity of soil by injection.
- They are extensively used in dams to fill the crack formed after the concrete sets and hardens.
- They are used to fill the spaces between tunnel walls and the surrounding earth to spread the earth stresses uniformly over the structures.
- They are used in hollow concrete blocks to develop the bond between steel reinforcement and concrete.

Properties of cement grout:

(a) Compressive strength : 7 – 20 N/mm^2
(b) Tensile strength : 1.5 – 3.5 N/mm^2
(c) Modulus of elasticity : 20 – 30 Gpa
(d) Flexural strength : 2 – 5 N/mm^2
(e) Co-efficient of thermal expansion : (7 – 12) × 10^{-6}/°C

Difference between cement grout and cement mortar:

Grout differes from mortar in its fluidity as it is to be poured and not spread into place with trowel.

Some of the major difference between grout and mortar are:

S.No.	Properties	Cement Grout	Cement Mortar
1.	Function	Used to fill gaps and create water tight seals.	Used to bind building blocks.
2.	Consistency	They are thin, fluid like.	They are like thick paste.
3.	Strength	Not as strong as mortar in terms o compressive strength.	Offer high compressive strength compared to grouts.
4.	Composition	Essentially composed of cement, fine or coarse sand, water and a small amount of grouting admisture	Essentially compesed of cement, water and sand in desired proportions.

Q.3. (b) Design a reinforced concrete isolated square footing for a column of size 500 mm × 500 mm subjected to an axial load of 1500 kN under dead and live load condition. The safe bearing capacity of the soil is 120 kN/m^2. Apply limit state method of design and use M25 and Fe500. Assume uniform thickness of footing as 600 mm. Nominal cover = 50 mm. No pedestal is to be provided. Given :

$\frac{100A_{st}}{b_d}$	≤ 0·15	0·25	0·50	0·75	1·00	1·25	1·50	1·75
τ_c N/mm^2	0·29	0·36	0·49	0·57	0·64	0·70	0·74	0·78

Use table 3 of SP – 16.

Show reinforcement details. The column is reinforced with 8 nos. of 20 ϕ bars.

TABLE 3 FLEXURE – REINFORCEMENT PERCENTAGE, p_t For Singly Reinforced Sections for f_{ck} = 25

M_u/bd^2, N/mm^2	f_y, N/mm^2: 240	250	415	480	500
0.30	0.146	0.140	0.084	0.073	0.070
0.35	0.171	0.164	0.099	0.085	0.082
0.40	0.195	0.188	0.113	0.198	0.094
0.45	0.220	0.211	0.127	0.110	0.106
0.50	0.245	0.236	0.142	0.123	0.118
0.55	0.271	0.260	0.156	0.135	0.130
0.60	0.296	0.284	0.171	0.148	0.142
0.65	0.321	0.309	0.186	0.161	0.154
0.70	0.347	0.333	0.201	0.174	0.167
0.75	0.373	0.358	0.216	0.186	0.179
0.80	0.399	0.383	0.231	0.199	0.191
0.85	0.425	0.408	0.246	0.212	0.204
0.90	0.451	0.433	0.261	0.215	0.216
0.95	0.477	0.458	0.276	0.239	0.229
1.00	0.504	0.483	0.291	0.252	0.242
1.05	0.530	0.509	0.307	0.265	0.255
1.10	0.557	0.535	0.322	0.279	0.267
1.15	0.584	0.561	0.338	0.292	0.280
1.20	0.611	0.687	0.353	0.306	0.293
1.25	0.638	0.613	0.369	0.319	0.306
1.30	0.666	0.639	0.385	0.333	0.320
1.35	0.693	0.692	0.401	0.347	0.333
1.40	0.721	0.666	0.417	0.360	0.346
1.45	0.749	0.719	0.433	0.374	0.359
1.50	0.777	0.746	0.449	0.388	0.373
1.33	0.805	0.775	0.466	0.403	0.381
1.60	0.834	0.800	0.482	0.417	0.400
1.65	0.862	0.828	0.499	0.431	0.414
1.70	0.891	0.856	0.515	0.446	0.428
1.75	0.920	0.883	0.532	0.460	0.442
1.80	0.949	0.911	0.549	0.475	0.456
1.83	0.979	0.940	0.566	0.489	0.470
1.90	1.009	0.968	0.583	0.504	0.484
1.95	1.038	0.997	0.601	0.519	0.498
2.00	1.068	1.026	0.618	0.534	0.513
2.05	1.099	1.055	0.635	0.549	0.527
2.10	1.129	1.084	0.653	0.565	0.562
2.15	1.160	1.114	0.671	0.580	0.557
2.20	1.191	1.143	0.689	0.596	0.572
2.25	1.222	1.173	0.707	0.611	0.587
2.30	1.254	1.204	0.725	0.627	0.602
2.35	1.285	1.234	0.743	0.543	0.617
2.40	1.317	1.265	0.762	0.659	0.632
2.45	1.350	1.296	0.781	0.675	0.648
2.50	1.382	1.327	0.799	0.691	0.663

M_u/bd^2, N/mm^2	f_y, N/mm^2: 240	250	415	480	500
2.55	1.415	1.358	0.818	0.708	0.679
2.60	1.448	1.390	0.837	0.724	0.695
2.65	1.482	1.422	0.857	0.741	0.711
2.70	1.515	1.455	0.876	0.758	0.727
2.75	1.549	1.487	0.896	0.775	0.744
2.80	1.584	1.520	0.916	0.792	0.760
2.85	1.618	1.554	0.936	0.809	0.777
2.90	1.653	1.587	0.956	0.827	0.794
2.95	1.689	1.621	0.977	0.844	0.811
3.00	1.724	1.655	0.997	0.862	0.828
3.05	1.760	1.690	1.018	0.880	0.845
3.10	1.797	1.725	1.039	0.898	0.863
3.15	1.834	1.760	1.061	0.917	0.880
3.20	1.871	1.796	1.082	0.936	0.898
3.25	1.909	1.832	1.104	0.954	0.916
3.30	1.947	1.869	1.126	0.973	0.935
3.32	1.962	1.884	1.135	0.981	0.942
3.34	1.978	1.899	1.144	0.989	
3.36	1.993	1.914	1.153		
3.38	2.009	1.929	1.162		
3.40	2.025	1.944	1.171		
3.42	2.040	1.959	1.180		
3.44	2.056	1.974	1.189		
3.46	2.072	1.989			
3.48	2.088	2.005			
3.50	2.104	2.020			
3.52	2.120	2.036			
3.54	2.137	2.051			
3.56	2.153	2.067			
3.58	2.170	2.083			
3.60	2.186	2.099			
3.62	2.203	2.115			
3.64	2.219	2.131			
3.66	2.236	2.147			
3.68	2.253	2.163			
3.70	2.270	2.179			
3.72	2.287	2.196			
3.74	2.304				

NOTE – Blanks Indicate inadmissible reinforcement percentage

Sol.

Given data

Size of column : 500 mm × 500 mm → b × D

Axial load: 1500 kN → P

Safe bearing capacity of soil = SBC = 120 kN/m^2

Grade of concrete: M25 → f_{ck} = 25 N/mm^2

Steel grade – Fe 500 – f_y = 500 N/mm^2

Thickness of footing: 600 mm → D

Nominal Cover : 50 mm

Size of footing

$$A = \frac{P + \frac{10}{100} \times P}{SBC} = \frac{1500 + \frac{10}{100} \times 50}{120} = 13.75\,m^2$$

A = Area of footing required

Self weight of footing $= \frac{10}{100} P$

Side of square footing required $= \sqrt{A} = \sqrt{13.75} = 3.7m$

Provide size of footing (B × B) = 3.8 m × 3.8 m

Net upward soil pressure

$$P_0 = \frac{P}{\text{Area of footing provided}}$$

$$P_0 = \frac{1500}{3.8 \times 3.8} = 10.387\ kN/m^2$$

= 104 kN/m^2 < 120 kN/m^2 ok safe in bearing

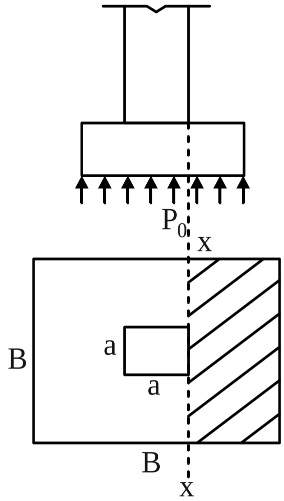
Effective depth of footing (d)

Assume = 20 mm dia bars to be provided in footing

d = D – nominal cover – φ/2 = 600 – 50 – 20/2 = 540 mm

Design B.M

The critical section for BM occurs at the face of column



$$M = \frac{P_0 B}{8}[B - a]^2$$

$$M = \frac{104 \times 3.8[3.8 - 0.5]^2}{8}$$

M = 537.96 kN-m

Factored BM

M_u = 1.5 M

= 1.5 × 537.96 = 807 kN-m

Effective depth of footing required.

$$d = \sqrt{\frac{M_u}{R_u B}} = \sqrt{\frac{807\times10^6}{0.138\times25\times3800}} = 248 \text{ mm} < 540 \text{ mm}$$

Therefore, the section can be treated as under reinforced.

Area of tension steel required in footing

$$\frac{M_u}{Bd^2} = \frac{807\times10^6}{3800\times540^2} = 0.728$$

From table $P = \frac{0.167+0.179}{2} = 0.173$

$$P = \frac{100\ A_{st}}{Bd}$$

$$A_{st} = \frac{0.173\times3800\times540}{100}$$

A_{st} = 3550 mm^2

$$\text{Spacing required } = \frac{B\times\text{Area of c/s of bar}}{A_{st}}$$

$$S = \frac{3800\times\frac{\pi}{4}\times16^2}{3550} = 215 \text{ mm}$$

Provide 16 mm and @ 180 mm c/c

$$\text{Provided steel } = \frac{3800\times\frac{\pi}{4}\times16^2}{180} = 4242.48 \text{ mm}^2$$

$$p_t = 100\times\frac{4242.48}{3800\times540} = 0.20$$

Check for one way shear,

Design S.F at critical section

$$V = P_o B\left[\frac{B}{2}-\frac{a}{2}-d\right]$$ (Critical section lies at d from face of column)

(d = 600 – 50 – 16/2 = 542 mm)

$$V = 104\times3.8\left[\frac{3.8}{2}-\frac{0.5}{2}-0.542\right]$$

= 437.8 kN

V_u = 1.5 V = 1.5 × 437.8 = 656.7 kN

Nominal shear stress

$$\tau_v = \frac{V_u}{bd} = \frac{656.7\times10^3}{3800\times542} = 0.318 \text{ N/mm}^2$$

Design shear strength of concrete from table,

Provide p_t = 0.2

$$\tau_c = 0.29 + \frac{0.36-0.29}{0.25-0.15}(0.20-0.15)$$

$\tau_c = 0.325 \text{ N/mm}^2$

Here $\tau_v < \tau_c$ (Safe in one-way shear)

Two way shear

Critical section for two way shear occurs at d/2 from face of column

$V = P_0 [B^2 - (a + d)^2]$

$= 104 [3.8^2 - (0.5 + 0.542)^2]$

$= 1389$ kN

$V_u = 1.5 V = 1.5 \times 1389 = 2083.5$ kN

Nominal shear

$$\tau_v = \frac{V_u}{bd} = \frac{V_u}{4(a+d)d} = \frac{2083.5\times10^3}{4(500+542)542} = 0.922 \text{ N/mm}^2$$

Design shear strength

$= k_s\tau_c$

$$k_s = 0.5 + \frac{b}{a} \ngtr 1$$

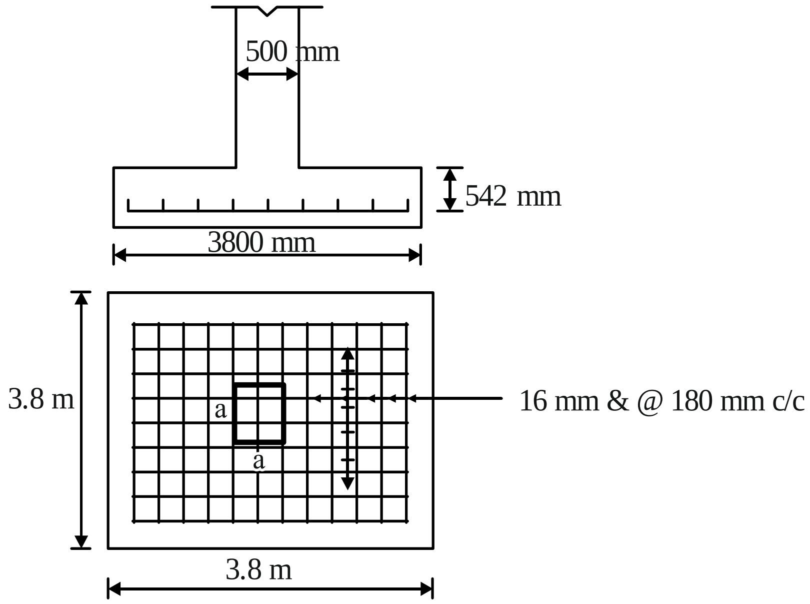
$$k_s = 0.5 + \frac{0.5}{0.5} \ngtr 1$$

$k_s = 1$

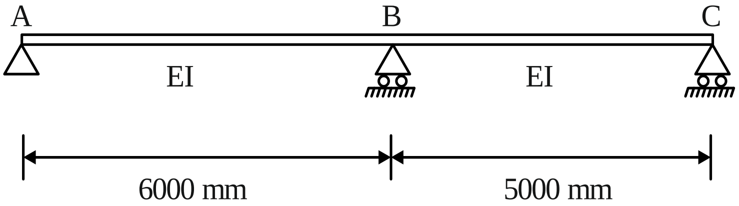
$$\tau_c = 0.25\sqrt{f_{ck}} = 0.5\times5 = 1.25 \text{ N/mm}^2$$

$\tau_v < k_s\,\tau_c$

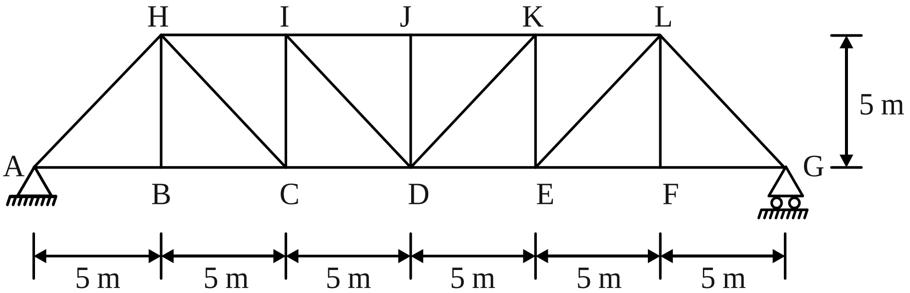
$\therefore$ Safe in two way shear



Q.3. (c) (i) Draw the influence line diagram for reaction R_A for continuous beam shown in the figure at 1 m interval. Assume flexural rigidity is constant throughout. Use Muller-Breslau Principle.



