MADE EASY

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Detailed Solutions

ESE-2026 Mains Test Series

Civil Engineering Test No : 9

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

1. (a) Solution:

Light Weight Concrete

Light weight concrete (LWC) is a concrete having lower density than conventional concrete and is mainly used to reduce dead load in structures. It is produced either by using lightweight aggregates such as pumice, clinker, foamed slag and blast furnace slag, or by entraining air in cement concrete. The density of LWC generally varies from 300-1200 kg/m^3. Depending upon the method of manufacture, it may be classified as no-fines concrete, cellular concrete, foam concrete and gas concrete. Lightweight concrete possesses good thermal insulation, excellent fire resistance and economy due to reduced self-weight. It also provides sound insulation and ease in handling, transportation and construction. The compressive strength generally ranges from 20-35 MPa, though higher strengths can also be achieved. Due to porosity, durability and corrosion resistance require special attention. Lightweight concrete is commonly used in precast floor and roofing units, load bearing walls, insulation cladding, shells, domes and prefabricated structural elements.

High Performance Concrete

High performance concrete (HPC) is a specially designed concrete that satisfies special performance and uniformity requirements which cannot be achieved routinely with conventional concrete. The concept of HPC evolved from the need for higher durability, strength and improved service life in structures. In addition to high compressive strength, HPC possesses characteristics such as low permeability, high durability, resistance to freezing and thawing, abrasion resistance and improved workability. It is generally produced using mineral admixtures like fly ash, silica fume and ground granulated blast furnace slag along with chemical admixtures such as high-range water reducers. HPC mixtures usually contain low water-cement ratio and carefully selected aggregates. The concrete is sensitive to variations in constituent materials and therefore requires strict quality control during production. High performance concrete is widely used in bridges, marine structures, pavements and high-rise buildings where environmental exposure and structural demands are severe. Examples include self-compacting concrete, roller compacted concrete and high strength concrete.

1. (b) Solution:

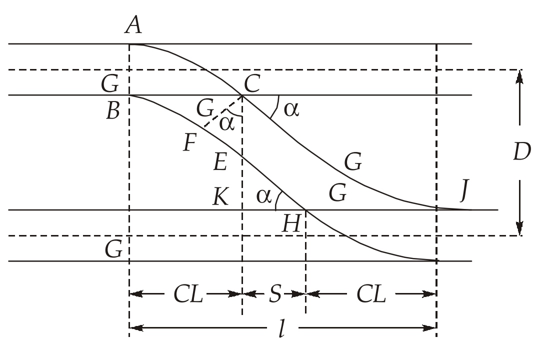
Given data

Crossing number, $N = 10$

Broad gauge, $G = 1.676$ m

Distance between centers of tracks, $D = 6$ m

Crossover with intermediate position straight and crossing angle equal for two parallel railways tracks is shown in figure.



where, D = Distance between centres of parallel tracks

α = Angle of crossing

S = Horizontal projection of immediate portion of main track

CL = Curve lead

G = Gauge

1. Overall length of crossover:

$$L = 4GN + (D - G)N - G\sqrt{1+N^2}$$

$$\Rightarrow \quad L = (4 \times 1.676 \times 10) + (6 - 1.676) \times 10 - 1.676\sqrt{1+10^2}$$

$$\Rightarrow \quad L = 93.436 \text{ m}$$

2. Intermediate straight distance:

$$S = (D - G)N - G\sqrt{1+N^2}$$

$$\Rightarrow \quad S = (6 - 1.676) \times 10 - 1.676\sqrt{1+10^2}$$

$$\Rightarrow \quad S = 26.396 \text{ m}$$

1. (c) Solution:

The dry process of manufacturing Ordinary Portland Cement (*OPC*) is a modern and fuel efficient method where the raw materials are crushed, ground and mixed in a dry state before being fed into the kiln.

Flow Diagram of Dry Process:

[Calcareous Materials (Limestone/Chalk)] & [Argillaceous Materials (Clay/Shale)]

↓

Crushing (To reduce size to about 25 mm)

↓

Drying (Removal of moisture in dryers)

↓

Grinding & Mixing (In ball mills to form "Raw Meal")

↓

Blending Silo (To ensure correct composition)

↓

Pre-heating (Using exit gases from kiln)

↓

Rotary Kiln (Burning at 1400°C-1500°C to form Clinker)

↓

Clinker Cooling

↓

Clinker Grinding + Gypsum (Gypsum 2-3% added for setting control)

↓

Cement storage and packing

Process Description

- **Collection of Raw Materials:** The primary raw materials are calcareous (lime-rich, like limestone) and argillaceous (silica/alumina-rich, like clay).
- **Crushing and Drying:** Materials are crushed in gyratory crushers to small sizes. If the raw materials contain moisture, they are passed through dryers to ensure they are completely dry before grinding.

- **Grinding:** The dried materials are ground into a very fine powder in ball or tube mills. This fine powder is known as the **raw meal**.
- **Blending:** The raw meal is stored in blending silos where compressed air is used to ensure a uniform and homogeneous mix of the chemical components.
- **Burning (The Kiln):** The blended meal is fed into a rotary kiln. As it moves toward the flame, it passes through different temperature zones. At the burning zone (approx. 1400°C to 1500°C), chemical reactions occur, and the material fuses into small, hard, dark-grey nodules called **clinker**.
- **Grinding of Clinker:** Once cooled, the clinker is ground in ball mills. During this final grinding, about 2-3% gypsum is added. The gypsum acts as a retarder to prevent the "flash set" of cement when mixed with water.
- **Storage and Packing:** The finished cement is then stored in silos and eventually packed into bags (typically 50 kg) for distribution.

1. (d) Solution:

Given:

First horizon

Thickness, $d_1 = 25$ cm

Field capacity, $FC_1 = 18\% = 0.18$

Permanent wilting point, $PWP_1 = 8\% = 0.08$

Dry density, $\gamma_{d1} = 1.6$ g/cm^3

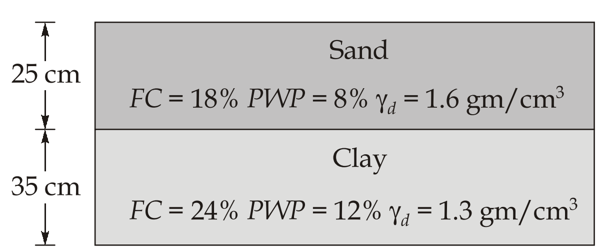
Second horizon

Thickness, $d_2 = 35$ cm

Field capacity, $FC_2 = 24\% = 0.24$

Permanent wilting point, $PWP_2 = 12\% = 0.12$

Dry density, $\gamma_{d2} = 1.3$ g/cm^3



Consumptive use, $C_u = 0.6$ mm/day $= 0.06$ cm/day

Density of water, $\gamma_w = 1$ g/cm^3

Moisture holding capacity of first horizon

$$d_{w1} = \frac{\gamma_{d1} d_1}{\gamma_w}(FC_1 - PWP_1)$$

$$\Rightarrow \quad d_{w1} = \frac{1.6 \times 25}{1}(0.18 - 0.08)$$

$$\Rightarrow \quad d_{w1} = 4 \text{ cm}$$

Moisture holding capacity of second horizon

$$d_{w2} = \frac{\gamma_{d2} d_2}{\gamma_w}(FC_2 - PWP_2)$$

$$\Rightarrow \quad d_{w2} = \frac{1.3 \times 35}{1}(0.24 - 0.12)$$

$$\Rightarrow \quad d_{w2} = 5.46 \text{ cm}$$

Total maximum available moisture

$$\text{Available moisture} = 4 + 5.46 = 9.46 \text{ cm}$$

Number of days the crop will survive

$$\text{Days} = \frac{\text{Maximum available moisture}}{\text{Consumptive use}}$$

$$\Rightarrow \quad \text{Days} = \frac{9.46}{0.06}$$

$$\Rightarrow \quad \text{Days} = 157.667 \text{ days}$$

1. (e) (i) Solution:

Workability: Workability is defined as the property of concrete which determines the amount of useful internal work necessary to produce full compaction. It can also be defined as the ease with which concrete can be compacted 100% with regard to mode of compaction and place of deposition. Workability is different than consistency. The latter indicates degree of fluidity or mobility.

1. (e) (ii) Solution:

(a) Size of aggregate: For big aggregate size, the total surface area to be wetted is less, also less paste is required for lubricating the surface to reduce internal friction. For a given water content big size aggregate give high workability.

(b) Cement content: Cement content influences the workability to a large extent. The higher the cement content, less leaner will be the concrete i.e., less workable.

(c) **Water cement ratio:** The fluidity of concrete increase with water cement ratio. At side, normal practice is to increase the water cement ratio to make the concrete workable which lowers strength.

(d) **Air Entraining Agents:** Air entrainment increases workability, and resistance of concrete of weathering. The possibility of bleeding and segregation and laitance is also reduced. However there is some loss in the strength of concrete.

2. (a) Solution:

Methods of tunnelling in hard rocks:

In hard rocks generally following methods of tunneling are more common:

1. Full face method
2. Heading and benching method
3. Drift method
4. Pilot tunnel method

1. Full face method. This method is adopted for tunnels whose length is not more than 3 metres. Large size tunnels in rocks are always driven by this method. With the development of drill carriage, this method is becoming more and more popular. In this method vertical columns are fixed at the face of the tunnel to which a large number of drills may be mounted or fixed at any suitable height. A series of drill holes are drilled at about 120 cm centre to centre in any number of desired rows, preferably in two rows. The size of the holes may vary from 10 mm to 40 mm. These holes are then charged with explosives and ignited. The muck is removed before the next operation of drilling holes. This tunnel is more suitable for diameters less than 6 m and face area less than 19 m^2.

Advantages of full face method:

1. It requires minimum equipment. Hence it is simple in operation.
2. The magnitude of ground disturbance and settlement is minimum in this method.
3. The work is easily and speedily completed by this method.
4. The mocking trucks can be laid once for all on the tunnel floor and extended progressively.
5. It is found advantageous in sensitive ground conditions where multiple phase excavation could generate excessive ground pressure and settlement effects.

2. **Heading and bench method:** The top portion is known as the heading and the bottom portion as bench. Usually this method is adopted for railway tunnels. In this method of tunnelling the top portion of heading will be about 3.70 m to 9.6 m ahead of the bottom portion. In hard rock which may permit the roof to withstand without supports, the top heading usually is advanced by one round of the bottom portion. If the rock is broken then heading may be driven well ahead of the bottom portion and after giving proper support to the roof, the bottom portion is completed. In hard rock the heading is bored first and the drill holes are driven for the bench or bottom portion at the same time as the removal of the muck. This is the main advantage of this method. It requires less explosives than full face method.

   The advantages and disadvantage of this method are similar to the drift method.

3. **Drift method:** Drift is a small tunnel, usually its size is 300 cm × 300 cm. In driving a large tunnel it has been found advantageous to drive a drift first through the full length or in a portion of the length of the tunnel prior to excavating the full bore. The drift may be provided at the centre, sides, bottom or top as desired. In this method after driving the drift, the drill holes are drilled all round the drift in the entire cross-section of the tunnel, filled with explosives and ignited. The rock shatters, the muck removed and the tunnel expanded to the full cross-section.

   Advantages of drift method:

   (a) By this method any bad rock or excessive water will be discovered prior to driving full tunnel enabling to take corrective measures at the earliest.

   (b) The drift assists in ventilating the tunnel during later operations.

   (c) The quantity of explosive required is reduced.

   (d) The side drifts provide facility to install timbering to provide support to the roof, specially when the tunnel is driven in broken rock.

   Disadvantages of drift method:

   (a) Driving of main tunnel get delayed until the drift is finished.

   (b) As the drift is a small hole, the cost of drilling and handling muck will be high as the work has to be performed manually instead of power driven equipment.

4. **Pilot tunnel method:** In this method usually two tunnels are to be driven (1) main tunnel (2) pilot tunnel.

Actually this method is adopted to expedite the driving of the main tunnel. The cross-section of the pilot tunnel is usually 240 cm × 240 cm and driven parallel to the main tunnel. The pilot tunnel which is first driven to the full length is connected to the centre line of the main tunnel. The main tunnel then can be started from a number of points. The pilot tunnel also serves the following purposes:

1. It helps in removing muck from the main tunnel quickly.
2. It helps in providing proper ventilation in the main tunnel.
3. It helps in providing proper lighting in the main tunnel.

Advantages of pilot tunnel method:

1. The cross headings can be used for storing tools and materials during construction period.
2. It is found cheaper than central shaft method.
3. Chances of falling materials into the tunnel during construction are reduced considerably.
4. It avoids dislocation of strata at the sides of the tunnel.
5. After completion of work, cross headings may be used as passage by workers engaged in repair and maintenance.
6. The shafts may be lowered to act as sump for the collection of seeping water. Thus these shafts may be used as means of pumping water and artificial ventilation by the use of pumps and fans respectively.

2. (b) (i) Solution:

Depth of water for each crop is calculated using:

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

Volume required for each crop:

$$V = A \times \Delta$$

Crop	B(days)	A(ha)	D(ha/cumec)	Δ(m)	Volume $A \times \Delta$ (ha-m)
Cotton	210	4000	1500	1.21	4840
Sugarcane	360	2500	900	3.456	8640
Wheat	120	5000	2000	0.518	2590
Rice	120	3000	800	1.296	3888

Total volume of water required at the field:

$$V_{total} = 4840 + 8640 + 2590 + 3888$$

$$\Rightarrow \quad V_{total} = 19958 \text{ ha-m}$$

Accounting for Canal Losses

Canal losses are 25%, so water required at the canal head:

$$\Rightarrow \quad V_{canal\ head} = \frac{V_{total}}{1-0.25}$$

$$\Rightarrow \quad V_{canal\ head} = \frac{19958}{0.75} = 26610.667 \text{ ha-m}$$

Accounting for Reservoir Losses

Reservoir losses are 10%, so live storage required in the reservoir:

$$V_{live\ storage} = \frac{V_{canal\ head}}{1-0.10}$$

$$\Rightarrow \quad V_{live\ storage} = \frac{26610.667}{0.9} = 29567.408 \text{ ha-m}$$

The required live storage of the reservoir is 29,567.408 ha-m.

