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Detailed Solutions

ESE-2024 Mains Test Series

Civil Engineering Test No : 9

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

Q.1 (a) Solution:

Defects in clay bricks:

1. **Efflorescence:** This defect is caused because of alkalis present in bricks. When bricks come in contact with moisture, water is absorbed by them. This absorbed water dries out by evaporation from the exposed faces, and as it does so, the soluble salts it contains crystallize out on the surface. On drying, grey or white powder patches appear on the brick surface. The process often continues for many years depending on the quantity of salts present in the bricks and their solubility. The less soluble salts, such as calcium sulphate, take much longer period to be leached out. Magnesium salts are very soluble and are the most destructive. Bricks which have been saturated before their placement in masonry will be more prone to efflorescence than those under dry conditions. Efflorescence can be minimized by selecting proper clay materials for brick manufacturing, preventing moisture to come in contact with the masonry. This can be achieved by providing waterproof coping and by using water repellent materials in mortars and by providing damp proof course.
2. **Over-burning of bricks**: Bricks should be burned at temperatures at which incipient, complete and viscous vitrification occur. However, if the bricks are overburnt, a soft molten mass is produced and the bricks loose their shape. Such bricks are not used for construction works.
3. **Under-burning of bricks**: When bricks are not burnt to cause complete vitrification, the clay is not softened because of insufficient heat and the pores are not closed. This

results in higher degree of water absorption and less compressive strength. Such bricks are not recommended for construction works.

4. **Bloating:** This defect is observed as spongy swollen mass over the surface of burned bricks and is caused due to the presence of excess carbonaceous matter and sulphur in brick-clay.
5. **Black core:** When brick-clay contains bituminous matter or carbon and they are not completely removed by oxidation, the brick results in black core mainly because of improper burning.
6. **Chuffs:** The deformation of the shape of bricks caused by the rain water falling on hot bricks is known as chuffs.
7. **Checks or Cracks**: This defect is because of lumps of lime or excess of water. In case of the former, when bricks come in contact with water, the absorbed water reacts with lime nodules causing expansion and a consequent disintegration of bricks. Plaster, applied over bricks containing lumps of lime, can be pushed off the walls because of the expansion of the later whereas shrinkage and burning cracks result when excess of water is added during brick manufacturing.
8. **Spots:** Iron sulphide, if present in the brick clay, results in dark surface spots on the brick surfaces. Such bricks though not harmful are unsuitable for exposed masonry work.
9. **Blisters:** Broken blisters are generally caused on the surface of sewer pipes and drain tiles due to air imprisoned during their moulding.
10. **Laminations:** These are caused by the entrapped air in the voids of clay. Laminations produce thin lamina on the brick faces which weather out on exposure. Such bricks are weak in structure

Q.1 (b) Solution:

Given data : Normal width, $B_0 = 15$ m,
Flume width, $B_f = 9$ m,
Length of transition, $L_f = 16$ m

Let V_0, V_f, V_x are velocities at original width, flumed width and at distance 'x' from the flumed section. And 'B_x' be the width at a distance 'x' from the flumed section.

Using Mitra's approach, rate of change of velocity per unit length of transition is kept constant.

$$\therefore \quad \frac{V_f - V_x}{x} = \frac{V_f - V_0}{L_f} \quad \text{... (i)}$$

Also water depth, D and discharge, Q are constant

So, $$Q = V_0 B_0 D = V_f B_f D = V_x . B_x . D \quad \text{... (ii)}$$

From (i) and (ii)

$$\frac{\left(\frac{Q}{B_f D} - \frac{Q}{B_x D}\right)}{x} = \frac{\left(\frac{Q}{B_f D} - \frac{Q}{B_0 D}\right)}{L_f}$$

$$\Rightarrow \quad \frac{(B_x - B_f) L_f}{B_x B_f} = \frac{(B_0 - B_f) x}{B_f B_0}$$

$$\Rightarrow \quad B_x = \frac{B_0 B_f L_f}{L_f B_0 - (B_0 - B_f) x}$$

$$\Rightarrow \quad B_x = \frac{B_0 B_f L_f}{L_f B_0 - (B_0 - B_f) x} = \frac{15 \times 9 \times 16}{16 \times 15 - (15 - 9) \times x}$$

$$= \frac{2160}{240 - 6x} = \left(\frac{360}{40 - x}\right)$$

x (m)	2	4	6	8	10	12	14
B_x(m)	9.47	10	10.59	11.25	12	12.86	12.85

Q.1 (c) Solution:

(i) High alumina cement (IS: 6452)

It's composition is very different from portland cement.

- The raw material used for its manufacture consists of 40% bauxite, 40% lime and 15% iron oxide with a little percentage of ferric oxide and silica, magnesia, etc. ground finely at a very high temperature.
- As since C_3A is not present, the cement has good resistance for attack by sulphate and some dilute acids, and is particularly suitable for sea and underwater work.
- It's rapid hardening properties arises due to $Al_2O_3.CaO$ (calcium aluminate) as the predominant compound in place of calcium silicate of OPC and for setting and hardening there is not free hydrated lime as in the case of OPC.
- High alumina cement has very high early compressive strength and has high heat of hydration in comparison to OPC 43 grade.
- High alumina cement has initial setting time of about 3.5-4 hrs and final setting time of 5-5.5 hrs.
- It hardens and develops strength very rapidly. One day strength is 30 N/mm^2 (which is equal to 28 days strength of OPC) and 3 day is 35N/mm^2 giving out a great amount

of heat.

- High alumina cement is preferred for use in cold region due to high heat of hydration.
- It should have a fineness not less than 225 m^2/kg.
- It is used for refractory concrete, in industries and is used widely for pre-casting.
- It is not recommended in tropical region and not mixed with any other type of cement.

(ii) Sulphate resisting portland cement (SRPC) (IS: 12330)

- It is similar to ordinary portland cement except that it contains very low C_3A content (< 5%).
- This cement is sulphate-resistant because the disintegration of harden concrete, caused by the chemical reaction of C_3A with soluble sulphate like $MgSO_4$, $CaSO_4$ and Na_2SO_4 is inhibited.
- Expansion of cement is limited to 10 mm and 0.8 per cent, when tested by Le-chatelier method and autoclave test, respectively.
- Setting times are same as that for ordinary Portland cement.
- It should have a fineness not less than 225m^2/kg.
- Initial setting time is more than 30 minutes and final setting time is less than 600 minutes.
- It can be used as an alternative of OPC, PPC or Portland slag cement under normal conditions
- But its use is restricted where the prevailing temperature is below 40°C and environment where chlorides are present. It is strongly recommended for structures in sea water, coastal area and marshy lands.

(iii) Air entraining cement:

- It is manufactured by grinding vinsol resin or vegetable fats and oils and fatty acids with ordinary cement; or by mixing a small amount (= 0.1%) by weight of an air entraining agent with OPC clinker at the time of grinding.

  Some of the air entraining agents are:

  1. Alkali salt of wood resins.
  2. Calcium salt of glues and other protein obtained in the treatment of animal hides.
  3. Calcium lignosulphate derived from the sulphite process in paper making.
  4. Synthetic detergents of the alkyl-aryl sulphonate type.

- These materials have the property to entrain air in the form of fine tiny air bubbles.
- Air entraining cements are used for the same purposes as that of OPC.

- Air entrainment improves workability and water-cement ratio can be reduced which in turn reduces shrinkage, etc.
- Minute voids are formed while setting of cement which increases resistance against freezing and scaling action of salts.
- It has higher initial setting time and longer final setting time than OPC.

Q.1 (d) Solution:

Following are the objectives of tunnel lining:

1. To provide the correct desired shape and cross-section to the tunnel.
2. To withstand the soil pressure and to prevent the tunnel from collapse more specially in soft grounds.
3. To keep the tunnel side portion free from water leakage.
4. To bind the loose rock and to provide stability to the tunnel.
5. To reduce the maintenance cost of the tunnel.
6. To provide better appearance to the tunnel.

Generally following types of lining can be used for tunnels:

1. **Timber lining:** It is the oldest material used for purpose of lining, but slowly concrete is taking its place. During construction, timber is used as a temporary support and afterwards as permanent support.
2. **Brick masonry:** Till the advent of concrete, brick masonry was the standard material for tunnel lining, but now it is used in case of under ground sewers as bricks are more acid resistant and suitable to carry sewage.
3. **Cast iron lining:** For shield driven tunnels, specially in subaqueous regions, cast iron lining has proved very useful.
4. **Plain & R.C.C. lining:** Now a days, cement concrete has become the standard material for tunnel lining in soft soils as well as in hard rocks.

Q.1 (e) Solution:

Let us convert Na^+, Ca^{2+}, Mg^{2+} concentration into meq/lit

Constituent (in mg/*l*)	Sample I	Sample II
Na^+	$\frac{114}{23} = 4.96$	$\frac{520}{23} = 22.61$
Ca^{2+}	$\frac{110}{20} = 5.5$	$\frac{104}{20} = 5.2$
Mg^{2+}	$\frac{30}{12} = 2.5$	$\frac{38}{12} = 3.17$

SAR of sample - I

Now, sodium absorption ratio, $\text{SAR} = \dfrac{Na^+}{\sqrt{\dfrac{Mg^{2+} + Ca^{2+}}{2}}} = \dfrac{4.96}{\sqrt{\dfrac{5.5+2.5}{2}}} = 2.48$

$\therefore$ Water is classified as low sodium water (S_1) based on SAR value.

Electrical conductivity, $EC = 1240\ \mu\text{mho/cm}$

This value lies in the range of 750 – 2250, high salinity water (C_3).

Hence, Sample I can be classified as C_3S_1.

SAR of sample II

$$\text{Sodium absorption ratio, SAR} = \frac{22.61}{\sqrt{\dfrac{5.2+3.17}{2}}} = 11.05$$

Hence this value of SAR lies in the range of 10-18 and water is classified as medium sodium water (S_2).

$$E.C. = 2340\ \mu\ \text{mho/cm} > 2250\ \mu\ \text{mho/cm}$$

The water is classified as very high salinity water (C_4) thus classification of sample is C_4S_2.