(ii) Draw the influence line diagram for member ID of the truss shown in the figure. Assume that the load moves along the bottom chord.



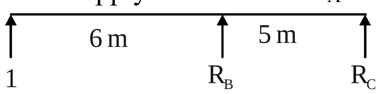
Sol.

(i)

Step 1)

ILD for Reaction at A, R_A

Let us apply unit load for R_A



Taking moment about C

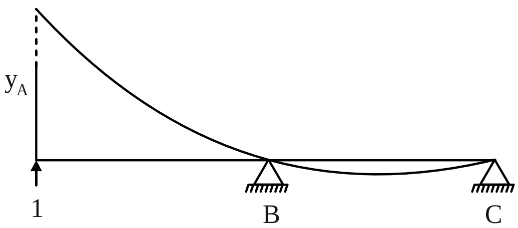
$$\sum M_C = 1\times 11 + 5R_B = 0$$

$$R_B = \frac{-11}{5} = -2.2$$

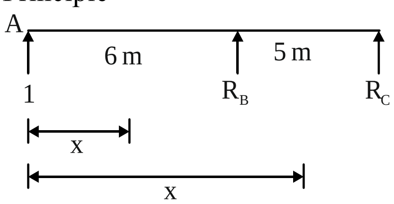
$$\sum F_y = 0\,;\ 1 + R_B + R_C = 0$$

$$1 - 2.2 + R_C = 0$$

$$R_C = 1.2$$



Let us apply unit load in the direction of R_A & draw deflected shape as per Muller Breslau Principle



At any section distance x from A the bending moment is given by

$$M_{xx} = 1x \vdots + R_B(x-6)$$

$$EI\frac{d^2y}{dx^2} = x \vdots - 2.2(x-6)$$

Integrating

$$EI\frac{dy}{dx} = \frac{x^2}{2} + C_1 \vdots - \frac{2.2}{2}(x-6)^2$$

Integrating again

$$EIy = \frac{x^3}{6} + C_1x + C_2 \vdots - \frac{2.2}{6}(x-6)^3 \qquad \ldots(1)$$

At x = 6m y = 0

From equation (1)

$$\frac{6^3}{6} + 6C_1 + C_2 = 0$$

$$6C_1 + C_2 + 36 = 0 \qquad \ldots(2)$$

At x = 11m y = 0

From equation (1)

$$\frac{11^3}{6} + 11C_1 + C_2 - \frac{275}{6} = 0$$

$$\frac{1331}{6} + 11C_1 + C_2 - \frac{275}{6} = 0$$

$1331 + 66\,C_1 + 6C_2 - 275 = 0$

$66\,C_1 + 6\,C_2 + 1056 = 0$

$11\,C_1 + C_2 + 176 = 0 \qquad \ldots(3)$

From equation (2) & (3)

$C_1 = -28$

$C_2 = 132$

New let us take equation (1) again

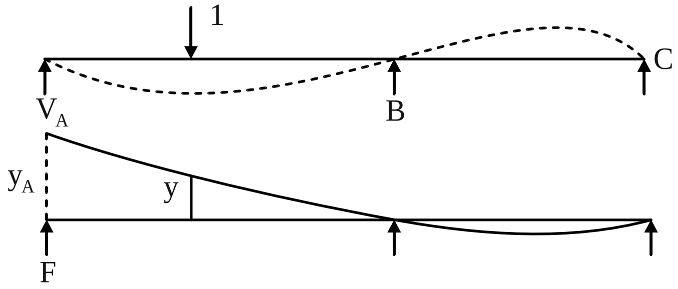
$$EIy = \frac{x^3}{6} + C_1x + C_2 \vdots - \frac{2.2(X-6)^3}{6}$$

$$y = \frac{1}{EI}\left[\frac{x^3}{6} - 28x + 132 \vdots - \frac{2.2(X-6)^3}{6}\right]$$

At x = 0 $y = y_A$

$$y_A = \frac{132}{EI}$$

Now let us use Muller Breslau principle & Bettis law



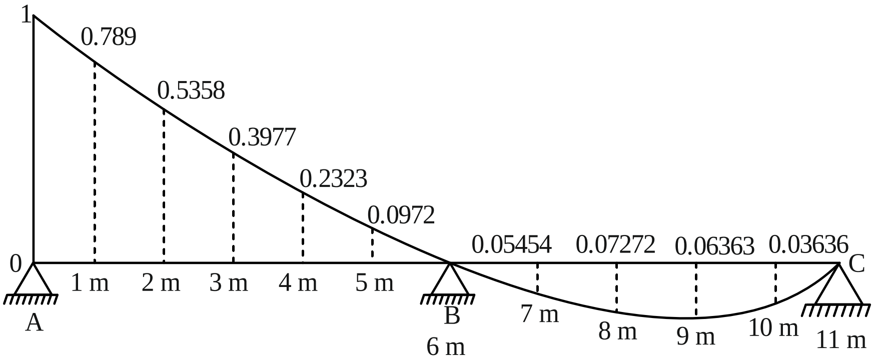
$$V_A y_A - 1y = F \times 0$$

$$V_A = \frac{y}{y_A}$$

$$V_A = \frac{EIy}{EIy_A}$$

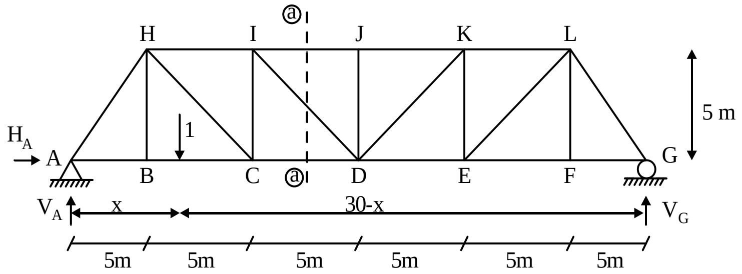
$$V_A = \frac{\left[\frac{x^3}{6} - 28x + 132 \vdots - \frac{2.2(x-6)^3}{6}\right]}{132}$$

x(m)	V_A
0	1
1	+0.789
2	+0.5858
3	+0.3977
4	+0.2323
5	+0.0972
6	0
7	– 0.05454
8	– 0.07272
9	– 0.06363
10	– 0.03636
11	0



ILD for V_A

(ii)



Case i) When unit load is left of ID member

$\Sigma F_x = 0 \Rightarrow H_A = 0$

$\Sigma F_y = 0 \Rightarrow V_A + V_G - 1 = 0$

$V_A + V_G = 1$ … (i)

$\Sigma M_G = V_A \times 30 - 1(30 - x) = 0$

$$V_A = \frac{30-x}{30}$$

From equation (i)

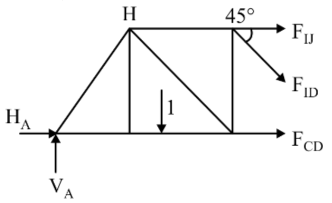
$V_A + V_G = 1$

$$V_G = 1 - V_A = 1 - \left(\frac{30-x}{30}\right)$$

$$V_G = \frac{30-30+x}{30}$$

$$\boxed{V_G = \frac{x}{30}}$$

Now, let's us consider section (a) – (a)



$$\Sigma F_y = 0 \Rightarrow V_A - F_{ID}\sin 45° - 1 = 0$$

$$\Rightarrow \left(\frac{30-x}{30}\right) - F_{ID} \times \frac{1}{\sqrt{2}} - 1 = 0$$

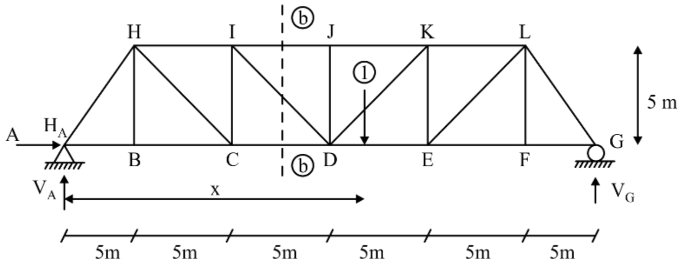
$$\frac{30-x}{30} - 1 = \frac{F_{ID}}{\sqrt{2}}$$

$$\frac{30-x-30}{30} = \frac{F_{ID}}{\sqrt{2}}$$

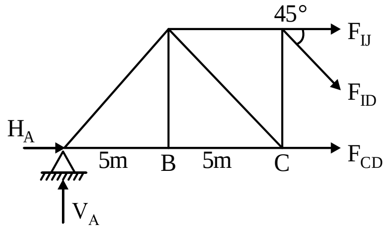
$$\boxed{F_{ID} = \frac{-x\sqrt{2}}{30}}$$

x	F_{ID}
0 m	0
5 m	– 0.2357
10 m	– 0.4714

Case ii) When unit load is right of ID member



Let us take section (b)–(b)



$\Sigma F_y = 0$

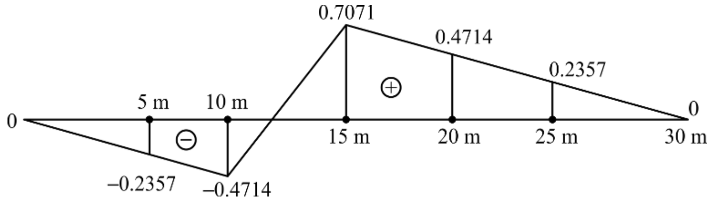
$V_A - F_{ID} \sin 45° = 0$

$$\frac{30-x}{30} = F_{ID} \times \frac{1}{\sqrt{2}}$$

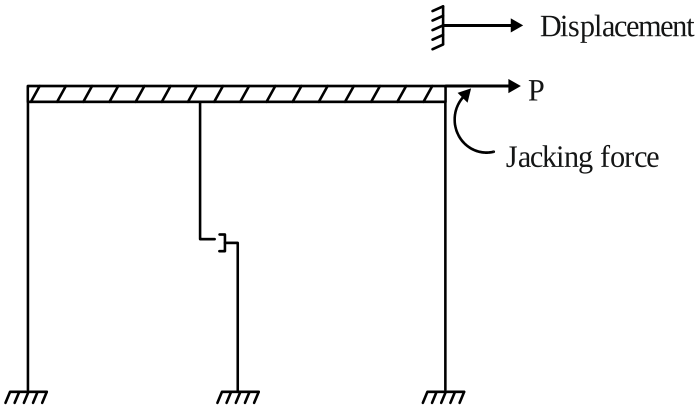
$$\boxed{F_{ID} = \left(\frac{30-x}{30}\right)\sqrt{2}}$$

X (m)	F_{ID}
15 m	+0.7071
20 m	+0.4714
25 m	+0.2354
30 m	0

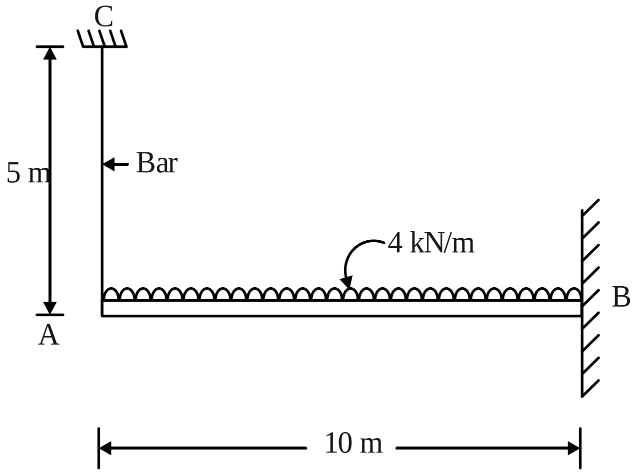
So, ILD for member ID is



Q.4. (a) (i) A single storey building is idealised as a rigid bar supported by weightless columns as shown in the figure. For dynamic response, using jack, a displacement of 5 mm of the girder was observed for a force of 10 kN. After instantaneous release of this initial displacement, the maximum displacement on the return swing was only 4 mm and the period of displacement cycle was 1·5 sec. Find approximately, the effective weight of the girder, damping factor and damping coefficient.



