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Detailed Solutions

ESE-2026 Mains Test Series

Civil Engineering Test No : 8

Section A : Water Resource Engineering + Building Materials + Railway, Airport Tunnelling & Harbour

1. (a) Solution:

Base Period (*B*): It is the total time elapsed between the first watering of a crop at the time of its sowing to its last watering before harvesting. It is expressed in days.

Duty (*D*): It is defined as the area of land in hectares which can be irrigated for growing any crop if one cumec (1 m^3/s) of water is supplied continuously to the land for the entire base period of the crop. It is expressed in hectares per cumec (ha/cumec).

Delta (Delta): It is the total depth of water applied over an irrigated land at different waterings throughout the entire base period of the crop. It is denoted by the symbol 'Δ' and is expressed in *cm* or *m*.

Relationship between Duty (*D*), Delta (Δ) and Base Period (*B*):

If D is the duty of water in hectares per cumec, Δ is the total depth of water supplied during the entire base period, and B is the base period of the crop in days, then for a field of area D hectares corresponding to the depth of water Δ metres:

The total quantity of water supplied for growing a crop on the field is:

$$\text{Volume} = D \times \Delta \text{ ha-m} = D \times \Delta \times 10^4 \text{ m}^3 \quad \text{...(i)}$$

Further, for the same field of area D hectares, if water is supplied at 1 m^3/s for the entire base period of B days, the total quantity of water supplied is:

$$\text{Volume} = 1 \times B \times 24 \times 60 \times 60 \text{ m}^3 \quad \text{...(ii)}$$

Equating (i) and (ii):

$$D \times \Delta \times 10^4 = B \times 24 \times 60 \times 60$$

$$D \times \Delta \times 10^4 = B \times 8.64 \times 10^4$$

$$D = \frac{8.64 \times B}{\Delta}$$

Where:

- D is the duty of crop in ha/cumec
- B is the base period in days
- D is the delta of crop in metres

1. (b) Solution:

1. **Efflorescence:** This defect is caused by the presence of soluble salts (alkalies) in the brick clay. When the brick absorbs moisture, these salts dissolve and migrate to the surface through capillary action. Upon evaporation of the water, a white or greyish crystalline powder is left on the surface. If the salt content is high, it can lead to the disintegration of the brick surface, a process known as "spalling."
2. **Bloating:** Bloating is characterized by a permanent swelling or spongy appearance of the brick. This occurs during the burning stage when the clay contains excess carbonaceous matter and sulfur. As these substances burn, they release gases that become trapped within the softened clay mass, causing the brick to expand and lose its intended dimensions and density.
3. **Chuffing:** Chuffing refers to the deformation or cracking of bricks caused by the impact of rain or sudden moisture on "hot" bricks. This usually happens in kilns where the bricks have not been allowed to cool down gradually. The rapid contraction caused by the temperature difference results in structural cracks and a significant loss of strength.
4. **Black Core:** When the brick clay contains bituminous matter or organic carbon, and the firing process is too rapid or the kiln atmosphere is not sufficiently oxidizing, the carbon in the center of the brick is not fully burnt out. This leaves a dark, unoxidized "black core" in the middle. Such bricks are generally more brittle and have lower compressive strength compared to fully oxidized bricks.
5. **Spots:** Iron spots or dark specks appear on the surface of the brick due to the presence of iron sulfide (pyrites) in the raw clay. During the burning process, these minerals oxidize and leave dark, unsightly marks. While often an aesthetic defect, in high concentrations, they can act as focal points for chemical weathering.

6. **Nodules of Free Lime:** When clay contains small lumps or nodules of limestone (calcium carbonate), they are converted into "quicklime" (calcium oxide) during burning. When the finished brick is later exposed to moisture, this quicklime slakes and expands in volume. This internal expansion exerts enough pressure to cause the brick to crack or "pop" from the inside, a defect commonly called "lime popping."

1. (c) Solution:

Alkali aggregate reaction is a chemical reaction that takes place in concrete between the alkalis present in cement and certain reactive constituents of aggregates in the presence of moisture. The alkalis mainly consist of sodium oxide and potassium oxide present in cement. This reaction produces an expansive gel, which absorbs water and swells, causing internal stresses, cracking, and deterioration of concrete.

Essential Conditions for Reaction

- High alkali content in cement
- Presence of reactive aggregates
- Availability of moisture

All three conditions must be present for the reaction to occur.

Causes of Alkali Aggregate Reaction

- Use of high alkali cement
- Use of aggregates containing reactive silica or dolomitic limestone
- Exposure to high moisture conditions
- High temperature which accelerates the reaction
- Poor material selection and lack of proper testing

Prevention of Alkali Aggregate Reaction

- Use of low alkali cement
- Use of non-reactive aggregates after petrographic examination
- Limiting total alkali content in concrete
- Use of supplementary cementitious materials such as fly ash, ground granulated blast furnace slag and silica fume
- Reducing permeability of concrete by proper mix design and compaction
- Providing good drainage and waterproofing to minimize moisture ingress

1. (d) (i) Solution:

The advantages of lined canals over unlined canals are as follows–

(a) Prevention of Seepage Loss: Lining significantly reduces water loss through the canal bed and banks, ensuring a higher percentage of diverted water reaches the fields.

(b) Prevention of Waterlogging: By eliminating or reducing seepage, lining prevents the surrounding water table from rising, which avoids the waterlogging and salinity of adjacent agricultural lands.

(c) Increased Discharge Capacity: Lined canals have smoother surfaces compared to unlined ones, reducing friction and allowing for higher flow velocities and increased discharge within the same cross-sectional area.

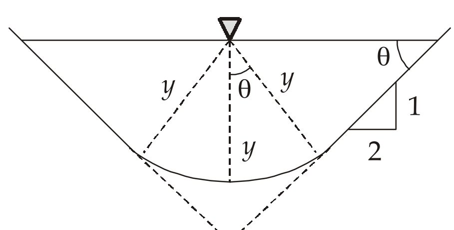
(d) Reduced Maintenance Costs: The hard surface of a lining prevents the growth of weeds and aquatic plants that often obstruct flow in unlined canals.

(e) Prevention of Siltation: The higher flow velocities achieved in lined canals keep silt in suspension, reducing the need for frequent desalting operations.

(f) Protection Against Erosion: Lining protects the canal structure from scouring and erosion caused by high-velocity water flow.

(g) Smaller Cross-Sectional Area: Due to improved hydraulic efficiency, a lined canal can be smaller than an unlined one while carrying the same discharge, potentially reducing land acquisition and earthwork costs.

1. (d) (ii) Solution:



For the given side slope,

$$\tan\theta = \frac{1}{2}$$

$$\Rightarrow \quad \cot\theta = 2$$

$$\Rightarrow \quad \theta = 26.565° = 0.4636 \text{ radians}$$

The area of flow for the given section is

$$A = y^2(\theta + \cot\theta)$$

$$\Rightarrow \quad A = y^2(0.4636 + 2)$$

$$\Rightarrow \quad A = 2.4636\, y^2$$

The wetted perimeter is

$$P = 2y(\theta + \cot\theta)$$

$$\Rightarrow \quad P = 2y(0.4636 + 2)$$

$$\Rightarrow \quad P = 4.9272\, y$$

The hydraulic radius is

$$R = \frac{A}{P} = \frac{2.4636y^2}{4.9272y}$$

$$\Rightarrow \quad R = \frac{y}{2}$$

Using Manning's equation,

$$Q = A \times \frac{1}{n} R^{2/3} S^{1/2}$$

$$\Rightarrow \quad 30 = 2.4636\, y^2 \times \frac{1}{0.012} \times \left(\frac{y}{2}\right)^{2/3} \times \left(\frac{1}{2000}\right)^{1/2}$$

$$\Rightarrow \quad y^{8/3} = 10.374$$

$$\Rightarrow \quad y = 2.4 \text{ m}$$

Therefore, the uniform flow depth is approximately 2.4 m.

1. (e) Solution:

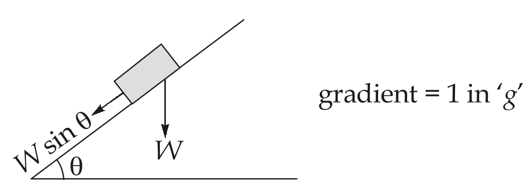
Given data

Number of wagons = 25

Weight of each wagon = 20 tonnes

Total weight of wagons,

$$W_w = 25 \times 20 = 500 \text{ tonnes}$$



Weight of locomotive,

$$W_l = 130 \text{ tonnes}$$

Total weight of train,

$$W = W_w + W_l = 500 + 130 = 630 \text{ tonnes}$$

Speed of train,

$$V = 60 \text{ kmph}$$

Tractive effort,

$$F_t = 18 \text{ tonnes}$$

Rolling resistance of wagons = 2.8 kg/tonne

Rolling resistance of locomotive = 3.8 kg/tonne

Speed dependent resistance,

$$R_2 = 3.15 \text{ tonnes}$$

Rolling resistance due to wagons and locomotive is given by

$$R_1 = \frac{(2.8 \times 500) + (3.8 \times 130)}{1000}$$

$$\Rightarrow \quad R_1 = 1.894 \text{ tonnes}$$

Resistance dependent on speed is given as

$$R_2 = 3.15 \text{ tonnes} \qquad \text{(given)}$$

Atmospheric resistance is given by

$$R_3 = 0.0000006 \times W \times V^2$$

$$\Rightarrow \quad R_3 = 0.0000006 \times 630 \times 60^2$$

$$\Rightarrow \quad R_3 = 1.361 \text{ tonnes}$$

Let the steepest gradient be 1 in g.

Resistance due to gradient is

$$R_4 = W \sin\theta = W \times \left(\frac{1}{g}\right)$$

$$\Rightarrow \quad R_4 = \frac{630}{g}$$

For the steepest gradient, total resistance is equal to the tractive effort.

$$F_t = R_1 + R_2 + R_3 + R_4$$

$$\Rightarrow \quad 18 = 1.894 + 3.15 + 1.361 + \frac{630}{g}$$

$$\Rightarrow \quad 18 = 6.405 + \frac{630}{g}$$

$$\Rightarrow \quad \frac{630}{g} = 18 - 6.405$$

$$\Rightarrow \quad \frac{630}{g} = 11.595$$

$$\Rightarrow \quad g = 54.334$$

The steepest gradient for the given conditions is

$$\text{Gradient} = 1 \text{ in } 54.334$$

2. (a) Solution:

First, the gross command area at 1 km is calculated.

$$GCA_1 = 30000 - 6000 = 24000 \text{ ha}$$

The culturable command area at 1 km is

$$CCA_1 = 0.80 \times 24000 = 19200 \text{ ha}$$

For Rabi season, the area to be irrigated is

$$A_r = 19200 \times 0.25 = 4800 \text{ ha}$$

Duty is calculated using

$$D = \frac{8.64B}{\Delta}$$

$$\Rightarrow \quad D_r = \frac{8.64 \times (4 \times 7)}{0.14} = 1728 \text{ ha/cumec}$$

Net discharge required for Rabi is

$$Q_{r(\text{net})} = \frac{A_r}{D_r}$$

$$Q_{r(\text{net})} = \frac{4800}{1728} = 2.778 \text{ m}^3/\text{s}$$

For Kharif season, the area to be irrigated is

$$A_k = 19200 \times 0.15 = 2880 \text{ ha}$$

Duty, $$D_k = \frac{8.64 \times (2.5 \times 7)}{0.18} = 840 \text{ ha/cumec}$$

Discharge, $$Q_{k(\text{net})} = \frac{2880}{840} = 3.429 \text{ m}^3/\text{s}$$

The maximum net irrigation discharge at 1 km is

$$Q_{\text{net}} = 3.429 \text{ m}^3/\text{s}$$

Next, conveyance losses below 1 km are calculated.

For reach 1-2 km, bed width B = 6.5 m, depth y = 1.2 m, side slope z = 0.5.

Wetted perimeter is

$$P = B + 2y\sqrt{1+z^2}$$

$$\Rightarrow \quad P = 6.5 + 2 \times 1.2\sqrt{1+0.5^2}$$

$$\Rightarrow \quad P = 9.183 \text{ m}$$

Wetted area over 1 km length is

$$A_w = P \times 1000 = 9183 \text{ m}^2$$

Loss in 1.2 km reach is

$$\text{Loss}_{1-2} = \frac{2.5}{10^6} \times 9183 = 0.023 \text{ cumec}$$

For reach 2-3 km, bed width B = 5.0 m, depth y = 1.2 m.

Wetted perimeter, $P = 5.0 + 2 \times 1.2\sqrt{1+0.5^2}$

$$\Rightarrow \quad P = 7.683 \text{ m}$$

Now, $A_w = 7.683 \times 1000 = 7683 \text{ m}^2$

$$Loss_{2-3} = \frac{2.5}{10^6} \times 7683 = 0.019 \text{ cumec}$$

Total losses below 1 km are

$$Loss_{\text{total}} = 0.023 + 0.019 + 0.8$$

$$Loss_{\text{total}} = 0.842 \text{ cumec}$$

Finally, the design discharge required at 1 km is

$$Q_{\text{design}} = Q_{\text{net}} + Loss_{\text{total}}$$

$$Q_{\text{design}} = 3.429 + 0.842 = 4.271 \text{ m}^3/\text{s}$$

The design canal discharge required at 1 km is 4.271 m^3/s.