Now, requirement of gypsym for sample II.

$$\text{SAR} = 11.05$$

$$\text{SAR required} = 4$$

So, $$\text{SAR} = \frac{Na^+}{\sqrt{\dfrac{Ca^{2+} + Mg^{2+}}{2}}} = 4$$

$$\Rightarrow \quad \frac{22.61}{\sqrt{\frac{Ca^{2+}+3.17}{2}}} = 4$$

$$\Rightarrow \quad Ca^{2+} = 60.73 \text{ meq}/l$$

$\therefore$ Concentration of Ca^{2+} required $= 60.73 \times 20$

$= 1214.6 \text{ mg}/l$

Concentration of Ca^{2+} already present $= 104 \text{ mg}/l$

$\therefore$ Additional requirement of $Ca^{+2} = (1214.6 - 104) = 1110.6 \text{ mg}/l$

Percentage of Ca in $CaSO_4$ (gypsum) $= \frac{40}{136} \times 100\% = 29.41\%$

$\therefore$ Quantity of gypsum ($CaSO_4$) $= \frac{1110.6}{0.294} \text{mg}/l = 3777.55 \text{ mg}/l = 3.78 \text{ kg/m}^3$

$\therefore$ Quantity of $CaSO_4$ required for 2 hectare-m of water

$$= 2 \times \frac{3.78 \times 10^4}{10^3} \text{ tonnes} = 75.6 \text{ tonnes}$$

Q.2 (a) Solution:

Workability:

- Workability is referred to as the ease with which a concrete can be transported, placed and fully compacted without excessive bleeding or segregation. For maximum strength in concrete, full compaction is required. It means a higher water to cement ratio than theoretical requirements. This is because workability can also be defined as the internal work done in overcoming the frictional forces between concrete ingredients for full compaction. So, water functions as a lubricant so that concrete can be compacted up to maximum possible extent.
- The optimal workability of concrete will depend upon the given job and hence will vary from situation to situation. It should be noted that workability is different from consistency. Consistency indicates fluidity or mobility. Concrete with high consistency may not be workable for a particular job and concrete having same consistency may vary in workability.

Factors affecting workability of concrete are as follow:

1. Water content:

- In order to obtain higher degree of workability, higher water content is required but with the increase in water-cement ratio, the strength of the cement concrete decreases. Hence, in order to maintain the strength, along with water content, proportion of cement should be increased.

- For a given maximum size of coarse aggregate, the slump or consistency of concrete is a direct function of the water content; i.e., within limits, it is independent of other factors such as aggregate grading and cement content.
- At a constant water-cement ratio, reduction in the aggregate-cement ratio causes increase in the water content, which consequently results into the increase in consistency of concrete.

2. Cement content:

- In normal concrete, at given water content, a considerable lowering of the cement content tends to produce harsh (i.e., low workable) mixtures with poor finishability.
- Concrete containing a very high proportion of cement or a very fine cement show excellent cohesiveness but tend to be sticky.

3. Mix proportions:

- Aggregate-cement ratio influences the workability to a large extent. The higher the ratio, leaner will be the concrete. In lean concrete the cement paste available for lubrication per unit surface area of aggregates is less and hence the workability gets reduced.

4. Aggregate size:

- If the aggregates will be of higher size then the total surface area to be wetted is less, also less paste will be required for lubricating the surface to reduce internal friction. For a given water content, large size aggregates give high workability.

5. Shape of aggregates:

- At a given water content, round and cubical shape aggregates are more workable than rough, angular or flaky aggregates, because the former type of aggregates requires less cement paste for lubrication as these have less surface area and lesser voids.
- In round aggregates, frictional resistance is small. So less lubrication is required. For this reason, river sand and gravel provide greater workability than crushed sand and aggregates.

6. Surface texture:

- Rough surface aggregate will have more surface area than a smooth round textured aggregate. Therefore, smooth textured aggregate will be more workable.

7. Grading of aggregates:

- Well graded aggregates are more workable. Because such a mix will have least voids and thus excess cement paste will be available as lubricant.

- This avoids segregation also.

8. **Admixtures:**
   - At given water content, air-entraining admixtures improve the consistency and cohesiveness of the concrete by increasing the volume of paste, making the concrete more workable. Because air forms bubbles, on which the aggregates slide past each other increasing the workability. Air entraining agents are surface active and they reduce the internal friction between the aggregates.
   - Pozzolanic admixtures tend to improve the cohesiveness of concrete. Fly ash, when used as a partial replacement for fine aggregate, generally increases the consistency at given water content. At a constant water content of a concrete mixture, the addition of a water-reducing admixture will increase the consistency.

Q.2 (b) Solution:

(i)

Radius of curve, R is given by:

$$R = \frac{1750}{\text{Degree of curve}} = \frac{1750}{30} = 583.33 \text{ m}$$

1. Extra clearance required on inside of the curve for platform

$$= \text{Overthrow + lean + sway} - 51 \text{ mm}$$

$$= \frac{C^2}{8R} + \frac{e.h}{G} + \frac{eh}{4G} - 51$$

Where, $C = 14780 \text{ mm} = 14.780 \text{ m}$

$R = 583.33$ m, $h = 0.80$ m, $e = 0.125$ m

$G = 1.75$ m, $L = 21.35$ m

Now, extra clearance on inside $= \dfrac{(14.780)^2}{8\times583.33} + \dfrac{0.125\times0.8}{1.75} + \dfrac{0.125\times0.8}{4\times1.75} - 0.051$

$$= 0.06723 \text{ m} = 67.23 \text{ mm}$$

2. Extra width on outside the curve for platform = End throw – 0.025 m

$$= \frac{L^2 - C^2}{8R} - 0.025 = \frac{(21.35)^2 - (14.78)^2}{8\times583.33} - 0.025$$

$$= 0.025866 \text{ m} = 25.866 \text{ mm}$$

(ii) Given $N = 8.5$

1. From Cole's method or Right angle method,

$$\cot\alpha = N \quad \Rightarrow \quad \tan\alpha = \frac{1}{N} \quad \Rightarrow \quad \tan\alpha = \frac{1}{8.5}$$

$\Rightarrow \quad \alpha = 6°42' 35.41''$

2. From centre line method:

$$\cot\frac{\alpha}{2} = 2N \quad \Rightarrow \quad \cot\frac{\alpha}{2} = 2\times 8.5 = 17 \quad \Rightarrow \quad \alpha = 6°43' 58.52''$$

3. From isosceles triangle method:

$$\sin\frac{\alpha}{2} = \frac{1}{2N} \quad \Rightarrow \quad \sin\frac{\alpha}{2} = \frac{1}{2\times 8.5} = \frac{1}{17}$$

$$\Rightarrow \quad \frac{\alpha}{2} = \sin^{-1}\left(\frac{1}{17}\right) \quad \Rightarrow \quad \alpha = 6°44' 40.40''$$

The value of angle of crossing obtained from the right angle/Cole's method is the least. Hence it is the best method. It is the standard method used in Indian Railways.

Q.2 (c) Solution:

(i) Gel-space ratio is defined as the ratio of volume of hydrated cement paste to the sum of the volume of the hydrated cement and that of the capillary pores.

For complete hydration,

$$\text{Gel-space ratio} = \frac{0.657C}{0.319C + W_0}$$

where C = Weight of cement (in gram)

W_0 = Volume of mixing water (in ml)

For partial hydration,

$$\text{Gel space ratio} = \frac{0.657C\alpha}{0.319C\alpha + W_0}$$

where α = Fraction of cement that has hydrated

Given: $W_0 = 500 \times 0.55 = 275$ ml

On partial hydration,

$$\text{Gel space ratio} = \frac{0.657C\alpha}{0.319C\alpha + W_0} = \frac{0.657\times 500\times 0.75}{0.319\times 500\times 0.75 + 275} = 0.6243$$

(ii)

Most commonly used chemical admixture are:

1. **Accelerators**: Normally reduce setting time, accelerate the rate of hydration of cement and consequently the rate of gain of strength.

Example: $AlCl_3$, $CaCl_2$, NaCl, NaOH, KOH, Calcium Formate, Formaldehyde etc.

2. **Retarders:** They normally increase the setting time and thus delay the setting of cement. Since these reduce the rate of hydration, more water is available and better is the workability.

   Example: Calcium sulphate, sugar, starch, cellulose, ammonium, ferrous and ferric chlorides, etc.

3. **Air entraining agents**: Air entrainment increases workability and resistance of concrete against weathering. The possibility of bleeding and segregation and laitance also get reduced. However, there is some loss in the strength of concrete.

Example:

- Surface active agents
  - Natural wood resins, vinsol resin
  - Darex
  - Animal or vegetable fats
- **Chemicals:** Zinc or aluminium powder
- **Dispersing agent:** Calcium lignosulphonate

4. **Plasticizers:** Plasticizers are organic or combination of organic and inorganic substances which allow water reduction for a given workability or give higher workability at same water content. They are surfactants and induce negative charge on individual cement particles.

   Example: Carboxylic acids, Calcium lignosulphonates, Hydroxy carboxylic acids etc.

5. **Superplasticizers**: They are hydrodynamic lubricants which impart high workability by reducing friction between the grains or by reducing the amount of water to be added. They are improved version of plasticizers which interact both physically and chemically with cement particles. They are anionic in nature and impart negative charge to the cement particles.

   Examples: Sulphonated melamine fomaldehyde, napthalene, sulphonated formaldehyde condensates, mixture of succharates and acid amines.

(iii)

Shotcrete is mortar or very fine concrete deposited by jetting or impacting it with high velocity (pneumatically projected or sprayed) on to a prepared surface. The system has different proprietary names in different countries such as Blastcrete, Blowcrete, Guncrete, Jet-crete, Nucrete, Pneukrete, Spraycrete. Torkrete, etc., though the principle is essentially the same. Shotcrete offers advantages over conventional concrete in a variety of new construction and repair works. Shotcrete is usually more economical than conventional concrete because of less formwork requirements, requiring only a small portable plant for manufacture and placement.

Shotcrete uses compressed air to shoot concrete onto the surface of structure. There are two application methods for shotcrete viz:

1. **Dry-mix:** In this method, the dry mixture of cement and aggregates is filled into the machine and conveyed with compressed air through the hoses. The water needed for the hydration is added at the nozzle.
2. **Wet-mix:** In this procedure, the mixes are prepared with all necessary water for hydration. The mixes are pumped through the hoses. At the nozzle, compressed air is added for spraying.

Properties of shotcrete The properties of the shotcrete are essentially the same as for conventional concrete of same materials and proportions. However, shortcrete has the following characteristics.

1. In shotcrete, generally, a small-maximum-size aggregate is used and cement content is high. These should enhance durability in most cases.
2. Whereas conventional concrete is consolidated by vibration, shotcrete is consolidated by the impact of a high-velocity jet impinging on the surface. This process not only increases the cement content due to rebound but also brings about different air-void systems affecting the durability of shotcrete.
3. The application procedures have a greater effect on the in-place properties of shotcrete than the mix proportions.
4. The low water-cement ratio enhances the durability of shotcrete, for most types of exposures. The preferred range of water-cement ratio for most shotcretes is 0.30 to 0.45.
5. Shotcrete has wide applications in different constructions, such as thin over-head vertical or horizontal surfaces, particularly the curved or folded sections.

   Shotcrete failures may occur by peeling off of sound shotcrete or by the delamination between shotcrete layers.

Q.3 (a) Solution:

(i) Elementary profile of a gravity dam is a right angled triangle having zero width at the water level and a base width (B) at bottom, i.e. the point where maximum hydro static pressure acts. It is subjected only to the external water pressure on the upstream side.

(i) Base width of elementary profile for no tension.

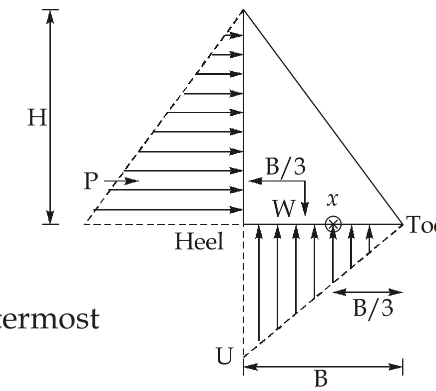
Here, P = Hydrostatic water pressure

W = Self weight of dam

U = Uplift pressure acting on base of dam

B = Base width of the dam.