(ii) A cantilever beam AB is fixed at B and is supported at A by the bar AC which serves as a yielding prop. Determine the tensile force in the bar if it is extensible, and if it is inextensible. EI is same for the beam and the bar. For the beam, take moment of inertia I = 0·05 m^4 and for the bar take L = 5 m and area of cross-section A = 1500 mm^2. (Refer the following figure for loading)



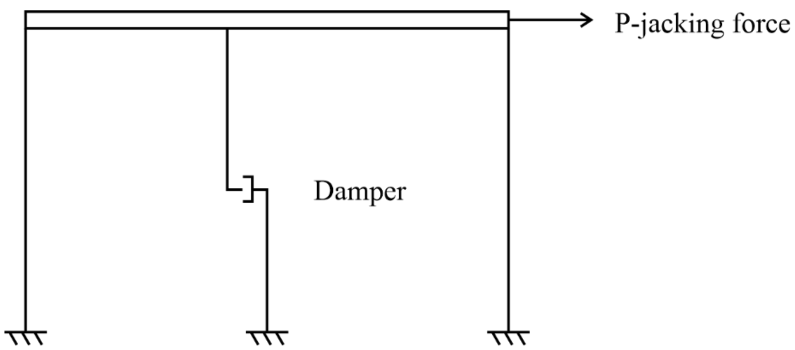
Sol.

(i)

Given data:

Displacement due to force of 10 kN = 5 mm or 0.005 m = δ

Force F = 10 kN



Let us calculate the stiffness (k) of the system

$$k = \frac{F}{\delta} = \frac{10 \times 1000 N}{0.005 m} = 2 \times 10^6 N/m$$

Effective mass (m)/weight (w) of the system:

Time period of the system is related to the stiffness k and the mass m

$$T = 2\pi\sqrt{\frac{m}{k}}$$

$T = 1.5$ sec. (Given)

$$1.5 = 2\pi\sqrt{\frac{m}{2\times10^6}}$$

$$\boxed{m = 114\times10^3 \text{ kg}}$$

$$\boxed{w = 114\times10^3\times9.81 = 1118340\text{N}}$$

Damping factor (ζ)

The logarithmic decrement (δ) is used to find damping factor

$$\delta = \ln\left(\frac{\text{initial displacement}}{\text{displacement after one cycle}}\right)$$

$$\delta = \ln\left(\frac{5}{4}\right) = 0.223$$

The damping factor ε is related to the logarithmic decrement δ

$$\delta = 2\pi\zeta\sqrt{1-\zeta^2}$$

For small damping

$$\delta \approx 2\pi\zeta$$

$$\boxed{\zeta = \frac{\delta}{2\pi} = \frac{0.223}{2\pi} = 0.0354}$$

Damping coefficient (c)

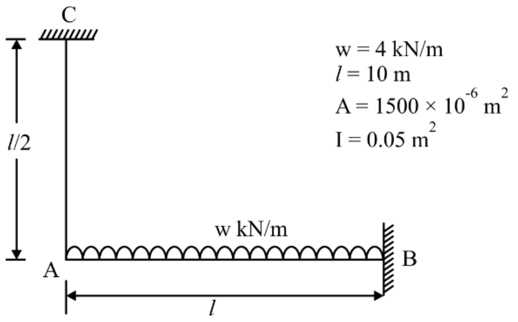
$$c = \zeta C_c = \zeta 2mw_n$$

$$w_n = \frac{2\pi}{T} = \frac{2\pi}{1.5} = 4.19 \text{ rad/sec}$$

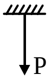
$$c = 0.0354\times2\times114\times10^3\times4.19$$

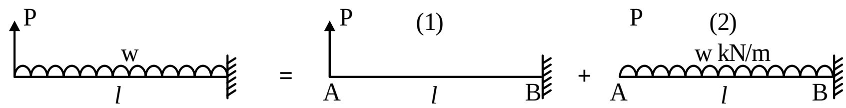
$$\boxed{c = 33808.56 \text{ Ns/m}}$$

(ii)



Let the bar is inextensible ($\Delta_A = 0$; by compatibility)



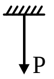


$\Delta_A = \Delta A_1 + \Delta A_2$

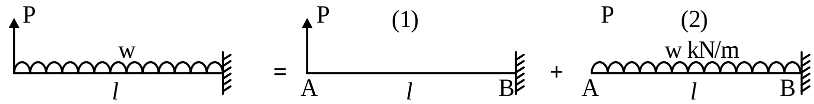
$$0 = \frac{-Pl^3}{3EI} + \frac{wl^4}{8EI}$$

$$P = \frac{3}{8}wl$$

If bar is extensible



$$\Delta_A = \frac{Pl}{2AE}$$



$\Delta_A = \Delta A_1 + \Delta A_2$

$$\frac{Pl}{2AE} = \frac{-Pl^3}{3EI} + \frac{wl^4}{8EI} \Rightarrow P\left(\frac{l}{2AE} + \frac{l^3}{3EI}\right) = \frac{wl^4}{8EI} \Rightarrow P\left(\frac{l}{2A} + \frac{l^3}{3I}\right) = \frac{wl^4}{8I}$$

$$P\left(\frac{10}{2\times1500\times10^{-6}} + \frac{10^3}{3\times0.05}\right) = \frac{4\times10^4}{8\times0.05}$$

P = 10 kN

Q.4. (b) (i) A small maintenance project consisting of jobs given m the following table, with normal time and crash time are given in days.

(I) What is the normal length and its cost ?

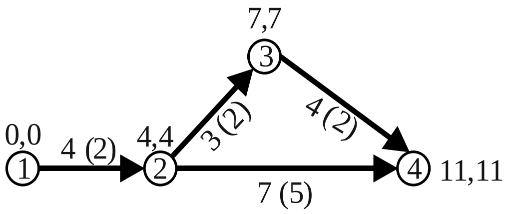
(II) If the project duration is to be crashed by 2 days, what is the total project cost? Overhead costs are ₹ 2,000 per day.

Activity	Duration (days)		Cost (₹)	
	Normal	Crash	Normal	Crash
1 – 2	4	2	4,000	12,000
2 – 3	5	2	3,000	6,000
2 – 4	7	5	4,000	6,000
3 – 4	4	2	8,000	12,000

(ii) Write a note on resources smoothing and resources levelling.

Sol.

(i) The network diagram of above project will be



Critical Path is 1 – 2 – 3 – 4

(I) Normal length of project is 13 days

Total Project Cost = Direct Cost + Indirect Cost

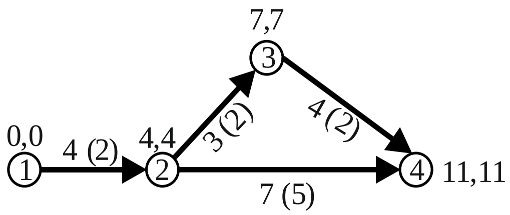
Direct Cost = 4000 + 3000 + 4000 + 8000 = 19000

Indirect cost = 13 × 2000 = 26000

Total Project Cost = 19000 + 26000 = 45000

(II)

Activity	Cost Slope $= \dfrac{C_c - C_n}{t_n - t_c}$
1 – 2	4000
2 – 3	1000
2 – 4	1000
3 – 4	2000



In the above network diagram critical activity (2-3) has least cost slope of 1000 and it has crashing potential of 3 days. But it can be crashed by 2 days only.

New project duration = 11 days

New Total Project Cost = 45000 + 2 × 1000 – 2 × 2000 = 43000

(ii) Resource Smoothing

In resource smoothing activities having floats are rescheduled such that project duration is not changed and uniform demand of resource is created.

Following conditions exist in Resource Smoothing

1. Resources are unlimited.
2. The critical path is not changed.
3. Project duration is not changed.

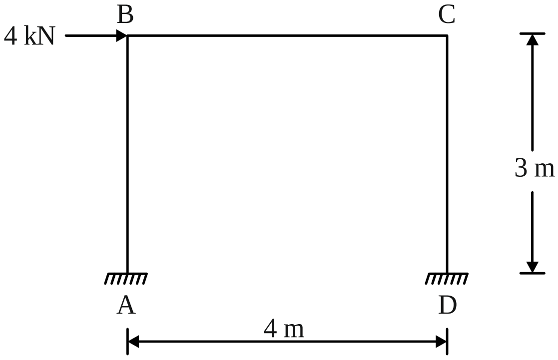
Resource Levelling

In resource levelling activities are rescheduled such that maximum demand of resources does not exceed availability of resources.

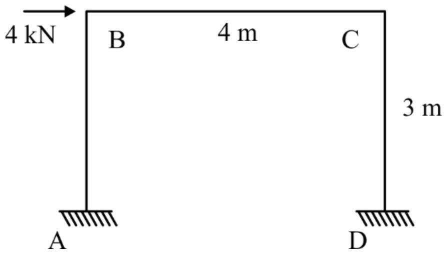
Following conditions exist in Resource Levelling

1. Resources are limited.
2. The Critical path may change.
3. Project duration may change.

Q.4. (c) Analyse the portal frame shown in the figure by using slope-deflection method. Take EI as constant and draw the bending moment diagram. Supports A and D are fixed.



Sol.



Unknown joint displacement

$D_K = 3j - re - n_r + r_r$

$j = 4$ $r_e = 6$ $n_r = 3$ $r_r = 0$

$D_K = 3 \times 4 - 6 - 3 + 0 = 3$

$D_K = 3$

θ_B, θ_C & $\Delta_{BH} = \Delta_{CH} = \Delta\}$ Unknown

$\theta_A = \theta_D = 0\}$ Known values

Fixed end moment

$$\bar{M}_{AB} = \bar{M}_{BA} = \bar{M}_{BC} = \bar{M}_{CB} = \bar{M}_{CD} = 0$$

Slope deflection equation

$$M_{AB} = \bar{M}_{AB} + \frac{2EI}{L}\left(2\theta_A + \theta_B - \frac{3\Delta}{L}\right)$$

$$= 0 + \frac{2EI}{L}\left(0 + \theta_B - \frac{3\Delta}{3}\right)$$

$$M_{AB} = \frac{2EI}{3}\theta_B - \frac{2}{3}EI\Delta$$

$$M_{BA} = \bar{M}_{BA} + \frac{2EI}{3}\left(2\theta_B + \theta_A - \frac{3\Delta}{L}\right)$$

$$M_{BA} = \frac{4}{3}EI\theta_B - \frac{2}{3}EI\Delta$$

$$M_{BC} = \bar{M}_{BC} + \frac{2EI}{4}\left(2\theta_B + \theta_C - \frac{3\Delta}{L}\right)$$

$$M_{BC} = EI\theta_B + \frac{EI\theta_C}{2}$$

$$M_{CB} = \bar{M}_{CB} + \frac{2EI}{4}\left(2\theta_C + \theta_B - \frac{3\Delta}{L}\right)$$

$$M_{CB} = EI\theta_C + \frac{EI\theta_B}{2}$$

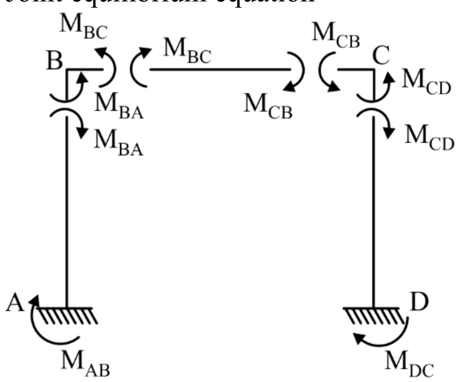
$$M_{CD} = \bar{M}_{CD} + \frac{2EI}{3}\left(2\theta_C + \theta_D - \frac{3\Delta}{L}\right)$$

$$M_{CD} = \frac{4}{3}EI\theta_C - \frac{2}{3}EI\Delta$$

$$M_{DC} = \bar{M}_{DC} + \frac{2EI}{3}\left(2\theta_D + \theta_C - \frac{3\Delta}{L}\right)$$

$$M_{DC} = {}^{2}\!/_{3}\,EI\theta_C - \frac{2}{3}EI\Delta$$

Joint equilibrium equation-



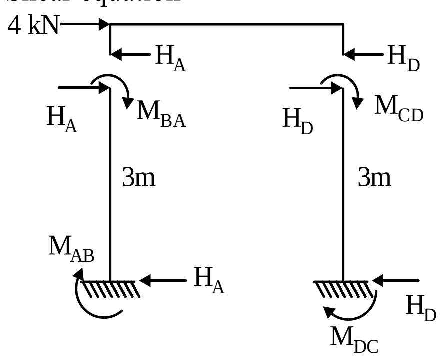
At joint B $\Sigma M_B = 0$ $\quad -M_{BA} - M_{BC} = 0$

$M_{BA} + M_{BC} = 0$ ...(1)

At joint C $\Sigma M_C = 0$ $\quad -M_{CD} - M_{CB} = 0$

$M_{CD} + M_{CD} = 0$ ...(2)

