



**MADE EASY**  
Leading Institute for ESE, GATE & PSUs

Detailed Solutions

**ESE-2026**  
**Mains Test Series**

**Civil Engineering**  
**Test No : 11**

Section - A

1. (a) Solution:

Given:

⇒

$$d_w = 25 \text{ cm}$$

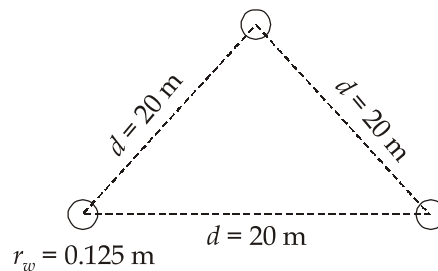
$$r_w = 0.125 \text{ m}$$

$$k = 25 \text{ m/day}$$

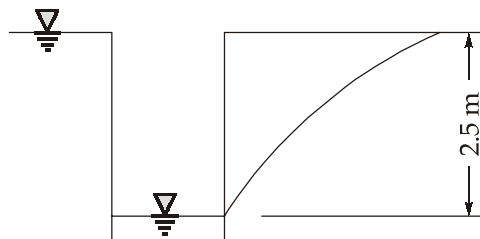
$$S = 2.5 \text{ m}$$

$$D = 15 \text{ m}$$

$$R = 350 \text{ m}$$



(a) When cones of depression do not intersect.



Discharge through each well,

$$Q = \frac{2\pi kDS}{\log_e\left(\frac{R}{r_w}\right)} = \frac{2\pi \times 25 \times 15 \times 2.5}{\log_e\left(\frac{350}{0.125}\right)} = 742.12 \text{ m}^3/\text{d}$$

Total discharge from three wells.

$$= 3 \times Q = 3 \times 742.12 = 2226.36 \text{ m}^3/\text{d}$$

**(b) When cones of depression of wells intersect**

Discharge through each well,

$$Q' = \frac{2\pi kDS}{\log_e\left(\frac{R^3}{r_w d^2}\right)} = \frac{2\pi \times 25 \times 15 \times 2.5}{\log_e\left(\frac{350^3}{0.125 \times 20^2}\right)}$$

$$= 431.165 \text{ m}^3/\text{d}$$

$\therefore$  Total discharge from three wells =  $3 \times Q' = 1293.496 \text{ m}^3/\text{d}$

**(c) % Change in discharge under interference through each well,**

$$\Delta Q = \frac{(Q - Q') \times 100}{Q} = \frac{(742.12 - 431.165) \times 100}{742.12}$$

$$= 41.90\%$$

**1. (b) Solution:**

(i) Power of sound,  $P = 0.0015 \text{ W}$

At distance of 8 m from source,

Intensity of sound,  $I = \frac{P}{4\pi r^2}$  where  $r$  is distance from source

$$= \frac{0.0015}{4\pi \times 8^2} = 1.865 \times 10^{-6} \text{ Watt/m}^2$$

Now, sound intensity level,  $L = 10 \log_{10}\left(\frac{I}{10^{-12}}\right) = 10 \log_{10}\left(\frac{1.865 \times 10^{-6}}{10^{-12}}\right)$

$$= 62.707 \text{ dB}$$

(ii) Now,  $L = 20 \log_{10}\left(\frac{P_{rms}}{P_0}\right)$

where  $P_{rms}$  is sound pressure and  $P_0$  is  $20 \mu Pa$

So, 
$$62.707 = 20 \log_{10} \left( \frac{P_{rms}}{20 \mu Pa} \right)$$

$$\Rightarrow P_{rms} = 27313.67 \mu Pa$$

(iii) At 8 m, there are two sounds of 62.707 dB and 50 dB

Now, sound pressure corresponding to 62.707 dB  

$$= 27313.67 \mu Pa$$

Also, 
$$50 = 20 \log_{10} \left( \frac{P_{rms}}{20 \mu Pa} \right)$$

$$\Rightarrow P_{rms} = 6324.56 \mu Pa$$

Hence, equivalent sound pressure

$$= \sqrt{(27313.67)^2 + (6324.56)^2} = 28036.36 \mu Pa$$

Now, sound pressure level = 
$$20 \log_{10} \left( \frac{P_{rms}}{20 \mu Pa} \right) = 20 \log_{10} \left( \frac{28036.36}{20} \right)$$
  