2. (b) (ii) Solution:

Objective of River Training

River training measures aim at achieving one or more of the following objectives:

(a) Flood protection: River floods of very small frequency inundate the fertile and thickly populated plains adjacent to the river, and thus, cause considerable loss to human life, property, agriculture, and public and private utilities.

During the years of large floods, damage is likely to be several times more. Flood control measures for thickly-populated flood plains, therefore, become essential, even if these measures do not assure complete protection under all conditions. River training for flood protection, also known as 'high water training' or 'training for discharge,' is achieved by one or more of the following four methods:

1. Construction of levees or embankments to confine water in a narrower channel.
2. Increasing the discharge capacity of natural channels by some means such as straightening, widening or deepings.
3. Provision of escapes or diversion from the main channel into an auxiliary channel for water in excess of the carrying capacity of the main channel.
4. Construction of reservoirs.

(b) **Navigation:** For a river to be navigable, sufficient depth and width required for navigation should be available even at low water level in the river. River training for navigation is also known as 'low water training'; or training for depth'. Measures to achieve adequate depth in a river for navigation include dredging the shallow reaches of the river and using spurs to contract the river channel, thus, increasing its depth. Sometimes, low flow is supplemented from another source. This is accomplished by building a series of small dams or weirs and locks. Sharp curves along the river need to be eliminated so that ships can move easily.

(c) **Sediment control:** River training for sediment control is also called 'mean water training' or 'training for sediment'. This types of training aims at rectification of river bed configuration and efficient movement of sediment load for keeping the channel in a state of equilibrium. River training methods for this purpose involve construction of such structures which would induce the desired local curvature to the flow. Spurs and pitched islands are normally used for training the river for sediment.

(d) **Guiding the flow:** Hydraulic structures, such as canal headworks, and communication structures such as bridges, have to be protected against outflanking and the direct attack of flow. This requires training of the river over its considerable reach by building a system of guide banks, known as Bell's guide banks, on one or both sides of the stream at the bridge site. The purpose of these guide banks is to make sure that water flows between the abutments of the bridge. The spacing between these guide banks conforms to the width required for the river to pass the design flood discharge. Similarly, guide banks are provided to guide the flow at the weir site. Marginal bund and lateral spurs guide the flow through the guide banks.

Sometimes the flow in a river needs to be deflected away from a bank in order to protect some portions of the river bank or for contracting the river. This is done by constructing one or more spurs projecting into the river from its banks.

(e) **Stabilization of river channel:** Weak river banks, which are likely to cave in or get eroded, need to be protected by training methods, such as stone pitching, lining, and so on. In some cases, the stability of the bed may also be endangered in some reaches due to increase in the bed shear on account of local flow conditions.

2. (c) Solution:

A gravity dam may fail in the following ways:

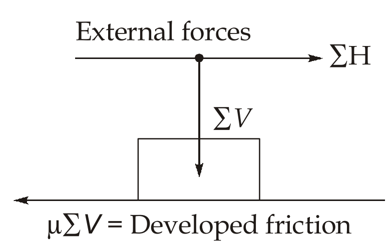
1. Overturning
2. Sliding
3. Compression or crushing
4. Tension

1. Overturning: If resultant of all the force acting on a dam at any of its sections cuts the base of the dam downstream of the toe (i.e., outside the body of dam), the dam shall rotate and overturn about the toe. On the other hand, if the resultant cuts the base within the body of the dam, there will be no overturning. Practically, the dam will fail by compression much earlier than overturning. For stability requirements, dam should be safe against overturning.

$$\text{FOS (against overturning)} = \frac{\Sigma \text{ restoring moment about toe (anticlockwise)}}{\Sigma \text{ overturning moment about toe (clockwise)}} = \frac{\Sigma M_R}{\Sigma M_o}$$

FOS against overturning should not be less than 1.5.

2. Sliding: A dam will fail in sliding at its base or at any other level, if the horizontal forces causing sliding are more than the frictional resistance developed at that level. As we know, fraction developed between two surfaces is equal to $\mu\Sigma V$ where μ is coefficient of friction between two surfaces and ΣV is algebraic sum of vertical forces.



Now, factor of safety against sliding, $\text{F.S.S.} = \dfrac{\mu \Sigma V}{\Sigma H}$ ...(i)

For dam to be safe in sliding, F.S.S. should be greater than 1.

Resistance against sliding in low dams is provided only by friction but in high dams, shear strength of joint also provide shear resistance and hence, it should also be considered and factor of safety against sliding is known as **shear friction factor** if shear strength is considered.

Now, Shear friction factor $(S.F.F.) = \dfrac{\mu \Sigma V + B.q}{\Sigma H}$ ...(ii)

where, B = Width of dam at joint

q = Average shear strength of joint

μ = Coefficient of friction

3. **Compression or Crushing:** A dam may fail in compression if the maximum compressive stress exceeds the allowable stress.

Now, Vertical stress, p at base = Direct stress + Bending stress

$$= \frac{\Sigma V}{B} \pm \frac{M}{I} y = \frac{\Sigma V}{B} \pm \frac{\Sigma Ve}{\frac{B^2}{6}}$$

$$= \frac{\Sigma V}{B}\left[1 \pm \frac{6e}{B}\right] \quad \text{...(i)}$$

Thus,

$$p_{max} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right) \quad \text{...(ii)}$$

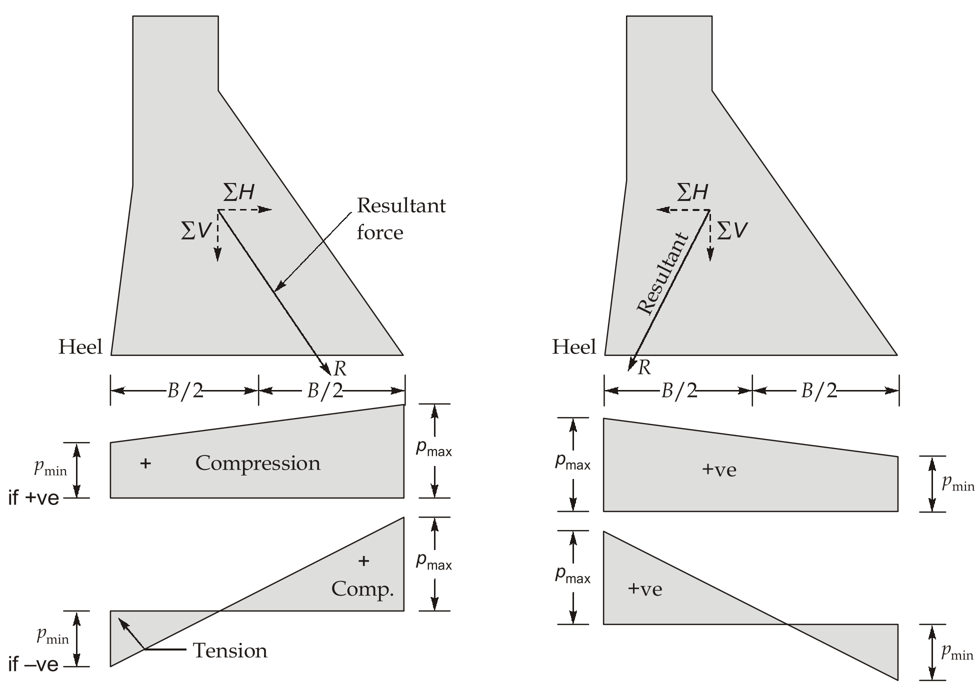
$$p_{min} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right) \quad \text{...(ii)}$$

where, e = Eccentricity of resultant force from centre of base

ΣV = Total vertical force

B = Base width

The maximum stress i.e. p_{max} will be on the end which is nearer to resultant as shown in figure (a) and figure (b) shown below.



(a) Reservoir full condition **(b)** Reservoir empty condition

Fig. Resultant force on the body of dam

It is to noted that in figure (a), resultant is near to toe and hence, p_{max} is produced at toe. This condition will exist when reservoir is full. In figure (b), resultant is near to heel therefore p_{max} is produced at heel. The condition will exist when reservoir is empty and horizontal earthquake wave is moving away from reservoir resulting in horizontal earthquake force toward the heel of dam.

4. **Tension:** Gravity dams are generally made of concrete or masonry. As we know that these materials cannot withstand tensile stresses and if these materials are subjected to tension, they may finally crack. Hence it is attempted that no such stresses are induced in dam.

Effect Produced by Transfer Crack

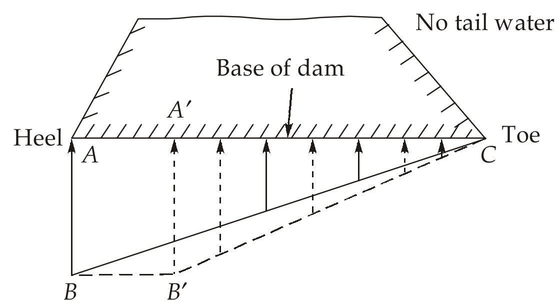


Fig. Effect of tension crack

If cracks are developed in a dam due to tension, then at location of crack it losses contact with bottom foundation and hence effective width reduces which will increase compressive stresses also. Moreover, uplift pressure also increases due to tension cracks as shown in figure.

As we know that uplift pressure reduces the major stabilizing force for the dam i.e. its self weight. If uplift pressure is increased due to cracks, then it will cause more reduction in stabilizing force and stability of dam will reduce. Therefore, a tension crack by itself does not fail the structure, but it leads to failure of structure by producing excessive compressive stresses.

In order to ensure that no tension is developed anywhere in dam, p_{min} should not be negative. It can have a minimum value equal to zero i.e. $p_{min} \geq 0$ for no tension.

$$p_{min} = \frac{\Sigma V}{B}\left[1 - \frac{6e}{B}\right] \geq 0$$

$$\Rightarrow \quad e \leq \frac{B}{6} \quad \text{...(i)}$$

Hence, maximum value of eccentricity that can be permitted on either side of centre is equal to $\left(\frac{B}{6}\right)$ i.e. resultant must lie within middle third.

3. (a) (i) Solution:

In a barrage, sheet piles are provided at the upstream and downstream ends of the impervious floor to ensure hydraulic stability against seepage forces.

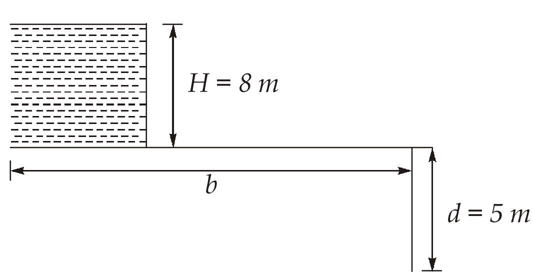
Upstream pile: The upstream pile is mainly provided to control uplift pressure beneath the floor. When water flows from the upstream to the downstream side, seepage develops under the floor, creating uplift pressure that tends to lift the structure. By driving a sheet pile at the upstream end, the seepage path is lengthened, and the hydraulic gradient is reduced. This results in a reduction in uplift pressure acting on the floor. The upstream pile also contributes to reducing the exit gradient at the downstream end to some extent, thereby improving overall safety against piping and subsoil erosion.

Downstream Pile: The downstream pile is primarily intended to safeguard the structure against piping failure. At the downstream end, water emerging from the subsoil creates an upward pressure. If the exit gradient exceeds the critical gradient of the soil, boiling or piping may occur, leading to failure of the foundation. The downstream sheet pile increases the length of the seepage path and significantly lowers the exit gradient at the point of emergence. Thus, it ensures that the exit gradient remains within safe limits and prevents undermining of the structure.

The depth of upstream and downstream piles is determined based on seepage analysis of the foundation soil. Traditionally, the design has been carried out using Bligh's creep theory, which assumes that seepage follows the contact surface between the structure and the subsoil, and the total creep length required is proportional to the head causing seepage. In more refined practice, Khosla's theory is adopted. This method is based on potential flow theory and considers the actual distribution of uplift pressure and exit gradient. By applying these theories, the required depth of sheet piles is calculated such that uplift pressures are within permissible limits and the exit gradient does not exceed the critical value for the soil.

3. (a) (ii) Solution

Given: $H = 8 \text{ m}, d = 5 \text{ m}$



The exit gradient according to Khosla's theory is given by

$$G_E = \frac{H}{d} \times \frac{1}{\pi\sqrt{\lambda}}$$

$$\Rightarrow \quad \frac{1}{5} = \frac{8}{5} \times \frac{1}{\pi\sqrt{\lambda}}$$

$$\Rightarrow \quad \lambda = 6.485$$

The parameter λ is also defined as

$$\lambda = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\Rightarrow \quad 6.485 = \frac{1+\sqrt{1+\alpha^2}}{2}$$

$$\Rightarrow \quad \alpha = 11.928$$

The horizontal floor length is calculated as

$$b = \alpha \times d$$

$$\Rightarrow \quad b = 11.928 \times 5$$

$$\Rightarrow \quad b = 59.64 \text{ m}$$

3. (b) (i) Solution:

Ferrocement: Ferrocement is a type of thin-wall reinforced concrete construction where small-diameter wire meshes are used as reinforcement and are impregnated with a rich cement mortar mix. It does not include coarse aggregates. The reinforcement consists of layers of mesh (such as chicken mesh or woven wire mesh), sometimes supported with small-diameter steel rods (skeletal reinforcement).

Advantages of Ferrocement:

1. High tensile strength due to closely spaced wire meshes.
2. Better crack control and durability.
3. Can be moulded into thin, curved sections.
4. Economical for small-scale applications.
5. Requires less skilled labour compared to *RCC*.

Applications of Ferrocement:

1. Water tanks, boats, silos, and biogas tanks.
2. Precast roofing channels and ferrocement shells.
3. Rural sanitation units and affordable housing.
4. Manhole covers, pipes, and cover slabs.