Shear equation



$\Sigma M_A = 0$

$H_A \times 3 + M_{AB} + M_{BA} = 0$

$$H_A = -\left(\frac{M_{AB} + M_{BA}}{3}\right)$$

$\Sigma M_C = 0$

$H_D \times 3 + M_{DC} + M_{CD} = 0$

$$H_D = -\left(\frac{M_{DC} + M_{CD}}{3}\right)$$

$\Sigma F_x = 0$

$4 - H_A - H_D = 0$

$$\frac{M_{AB}+M_{BA}}{3}+\frac{M_{CD}+M_{DC}}{3}+4=0 \qquad \ldots(3)$$

From equation (1)

$M_{BA} + M_{BC} = 0$

$$\frac{4}{3}EI\theta_B-\frac{2}{3}EI\Delta+EI\theta_B+\frac{EI\theta_C}{2}=0$$

$$\frac{7}{3}\theta_B+0.5\,\theta_C-\frac{2}{3}\Delta=0 \qquad \ldots(4)$$

From equation (2)

$M_{CB} + M_{CD} = 0$

$$EI\theta_C+\frac{EI\theta_B}{2}+\frac{4}{3}EI\theta_C-\frac{2}{3}EI\Delta=0$$

$$\frac{7}{3}\theta_C+\frac{\theta_B}{2}-\frac{2}{3}\Delta=0 \qquad \ldots(5)$$

From equation (3)

$M_{BA} + M_{AB} + M_{CD} + M_{DC} = -12$

$$\frac{4}{3}EI\theta_B-\frac{2}{3}EI\Delta+\frac{2EI}{3}\theta_B-\frac{2}{3}EI\Delta+\frac{4}{3}EI\theta_C-\frac{2}{3}EI\Delta+\frac{2}{3}EI\theta_C-\frac{2}{3}EI\Delta=-12$$

$$2\theta_B+2\theta_C-\frac{8\Delta}{3}=\frac{-4}{EI} \qquad \ldots(6)$$

From equation (4), (5), & (6)

$\theta_B=\frac{1.636}{EI}$, $\qquad \theta_C=\frac{1.636}{EI}$,

$\Delta=\frac{6.955}{EI}$

Moments:

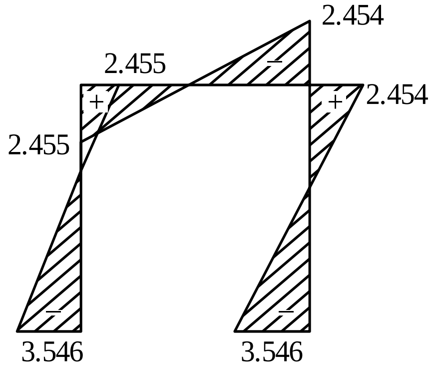
$M_{AB} = -3.546$ kNM

$M_{BA} = -2.455$ kNm

$M_{BC} = +2.455$ kNm

$M_{CD} = -2.454$ kNm

$M_{DC} = -3.546$ kNm



Bending Moment Diagram

SECTION-B

Q.5. (a) (i) What is the difference between soft wood and hard wood ?
(ii) What are the different phases of a project ?

Sol.

(i) Difference between soft wood and Hard wood:

S.No.	Property	Soft wood	Hard wood
1.	Colour	Soft wood are light coloured.	Hard wood are dark in colour.
2.	Growth	Growth of soft wood is faster.	Growth of hard wood is slower.
3.	Weight	Soft wood are light weighted.	Hard wood are heavy weighted.
4.	Density	Density of soft woods are low.	Hard wood have high density.
5.	Strength	Soft woods are strong along the grains.	Hard woods are strong along and across the grains.
6.	Heart wood and sap wood	Heart wood and sap wood cannot be distinguished in soft wood.	Heart wood and sap wood can be distinguished in hard wood.
7.	Resinous materials	Exists in pores.	Does not exist.
8.	Conversion	Conversion is easy.	Conversion is difficult.
9.	Annular rings	Distinct annular ring	Indistinct annular rings.
10.	Example	Chir, fir, deodar, pine and other conifer tress.	Sal, Teak Shisham, Oak and other deciduous trees.

(ii)

Generally a project has four phases,

1. Project initiation: This is the first phase of the project. In this phase, the purpose and scope, justification for initiating it and the solution to be implemented are defined. Also, the recruitment of skilled project team, setting up of a project office and performing an end review of this phase are done in this phase.
2. Project planning: After the project initiation phase, the next phase is project planning. It involves creating the planning documents to guide the team throughout the project delivery. During this phase all the required plans are made in complete detail like resource plan , financial plan, quality plan, risk plan and etc.
3. Project execution: It is third phase of the project in this phase, the physical deliverables are built and handed over to the customer. It is the longest phase in the project, and it consumes lot of energy and resources.
4. Project closure: It is the last phase of the project, which formally closes the project and reports all the achievement of the project the owner of the project. This phase include

handing over the deliverables to the customer, passing the documentation to the business, cancelling supplier contracts, releasing staff and equipment which were used in the project and informing stakeholders of the closure of the project.

Q.5. (b) A suspension bridge of 116 m span has three-hinged stiffening girder, and is subjected to two concentrated loads of 250 kN and 340 kN at a distance of 30 m and 58 m respectively from the left support. Determine the shear force and bending moment for the girder at a distance of 35 m from the left end. The supporting cable has a central dip of 12 m. Also determine maximum tension and its slope in the cable.

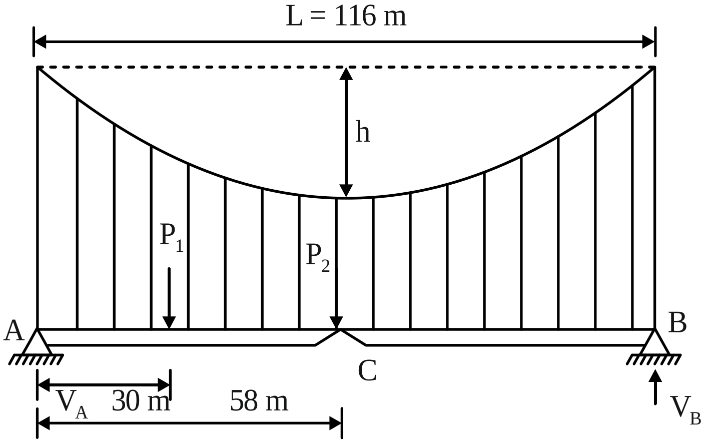
Sol.

Span of the bridge = L = 116 m

Loads P_1 = 250 kN at 30 m from the left support

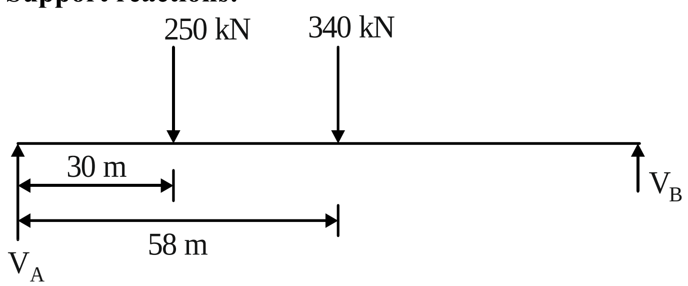
Loads P_2 = 340 kN at 58 m from the left support

Central dip of the cable = h = 12 m



Considering the stiffening grider as simply supported beam supporting the given external load system.

Support reactions:



$\Sigma M_A = 0$; $V_B \times 116 - 340 \times 58 - 250 \times 30 = 0 \Rightarrow V_B = 234.65$ kN

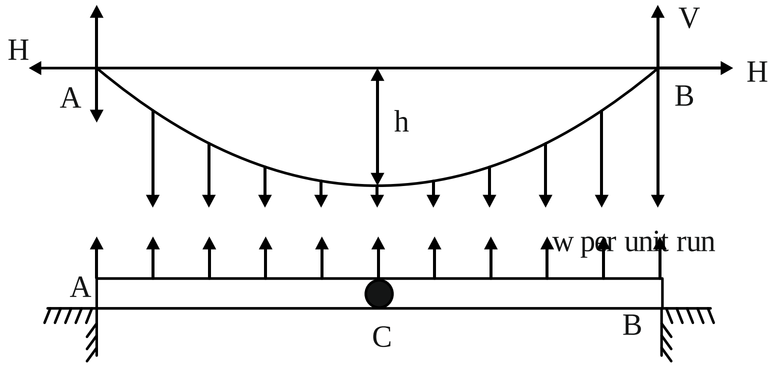
$\sum F_y = 0 \Rightarrow V_A + V_B = 250 + 340 = 590 kN$

So, $V_A = 590 - 234.65 = 355.35$ kN

Max tension & its slope in the table

Now, let us calculate beam moment at C, the middle point of the grider.

$M_C = V_A \times 58 - 250 \times 28 = 355.35 \times 58 - 250 \times 28 = 13610.3$ kNm



Bending moment at any section is given by M = $M_{\text{beam moment}} - H_{\text{moment}}$

Using this condition to the hinge C of the grider, where bending moment is zero.

$M = M_C - Hh$

Where M_C is B_M at C due to the given external load system

$M = 0 \Rightarrow M_C = Hh$

$$\boxed{H = \frac{M_C}{h} = \frac{13610.3}{12} = 1134.19kN}$$

Let the Udl transferred to the cable be w per unit length

$$H = \frac{wl^2}{8h};\ 1134.19 = \frac{w \times 116^2}{8 \times 12}$$

$$\boxed{w = 8.09kN/m}$$

Vertical reaction for the cable $\boxed{V = \frac{wl}{2} = \frac{8.09 \times 116}{2} = 469.22kN}$

Maximum tension in the cable = $T_{max} = \sqrt{V^2 + H^2}$

$$T_{max} = \sqrt{469.22^2 + 1134.19^2}$$

$\boxed{T_{max} = 1227.41kN}$ at A or B in cable

For the cable, at any point

$$\tan\theta = \frac{4h}{L^2}(L - 2x)$$

So, when x = 0 ⇒ tan θ @ A = $\frac{4h}{L^2}(L - 2 \times 0)$

$$= \frac{4h}{L} = \frac{4 \times 12}{116}$$

$$\tan\theta = \frac{12}{29}$$

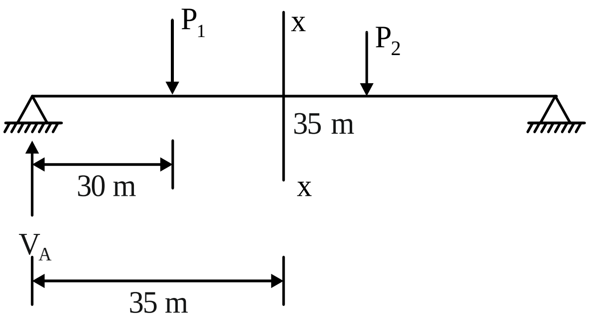
$\boxed{\theta = 22.48}$ slope in the cable @ A and B

Shear force & Bending moment for the grider at a distance of 35 from the left end.

$M_{\text{actual BM}}$ = Actual BM for the grider

$M_{\text{Beam moment}}$ = Beam moment

$M_{\text{actual}} = M_{\text{Beam}} - Hy$



M_{Beam} = Let us calculate beam moment at 35 m from the left end

$= V_A \times 35 - P_1 \times 5 = 355.35 \times 35 - 250 \times 5 = 11187.25$ kNm

At 35 m from the left end, for the cable

$$y = \frac{4hx}{L^2}(L-x) = \frac{4\times12\times35}{116^2}(116-35) = 10.1129\text{m}$$

Actual BM at 35 m from left end

$= M_{Beam} - H_y$

$= 11187.25 - 1134.19 \times 10.1129$

$= -282.70$ kNm

For the girder, the SF at any section is given by S = Beam shear – H tan θ

For the cable, at any point, $= \frac{4h}{L^2}(L-2x) = \tan\theta = \frac{4\times12}{116^2}(116 - 2\times35) = 0.16409$ m

Beam shear at 35 m from left end $= V_A - 250$

$= 355.35 - 250$

$= 105.35$ kN

Actual SF at 35 m from left end $= 105.35 - 1134.19 \times 0.16409 = 86.73$ kN

Q.5. (c) What are the four classes of steel sections as per IS : 800 - 2007 ? Explain with a moment-rotation diagram and a stress diagram.

Sol.

In the code (IS 800), cross sections are placed into four behavioural classes depending upon the material yield strength, the uidth-to-thickness ratios of the individual components (e.g., webs and flanges) within die cross section, and tire loading arrangement, rhe four classes of sections are defined as follows.

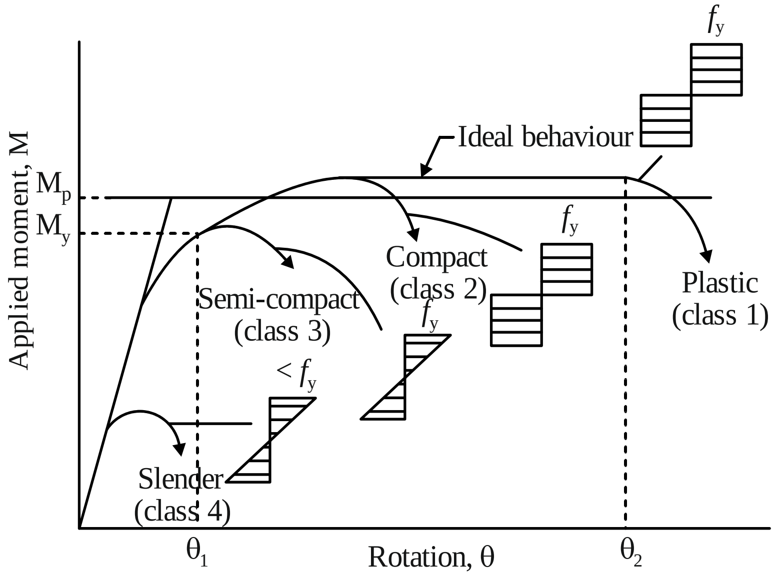
(a) Plastic or class 1 Cross sections which can develop plastic hinges and have the rotation capacity required for the failure of the structure by the formation of a plastic mechanism (only these sections are used in plastic analysis and design).