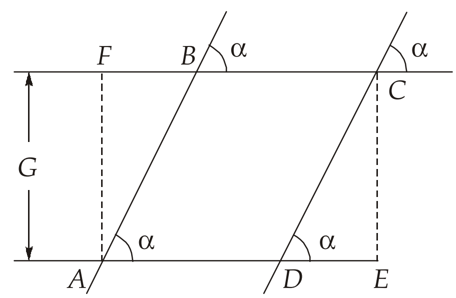
2. (b) (i) Solution:

Given data

$$\text{Crossing number, } N = 12$$

$$\text{Gauge, } G = 1.676 \text{ m}$$

$$\tan \alpha = \frac{1}{12}$$



Angle of crossing:

$$\alpha = \tan^{-1}\left(\frac{1}{12}\right) = 4.764°$$

1. Length of sides of diamond ($AB = BC = CD = DA$):

$$AB = G \text{ cosec } \alpha$$

$$\Rightarrow \quad AB = 1.676 \times \text{cosec } 4.764°$$

$$\Rightarrow \quad AB = 20.181 \text{ m}$$

2. Length of shorter diagonal (BD):

$$BD = G \sec\left(\frac{\alpha}{2}\right)$$

$$\Rightarrow \quad BD = 1.676 \times \sec\left(\frac{4.764°}{2}\right)$$

$$\Rightarrow \quad BD = 1.677 \text{ m}$$

3. Length of longer diagonal (AC):

$$AC = G \text{ cosec}\left(\frac{\alpha}{2}\right)$$

$$\Rightarrow \quad AC = 1.676 \times \text{cosec}\left(\frac{4.764°}{2}\right)$$

$$\Rightarrow \quad AC = 40.326 \text{ m}$$

4. Horizontal projections ($BF = ED$):

$$BF = G \cot \alpha$$

$$\Rightarrow \quad BF = 1.676 \times 12$$

$$\Rightarrow \quad BF = 20.112 \text{ m}$$

Summary of design:

Side lengths, $AB = BC = CD = DA = 20.181$ m

Short diagonal (BD) = 1.677 m

Long diagonal (AC) = 40.326 m

Horizontal projection, $BF = ED$ = 20.112 m

2. (b) (ii) Solution:

Given data

Turning speed, V = 40 kmph

Wheelbase, W = 22 m

Tread of main landing gear, T' = 5.5 m

Coefficient of friction, f = 0.15

Width of taxiway pavement, T = 22 m

The radius of the taxiway will be the maximum value obtained from the following three criteria.

1. Based on turning speed and friction,

$$R = \frac{V^2}{125 \times f} = \frac{40^2}{125 \times 0.15}$$

$$\Rightarrow \quad R = 85.333 \text{ m}$$

2. Based on Horonjeff's equation,

$$R = \frac{0.388 \times W^2}{\frac{T}{2} - S}$$

Where S is the distance between the midway point of the main gears and the edge of the taxiway pavement.

Assuming an oleo distance (clearance) of 6 m,

$$S = \text{Oleo distance} + \frac{T'}{2}$$

$$\Rightarrow \quad S = 6 + \frac{5.5}{2} = 8.75 \text{ m}$$

Substituting the values,

$$\Rightarrow \qquad R = \frac{0.388 \times 22^2}{\frac{22}{2} - 8.75} = \frac{187.792}{2.25}$$

$$\Rightarrow \qquad R = 83.463 \text{ m}$$

3. **Minimum radius requirement for subsonic aircraft,**

   For subsonic aircraft, the radius of the taxiway should not be less than 120 m.

   Comparing the values obtained, The maximum of the above values governs the design. Therefore, the radius of the taxiway is 120 m.

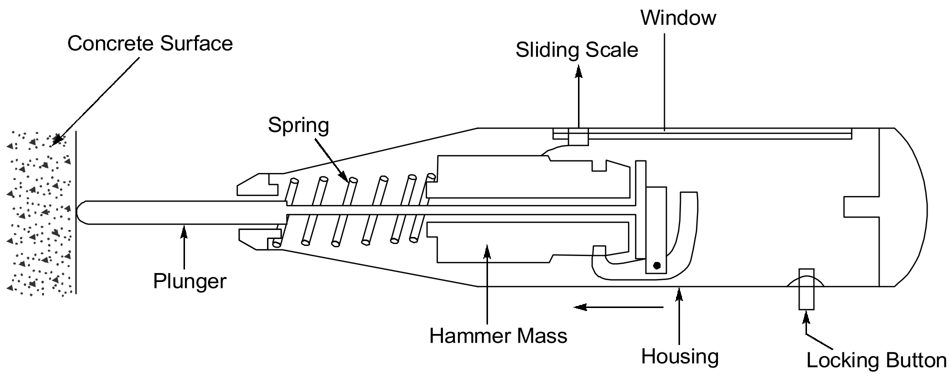
2. (c) Solution:

Non-destructive testing (NDT) is a method of evaluating the properties of a concrete structure without causing any physical damage to the member being tested. Unlike cube testing, which tells us the potential strength of the concrete mix, NDT provides information on the actual "in-situ" strength and quality of the concrete in the finished structure.

1. **Relative Merits of NDT**
   - **Preservation of Structure:** Tests can be performed on the actual structural members without damaging or weakening them.
   - **Assessment of In-situ Concrete:** It allows for the evaluation of concrete as it exists in the structure, accounting for the effects of placement, compaction, and curing.
   - **Speed and Efficiency:** Most NDT methods are fast, allowing for a large number of tests to be conducted in a short period.
   - **Cost-Effective:** It eliminates the need for expensive core drilling and subsequent structural repairs.
   - **Monitoring over Time:** Enables periodic monitoring of the health and durability of a structure throughout its service life.
2. **Common NDT Methods**
   - Rebound Hammer Test (Schmidt Hammer)
   - Ultrasonic Pulse Velocity (UPV) Test
   - Pull-out Test
   - Penetration Resistance Test (Windsor Probe)
   - Radio-active Methods (X-ray and Gamma-ray)
   - Corrosion Potential Assessment

3. **Detailed Explanation: Rebound Hammer Test**

The Rebound Hammer test is the most commonly used NDT method due to its simplicity and portability.



Principle:

The test is based on the principle that the rebound of an elastic mass depends on the hardness of the surface against which the mass strikes. When the plunger of the rebound hammer is pressed against the surface of the concrete, a spring-controlled mass rebounds and takes a reading on a graduated scale, known as the **Rebound Number.**

Procedure:

1. **Surface Preparation:** The surface of the concrete should be smooth, clean, and dry. Any loose material or plaster should be removed using a grinding stone.
2. **Testing:** The hammer is held perpendicular to the surface and pressed until the impact spring is released.
3. **Recording:** The rebound number is recorded from the scale. Usually, 10 to 12 readings are taken in a small area, and the average is calculated.
4. **Correlation:** The average rebound number is then correlated with compressive strength using a calibration chart provided by the manufacturer.

Factors Affecting Results:

- **Type of Aggregate:** Harder aggregates yield higher rebound numbers.
- **Surface Smoothness:** Trowelled surfaces give higher readings than floated surfaces.
- **Moisture Content:** Wet concrete typically shows a lower rebound number than dry concrete.
- **Carbonation:** Concrete with a carbonated surface layer can give significantly higher (misleading) rebound numbers.

3. (a) (i) Solution:

Field Capacity

Field capacity is the maximum amount of water that a soil can retain against the force of gravity. It is attained after excess gravitational water has drained away, generally 2 to 3 days after heavy rainfall or irrigation. At this stage, micropores of the soil are filled with water while macropores contain air.

Permanent Wilting Point

Permanent wilting point is the soil moisture content at which plants are no longer able to extract sufficient water to meet their transpiration requirements. The water is held so tightly by soil particles that plants wilt permanently and do not recover even under humid conditions.

Average Moisture Content

In irrigation practice, the average moisture content refers to the soil moisture level maintained in the root zone between two irrigations. For healthy crop growth, soil moisture is not allowed to fall to the permanent wilting point. Instead, irrigation is applied when moisture content reaches a predetermined optimum value to avoid plant stress.

Available Moisture

Available moisture is the difference between field capacity and permanent wilting point,

$$\text{Available moisture} = FC - PWP$$

Net Irrigation Requirement

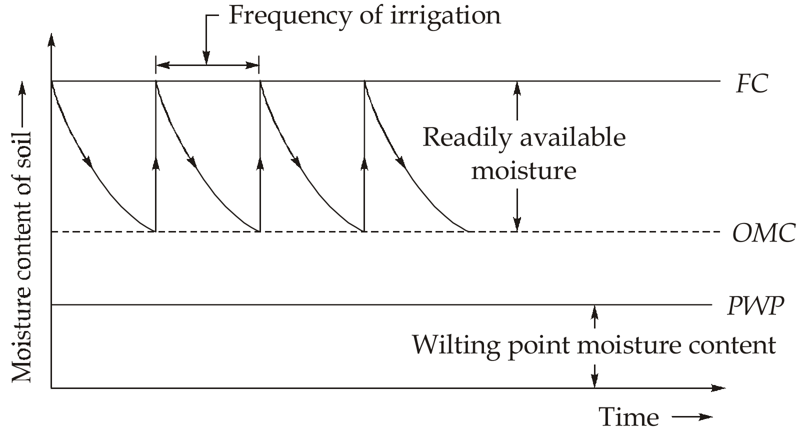
If irrigation is scheduled at an optimum moisture content *OMC*, the depth of water to be applied is

$$d_w = \frac{\gamma_d D}{\gamma_w}(FC - OMC)$$

Irrigation Frequency

The frequency of irrigation is determined by the rate at which available moisture is depleted by consumptive use. The time interval between two irrigations is

$$\text{Frequency} = \frac{d_w}{C_U}$$

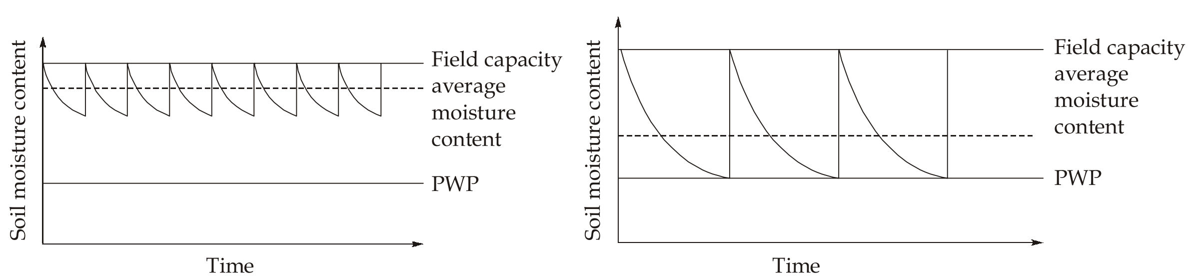


More frequent irrigation

If irrigation is applied soon after field capacity is reached, the depth of water applied per irrigation is small, but the frequency is high. This maintains higher average soil moisture and minimizes plant stress.

Less frequent irrigation

If soil moisture is allowed to fall close to the permanent wilting point, the depth of water applied per irrigation is large, but the frequency is low. This may result in plant stress and reduced crop yield.



(a) More frequent irrigation (b) Less frequent irrigation

3. (a) (ii) Solution:

Given Data

Moisture holding capacity = 160 mm/m

Root depth (D) = 50 cm = 0.5 m

Allowable depletion = 40%

Daily water use (C_U) = 6 mm/day

Area to be irrigated (A) = 45 *ha* = 45×10^4 m^2

Rate of delivery = 35 *l*ps

Irrigation application efficiency (η_a) = 65% = 0.65

1. Allowable depletion depth between irrigations

Total moisture storage capacity = 160 × 0.5 = 80 mm

$$\text{Allowable depletion depth} = \frac{40}{100} \times 80 = 32 \text{ mm}$$

2. Frequency of irrigation

$$\text{Frequency} = \frac{32}{6} = 5.333 \text{ days}$$

3. Net application depth of water

$$\text{Net application depth} = \frac{32}{0.65} = 49.231 \text{ mm}$$

4. Volume of water required

$$\text{Volume} = 45 \times 10^4 \times 49.231 \times 10^{-3} = 22.154 \times 10^3 \text{ m}^3$$

$$\Rightarrow \quad V = 22.154 \times 10^6 \text{ litres} = 22.154 \text{ mL}$$

5. Time to irrigate 5 ha plot

$$\text{Volume for 5 ha} = 5 \times 10^4 \times 49.231 \times 10^{-3} = 2.462 \times 10^3 = 2.462 \times 10^6 \text{ litres}$$

$$\text{Time} = \frac{2.462 \times 10^6}{35} = 70342.857\ s = \frac{70342.857}{3600} \text{ hrs}$$

$$\Rightarrow \quad T = 19.540 \text{ hrs}$$

3. (b) (i) Solution:

1. **Definition of Setting Times:** Setting of cement refers to the transition of cement paste from a fluid or plastic state to a rigid state. This is distinct from "hardening," which refers to the gain of mechanical strength.
   - **Initial Setting Time:** This is the time elapsed between the moment water is added to the cement and the time when the paste starts losing its plasticity. It marks the limit of the time available for mixing, transporting, and placing the concrete. For Ordinary Portland Cement (OPC), this should not be less than 30 minutes.
   - **Final Setting Time:** This is the time elapsed between the moment water is added to the cement and the time when the paste has completely lost its plasticity and attained sufficient firmness to resist a certain pressure. For OPC, this should not exceed 600 minutes (10 hours).

2. **Experimental Determination (Vicat Apparatus Test):** The setting times are determined using the Vicat Apparatus, which consists of a frame with a movable rod and a set of interchangeable needles.

Preparation of Test Block:

Before starting, the quantity of water required to produce a paste of standard consistency (P) must be known. For the setting time test, a paste is prepared by gauging cement with 0.85 times P water by weight. The paste is filled into the Vicat mould and leveled.