For no tension to be developed, the resultant of all forces, P, W and U must pass through the outermost middle third point X at distance B/3 from toe.



Thus taking the moment of all the forces about point 'X' as shown in figure

$$W \times \frac{B}{3} - U \times \frac{B}{3} - P \times \frac{H}{3} = 0$$

$$\Rightarrow \quad (W-U)\frac{B}{3} - \frac{PH}{3} = 0$$

But, $$W = \frac{1}{2} BH \times 1 \times S_C \times \gamma_w$$

where S_C is specific gravity of dam material and γ_w is unit weight of water.

$$U = \frac{1}{2} \times C\gamma_w H \times B$$

where C is a constant which is taken as equal to 1 in calculation and will be equal to zero when no uplift is considered.

$$P = \frac{1}{2} \times \gamma_w H^2$$

$$\therefore \quad \left(\frac{1}{2} BHS_C\gamma_w - \frac{1}{2} C\gamma_w HB\right) \times \frac{B}{3} - \frac{1}{2}\gamma_w H^2 \times \frac{H}{3} = 0$$

$$\Rightarrow \quad \frac{1}{2} \times \frac{1}{3} H\gamma_w \left(B^2 S_c - B^2 C\right) = \frac{1}{2} \times \gamma_w H^2 \times \frac{H}{3}$$

$$\Rightarrow \quad B^2 (S_C - C) = H^2$$

$$\Rightarrow \quad B = \frac{H}{\sqrt{S_C - C}}$$

Hence, if B is taken equal to or greater than $\frac{H}{\sqrt{S_C - C}}$, no tension will be developed anywhere in the base of dam.

For no sliding, the frictional resistance $\mu(W-U)$ should be equal to or more than the horizontal force, i.e. P

$$\therefore \qquad \mu(W-U) \geq P$$

$$\Rightarrow \qquad \mu\left[\frac{1}{2}BHS_c\gamma_w - \frac{1}{2}C\gamma_w.HB\right] \geq \frac{1}{2}\gamma_w H^2$$

$$\Rightarrow \qquad \mu BS_C - CB \geq H$$

$$\Rightarrow \qquad B \geq \frac{H}{\mu(S_C - C)}$$

Now, for no tension case, $B = \dfrac{H}{\mu(S_C - C)} = \dfrac{30}{0.8(2.65-1)} = 22.73$ m

Hence adopt $B = 22.73$ m

(ii)

1. **Flexibility:** It is defined as the ratio of rate of the change of discharge of the outlet to the rate of the change of discharge of the distributary channel.

$$F = \frac{da/q}{dQ/Q}$$

where, F = flexibility of the outlet, q = discharge passing through the outlet, Q = discharge in the distributary channel.

2. **Proportionality:** The outlet is said to be proportional when rate of change of outlet discharge is equal to the rate of change of channel discharge. In other words, the outlet is 'Proportional' when 'Flexibility' is equal to unity.

So for a proportional outlet,

$$F = \frac{m}{n} \times \frac{y}{H} = 1$$

$$\Rightarrow \qquad \frac{H}{y} = \frac{m}{n}$$

where H = head acting on the outlet, y = depth of water in distributary.

m = outlet index, n = channel index.

3. **Setting:** Ratio $\dfrac{H}{y}$, i.e ratio of depth of sill level of the outlet below the FSL of the distributary to the full supply depth of the distributary is known as Setting.

4. **Sensitivity:** It is ratio of the rate of discharge through the outlet to the rate of change of water level of the distributary, referred to the normal depth of the channel.

$$\text{Sensitivity, } S = \frac{dq/q}{dG/y}$$

Q.3 (b) Solution:

(i) Theoretical super elevation of main line, e_T is given by:

$$e_T = \frac{GV^2}{127.R}$$

where V is permissible speed on main line

$$\therefore \quad e_T = \frac{1.75\times(70)^2}{127\times\left(\frac{1750}{3}\right)} = 0.1157\text{ m} = 11.57\text{ cm}$$

Now, $$e_{Theor} = e_{act.} + \text{C.D.}$$

where e_{act} is actual cant and CD is cant deficiency.

$$11.57 = e_{act.} + 7.5$$

$$\Rightarrow \quad e_{act.} = 4.07\text{ cm}$$

Now, $$(e_{act.})_{\text{Branch line}} = (e_{act.})_{\text{Main line}}$$

$$\Rightarrow \quad (e_{act.})_{\text{Branch line}} = -4.07\text{ cm}$$

Now, for branch line,

$$\Rightarrow \quad e_{Theor} = e_{act.} + \text{C.D.}$$

$$e_{Theor} = -4.07 + 7.5$$

$$\Rightarrow \quad \frac{GV^2}{127.R} = 3.43\text{ cm}$$

where V is permissible speed on branch line

$$\therefore \quad \frac{1.75\times V^2}{127.\left(\frac{1750}{5^\circ}\right)} = \frac{3.43}{100}$$

$$\Rightarrow \quad V = 29.52\text{ kmph}$$

(ii)

- Basic runway take-off length, L_T = 2200 m
- Basic runway landing length, L_L = 2000 m
- Elevation aerodrome = 350 m
- Reference temperature, T_R = 25° C
- Standard temperature, T_S = 13° C
- Runway slope, g = 0.5%

1. **Correction to runway take-off length.**

Correction for elevation:

$$\text{Correction for elevation} = \frac{7}{100}\times\frac{350}{300}\times 2200 = 179.67 \text{ m}$$

$$\text{Corrected length, } L_1 = 2200 + 179.67 = 2379.67 \text{ m}$$

Correction for temperature:

$$\text{Correction for temperature} = \left(\frac{T_R - T_S}{100}\right)\times L_1 = \left(\frac{25^\circ - 13^\circ}{100}\right)\times 2379.67 = 285.56 \text{ m}$$

$$\text{Corrected length, } L_2 = 2379.67 + 285.56 = 2665.23 \text{ m}$$

Check:

$$\text{Total correction (in percentage)} = \frac{(179.67+285.56)}{2200}\times 100 = 21.15\% < 35\% \text{ (OK)}$$

Correction for gradient:

$$\text{Correction for gradient} = \frac{20}{100}\times\frac{0.5}{1}\times 2665.23 = 266.523 \text{ m}$$

$$\text{Final runway take-off length} = 2665.23 + 266.523$$

$$= 2931.75 \text{ m} \simeq 2932 \text{ m (say)} \qquad \text{... (i)}$$

2. **Correction to runway landing length ,**

Correction for elevation:

$$\text{Correction for elevation} = \frac{7}{100}\times\frac{350}{300}\times 2000 = 163.33 \text{ m}$$

Corrected runway landing length = 2000 + 163.33 = 2163.33 m **... (ii)**

- No corrections are applied to landing length for temperature & gradient.
- Actual runway length to be provided will be greater of (1) & (2).

Therefore, Actual runway length = 2932 m

Q.3 (c) Solution:

(i) Since side slope is 1 : 1, $\theta = 45°$

Now,
$$\frac{\tau_c^i}{\tau_c} = \sqrt{1-\frac{\sin^2\theta}{\sin^2\phi}} = \sqrt{1-\frac{\sin^2 45°}{\sin^2 50°}} = 0.38$$

Therefore, minimum shear stress required to dislodge the grain on side slope is given by

$$\tau_c' = 0.38\,\tau_c$$

Hence for stability, shear stress actually going to be generated on the slopes of a channel of given R and S must be less than or equal to 0.38 τ_c.

But shear stress actually going to be generated on the side slopes of a channel of given R and S is

$$\tau'_o = 0.75\,\gamma_w RS$$

$$\Rightarrow \quad 0.75\,\gamma_w RS \leq 0.38\,\tau_c$$

But $$\tau_c = 0.56\,\gamma_w\, d\,(G_S - 1) \qquad \{\text{Given: } G_S = 2.65\}$$

$$\Rightarrow \quad \tau_c = 0.056 \times (2.65 - 1) \times d \times \gamma_w$$

$$= 0.924\,\gamma_w d$$

$$\therefore \quad 0.75\,\gamma_w RS \leq 0.38 \times 0.0924 \times \gamma_w \times d$$

$$\Rightarrow \quad R \times \frac{1}{1500} \leq \frac{0.38 \times 0.0924}{0.75} \times (0.05) \qquad (\because d = 0.05 \text{ m})$$

$$\Rightarrow \quad R \leq 3.5 \text{ m}$$

Let us assume,

Depth of the section, d = 2.8 m and find out b for the discharge of 50 m^3/s.

Now, $$\text{Area, } A = 2.8 \times (b + 2.8) = 2.8 + 7.84$$

$$\text{Perimeter, } P = b + 2 \times \sqrt{2} \times 2.8 = b + 7.92$$

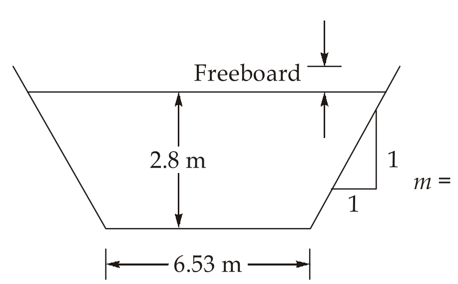
$$\text{Manning's } n = 0.02$$

$$\text{Velocity, } V = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.02} \times R^{2/3} \times \left(\frac{1}{1500}\right)^{1/2}$$

$$\Rightarrow \quad V = 1.29\, R^{2/3}$$

But, $$R = \frac{A}{P} = \frac{2.8b + 7.84}{b + 7.92}$$

Now, $$\text{Discharge, } Q = AV$$



$$\Rightarrow (2.8b + 7.84) \times 1.29 \left(\frac{2.8b + 7.84}{b + 7.92}\right)^{2/3} = 50$$

Solving by trial and error

$$b = 6.532 \text{ m}$$

$$\text{Hydraulic radius, } R = \frac{2.8 \times 6.532 + 7.84}{6.532 + 7.92} = 1.8 < 3.5$$

$\therefore$ Adopt the width as 6.53 m and depth of the canal as 2.8 m.

(ii) Water logging is phenomenon in which productivity of land gets affected due to rise in water table leading to the flooding of root zone of the plants.

Causes of water logging:

1. Over and intensive irrigation of fields
2. Seepage of water through canals.
3. Inadequate surface drainage.
4. Inadequate natural drainage.
5. Obstruction of natural drainage.

Effects of water logging:

1. Inhibiting activity of soil bacterial.
2. Fall in soil temperature.
3. Rise in level of salts in the surface soil.
4. Growth of wild flora.
5. Difficulty in cultivation operations.
6. Breeding of mosquitos and other insects due to wet/humid and warm temperature.