(b) Compact or class 2 Cross sections which can develop their plastic moment resistance, but have inadequate plastic hinge rotation capacity because of local buckling.

(c) Semi-compact or class 3 Cross sections in which the elastically calculated stress in the extreme compression fibre of the steel member, assuming an elastic distribution of stresses, can reach the yield strength, but local buckling is liable to prevent the development of the plastic moment resistance.

(d) Slender or class 4 Cross sections in which local buckling will occur even before the attainment of yield stress in one or more parts of the cross section. In such cases, the effective sections for design are calculated by deducting the width of the compression plate element in excess of the semi-compact section limit.

The moment-rotation characteristics of these four classes of cross sections are shown in figure below.. As seen from this figure, class 1 (plastic) cross sections are fully effective under pure compression, and capable of reaching and maintaining their full plastic moment in bending and hence used in plastic design. These sections will exhibit sufficient ductility ($\theta_2 > 6\ \theta_1$), where θ_1 is the rotation at the onset of plasticity and θ_2 is t,he lower limit of rotation for treatment as a plastic section).



Q.5. (d) Find the development length for a reinforcement bar of diameter 20 mm under tension and compression embedded in concrete of M30 grade. The design bond stress τ_{bd} in limit state method for plain bar in tension is 1·5 MPa for M30. The grade of reinforcement bars is Fe500. Also find the corresponding anchorage lengths of the bar for 90° bend.

Sol.

Given,
Diameter of renforcing bars = 20 mm
Grade of concrete = M30
τ_{bd} (Plain bars) = 1.5 Mpa
Grade of steel = Fe 500

(i) Development length in tension,

The value of design bond stress increased by 60% for deformed bars

$$(L_d)_{Tension} = \frac{0.87 f_y \phi}{4\tau_{bd} \times 1.6}$$

$$= \frac{0.87 \times 500 \times 20}{4 \times 1.5 \times 1.6} = 906.25 \text{ mm}$$

(ii) **Development length in compression**,

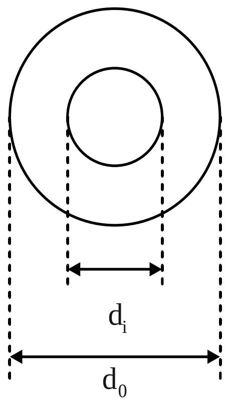
The value of design bond stress increased by 60% for deformed bars and 25% increased for bar in compression

$$(L_d)_{Compression} = \frac{0.87 f_y \phi}{4\tau_{bd} \times 1.6 \times 1.25} = 725 \text{ mm}$$

(iii) The anchorage value of 90° bend
l_o= 8 times the diameter of the bar.
= 8 × 20 = 160 mm

Q.5. (e) The maximum allowable shear stress in a hollow shaft of external diameter equal to twice the internal diameter is 120 N/mm^2. Determine the diameter of the shaft if it is subjected to a torque of 12 kNm and a bending moment of 5 kNm.

Sol.



Inner diameter = d_i
Outer diameter = d_o
Given do = $2d_i$
Assume di = d
$d_o = 2d$
τ_{max} = maximum allowable shear stress = 120 N/mm^2
Tarque = 12 kNm =T
Bending moment = M = 5 kNm
Maximum shear stress due to combined bending and twisting should be less than equal to Maximum allowable shear stress.

$$\text{Normal stress } = \frac{M}{Z} = \frac{M\, d_0/2}{\frac{\pi}{64}\left(d_0^4 - d_i^4\right)}$$

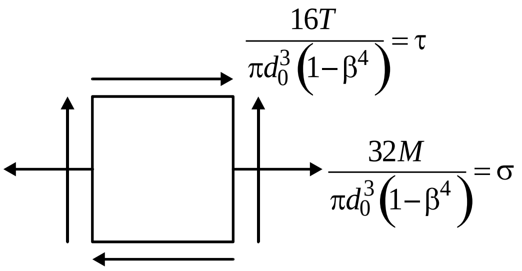
$$= \frac{32M}{\pi d_0^3\left(1-\beta^4\right)}$$

$$\text{Where } \beta = \left(\frac{d_i}{d_o}\right) = \frac{1}{2} = 0.5$$

$$\text{Shear stress } = \frac{T}{Z_P} = \frac{T\, d_0/2}{\frac{\pi}{32}\left(d_0^4 - d_i^4\right)}$$

$$= \frac{16T}{\pi d_0^3\left(1-\beta^4\right)}$$

$$\text{Where } \beta = \left(\frac{d_i}{d_o}\right) = \frac{1}{2} = 0.5$$



$$\frac{16T}{\pi d_0^3\left(1-\beta^4\right)} = \sigma$$

$$\frac{32M}{\pi d_0^3\left(1-\beta^4\right)} = \tau$$

Maximum shear stress

$$\tau_{max} = \sqrt{\left(\frac{\sigma}{2}\right)^2 + \tau^2}$$

$$= \frac{16}{\pi d_0^3\left(1-\beta^4\right)}\sqrt{M^2+T^2}$$

$$= \frac{16}{\pi d_0^3\left(1-\beta^4\right)}\sqrt{M^2+T^2} = \tau_{max} = 120\ \text{N/mm}^2$$

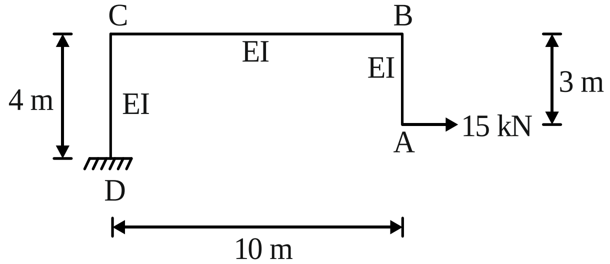
$$\Rightarrow d_0 = \left(\frac{16\sqrt{M^2+T^2}}{\pi\left(1-\beta^4\right)\times 120}\right)^{1/3}$$

$$= \left(\frac{16\sqrt{5^2+12^2}\times 10^6}{\pi\left(1-0.5^4\right)\times 120}\right)^{1/3}$$

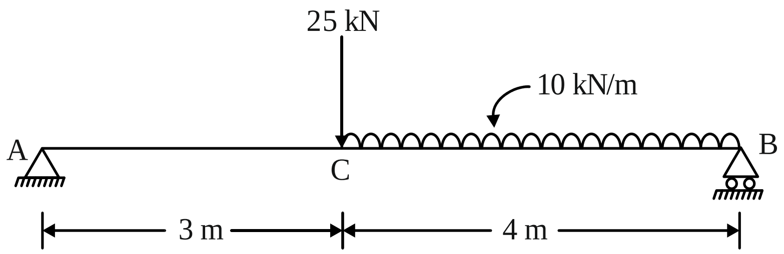
$d_0 = 83.80$ mm

$d_i = 41.90$ mm

Q.6. (a) (i) Using the unit load method, determine the horizontal deflection of the free end (Point A) of the frame shown in the figure below. Support D is fixed, C and B are rigid joints. Assume flexural rigidity EI as constant.



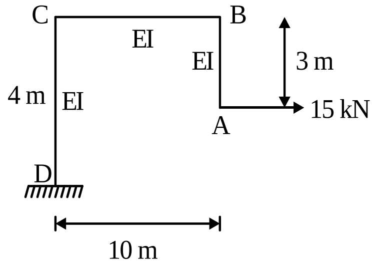
(ii) Use Castigliano's theorem and determine vertical displacement of point C of the beam shown in the figure. Assume E = 210 GPa and I = 150 × 10^6 mm^4.



EI same for AC and CB

Sol.

(i)



As per virtual work method:

External work done = Internal work done

$$y = \int_0^l \frac{Mm_1}{EI} dl \rightarrow \text{deflection}$$

M= B.M at any section

m_1 = BM at any section, where all the loads are removed and placing a unit load

Support reaction:

$\Sigma Fy = 0; \quad V_D = 0$

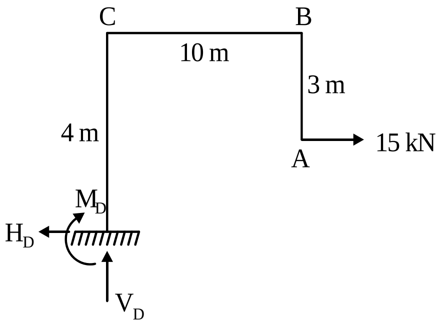
$\Sigma Fx = 0; \quad -H_D + 15 = 0$

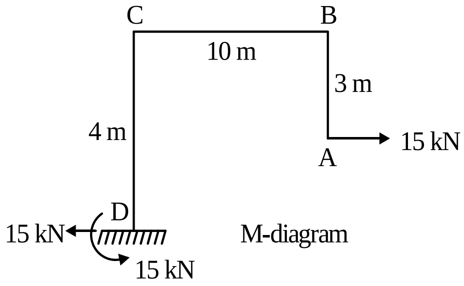
$\boxed{H_D = 15kN}$

$\Sigma M_D = 0$

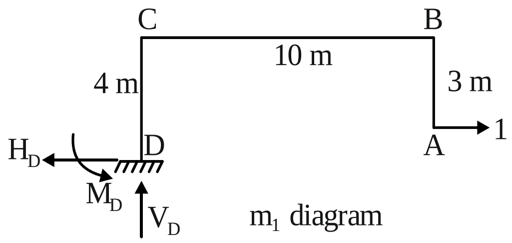
$M_D + 15 \times 1 = 0$

$\boxed{M_D = -15kNm}$





Remove all loads and apply unit load at A in horizontal direction:



m_1 diagram

$\Sigma Fy = 0$

$\boxed{V_D = 0}$

$\Sigma Fx = 0$

$\boxed{H_D = 1}$

$\Sigma M_D = 0$

$M_D - 1 = 0$

$\boxed{M_D = 1}$

Unit load table

Member	Origin	Limits	M	m_1
AB	A	0 – 3 m	+ 15 x	X
BC	B	0 – 10 m	+ 45 kNm	3
CD	D	0 – 4 m	15 x – 15	x – 1

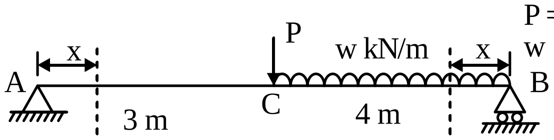
$$\Delta_{AH} = \int_0^3 \frac{15x.x}{EI}dx + \int_0^{10} \frac{(45)\times 3}{EI}dx + \int_0^4 \frac{(15x-15)(x-1)}{EI}dx$$

$$\Delta_{AH} = \left[\frac{15\times 3}{3EI}\right]_0^3 + \left[\frac{135x}{EI}\right]_0^{10} + \frac{15}{EI}\left[\frac{x3}{3} + x - \frac{2x^2}{2}\right]_0^4$$

$$= \frac{15\times 3^3}{3EI} + \frac{135\times 10}{EI} + \frac{15}{EI}\left[\frac{4^3}{3} + 4 - 4^2\right]$$

$$\Delta_{AH} = \frac{1625}{EI}$$

(ii)



P = 25 kN
w = 10 kN/m

Reactions

$\Sigma M_A = 0$; $R_B \times 7 - w \times 4 \times 5 - P \times 3 = 0$

$$R_B = \left(\frac{200 + 3P}{7}\right)$$

$\Sigma F_y = 0$; $R_A + R_B - 4w - P = 0$

$$R_A + \left(\frac{200 + 3P}{7}\right) - 4\times 10 - P = 0$$

$$R_A = \left(\frac{80 + 4P}{7}\right)$$

As per Castigliano's theorem $\frac{\partial U}{\partial P} = \Delta_C$

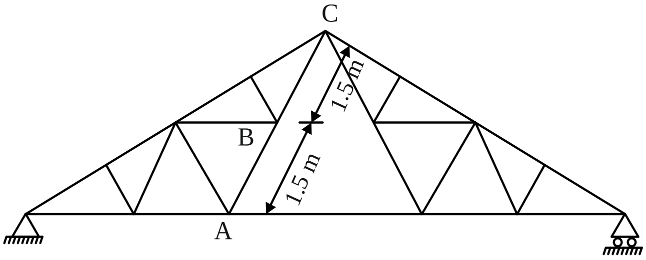
$$\Delta = \frac{1}{EI}\int M_x \frac{\partial M_x}{\partial P} dx$$

$$EI\Delta_C = \int_{AC,\,0}^{3} \left(\frac{80+4P}{7}\right)x \cdot \frac{4}{7}xdx + \int_{BC,\,0}^{4} \left\{\left(\frac{200+3P}{7}\right)x - \frac{wx^2}{2}\right\}\frac{3}{7}xdx$$

$$= \frac{1}{EI}\left[14.69\left[\frac{x^3}{3}\right]_0^3 + \frac{3}{7}\left[39.28\frac{x^3}{3} - \frac{5x^4}{4}\right]_0^4\right] = \frac{1}{EI}[132.21 + 221.98] = \frac{354.20}{EI}$$

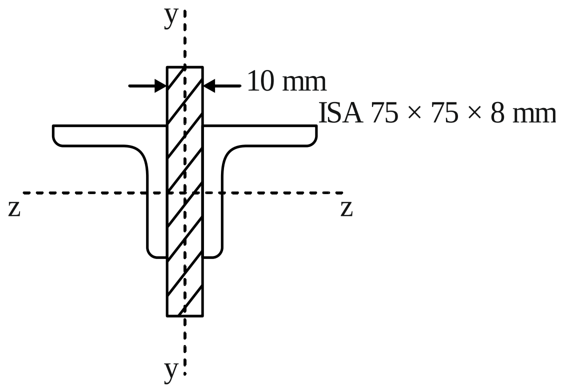
$$\Delta_C = \frac{354.20 \times 10^3}{210 \times 10^9 \times 150 \times 10^6 \times 10^{-12}} = 0.011\text{mm}$$

Q.6. (b) The member A-B-C in a truss consists of two angles $\angle 75 \times 75 \times 8$ back to back on both sides of the gusset of thickness 10 mm. The length of members AB and BC is 1·5 m each. Find the maximum (factored) compressive load carrying capacity of the member A-B-C. Assume E250 grade of steel and all joints are welded. Properties of $\angle 75 \times 75 \times 8$ are : $A = 1140$ mm^2, $C_y = C_z = 21{\cdot}4$ mm, $I_{yy} = I_{zz} = 59 \times 10^4$ mm^4.