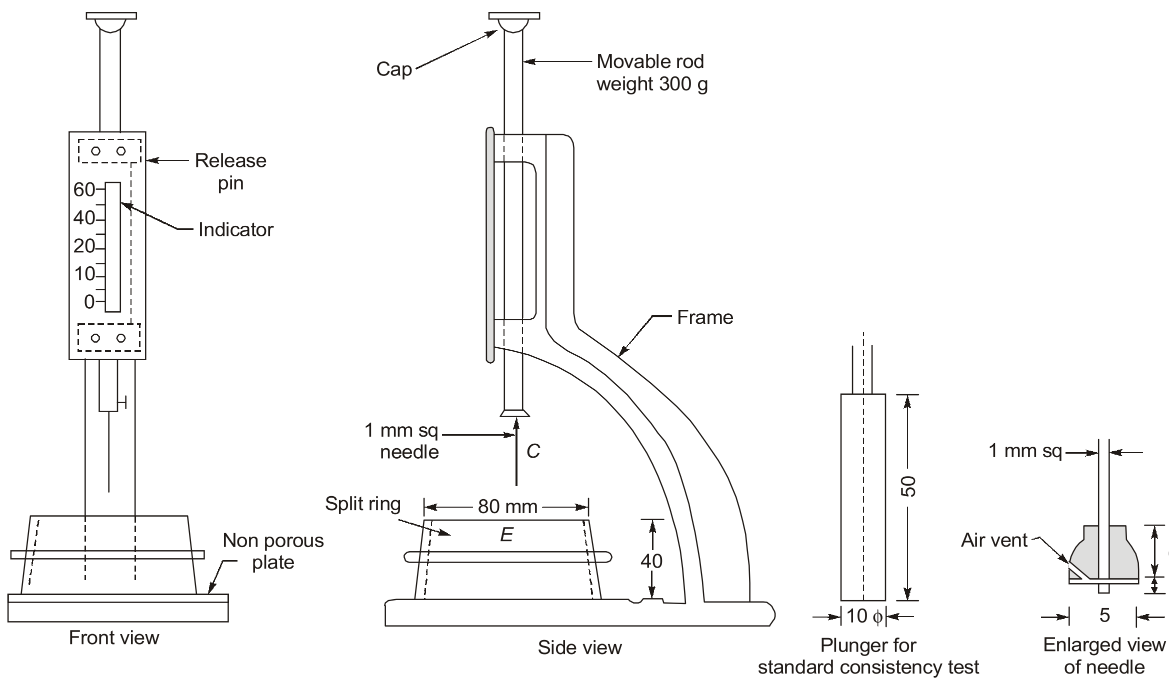


Fig. *Vical apparatus with needle for initial section time test*

Determination of Initial Setting Time:

- The 1 mm^2 square needle (Needle C) is attached to the rod of the apparatus.
- The needle is lowered gently to touch the surface of the test block and then quickly released.
- Initially, the needle pierces the block completely. The procedure is repeated at regular intervals.
- The time at which the needle fails to pierce the block beyond 5 mm to 7 mm from the bottom of the mould is recorded as the initial setting time.

Determination of Final Setting Time:

- The square needle is replaced by the needle with an annular attachment (Needle *F*).
- The cement is considered finally set when, upon lowering the needle gently to the surface, the needle makes an impression, but the circular attachment fails to do so.
- The time elapsed since the addition of water to this point is the final setting time.

3. **Roles of Gypsum and Calcium Chloride**

These substances are known as admixtures that modify the hydration rate of the cement clinker.

Role of Gypsum ($CaSO_4 \cdot 2H_2O$):

Gypsum acts as a Retarder. During the grinding of cement clinker, a small amount of gypsum is added. Without gypsum, the Tricalcium Aluminate (CA) would react almost instantly with water, leading to "flash set" (premature hardening). Gypsum reacts with C_3A to form an insoluble layer of ettringite (calcium sulfoaluminate) around the cement particles, which slows down the hydration process and provides sufficient time for placing the concrete.

Role of Calcium Chloride ($CaCl_2$):

Calcium Chloride acts as an Accelerator. It is the most common accelerator used to speed up the setting time and the rate of early strength gain. It increases the rate of hydration of Tricalcium Silicate (C_3S). It is particularly useful in:

- **Cold-weather concreting:** To prevent damage from early freezing.
- **Emergency repairs:** Where a quick set is required to restore service.
- **Pre-cast concrete:** To allow for faster reuse of moulds and forms.

3. **(b) (ii) Solution:**

The strength of concrete at any water-cement ratio is a function of the degree of hydration of cement and the physical and chemical properties of cement. When cement hydrates, it forms hydration products known as gel. The space available in the paste consists of gel and capillary pores. The gel-space ratio is defined as the ratio of the volume of hydrated cement paste (gel) to the total volume of gel and capillary pores.

$$\text{Gel-space ratio, } x = \frac{\text{Volume of gel}}{\text{Volume of gel + Volume of capillary pores}}$$

The relationship between compressive strength and gel-space ratio is given by

$$S = 240\, x^3$$

Where S is the strength of concrete in N/mm^2, x is the gel-space ratio, and 240 N/mm^2 is the intrinsic strength of the gel.

Given data

Weight of cement, $C = 500$ g

Water-cement ratio, $\frac{W}{C} = 0.50$

Weight of mixing water, $W = 500 \times 0.50 = 250$ g

Volume of mixing water, $W_0 = 250$ ml

Fraction of hydration, $\alpha = 1$ for full hydration

Fraction of hydration, $\alpha = 0.75$ for 75% hydration

For partial hydration, the gel-space ratio is given by

$$x = \frac{0.657\, C\, \alpha}{0.319\, C\, \alpha + W_0}$$

where, C = Weight of cement in gm

W_0 = Volume of mixing water in ml

α = Fraction of cement that has hydrated

At full hydration, $\alpha = 1$

$$\Rightarrow \quad x = \frac{0.657 \times 500 \times 1}{0.319 \times 500 \times 1 + 250}$$

$$\Rightarrow \quad x = 0.802$$

The theoretical strength is

$$S = 240 \times (0.802)^3$$

$$\Rightarrow \quad S = 123.804 \text{ N/mm}^2$$

At 75% hydration, $\alpha = 0.75$

$$\Rightarrow \quad x = \frac{0.657 \times 500 \times 0.75}{0.319 \times 500 \times 0.75 + 250}$$

$$\Rightarrow \quad x = 0.667$$

The theoretical strength is

$$S = 240 \times (0.667)^3$$

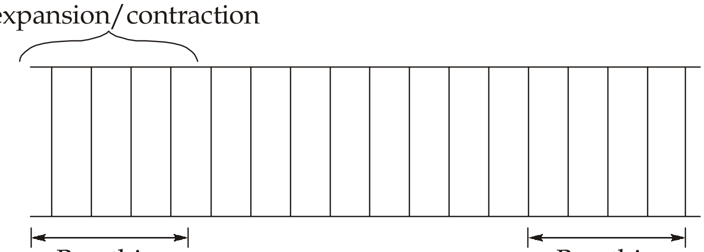
$$\Rightarrow \quad S = 71.22 \text{ N/mm}^2$$

Therefore, the theoretical strength of concrete is 123.804 N/mm^2 at full hydration and 71.22 N/mm^2 at 75% hydration.

3. (c) (i) Solution:

Breathing length of LWR:

The breathing length is the end portion of a Long-Welded Rail that undergoes longitudinal expansion or contraction due to temperature changes. In this region, the thermal forces are not fully neutralized by the longitudinal resistance provided by the sleepers and ballast. Consequently, the rail moves at the ends, while the central portion remains immobile because the cumulative resistance of the track equals or exceeds the thermal force.



3. (c) (ii) Solution:

Given:
$$\Delta T = 35°C$$
$$A = 76.5 \text{ cm}^2, E = 2.15 \times 10^6 \text{ kg/cm}^2$$
$$\alpha = 12 \times 10^{-6}/°C, \text{ spacing } (s) = 0.65 \text{ m}$$

Thermal force in the rail:
$$P = A\,E\,\alpha\,\Delta T$$
$$\Rightarrow \quad P = 76.5 \times (2.15 \times 10^6) \times (12 \times 10^{-6}) \times 35$$
$$\Rightarrow \quad P = 69079.5 \text{ kg}$$

Resistance per unit length:
$$R_p = \frac{r}{S} = \frac{350}{0.65} = 538.462 \text{ kg/m}$$

Breathing length:
$$L_b = \frac{P}{R_p} = \frac{69079.5}{538.462} = 128.291 \text{ m}$$

Minimum theoretical length of LWR:
$$L = 2 \times L_b = 2 \times 128.291 = 256.582 \text{ m}$$

3. (c) (iii) Solution:

Classification of rocks: Building stones are obtained from rocks which are classified in three ways:

1. Geological classification
2. Physical classification
3. Chemical classification

1. Geological classification: According to this classification, rocks are of the following three types:

(i) Igneous rocks

(ii) Sedimentary rocks

(iii) Metamorphic rocks

(i) Igneous rocks: Inside portion of the earth's surface has high temperature so as to cause *fusion* by heat at even ordinary pressures. Molten or pasty rocky material is known as *magma* and this magma occasionally tries to come out to the earth's surface through cracks or weak portions. Rocks which are formed by the cooling of magma are known as *igneous rocks*. e.g., ganite, gabbro etc.

(ii) Sedimentary rocks: These rocks are formed by the deposition of products of weathering on the pre-existing rocks. All the products of weathering are ultimately carried away from their place of origin by the agents of transport. Such agents are frost, rain, wind, flowing water etc. e.g., limestone, gypsum etc.

(iii) Metamorphic rocks: These rocks are formed by the change in character of the pre-existing rocks. Igneous as well as sedimentary rocks are changed in character when they are subject to great heat and pressure. This process of change is known as metamorphism. e.g., marble, schist etc.

2. Physical classification: This classification is based on general structure of rocks. According to this classification, rocks are of the following three types:

(i) Stratified rocks

(ii) Unstratified rocks

(iii) Foliated rocks

(i) Stratified rocks: These rocks possess planes of stratification or cleavage and such rocks can easily be split up along these planes. Sedimentary rocks are distinctly stratified rocks.

(ii) **Unstratified rocks:** These rocks are unstratified. The structure may be crystalline granular or compact granular. Igneous rocks of volcanic agency and sedimentary rocks affected by movements of the earth are of this type of rocks.

(iii) **Foliated rocks:** These rocks have a tendency to be split up in a definite direction only. Foliated structure is very common in case of metamorphic rocks.

3. **Chemical classification:** According to this classification, rocks are of the following three types:

(i) Silicious rocks

(ii) Argillaceous rocks

(iii) Calcareous rocks

(i) **Silicious rocks:** In these rocks, silica predominates. The rocks are hard and durable. They are not easily affected by the weathering agencies. Silica, however, in combination with weaker minerals, may disintegrate easily. It is therefore necessary that these rocks should contain maximum amount of free silica for making them hard and durable. Granites, quartzites, etc. are examples of silicious rocks.

(ii) **Argillaceous rocks:** In these rocks, clay or argil predominates. Such rocks may be dense and compact or they may be soft. These stones are hard and durable but brittle. Slates, laterites, etc. are examples of silicious rocks.

(iii) **Calcareous rocks:** In these rocks, calcium carbonate predominates. The durability of these rocks will depend upon the constituents present in surrounding atmosphere. Limestones, marbles, etc. are examples of calcareous rocks.

4. (a) (i) Solution:

Micro-irrigation, often referred to as localized or low-flow irrigation, involves the slow and precise application of water directly to the plant's root zone through a network of pipes and emitters. Unlike surface irrigation, which floods the entire soil surface, microirrigation targets specific areas to minimize waste.

Micro-Irrigation Methods

1. **Drip (Trickle) Irrigation:** This is the most common form of micro-irrigation. Water is delivered at a very low rate (2– 20 liters per hour) through small plastic pipes fitted with outlets called emitters or drippers.

2. **Micro-Sprinkler Irrigation:** This method uses small-diameter sprinklers (micro-jets) that operate at low pressure. They throw water in a circular or semi-circular pattern over a small area, making them ideal for orchards where a larger wetting pattern than a single dripper is required.

Advantages Over Surface Irrigation

- **High Water Use Efficiency:** Efficiency can exceed 90%, compared to 40-60% in surface irrigation, as it significantly reduces evaporation and deep percolation.
- **Reduced Weed Growth:** Since water is applied only to the root zone, the areas between rows remain dry, discouraging weed germination.
- **Fertigation:** Fertilizers and chemicals can be injected directly into the water stream, ensuring they reach the roots directly and reducing chemical runoff.
- **Adaptability:** It can be used on undulating or sloping terrains where surface irrigation would cause heavy erosion or require expensive land levelling.
- **Improved Crop Yield and Quality:** Maintaining optimum soil moisture levels prevents plant stress, leading to better growth and uniform harvests.

Disadvantages Over Surface Irrigation

- **High Initial Cost:** The investment for pipes, pumps, filters, and emitters is significantly higher than the cost of digging simple furrows or channels.
- **Clogging of Emitters:** Because the openings in drippers are very small, they are highly susceptible to blockage from silt, algae, or mineral deposits. This requires high-quality filtration systems.
- **Salt Accumulation:** In arid regions, salts tend to accumulate at the edges of the "wetting front." If not managed with occasional leaching, this can harm the plants.
- **Technical Skill Requirement:** It requires a higher level of technical knowledge for design, installation, and regular maintenance compared to traditional flooding.
- **Energy Consumption:** While surface irrigation often relies on gravity, microirrigation typically requires a pump to maintain the necessary operating pressure.