Q.4 (a) Solution:

Design of M40 Concrete:

1. Target mean strength : $f_m = f_{ck} + 1.65 \times \sigma = 40 + 1.65 \times 5 = 48.25 \text{ N/mm}^2$

2. Calculation for w/c ratio:

Maximum water – cement ratio = 0.45 [for severe, RCC]

Adopt water – Cement ratio = 0.4

3. Calculation of water content:

Water content = 0.4

For 20 mm aggregate, maximum water content = 186 kg/m^3 [For 25 to 50 mm slump]

$$\text{Estimated water content for 100 mm slump} = 186+\frac{6}{100}\times 186 \simeq 197 \text{ kg/m}^3$$

[Water content is increased by 3% for every additional 25 mm slump]

As superplasticizer is used, the water content can be reduced up to 20% as above.

Now, arrived water content = $197 \times 0.8 = 157.73 \simeq 158$ kg/m^3

4. Calculation of weight of cement:

Water-cement ratio = 0.4

$$\therefore \quad \text{Cement content} = \frac{158}{0.4} = 395 \text{ kg/m}^3$$

From table, minimum cement content for 'Severe exposure' condition = 250 kg/m^3

As 395 kg/m^3 > 250 kg/m^3 (OK)

5. Calculation for volume of cement and water

$$\text{Volume of Cement} = \frac{395}{3150} = 0.125 \text{ m}^3$$

$$\text{Volume of water} = \frac{158}{1000} = 0.158 \text{ m}^3$$

$$\text{Total Volume } (V_1) = 0.125 + 0.158 = 0.283 \text{ m}^3$$

Taking 1m^3 volume,

$$\text{Remaining volume} = 1 - V_1 = 1 - 0.283 = 0.717 \text{ m}^3$$

6. Finding the volume of coarse aggregate for zone-1 of find aggregate: Table gives the volume of coarse aggregate corresponding to 20 mm size aggregate and fine aggregate (zone : 1) for water cement ratio of 0.5.

In our case, adopted water content ratio (0.4) is less. There should be lesser fine aggregate and hence the volume of coarse aggregate has to be increased.

$\therefore$ for water content of 0.4, coarse aggregate = 0.6 + 0.02 = 0.62

[$\because$ For every decrease of 0.05 in water content ratio from 0.5, we increase the proportion of volume of coarse aggregate as given by table by 0.01]

For pumpable concrete, the amount of coarse aggregate is to be reduced by 10%.

Hence, $\quad$ % of coarse aggregate = 0.62 × 0.9 = 0.56

% of fine aggregate = 1 – 0.56 = 0.44

Mass of coarse aggregate = 0.56 × 0.717 × 2740 = 1100.2 kg $\simeq$ 1100 kg

Mass of fine aggregate = 0.717 × 0.44 × 2740 = 864.42 kg $\simeq$ 864 kg

Final mix proportion for trial mix of 1 m^3 of concrete :

Cement = 395 kg/m^3

Water = 158 kg/m^3

Fine aggregate = 864 kg/m^3 (SSD)

Coarse aggregate = 1100 kg/m^3 (SSD)

Water cement ratio = 0.4

Q.4 (b) Solution:

- Firstly, we have to find maximum speed, $V_{\max}$.
- By railway board, $\quad V_{\max} = 100$ km/h
- By superelevation, $\quad e_{th} = e_{act} + CD$

$$\frac{1.75V_{\max}^2}{127 \times \dfrac{1750}{5^\circ}} = (12 + 7.5) \times 10^{-2}$$

$$V_{max} = 70.38 \text{ km/h}$$

Hence, permissible speed, V_{max} = 70.38 kmph

(i) As per Indian railways, the length of the transition curve is the maximum of the following three values:

1. $$L(\text{in m}) = 0.72\, e_{act} \quad \text{where } e_{act} \text{ is cant provided in mm.}$$
$$= 0.72 \times 120 = 86.4 \text{ m}$$

2. $$L(\text{in m}) = 0.008\,(e_{act})(V_{max})$$

where e_{act} is cant provided in mm and V_{max} is maximum design speed in kmph.

$$= 0.008 \times 120 \times 70.38 = 67.56 \text{ m}$$

3. $$L(\text{in m}) = 0.008\,(CD)(V_{max}) \quad \text{where } CD \text{ is cant deficiency in mm.}$$
$$= 0.008\,(75)\,(70.38) = 42.23 \text{ m}$$

Hence, length of transition curve is to be taken as maximum of above 3 values i.e. 86.4 m (90 m say)

The equation of cubic parabola which is used as transition curve is given by:

$$y = \frac{x^3}{6.R.L}$$

where R is radius of curve.

$$\therefore \quad y = \frac{x^3}{6\left(\frac{1750}{5^\circ}\right).(90)} = \frac{x^3}{189000} \times 100 \text{ cm}$$

Taking offsets at 20 m intervals, we get

$$y_{20m} = \frac{(20)^3}{189000} \times 100 = 4.23 \text{ cm}$$

$$y_{40m} = \frac{(40)^3}{189000} \times 100 = 33.86 \text{ cm}$$

$$y_{60m} = \frac{(60)^3}{189000} \times 100 = 114.28 \text{ cm}$$

$$y_{80m} = \frac{(80)^3}{189000} \times 100 = 270.90 \text{ cm}$$

$$y_{90m} = \frac{(90)^3}{189000} \times 100 = 385.7 \text{ cm}$$

(ii) Based on location, harbours can be classified into the following four categories:

1. **Canal harbour:** The harbour located along the canal for sea and inland navigation is known as Canal Harbour Dredging of canal harbour basins is negligible.
2. **Lake harbour:** The harbour constructed along the shore of a lake is known as lake harbour. In case the lake is large, then the conditions are similar to those of ocean harbour, except that tidal action does not take place in this case.
3. **River harbour:** The harbour constructed along the banks of a river is known as river or estuary harbour. Rivers and estuaries create the main transportation route to join the hinterland and the sea.
4. **Sea harbours:** The harbours located on the coast of a ocean or sea are known as ocean harbours. They are meant for sea going vessels.

Following are the requirements of a commercial harbour:

1. Strong sheds for cargo.
2. To avoid delay, good and quick repair facilities must be avilability.
3. Spacious accommodation for the mercantile marine.
4. Good and sufficient sheltered conditions as loading and unloading are done efficiently in calmer water.

Q.4 (c) Solution:

(i) Various components of a diversion head work are as follow:

1. **Weir or barrage:** The weir is a solid obstruction constructed across the river to raise its water level and divert the water into the canal. Normally, the water level of any perennial river is such that it cannot be diverted to the irrigation canal. The bed level of the canal may be higher than the water level of existing in the river. Hence, to divert the water into the channel, weir helps in raising the water level.
2. **Divide wall:** Divide wall is a long masonry or concrete wall constructed at right angles to the axis of the weir to separate the undersluices from the rest of the weir.

   It controls the eddy current or cross current in front of the canal head and forms a still water pocket so that the suspended silt can settle and scoured later through scouring sluices from time to time.
3. **Undersluices:** Undersluices are the opening provided at the base of the weir or barrage. The main function of the undersluices is to preserve a clear and well defined river channel towards canal head regulator. They also help in scouring the silt deposited on the river bed in the pocket upstream of the canal head regulator.

4. **River works:** River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus to prevent the river from outflanking the works due to change in its course.

5. **Fish ladder:** The fish ladder is provided just by the side of the divide wall for the free movement of fishes. Fish has a tendency to move from upstream part of the river to the downstream part in winters for search of warmth and from downstream part to upstream in monsoons for clearer water. This movement gets obstructed due to construction of weirs and barrages. Hence, to enable the fishes to pass upstream this structure is provided. Fish ladder is a device by which flow energy can be dissipated in such a manner as to provide smooth flow at sufficiently low velocity.

6. **Canal head regulators:** A canal head regulator is provided to serve the following function:
   - It regulates the supply of water entering the canal.
   - It controls the entry of silt in the canal.
   - It prevents the river floods from entering the canal.

7. **Silt regulation works:** The entry of silt into a canal which offtakes from a head work can be reduced by constructing certain special works called silt control works.
   - **Silt excluder:** It is constructed on the bed of river upstream of head regulator. In this type of work, the silt is removed from water before it enters canal.
   - **Silt ejector:** It extracts the silt from the canal water after the silted water has travelled a certain distance in the offtaking canal. These works are therefore constructed on the bed of the canal and a little distance downstream from the head regulator.

(ii) Precautions in using mortar: Following precautions are to be taken while making use of mortar:

(a) **Consumption of mortar:** After preparation, the mortar should be consumed as early as possible. The lime mortar should be consumed within 36 hours after its preparation and it should be kept wet or damp. The cement mortar should be consumed within 30 minutes after adding water and for this reason, it is advisable to prepare cement mortar of one bag of cement at a time. The gauged mortar or composite mortar should be used within 2 hours of the addition of cement.

(b) **Frost action:** The setting action of mortar is affected by the presence of frost. It is therefore advisable to stop the work in frosty weather or to execute it with cement mortar which will set before it tries to freeze.

(c) **Sea water:** In absence of pure water, the sea water may be used with hydraulic lime or cement. It helps in preventing too quick drying of the mortar. However it is not advisable to use sea water in making pure lime mortar or surkhi mortar because it will lead to efflorescence.

(d) **Soaking of building units**: The presence of water in mortar is essential to cause its setting action. Hence, the building units should be soaked in water before mortar is applied. If this precaution is not taken, the water of mortar will be absorbed by the building units and the mortar will become weak.

(e) **Sprinkling of water:** The construction work carried out by mortar should be kept damp or wet by sprinkling water to avoid rapid drying of mortar. The water may be sprinkled for about 7 to 10 days. The exposed surfaces are sometimes covered to give protection against sun and wind.

(f) **Workability:** The mortar should not contain excess water and it should be as stiff as can be conveniently used. The joints should be well formed and the excess mortar from joints should be neatly taken off by a trowel. The surfaces formed by mortar for building units should be even.