Fibre reinforced concrete:

Fibre Reinforced Concrete (FRC) is concrete containing fibrous materials uniformly distributed and randomly oriented. These fibres act as crack arrestors and improve the tensile strength, toughness, and ductility of concrete. Common types of fibres include steel, glass, synthetic fibres (like polypropylene), and natural fibres. FRC improves the behavior of concrete under load and especially enhances post-cracking performance. It is used where control of plastic shrinkage cracks and improved impact resistance are important.

Advantages of Fibre Reinforced Concrete:

1. Reduces microcracks and plastic shrinkage.
2. Enhances tensile and flexural strength.
3. Improves resistance to impact and fatigue.
4. Increases ductility and post-cracking toughness.
5. Enhances abrasion and freeze-thaw resistance.

Applications of Fibre Reinforced Concrete:

1. Industrial floors, pavements, and precast segments.
2. Tunnel linings and slope stabilisation (shotcrete).
3. Airport runways and bridge decks.
4. Earthquake-resistant and blast-resistant structures.

3. (b) (ii) Solution:

Given data

$$\text{Water content} = 172 \text{ kg/m}^3$$

$$\text{Target}\left(\frac{w}{c}\right) = 0.50$$

$$\text{Maximum permissible}\left(\frac{w}{c}\right) = 0.40$$

$$\text{Mortar content} = 56\% = 0.56 \text{ (by volume)}$$

$$S_c = 3.15$$

$$S_{fa} = 2.65$$

$$S_{ca} = 2.72$$

For durability, the lower water-cement ratio is adopted.

$$\frac{w}{c} = 0.40$$

Quantity of Cement

$$\text{Cement content} = \frac{172}{0.40}$$

$$\Rightarrow \quad \text{Cement content} = 430 \text{ kg/m}^3$$

Absolute Volumes

Volume of water, $$V_w = \frac{172}{1000}$$

$$\Rightarrow \quad V_w = 0.172 \text{ m}^3$$

Volume of cement, $$V_c = \frac{430}{3.15 \times 1000}$$

$$\Rightarrow \quad V_c = 0.1365 \text{ m}^3$$

Volume of Fine Aggregate

Mortar volume consists of water, cement, and fine aggregate.

$$V_w + V_c + V_{fa} = 0.56$$

$$\Rightarrow \quad 0.172 + 0.1365 + V_{fa} = 0.56$$

$$\Rightarrow \quad V_{fa} = 0.2515 \text{ m}^3$$

Volume of Coarse Aggregate

Since air voids are neglected, total volume is 1 m^3.

$$V_w + V_c + V_{fa} + V_{ca} = 1$$

$$\Rightarrow \quad 0.172 + 0.1365 + 0.2515 + V_{ca} = 1$$

$$\Rightarrow \quad 0.56 + V_{ca} = 1$$

$$\Rightarrow \quad V_{ca} = 0.44 \text{ m}^3$$

Final Quantities

Mass of fine aggregate,

$$M_{fa} = 0.2515 \times 2.65 \times 1000$$

$$M_{fa} = 665.475 \text{ kg/m}^3$$

Mass of coarse aggregate,

$$M_{ca} = 0.44 \times 2.72 \times 1000$$

$$M_{ca} = 1196.8 \text{ kg/m}^3$$

$$\text{Cement content} = 430 \text{ kg/m}^3$$

$$\text{Fine aggregate content} = 665.475 \text{ kg/m}^3$$

$$\text{Coarse aggregate content} = 1196.8 \text{ kg/m}^3$$

3. (c) Solution:

Given data

Number of pairs of driving wheels = 4

$$\text{Axle load} = 24 \text{ tons}$$

$$\text{Initial speed, } V = 90 \text{ kmph}$$

$$\text{Coefficient of friction, } \mu = 0.18$$

$$\text{Rising gradient} = 1 \text{ in } 150$$

$$\text{Degree of curve, } D = 3°$$

Maximum hauling capacity of the locomotive is

$$H = \mu \times (\text{number of driving axles} \times \text{axle load})$$

$$\Rightarrow \quad H = 0.18 \times 4 \times 24$$

$$\Rightarrow \quad H = 17.28 \text{ tons}$$

Let the maximum permissible load be W tons.

Resistance independent of speed is

$$R_{t1} = 0.0016W$$

Resistance dependent on speed is

$$R_{t2} = 0.00008WV$$

$$\Rightarrow \quad R_{t1} = 0.00008 \times W \times 90 = 0.0072W$$

Atmospheric resistance is

$$R_{t3} = 0.0000006WV^2$$

$$\Rightarrow \quad R_{t3} = 0.0000006 \times W \times 90^2 = 0.00486W$$

Total resistance on level track is

$$R = 0.0016W + 0.0072W + 0.00486W$$

$$\Rightarrow \quad R = 0.01366W$$

For maximum load, hauling capacity equals total resistance.

$$17.28 = 0.01366W$$

$$\Rightarrow \quad W = 1265.007 \text{ tons}$$

Reduction in speed on rising gradient

Let the new speed be V'.

Gradient resistance is

$$R_g = W \times \frac{1}{150}$$

$$\Rightarrow \quad R_g = 1265.007 \times \frac{1}{150} = 8.433$$

Total resistance on gradient is

$$17.28 = (0.0016 \times 1265.007) + (0.00008 \times 1265.007 \times V') + (0.0000006 \times 1265.007 \times V'^2) + 8.433$$

$$\Rightarrow \quad 17.28 = 2.024 + 0.1012V' + 0.000759V'^2 + 8.433$$

$$\Rightarrow 0.000759V'^2 + 0.1012V' - 6.823 = 0$$

Solving the quadratic equation,

$$V' = 49.238 \text{ kmph}$$

Reduction in speed is

$$\Delta V_1 = 90 - 49.238$$

$$\Rightarrow \quad \Delta V_1 = 40.762 \text{ kmph}$$

Further reduction in speed on 3° curve

Curve resistance for *B.G.* track is

$$R_c = 0.0004\,WD$$

$$\Rightarrow \quad R_c = 0.0004 \times 1265.007 \times 3 = 1.518 \text{ tons}$$

Total resistance on gradient and curve is

$$17.28 = 10.457 + 0.1012V'' + 0.000759V''^2 + 1.518$$

$$0.000759V''^2 + 0.1012V'' - 5.305 = 0$$

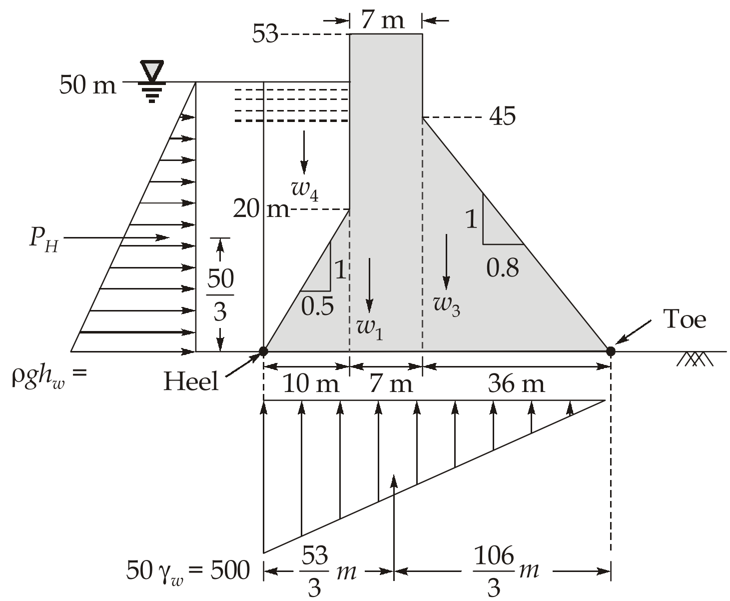
Solving the quadratic equation, $V'' = 40.263$ kmph

Further reduction in speed is

$$\Delta V_2 = 49.238 - 40.263$$

$$\Delta V_2 = 8.975 \text{ kmph}$$

4. (a) Solution:



(All levels and dimensions are in metres)

Given data

$$\gamma_c = 24 \text{ kN/m}^3$$
$$\gamma_w = 10 \text{ kN/m}^3$$
$$\mu = 0.65$$

Load and Moment Calculation Table

S.No.	Load Component	ΣV(kN)	ΣH(kN)	Lever Arm from Toe (m)	Moment about Toe (kN-m)
1	W_1 (US Triangle)	0.5 × 10 × 20 × 24 = 2400	–	36 +7 + 10/3 = 46.333	2400 × 46.333 = 111199.2 (↺)
2	W_2 (Rectangular)	7 × 53 × 24 = 8904	–	36 + 7/2 = 39.5	8904 × 39.5 = 351708 (↺)
3	W_3 (DS Triangle)	0.5 × 36 × 45 × 24 = 19440	–	36 × 2/3 = 24	19440 × 24 = 466560 (↺)
4	W_4 (Water on US)	0.5 × (50 + 30) × 10 × 10 = 4000	–	$36+7+\frac{(30+2\times 50)}{(30+50)}\times\frac{10}{3}=48.42$	4000 × 48.42 = 193680 (↺)
5	P_H (Horiz. Water)	–	0.5 × 10 × 50^2 = 12500	50/3 = 16.667	12500 × 16.667 = 208337.5 (↻)
6	U(Uplift Force)	–(0.5 × 500 × 53) = –13250	–	2/3 × 53 = 35.333	13250 × 35.33 = 468162.25 (↻)
Total		**21494**	**12500**		**Net M: 446647.45 kN-m**

$$\Sigma M_R = \text{Sum of all resisting moment}$$
$$= 111199.2 + 351708 + 466560 + 193680 = 1123147.2 \text{ kN-m}$$
$$\Sigma M_O = \text{Sum of all overturning moment}$$
$$= 208337.5 + 468162.25 = 676499.75 \text{ kN-m}$$

$$M_{net} = 1123147.2 - 676499.75 = 446647.45 \text{ kN-m}$$

Factor of Safety against Overturning

$$FOS_O = \frac{\Sigma M_R}{\Sigma M_O} = \frac{1123147.2}{676499.75}$$

$$\Rightarrow \quad FOS_O = 1.66$$

Factor of Safety against Sliding

$$FOS_S = \frac{\mu \Sigma V}{\Sigma H}$$

$$\Rightarrow \quad FOS_S = \frac{0.65 \times 21494}{12500}$$

$$\Rightarrow \quad FOS_S = 1.12$$

Stresses at the Toe

First, determine the location of the resultant from the toe.

$$x = \frac{\Sigma M}{\Sigma V} = \frac{446647.45}{21494} = 20.78 \text{ m}$$

Eccentricity, $$e = \frac{B}{2} - x = \frac{53}{2} - 20.78 = 5.72 \text{ m}$$

$$\because \quad e < \left(\frac{B}{6} = 8.83 \text{ m}\right)$$

So, No tension occur in base of dam.

Vertical stress at the toe,

$$\sigma_v = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right) = \frac{21494}{53}\left(1 + \frac{6 \times 5.72}{53}\right)$$

$$\Rightarrow \quad \sigma_v = 668.16 \text{ kN/m}^2$$

Principal stress, $$\sigma = \sigma_v \sec^2\theta = 668.16 \times (1 + 0.8^2)$$

$$\sigma = 1095.78 \text{ kN/m}^2$$

Maximum shear stress, $$\tau_{max} = \sigma_v \tan\theta = 668.16 \times 0.8$$

$$\tau_{max} = 534.53 \text{ kN/m}^2$$

4. (b) (i) Solution:

A good building stone must possess specific physical, mechanical, and chemical properties to ensure durability, strength, and economy in construction. The following are the key characteristics:

1. **Appearance :** For face work, the stone should have a fine, compact texture and a uniform color. Light-colored stones are generally preferred as dark shades tend to fade with time. The stone should also be free from visible stratifications and undesirable patches.
2. **Structure :** A good stone should be uniform in texture and free from cavities, cracks, loose materials, or patches of soft or decomposed matter. A broken surface should appear crystalline and dense, without laminated or foliated planes.
3. **Strength :** The compressive strength of a good building stone should generally range from 60 to 200 N/mm^2. Stones should resist crushing, weathering, and external loads, especially in structural applications like load-bearing walls and dams.
4. **Weight :** The weight of a stone is an indicator of its density and porosity. For structural stability, heavier stones are preferred in retaining walls and dams, whereas lighter stones may be used in arches, domes, and vaults.
5. **Hardness :** Stones used for pavements, aprons of bridges, and flooring must be hard enough to resist abrasion and wear. Hardness is usually measured using the Mohs scale.
6. **Toughness :** Stones should have adequate toughness to resist impact and vibrations, especially in road work. It is determined by the amount of energy the stone can absorb before failure.
7. **Porosity and Water Absorption :** Stones should be less porous to prevent damage due to freezing and thawing. The permissible water absorption by volume is:

   Sandstone, Limestone, Shale – up to 10%

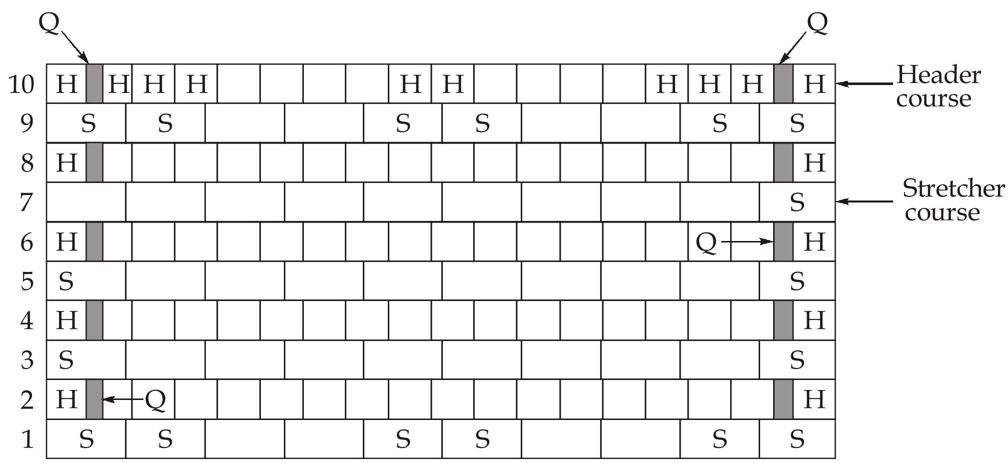
   Granite, Trap, Gneiss, Slate – up to 1%

   Quartzite – up to 3%
8. **Fire Resistance :** Good building stones should withstand high temperatures. Some stones like granite can resist fire up to 800°C, while limestone disintegrates above 575°C.
9. **Specific Gravity :** The specific gravity of building stones should lie between 2.4 to 2.8, indicating a dense and strong material.
10. **Durability and Weathering Resistance :** Stones should be able to resist the action of weather, moisture, and temperature changes. Stones from exposed faces and upper beds are usually less durable. Hence, durability is critical in external applications.
11. **Workability:** Stones should be workable so they can be easily cut, shaped, and dressed to required sizes without much wastage. However, excessive softness may reduce durability.
12. **Thermal Stability :** Stones should withstand minor thermal movements without cracking or spalling. Stones with homogeneous texture and stable mineral composition offer better thermal performance.