$$= 62.93 \text{ dB}$$

**Q.1 (c) Solution:**

Value (i)	Mean (ii)	Deviation (d) (iii) = (i) - (ii)	d <sup>2</sup> (iii) <sup>2</sup>
162° 20' 00"	162° 20' 46.67"	-46.67"	36' 18.09"
162° 21' 20"		+33.33"	18' 31.56"
162° 21' 40"		+53.33"	47' 24.09"
162° 20' 40"		-6.67"	44.49"
162° 19' 40"		-1' 6.67"	74' 4.89"
162° 21' 20"		+33.33"	18' 30.9"
		$\Sigma d = 0$	$\Sigma d^2 = 30^{\circ}15' 34.02''$

$$\text{Mean} = \frac{162^{\circ}20'00'' + 162^{\circ}21'20'' + 162^{\circ}21'40'' + 162^{\circ}20'40'' + 162^{\circ}19'40'' + 162^{\circ}21'20''}{6}$$

$$= 162^{\circ} 20' 46.67''$$

(a) Probable error of a single observation,

$$E_s = \pm 0.6745 \sqrt{\frac{\sum d^2}{n-1}} = \pm 0.6745 \sqrt{\frac{3^\circ 15' 34.02''}{(6-1)}}$$

$$E_s = 32.675''$$

(b) Probable error of the mean,

$$E_m = \frac{E_s}{\sqrt{n}} = \frac{32.675''}{\sqrt{6}} = 13.34''$$

(c) Most probable value of the angle

$$= 162^\circ 20' 46.67'' \pm 13.34''$$

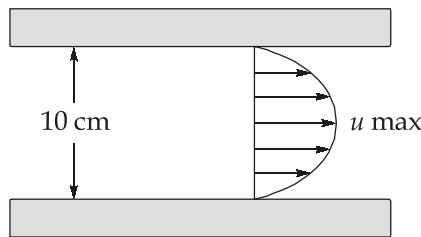
$$= 162^\circ 21' 0.01''$$

1. (d) **Solution:**

1. **Afflux:** Afflux is defined as the rise in the maximum flood level of the river upstream of the barrage as a direct result of the construction of the structure. When a barrage constricts the natural waterway of the river, the water level must rise to generate the head required to pass the discharge through the gates. Generally, an afflux of 1 m to 1.2 m is allowed in the upstream during high floods. It is a critical design parameter as it determines the height of the guide banks and marginal bunds.
2. **Under sluices:** Undersluices are the gate-controlled openings located at the end of the barrage, usually on the side of the canal head regulator. Their crest level is kept lower than the crest level of the main weir bays. Their primary functions are:
  - To preserve a clear, deep channel toward the canal head regulator.
  - To scour away the silt deposited in front of the canal intake.
  - To pass low-season discharge without needing to operate the main barrage gates.
3. **Divide Wall:** The divide wall is a long masonry or concrete wall built perpendicular to the barrage axis, separating the undersluice bays from the main weir bays. It extends upstream beyond the canal head regulator. Its main purposes are:
  - To create a "stilling pond" or quiet pocket of water in front of the canal intake to facilitate silt settlement.
  - To prevent the formation of cross-currents and vortices that could cause heavy scour near the structure.
  - To provide a base for the silt excluder tunnels.

4. **Silt Excluder:** The silt excluder is a diaphragm-like structure constructed on the riverbed within the undersluice pocket. It consists of several horizontal tunnels of varying lengths. The roof of these tunnels is at the same level as the sill of the canal head regulator. The device works on the principle that the bottom layers of water carry the heaviest silt load; these layers are trapped by the tunnels and "excluded" through the undersluices, while the cleaner top water is diverted into the canal.

1. (e) **Solution:**



Shear stress, 
$$\tau = -\frac{1}{2}\left(\frac{\partial P}{\partial x}\right)(H - 2y)$$

For shear stress at plates, 
$$\tau = -\frac{1}{2}\frac{\partial P}{\partial x}H$$

Pressure gradient, 
$$\left(-\frac{\partial P}{\partial x}\right) = \frac{12\mu\bar{v}}{H^2}$$

Average velocity, 
$$\bar{v} = \frac{u_{\max}}{1.5}$$

$\Rightarrow \bar{v} = \frac{1.5}{1.5} = 1 \text{ m/s}$

$\therefore \left(-\frac{\partial P}{\partial x}\right) = \frac{12 \times 2.5 \times 1}{(0.10)^2} = 3000 \text{ N/m}^2/\text{m}$

Shear stress at plates, 
$$\tau = -\frac{1}{2}\left(\frac{\partial P}{\partial x}\right)H$$

$\Rightarrow \tau = -\frac{1}{2}(-3000) \times (0.10) = 150 \text{ N/m}^2$

Pressure difference, 
$$P_1 - P_2 = \frac{12\mu\bar{v}L}{H^2}$$

$\Rightarrow P_1 - P_2 = \frac{12 \times 2.5 \times 1 \times 12}{(0.10)^2} = 36000 \text{ N/m}^2$