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

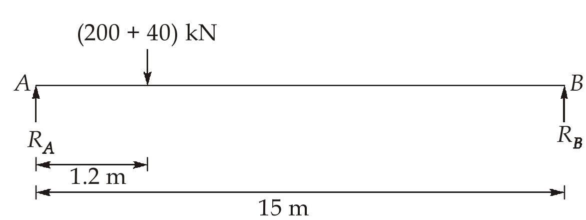
Q.5 (a) Solution:

(i) **Vertical loading**

Calculation of maximum static wheel load:

Maximum static wheel load due to the weight of the crane = $\frac{200}{4} = 50$ kN

Maximum static wheel load due to crab and crane load



$$\sum M_B = 0$$

$$\Rightarrow \quad 240 \times (15 - 1.2) = R_A \times 15$$

$$\Rightarrow \quad R_A = 220.8 \text{ kN}$$

$$\text{Load on each wheel} = \frac{R_A}{2} = 110.4 \text{ kN}$$

Total load due to the weight of the crane and the crane load = 50 + 110.4 = 160.4 kN

To allow for impact, etc. this load should be multiplied by impact factor,

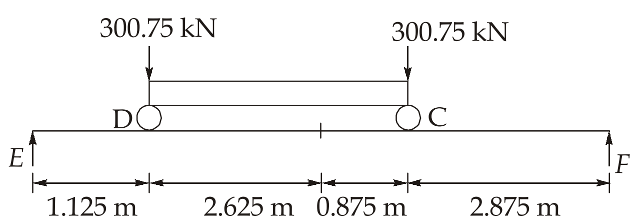
$\therefore$ Design load $= 160.4 \times 1.25 = 200.5$ kN

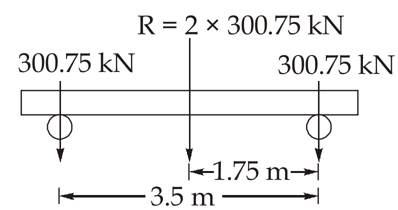
$\therefore$ Factored load on each wheel,

$$W_C = 200.5 \times 1.5 = 300.75 \text{ kN}$$

1. **Maximum bending moment due to vertical load**

Without considering the self weight





To get the maximum bending moment anywhere in the beam, the wheels should be placed such that the resultant load and one of the wheels are equidistant from the centre. The maximum bending moment occurs under the chosen wheel which is calculated as below.

$$R_A + R_B = 2 \times 300.75 \text{ kN}$$

$$\sum M_A = 0$$

$\Rightarrow$ $300.75 \times 1.125 + 300.75 \times 4.625 - R_B \times 7.5 = 0$

$\Rightarrow$ $R_B = 230.575$ kN

$\therefore$ Bending moment at C $= 230.575 \times 2.875 = 662.90$ kN-m

Now, let us find out bending moment due to dead load.

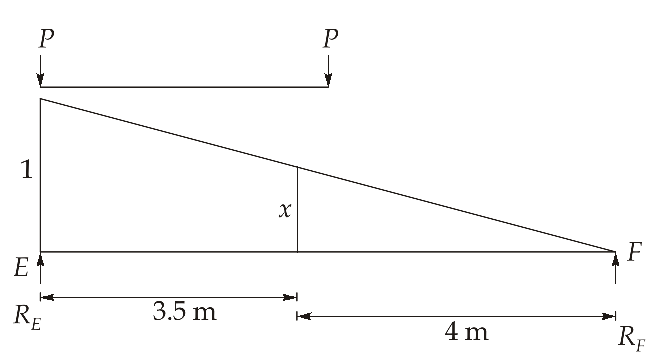
Total dead load $= 1600 + 300 = 1900$ N/m $= 1.9$ kN/m

Factored dead load $= 1.5 \times 1.9 = 2.85$ kN/m

Bending moment due to dead load $= wl^2/8 = 2.85 \times \dfrac{7.5^2}{8} = 20.04$ kN/m

Shear force:

To find the maximum shear force, let's draw ILD for reaction at E



To find maximum shear force, place the loads as shown in figure above.

$$\frac{1}{7.5} = \frac{x}{4}$$

$$\Rightarrow \qquad x = \frac{4}{7.5}$$

$$\therefore \qquad R_A = 1\times300.75+\frac{4}{7.5}\times300.75 = 461.15 \text{ kN}$$

$$\text{Shear force due to dead load} = \frac{wl}{2} = 2.85\times\frac{7.5}{2} = 10.69 \text{ kN}$$

$\therefore$ Maximum vertical shear force = 461.15 + 10.69 = 471.84 kN

Q.5 (b) Solution:

The following factors affect flood hydrograph of a basin:

- **Physiographic factors:**

  Basin Characteristics:

1. Shape	2. Size	3. Slope
4. Nature of Valley	5. Elevation	6. Drainage density

- **Infiltration characteristics:**
  1. Land use and cover
  2. Soil type and geological conditions
  3. Lakes, swaps and other storage
- **Climatic factors**
  1. Precipitation, intensity, duration, magnitude and movement of storm.
  2. Initial loss
  3. Evapotranspiration

There are three limbs/segments of a hydrograph:

1. **Rising limb:** Rising limb is also known as concentration curve. It represents building up of storage in channels. The initial losses and high infiltration losses during early period of storm cause discharge to rise rather slowly.
   - Basin with large slope leads to steep rising limb as well as quick depletion of storage.
   - More the intensity of storm, steeper the rising limb.
   - Elongated catchments have elongated rising limb.
   - Nearly semicircular fan shaped catchments give small length of rising limb.

2. **Crest segment:**
   - It contains the peak flow.
   - Steeper the slope of catchment, more will be the peak flow and vice versa.
   - Fan shaped catchments give high peak value.
   - Larger the catchment area, more will be peak discharge.
   - More the vegetation cover and infiltration losses, less will be the peak discharge.
3. **Recession limb:**
   - Extends from point of inflection at the end of crest segment to commencement of natural ground flow.
   - Large slope of catchment gives rise to quicker depletion of storage and hence a steep recession limb or vice-versa.
   - High drainage density implies steeper recession limb.

Q.5 (c) Solution:

Various type of loads on Roof Truss:

1. **Dead load:** The dead loads on roof truss consists of

   (i) Weight of roof covering: (ii) Weight of purlins;

   (iii) Weight of bracings; and (iv) Self-weight of the trusses.

   (i) Weight of roof covering

Type of covering	Weight per m² of plan area
1. Trafford asbestos sheets	**159 N/m²**
2. 20 gauge CGI sheets	**112.7 N/m²**

   (ii) Weight of purlins: The load due to weight of purlins per square meter of plan area, may be assumed as 70 to 120 N for glazed roofing, 60 to 90 N for G.I. sheeting and 90 to 150 N for A.C. sheeting.

   (iii) Weight of bracings: The load due to the weight of bracings may be assumed as 12 to 15 N/m² of plan area.

   Generally, the load due to self-weight of the truss is estimated from the following empirical expression applicable for pitch equal to 1 in 4 and spacing of 4 m, with corrugated Gl. sheets.

$$w = 10\left(\frac{L}{3}+5\right)$$

   where, w = load per square meter of plan area, due to weight of the truss, in N/m².
   L= span of the truss, in meters.

2. **Imposed load:** IS 875 recommends that for the roofs with slope upto and including 10°, live load measured on plan should be taken as 1500 N/m² if access to roof in

provided, and as 750 N/m² if access to roof is not provided except for the maintenance purpose. For sloping roofs with slope greater than 10°, the live load may be taken as 750 N/m² less 20 N/m² for every degree increase in slope over 10°, subject to a minimum of 400 N/m² of the plan area. For members supporting the roof members and roof purlins, such as trusses, beam, girders etc, live load may be taken equal to 2/3rd of the above load.

3. **Snow load:** IS 875 recommends snow load of 2.5 N/m² per mm depth of snow. No snow load may be considered if the slopes are greater than 50°.

4. **Wind load:** The load due to wind is one of the most important loads to be considered in the design of roof trusses and other types of pitched roofs. The design wind pressure p, as per IS code 875 Part-3 2015 is given by

$$p_z = 0.6V_z^2 = 0.6(k_1\, k_2\, k_3\, k_4\, V_b)^2$$

where, p_z = Wind pressure at any height zabove MSL

V_b = basic wind speed in m/s at 10 m height

k_1 = Probability factor (or risk coefficient)

k_2 = Terrain, height and structure size factor

k_3 = Topography factor

k_4 = Importance factor

Design wind pressure, $p_d = k_d k_a k_c p_z$

where, k_d = Wind directionality factor

k_a = Area averanging factor

k_c = Combination factor

The wind force F acting in a direction normal to the individual structural element or cladding unit is

$$F = (C_{pe} - C_{pi}) \times A \times p_d$$

Where, C_{pe} = External pressure coefficient

C_{pi} = Internal pressure

Q.5 (d) Solution:

Time (1)	Ordinates of storm hydrograph (m^3/s) (2)	Base flow (3)	Direct Runoff hydrograph (DRH) (4)	Ordinates of 2h - UH $\left(\frac{\text{Column (4)}}{15}\right)$
0	30	30	0	0
2	50	30	20	1.33
4	60	30	30	2
6	120	30	90	6
8	200	30	170	11.33
10	250	30	220	14.67
12	280	30	250	16.67
14	300	30	270	18
16	250	30	220	14.67
18	150	30	120	8
20	80	30	50	3.33
22	50	30	20	1.33
24	30	30	0	0
			$\sum = 1460\ m^3/sec$	

$$\text{Direct runoff depth} = \frac{\text{Volume of DRH}}{\text{Drainage area}}$$

$$\text{Volume of DRH} = (\text{Sum of DRH ordinates}) \times \text{Time of rainfall}$$

$$= 1460 \times 2 \times 60 \times 60 = 10.512 \times 10^6\ m^3$$

Now,
$$\text{Direct runoff depth} = \frac{10.512 \times 10^6\ m^3}{70 \times 10^6 m^2} = 0.150\ m = 15\ cm$$

Thus the ordinates of DRH (Col. 4) are divided by 15 cm to obtain the ordinates of 2h unit hydrograph as tabulated above.

Q.5 (e) Solution:

$$\text{For Fe 410 grade of steel, } f_y = 250\ N/mm^2$$

$$\text{Partial factor of safety, } \gamma_{mo} = 1.10$$

$$\text{Bearing pressure from concrete, } f_b = 0.45\, f_{ck} = 0.45 \times 25 = 11.25\ N/mm^2$$

$$\text{Area of bearing plate required, } A = \frac{500 \times 10^3}{11.25} = 44444.44\ mm^2$$

Width of bearing plate = Width of the concrete pedestate = Thickness of wall = 300 mm

$$\text{Length of the bearing plate, } b = \frac{44444.44}{300} = 148.15 \text{ mm}$$

Let us provide 200 mm × 300 mm bearing plate

$$\text{Area of bearing plate provided} = 60000 \text{ mm}^2$$

$$\text{Thickness of bearing plate, } t = \sqrt{\frac{2.75 \times R \times n^2}{Af_y}}$$

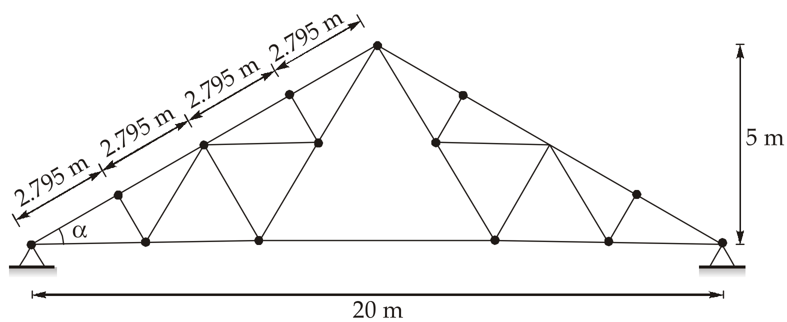
where $n = \frac{b_f}{2} - \frac{t_w}{2} - R_1 = \frac{210}{2} - \frac{12}{2} - 20 = 79 \text{ mm}$

$$\therefore \quad t = \sqrt{\frac{2.75 \times 500 \times 10^3 \times 79^2}{6 \times 10^4 \times 250}} = 23.92 \text{ mm}$$