4. (b) (ii) Solution:

(I) English Bond :

- This bond consists of alternate courses of headers and stretchers.
- In this arrangement, vertical joints in the header courses come over each other and the vertical joints in the stretcher courses are also in the same vertical line.
- For breaking of vertical joints in the successive courses, it is essential to place queen closer, after the first header in each header course.



S = Stretcher : H = Header : Q = Queen closer

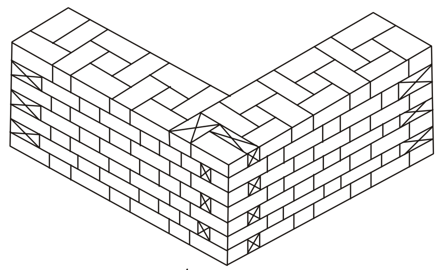
ENGLISH BOND

- The following additional points should be noted in English bond construction:
  1. A header course should never start with a queen closer as it is liable to get displaced from this position.
  2. In the stretcher course, the stretchers should have a minimum lap of $1/4^{th}$ of their length over the headers.
  3. In walls having thickness equal to an odd number of half brick, i.e., $1\frac{1}{2}$ brick thick walls or $2\frac{1}{2}$ brick thick walls and so on, the same course will show stretchers on one face and headers on the other.

(II) Flemish Bond

- In this arrangement of bonding brick work, each course consists of alternate headers and stretchers.

- The alternate headers of each course are centered over the stretchers in the course below.



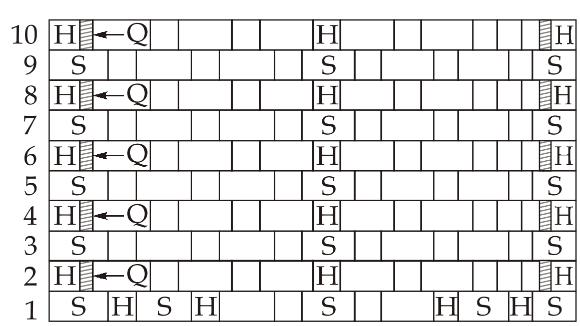
$1\frac{1}{2}$ Brick wall

Fig. *Flemish Bond*

- Every alternate course starts with a header at the corner.
- For breaking the vertical joints in the successive courses, closers are inserted in alternate courses next to the quoin header.
- In walls having thickness equal to odd number of half bricks, bats are essentially used to achieve the bond.
- Flemish bond is further divided into two different types viz. Single Flemish bond and Double Flemish bond.

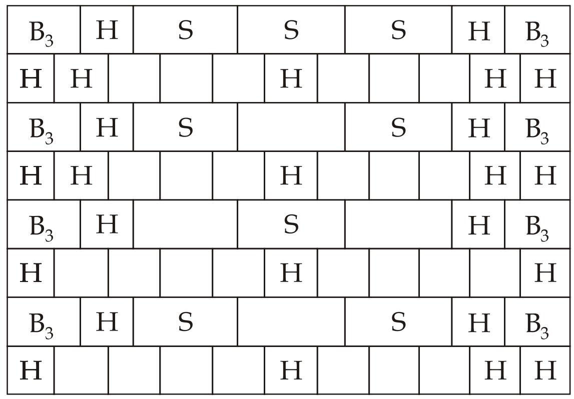
(III) Double Flemish Bond

- In this system of bonding brick work, each course presents the same appearance both in the front and back elevations.
- Every course consists of headers and stretchers laid alternately.
- This type of bond is best suited from considerations of economy and appearance.
- It enables the one brick thick wall to have flush and uniform faces on both the sides.



DOUBLE FLEMISH BOND (ELEVATION)

(III) Dutch bond: This bond is a modification of the old English cross bond and consists of alternate courses of headers and stretchers.



DUTCH BOND ($B_3 \rightarrow 3/4$ Bat)

4. (c) (i) Solution:

Hogging in rails: Rail ends gets hogged due to poor maintenance of rail joint, yield formation loose and faulty fastening and other such reason. Due to loose joint, impart of wheel on rail ends result into bend in rail end and is known as hogging.

Composite sleeper index: The CSI evolved from a combination of properties of strength and hardness, is an index used to determine suitability of a particular timber for use as sleeper from point of view of mechanical strength.

$$\text{CSI} = \frac{S + 10H}{20}$$

where,

S = Strength index defined at 12% moisture content for both green and dry timber.

H = Hardness index at 12% moisture content for both green and dry timber.

Type of timber	Minimum CSI
Track sleeper	783
Crossing sleeper	1352
Bridge sleeper	1455

4. (c) (ii) Solution:

Given data

Basic runway length, $L = 2500$ m

Elevation of airport site, $h = 750$ m

Monthly mean of average daily temperature, $T_a = 20°C$

Monthly mean of maximum daily temperature, $T_m = 29.3°C$

Effective gradient, $g = 0.8\%$

Correction for elevation as recommended by ICAO is 7% per 300 m of elevation.

$$\text{Correction} = \frac{7}{100} \times \frac{750}{300} \times 2500$$

$$\text{Correction} = 437.5 \text{ m}$$

Corrected length,

$$L' = 2500 + 437.5 = 2937.5 \text{ m}$$

Standard atmospheric temperature at given elevation,

$$T_s = 15 - 0.0065 \times 750$$

$$T_s = 10.125°C$$

Airport reference temperature,

$$T_R = T_a + \frac{T_m - T_a}{3}$$

$$\Rightarrow \quad T_R = 20 + \frac{29.3 - 20}{3}$$

$$\Rightarrow \quad T_R = 23.1°C$$

$$\text{Rise in temperature} = 23.1 - 10.125 = 12.975°C$$

As per ICAO, the correction for temperature is 1% for every 1°C rise in temperature.

$$\text{Correction} = \frac{1}{100} \times 12.975 \times 2937.5$$

$$\text{Correction} = 381.141 \text{ m}$$

$$\text{Corrected length, } L'' = 2937.5 + 381.141 = 3318.641 \text{ m}$$

Total correction for elevation and temperature,

$$= 437.5 + 381.141 = 818.641 \text{ m}$$

Total correction in percentage,

$$= \frac{818.641}{2500} \times 100 = 32.746\%$$

Since 32.746% < 35%, the correction is within permissible limit.

FAA recommends that runway length after being corrected for elevation and temperature should be further increased at the rate of 20% for every 1% of effective gradient.

$$\text{Correction of gradient} = \frac{20}{100} \times 0.8 \times 3318.641$$

$$\text{Correction} = 530.983 \text{ m}$$

$$\text{Final runway length, } L_f = 3318.641 + 530.983$$

$$= 3849.624 \text{ m}$$

$$\text{Actual runway length} = 3849.624 \text{ m.}$$

Section B : Design of Steel Structure-2 + Hydrology-2

5. (a) Solution:

Factors affecting the strength of a compression member

The strength of a compression member such as a column or strut depends on several parameters which influence its ability to resist compressive load without buckling or crushing.

1. **Length of the member:** As the length of the compression member increases, the tendency to buckle also increases. Long members fail mainly due to buckling, while short members fail by crushing.
2. **Slenderness ratio:** The slenderness ratio is the ratio of effective length to the radius of gyration $\frac{L_e}{r}$. A higher slenderness ratio reduces the load carrying capacity of the member and increases the probability of buckling.
3. **Cross-sectional area and shape:** The load carrying capacity depends on the area of the cross section. Shapes with higher radius of gyration such as I-sections, box sections and circular sections provide better resistance to buckling.
4. **End conditions of the member:** The effective length of the compression member depends on the type of end supports such as hinged, fixed or free. Different end conditions change the effective length and therefore affect the buckling strength.
5. **Material properties:** Mechanical properties of the material such as yield strength and modulus of elasticity influence the compressive strength and buckling resistance.
6. **Eccentric loading:** If the compressive load does not act through the centroid of the section, bending stresses are produced along with compressive stresses, reducing the strength of the member.
7. **Initial imperfections and residual stresses:** Manufacturing defects, residual stresses and slight curvature in the member reduce its effective strength.

8. **Lateral support or bracing:** Providing lateral supports reduces the effective length of the member and increases its load carrying capacity.

Reason why the design of a compression member is not a direct method:

The design of a compression member is not a direct method because the load carrying capacity cannot be determined by a single simple formula. The strength depends on several factors such as slenderness ratio, effective length, cross-section and end conditions.

Compression members may fail either by crushing or by buckling, and the critical stress varies with the slenderness ratio. Therefore, the designer must first assume a trial section, calculate its slenderness ratio and then determine the design compressive stress from design tables or curves. If the section is not adequate, another section must be selected and checked again. Since this process involves trial selection and repeated checking, the design of compression members is considered an iterative or indirect method rather than a direct method.

5. **(b) Solution:**

A plastic hinge is a region in a structural member where the bending moment reaches the plastic moment capacity of the section and the entire cross section yields. At this stage, large rotations occur without any increase in bending moment. The section behaves like a hinge but is capable of carrying a constant plastic moment. Plastic hinges are the basis of plastic analysis of beams and frames.

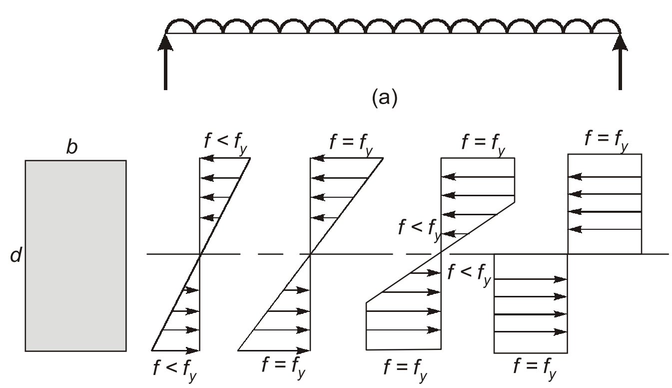


Fig. : Plastic moment capacity of a beam section

The shape factor is defined as the ratio of plastic moment capacity to yield moment of a section.

$$\text{Shape factor} = \frac{M_p}{M_y}$$

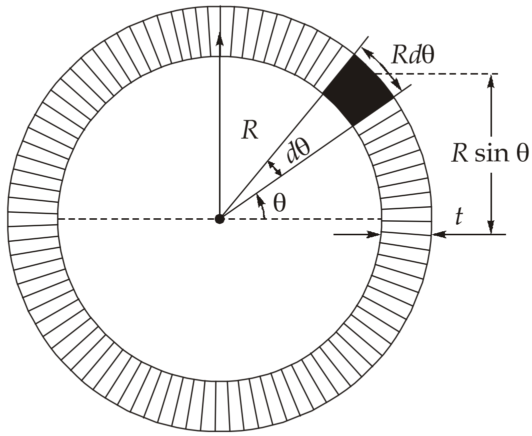
where M_p is the plastic moment capacity and M_y is the yield moment capacity.

For a thin circular tube, the wall thickness is very small compared to its radius. Therefore, the entire area of the tube can be considered concentrated along the circumference.

Area of the thin tube

$$A = 2\pi Rt$$

For bending about a diameter, half the area is in compression and half in tension.



Plastic section modulus

$$Z_p = \int y\, dA$$

Consider a small element at angle θ.

$$y = R\sin\theta$$

$$dA = tR\, d\theta$$

Plastic section modulus for half the section

$$Z_p = 4\int_0^{\pi/2} (R\sin\theta)(tR\, d\theta)$$

$$\Rightarrow \quad Z_p = 4tR^2 \int_0^{\pi/2} \sin\theta\, d\theta$$

$$\Rightarrow \quad Z_p = 4tR^2 [-\cos\theta]_0^{\pi/2}$$

$$\Rightarrow \quad Z_p = 4tR^2(1) = 4tR^2$$

Plastic moment capacity,

$$M_p = \sigma_y Z_p$$

$$M_p = \sigma_y(4tR^2)$$

Now determine the elastic section modulus.