$\frac{KL}{r}$	f_{cd} (MPa)
20	224
30	211
40	198
50	183
60	168
70	152
80	136
90	121
100	107
110	94·6
120	83·7
130	74·3
140	66·2
150	59·2
160	53·3
170	48·1
180	43·6

Sol.



For steel of grade E250

$f_y = 250$ Mpa

$f_u = 410$ Mpa

Angles placed on opposite side of gusset plate.

r ⟶ radius of gyration

$$r_{zz}^{*} = \sqrt{\frac{I_{zz}}{A}} = \sqrt{\frac{59\times10^4}{1140}} = 22.75mm$$

$$I_{yy}^{*} = 2\left[I_{yy} + A\left(C_y + t_y/2\right)^2\right]$$

$$= 2\left[59\times10^4 + 1140\left(21.4 + \frac{10}{2}\right)^2\right]$$

= 2769068.8

$$\therefore r_{yy}^{*} = \sqrt{\frac{I_{yy}^{*}}{2A}} = \sqrt{\frac{2769068.8}{2\times1140}} = 34.85\text{mm}$$

The asterik sign * is used for the properties of double angle strut

$$r_{\min} = r_{zz}^{*} = 22.75\text{mm}$$

For continuous double angle strut.

$L_{eff} = kl = l = 1.5 + 1.5 = 3$m

$$\therefore \frac{kl}{r_{\min}} = \frac{3\times10^3}{22.75} = 131.868$$

From the table,

$$f_{cd} = 90 - \left(\frac{90-80}{136-121}\right)\times(131.868-121)$$

= 82.75 MPa

∴ The design compressive strength of the member A–B–C is,

$P_d = A_e \times f_{cd} = 2 \times 1140 \times 82.75 \times 10^{-3}$ kN

$= \boxed{188.67\text{kN}}$

Q.6. (c) The cost of a machine required at the construction site is ₹ 1,20,000 and its salvage value is ₹ 20,000. The expected life of the machine is 5 years only It is also expected to work 2000 hours in a year. Compute the yearly depreciation for the machine by using the following methods :

- Straight line method
- Sinking fund method

Sol.

Given data

C_i = 1,20,000, C_s = 20,000, n = 5 years

Straight line depreciation method

Depreciation per year, $D = \frac{C_i - C_s}{n}$

$$D = \frac{120000 - 20000}{5} = 20,000$$

Depreciation in 1^{st}, 2^{nd}, 3^{rd}, 4^{th} & 5^{th} year will be

$D_1 = D_2 = D_3 = D_4 = D_5 = 20,000$

Sinking Fund Method

Depreciation in m^{th} year = $D(1 + i)^{m-1}$

Where $D = (C_i - C_s) \times \frac{i}{(1+i)^n - 1}$

Since in the question rate of interest for sinking fund is not given, hence let us assume i = 10% per year.

$$D = (C_i - C_s) \times \frac{i}{(1+i)^n - 1} = 100000 \times \frac{0.1}{1.1^5 - 1}$$

D = 16,379·75

$\because D_m = D(1 + i)^{m-1}$

Depreciation in 1^{st}, 2^{nd}, 3^{rd}, 4^{th} & 5^{th} year will be

$D_1 = 16,379\cdot75 \times 1\cdot1^0 = 16,379\cdot75$

$D_2 = 16,379\cdot75 \times 1\cdot1^1 = 18,016\cdot90$

$D_3 = 16,379\cdot75 \times 1\cdot1^2 = 19,818\cdot59$

$D_4 = 16,379\cdot75 \times 1\cdot1^3 = 21,800\cdot45$

$D_5 = 16,379\cdot75 \times 1\cdot1^4 = 23,980\cdot49$

Q.7. (a) A cylindrical shell of mild steel sheet and 1250 mm diameter is to be subjected to an internal pressure of 1·55 MN/mm^2. If the mild steel yields at 225 MN/m^2, determine the thickness of the mild steel sheet on the basis of the following theories of failure. Assume a F.O.S. = 3.

(i) Maximum principal stress theory
(ii) Maximum shear stress theory
(iii) Maximum shear strain energy theory

Sol.

For cylindrical shell

Thickness = t

Diameter = d = 1250 mm

Internal pressure = P = 1.55 MN/m^2

(in question it is given 1.55 MN/mm^2)

Yield strength = f_y = 225 MN/m^2

Factor of safety = F = 3.

(i) **Thickness as per maximum principal stress theory**

$$\text{Maximum principal stress} = \text{Hoop stress } = \sigma_1 = \frac{pd}{2t}$$

$$\text{Factor of safety} = \frac{f_y}{\sigma_1} = 3$$

$$\Rightarrow \sigma_1 = \frac{f_y}{3}$$

$$\Rightarrow \frac{pd}{2t} = \frac{225}{3}$$

$$\Rightarrow t = \frac{3 \times pd}{225 \times 2} = \frac{3 \times 1.55 \times 1250}{225 \times 2} = 12.92 mm$$

(ii) **Thickness as per maximum shear stress theory**

Maximum shear stress

$$\tau_{max} = \frac{\sigma_1 - \sigma_2}{2} = \frac{\frac{pd}{2t} - \frac{pd}{4t}}{2} = \frac{pd}{8t}$$

$\sigma_1 =$ Hoop stress

$\sigma_2 =$ Longitudinal stress

$$\text{Factor of safety } = \frac{\frac{f_y}{2}}{\tau_{max}} = 3$$

$$\Rightarrow \tau_{max} = \frac{f_y}{6}$$

$$\Rightarrow \frac{pd}{8t} = \frac{f_y}{6}$$

$$\Rightarrow t = \frac{3pd}{4f_y} = \frac{3 \times 1.55 \times 1250}{4 \times 225} = 6.46 \text{ mm}$$

(iii) **Thickness as per maximum shear strain energy theory**

$$FOS = \sqrt{\frac{2f_y^2}{\sigma_1^2 + \sigma_2^2 + (\sigma_1 - \sigma_2)^2}}$$

$$= \sqrt{\frac{2 \times 225^2}{\left(\frac{pd}{2t}\right)^2 + \left(\frac{pd}{4t}\right)^2 + \left(\frac{pd}{4t}\right)^2}}$$

$$\Rightarrow F = \frac{1}{\frac{pd}{4t}} \sqrt{\frac{2 \times 225^2}{(4+1+1)}}$$

$$\Rightarrow 3 = \frac{129.9 \times 4t}{pd}$$

$$\Rightarrow t = \frac{3pd}{129.9 \times 4} = \frac{3 \times 1.55 \times 1250}{129.9 \times 4}$$

= 11.19 mm.

Alternatively:

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

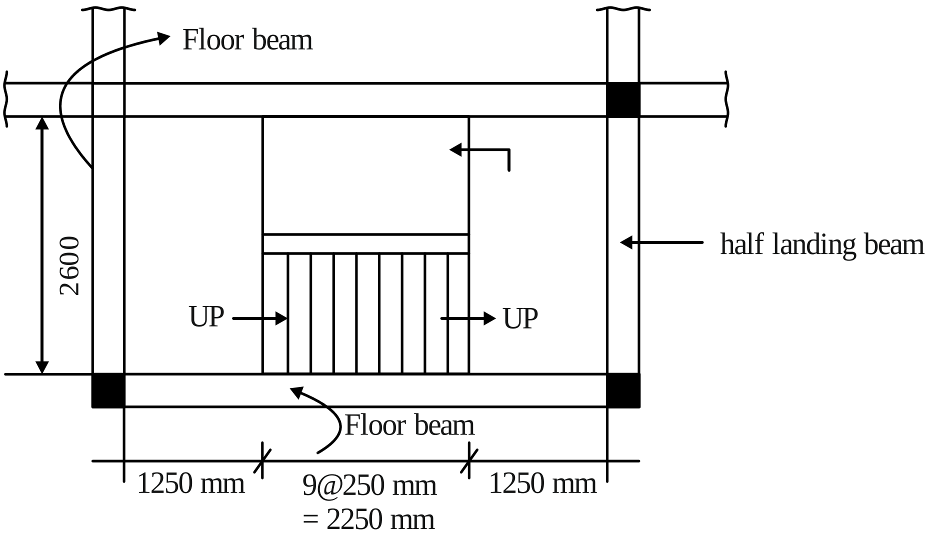
$$\Rightarrow \left(\frac{pd}{2t}\right)^2 + \left(\frac{pd}{4t}\right)^2 - \frac{p^2d^2}{8t^2} = \frac{225^2}{3^2}$$

$$\Rightarrow \frac{p^2d^2}{t^2}\left[\frac{1}{4} + \frac{1}{16} - \frac{1}{8}\right] = 5625$$

$$\Rightarrow t^2 = \frac{1.55^2 \times 1250^2}{5625} \times \frac{3}{16}$$

$$\Rightarrow t = 11.19\text{mm.}$$

Q.7. (b) A dog-legged stair is to be constructed in a building resting on beams and columns as shown in the figure. The floor to floor height of the building is 3 m. The cross-sectional dimensions of beams including half landing beams are 250 mm × 400 mm. Intensity of live load is 3 kN/m^2. Design and detail the reinforcement (with sketches) of a typical flight of the stair, applying 'limit state method' of design. Use M25 and Fe500. Deflection check is not needed. Assume the depth of waist slab as 150 mm.



PLAN

TABEL 3 FLEXURE – REINFORCEMENT PERCENTAGE, p_t FOR SINGLY REINFORCED SECTIONS

M_u/bd^2, N/mm²	f_y, N/mm²				
	240	250	415	480	500
0.30	0.146	0.140	0.084	0.073	0.070
0.35	0.171	0.164	0.099	0.085	0.082
0.40	0.195	0.188	0.113	0.198	0.094
0.45	0.220	0.211	0.127	0.110	0.106
0.50	0.245	0.236	0.142	0.123	0.118
0.55	0.271	0.260	0.156	0.135	0.130
0.60	0.296	0.284	0.171	0.148	0.142
0.65	0.321	0.309	0.186	0.161	0.154
0.70	0.347	0.333	0.201	0.174	0.167
0.75	0.373	0.358	0.216	0.186	0.179
0.80	0.399	0.383	0.231	0.199	0.191
0.85	0.425	0.408	0.246	0.212	0.204
0.90	0.451	0.433	0.261	0.215	0.216
0.95	0.477	0.458	0.276	0.239	0.229
1.00	0.504	0.483	0.291	0.252	0.242
1.05	0.530	0.509	0.307	0.265	0.255
1.10	0.557	0.535	0.322	0.279	0.267
1.15	0.584	0.561	0.338	0.292	0.280
1.20	0.611	0.687	0.353	0.306	0.293
1.25	0.638	0.613	0.369	0.319	0.306
1.30	0.666	0.639	0.385	0.333	0.320
1.35	0.693	0.692	0.401	0.347	0.333
1.40	0.721	0.666	0.417	0.360	0.346
1.45	0.749	0.719	0.433	0.374	0.359
1.50	0.777	0.746	0.449	0.388	0.373
1.33	0.805	0.775	0.466	0.403	0.381
1.60	0.834	0.800	0.482	0.417	0.400
1.65	0.862	0.828	0.499	0.431	0.414
1.70	0.891	0.856	0.515	0.446	0.428
1.75	0.920	0.883	0.532	0.460	0.442
1.80	0.949	0.911	0.549	0.475	0.456
1.83	0.979	0.940	0.566	0.489	0.470
1.90	1.009	0.968	0.583	0.504	0.484
1.95	1.038	0.997	0.601	0.519	0.498
2.00	1.068	1.026	0.618	0.534	0.513
2.05	1.099	1.055	0.635	0.549	0.527
2.10	1.129	1.084	0.653	0.565	0.562
2.15	1.160	1.114	0.671	0.580	0.557
2.20	1.191	1.143	0.689	0.596	0.572
2.25	1.222	1.173	0.707	0.611	0.587
2.30	1.254	1.204	0.725	0.627	0.602
2.35	1.285	1.234	0.743	0.543	0.617
2.40	1.317	1.265	0.762	0.659	0.632
2.45	1.350	1.296	0.781	0.675	0.648
2.50	1.382	1.327	0.799	0.691	0.663