4. (a) (ii) Solution:

Given: Consumptive use coefficient, $k = 0.85$

Water application efficiency, $\eta_a = 70\% = 0.7$

Blaney-Criddle formula:

$$C_u = k \times \Sigma(f)$$

$$f = \frac{P}{40} \times (1.8t + 32)$$

Month	Mean Monthly Temperature 't' (°C)	Monthly percentage of sunshine hours 'P'	Effective rainfall 'R_e' (cm)	f(cm)
November	20	7.50	2.50	12.75
December	16	7.20	3.00	10.944
January	14	7.40	3.20	10.582
February	15	7.20	2.10	10.62
			Σ 10.80	**44.896 cm**

Seasonal consumptive use is calculated using the Blaney-Criddle equation.

$$C_u = k \times \Sigma(f)$$

$$\Rightarrow \quad C_u = 0.85 \times 44.896$$

$$\Rightarrow \quad C_u = 38.162 \text{ cm}$$

Consumptive Irrigation Requirement is obtained by subtracting effective rainfall from seasonal consumptive use.

$$CIR = C_u - \Sigma R_e$$

$$\Rightarrow \quad CIR = 38.162 - 10.8$$

$$\Rightarrow \quad CIR = 27.362 \text{ cm}$$

Field Irrigation Requirement accounts for application efficiency.

$$FIR = \frac{CIR}{\eta_a} = \frac{27.362}{0.7}$$

$$\Rightarrow \quad FIR = 39.089 \text{ cm}$$

4. (b) Solution:

Given data

Area for Rabi season, $A_r = 4500$ ha

Kor depth for Rabi season, $\Delta_r = 13 \text{ cm} = 0.13 \text{ m}$

Kor period for Rabi season, $B_r = 4 \text{ weeks} = 28 \text{ days}$

Area for Kharif season, $A_k = 1800$ ha

Kor depth for Kharif season, $\Delta_k = 19$ cm $= 0.19$ m

Kor period for Kharif season, $B_k = 3$ weeks $= 21$ days

Silt factor, $f = 0.9$

Side slope, $0.5\,H : 1V$

First, the duty for each season is calculated using the relation

$$D = \frac{8.64B}{\Delta}$$

For Rabi season, $$D_r = \frac{8.64 \times 28}{0.13}$$

$\Rightarrow$ $$D_r = 1860.923 \text{ ha/cumec}$$

The discharge required for Rabi season is

$$Q_r = \frac{A_r}{D_r} = \frac{4500}{1860.923}$$

$\Rightarrow$ $$Q_r = 2.418 \text{ m}^3\text{/s}$$

For Kharif season,

$$D_k = \frac{8.64 \times 21}{0.19}$$

$\Rightarrow$ $$D_k = 954.947 \text{ ha/cumec}$$

The discharge required for Kharif season is

$$Q_k = \frac{A_k}{D_k} = \frac{1800}{954.947}$$

$\Rightarrow$ $$Q_k = 1.885 \text{ m}^3\text{/s}$$

The design discharge is taken as the maximum of the two seasonal discharges.

$$Q = 2.418 \text{ m}^3\text{/s}$$

Now, using Lacey's theory, the velocity is calculated from

$$V = \left(\frac{Qf^2}{140}\right)^{1/6} = \left(\frac{2.418 \times 0.9^2}{140}\right)^{1/6}$$

$\Rightarrow$ $$V = 0.491 \text{ m/s}$$

The hydraulic mean radius is given by

$$R = \frac{5}{2}\left(\frac{V^2}{f}\right) = \frac{5}{2}\left(\frac{0.491^2}{0.9}\right)$$

$\Rightarrow$ $R = 0.670$ m

The area of flow is

$$A = \frac{Q}{V} = \frac{2.418}{0.491}$$

$\Rightarrow$ $A = 4.925 \text{ m}^2$

The wetted perimeter is obtained from Lacey's equation

$$P = 4.75\sqrt{Q}$$

$\Rightarrow$ $P = 4.75\sqrt{2.418}$

$\Rightarrow$ $P = 7.386$ m

Assuming a trapezoidal section with side slope $0.5H : 1V$, the wetted perimeter is

$$P = B + 2y\sqrt{1+m^2}$$

$$7.386 = B + 2y\sqrt{1+0.5^2}$$

$\Rightarrow$ $B + 2.236y = 7.386$ ...(i)

The area of flow is

$$A = (B + my)y = By + 0.5y^2$$

$$4.925 = By + 0.5y^2 \quad \text{...(ii)}$$

On putting the value of B from equation (i) in equation (ii) we get,

$$4.925 = (7.386 - 2.236y)y + 0.5y^2$$

$\Rightarrow$ $4.925 = 7.386y - 1.736y^2$

$\Rightarrow$ $1.736y^2 - 7.386y + 4.925 = 0$

Solving for depth, $y = 3.427$ m and 0.828 m

For $y = 3.427$ m

From equation (i), $B = 7.386 - 2.236 \times 3.427$

$B = -0.276$ m (not ok)

For $y = 0.828$ m

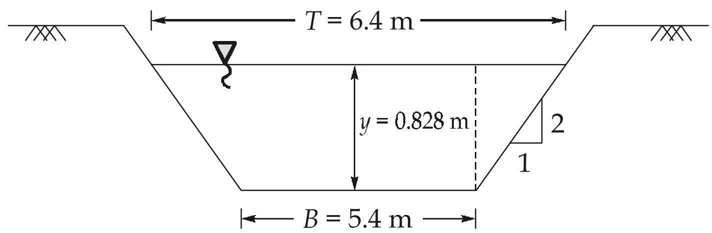
From equation (i), $B = 7.386 - 2.236 \times 0.828$

$B = 5.535 \text{ m} \simeq 5.54 \text{ m}$ (OK)

The bed slope is

$$S = \frac{f^{5/3}}{3340\,Q^{1/6}} = \frac{0.9^{5/3}}{3340 \times 2.418^{1/6}} = 2.168 \times 10^{-4}$$

$$S = \frac{1}{4612.308}$$



(Cross-section of canal)

4. (c) (i) Solution:

The hydration of Ordinary Portland Cement is a complex chemical process where the anhydrous compounds in cement react with water to form a hardened paste. During the fusion process in the kiln, four primary mineralogical compounds are formed, collectively known as Bogue's Compounds. These compounds are Tricalcium Silicate, Dicalcium Silicate, Tricalcium Aluminate, and Tetracalcium Aluminoferrite. Each plays a distinct role in defining the engineering properties of the resulting concrete.

1. **Tricalcium Silicate ($3\,CaO{\cdot}SiO_2$):** Often referred to as Alite, constitutes approximately 40% to 60% of the total volume of OPC. It is considered the most important constituent for structural engineering purposes.
   - **Influence on Strength:** hydrates rapidly and is primarily responsible for the "early strength" of concrete, specifically within the first 7 days. It allows for the early removal of formwork in construction projects.
   - **Heat Generation:** It produces a significant amount of heat during hydration (approximately 500 J/g).
   - **Setting Time:** It contributes to the initial hardening and setting of the cement paste. A higher percentage of results in a cement that is suitable for cold-weather concreting due to its rapid heat evolution.
2. **Dicalcium Silicate ($2\,CaO{\cdot}SiO_2$):** Or Belite, makes up about 15% to 35% of the cement. Unlike, it reacts very slowly with water.
   - **Influence on Strength:** It contributes very little to the strength during the first week. However, it is the primary compound responsible for the "ultimate strength" or long-term strength gain of concrete beyond 28 days.

- **Heat Generation:** It produces the least amount of heat among the silicate and aluminate compounds (approximately 260 J/g).
- **Setting Time:** It has negligible impact on the initial setting time but ensures the continued durability and chemical resistance of the hardened mass over years.

3. **Tricalcium Aluminate ($3\ CaO{\cdot}Al_2O_3$):** constitutes about 5% to 15% of OPC. It is the most reactive compound when it comes into contact with water.
   - **Influence on Strength:** It contributes only a small amount to the strength in the first 24 hours and has no significant contribution to the ultimate strength.
   - **Heat Generation:** It generates the highest amount of heat of hydration (approximately 865 J/g). Excessive can lead to volume instability and cracks.
   - **Setting Time:** It is responsible for the "flash set" or immediate stiffening of cement. To prevent this, gypsum is added during the grinding of clinker to retard the reaction of. It also makes concrete vulnerable to sulphate attacks.
4. **Tetracalcium Aluminoferrite ($4\ CaO{\cdot}Al_2O_3{\cdot}Fe_2O_3$):** known as Celite, makes up about 10% to 15% of the cement.
   - **Influence on Strength:** It is a poor cementing material and contributes very little to the overall strength of the concrete.
   - **Heat Generation:** It produces a moderate amount of heat (approximately 420 J/g).
   - **Setting Time:** It reacts relatively quickly, though slower than . Its primary role is acting as a flux during the manufacturing process, lowering the temperature required for clinker formation. It is also responsible for the characteristic grey color of cement.

4. (c) (ii) Solution:

The height of capillary rise is given by

$$H_C = \frac{4\sigma\cos\theta}{\gamma_w D} = \frac{4\times 0.054\times\cos 0^\circ}{9810\times 0.08\times 10^{-3}}$$

$$H_C = 0.275 \text{ m}$$

The top of the capillary zone below ground level is

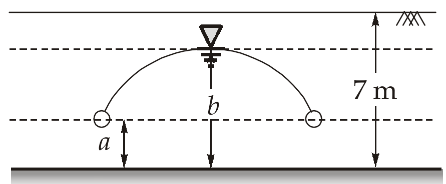
$$= 2 - 0.275 = 1.725 \text{ m}$$

Since the root zone extends up to 1.8 m below ground level and the capillary water rises up to 1.725 m below ground level, the capillary water enters the root zone. Therefore, the field is slight waterlogged.

Discharge per unit length of drain is, $q = 0.116 \dfrac{\text{cumecs}}{\text{Km}^2}$ Area in km^2

$$\Rightarrow \quad q = 0.116\left(\frac{15}{1000}\times\frac{1}{1000}\right)$$

$$\Rightarrow \quad q = 1.74 \times 10^{-6}\ \text{m}^3/\text{s per m length}$$



To prevent interference of capillary fringe with the root zone, the maximum height of the water table above the impervious stratum is

$$b = 7 - 1.725$$

$$\Rightarrow \quad b = 5.275\ \text{m}$$

Using the discharge equation,

$$q = \frac{4k}{S}\left(b^2 - a^2\right)$$

$$\Rightarrow \quad 1.74 \times 10^{-6} = \frac{4\times1\times10^{-6}}{15}(5.275^2 - a^2)$$

$$\Rightarrow \quad 5.275^2 - a^2 = 6.525$$

$$a = 4.62\ \text{m}$$

Depth of tile drains below ground surface is

$$\text{Depth} = 7 - 4.62$$

$$\text{Depth} = 2.38\ \text{m}$$

The closed drains should be laid at a depth of 2.38 m below the ground level.

Section B : Design of Steel Structure-1+ Hydrology-1, Structural Analysis-2+CPM PERT-2

5. (a) Solution:

Minimum weld size for t_g = 10 mm: S_{min} = 3 mm

Maximum weld size for web thickness: $S_{max} = t_w - 1.5 = 6.7 - 1.5 = 5.2$ mm

Select weld size: $S = 5$ mm

Effective throat thickness of fillet weld: $t_t = 0.7 \times S = 0.7 \times 5 = 3.5$ mm

Strength of weld per mm length:

$$f_w = \frac{t_t f_u}{\sqrt{3}\gamma_{mw}} = \frac{3.5 \times 410}{\sqrt{3} \times 1.5} = 552.332 \text{ N/mm}$$

Length of weld required,

$$(l_w)_{req} = \frac{P}{f_w} = \frac{800 \times 10^3}{552.333} = 1448.404 \text{ mm}$$

Weld arrangement

Available weld length with 300 mm overlap (top, bottom, and end):

$$L_{available} = 2 \times 300 + 300 = 900 \text{ mm}$$

Since $L_{available} < (l_w)_{req}$ longitudinal and transverse welds alone are insufficient. Supplementary slot welds are required.

Design of slot welds

Width of slot:

$$w = \max(3 \times t_w, 25) = \max(20.1, 25) = 25 \text{ mm}$$

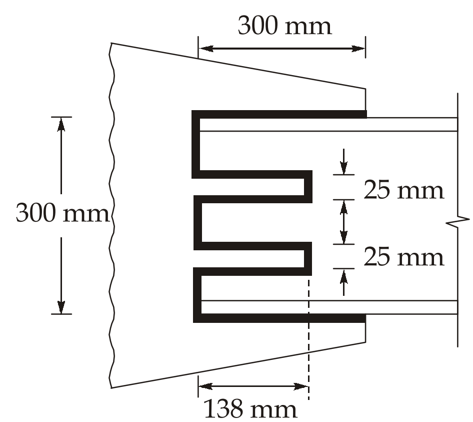
Let two slots of length l_1 each be provided. Total weld length including slots:

$$l_w = 900 + 4l_1$$

$$\Rightarrow \quad 1448.404 = 900 + 4l_1$$

$$\Rightarrow \quad l_1 = 137.101 \text{ mm} \approx 138 \text{ mm}$$

Provide a 5 mm fillet weld with an overlap of 300 mm. To supplement the strength, provide two slot welds of size 138 mm × 25 mm each.