Moment of inertia of a thin circular tube about a diameter

$$I = \frac{\pi}{4}\left[R^4 - (R-t)^4\right]$$

$$\Rightarrow \quad I = \frac{\pi}{4}R^4\left[1 - \left(1 - \frac{t}{R}\right)^4\right]$$

$$\Rightarrow \quad I = \frac{\pi}{4}R^4\left[1 - 1 + \frac{4t}{R}\right] \qquad \because \frac{t}{R} <<< 1$$

$$\Rightarrow \quad I = \pi R^3 t$$

Elastic section modulus

$$Z_e = \frac{I}{R}$$

$$\Rightarrow \quad Z_e = \frac{\pi R^3 t}{R}$$

$$\Rightarrow \quad Z_e = \pi R^2 t$$

Yield moment capacity,

$$M_y = \sigma_y Z_e$$

$$M_y = \sigma_y(\pi R^2 t)$$

Shape factor,

$$\text{Shape factor} = \frac{M_p}{M_y} = \frac{4\sigma_y t R^2}{\pi \sigma_y R^2 t}$$

$$\Rightarrow \quad \text{Shape factor} = \frac{4}{\pi} \approx 1.27$$

5. (c) Solution:

Given data

$$(L_{eff})_{zz} = (KL)_{zz} = 6 \text{ m} = 6000 \text{ mm}$$

$$(L_{eff})_{yy} = (KL)_{yy} = 3 \text{ m} = 3000 \text{ mm}$$

$$f_y = 250 \text{ MPa}$$

$$D = 500 \text{ mm}$$

$$B = 200 \text{ mm}$$

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

Gross area of section, $A_g = 2 \times 200 \times 20 + 2 \times 460 \times 20$

$$\Rightarrow \quad A_g = 26400 \text{ mm}^2$$

Moment of inertia about major axis

$$I_{zz} = 2\times\frac{20\times 460^3}{12}+2\left[\frac{200\times 20^3}{12}+200\times 20\times 240^2\right]$$

$\Rightarrow$ $$I_{zz} = 785520000 \text{ mm}^4$$

Moment of inertia about $(y - y)$ axis

$$I_{yy} = 2\times\frac{20\times 20^3}{12}+2\left[\frac{460\times 20^3}{12}+460\times 20\times 85^2\right]$$

$$I_{yy} = 160220000 \text{ mm}^4$$

$\because$ $$I_{zz} > I_{yy}$$

So, z-z axis is major axis

Radius of gyration about z-z axis,

$$r_{zz} = \sqrt{\frac{785520000}{26400}} = 172.495 \text{ mm}$$

Radius of gyration about y-y axis,

$$r_{yy} = \sqrt{\frac{160220000}{26400}} = 77.903 \text{ mm}$$

Slenderness Ratio Calculation

About major axis, $$\lambda_{zz} = \left(\frac{KL}{r}\right)_{zz} = \frac{6000}{172.495} = 34.784$$

About minor axis, $$\lambda_{yy} = \left(\frac{KL}{r}\right)_{yy} = \frac{3000}{77.903} = 38.509$$

$\therefore$ Minimum, $$\lambda = 36.709$$

Safe load calculation about z-z axis:

$$\lambda_{zz} = 34.784$$

From given table, $$(f_{cd})_{zz} = 211+\frac{(198-211)}{(40-30)}\times(34.784-30)$$

$$= 204.78 \text{ N/mm}^2$$

$\therefore$ $$(P_u)_{zz} = A\times(f_{cd})_{zz} = 204.78\times 26400 = 5406.192 \text{ kN}$$

$$(P_{safe})_{zz} = 3604.128 \text{ kN}$$

Safe load calculation about y-y axis:

$$\lambda_{yy} = 38.509$$

From given table, $$(f_{cd})_{yy} = 211+\frac{(198-211)}{(40-30)}\times(38.509-30)$$

$$= 199.938 \text{ N/mm}^2$$

$\therefore$ $$(P_u)_{yy} = A \times (f_{cd})_{yy} = 199.938 \times 26400 = 5278.363 \text{ kN}$$

$$(P_{\text{safe}})_{yy} = 3518.91 \text{ kN}$$

Hence, safe load, $$P_{\text{safe}} = 3518.91 \text{ kN}$$

5. (d) Solution:

Given:

Storage equation is as under:

$$S = K[xI + (1-x)O] \quad \text{...(i)}$$

Also, the basic routing equation is

$$\left(\frac{I_1+I_2}{2}\right)t-\left(\frac{O_1+O_2}{2}\right)t = (S_2-S_1) \quad \text{...(ii)}$$

Substituting the values of S_1 and S_2 from (i) into (ii), we have

$$S_1 = K\left[xI_1+(1-x)O_1\right]$$

$$S_2 = K\left[xI_2+(1-x)O_2\right]$$

$$\therefore \quad \left(\frac{I_1+I_2}{2}\right)t-\left(\frac{O_1+O_2}{2}\right)t = K\left[xI_2+(1-x)O_2\right]-K\left[xI_1+(1-x)O_1\right]$$

$$\Rightarrow \left(\frac{I_1+I_2}{2}\right)t+K\left[xI_1+(1-x)O_1\right] = \left(\frac{O_1+O_2}{2}\right)t+K\left[xI_2+(1-x)O_2\right]$$

$$\Rightarrow (I_1+I_2)+\frac{2K}{t}\left[xI_1+(1-x)O_1\right] = (O_1+O_2)+\frac{2K}{t}\left[xI_2+(1-x)O_2\right]$$

$$\Rightarrow \left[I_1+\frac{KxI_1}{0.5t}\right]+\left[I_2-\frac{KxI_2}{0.5t}\right]+\left[\frac{K(1-x)O_1}{0.5t}-O_1\right]=\left[O_2+\frac{K(1-x)}{0.5t}O_2\right]$$

$$\Rightarrow O_2\left[\frac{0.5t+K(1-x)}{0.5t}\right]=I_1\left[\frac{0.5t+Kx}{0.5t}\right]+I_2\left[\frac{0.5t-Kx}{0.5t}\right]+O_1\left[\frac{K(1-x)-0.5t}{0.5t}\right]$$

$$\Rightarrow O_2(K-Kx+0.5t)=I_1(Kx+0.5t)+I_2(0.5t-Kx)+O_1(K-Kx-0.5t)$$

$$\Rightarrow O_2 = I_1\left[\frac{Kx+0.5t}{K-Kx+0.5t}\right]+I_2\left[\frac{-Kx+0.5t}{K-Kx+0.5t}\right]+O_1\left[\frac{K-Kx-0.5t}{K-Kx+0.5t}\right]$$

$$\Rightarrow \qquad O_2 = C_0I_2 + C_1I_1 + C_2O_1$$

which is the required Muskingum equation, where

$$C_0 = \frac{-Kx+0.5t}{K-Kx+0.5t},\ C_1 = \frac{Kx+0.5t}{K-Kx+0.5t} \text{ and } C_2 = \frac{K-Kx-0.5t}{K-Kx+0.5t}$$

Sum of coefficients,

$$C_0 + C_1 + C_2 = \frac{1}{K-Kx+0.5t}[-Kx+0.5t+Kx+0.5t+K-Kx-0.5t] = 1$$

5. (e) Solution:

Total number of occurrences,

$$N = 35 + 240 + 310 + 420 + 320 + 210 + 165 + 85 + 35 + 10 = 1830$$

Flow Duration Analysis Table

Arrange data in descending order of discharge and cumulative frequency m and corresponding probability P using Hazen method.

$$P\% = \left(\frac{m-0.5}{N}\right)\times 100$$

where m is cumulative frequency.

Mean Discharge (Q_m)	Frequency	Cumulative Frequency (m)	$P\% = \frac{m-0.5}{N} \times 100$
> 450.0	10	10	0.519
374.95	35	45	2.432
249.95	85	130	7.076
162.45	165	295	16.092
97.45	210	505	27.568
57.45	320	825	45.055
37.45	420	1245	68.005
24.95	310	1555	84.945
14.95	240	1795	98.060
< 10.0	35	1830	99.973

Calculation of Dependable Discharges

For 90% Dependable Flow:

The 90% value lies between Q = 24.95 (at P = 84.945%) and Q = 14.95 (at P = 98.060%).

Using linear interpolation:

$$Q_{90\%} = 24.95 + \frac{(14.95 - 24.95)}{(98.060 - 84.945)} \times (90 - 84.945)$$

$$\Rightarrow \qquad Q_{90\%} = 21.1 \text{ m}^3/\text{s}$$

For 65% Dependable Flow:

The 65% value lies between Q = 57.45 (at P = 45.055%) and Q = 37.45 (at P = 68.005%).

Using linear interpolation:

$$Q_{65\%} = 57.45 + \frac{(37.45 - 57.45)}{(68.005 - 45.055)} \times (65 - 45.055)$$

$$\Rightarrow \qquad Q_{65\%} = 40.07 \text{ m}^3/\text{s}$$

6. (a) (i) Solution:

Assumptions of unit hydrograph theory: The various assumptions of unit hydrograph theory are as follows:

1. The effective rainfall is uniformly distributed within its duration of specified period of time.
2. The effective rainfall is uniformly distributed throughout the whole area of the drainage basin.
3. The base or time duration of the hydrograph of direct run-off due to an effective rainfall of unit duration is constant.
4. The ordinates of direct run-off of common base-time are directly proportional to the total amount of direct run-off represented by each hydrograph.
5. For a given drainage basin, the hydrograph of run-off due to a given period of rainfall reflects all the combined physical characteristics of the basin.

6. (a) (ii) Solution:

Given:

$$\text{Duration of rainfall excess} = 3 \text{ hr}$$

$$\text{Infiltration loss for 3 hr} = 0.3 \times 3 = 0.9 \text{ cm}$$

$$\text{Total depth of rainfall} = 5.9 \text{ cm}$$

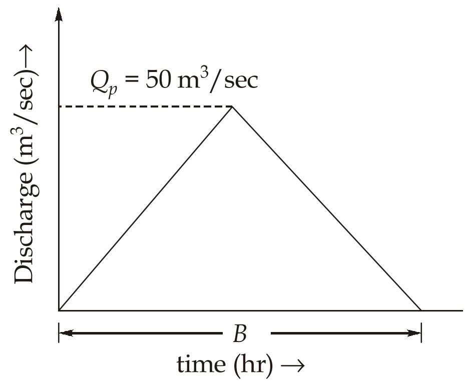
$$\therefore \qquad \text{Runoff depth} = 5.9 - 0.9 = 5.0 \text{ cm}$$

Now,

$$\text{Peak of flood hydrograph} = 270 \text{ m}^3/\text{sec}$$

$$\text{Peak of DRH} = 270 - \text{Base flow} = 270 - 20 = 250 \text{ m}^3/\text{sec}$$

$$\therefore \qquad \text{Peak of 3 hr UH} = \frac{\text{Peak of DRH}}{\text{Runoff depth}} = \frac{250}{5} = 50 \text{ m}^3/\text{sec}$$



$$\text{Volume} = 576\times10^6\times\frac{1}{100} = \frac{1}{2}B(50)\times3600$$

$$\Rightarrow \quad B = 64 \text{ hr}$$

6. (a) (iii) Solution:

Time in hr	3 hr unit hydrograph ordinates (cumecs)	Imaginary off-setted S-curve (shifted by $t_1 = 3$ hr) cumecs	S-curve ordinates cumecs	S-curve lagged by 4 hours	Difference (4)-(5)	Required ordinates of 4 hr U.H = (6) × t_1/t_2 = 0.75 × col(6)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
0	0	–	0		0	0
1	12	–	12		12	9
2	45	–	45		45	33.75
3	80	0	80		80	60
4	105	12	117	0	117	87.75
5	90	45	135	12	123	92.25
6	60	80	140	45	95	71.25
7	35	117	152	80	72	54
8	15	135	150	117	33	24.75
9	0	140	140	135	5	3.75
10	0	152	152	140	12	9
11	0	150	150	152	–2	0
12	0	152	152	150	2	0

6. (b) Solution:

$$\text{For Fe410 steel, } f_u = 410 \text{ MPa}, f_y = 250 \text{ MPa}$$

$$\text{For 4.6 grade bolts, } f_{ub} = 400 \text{ MPa}, f_y = 0.6 \times 400 = 240 \text{ MPa}$$

$$A_{nb} = \text{Stress area of bolts}$$

$$= 0.78 \times \frac{\pi}{4} \times d^2 = 0.78 \times \frac{\pi}{4} \times 20^2 = 245 \text{ mm}^2$$

$$\text{Diameter of bolt hole, } d_0 = 20 + 2 = 22 \text{ mm}$$

$$\gamma_{mb} = \text{Partial factor of safety of bolt material} = 1.25$$

$\because$ Bolts are in single shear and strength of bolt in single shear,

$$V_{sb} = A_{nb} \times \frac{f_{ub}}{\sqrt{3}\gamma_{mb}} \times 10^{-3} = 245 \times \frac{400}{\sqrt{3} \times 1.25} \times 10^{-3} = 45.26 \text{ kN}$$

$$\text{Strength of bolt in bearing, } V_{pd} = 2.5 k_b d t \frac{f_u}{\gamma_{mb}} \qquad (f_u = \text{minimum of } f_u \text{ and } f_{ub})$$

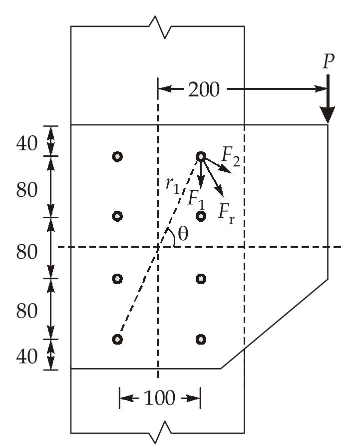
Taking, $k_b = 0.606$ (given)

$$V_{pd} = 2.5 \times 0.606 \times 20 \times 9.1 \times \frac{400}{1.25} \times 10^{-3}$$

$$= 88.23 \text{ kN}$$

So, Bolt value $=$ Minimum of V_{sb} and $V_{pb} = 45.26$ kN

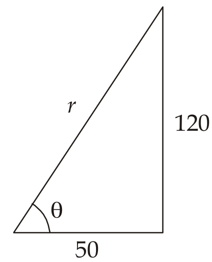
Critical bolt is top-right bolt,



(All dimensions are in mm)

$$\text{Direct force in bolt, } F_1 = \frac{P'}{n} = \frac{P'}{8}$$

$$\text{Force in bolt due to torque, } F_2 = \frac{P'er}{\Sigma r^2}$$



$$r = \sqrt{50^2 + 120^2} = 130\,\text{mm}$$

$$\Sigma r^2 = 4 \times [(50^2 + 40^2)] + [(50^2 + 120^2)]$$
$$= 84000\ \text{mm}^2$$

$$\cos\theta = \frac{50}{130} = 0.3846$$

$$\Rightarrow \quad F_2 = \frac{P' \times 200 \times 130}{84000} = \frac{P'}{3.23}$$