$\Rightarrow P_1 - P_2 = 36 \text{ kN/m}^2$

Velocity at 2.5 cm from the plate,

$$u = \frac{1}{2\mu} \left( \frac{-\partial P}{\partial x} \right) (Hy - y^2)$$

$$\Rightarrow u = \frac{1}{2 \times 2.5} (3000) (0.10 \times 0.025 - 0.025^2)$$

$$\Rightarrow u = 1.125 \text{ m/s}$$

## 2. (a) (i) Solution:

Objective of providing extra widening of pavements on horizontal curve are as follows:

1. An automobile has a rigid wheel base and only the front wheels can be turned using steering wheel so, when this vehicle takes a turn to negotiate a horizontal curve, the rear wheels do not follow the same path as that of the front wheels. This phenomenon is called *off tracking*. If inner front wheel takes a path on the inner edge of a pavement at a horizontal curve, inner rear wheel will be off the pavement on the inner shoulder. The off-tracking depends on the length of the wheel base of the vehicle and the turning angle or the radius of the horizontal curve being negotiated.
2. At speeds higher than the design speed, when the super elevation and lateral friction developed are not fully able to counteract the outward thrust due to the centrifugal force, some transverse skidding may occur and the rear wheels may take paths on the outside of those traced by the front wheels on the horizontal curves.
3. In order to take curved path with larger radius and to have greater visibility at curve, the drivers have tendency not to follow the central path of the lane, but to use the outer side at the beginning of a curve.
4. While two vehicles cross or overtake at horizontal curve there is a psychological tendency to maintain a greater clearance between the vehicles, than on straights for increased safety.

Thus, the required extra widening of the pavement at the horizontal curve,  $W_e$  depends on the length of wheel base of the vehicle, radius of the curve negotiated  $R$ , and the psychological factor which is a function of the speed of the vehicle and the radius of the curve.

## 2. (a) (ii) Solution:

For rolling terrain, minimum design speed for National Highway is 65 kmph i.e.,  $V = 65$  kmph, radius of curve ( $R$ ) = 230 m (given), No. of lanes ( $n$ ) = 2, length of wheel base ( $l$ ) = 6.1 m.

Allowable rate of introduction of superelevation (if pavement is rotated about centre line) is 1 in  $N = 1$  in 150.

**Length of Transition curve ( $L_{TC}$ ):**

1. On the basis of change of centrifugal acceleration

$$L = \frac{v^3}{CR}$$

where,

$v$  = design speed (m/s)

$R$  = Radius of curve (m) = 230 m

$$\therefore v = \frac{65}{3.6} = 18.055 \text{ m/s}$$

$C$  = Rate of change of centrifugal acceleration

$$\Rightarrow C = \frac{80}{75 + V(\text{kmph})}$$

$$\Rightarrow C = \frac{80}{75 + 65} = 0.5714 \text{ m/s}^3$$

$$\therefore L = \frac{(18.055)^3}{0.5714 \times 230} = 44.784 \text{ m}$$

2 On the basis of introduction of superelevation

$$L = e(W + W_e) N \text{ (when rotated about inner edge)}$$

$$\text{Extra widening, } W_e = \frac{nl^2}{2R} + \frac{V(\text{kmph})}{9.5\sqrt{R}}$$

$$= \frac{2 \times 6.1^2}{2 \times 230} + \frac{65}{9.5\sqrt{230}} = 0.613 \text{ m}$$

$W_T$  = Total Width of pavement

$$W_T = 7 + 2 \times 1.5 + 0.613 = 10.613 \text{ m}$$

$$\text{Superelevation, } e = \frac{V^2}{225R}$$

$V$  = Design speed (kmph)