5. (b) Solution:

Rivet diameter, $\phi = 20$ mm

Gross diameter of rivet hole, $d = 20 + 1.5 = 21.5$ mm

Deduction for holes in the connected leg (125 mm leg)

Along a straight section with one hole, Deduction = d = 21.5 mm

Along a staggered section with two holes, Deduction = $2d - \frac{s^2}{4g}$

$$= 2 \times 21.5 - \frac{50^2}{4 \times 60}$$

$$= 43 - 10.417 = 32.583 \text{ mm}$$

Maximum deduction = 32.583 mm

Net area of connected leg, A_1

$$A_1 = \left(125 - 32.583 - \frac{t}{2}\right)t$$

$\Rightarrow$ $A_1 = (125 - 32.583 - 5) \times 10$

$\Rightarrow$ $A_1 = 874.17 \text{ mm}^2$

Area of outstanding leg, A_2

$$A_2 = \left(75 - \frac{t}{2}\right)t$$

$\Rightarrow$ $A_2 = (75 - 5) \times 10$

$\Rightarrow$ $A_2 = 700 \text{ mm}^2$

Reduction factor for single angle connected by one leg,

$$k = \frac{3A_1}{3A_1 + A_2} = \frac{3 \times 874.17}{3 \times 874.17 + 700}$$

$\Rightarrow$ $k = 0.789$

Effective net area,

$$A_{net} = A_1 + kA_2$$

$\Rightarrow$ $A_{net} = 874.17 + 0.789 \times 700$

$\Rightarrow$ $A_{net} = 1426.47 \text{ mm}^2$

Allowable load

$$P_{allow} = \sigma_{at} \times A_{net}$$

$\Rightarrow$ $P_{allow} = 150 \times 1426.47$

$\Rightarrow$ $P_{allow} = 213970.5 \text{ N}$

$\Rightarrow$ $P_{allow} = 213.970 \text{ kN}$

5. (c) Solution:

Given data

Total volume of excavation, $V = 350000 \text{ m}^3$

Ideal output of shovel, $Q_i = 140 \text{ m}^3/\text{hr}$

Depth-swing correction factor, $F_d = 0.88$

Job-management factor, $F_j = 0.82$

Working hours per week, $H_w = 45 \text{ hr/week}$

Working weeks per year, $W_y = 48 \text{ weeks/year}$

Operating efficiency, $\eta = \frac{50}{60}$

Target completion time, $T = 1200 \text{ hr}$

Actual capacity of power shovel is calculated by considering operating efficiency and correction factors.

$$\text{Actual Capacity} = Q_i \times \eta \times F_d \times F_j$$

$\Rightarrow$ $\text{Actual Capacity} = 140 \times \frac{50}{60} \times 0.88 \times 0.82$

$\Rightarrow$ $\text{Actual Capacity} = 84.187 \text{ m}^3/\text{hr}$

Time required in years to complete the project is

$$\text{Total time (years)} = \frac{V}{\text{Actual Capacity} \times H_w \times W_y}$$

$$\Rightarrow \quad \text{Total time (years)} = \frac{350000}{84.187 \times 45 \times 48}$$

$$\Rightarrow \quad \text{Total time (years)} = 1.925 \text{ years}$$

For completion in 1200 hours, required total capacity is

$$\text{Total required capacity} = \frac{V}{T} = \frac{350000}{1200}$$

$$\text{Total required capacity} = 291.667 \text{ m}^3/\text{hr}$$

Number of shovels required is

$$\text{Number of shovels} = \frac{\text{Total required capacity}}{\text{Actual Capacity of one shovel}}$$

$$\Rightarrow \quad \text{Number of shovels} = \frac{291.667}{84.187}$$

$$\Rightarrow \quad \text{Number of shovels} = 3.465 \approx 4 \text{ power shovels}$$

5. (d) Solution:

Given data

Capacity of tank, V = 20000 liters = 20 m^3

Natural period when empty, T_1 = 1.2 sec

Natural period when full, T_2 = 2.5 sec

Density of water, ρ = 1000 kg/m^3

Mass of water, $m_w = \rho \times V = 1000 \times 20 = 20000$ kg

The natural period of a single degree of freedom (SDOF) system is given by:

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

For the empty tank: $T_1 = 2\pi\sqrt{\frac{m}{k}}$

Where, m = mass of empty tank.

$$\Rightarrow \quad 1.2 = 2\pi\sqrt{\frac{m}{k}} \qquad \text{...(i)}$$

For the full tank: $T_2 = 2\pi\sqrt{\frac{m+m_w}{k}}$

$$\Rightarrow \quad 2.5 = 2\pi\sqrt{\frac{m+20000}{k}} \qquad \text{...(ii)}$$

Taking the ratio of the two periods:

$$\frac{2.5}{1.2} = \sqrt{\frac{m+20000}{m}}$$

$\Rightarrow$ $$m = 5987.526 \text{ kg}$$

Substituting the value of m in equation (i), we get

$$1.2 = 2\pi\sqrt{\frac{5987.526}{k}}$$

$\Rightarrow$ $$k = 164151.425 \text{ N/m}$$

5. (e) Solution:

Fixed End Moments (FEMs)

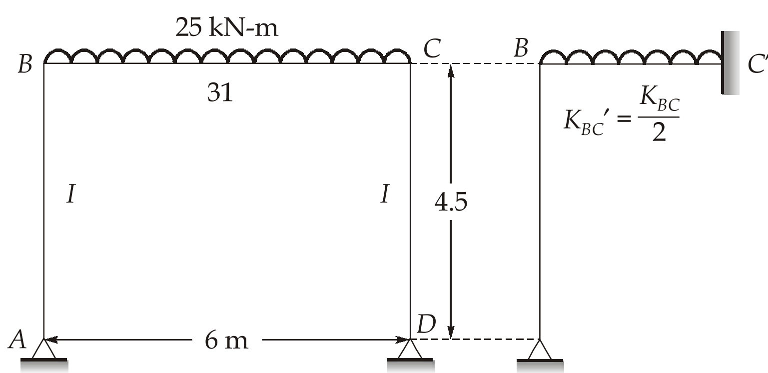
For span *BC*:

$$M_{FBC} = -\frac{25\times 6^2}{12} = -75 \text{ kN-m}$$

$$M_{FCB} = \frac{25\times 6^2}{12} = 75 \text{ kN-m}$$

For columns AB and CD:

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



Stiffness and Distribution Factors (*DF*)

Using the symmetry modification for the beam and the hinged far-end modification for the columns:

Stiffness of *BA*, $$K_{BA} = \frac{3\times E\times I}{4.5} = \frac{2EI}{3}$$

Stiffness of *BC*, $$K_{BC}' = \frac{4\times E\times 3I}{6\times 2} = EI$$

Total stiffness at joint B, $\Sigma K = \dfrac{2EI}{3} + EI = \dfrac{5EI}{3}$

Distribution factors: $DF_{BA} = \dfrac{0.667EI}{1.667EI} = 0.4$

$$DF_{BC}' = \frac{1EI}{1.667EI} = 0.6$$

Moment Distribution at Joint B

Joint B	BA	BC′
DF	0.4	0.6
FEM (kN-m)	0	–75
Balance (kN-m)	30	45
Final Moment (kN-m)	30	–30

Final End Moments

By symmetry:

$$M_{AB} = 0 \text{ kN-m}$$
$$M_{BA} = 30 \text{ kN-m}$$
$$M_{BC} = -30 \text{ kN-m}$$
$$M_{CB} = 30 \text{ kN-m}$$
$$M_{CD} = -30 \text{ kN-m}$$
$$M_{DC} = 0 \text{ kN-m}$$

Maximum Span Moment in BC

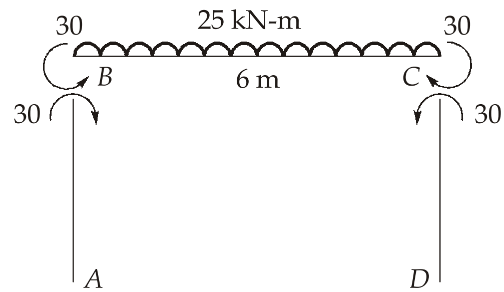
Free bending moment at mid-span:

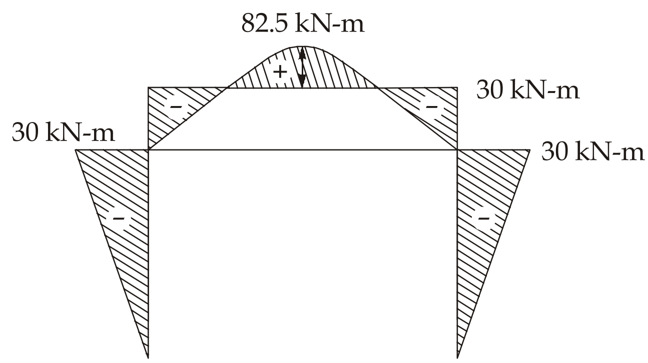
$$\text{Free } BM = \frac{w \times L_{BC}^2}{8} = \frac{25 \times 6^2}{8} = 112.5 \text{ kN-m}$$

Maximum span moment:

$$M_{\text{max span}} = 112.5 - 30 = 82.5 \text{ kN-m}$$

Thus, the maximum span moment in BC is 82.5 KN-m.





(BMD)

6. (a) (i) Solution:

$$\Sigma P = 112 + 178 + 95 + 139 + 101 = 625 \text{ cm}$$

$$\therefore \quad \bar{P} = \frac{625}{5} = 125 \text{ cm}$$

Optimum number of stations can be calculated as

$$N = \left(\frac{C_v}{\in}\right)^2$$

C_v = Coefficient of variation of rainfall

$\in$ = Allowable degree of error in%

$$C_v = \frac{\sigma_{m-1} \times 100}{\bar{P}}$$

$$\Rightarrow \quad \sigma_{m-1} = \sqrt{\sum_{i=1}^{m} \frac{\left(P_i - \bar{P}\right)^2}{m-1}}$$

$$\sigma_{m-1} = \sqrt{\frac{(112-125)^2 + (178-125)^2 + (95-125)^2 + (139-125)^2 + (101-125)^2}{5-1}}$$

$$= 34.095 \text{ cm}$$

$$C_v = \frac{34.095 \times 100}{125} = 27.276\%$$

$$N = \left(\frac{27.276}{8}\right)^2 = 11.625 \simeq 12$$

∴ In watershed 10% of the raingauge stations must be recording type.

∴ So, number of recording raingauges = 2

Number of non-recording raingauges = 10

6. (a) (ii) Solution:

Given data

Surface area, $A = 12 \text{ km}^2 = 12 \times 10^6 \text{ m}^2$

Relative humidity, $RH = 0.4$

Wind velocity at 3 m height, $u_3 = 14.4$ km/h

Saturation vapour pressure, $e_s = 35.66$ mm Hg

Meyer•fs coefficient, $k_m = 0.36$

Actual vapour pressure,

$$e_a = e_s \times RH = 35.66 \times 0.4 = 14.264 \text{ mm Hg}$$

Vapour pressure difference,

$$e_s - e_a = 35.66 - 14.264 = 21.396 \text{ mm Hg}$$

Wind speed at 9 m height using $\frac{u_9}{u_3} = \left(\frac{9}{3}\right)^{1/7}$

$$u_9 = 14.4 \times 3^{1/7} = 14.4 \times 1.1699 = 16.847 \text{ km/h}$$

Meyer's formula,

$$E = k_m (e_s - e_a)\left(1 + \frac{u_9}{16}\right)$$

$$\Rightarrow \quad E = 0.36 \times 21.396 \left(1 + \frac{16.847}{16}\right)$$

$$\Rightarrow \quad E = 0.36 \times 21.396 \times 2.053 = 15.813 \text{ mm/day}$$

Evaporation depth in meters,

$$E = 0.015813 \text{ m/day}$$

Daily evaporation loss,

$$\text{Volume} = A \times E = 12 \times 10^6 \times 0.015813 \text{ m}^3/\text{day}$$

$$= 189756 \text{ m}^3/\text{day}$$

$$\text{Daily evaporation loss} = 189756 \text{ m}^3/\text{day}.$$

6. (b) Solution:

Given data

Factored axial load: $P = 1200$ kN

Column length: $L = 10$ m = 10,000 mm

Effective length for pinned ends: l = 10,000 mm

assume bolt diameter: d = 20 mm, Grade 4.6, f_{ub} = 400 MPa

Step 1: Column Capacity and Slenderness Ratio

Total area provided: $A_{\text{total}} = 4 \times 2106 = 8424 \text{ mm}^2$

Design compressive stress required:

$$f_{cd} = \frac{P}{A_{total}} = \frac{1200 \times 10^3}{8424} = 142.45 \text{ N/mm}^2$$

Interpolating for buckling class C

between f_{cd} = 152 MPa(λ = 70) and f_{cd} = 136 MPa(λ = 80):

$$\lambda = 70 + \frac{10}{152 - 136} \times (152 - 142.45) = 75.97$$

Effective slenderness ratio for laced column:

$$\lambda_{\text{eff}} = 1.05 \times 75.97 = 79.77$$

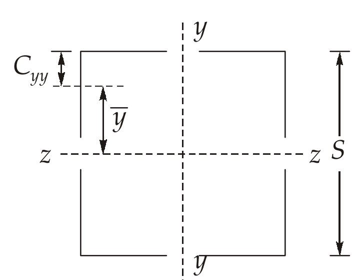
Step 2: Spacing of Angles

Required radius of gyration:

$$r = \frac{l}{\lambda_{eff}} = \frac{10000}{79.77} = 125.37 \text{ mm}$$

Required moment of inertia:

$$I_{\text{required}} = A_{\text{total}} \cdot r^2 = 8424 \times 125.37^2 = 132.39 \times 10^6 \text{ mm}^4$$



Equating to provided moment of inertia:

$$I_{\text{required}} = 4 \times [I_{\text{angle}} + A_{\text{angle}}\, \overline{y}^2]$$

$$132.39 \times 10^6 = 4 \times [2.38 \times 10^6 + 2106\ \bar{y}^2]$$

$$\bar{y} = 120.77 \text{ mm}$$

Spacing of angles:

$$S = 2 \times \left(\bar{y} + C_{yy}\right) = 2 \times (120.68 + 30.8) = 303.14 \text{ mm} \approx 310 \text{ mm}$$

Step 3: Forces in Lacing Bars

Let us provide the single lacing system with lacing flats inclined at angle of 45 degrees with the vertical.