Resultant force on bolt, $F_R = \sqrt{F_1^2 + F_2^2 + 2F_1F_2\cos\theta}$

$$= P'\sqrt{\frac{1}{8^2} + \frac{1}{3.23^2} + 2 \times \frac{1}{8} \times \frac{1}{3.23} \times 0.3846}$$

$$= 0.376\,P'$$

Now, Resultant force $\leq$ Bolt value

$\therefore$ $0.376\,P' = 45.26$ kN

$\therefore$ Factored Load, $P' = 120.37$ kN

Service load, $P = \dfrac{P'}{1.5} = \dfrac{120.37}{1.5} = 80.25\,\text{kN} \simeq 80\,\text{kN}$

6. (c) Solution:

- **For section A:**

$$y_1 = 4.5\ \text{m},$$
$$B_1 = 10\ \text{m}$$

$\therefore$ Area $(A_1) = y_1 \times B_1 = 4.5 \times 10 = 45\ \text{m}^2$

and Perimeter $(P_1) = B_1 + 2y_1 = 10 + 2 \times 4.5 = 19$ m

Now, hydraulic mean radius,

$$R_1 = \frac{A_1}{P_1} = \frac{45}{19} = 2.37\ \text{m}$$

and conveyance,

$$K_1 = \frac{1}{n} \times A_1 R_1^{2/3}$$

$$= \frac{1}{0.02} \times 2.37^{2/3} \times 45 = 3999.58$$

- **For section B:**

$$y_2 = 3.2 \text{ m},$$

$$B_2 = 10 \text{ m}$$

$\therefore$ Area $(A_2) = y_2 \times B_2 = 3.2 \times 10 = 32 \text{ m}^2$

and Perimeter $(P_2) = 2y_2 + B_2 = 2 \times 3.2 + 10 = 16.4$ m

Now, hydraulic mean radius,

$$R_2 = \frac{A_2}{P_2} = \frac{32}{16.4} = 1.95 \text{ m}$$

and conveyance,

$$K_2 = \frac{1}{n} \times A_2 R_2^{2/3}$$

$$= \frac{1}{0.02} \times 1.95^{2/3} \times 32 = 2497.33$$

Average conveyance is given by,

$$K_{avg.} = \sqrt{K_1 \cdot K_2} = \sqrt{3999.58 \times 2497.32} = 3160.42 \text{ m}$$

1. Ist Iteration:

Assuming, $V_1 = V_2$

$$\therefore \quad h_f = (h_1 - h_2) + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) - K\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$$

$$\Rightarrow \quad h_f = (104.771 - 103.852) = 0.919 \text{ m}$$

Now, $$Q = K_{avg}\sqrt{\frac{h_f}{L}} = 3160.42\sqrt{\frac{0.919}{5000}} = 42.847 \text{ m}^3/\text{sec}$$

$$\therefore \quad V_1 = \frac{Q}{A_1} = \frac{42.846}{45} = 0.952 \text{ m/sec}$$

$$V_2 = \frac{Q}{A_2} = \frac{42.847}{32} = 1.34 \text{ m/sec}$$

2. 2st Iteration:

Take $K = 0.1$ for gradual contraction

$$\therefore \quad h_f = (h_1 - h_2) + \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) - K\left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$$

$$= (104.771 - 103.852) + \left(\frac{0.952^2}{2 \times 9.81} - \frac{1.34^2}{2 \times 9.81}\right)$$

$$-0.1\left(\frac{0.952^2}{2 \times 9.81} - \frac{1.34^2}{2 \times 9.81}\right)$$

$$\Rightarrow \quad h_f = 0.878 \text{ m}$$

Now,

$$Q = K_{avg}\sqrt{\frac{h_f}{L}} = 3160.42\sqrt{\frac{0.878}{5000}} = 41.88 \text{ m}^3/\text{sec}$$

$$\therefore \quad V_1 = \frac{Q}{A_1} = \frac{41.88}{45} = 0.9307 \text{ m/sec}$$

$$V_2 = \frac{Q}{A_2} = \frac{41.88}{32} = 1.3087 \text{ m/sec}$$

3. 3rd Iteration:

$$h_f = (104.771 - 103.852) + \left(\frac{(0.9307)^2}{2 \times 9.81} - \frac{(1.3087)^2}{2 \times 9.81}\right)$$

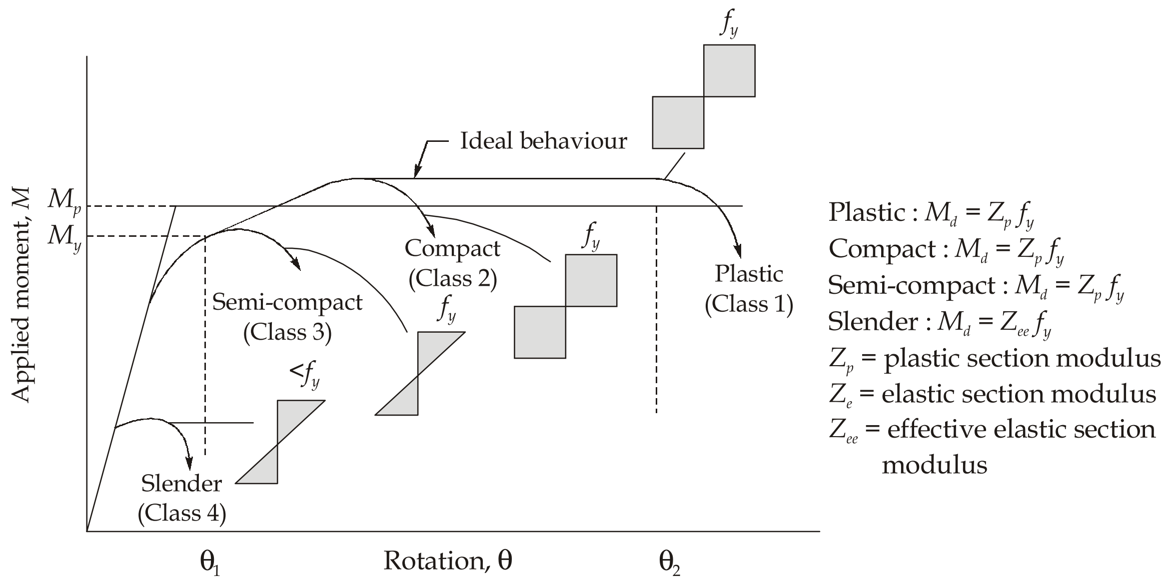
$$-0.1 \times \left(\frac{(0.9307)^2}{2 \times 9.81} - \frac{(1.3087)^2}{2 \times 9.81}\right) = 0.8802 \text{ m}$$

$$\therefore \quad Q = K_{avg}\sqrt{\frac{h_f}{L}} = 3160.42\sqrt{\frac{0.8802}{5000}} = 41.93 \text{ m}^3/\text{sec}$$

7. (a) (i) Solution:

Table: Classification of cross-sections and their characteristics

S.No.	Types of Sections	Characteristics
1.	Plastic (Class 1)	Cross-sections, which can develop plastic hinges and have the rotation capacity required for failure fo the structure by formation of a plastic mechanism, are called plastic sections. Only these sections are used in plastic analysis and design. Such a stocky section will exhibit considerable ductility. Further, these are the only sections used in indeterminate frames forming plastic collapse mechanism. The stress distribution for these section is rectangular.
2.	Compact (Class 2)	Cross-sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism before buckling are classed as compact sections. These cross-sections may develop fully plastic stress distribution across the entire cross-section but do not have adequate ductility. These can be used for all the structural elements. The stress distribution for these sections is rectangular.
3.	Semi-compact (Class 3)	Cross-sections, in which the extreme fibre in compression can reach yield stress (assuming an elastic distribution of stress), but cannot develop the plastic moment of resistance due to local buckling are called semi-compact or non-compact sections. In a semi-compact section the yield stress can be reached only in some parts of compression elements before buckling occurs. It is not capable of reaching a fully plastic stress distribution. These sections are used in elastic design. The stress distribution for such sections is triangular.
4.	Slender (Class 4)	Cross-sections in which the elements buckle locally even before attainment of yield stress are classed as slender sections. These sections are used in cold-formed members. In such cases, the effective sections for design should be calculated by deducting width of compression plate element in excess of the semi-compact section limit.



Moment-rotation behaviour of the four classes of cross-section

7. (a) (ii) Solution:

For ISMC 350

$$A_g = 5366 \text{ mm}^2$$

Two channels are used, $A_{\text{provided}} = 2 \times 5366 = 10732 \text{ mm}^2$

Effective length, $kL = 6 \text{ m}$

$$r_z = 136.6 \text{ mm}$$
$$I_{zz} = 10008 \times 10^4 \text{ mm}^4$$
$$r_y = 28.3 \text{ mm}$$
$$I_{yy} = 430.6 \times 10^4 \text{ mm}^4$$
$$C_{yy} = 24.4 \text{ mm}$$

Effective slenderness ratio $= \dfrac{kL}{r_z} = \dfrac{1.1 \times 6 \times 10^3}{136.6} = 48.32$

For effective slenderness ratio of 48.32 and buckling class c, the design compressive stress is given by,

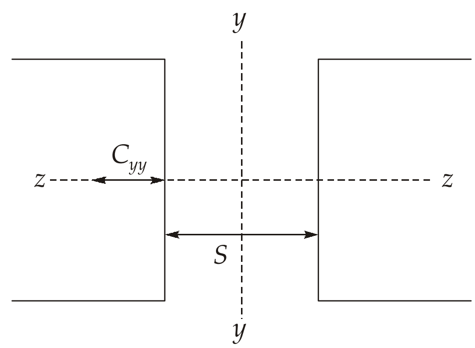
$$f_{cd} = 198 - \frac{(198 - 183)}{(50 - 40)} \times (48.32 - 40)$$
$$= 185.52 \text{ N/mm}^2$$

Design compressive strength, $P_d = f_{cd} \times A_{\text{provided}}$

$$= 185.52 \times 10732 \times 10^{-3} \text{ kN}$$
$$= 1991.00 \text{ kN} > 1800 \text{ kN}$$

Which is safe.

Spacing of channels:



$$2I_{zz} = 2\left[I_{yy} + A\left(C_{yy} + \frac{S}{2}\right)^2\right]$$

$$\Rightarrow \quad 2 \times 10008 \times 10^4 = 2\left[430.6 \times 10^4 + 5366\left(24.4 + \frac{S}{2}\right)^2\right]$$

$$\Rightarrow \quad S = 218.4 \simeq 220 \text{ mm}$$

Provide the 2 ISMC 350 at a spacing of 220 mm back to back.

Spacing of battens:

$$\frac{C}{r_y} < 0.7 \times \text{slenderness ratio of column as a whole}$$

$$\Rightarrow \quad C < 0.7 \times 48.32 \times 28.3$$

$$\Rightarrow \quad C < 957.22 \text{ mm}$$

Also,
$$\frac{C}{r_y} < 50$$

$$\Rightarrow \quad C < 50 \times 28.3$$

$$\Rightarrow \quad C < 1415 \text{ mm}$$

Provide battens at a spacing of 950 mm

Size of end battens:

$$\begin{aligned}\text{Overall depth of the batten} &= \text{Effective depth of the batten}\\ &= S + 2C_{yy}\\ &= 220 + 2(24.4)\\ &= 268.8 \text{ mm} \simeq 270 \text{ mm} \nless 2b_f (= 200 \text{ mm}) \qquad \text{(O.K.)}\end{aligned}$$

$$\text{Thickness of batten} = \frac{S}{50} = \frac{220}{50} = 4.4 \text{ mm} \simeq 8 \text{ mm (Say)}$$

Provide a 70 mm overlap of battens on channel flange.

For welding (> 4 × 8 = 32 mm) (O.K.)

Length of batten = 220 + 2 × 70 = 360 mm

Provide 360 × 270 × 8 mm end batten plates.

Design forces:

Transverse shear, V_t = 2.5% of axial load

$$= \frac{2.5}{100} \times 1800 \text{ kN} = 45 \text{ kN}$$

Longitudinal shear $V_b = \frac{V_t C}{NS} = \frac{45 \times 950}{2 \times \left(220 + 2 \times \frac{70}{2}\right)} = 73.71 \text{ kN}$

Moment, $M = \frac{V_t C}{2N} = \frac{45 \times 950 \times 10^{-3}}{2 \times 2} = 10.69 \text{ kNm}$

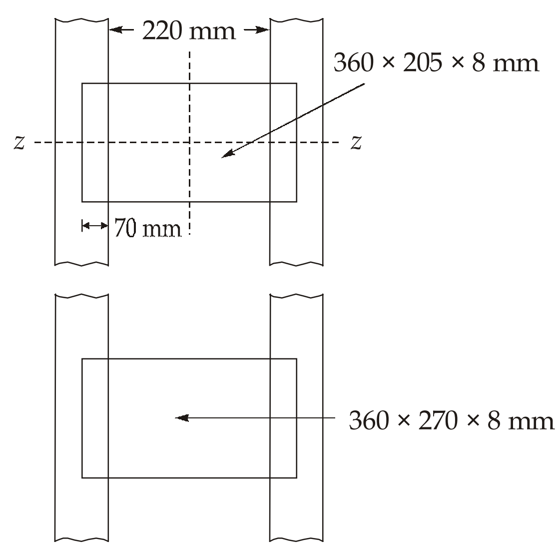
For end battens, shear stress $= \frac{73.71 \times 10^3}{270 \times 8} = 34.13 \text{ N/mm}^2$

$$\left(< \frac{250}{\sqrt{3} \times 1.1} = 131.27 \text{ N/mm}^2 \right)$$

Bending stress $= \frac{10.69 \times 10^6 \times 6}{8 \times 270^2} = 109.98 \text{ N/mm}^2$

$$\left(< \frac{250}{1.1} = 227.23 \text{ N/mm}^2 \right)$$

Hence it is safe.