$$R = \text{Radius} = 230 \text{ m}$$

$$\therefore e = \frac{65^2}{225 \times 230} = 0.0816 > 0.07$$

Since  $e$  is more than maximum recommended value by IRC, so we will provide

$$e = 0.07 \text{ and check for } f.$$

$$\therefore e + f = \frac{V^2}{127R}$$

$$\Rightarrow 0.07 + f = \frac{65^2}{127 \times 230}$$

$$\Rightarrow f = 0.074 < 0.15 \quad (\text{Ok})$$

$$\therefore \text{Take } e = 0.07$$

$$\therefore L = 0.07 (10 + 0.613) \times 150 = 111.43 \text{ m}$$

3. For plain and rolling terrain, using empirical formula

$$L = \frac{2.7V^2}{R}$$

$$V = \text{Design speed (kmph)}$$

$$\therefore L = \frac{2.7 \times 65^2}{230} = 49.60 \text{ m}$$

Adopt the highest value of above three criteria i.e.,

$$L_{TC} = 111.43 \text{ m} \simeq 112 \text{ m}$$

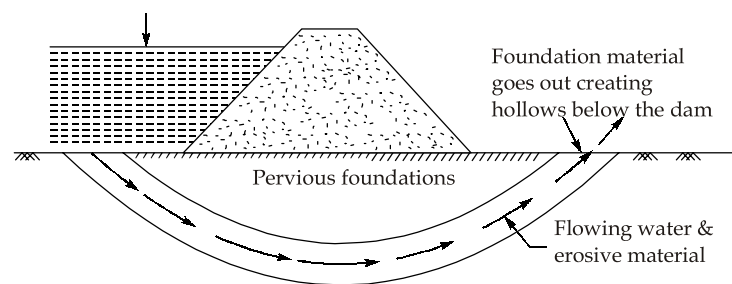
## 2. (b) (i) Solution:

The various causes of failure of earth dams can be grouped into the following three classes:

1. **Hydraulic failures:** The failure under this category, may occur due to the following reasons:
  - (a) By over toppling
  - (b) Erosion of upstream face
  - (c) Cracking due to frost action
  - (d) Erosion of downstream face by gully formation
2. **Seepage failure:** Controlled seepage or limited uniform seepage is inevitable in all earth dams, and ordinarily it does not produce any harm. However, uncontrolled or concentrated seepage through the dam body or through its foundation may lead

to piping or sloughing and the subsequent failure of the dam. Piping is the progressive erosion and subsequent removal of the soil grains from within the body of the dam or the foundation of the dam. Sloughing is the progressive removal of soil from the wet downstream face. More than 1/3rd of the earth dams have failed because of these reasons.

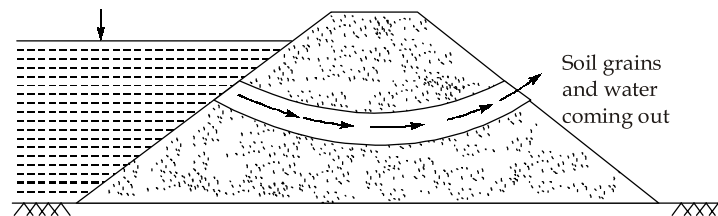
- (a) **Piping through foundations:** Sometimes, when highly permeable cavities or fissures or strata of coarse sand or gravel are present in the foundation of the dam, water may start seeping at a huge rate through them. This concentrated flow at a high gradient, may erode the soil. This leads to increased flow of water and soil, ultimately resulting in a rush of water and soil, thereby creating hollows below the foundation. The dam may sink down into the hollow so formed, causing its failure.



- (b) **Piping through the dam body:** When the concentrated flow channels get developed in the body of the dam, soil may be removed in the same manner as in foundation piping, leading to the formation of hollows in the dam body, and subsequent subsidence of the dam. These flow channels may develop due to faulty construction, insufficient compaction, cracks developed in embankment due to foundation settlement, shrinkage cracks, animal burrows, etc. All these causes can be removed by better construction and better maintenance of the dam embankments.

Piping through the dam body, generally get developed near the pipe conduits passing through the dam body. Contact seepage along the outer side of conduits may either develop into piping, or seepage through leaks in the conduits may develop into piping. This can be avoided by thoroughly and properly compacting the soils near the outlet conduits and by preventing the possibilities of leakage through conduits, but preventing the formation of cracks in the conduits. These cracks in the conduits are caused by differential settlement

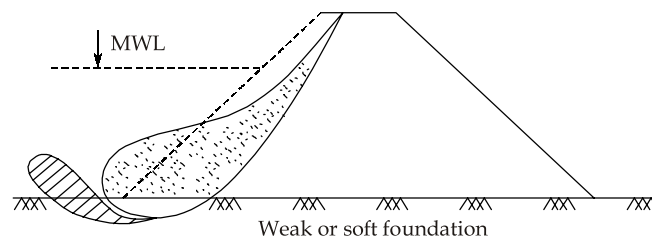
and by overloading from the embankment. When these factors are controlled, automatically, the possibility of piping due to leakage through the conduits is reduced.



(c) **Sloughing of downstream toe:** The process behind the sloughing of the toe is somewhat similar to that of piping. The process of failure due to sloughing starts when the downstream toe becomes saturated and get eroded, producing a small slump or a miniature slide. The miniature slide leaves a relatively steep face which becomes saturated by the seepage from the reservoir and slumps again, forming a more unstable surface. The process continues till the remaining portion of the dam is too thin to withstand the horizontal water pressure, leading to the sudden failure of the dam.