Provide bearing plate of size 200 × 300 × 25 mm

Q.6 (a) Solution:

Let α be the inclination of the roof with the horizontal.



$$\alpha = \tan^{-1}\frac{5}{10} = 26.57°$$

$$\text{Length of rafter} = \sqrt{10^2 + 5^2} = 11.18 \text{ m}$$

$$\text{Length of each panel } L_0 U_1 = U_1 U_2 = U_2 U_3 = U_3 U_4 = \frac{11.18}{4} = 2.795 \text{ m}$$

(i) Dead load

Dead weight of roofing and coverings = 170 N/m^2

Self weight of purlin = 318 N/m

Total weight of purlin = (318 × 6) N = 1908 N

$$\text{Self weight of truss} = \left(\frac{\text{Span}}{3} + 5\right) \times 10 = \left(\frac{20}{3} + 5\right) \times 10 = 116.67 \text{ N/m}^2$$

Panel length = 2.795 m

Panel length in plan = 2.795 × cos 26.57° = 2.5 m

Load on each intermediate panel due to dead load = (170 + 116.67) × (6 × 2.5) + 1908

= 4300.05 + 1908 = 6208.05 N = 6.21 kN

Load on end panel points of the rafter = $\frac{6.21}{2} = 3.105$ kN

(ii) Live load

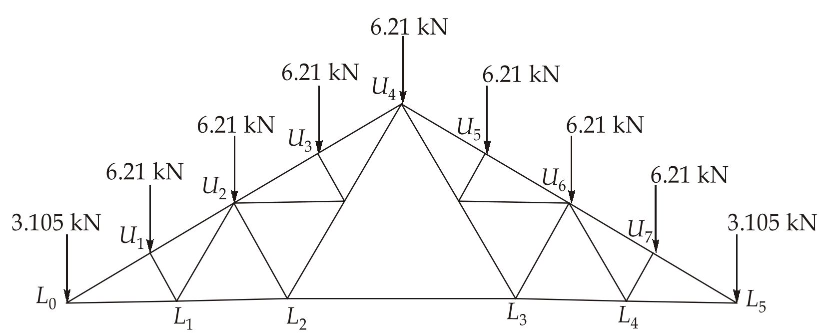
$\alpha = 26.57°$

Let us assume that no access is provided to the roof. The live load is reduced by 20 N/m^2 for each 1° above 10° slope.

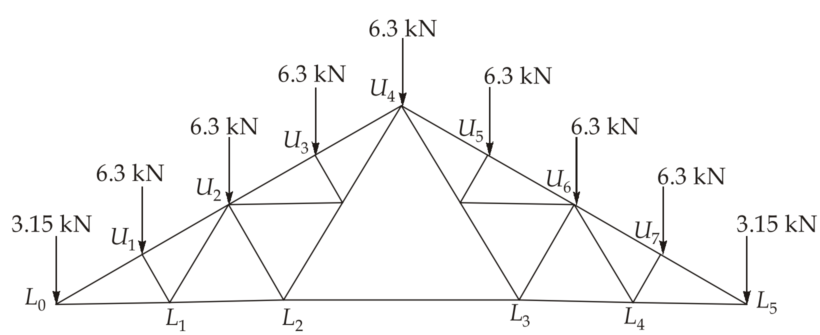
$\therefore$ Live load = 750 – 20 × (26.57° – 10) = 418.6 N/m^2

Load on each intermediate panel = 418.6 × 6 × 2.5 = 6279 N $\simeq 6.3$ kN

Resultant panel point loads due to dead and live loads are shown below.



Dead load at panel points



Live load at panel points

Wind loads:

Calculation of critical wind loads on panel points are as follows.

1. Windward side

$F_1 = (C_{Pe} - C_{Pi}) \times p_d \times A = (-0.8 - 0.2) \times 1.2 \times (6 \times 2.795) = -20.124$ kN

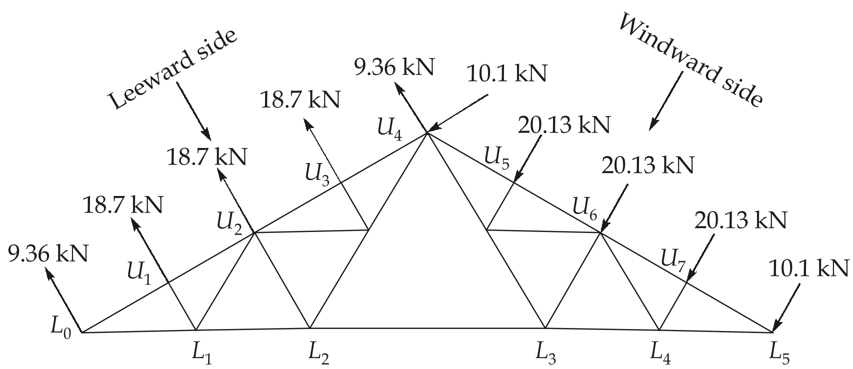
$\simeq -20.13$ kN (intermediate panel points)

$$F_2 = -\frac{20.13}{2} \simeq -10.1\,\text{kN (End panel points)}$$

2. Leeward side

$F_3 = (C_{Pe} - C_{Pi}) \times P_d \times A = (-0.73 - 0.2) \times 1.2 \times (6 \times 2.795) \cong -18.72$ kN (intermediate panel point)

$$F_4 = -\frac{18.72}{2} \simeq -9.36\,\text{kN (End panel points)}$$



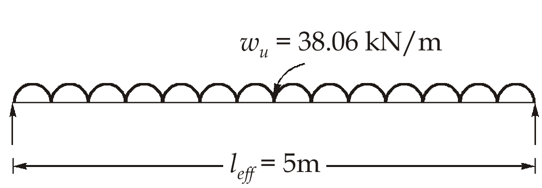
Wind loads at panel points (wind from right)

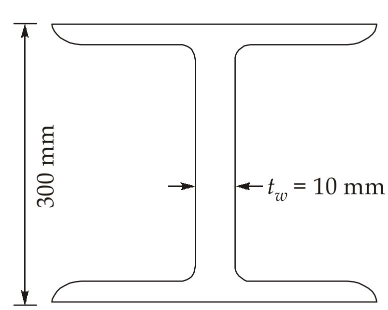
Q.6 (b) Solution:

Time (hrs) (1)	Ordinates of UH (m^3/S) (2)	Ordinates of First Hydrograph 0.5 × (Col. 2) (3)	Ordinates of 2nd Hydrograph 0.8 × (Col. 2) tabulated after 3 hrs. (4)	Ordinates of 3rd Hydrograph 1.2 × (Col. 2) tabulated after 6 hrs. (5)	Ordinates of 4th Hydrograph 1.5 × (Col. 2) tabulated after 9 hrs.	Ordinates of flood Hydrograph
0	0	0	–	–	–	0
1	15	7.5	–	–	–	7.5
2	60	30	–	–	–	30
3	170	85	0	–	–	85
4	350	175	12	–	–	187
5	450	225	48	–	–	273
6	400	200	136	0	–	336
7	280	140	280	18	–	438
8	200	100	360	72	–	532
9	160	80	320	204	0	604
10	120	60	224	420	22.5	726.5
11	90	45	160	540	90	835
12	50	25	128	480	255	888
13	30	15	96	336	525	972
14	15	7.5	72	240	675	**994.5**
15	0	0	40	192	600	832
16			24	144	420	588
17			12	108	300	420
18			0	60	240	300
19			–	36	180	216
20			–	18	135	153
21			–	0	75	75
22			–	–	45	45
23			–	–	22.5	22.5
24			–	–	0	0

Peak rate of discharge = 994.5 m^3/s.

Q.6 (c) Solution:





Live load = 25 kN/m

Dead load = 0.377 kN/m

$$\text{Total factored load, } w_u = 1.5(25 + 0.377) \text{ kN/m} = 38.06 \text{ kN/m}$$

$$\text{Effective length, } l = 5 \text{ m}$$

$$\therefore \text{ Maximum factored bending moment, } M_u = \frac{w_u l^2}{8} = \frac{38.06 \times 5^2}{8} = 118.94 \text{ kN-m}$$

1. Check for bending:

Plastic section modulus required,

$$Z_{PZ,\text{required}} = \frac{M_u}{\frac{f_y}{\gamma_{mo}}} = \frac{118.94 \times 10^6}{\frac{250}{1.1}} = 523336 \text{ mm}^3$$

Now, $Z_{eZ,\text{ available}} = 488.9 \times 10^3 \text{ mm}^3$

Shape factor $= 1.12$

So, $Z_{PZ,\text{ available}} = 488.9 \times 10^3 \times 1.12 = 547.568 \times 10^3 \text{ mm}^3$

$\therefore$ $Z_{PZ,\text{ available}} > Z_{PZ,\text{required}}$ (OK)

$$\text{Design bending moment, } M_d = \beta_b Z_P\left(\frac{f_y}{\gamma_{mo}}\right) \leq 1.2 Z_e\left(\frac{f_y}{r_{\text{mo}}}\right) \quad (\beta_b = 1 \text{ for plastic section})$$

$$\Rightarrow \quad M_d = 1.0 \times 547568 \times \frac{250}{1.1} \leq 1.20 \times 488.9 \times 10^3 \times \frac{250}{1.1}$$

$$= 124.45 \times 10^6 \text{ N-mm} \leq 133.34 \times 10^6 \text{ N-mm (OK)}$$

(ii) Check for shear

$$\text{Maximum shear force, } V_u = \frac{w_u l}{2} = \frac{38.06 \times 5}{2} = 95.15 \text{ kN}$$

$$\text{Design shear strength, } V_d = \frac{f_y}{\sqrt{3}\gamma_{mo}}(ht_w) = \frac{250}{\sqrt{3} \times 1.1} \times (300 \times 10) N = 393.65 \text{ kN}$$

$$0.6\ V_d = 0.6 \times 393.65 = 236.19 \text{ kN}$$

$\because$ $V_u < 0.6\ V_d$

$\therefore$ Section is safe and it is low shear case.

(c) **Check for deflection**

Maximum deflection of the beam due to UDL,

$$\delta = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384} \times \frac{(25 + 0.377) \times (5000)^4}{2 \times 10^5 \times 7332.9 \times 10^4} = 14.082 \text{ mm}$$

As per IS 800 : 2007, the maximum deflection for beam not susceptible to cracking should,

$$\delta_{\text{max}} = \frac{l}{300} = \frac{5000}{300} = 16.67 \text{ mm} > 14.082 \text{ mm} \qquad \text{(OK)}$$

Hence the beam is safe in flexure, shear and deflection.

Q.7 (a) Solution:

(i) Infiltration characteristics of a soil at a given location can be estimated by following methods:

1. Using flooding- type infiltrometers.
2. Measurement of subsidence of free water in a large basin or pond.
3. Rainfall simulator
4. Hydrograph analysis.

1. Flooding-type infiltrometer:

Flooding-type infiltrometers are experimental devices used to obtain data relating to variation of infiltration capacity with time. Two types of flooding type infiltrometers are in common use. They are (1) Tube-type (or simple) infiltrometer, and (2) Double ring infiltrometer.