7. **(b) (i) Solution:**

Calculation of Maximum Uniform Draft

Total annual inflow,

$$V_{\text{total}} = 6200 + 4800 + 5500 + 3000 + 1800 + 1200 + 1400 + 2100 + 2500 + 3400 + 5800 + 7000$$
$$= 44700 \text{ m}^3$$

Maximum uniform monthly draft,

$$D_{\text{monthly}} = \frac{44700}{12} = 3725 \text{ m}^3/\text{month}$$

Daily draft,

$$D_{\text{daily}} = \frac{3725}{30} = 124.167 \text{ m}^3/\text{day}$$

Mass Curve Analysis

Calculate cumulative inflow, cumulative outflow, and excess supply or demand:

Month	Inflow (m^3)	Outflow (m^3)	Cum. Inflow (m^3)	Cum. Outflow (m^3)	Excess Supply (+)/ Demand (–)
Jan	6200	3725	6200	3725	+2475
Feb	4800	3725	11000	7450	+3550
Mar	5500	3725	16500	11175	+5325
Apr	3000	3725	19500	14900	+4600
May	1800	3725	21300	18625	+2675
Jun	1200	3725	22500	22350	+150
Jul	1400	3725	23900	26075	–2175
Aug	2100	3725	26000	29800	–3800
Sep	2500	3725	28500	33525	–5025
Oct	3400	3725	31900	37250	–5350
Nov	5800	3725	37700	40975	–3275
Dec	7000	3725	44700	44700	0

Maximum excess supply (surplus) = 5325 m^3 (at the end of March)

Maximum excess demand (deficit) = 5350 m^3 (at the end of October)

Reservoir capacity required,

$$C = 5325 + 5350 = 10675 \text{ m}^3$$

7. (b) (ii) Solution:

Given: Mean discharge, $\bar{x} = 35000\ \text{m}^3/s$

Standard deviation, $\sigma = 18000\ \text{m}^3/s$

Acceptable risk, $R = 15\% = 0.15$

Service life, $n = 60$ years

Reduced mean, $\bar{y}_n = 0.5436$

Reduced standard deviation, $S_n = 1.1413$

1. Estimate the Design Flood Discharge

Calculate the Return Period T:

$$R = 1-\left(1-\frac{1}{T}\right)^n$$

$$0.15 = 1-\left(1-\frac{1}{T}\right)^{60}$$

$$\Rightarrow \quad \left(1-\frac{1}{T}\right)^{60} = 0.85$$

$$1-\frac{1}{T} = 0.85^{\frac{1}{60}}$$

$$T = \frac{1}{1-0.85^{1/60}} = 369.688 \text{ years}$$

Calculate the Reduced Variate y_T:

$$y_T = -\ln\left[\ln\left(\frac{369.688}{369.688-1}\right)\right]$$

$$y_T = 5.91$$

Calculate the Frequency Factor K:

$$K = \frac{y_T - \bar{y}_n}{S_n}$$

$$K = \frac{5.91-0.5436}{1.1413} = 4.702$$

Calculate the Design Flood Discharge x_T :

$$x_T = \bar{x} + K \times \sigma$$

$$x_T = 35000 + 4.702 \times 18000 = 119636\ m^3/s$$

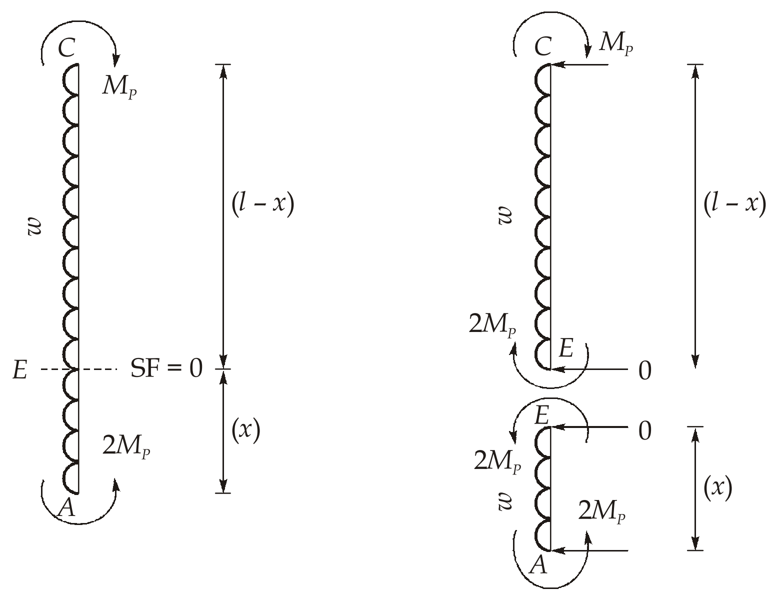
2. **Safety Margin Calculation**

The safety margin is the difference between the adopted design flood and the calculated estimated flood.

$$\text{Adopted Flood Value} = 135000 \text{ m}^3/s$$
$$\text{Calculated Flood Value} = x_T = 119636 \text{ m}^3/s$$
$$\text{Safety Margin} = 135000 - 119636 = 15364 \text{ m}^3/s$$

Q.7 (c) Solution:

(i) **Beam mechanism**



$$\Sigma M_A = 0$$

Segment AE, $$-2M_p - 2M_p + \frac{wx^2}{2} = 0$$

$$\Rightarrow \quad 4M_p = \frac{wx^2}{2} \quad \text{...(i)}$$

$$\Sigma M_c = 0$$

Segment *EC*, $$2M_p + M_p - \frac{w(l-x)^2}{2} = 0$$

$$\Rightarrow \quad 3M_p = \frac{w(l-x)^2}{2} \quad \text{...(ii)}$$

From eq. (i) and (ii)

$$\frac{wx^2}{8} = \frac{w(l-x)^2}{6}$$

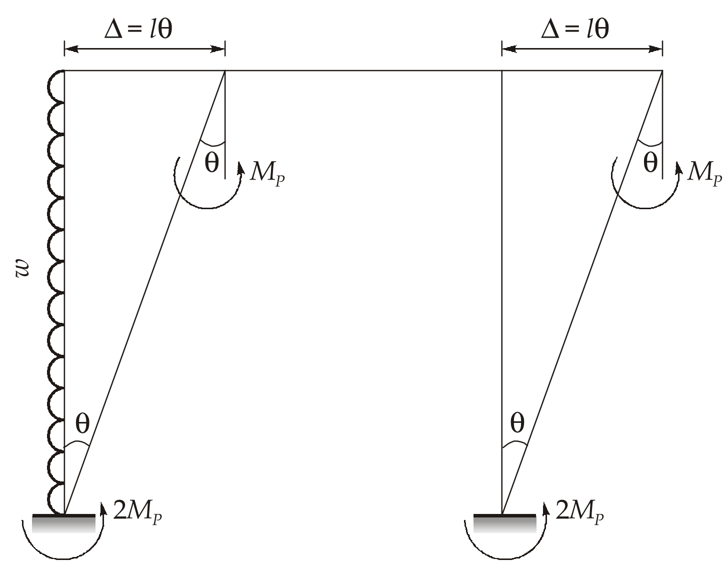
$$\Rightarrow \quad 3x^2 = 4(l^2 + x^2 - 2lx)$$

$$\Rightarrow \quad 3x^2 = 4l^2 + 4x^2 - 8lx$$

$$\Rightarrow \quad x = 0.536\, l$$

$$\therefore \quad w = w_u = \frac{8M_P}{x^2} = \frac{8M_P}{(0.536l)^2} = \frac{27.846M_P}{l^2} \quad \text{...(iii)}$$

(ii) Sway mechanism

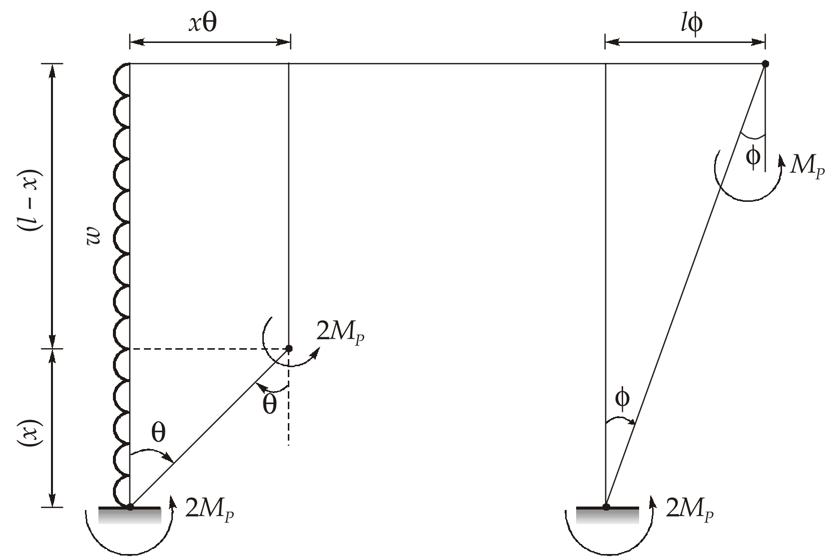


By principle of virtual work

$$2M_p\theta + M_p\theta + M_p\theta + 2M_p\theta = \frac{1}{2}wl(l\theta)$$

$$\Rightarrow \quad w = w_u = \frac{12M_P}{l^2} \quad \text{...(iv)}$$

(iii) Combined mechanism



$$x\theta = l\phi$$

$$\Rightarrow \qquad \phi = \left(\frac{x\theta}{l}\right)$$

By the principle of virtual work

$$2M_p\theta + 2M_p\phi + M_p\phi + 2M_p\theta = \frac{1}{2}x \times w \times (x\theta) + x\theta \times (l-x)w$$

$$\Rightarrow \qquad 4M_p\theta + 3M_p\frac{x\theta}{l} = \frac{1}{2}wx^2\theta + wxl\theta - wx^2\theta$$

$$\Rightarrow \qquad M_P\left(\frac{4l+3x}{l}\right) = wxl - \frac{1}{2}wx^2$$

$$\Rightarrow \qquad M_P = \left\{\frac{2wxl - wx^2}{2(4l+3x)}\right\}l$$

For maximum M_P,

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

$$\Rightarrow \qquad \frac{d}{dx}\left[\left\{\frac{2wxl - wx^2}{2(4l+3x)}\right\}l\right] = 0$$

$$\Rightarrow (2wl - 2wx)(8l + 6x) - (2wxl - wx^2)(6) = 0$$

$$\Rightarrow \qquad 16l^2 + 12lx - 16lx - 12x^2 - 12xl + 6x^2 = 0$$

$$\Rightarrow \qquad -6x^2 - 16xl + 16l^2 = 0$$

$$x = 0.77485\, l$$

Putting the value of x, $$M_P = 0.07505\, wl^2$$

$$\Rightarrow \quad w = w_u = \frac{13.324 M_P}{l^2} \quad \ldots(v)$$

So, the collapse load is lowest of (iii), (iv) and (v)

So, $$w_u = \frac{12 M_P}{l^2}$$

8. (a) (i) Solution:

Effective rainfall = Rainfall depth – ϕ-Index

Now, 1st hr = 4 – 2 × 1 = 2 cm

2nd hr = 3 – 2 × 1 = 1 cm

3rd hr = 2.5 – 2 × 1 = 0.5 cm

Time (1)	Ordinates of *U.H.* (m^3/sec) (2)	DRH due to 1st storm i.e. 2 cm (3)	DRH due to 2nd storm i.e. 1 cm ER- for eff. rainfall (4)	DRH due to 3rd storm i.e. 0.5 cm ER- for eff. rainfall (5)	Add ordinates of 3 rainfall for DRH ER- for eff. rainfall (6)	Storm Hydrograph = (6) + 2 Base flow (7)
0	0	0	0	0	0	0
1	58	116	0	0	116	118
2	110	220	58	0	278	280
3	96	192	110	29	331	333
4	53	106	96	55	257	259
5	26	52	53	48	153	155
6	14	28	26	26.5	80.5	82.5
7	8	16	14	13	43	45
8	5	10	8	7	25	27
9	4	8	5	4	17	19
10	3	6	4	2.5	12.5	14.5
11	1.5	3	3	2	8	10
12	1	2	1.5	1.5	5	7
13	0	0	1	0.75	1.75	3.75
14	0	0	0	0.5	0.5	2.5
15	0	0	0	0	0	0

Q.8 (a) (ii) Solution:

Flood hydrograph shows stream flow due to a storm over a catchment. Flood hydrograph is used for short term study. The factors that affect the shape of the hydrograph can be broadly grouped into two types:

(i) Physiographic factor

(ii) Climate factor

Physiographic factors	Climate factors
1. Basin characteristics: (a) Shape (b) Size (c) Slope (d) Nature of the valley (e) Elevation (f) Drainage density 2. Infiltration characteristics: (a) Land use and cover (b) Soil type and geological conditions (c) Lakes, swamps and other storage 3. Channel characteristics: cross-section, oughness and storage capacity	1. Storm characteristics: precipitation, intensity, duration, magnitude and movement of storm. 2. Initial loss 3. Evapotranspiration

Generally, the rising limb is affected by the climatic factors and recession limb is affected by the physiographic factor.

The factors which affect the shape of hydrograph are as follows:

- **Shape of Catchment**: The shape of the catchment influences the time taken for water from the remote parts of the catchment to arrive at the outlet of catchment. The occurrence of peak and the shape of hydrograph are affected by the catchment shape.