3. **Structural failures:** About 25% of the dam failures have been attributed to structural failures. Structural failures are generally caused by shear failures, causing slides.

(a) **Foundation slide, (i.e. overall stability of the dam):** When the foundation of the earth dams-are made of soft soils, such as silt, soft clay, etc., the entire dam may slide over the foundation. Sometimes, seams of fissured rocks, shales or soft clay, etc. may exits under the foundation, and the dam may slide over some of them, causing its failure. In this type of failure, the top of embankment gets cracked and sub-sides, the lower slope moves outward forming large mud waves near the heel, as shown in figure.

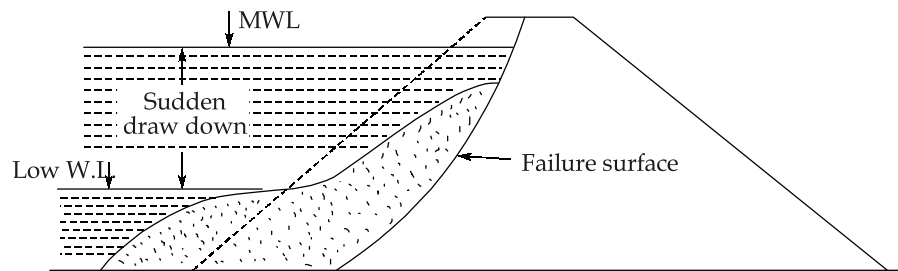


Excessive pore water pressure in confined seams of sand and silt, artesian pressure in abutments, or hydrostatic excess developed due to consideration of clay seams embedded between sands or silts, etc. may reduce the shear strength of the soil, until it becomes incapable of resisting the induced shear

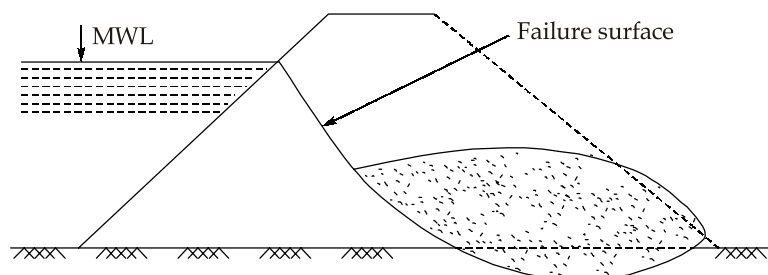
stresses, leading to the failure of the dam foundation without warning. Loose sand foundations may fail by the liquefaction or flow slides.

- (b) *Slide in embankments:* When the embankment slopes are too steep for the strength of the soil, they may slide causing dam failure.

The most critical condition of the slide of the upstream slope is the sudden drawdown of the reservoir; and the downstream slope is most likely to slide, when the reservoir is full. The upstream slope failures seldom lead to catastrophic failures, but the downstream slope failures are very serious. These failures, generally occur due to development of excessive unaccounted pore pressures which may reduce the shearing strength of the soils as explained in the previous article. Many embankments may fail during the process of consolidation, at the time of construction or after the construction.



U/S slope slide due to sudden draw down



D/S slope slide during full reservoir condition

## 2. (b) (ii) Solution:

Given: Particle mean diameter,

$$d = 6 \text{ mm} = 6 \times 10^{-3} \text{ m}$$

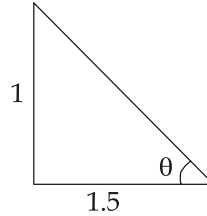
Angle of repose,

$$\phi = 35^\circ$$

Bed width of canal,

$$B = 6 \text{ m}$$

Side slope, 1.5H : 1V



So,

$$\theta = \tan^{-1}\left(\frac{1}{1.5}\right) = 33.69^\circ$$

According to Shield's equation, the critical tractive stress acting on bottom of canal is given by,

$$\tau_c = 0.056\gamma_w d(G_s - 1)$$

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

Now if  $\tau_0$  is the average unit tractive force acting on the bottom of canal, then for no sediment movement from bottom,

$$\tau_0 \leq \tau_c$$

$$\Rightarrow \gamma_w R S \leq 0.056 \times \gamma_w d \times (1.65)$$

$$\Rightarrow \frac{R \times 1}{4500} \leq 0.056 \times 6 \times 10^{-3} \times 1.65$$

$$\Rightarrow R \leq 2.49 \text{ m} \quad \dots(i)$$

Also, if  $\tau_0'$  is the average unit tractive force acting on the sides of the canal, then for no sediment movement from the sides,

$$\tau_0' \leq \tau_c'$$

where  $\tau_c'$  is critical tractive stress acting on side slope of canal.