- **Simple (Tube type) infiltrometer:** This is a simple instrument consisting essentially of a metal cylinder 30 cm in diameter and 60 cm long, open at both ends. The cylinder is driven into the ground to a depth of 50 cm.
- **Double-ring infiltrometer:** This the most commonly used infiltrometer which is designed to overcome the basic drawback of the tube infiltrometer, viz. the tube area is not representative of the infiltrating area.

2. Rain fall simulator: Rainfall simulator-type infiltrometer gives lower values than flooding type infiltrometers. This is due to effect of the rainfall impact and turbidity of the in otherwise pure rainfall.

3. Hydrograph analysis: Reasonable estimation of the infiltration capacity of a small watershed can be obtained by analysis of runoff hydrographs and corresponding rainfall records. If sufficiently good rainfall records and runoff hydrographs corresponding to isolated storms in a small watershed with fairly homogenous soils are available, the water-budget equation can be applied to estimate the abstraction by infiltration. In this, the evapotranspiration losses are estimated by knowing the land cover/use of the watershed.

(ii)

- **ϕ-index:** It is the average rainfall above which the rainfall volume is equal to the runoff volume. The ϕ-index is derived from the rainfall hyetograph with the knowledge of the resulting runoff volume. ϕ-index is the average infiltration rate during the period of rainfall excess. Rainfall excess is that rainfall which contribute to runoff. The period during which such a rainfall excess takes place is called period of rainfall excess. Initial loss is also considered as infiltration. If the rainfall intensity (i) is less than $\phi(i < \phi)$, then

$$f = i$$

(ii)If the rainfall intensity is greater than $\phi (i > \phi)$, then

$$f = \phi\text{-index}$$

Here, f = infiltration rate.

- **W-index**: It is the average infiltration rate during the time period when rainfall intensity exceeds the infiltration capacity rate.
- W-index is a refined version of ϕ-index. In it, initial losses are separated from the total abstractions and an average value of infiltration rate is arrived at.

1. In the first 0.5 hour

$$\text{Infiltration depth, } F_{P1} = \int_0^{0.5} \left(4 + e^{-2t}\right) dt = \left[4t + \left[\frac{e^{-2t}}{-2}\right]\right]_0^{0.5}$$

$$= \left[4t - \frac{e^{-2t}}{2}\right]_0^{0.5} = \left[\left(4 \times 0.5 - \frac{1}{2} . e^{-2\times 0.5}\right) - \left(0 - \frac{e^0}{2}\right)\right]$$

$$= \left[(2 - 0.1839) - (-0.5)\right] = 2.3161 \text{ cm}$$

2. In the second 0.5 hour,

$$\text{Infiltration depth, } F_{P2} = \int_{0.5}^{1} \left(4 + e^{-2t}\right) dt = \left[4t - \frac{1}{2} . e^{-2t}\right]_{0.5}^{1}$$

$$= \left[\left(4 - \frac{1}{2} e^{-2}\right) - \left(4 \times 0.5 - \frac{e^{-2\times 0.5}}{2}\right)\right]$$

$$= \left[(4 - 0.0677) - (2 - 0.1839)\right] = 2.1162 \text{ cm}$$

Q.7 (b) Solution:

(i) Cl. 4. 5. 2 of IS : 800-2007 stipulates that following conditions that should be satisfied in order to use the plastic method of analysis.

1. The yield stress should not be greater than 450 MPa.
2. The stress strain characteristics of the steel used should obey the following conditions, in order to ensure plastic moment redistributions:
   (i) The yield plate (horizontal portion of the stress-strain curve) should be greater than 6 times the yield strain.
   (ii) The ratio of the ultimate tensile stress to the yield stress should be more than 1.2.
   (iii)The elongation on the standard gauge length should be more than 15%.

(iv) The steel should exhibit strain hardening capacity.

3. The members should be hot rolled or fabricated using hot rolled plates.
4. The cross sections of the members not containing plastic hinges should be 'compact' and those of members containing plastic hinges should be plastic.
5. The cross-section should be symmetrical about its axis perpendicular to the axis of the plastic hinge rotation.

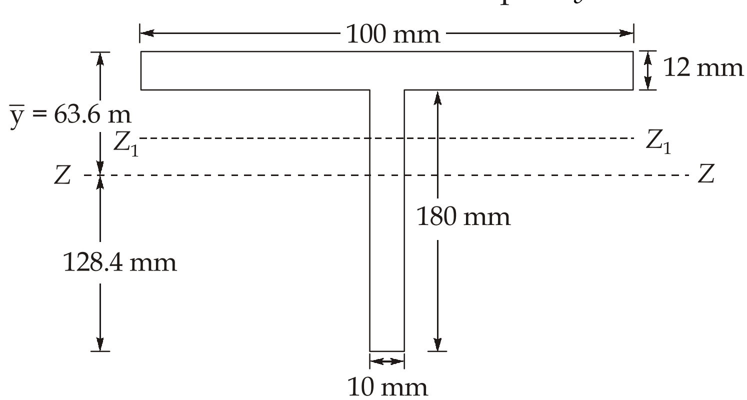
Advantages offered by plastic design.

1. Realization of uniform and realistic factor of safety for all parts of the structure.
2. Simplified analytical procedure and speed in of obtaining the design moments because there is no need to satisfy elastic strain compatibility conditions.
3. Saving of material over elastic methods resulting in lighter structures.
4. Gives some idea of the collapse mode and strength of the structure.
5. No effect due to temperature change, settlement of supports, imperfections, erection method etc. because their only effect is to change the amount of rotation required.

Disadvantages

1. Obtaining collapse load is difficult if the structure is reasonably complicated.
2. There is little saving in column design.
3. Difficult to design for fatigue.
4. Lateral bracing requirements are more stringent than for elastic design.
5. Calculation for elastic deformations require careful considerations.

(ii) Let the distance of the neutral axis from the top be $\bar{y}$.



Now, $$\bar{y} = \frac{100\times12\times\frac{12}{2}+180\times10\times(90+12)}{100\times12+180\times10} = 63.6 \text{ mm}$$

Moment of inertia about Z -Z axis

$$I_{ZZ} = 100 \times \frac{12^3}{12} + 100 \times 12 \times (63.6 - 6)^2 + 10 \times \frac{180^3}{12} + 10 \times 180 \times (102 - 63.2)^2$$
$$= 11.51 \times 10^6 \text{ mm}^4$$

Elastic section modulus

$$Z_{ZZ} = \frac{I_{ZZ}}{y_{\max}} = \frac{11.51 \times 10^6}{(128.4)} = 89641.74 \text{ mm}^3$$

Let $Z_1 Z_1$ be the axis that divides the total area into two equal areas at a distance y_1 from the top.

Total area of the section, $A = 100 \times 12 + 180 \times 10 = 3000 \text{ mm}^2$

Thus, $100 \times 12 + (y_1 - 12) \times 10 = \frac{3000}{2}$

$\Rightarrow$ $y_1 = 42$ mm

$\bar{y}_1$ = distance of the CG of the area above the equal-area axis.

$$= \frac{100 \times 12 \times \left(42 - \frac{12}{2}\right) + (42 - 12) \times 10 \times \left(\frac{42 - 12}{2}\right)}{100 \times 12 + 10 \times 30} = 31.8 \text{ mm}$$

$\bar{y}_2$ = distance of the CG of the area below the equal-area axis.

$$= \frac{192 - 42}{2} = 75 \text{ mm}$$

Now, plastic section modulus, $Z_p = \frac{A}{2}(\bar{y}_1 + \bar{y}_2) = \frac{3000}{2} \times (31.8 + 75) = 160200 \text{ mm}^3$

Shape factor, $\frac{Z_P}{Z_e} = \frac{160200}{89641.74} = 1.787 \simeq 1.79$

Plastic moment capacity, $M_P = Z_P f_y = 160200 \times 250 \times 10^{-6} = 40.05$ kN-m

Q.7 (c) Solution:

(i) Rainfall is a constant at a rate of 8 cm/hr, out of which, infiltration of takes place at a constant rate of 1.5 cm/hr. The remaining water appears as excess rain.

$\therefore$ Excess rain $= (8 - 1.5) \times \frac{90}{60} = 9.75$ cm

Total excess rainfall volume,

$$R_e = \left(\frac{9.75}{100}\right) \times 500 \times 10^4 \text{ m}^3 = 487500 \text{ m}^3$$

Since the peak percentage of distribution graph is based on 10 minutes interval. The volume of excess rainfall (R_e) must, first of all be converted into discharge in m^3/s as:

$$R_e \text{ (in m}^3\text{/s)} = \frac{487500}{10\times60} = 812.5 \text{ m}^3\text{/s}$$

The peak percentage from the distribution graph, based on 10 minutes unit duration = 18% (given)

$\therefore$ Peak rate of runoff = $812.5 \times 0.18 = 146.25 \text{ m}^3\text{/s}$

Hence, the maximum runoff rate = $146.25 \text{ m}^3\text{/s}$.

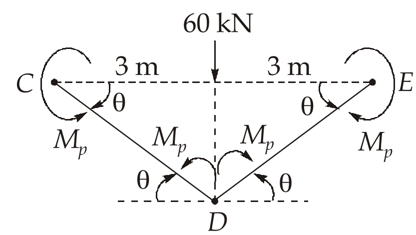
(ii)

The possible locations of plastic hinges are *B, C, D* and *E*. The number of possible plastic hinges, $N = 4$,

Degree of redundancy, $r = 4 - 3 = 1$

Number of possible independent mechanisms, $n = N - r = 4 - 1 = 3$

Beam mechanism for span *CE*



External work done = $60 \times 3\theta = 180\,\theta$

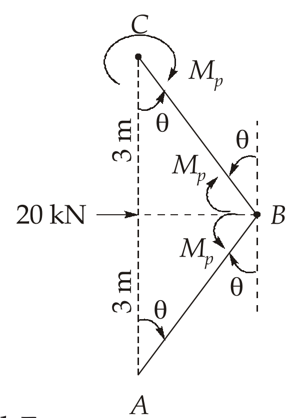
Internal work done = $M_p\theta + M_p\,2\theta + M_p\theta = 4\,M_p\,\theta$

By principle of virtual work,

$$4M_p\theta = 180\,\theta$$

$$\Rightarrow \quad M_p = 45 \text{ kN-m}$$

Beam mechanism for span *AC*



External work done = $20 \times 3\theta = 60\,\theta$

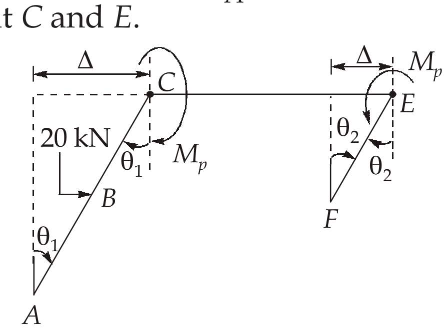
Internal work done = $M_p\theta + M_p\,(2\theta) = 3M_p\theta$