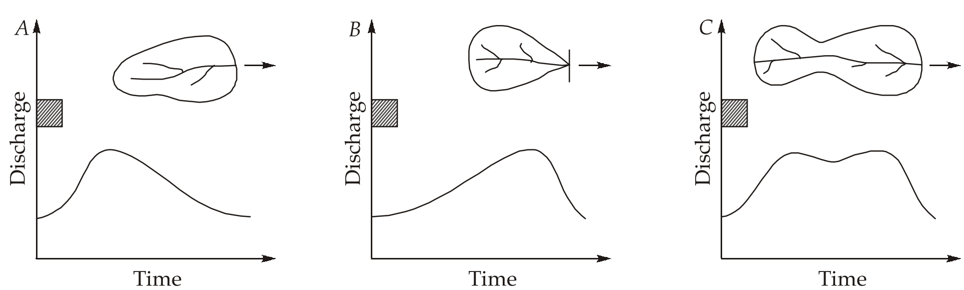


Fig. Effect of Catchment Shape on the Hydrograph

- **Size:** In small catchment area, the overland flow is predominant over the channel flow. Hence the land use and intensity of rainfall have important role on peak flood, while on large basins these effects are suppressed, because on large basin the, channel flow phase is more predominant. The time base of hydrograph from larger basins will be larger than those of corresponding hydrograph from smaller basin.

- **Slope:** The recession limb of hydrograph represents the depletion of storage, so the slope will have a pronounced effect on this part of hydrograph. Steeper slope give rise to quicker depletion of storage and hence result in steeper recession limbs of hydrograph and also the smaller time base. The basin slope is important in small catchment where the overland flow is relatively more important, the steeper slope of catchment results in larger peak discharge.
- **Drainage Density**: The drainage density is the ratio of the total channel length to the total drainage area. A large drainage density creates a situation for quick disposal of runoff down. This part response is reflected in a pronounced peak discharge.

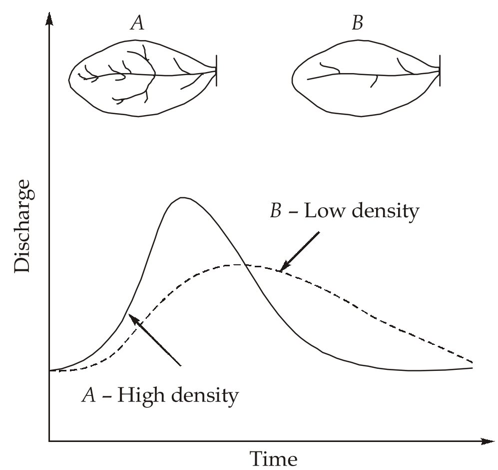


Fig. Role of Drainage Density on the Hydrograph

In those catchments where the drainage density is less, the overland flow is predominant and resulting hydrograph is with a slowly rising limb.

- **Land Use**: An area covered by vegetation and forest shows increase in the infiltration and storage capacities of the soils. The vegetation causes considerable retardance to overland flow, thus the vegetal cover reduces peak flow.
- **Climatic Factor**: Among all the climatic factors; intensity, duration and direction of rainfall movement are the three important factors affecting the shape of a flood hydrograph.
  - **(a) Rainfall Intensity**: For a given specified duration, the peak and volume of surface runoff are proportional to intensity of rainfall. The shape of the hydrograph can also be affected by the intensity of rainfall.
  - **(b) Rainfall Duration**: For the same volume of rainfall, if the rainfall duration increases then the peak of hydrograph reduces.
  - **(c) Direction of Rainfall**: If the rainfall direction is in the direction of outlet then it gives left skewed hydrograph or if the rainfall direction is in the opposite direction of outlet then it gives right skewed hydrograph.

8. (b) Solution:

For Fe410, $f_u = 410\ \text{N/mm}^2$

$f_y = 250\ \text{N/mm}^2$

The outstand of flange element,

$$b = \frac{b_f}{2} = \frac{165}{2} = 82.5\ \text{mm}$$

Section Classification

Depth of web, $d = h - 2\,(t_f + R)$

$= 350 - 2\,(11.4 + 16)$

$= 295.2$ mm

$$\frac{b}{t_f} = \frac{82.5}{11.4} = 7.24 < 9.4\epsilon = 9.4 \qquad \left(\because \epsilon = \sqrt{\frac{250}{f_y}} = 1\right)$$

$$\frac{d}{t_w} = \frac{295.2}{7.4} = 39.89 < 84\epsilon = 84$$

Hence the section is plastic,

Check for shear capacity:

Design shear force, $V = 210$ kN

Design shear strength of the section,

$$V_d = \frac{f_y}{\sqrt{3}\,\gamma_{m0}} \times h t_w = \frac{250}{\sqrt{3} \times 1.1} \times 350 \times 7.4 \times 10^{-3}\ \text{kN}$$

$= 339.85\ \text{kN} > 210\ \text{kN}$

Which is OK.

Check for high/low shear:

$0.6\ V_d = 0.6 \times 339.85$

$= 203.91\ \text{kN} < 210\ \text{kN}$

This case is for high shear since

$$V > 0.6\ V_d$$

Check for design bending strength:

$$M_{dv} = M_d - \beta\left(M_d - M_{fd}\right) \leq 1.2\frac{Z_e f_y}{\gamma_{m0}}$$

$$\beta = \left(\frac{2V}{V_d} - 1\right)^2 = \left(\frac{2\times210}{339.85} - 1\right)^2 = 0.0556$$

$$M_d = Z_{Pz} \times \frac{f_y}{\gamma_{m0}} = 851.11\times10^3 \times \frac{250}{1.1} \times 10^{-6} \text{ kNm}$$

$$= 193.434 \text{ kNm}$$

$$Z_{fd} = Z_{pz} - A_w y_w$$

$$= 851.11\times10^3 - 350\times7.4\times\left(\frac{350}{4}\right)$$

$$= 624485 \text{ mm}^3 = 624.485 \times 10^3 \text{ mm}^3$$

$$M_{fd} = \frac{Z_{fd} f_y}{\gamma_{m0}} = 624.485\times10^3 \times \frac{250}{1.1} \times 10^{-6}$$

$$= 141.93 \text{ kNm}$$

$$M_{dv} = M_d - \beta\left(M_d - M_{fd}\right)$$

$$= 193.434 - 0.0556\,(193.434 - 141.93)$$

$$= 190.57 \text{ kNm} > 150 \text{ kNm} \qquad \text{(OK)}$$

$$\leq 1.2\times\frac{Z_e f_y}{\gamma_{m0}}$$

$$= 1.2\times751.9\times10^3 \times \frac{250}{1.1} \times 10^{-6} = 205.06 \text{ kNm}$$

(OK)

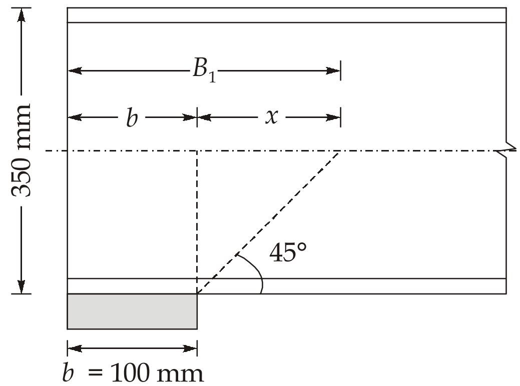
Check for web buckling (at support):

Stiff bearing length, $b = 100$ mm

$$A_b = B_1 t_w = (b + x)\, t_w$$

$$= (100 + 175) \times 7.4 = 2035 \text{ mm}^2$$

$$\left\{x = \frac{h}{2} = \frac{350}{2} = 175 \text{ mm}\right\}$$



Effective length of web, $kL = 0.7 \times d = 0.7 \times 295.2 = 206.64$ mm

I_{eff} of web, $\dfrac{bt_w^{\ 3}}{12} = \dfrac{100 \times 7.4^3}{12} = 3376.87 \text{ mm}^4$

A_{eff} of web , $bt_w = 100 \times 7.4 = 740 \text{ mm}^2$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{3376.87}{740}} = 2.1362 \text{ mm}$$

$$\lambda = \frac{kL}{r} = \frac{206.64}{2.1362} = 96.733$$

From buckling curve '*c*'

Interpolating the value for given table,

$$\frac{100-90}{107-121} = \frac{96.733-90}{f_{cd}-121}$$

$\Rightarrow$ $f_{cd} = 111.574 \text{ N/mm}^2$

Capacity of web section, $F_{wb} = A_b f_{cd}$

$= 2035 \times 111.574 \times 10^{-3}$ kN

$= 227.05 \text{ kN} > 210 \text{ kN}$ (OK)

Check for web bearing:

$$f_w = (b+n_1)t_w \times \frac{f_y}{\gamma_{m0}}$$

$$n_1 = 2.5\left(t_f + R\right) = 2.5\,(11.4 + 16) = 68.5 \text{ mm}$$

(Dispersion ratio is 1 : 2.5)

Stiff bearing length, $b = 100$ mm

$$F_w = (100+68.5) \times 7.4 \times \frac{250}{1.1} \times 10^{-3}$$

$F_w = 283.386 \text{ kN} > 210 \text{ kN}$ (OK)

8. (c) Solution:

The various elements of a riveted/bolted girder are depicted in figure (a) and that of a welded plate girder in figure (b). These elements are as listed below:

1. Web plate
2. Flange angles with or without flange cover plates for riveted/bolted plate girder and only flange plates for welded plate girders.
3. Stiffeners—bearing, transverse and longitudinal.
4. Splices—for web and flange

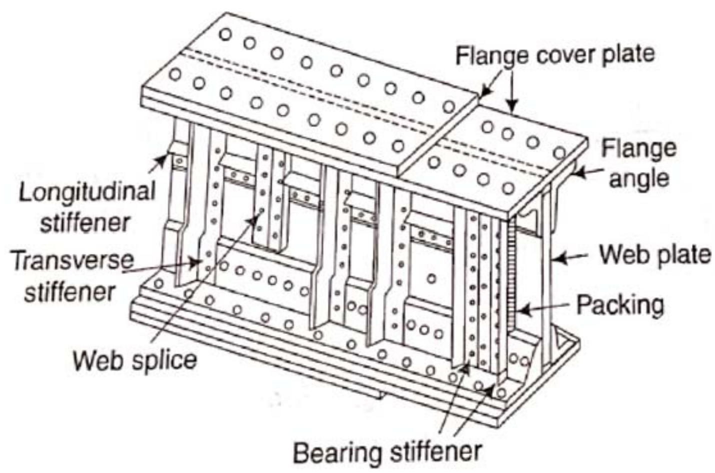


Fig. (a) ***Bolted Plate Girder***

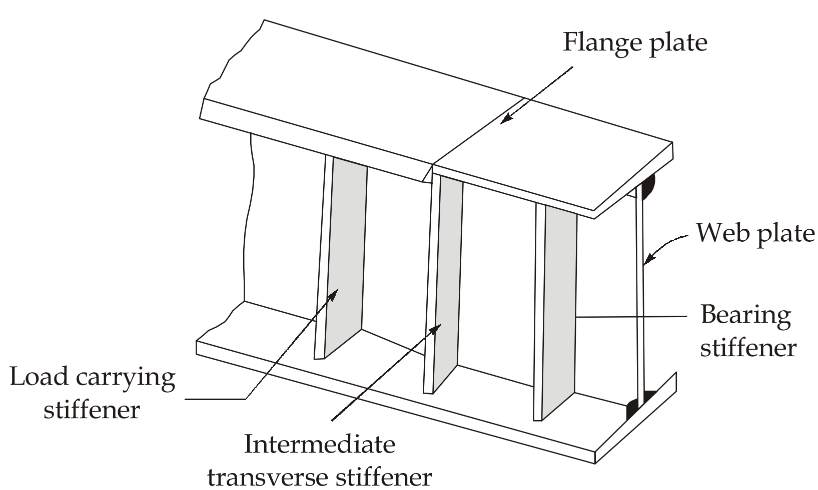


Fig. (b) ***Welded Plate Girder***

(a) Minimum web thickness based on serviceability requirement

As per Cl. 8.6.1.1 of IS 800 : 2007

(i) When transverse stiffeners are not provided and web is connected to flange along both longitudinal edges,

$$\frac{d}{t_w} \leq 200\ \varepsilon_w$$

(ii) When transverse stiffness are not provided and web is connected to flanges along one longitudinal edge only

$$\frac{d}{t_w} \leq 90\ \varepsilon_w$$

(iii) When only transverse stiffeners are provided in webs connected to flanges along both longitudinal edges.

Case-1: $3\,d \geq c \geq d$

$$\frac{d}{t_w} \leq 200\ \varepsilon_w$$

Case-2: $0.74\,d \leq c \leq d$

$$\frac{c}{t_w} \leq 200\ \varepsilon_w$$

Case-3: $c < d$

$$\frac{d}{t_w} \leq 270\ \varepsilon_w$$

Case-4: $c > 3d$

Here the web shall be considered as unstiffened.

(iv) When transverse stiffeners are provided along with longitudinal stiffener at one level only i.e., at 0.2 d from the compression flange.

Case-1: $2.4\,d \geq c \geq d$

$$\frac{d}{t_w} \leq 250\ \varepsilon_w$$

Case-2: $0.74d \leq c \leq d$

$$\frac{c}{t_w} \leq 250\ \varepsilon_w$$

Case-3: $c < 0.74d$

$$\frac{d}{t_w} \leq 340\ \varepsilon_w$$

(v) When a second longitudinal stiffener is also provided (at neutral axis of the section)

$$\frac{d}{t_w} \le 400\,\varepsilon_w \qquad \left(\text{where, } \varepsilon_w = \sqrt{\frac{250}{f_{yw}}}\right)$$

(b) Minimum web thickness based on compression flange buckling requirement

As per Cl. 8.6.1.2 of IS 800 : 2007, in order to avoid buckling of compression flange, the web thickness shall comply with the following requirements:

(i) When transverse stiffeners are not provided

$$\frac{d}{t_w} \le 345\,\varepsilon_f^2 \qquad \left(\text{where, } \varepsilon_f = \sqrt{\frac{250}{f_{yf}}}\right)$$

(ii) When transverse stiffeners are provided

Case-1: $c \ge 1.5\,d$

$$\frac{d}{t_w} \le 345\,\varepsilon_f^2$$

Case-2: $c < 1.5\,d$

$$\frac{d}{t_w} \le 345\,\varepsilon_f$$