$$\therefore \tau_c' = 0.056\gamma_w d(G_s - 1) \times \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}}$$

$$\begin{aligned} \Rightarrow \tau'_c &= \tau_c \times \sqrt{1 - \frac{\sin^2 \theta}{\sin^2 \phi}} \\ \Rightarrow \tau'_c &= 0.056 \times \gamma_w \times 6 \times 10^{-3} \times 1.65 \times \sqrt{1 - \left( \frac{\sin 33.69^\circ}{\sin 35^\circ} \right)^2} \\ \Rightarrow \tau'_c &= 1.4106 \times 10^{-4} \times \gamma_w \\ \text{Also, } \tau'_0 &= 0.75 \gamma_w R S_0 \\ \text{So, } \frac{0.75 \times \gamma_w \times R \times 1}{4500} &\leq 1.4106 \times 10^{-4} \times \gamma_w \\ \Rightarrow R &\leq 0.846 \text{ m} \quad \dots(\text{ii}) \\ \text{So, maximum value of R will be minimum of (i) and (ii)} \\ \therefore R_{\max} &= 0.846 \text{ m} \end{aligned}$$

## 2. (c) (i) Solution:

The possible four mechanisms of coagulation are as below:

### 1. Ionic layer compression:

The quantity of ions in water surrounding a colloid has an effect on decay function of electrostatic potential. A high ionic concentration compresses the layer that is composed predominantly of counter ions towards the surface of celluloid. If this layer is sufficiently compressed, then the Van-der Waals forces will be predominant across the entire area of influence, so that net force will be attractive and no energy barriers will exist. An example of ionic layer compression occurs in nature when a turbid stream flows into the ocean. There, the ion content of water increases drastically and coagulation and settling occur. Eventually, deposits are formed from material which was originally so small that it could not have settled without coagulation

### 2. Adsorption and charge neutralization:

Unlike the ionic layer compression, nature rather than quantity of ions is of prime importance in theory of adsorption and charge neutralization. For instance, when alum is used as coagulant in water its ionization produces sulphate anions and aluminium cations. Sulphate ions which may remain in the form of ion or combine with other cation. However, the AP cations react immediately with water to form a variety of aquometallic ions and hydrogen. These aquometallic ions become part of ionic cloud surrounding colloid, and are adsorbed on the surface of colloid where they neutralize surface charge. Once the surface charge has been neutralized, ionic cloud dissipates and electrostatic potential disappears so that contact occurs freely.

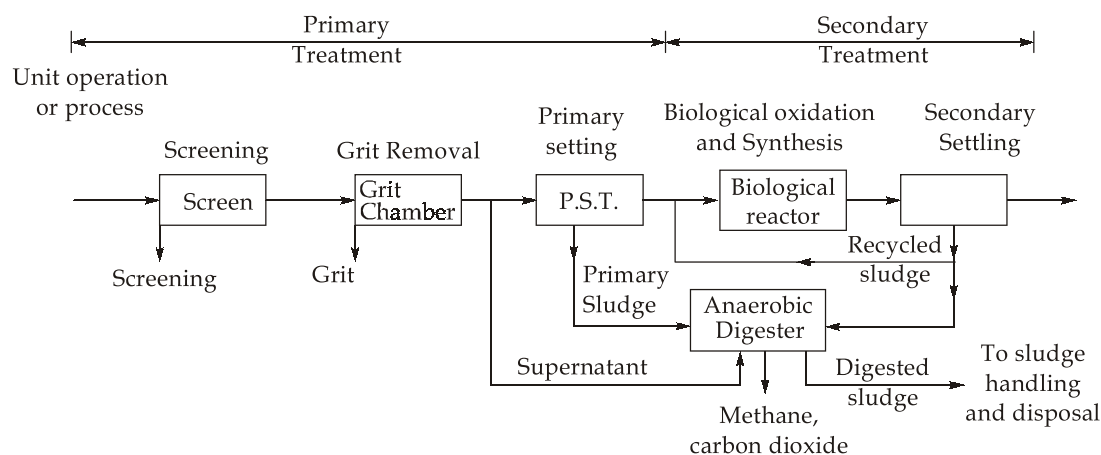
### 3. Sweep coagulation:

The last product in hydrolysis of alum when used as a coagulant is aluminium hydroxide. The  $Al(OH)_3$  forms in amorphous, gelatinous flocs that are heavier than water and settle by gravity. Colloids may become entrapped in a floc as it is formed, or they become sticky as the flocs settle. The process by which colloids are swept from suspension in this manner is known as sweep coagulation.