Transverse shear in column:

$$V = 0.025 \times 1200 \times 10^3 = 30{,}000 \text{ N}$$

Assuming a single lacing system on two faces:

$$V_{\text{per face}} = \frac{30000}{2} = 15{,}000 \text{ N}$$

Compressive/tensile force in each lacing bar:

$$F = \frac{V_{\text{per face}}}{\sin\theta} = \frac{15000}{\sin 45°} = 21213 \text{ N}$$

Step 4: Design of Lacing Flat

Assume 20 mm dia bolt, assuming 50 mm gauge distance

Minimum width:

$$b_{\min} = 3 \times d = 3 \times 20 = 60 \text{ mm}$$

Length of lacing flat:

$$L_1 = (S - \text{gauge} \times 2) \text{ cosec } 45° = (310 - (50 \times 2)) \text{ cosec } 45° = 296.99 \text{ mm}$$

Minimum thickness: $$t_{\min} = \frac{L_1}{40} = \frac{296.99}{40} = 7.42 \text{ mm}$$

Provide a lacing flat: 60 × 8 mm

Radius of gyration of flat: $$r = \frac{t}{\sqrt{12}} = \frac{8}{\sqrt{12}} = 2.31 \text{ mm}$$

Slenderness ratio: $$\lambda_{\text{flat}} = \frac{L_1}{r} = \frac{296.99}{2.31} = 128.567 < 145 \text{ (safe)}$$

Step 5: Check Compressive and Tensile Capacity of Lacing

Compressive capacity: For $\lambda = 128.62$ (Class C),

interpolate f_{cd} between 121 MPa ($\lambda = 90$) and 107 MPa ($\lambda = 100$):

$$f_{cd} = 121 - \frac{121-107}{100-90}(128.62 - 90) = 66.93 \text{ MPa}$$

Capacity of lacing flat:

$$P_c = f_{cd}\, b\, t = 66.93 \times 60 \times 8 = 32{,}127 \text{ N} > 21{,}213 \text{ N (Safe)}$$

Tensile capacity:

$$A_{net} = (b - d_0)\, t = (60 - 22) \times 8 = 304 \text{ mm}^2$$

$$T_{dn} = \frac{0.9 f_u A_{net}}{\gamma_{m1}} = \frac{0.9 \times 410 \times 304}{1.25} = 89{,}741 \text{ N} > 21{,}213 \text{ N (Safe)}$$

Step 6: Connection Design

Shear capacity of M20 bolt (single shear):

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \times \gamma_{mb}} \times A_{nb} = \frac{400}{\sqrt{3} \times 1.25} \times 245.044 = 45{,}272 \text{ N}$$

Bearing capacity of flat: (Assuming $k_b = 0.5$)

$$V_{dpb} = \frac{2.5\, k_b\, d\, t\, f_u}{\gamma_{mb}} = \frac{2.5 \times 0.5 \times 20 \times 8 \times 41}{1.25} = 65{,}600 \text{ N}$$

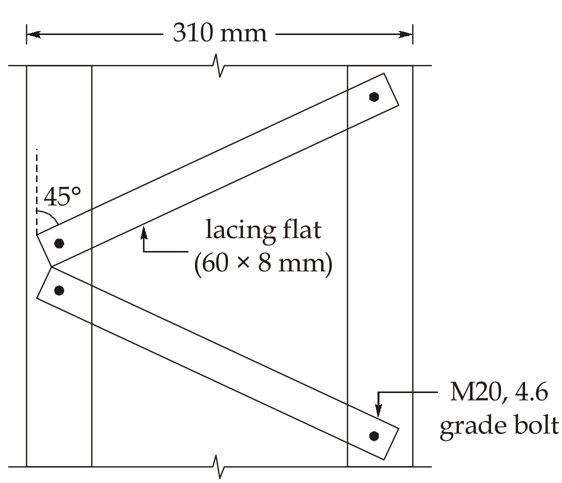
Bolt value:

$$V_{bolt} = \text{Min}(V_{dsb}, V_{dpb}) = 45{,}272 \text{ N}$$

Number of bolts required:

$$n_{bolt} = \frac{F}{V_{bolt}} = \frac{21213}{45272} \approx 0.469$$

Provide one M20 bolt at each end of the lacing bar.



6. (c) Solution:

Given

Area of each member, $A = 1500 \text{ mm}^2$

Modulus of elasticity, $E = 2 \times 10^5 \text{ N/mm}^2$

Length of each member, $L = 4 \text{ m} = 4000 \text{ mm}$

Loads: $W_B = 20 \text{ kN}, W_C = 40 \text{ kN}$

All members form equilateral triangles, hence

$$\sin 60° = \frac{\sqrt{3}}{2}, \cos 60° = \frac{1}{2}$$

Support Reactions

Taking moments about A:

$$\Sigma M_A = 0$$

$$\Rightarrow \quad V_D \times 8 - 20 \times 2 - 40 \times 6 = 0$$

$$\Rightarrow \quad 8V_D = 40 + 240 = 280$$

$$\Rightarrow \quad V_D = 35 \text{ kN} (\uparrow)$$

Vertical equilibrium: $\Sigma F_y = 0$

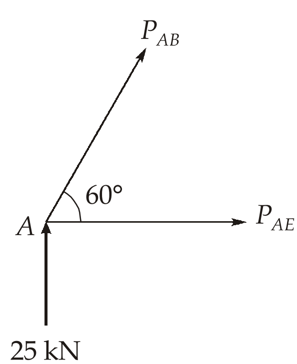
$$\Rightarrow \quad V_A + V_D = 20 + 40$$

$$\Rightarrow \quad V_A = 25 \text{ kN} (\uparrow)$$

Member Forces due to Actual Loads (P - forces system)

Using method of joints.

Joint *A*:



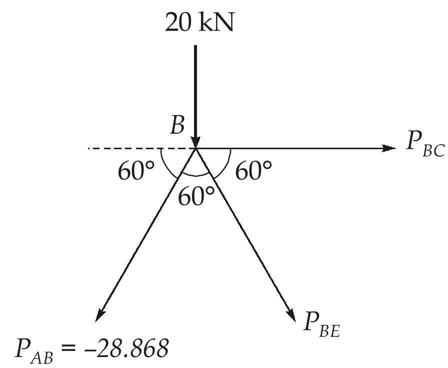
$$\Sigma F_y = 0 \quad \Rightarrow \quad P_{AB} \sin 60° = -25$$

$$P_{AB} = -28.868 \text{ kN}$$

$$\Sigma F_x = 0 \quad \Rightarrow \quad P_{AE} = -P_{AB} \cos 60°$$

$$P_{AE} = 14.434 \text{ kN}$$

Joint *B*:



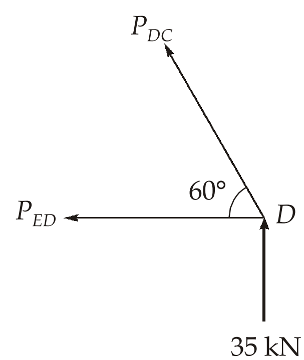
$\Sigma F_y = 0 \quad \Rightarrow \quad -P_{BE}\sin 60° - 20 - (-28.868\sin 60°) = 0$

$$P_{BE} = 5.774 \text{ kN}$$

$\Sigma F_x = 0 \quad \Rightarrow \quad P_{BC} = 5.774 \cos 60° + (-28.868) \cos 60°$

$$P_{BC} = -17.321 \text{ kN}$$

Joint *D*:



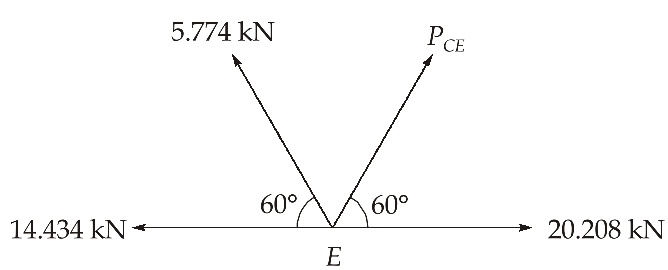
$\Sigma F_y = 0 \quad \Rightarrow \quad P_{DC}\sin 60° = -35$

$$P_{DC} = -40.415 \text{ kN}$$

$\Sigma F_x = 0 \quad \Rightarrow \quad P_{ED} = -P_{DC}\cos 60°$

$$P_{ED} = 20.208 \text{ kN}$$

Joint E:



$\Sigma F_y = 0 \quad \Rightarrow \quad P_{CE}\sin 60° + P_{BE}\sin 60° = 0$

$$P_{CE} = -5.774 \text{ kN}$$

Member Forces due to Unit Load at *E* (*k* – forces system)

Apply a vertical unit load of 1 kN downward at joint *E*.

By symmetry:

$$V_A = V_D = 0.5 \text{ kN}$$

Member forces:

$$k_{AB} = k_{DC} = -\frac{1}{\sqrt{3}} = -0.577 \text{ kN}$$

$$k_{AE} = k_{ED} = \frac{1}{2\sqrt{3}} = 0.289 \text{ kN}$$

$$k_{BE} = k_{CE} = \frac{1}{\sqrt{3}} = 0.577 \text{ kN}$$

$$k_{BC} = -\frac{1}{\sqrt{3}} = -0.577 \text{ kN}$$

Deflection at Joint *E*

Using unit load method: $\delta_E = \sum\left(\frac{PkL}{AE}\right)$

Table of calculations:

Member	*L* (mm)	*P* (kN)	*k*	*P k L* kN-mm
AB	4000	–28.868	–0.577	66627.34
BC	4000	–17.321	–0.577	39976.87
CD	4000	–40.415	–0.577	93277.82
DE	4000	20.208	0.289	23360.45
EA	4000	14.434	0.289	16685.70
BE	4000	5.774	0.577	13326.39
CE	4000	-5.774	0.577	-13326.39

$\Sigma(PkL) = 239928.18$ kN-m

Therefore,

$$\delta_E = \frac{239928.18 \times 10^3}{1500 \times 2 \times 10^5}$$

$$\Rightarrow \quad \delta_E = 0.799 \text{ mm}$$

$\Rightarrow \quad \delta_E = 0.8$ mm downward

7. (a) (i) Solution:

Given data

Catchment area, $A = 5\text{ km}^2$

Slope, $S = 1/400 = 0.0025$

Length of travel, $L = 3\text{ km} = 3000\text{ m}$

Runoff coefficient, $C = 0.35$

Return period, $T = 50$ years

Time of concentration (t_c) using Kirpich Formula

$$t_c = 0.01947 \times L^{0.77} \times S^{-0.385}$$

$$\Rightarrow \quad t_c = 0.01947 \times 3000^{0.77} \times 0.0025^{-0.385}$$

$$\Rightarrow \quad t_c = 93.013\text{ min} = 1.55\text{ hr}$$

Average rainfall intensity (i)

Using linear interpolation from the provided table for $t_c = 93.013$ min:

$$d = 13.3 + \frac{13.7 - 13.3}{100 - 90} \times (93.013 - 90)$$

$$\Rightarrow \quad d = 13.421\text{ cm}$$

Intensity, $$i = \frac{d}{t_c}$$

$$i = \frac{13.421}{1.55}$$

$$i = 8.658\text{ cm/hr} = 86.58\text{ mm/hr}$$

Peak flow rate (Q_p) using Rational Formula

$$Q_p = \frac{1}{3.6} C\, i\, A$$

Where, Q_p = Peak discharge (in m^3/sec)

i = Intensity (in mm/hr)

A = Area (in km^2)

$$\Rightarrow \quad Q_p = 0.35 \times 86.58 \times 5/3.6$$

$$\Rightarrow \quad Q_p = 42.087\text{ m}^3/\text{s}$$

7. (a) (ii) Solution:

Given:

Duration of storm, $T = 2$ hr

Total rainfall depth, $P = 47$ mm

Peak discharge of flood hydrograph, $Q_p = 220\ m^3/s$

Base flow, $Q_b = 15\ m^3/s$

Infiltration index, $\phi = 2.5$ mm/hr

Catchment area, $A = 445\ km^2 = 445 \times 10^6\ m^2$

First, the effective rainfall depth is calculated by subtracting the infiltration loss during the storm period from the total rainfall.

$$R_e = P - (\phi \times T)$$

$$\Rightarrow \quad R_e = 47 - (2.5 \times 2)$$

$$\Rightarrow \quad R_e = 42\ \text{mm}$$

$$\Rightarrow \quad R_e = 4.2\ \text{cm}$$

Next, the peak of the direct runoff hydrograph is obtained by subtracting the base flow from the observed peak discharge.

$$Q_{p(\text{DRH})} = Q_p - Q_b$$

$$\Rightarrow \quad Q_{p(\text{DRH})} = 220 - 15$$

$$\Rightarrow \quad Q_{p(\text{DRH})} = 205\ m^3/s$$

The peak of the 2-hour unit hydrograph is calculated by dividing the peak of direct runoff hydrograph by the effective rainfall depth in cm.

$$U_p = \frac{Q_{p(DRH)}}{R_e}$$

$$\Rightarrow \quad U_p = \frac{205}{4.2}$$

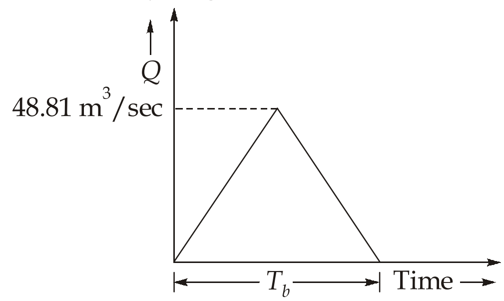
$$\Rightarrow \quad U_p = 48.81\ m^3/s \text{ per cm}$$

Now, assuming the unit hydrograph to be triangular in shape, the volume of direct runoff due to 1 cm of effective rainfall over the entire catchment area is calculated.

$$\text{Volume of 1 cm runoff} = A \times 0.01$$

$$= 445 \times 10^6 \times 0.01$$

$$= 4.45 \times 10^6\ m^3$$



For a triangular hydrograph, the volume is given by

$$\text{Volume} = \frac{1}{2} \times B \times U_p \times 3600$$

Equating the two volumes,

$$\frac{1}{2} \times B \times 48.81 \times 3600 = 4.45 \times 10^6$$

$$\Rightarrow \quad B = \frac{445 \times 10^6 \times 0.01 \times 2}{48.81 \times 3600}$$

$$\Rightarrow \quad B = 50.649 \text{ hr}$$

7. (b) (i) Solution:

Given : Section 1-1: $T = 240$ kN

$$\cos\theta = \frac{4}{5}, \sin\theta = \frac{3}{5}$$

$$d = 20 \text{ mm}$$

Bolt grade 4.6: $f_{ub} = 400$ MPa, $f_{yb} = 240$ MPa

$$n = 8 \text{ bolts}$$

$$f_u = 410 \text{ MPa}, \gamma_{mb} = 1.25$$

$$d_h = 22 \text{ mm}$$

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times 20^2 = 245.044 \text{ mm}^2$$

$$A_{sb} = \frac{\pi}{4} \times 20^2 = 314.159 \text{ mm}^2$$

Analysis of Bolts at Section 1-1

Design shear strength of a bolt:

$$V_{dsb} = \frac{f_{ub}}{\sqrt{3} \cdot \gamma_{mb}} \times A_{nb} = \frac{400}{\sqrt{3} \times 1.25} \times 245.044 \times 10^{-3} = 45.272 \text{ kN}$$

Design tensile strength of a bolt:

$$T_{nb} = 0.9 f_{ub} A_{nb} \ngtr f_{yb} \frac{\gamma_{mb}}{\gamma_{m0}} A_{sb}$$

$$\Rightarrow \quad T_{nb} = 0.9 \times 400 \times 245.044 \ngtr 240 \times \frac{1.25}{1.1} \times 314.159$$

$$\Rightarrow \quad T_{nb} = 88.22 \text{ kN} \ngtr 85.68 \text{ kN}$$

$$\Rightarrow \quad T_{nb} = 85.68 \text{ kN}$$

$$T_{db} = \frac{T_{nb}}{\gamma_{mb}} = \frac{85.680}{1.25} = 68.544 \text{ kN}$$

Applied forces per bolt:

$$T_H = T \cdot \cos\theta = 240 \times \frac{4}{5} = 192 \text{ kN}$$

$$T_V = T \cdot \sin\theta = 240 \times \frac{3}{5} = 144 \text{ kN}$$

$$T_b = \frac{T_H}{n} = \frac{192}{8} = 24 \text{ kN}$$

$$V_{sb} = \frac{T_V}{n} = \frac{144}{8} = 18 \text{ kN}$$

Interaction check:

$$\left(\frac{V_{sb}}{V_{dsb}}\right)^2 + \left(\frac{T_b}{T_{db}}\right)^2 = \left(\frac{18}{45.271}\right)^2 + \left(\frac{24}{68.544}\right)^2 = 0.281 \le 1.0 \qquad \text{(Ok)}$$

Hence, the joint at section 1-1 is safe.