M_u/bd^2, N/mm²	f_y, N/mm²				
	240	250	415	480	500
2.55	1.415	1.358	0.818	0.708	0.679
2.60	1.448	1.390	0.837	0.724	0.695
2.65	1.482	1.422	0.857	0.741	0.711
2.70	1.515	1.455	0.876	0.758	0.727
2.75	1.549	1.487	0.896	0.775	0.744
2.80	1.584	1.520	0.916	0.792	0.760
2.85	1.618	1.554	0.936	0.809	0.777
2.90	1.653	1.587	0.956	0.827	0.794
2.95	1.689	1.621	0.977	0.844	0.811
3.00	1.724	1.655	0.997	0.862	0.828
3.05	1.760	1.690	1.018	0.880	0.845
3.10	1.797	1.725	1.039	0.898	0.863
3.15	1.834	1.760	1.061	0.917	0.880
3.20	1.871	1.796	1.082	0.936	0.898
3.25	1.909	1.832	1.104	0.954	0.916
3.30	1.947	1.869	1.126	0.973	0.935
3.32	1.962	1.884	1.135	0.981	0.942
3.34	1.978	1.899	1.144	0.989	
3.36	1.993	1.914	1.153		
3.38	2.009	1.929	1.162		
3.40	2.025	1.944	1.171		
3.42	2.040	1.959	1.180		
3.44	2.056	1.974	1.189		
3.46	2.072	1.989			
3.48	2.088	2.005			
3.50	2.104	2.020			
3.52	2.120	2.036			
3.54	2.137	2.051			
3.56	2.153	2.067			
3.58	2.170	2.083			
3.60	2.186	2.099			
3.62	2.203	2.115			
3.64	2.219	2.131			
3.66	2.236	2.147			
3.68	2.253	2.163			
3.70	2.270	2.179			
3.72	2.287	2.196			
3.74	2.304				

NOTE – Blanks Indicate inadmissible reinforcement percentage

Sol.

Given data:

Floor to floor height : 3 m → H

Size of beam : 250 mm × 400 mm → b × D

Live load : 3 kN/m²

Concrete grade : M 25

Steel grade : Fe-500

Waist slab thickness : 150 mm

Treads : 9 @ 250 mm

Width of landing : 1250 mm

$$\text{Height of each flight} = \frac{\text{Floor to floor height}}{\text{No. of flights}}$$

$$= \frac{3}{2} = 1.5\text{m} = 1500\text{mm}$$

Number of risers = Number of treads + 1 = 9 + 1 = 10

$$\text{Height of riser, } R = \frac{\text{Height of flight}}{\text{No. of risers}} = \frac{1500}{10} = 150\text{mm}$$

Dead load of flight

$$\text{Weight of step} = \left(\frac{1}{2}RT\right)\gamma = \frac{1}{2} \times 0.15 \times 0.25 \times 25 = 0.468\text{kN/m}$$

Weight of waist slab = (WB)γ = 0.15 × 0.292 × 25 = 1.095 kN/m

$$\left(B = \sqrt{R^2 + T^2} = \sqrt{0.15^2 + 0.25^2} = 0.292\text{m}\right)$$

Assume wt of floor finishes = 0.437 kN/m

Total dead load per m^2 on plan

$$= (0.468 + 1.095 + 0.437)\frac{1}{0.25} = 8\text{kN/m}^2$$

Live load on plan = 3 kN/m^2

Total load = 8 + 3 = 11 kN/m^2

Factored load w_u = 1.5 × 11 = 16.5 kN/m^2

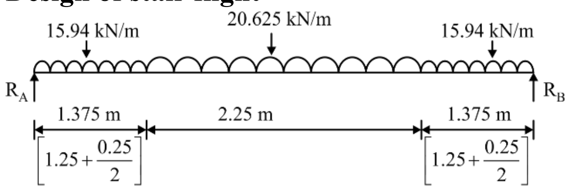
Taking width of slab 1.25 m, load = 1.25 × 16.5 = 20.625 kN/m

Landing

Self weight of slab	= 0.2 × 25 = 5 kN/m^2
Weight of floor fineness	= 0.5 kN/m^2
Live load	= 3 kN/m^2
Total load	= 8.5 kN/m^2
Factored load	= 1.5 × 8.5 = 12.75 kN/m^2

Taking 1.25 cm width of slab load = 1.25 × 12.75 = 15.94 kN/m

Design of stair flight



$$\sum V = 0$$

$$R_A + R_B = 2(15.94 \times 1.375) + (20.625 \times 2.25) = 90.24\text{kN}$$

Taking moment about A to find reaction R_B

$$R_B \times 5 - (15.94 \times 1.375)\left(\frac{1.375}{2} + 2.25 + 1.375\right) - (20.625 \times 2.25)\left(\frac{2.25}{2} + 1.375\right) - (15.94 \times 1.375)\left(\frac{1.375}{2}\right) = 0$$

R_B = 45.12 kN

R_A = 45.12 kN

Max BM

$$M_u = 45.12 \times 2.5 - (15.94 \times 1.375)\left(\frac{1.375}{2} + \frac{2.25}{2}\right) - 20.625 \times \frac{2.25}{2} \times \left(\frac{2.25}{2} + 1.375\right)$$

= 15 kNm

Area of reinforcement required from table assume cover 20 mm and bar dia 10 mm for main steel

effective depth of slab.

$$d = 150 - 20 - \frac{10}{2} = 125\text{mm}$$

Now, $\frac{M_u}{bd^2} = \frac{15\times10^6}{1250\times125^2} = 0.77$

$p = \frac{0.179+0.191}{2} = 0.185\% > 0.12\%$

Area of steel required

$A_{st} = \frac{0.185\times1250\times125}{100} = 289\text{mm}^2$

Spacing required for main steel

$S = 1000\frac{a_{st}}{A_{st}} = 1000\times\frac{\frac{\pi}{4}\times10^2}{289} = 271\text{mm}$

Provide 10 mm & @ 250 mm c/c

Distribution steel

Let us provide mm steel

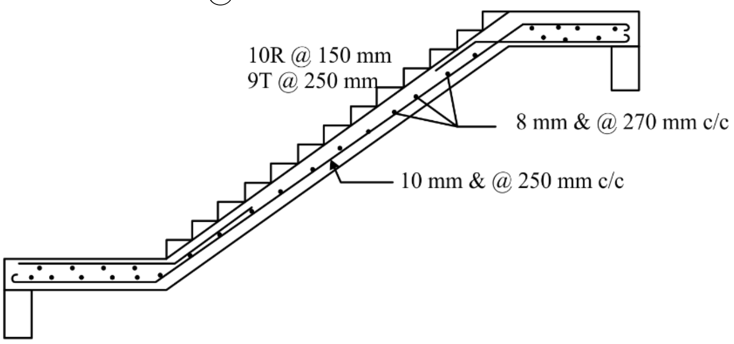
0.12% of bD

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

Spacing req. per m width

$S = 1000\frac{ast}{Ast} = 1000\times\frac{\frac{\pi}{4}\times8^2}{180} = 279\text{mm}$

Provide 8 mm & @ 270 mm c/c



Q.7. (c) (i) A batch of concrete consists of the following ingredients :

Ingredients	Batch Mass (kg)	Specific heat cal/gm/°C	Initial Temperature (°C)
Cement	86	0·27	44
Sand	320	0·25	26·0
Coarse Aggregate	1498	0·23	4·2
Water	35	1·00	1·8
Free Moisture in Sand (3%)	9·0	1·00	26·0
Free Moisture in Coarse Aggregate (1%)	14·5	1·00	4·2
Ice	X	0·50	– 4·0

If the desired placement temperature is 11°C and concrete gains 4 C after cooling has occurred, find the quantity of ice to be added for the given set of materials.

(ii) What are the causes of accidents in the construction industry ? How can they be reduced ?

Sol.

(i)

Ingredients	Batch Mass (kg) (1)	Specific heat cal/gm/°C or k cal/kg/°C (2)	Initial Temperature (°C) (3)	K cal to vary temperature, 1°C (4) = (1 × 2)	Total k cal in material (5) = (3 × 4)
Cement	86	0·27	44	23.22	1021.68
Sand	320	0·25	26·0	80	2080
Coarse Aggregate	1498	0·23	4·2	344.54	1447.068
Water	35	1·00	1·8	35	63
Free Moisture in Sand (3%)	9·0	1·00	26·0	9	234
Free Moisture in Coarse Aggregate (1%)	14·5	1·00	4·2	14.5	60.9
Ice	X	0·50	– 4·0	0.5X	2X
Minus X × [Latent heat of fusion] = 79.6X				= 506.26 + 0.5x	$\frac{-79.6X}{4906.648-77.6X}$

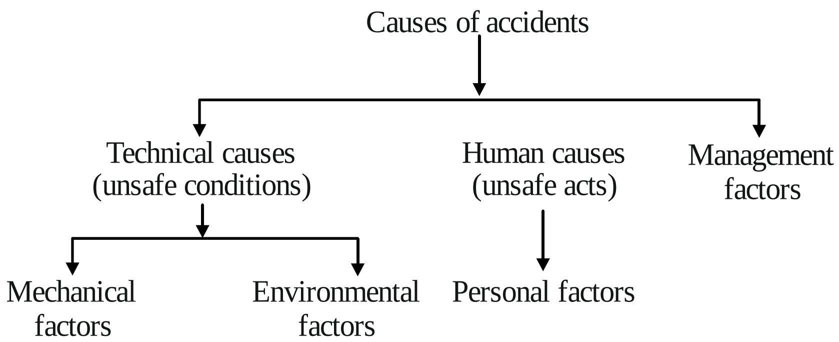
$$\therefore \text{ concrete temperature at mixing} = \left[\frac{4906.648-77.6X}{506.26+0.5X}\right]$$

To achieve desired placement temperature of 11°C and concrete gains 4°C after cooling, concrete temperature at mixing = [11 – 4] = 7°C

$$\Rightarrow 7 = \left[\frac{4906.648-77.6X}{506.26+0.5X}\right] \Rightarrow X = 16.8043\, kg$$

(ii)

Accident is an unplanned incident ,which usually has a specific cause or causes, if one could but discover them.



1. Mechanical Factors: It signifies the unsafe conditions, reflect deficiencies in plant, equipment, tools, materials handling system etc.
2. Environmental Factors: It signifies the unsafe conditions of work environment indicating physical and atmospheric conditions of workplace which indirectly leads to occurrence of accidents.
3. Personal (human) factors: It signifies the unsafe acts by persons concerned are due to ignorance, carelessness, forgetfulness etc.
4. Management factors: Insensitiveness on the part of the management in imparting accident prevention programmes cause accidents.

Accidents can be reduced by following measures.

1. Secure the full support of the top management: The management gets the greatest benefit from an effective safety programme; therefore, it is compulsory on the part of' the top management to come forward and assume a leadership in promoting safety programmes and making the construction point a better and safer place for employees to earn their livelihood.

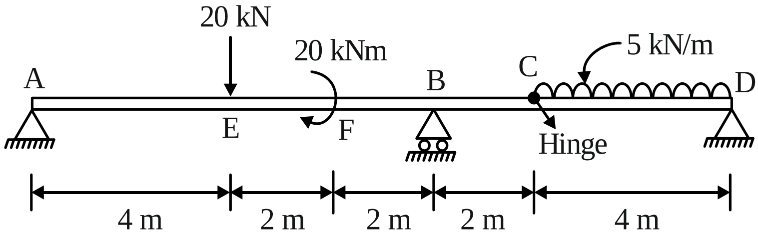
2. Designate someone in the organization to direct safety programmes: Any safety programme should be placed under the direction of a capable person. For small concern, the supervisor of the unit/operation is the right person who Can implement the safety program files along. With his main duties. Depending on size of the industry and hazardousness of operation, a full-fledged safety department may be created with the safety director or safety manager as its chief executive. He/she should be made responsible for all safety training and should have authority to inspect all operations to ensure that adequate safety practices are adopted.

3. Publicize your safety programme: Every employee should be made conscious· of the fact that safety is not the responsibility of one person. Safety is everybody's job.

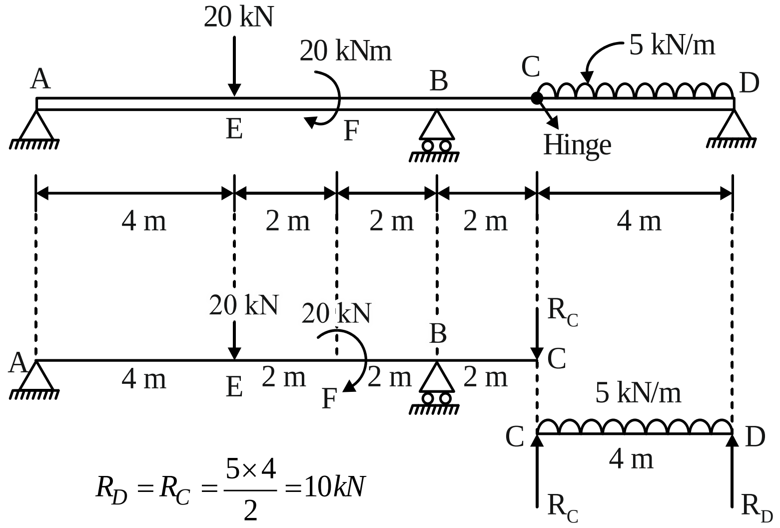
4. Develop a safety programme to each job: Every job has its own hazards; it is not possible to develop a standard safety programme which is equally effective for all types of jobs.