By principle of virtual work, $3M_p\theta = 60\,\theta$

$$\Rightarrow \quad M_p = 20 \text{ kN-m}$$

Sway mechanism

A sway mechanism will be formed with plastic hinges at *C* and *E*.



$$\Delta = 6\theta_1 = 4\theta_2$$

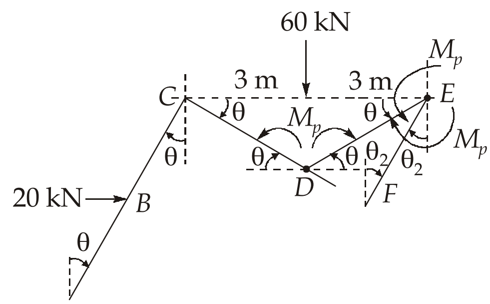
$$\Rightarrow \quad \theta_2 = 1.5\,\theta_1$$

$$\text{External work done} = 20 \times 3\theta_1 = 60\,\theta_1$$

$$\begin{aligned}\text{Internal work done} &= M_p\,\theta_1 + M_p\,\theta_2 \\ &= M_p\,\theta_1 + 1.5\,M_p\,\theta_1 \\ &= 2.5\,M_p\,\theta_1\end{aligned}$$

By the principle of virtual work, 2.5 $M_p\theta_1 = 60\ \theta_1$

$\Rightarrow$ $M_p = 24$ kN-m



Combined mechanism (sway and span *CE*)

External work done = $60 \times 3\theta + 20 \times 3\theta = 240\theta$

Internal work done = $M_p \times 2\theta + M_p(\theta + \theta_2)$

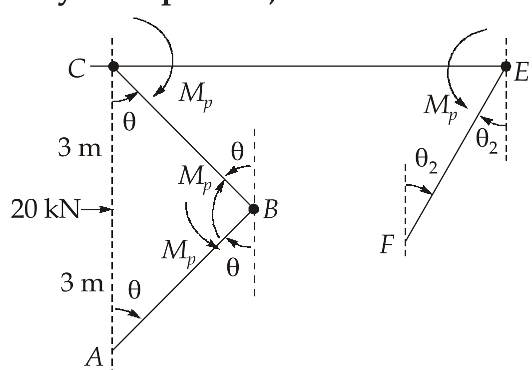
$= 2M_p\theta + M_p(\theta + 1.5\theta)$

$= 4.5\ M_p\theta$

By the principle of virtual work, 4.5 $M_p\theta = 240\ \theta$

$\Rightarrow$ $M_p = 53.33$ kN-m

Combined mechanism (sway and span *AC*)



$$\text{External work done} = 20 \times 3\theta = 60\ \theta$$

$$\text{Internal work done} = M_p\, 2\theta + M_p\, \theta + M_p\, \theta_2 = 3M_p\theta + M_p\theta_2$$

$$= 3M_p\theta + 1.5\ M_p\,\theta = 4.5\ M_p\theta$$

By the principle of virtual work, 4.5 $M_p\theta = 120\ \theta$

$\Rightarrow$ $M_p = 26.67$ kN-m

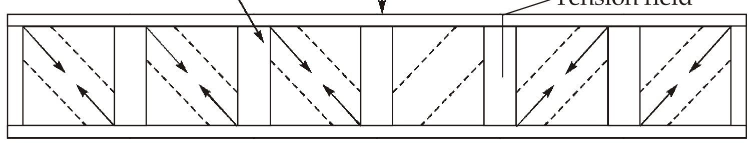
Therefore the section has to have $M_p = 53.33$ kN-m

Q.8 (a) Solution:

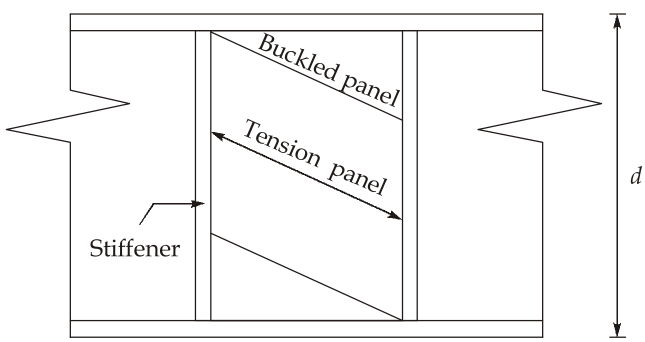
(i) The shear capacity of the web (in plate girders) has two components, namely, strength before onset of buckling and strength after buckling. As the shear load is increased on a stiffened web panel, the web panel buckles. This load does not indicate the maximum shear capacity of the web. The load can still be increased and the web panel continues to carry further load relying on the tension field action part of the buckled web which takes the load in tension.

This tension member action is across the web panel in an inclined direction to the web panel diagonal as shown in figure. At this stage, the girder acts like a N-turns with the compression forces being carried by the flanges and the intermediate stiffeners and the buckled web resisting the tension force.

This additional reserve strength is termed as 'tension field action'. If no intermediate stiffeners are present or their spacing is large, it is not possible for tension field action to take place and the shear capacity is restricted to the strength before buckling. The stiffener spacing influences both buckling and post buckling behavior of the web under shear.



Tension field in individual sub panel



Panel showing tension field

(ii)

(a) Determination of requirement of stiffeners

$$\frac{d}{t} = \frac{1500}{15} = 100, \ \frac{c}{d} = \infty \text{ (assuming no stiffeners have been provided)}$$

From table, let us find out shear buckling strength (τ_b) of web which comes out to be,

$$\tau_b = 95.7 \text{ N/mm}^2$$

Hence, shear strength of the web = $dt\,\tau_b = 1500 \times 15 \times \frac{95.7}{10^3}$ kN

$$= 2153.25 \text{ kN} < 2500 \text{ kN}$$

Thus, shear strength of the web is less than the applied factored shear.

Hence, stiffener is required.

Shear stress induced in web, $\tau_b = \frac{2500 \times 10^3}{1500 \times 15} = 111.11 \text{ N/mm}^2$

From table for $d/t = 100$, maximum c/d corresponding to this strength is 1.6.

Hence spacing of stiffness = 1.6 × 1500 = 2400 mm

Hence stiffeners at 2400 mm interval should be provided.

(b) Thickness of the web without stiffeners, $\tau_b = 111.11\ \text{N/mm}^2$

From table, for no stiffeners (i.e. $\frac{c}{d} = \infty$), let's find out d/t

$$\frac{116.9-109.8}{85-90} = \frac{116.9-111.11}{85-(d/t)}$$

$$\Rightarrow \quad \frac{d}{t} = 89.077$$

$$\therefore \quad t = \frac{1500}{89.077} = 16.84\ \text{mm}$$

Hence, provide a web thickness of 17 mm for no requirement of stiffeners.

Q.8 (b) Solution:

(i)

Time (hrs) (1)	4 hr UH ordinates (cumecs) (2)	Imaginary offsetted S-curve (Shifted t_1 = 4 hrs) (cumecs) (3)	S-curve ordinates (cumecs) (4)	S-curve legged by t_2 = 5 hrs (cumecs) (5)	Difference (4) – (5) (6)	Required ordinates of 5 hrs UH COL. (6) × 4/5 (7)
0	0	–	0	–	0	0
1	6	–	6	–	6	4.8
2	36	–	36	–	36	28.8
3	66	–	66	–	66	52.8
4	91	0	91	–	91	72.8
5	106	6	112	0	112	89.6
6	93	36	129	6	123	98.4
7	79	66	145	36	109	87.2
8	68	91	159	66	93	74.4
9	58	112	170	91	79	63.2
10	49	129	178	112	66	52.8
11	41	145	186	129	57	45.6
12	34	159	193	145	48	38.4
13	27	170	197	159	38	30.4
14	23	178	201	170	31	24.8
15	17	186	203	178	25	20
16	13	193	206	186	20	16
17	10	197	207	193	14	11.2
18	06	201	207	197	10	08
19	04	203	207	201	06	4.8
20	01	206	207	203	04	3.2
21	0	207	207	206	01	0.8
22	0		207	207	0	0

(ii)

(a) **Stream Density:** The stream density of a drainage basin may be expressed by relating the number of streams to the area drained. If N_s is the number of streams in the basin, and A is the total area, then the stream density D_s can be expressed as;

$$D_s = \frac{N_s}{A}$$

(b) **Drainage Density:** The drainage density is expressed as the length of steam per unit area. Let D_d represents the drainage density, L is the total length of the perennial and intermittent streams in the basin, and A is the basin area; then

$$D_d = \frac{L}{A}$$

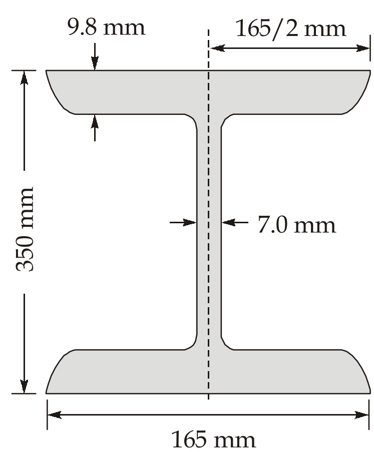
Q.8 (c) Solution:

(i) For Fe 410 grade steel, $f_u = 410\ \text{N/mm}^2$, $f_y = 250\ \text{N/mm}^2$

Section classification:

Outstand of flange, $$\frac{b}{t_f} = \frac{165/2}{9.8} = 8.42 < 9.4\epsilon (= 9.4)$$

$$\left[\because \epsilon = \sqrt{\frac{250}{f_y}} = \sqrt{\frac{250}{250}} = 1\right]$$



Flange is plastic

Now, $$d = h - 2(t_f + R_1) = 325 - 2(9.8 + 16) = 273.4\ \text{mm}$$

$$\frac{d}{t_w} = \frac{273.4}{7} = 39.06 < 84\epsilon\ (= 84)\ (\because \epsilon = 1)$$

Web is plastic.

∴ Section is plastic. Also it is given that it is low shear case,

$\therefore$ Design bending strength, $M_d = \beta_b Z_{pz} \dfrac{f_y}{\gamma_{m0}} \leq 1.2 Z_e \dfrac{f_y}{\gamma_{m0}}$

$$\beta_b = 1.0 \text{ for plastic section.}$$

$$M_d = 1.0 \times 687.88 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 156.34 \text{ kN-m}$$

$$< 1.2 \times 607.7 \times 10^3 \times \frac{250}{1.1} \times 10^{-6} = 165.736 \text{ kN-m} \quad \text{(OK)}$$

$\therefore$ $M_d = 156.34$ kN-m

(ii) 8% risk means that there is a probability of 0.08 for the design flood to occur at least once in successive 20 years. In other words, for 0.92 (i.e. 92%) probability, the flood should not occur.

Now, Risk, $R = 1 - (1-p)^n$

$\because$ $R = 0.08$

$\therefore$ $0.08 = 1 - (1 - p)^{20}$

$\Rightarrow$ $p = 4.1604 \times 10^{-3}$

$\therefore$ Return period, $T = \dfrac{1}{p}$

$\Rightarrow$ $T = \dfrac{1}{4.1604 \times 10^{-3}} = 240.36$ years

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