Connection of Double Angle to Tee-bracket

Double Angle to Tee-bracket:

$$T = 240 \text{ kN}$$

$$d = 16 \text{ mm}, d_0 = 18 \text{ mm}$$

Bolt grade 4.6: $f_{ub} = 400 \text{ MPa}$

$$t_a = 10 \text{ mm}, t_w = 12 \text{ mm}$$

$$f_u = 410 \text{ MPa}, \gamma_{mb} = 1.25$$

Net tensile area of bolt:

$$A_{nb} = 0.78 \times \frac{\pi}{4} \times 16^2 = 156.828 \text{ mm}^2$$

The angle are placed on the opposite side of web of Tee-bracket.

Therefore, the bolts will be in double shear and bearing.

Double shear design strength:

$$V_{dsb} = 2 \times \frac{f_{ub}}{\sqrt{3} \cdot \gamma_{mb}} \times A_{nb} = 2 \times \frac{400}{\sqrt{3} \times 1.25} \times 156.828 \times 10^{-3} = 57.949 \text{ kN}$$

Bearing capacity calculation:

Minimum end distance: $e = 1.5 \times d_0 = 1.5 \times 18 = 27$ mm

Minimum pitch: $p = 2.5 \times d = 2.5 \times 16 = 40$ mm

Calculation of K_b:

$$\frac{e}{3d_o} = \frac{27}{3 \times 18} = 0.5$$

$$\frac{p}{3d_o} - 0.25 = \frac{40}{3 \times 18} - 0.25 = 0.491$$

$$\frac{f_{ub}}{f_u} = \frac{400}{410} = 0.976$$

$$K_b = \min(0.5, 0.491, 0.976, 1.0) = 0.491$$

Thickness for bearing:

$$t = \min(10 + 10, 12) = 12 \text{ mm}$$

Design bearing strength:

$$V_{dpb} = 2.5\, K_b dt \frac{f_u}{\gamma_{mb}} = \frac{2.5 \times 0.491 \times 16 \times 12 \times 410}{1.25} \times 10^{-3} = 77.303 \text{ kN}$$

Strength of a single bolt:

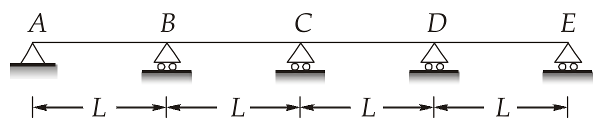
$$V_b = \min(V_{dsb}, V_{dpb}) = \min(57.949, 77.303) = 57.949 \text{ kN}$$

Number of bolts required:

$$n = \frac{T}{V_b} = \frac{240}{57.949} = 4.142 \approx 5$$

Therefore, provide 5 bolts of 16 mm diameter.

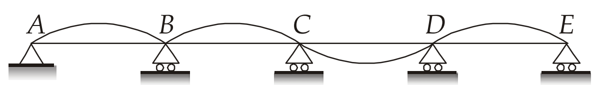
7. (b) (ii) Solution:



Dead load will act throughout the continuous beam over whole span *AE*.

1. Arrangement of live load to produce hogging maximum *BM* at support *B*.

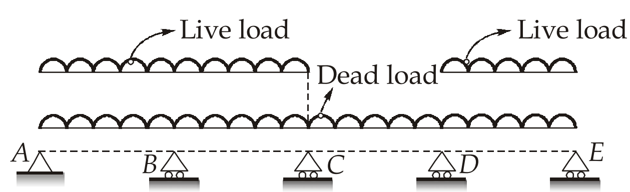
   As per Muller Breslau Principle, influence line diagram for bending moment at support *B* is qualitatively given as:



(For maximum Hogging *BM* at support *B*)

∴ Live load should be place over span *AB*, *BC* and *DE*.

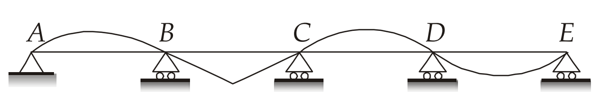
Arrangement of loads:



(For maximum Hogging *BM* at support *B*)

2. Arrangement of live load to produce maximum hogging *BM* at span *BC*.

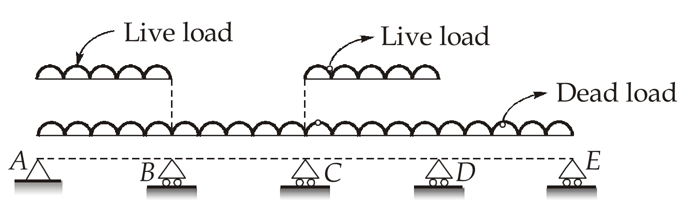
   ILD for *BM* at span *BC* as per Muller Breslau principle is qualitatively represented as:



(For maximum hogging *BM* at span *BC*)

∴ Either span *AB* and *CD* to be loaded or span *BC* and *DE* to be loaded.

Arrangement of loads:



(For maximum hogging *BM* at span *BC*)

7. (c) Solution:

Joint	Member	Relative Stiffness	Total stiffness	D.F.
B	BA	$\frac{3EI}{7.5} = \frac{2EI}{5}$	$\frac{26EI}{15}$	0.231
	BC	$\frac{4(2EI)}{6} = \frac{4EI}{3}$		0.769
C	CB	$\frac{4(2EI)}{6} = \frac{4EI}{3}$	$\frac{29EI}{15}$	0.690
	CD	$\frac{3EI}{5}$		0.310

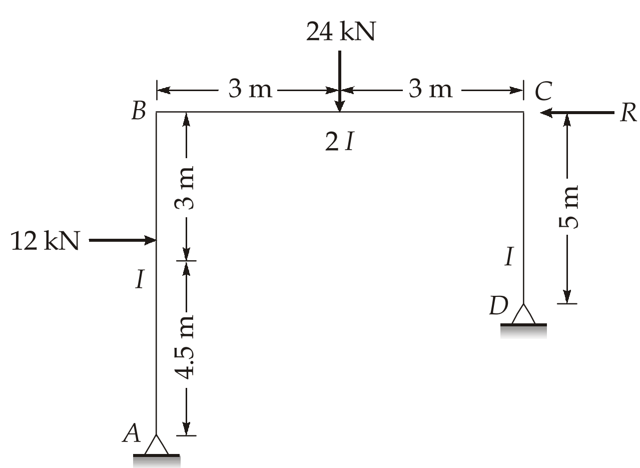
Non sway Analysis

$$M_{FAB} = \frac{12(4.5)(3)^2}{(7.5)^2} = -8.84 \text{ kNm}$$

$$M_{FBA} = \frac{12(3)(4.5)^2}{(7.5)^2} = 12.96 \text{ kNm}$$

$$M_{FBC} = \frac{-24\times 6}{8} = -18 \text{ kNm}$$

$$M_{FCB} = 18 \text{ kNm}$$



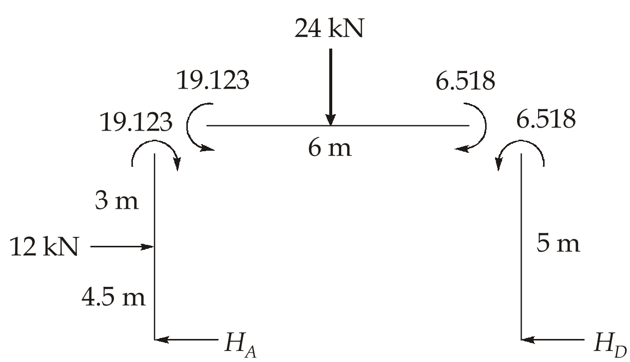
After transferring

$$M_{FAB} = 0$$

$$M_{FBA} = 12.96 + \frac{8.64}{2} = 17.28 \text{ kN-m}$$

A	0.231 B	0.769	0.690 C	0.310	D	
AB	BA	BC	CB	CD	DC	
	17.28	–18	18	0	0	FEM
		0.554	–12.42			Bal
		–6.21	0.277			C/O
		4.775	–0.191			Bal
		–0.096	2.388			C/O
		0.073	–1.648			Bal
		–0.824	0.037			C/O
		0.633	–0.025			Bal
		–0.013	0.317			C/O
		0.010	–0.219			Bal
		–0.109	0.005			C/O
		0.084	–0.003			Bal
	19.123	–19.123	6.518	–6.518		

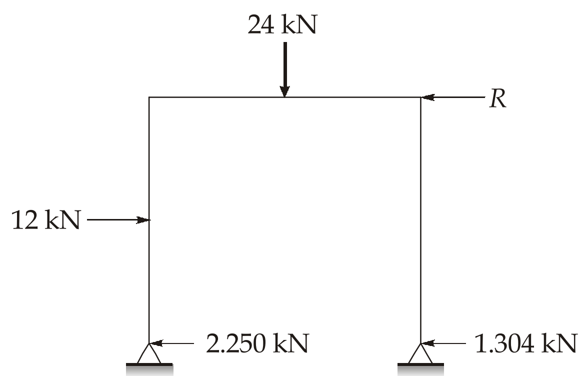
The free body diagram is as shown below:



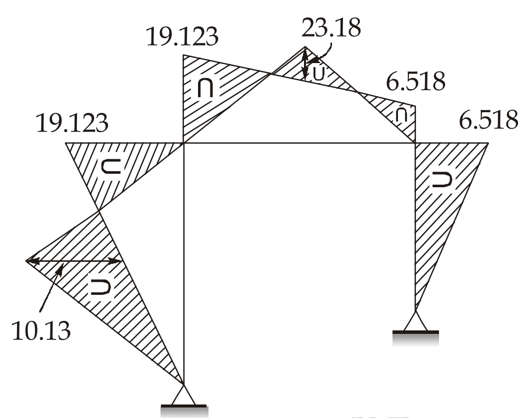
$$H_A = \frac{12 \times 3 - 19.123}{7.5} = 2.250 \text{ kN } (\leftarrow)$$

$$H_D = \frac{6.518}{5} = 1.304 \text{ kN } (\leftarrow)$$

The restraining force is calculated as follows:

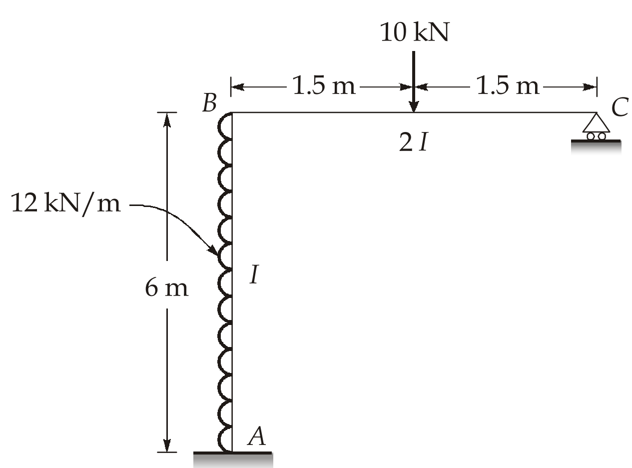


$$R = 12 - 2.250 - 1.304 = 8.446 \text{ kN}$$



BMD
(all values are in kN-m)

8. (a) Solution:



$$DOF = \theta_B, \theta_C \text{ and horizontal sway } \delta$$

Fixed end moments

$$M_{FAB} = \frac{wL_{AB}^2}{12} = -\frac{12 \times 6^2}{12} = -36 \text{ kNm}$$

$$M_{FBA} = +\frac{wL_{AB}^2}{12} = +\frac{12 \times 6^2}{12} = +36 \text{ kNm}$$

$$M_{FBC} = -\frac{WL_{BC}}{8} = -\frac{10 \times 3}{8} = -3.75 \text{ kNm}$$

$$M_{FCB} = +\frac{WL_{BC}}{8} = +\frac{10 \times 3}{8} = +3.75 \text{ kNm}$$

Slope deflection equations

Let the rotations at joints B and C be θ_B and θ_C and the horizontal sway be δ. Since A is fixed, $\theta_A = 0$.

$$M_{AB} = -36 + \frac{2EI}{6}\left[\theta_B - \frac{3\delta}{6}\right] = -36 + \frac{EI}{3}\left[\theta_B - \frac{\delta}{2}\right]$$

$$M_{BA} = 36 + \frac{2EI}{6}\left[2\theta_B - \frac{3\delta}{6}\right] = 36 + \frac{EI}{3}\left[2\theta_B - \frac{\delta}{2}\right]$$

$$M_{BC} = -3.75 + \frac{2E(2I)}{3}[2\theta_B + \theta_C]$$

$$M_{CB} = 3.75 + \frac{2E(2I)}{3}[2\theta_C + \theta_B]$$

Now, $M_{CB} = 0$,

$$\theta_C = -\frac{\theta_B}{2} - \frac{1.40625}{EI}$$

Substituting θ_C into M_{BC},

$$M_{BC} = -3.75 + \frac{4EI}{3}\left[2\theta_B - \frac{\theta_B}{2} - \frac{1.40625}{EI}\right] = 2EI\theta_B - 5.625$$