5. Install a safety programme on a competitive basis: There should be a system of awards on annual basis to those supervisors who produce the best safety records. Cash awards and other incentives are good for the purpose. Name. of winners should be published publicly.

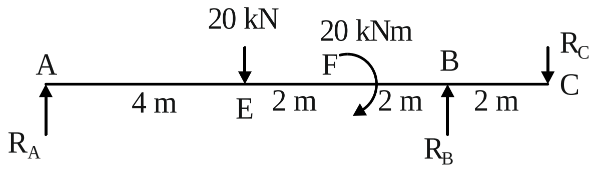
Q.8. (a) Draw the shear force and bending moment diagrams for the beam loaded, as shown in the figure below. There is an internal hinge at C. Determine the values of maximum bending moment and maximum shearing force.



Sol.



$$R_D = R_C = \frac{5\times4}{2} = 10kN$$

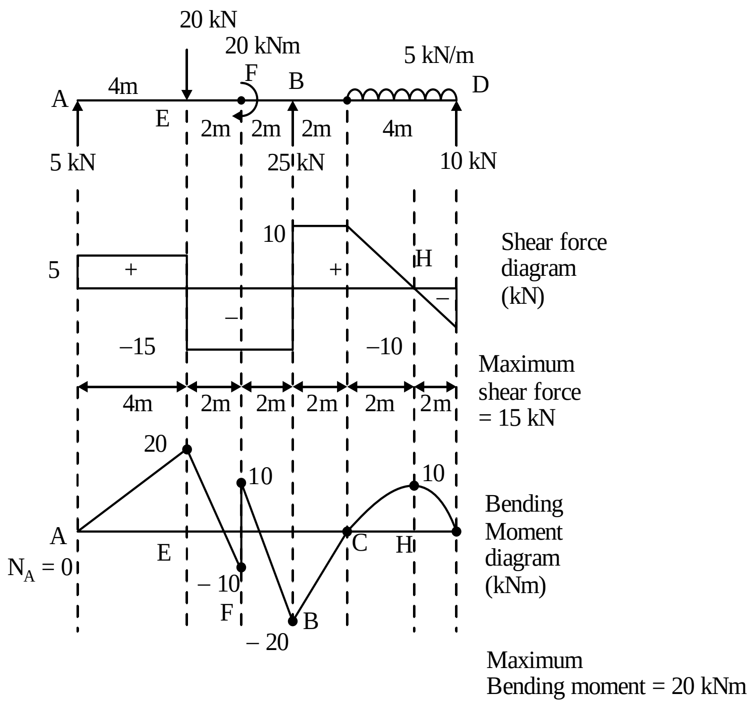


$R_A + R_B = R_C + 20 = 30$ kN

$\Sigma M_B = 0 \Rightarrow R_A \times 8 - 20 \times 4 + 20 + R_C \times 2 = 0$

$\Rightarrow R_A = 5$ kN

$R_B = 30 - 5 = 25$ kN



$M_E - M_A$ = Area of SFD between A and E

$M_E = 5 \times 4 = 20$ kNm

$M_F - M_E$ = Area of SFD between F and E = $-15 \times 2 = -30$

$M_F = M_E - 30 = 20 - 30 = -10$ kNm

- There is sudden jump in bending moment at F due to clockwise moment at F.
- $M_B - M_F$ = Area of SFD between B and F

$= -15 \times 2 = -30$

$\rightarrow M_B = M_F - 30 = 10 - 30 = -20$ kNm

- $M_C - M_B$ = Area of SFD between C and B
  $= 2 \times 10 = 20$
  $\rightarrow M_C = M_B + 20 = -20 + 20 = 0$
- Bending moment diagram has zero slope at mid of CD as shear force is there. Let that that point be H.
- $M_H - M_C$ = Area of SFD between C and H
  $= \frac{1}{2} \times 10 \times 2 = 10$
  $\rightarrow M_H = M_C + 10 = 10$ kNm

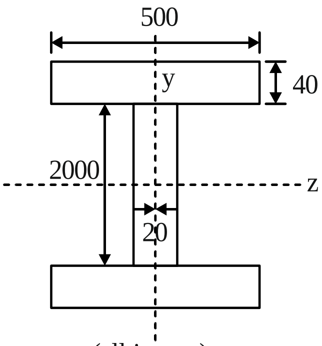
Q.8. (b) (i) A welded plate girder is made of a web 2000 mm deep and 20 mm thick and flange 500 mm wide and 40 mm thick. Design a suitable welded connection between the flange and web. The span of the girder is 30 m (simply supported) and the total load (udl) including its self weight is 160 kN/m. Assume E250 grade of steel. Assume field weld.

(ii) Find the moment of resistance of reinforced concrete beam 250 mm × 500 mm. The beam is reinforced with 3 Nos. 20 ϕ bar at bottom and 2 Nos. 12 ϕ bar at top. The beam is simply supported and under vertical load. Assume effective cover to bar at the top and the bottom is 50 mm. Use M25 and Fe415. Apply limit state method. Assume the stress strain values of a steel bar from the table.

Strain	Stress (MPa)
0·00174	347·8
0·00195	369·6
0·00226	391·3
0·00277	413·0
0·00312	423·9
0·00417	434·8

Sol.
(i)

$l = 30$ m
w = 160 kN/m
Fc 250
Field weld, so $\gamma_{mw} = 1.5$



(all in mm)

Let welding is done on both sides of the web

q_w = shear force in weld per unit length

$$q_w = \frac{VA\bar{y}}{2I_{zz}}$$

$$I_{zz} = \frac{500\times(2080)^3}{12} - \left(\frac{480\times 2000^3}{12}\right)$$

$= 5.495 \times 10^{10}$ mm^4

For simply supported girder shear force $= \dfrac{wL}{2}$

w_u = factored load = 1.5 × 160 kN/m

$$V = \frac{1.5\times160\times30}{2} = 3600kN.$$

$$A\bar{y} = 500\times40(1000+20)$$

$$q_w = \frac{1}{2}\left(\frac{3600\times10^3\times500\times40\times1020}{5.495\times10^{10}}\right)$$

= 668.24 N/mm

Assuming size of weld 's'

= q_w < strength of weld

$$= 668.24 < \frac{f_y \times ks}{\sqrt{3}\,\gamma_{mw}}$$

$$= 668.24 < \frac{250\times0.7\times s}{\sqrt{3}\times1.5}$$

= 9.92 < b.

= Let's provide size of weld = 10 mm

At both side of junction of flange and web.

(ii)

Given data

Size of beam = (250 × 500) mm ⟶ b × D

Tension steel = 3 – 20 mm φ – A_{st}

Compression steel = 2 – 12 mm φ – A_{sc}

Type of beam : Simply supported

Effective cover : 50 mm – (for both tension & compression)

Concrete grade : M25

Steel grade : Fe 415

Effective depth of beam

d = D – effective cover

= 500 – 50 = 450 mm

Limiting Depth,

$x_{u,lim}$ = 0.48 d ⟶ for Fe415

$x_{u,lim}$ = 0.48 × 450 = 216 mm

Actual Depth of N.A,

$0.36\, f_{ck}\, b x_u + (f_{sc} - f_{cc})A_{sc} = 0.87\, f_y\, A_{st}$

$0.36 \times 25 \times 250 \times x_u + (f_{sc} - 0.446 \times 25)\ 2 \times \frac{\pi}{4} \times 12^2$

$= 0.87 \times 415 \times 3 \times \frac{\pi}{4} \times 20^2$

$2250\ x_u (f_{sc} - 11.15)\ 226 = 340.28 \times 10^3$ -------------(1)

In the above equation, there are two unknown which can be determined by trial and error.

Trial–1

Assume $x_u = x_{u,lim} = 216$ mm

$$\in_{sc} = 0.0035\left[1 - \frac{d'}{x_u}\right]$$

$$= 0.0035\left[1 - \frac{50}{216}\right] = 0.00269$$

f_{sc} from table

$$f_{sc} = 391.3 + \frac{413 - 391.3}{0.00277 - 0.00226} \times (0.00269 - 0.00226)$$

$= 409.59$ N/mm^2

Put f_{sc} in equation (1) we get x_u

$2250\ x_u + (409.59 - 11.15) \times 226 = 340.28 \times 10^3$

$x_u = 111.21$ mm < 216 mm $\therefore$ Not ok

Hence go to next trial

Trial (2)

Assume $x_u = 111.21$ mm

$$\in_{sc} = 0.0035\left[1 - \frac{50}{111.21}\right] = 0.00193$$

$$f_{sc} = 347.8 + \frac{369.6 - 347.8}{0.00195 - 0.00174} \times (0.00193 - 0.00174)$$

$f_{sc} = 348.7$ N/mm^2

put equation (i)

$2250 \times x_u + (348.7 - 11.15)\ 226 = 340.28 \times 10^3$

$x_u = 117.3$ mm > 111.21 mm $\therefore$ Not ok

Hence go to next trial

Trial (3)

Assume $x_u = 117.3$ mm

$$\in_{sc} = 0.0035\left[1 - \frac{50}{117.3}\right] = 0.002$$

$$f_{sc} = 369.6 + \left(\frac{391.3 - 369.6}{0.00226 - 0.00195}\right) \times (0.002 - 0.00195)$$

$f_{sc} = 373.1$ N/mm^2

put equation (1)

$= 2250\ x_u + (373.1 - 11.15)\ 226 = 340.\ 28 \times 10^3$

$x_u = 114.88 < 117.3$ mm $\therefore$ Not ok

hence go to next trial

Trial 4

Assume x_u = 114.88 mm

$$\in_{sc} = 0.0035 \times \left[1 - \frac{50}{114.88}\right] = 0.00198$$

$$f_{sc} = 369.6 + \left(\frac{391.3 - 369.6}{0.00226 - 0.00195}\right) \times (0.00198 - 0.00195)$$

f_{sc} = 371.7 N/mm^2

Put in equation (i)

2250 x_u + (371.7 – 11.15) 226 = 340.28 × 28 × 10^3

x_u = 115 = 114.88 mm ∴ ok

Correct f_{sc} = 371.7 N/mm^2 and x_u = 115 mm

Here $x_u < xu_{lim}$

∴ under reinforced section

MOR = 0.36 b_{ck} × b × x_u (d – 0.42 xu) + (f_{sc} – f_{cc}) A_{sc} [d – d']

= 0.36 × 25 × 250 × 115 (450 – 0.42 × 115) + (371.7 – 0.42 × 25) × 226 × (450 – 50)

MOR = (103.94 × 10^6 + 32.59 × 10^6) N-mm

MOR = 136.53 kN-m

Q.8. (c) (i) What is a sheep's foot roller? How does it compact the earth?

(ii) Explain the following terms :

- Total float
- Free float
- Independent float
- Interfering float

Sol.

(i)

Sheep- Foot rollers has a hollow steel drum with cylindrical pads (or feet) as a projections on its surface . These projections are generally 8-10 inches in length which are arranged around the drum (also called wheel) at 100-200 mm c/c along the axis. They have weight around 15 tonnes or more and travel at a speed of 25 Kmph using wheel tractor for towing.

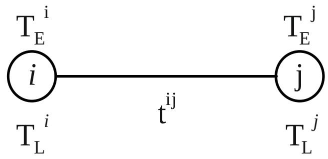
They do the compaction is by kneading action, when roller travels over the earth surface, the feet penetrate the soil by kneading action to mix and compact the soil from bottom to top layer. Penetration of feet decreases with number of passages. Depth of a layer of soil to be compacted is limited to approximately the length of the feet.

Sheep's foot rollers are suitable for compacting fine grained materials such as clays and mixtures of sand and clay. These cannot compact granular soils such as sand and gravel.

(ii)

1. Total Float (F_T)

- Total Float is the maximum time by which an activity can be delayed without affecting project completion time.
- Also, Total Float is maximum time over activity time i.e., when activity starts as early as possible and completes as late as possible.
- Total Float = Maximum Available Time – Activity Time



$$F_T = \left(T_L^j - T_E^i\right) - t^{ij}$$

F_T = LST – EST = LFT – EFT

2. Free Float (FF)

Free Float is the extra time by which an activity can be delayed without affecting earliest start time of succeeding activity.

Also, Free Float is excess time over activity time such that when all the activities start as early as possible.

$$F_F = \left(T_E^j - T_E^i\right) - t^{ij}$$

3. Independent Float (FID)

Independent Float is the minimum time by which an activity can be delayed.

Independent float is excess of minimum time over activity time such that when preceding activity completes as late as possible and succeeding activity starts as early as possible.

$$F_{ID} = \left(T_E^j - T_L^i\right) - t^{ij}$$

$$F_{ID} = F_F - S_i$$

4. Interfering Float (FIN)

It is difference between total float and free float, which is also equal to head event slack.

$F_{IN} = S_j = F_T - F_F$

THANK YOU!