Joint B equilibrium,

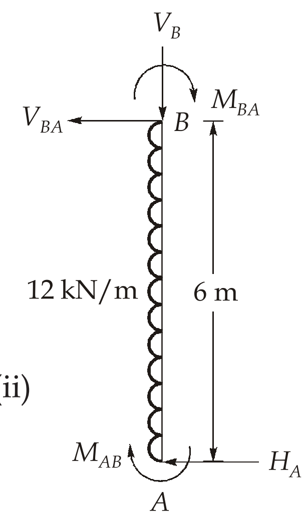
$$M_{BA} + M_{BC} = 0$$

$$\Rightarrow \quad 36 + \frac{2}{3}EI\theta_B - \frac{EI\delta}{6} + 2EI\theta_B - 5.625 = 0$$

$$2.667\, EI\theta_B - 0.167\, EI\delta = -30.375 \qquad \text{...(i)}$$

Shear condition for column AB

$$H_A = 12 \times 6 = 72 \text{ kN}$$



Taking moment about B,

$$M_{AB} + M_{BA} + H_A \times 6 - \frac{12\times 6^2}{2} = 0$$

$$\Rightarrow \quad M_{AB} + M_{BA} + 432 - 216 = 0$$

$$\Rightarrow \quad \frac{EI}{3}\left[3\theta_B - \delta\right] = -216$$

$$\Rightarrow \quad 3EI\theta_B - EI\delta = -648 \quad \text{...(ii)}$$

Solving equation (i) and (ii)

$$\delta = \frac{755.813}{EI}$$

$$\theta_B = \frac{35.937}{EI}$$

Final moments

$$M_{AB} = -36 + \frac{1}{3}\left[35.937 - \frac{755.813}{2}\right] = -150 \text{ kNm}$$

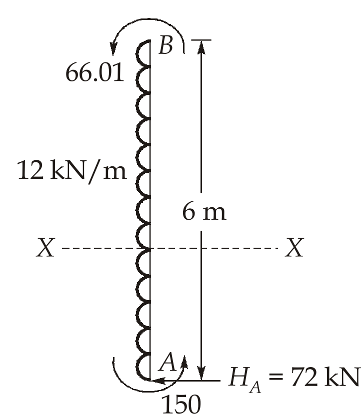
$$M_{BA} = 36 + \frac{1}{3}\left[2\times 35.937 - \frac{755.813}{2}\right] \simeq -66.2 \text{ kNm}$$

$$M_{BC} = 2 \times 35.937 - 5.625 = 66.2 \text{ kNm}$$

$$M_{CB} = 0$$

Point of contraflexure in member AB

Let the point of contraflexure be at a distance x from A.



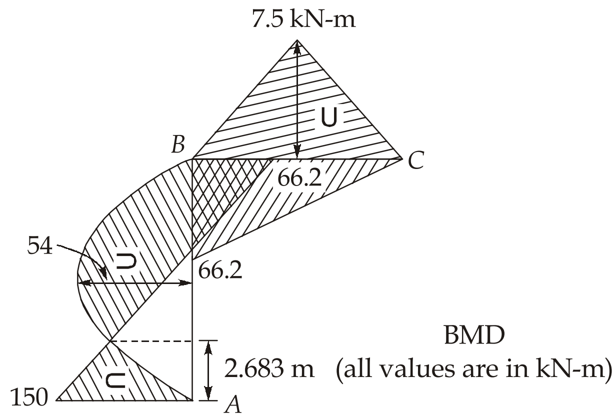
$$M_{xx} = H_A x + M_{AB} - \frac{12x^2}{2} = 0$$

$$72x - 150 - 6x^2 = 0$$

$$6x^2 - 72x + 150 = 0$$

Solving the quadratic equation, $x = 9.31 \text{ m} > 6 \text{ m}$ (not ok)

$$x = 2.689 \text{ m} < 6 \text{ m (ok)}$$



8. (b) (i) Solution:

Diameter of bolt hole, $d_0 = 20 + 2 = 22$ mm

Number of bolts, $n = 5$

Pitch, $p = 50$ mm

Calculation of Areas for Block Shear (for one angle)

Total length of shear plane, $L_v = 35 + (4 \times 50) = 235$ mm

Length of tension plane, $L_t = 35$ mm

Gross area in shear, $A_{vg} = L_v \times t = 235 \times 6 = 1410 \text{ mm}^2$

Net area in shear, $A_{vn} = [L_v - (4.5 \times d_0)] \times t$

$A_{vn} = [235 - (4.5 \times 22)] \times 6 = 816 \text{ mm}^2$

Gross area in tension, $A_{tg} = L_t \times t = 35 \times 6 = 210 \text{ mm}^2$

Net area in tension, $A_{tn} = [L_t - (0.5 \times d_0)] \times t$

$A_{tn} = [35 - (0.5 \times 22)] \times 6 = 144 \text{ mm}^2$

Block Shear Strength-

Case I: Shear Yielding and Tension Rupture

$$T_{db1} = \frac{A_{vg} f_y}{\sqrt{3}\gamma_{mo}} + \frac{0.9 A_{tn} f_u}{\gamma_{m1}}$$

$$\Rightarrow \quad T_{db1} = \frac{1410 \times 250}{\sqrt{3} \times 1.1} + \frac{0.9 \times 144 \times 410}{1.25}$$

$$\Rightarrow \quad T_{db1} = 227.523 \text{ kN}$$

Case II: Shear Rupture and Tension Yielding

$$T_{db2} = \frac{0.9A_{vn}f_u}{\sqrt{3}\gamma_{m1}} + \frac{A_{tg}f_y}{\gamma_{m0}}$$

$$\Rightarrow \quad T_{db2} = \frac{0.9 \times 816 \times 410}{\sqrt{3} \times 1.25} + \frac{210 \times 250}{1.1}$$

$$\Rightarrow \quad T_{db2} = 186.801 \text{ kN}$$

Block shear strength for one angle is the minimum value,

$$T_{db(\text{single})} = 186.801 \text{ kN}$$

Total Block Shear Strength of the Member

$$T_{db(\text{total})} = 2 \times 186.801 = 373.602 \text{ kN}$$

$$\Rightarrow \quad T_{db(\text{total})} = 373.602 \text{ kN}$$

8. (b) (ii) Solution:

Given data

Factored load, $P = 160$ kN

Eccentricity, $e = 150$ mm

Length of weld, $l = 450$ mm

Inclination angle, $\theta = 45°$

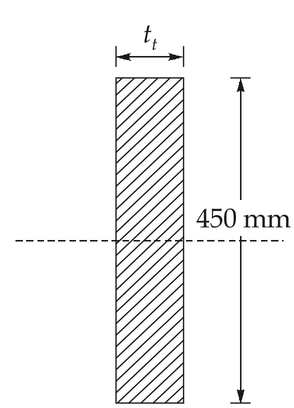
Components of factored load

Vertical component, $P_v = 160 \sin 45° = 113.137$ kN

Horizontal component, $P_h = 160 \cos 45° = 113.137$ kN

Sectional properties of weld

Let t_t be the effective throat thickness.



Area of weld, $A_w = 450 \times t_t$

Moment of inertia, $I_{xx} = \frac{t_t \times 450^3}{12}$

$\Rightarrow$ $I_{xx} = 7593750 \times t_t \text{ mm}^4$

$\because$ $I_{yy} \approx 0$

$\therefore$ Polar MOI, $I_p = I_{xx} + I_{yy} = 7593750 \times t_t \text{ mm}^4$

Shear stresses

Critical point at extreme end of weld,

$$r_{max} = \frac{450}{2} = 225 \text{ mm}$$

Direct vertical shear stress, $q_1 = \frac{113137}{450 \times t_t} \text{ (N/mm}^2\text{)}$

$\Rightarrow$ $q_1 = \frac{251.416}{t_t} \text{ (MPa)}$

Direct horizontal shear stress, $q_2 = \frac{113137}{450 \times t_t}$

$\Rightarrow$ $q_2 = \frac{251.416}{t_t} \text{(MPa)}$

Moment, $M = 113137 \times \left(\frac{450}{2} - 150\right) = 8485275 \text{ N-mm}$

Torsional shear stress, $q_3 = \frac{M \times r_{max}}{I_p}$

$\Rightarrow$ $q_3 = \frac{8485275 \times 225}{7593750 \times t_t}$

$\Rightarrow$ $q_3 = \frac{251.416}{t_t}(MPa)$

Resultant stress

Total horizontal stress,

$$f_h = q_2 + q_3 = \frac{502.832}{t_t} \text{(MPa)}$$

Total vertical stress, $f_v = \frac{251.416}{t_t} \text{ (MPa)}$

Resultant stress, $f_r = \sqrt{\left(\frac{502.831}{t_t}\right)^2 + \left(\frac{251.416}{t_t}\right)^2}$

$\Rightarrow$ $f_r = \frac{562.182}{t_t}(\text{MPa})$

Determination of weld size

Design weld strength, $f_{wd} = \frac{410}{\sqrt{3} \times 1.25}$

$\Rightarrow$ $f_{wd} = 189.37$ MPa

For safe connection, $\frac{562.182}{t_t} \leq 189.37$

$\Rightarrow$ $t_t \geq 2.968$ mm

$\Rightarrow$ $0.7s \geq 2.968$

$\Rightarrow$ $s \geq 4.240$ mm

Provide a fillet weld size of 5 mm.

8. (c) Solution:

The *ILD* for shear force at centre of span is as shown in figure. For maximum negative shear force the leading load should be on the section. The maximum *ILD* ordinate for shear force at *C*.

$$= \frac{12}{24} = 0.5$$

Maximum negative S.F. at mid-span

$$= 60 \times 0.5 + \left[90 \times \frac{3}{8} + 100 \times \frac{7}{24} + 40 \times \frac{1}{6} + 40 \times \frac{1}{12}\right]$$

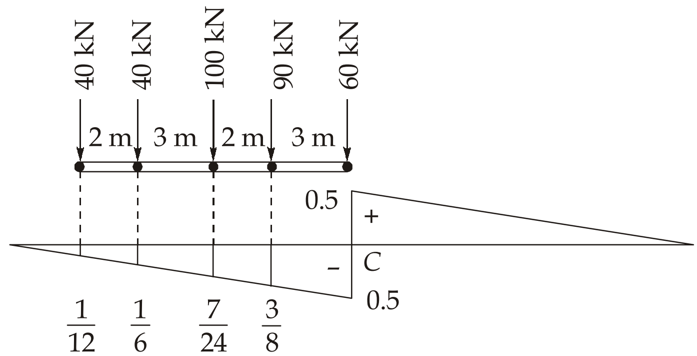
$$= 102.916 \text{ kN}$$

For maximum +ve shear force at *C*, the trailing load should be at *C*.

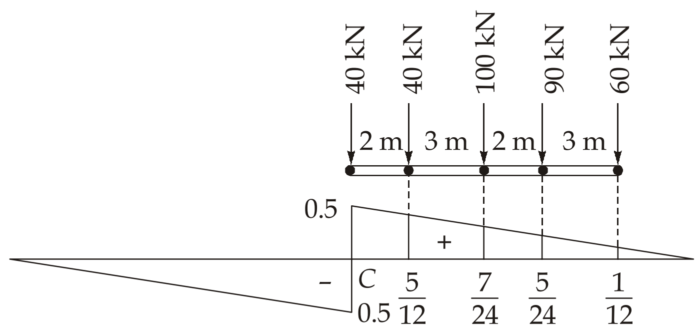
Maximum positive shear force at *C*.

$$= 40 \times 0.5 + \left[40 \times \frac{5}{12} + 100 \times \frac{7}{24} + 90 \times \frac{5}{24} + 60 \times \frac{1}{12}\right]$$

$$= 89.583 \text{ kN}$$

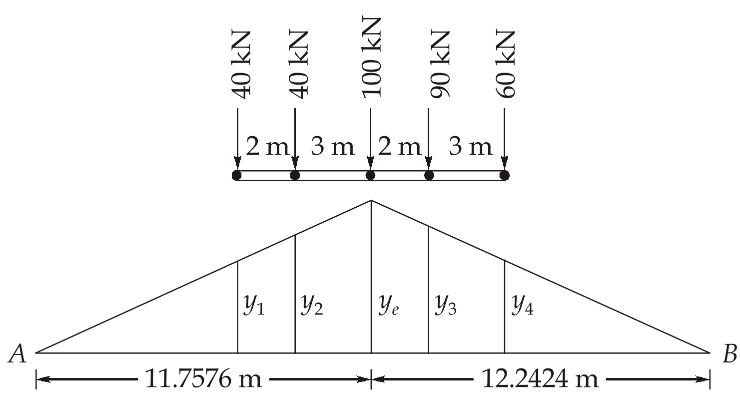


(ILD for SF at C)



(ILD for SF at C)

For absolute maximum bending moment:



Let C.G. of loads from leading wheel load be at a distance x.

Then,
$$x = \frac{90\times3+100\times5+40\times8+40\times10}{60+90+100+40+40}$$
$$= 4.5152 \text{ m}$$

This is near to 100 kN load. Hence, maximum moment is likely to occur under this load. Its distance from CG.

$$= 5 - 4.5152 = 0.4848 \text{ m}$$

Therefore, its position from end A

$$= 12 - \frac{0.4848}{2} = 11.7576 \text{ m}$$

Therefore, ILD ordinate under this load is

$$y_c = \frac{z(L-z)}{L} = \frac{11.7576(24 - 11.7576)}{24} = 5.99$$

Referring to figure

Therefore, absolute maximum $BM = 40y_1 + 40y_2 + 100y_c + 90y_3 + 60y_4$

$$= \left[40 \times \frac{6.7576}{11.7576} + 40 \times \frac{8.7576}{11.7576} + 100 + 90 \times \frac{10.2424}{12.2424} + 60 \times \frac{7.2424}{12.2424}\right] 5.99$$

$$= 1578.82 \text{ kNm}$$

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