### 4. Inter-particle bridging:

Large molecules may be formed when aluminium or ferric salts dissociate in water. Synthetic polymers also may be used instead of or in addition to, metallic salts. These polymers may be linear or branched and are highly surface reactive. Thus, several colloids may become attached to one polymer or series of polymer, colloid. groups may become enmeshed resulting in a settle able mass. In addition to adsorption forces, charges on polymer may assist in coagulation process. Metallic polymers formed by addition of aluminium or ferric salts are positively charged, while synthetic polymers may carry positive or negative charge or may be neutral.

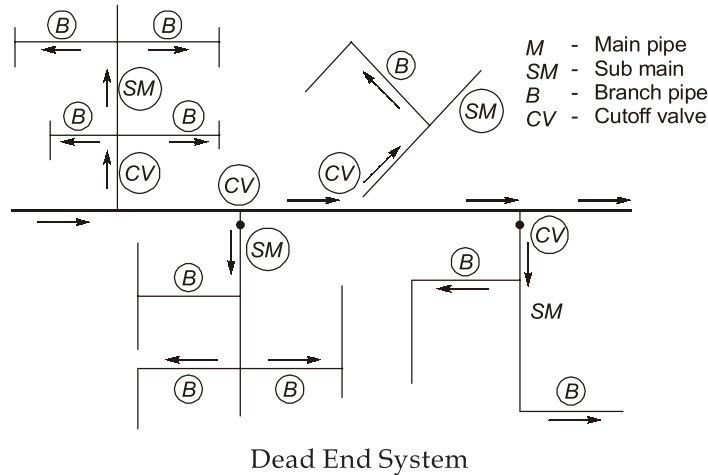
## 2. (c) (ii) Solution:



## 3. (a) (i) Solution:

In general there are four types of water distribution network which are as follows:

- (i) **Dead end system:** This network system is adopted for cities which have developed in a haphazard manner with no definite pattern of growth. Ex- old cities.

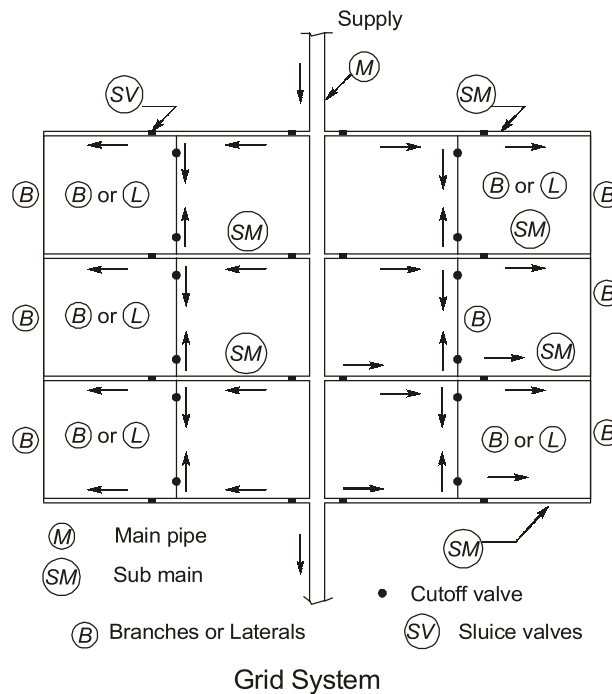
**Advantages:**

- It is relatively a cheap system.
- Determination of discharge and pressure is easy due to less number of valves being employed in the network.
- Laying of pipes is easy.
- Pipes used are of lesser diameter and hence economical.

**Disadvantages:**

- Owing to a large number of dead ends, stagnation of water occurs which cannot be utilized at locations where demand is high.
- During repairs and maintenance, entire downstream portion below the valve has to be cut-off.

- (ii) **Grid iron system:** It is suitable for cities with rectangular layout where water mains and branches are laid in a rectangular fashion.



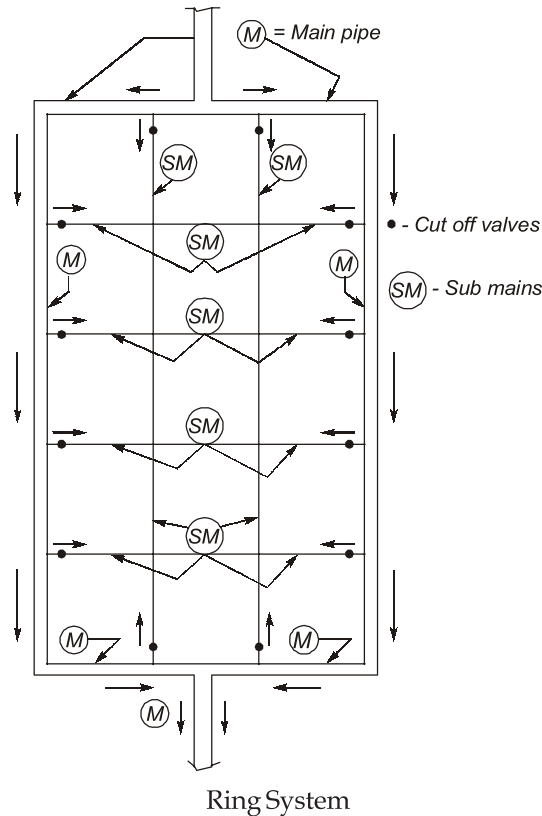
#### Advantages:

- Due to the absence of dead ends, whole water remains in circulation.
- Due to continuously circulating water, there are less chances of water getting contaminated as compared to still water.
- In case of repairs and breakdown, water is available from other locations and only a part of network needs to be isolated.

#### Disadvantages:

- Due to large number of valves required to be provided at all branches, it is difficult to compute discharge and pressure and thus exact calculation of pipe size required is difficult.

(iii) **Circular/Ring system:** In this system of water distribution, the main line is laid all along the periphery roads and sub-mains branch out from the main system of pipes.



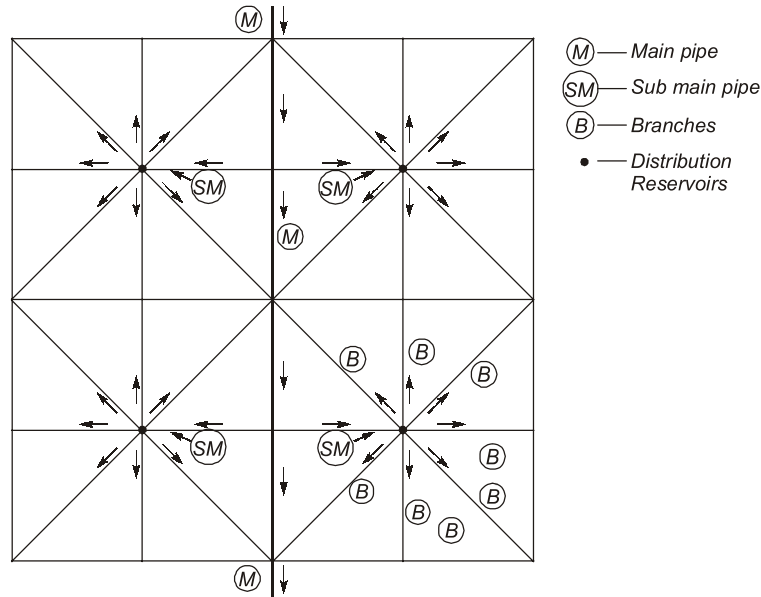
**Advantages:**

- The biggest advantage of this system is that water can be supplied to any point from at least two directions thereby ensuring availability of water for longer time per day.

**Disadvantages:**

- More pipe material is required.
- It involves high maintenance cost.

(iv) **Radial system:** Here the area is divided into different zones and water is kept in the distribution reservoir provided in the middle of each zone. Supply pipes are laid radially towards the periphery of the system.



Radial System

**Advantages:**

- Here calculation of pipe size is very easy.
- Water is made available in a short time as compared to other distribution systems.
- It is the most economical system if combined pumping and gravity flow are adopted.

**Disadvantages:**

- House at the distance end faces fluctuations.
- Design is very complex.

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

$$EW \text{ of } \text{CaCO}_3 = 50$$

total alkalinity of water sample in milli equivalents = (Volume of acid in ml)  $\times$  0.02

$\therefore$  total alkalinity in mg/l as  $\text{CaCO}_3$  = (Volume of acid in ml)  $\times$  0.02

$$\times 50 = (\text{Volume of acid in ml}) \text{ mg/l}$$

Given that, Initial pH = 9.5

So, pOH = 14 - 9.5 = 4.5

i.e.  $-\log_{10}[\text{OH}^-] = 4.5$

$$\Rightarrow [OH^-] = 10^{-4.5} \text{ moles/litre}$$

$$\text{So, Number of gram equivalents of } [OH^-] = 10^{-4.5} \times 1 = 10^{-4.5}$$

$$\text{So, Caustic alkalinity of water sample} = 10^{-4.5} \times 50 \times 1000 = 1.58 \text{ mg/l as CaCO}_3$$

$$\text{Now, acid consumed till 200 ml sample to reach pH of 8.3} = 6.8 \text{ ml}$$

$$\text{So, acid consumed till 1 l sample reaches pH of 8.3} = \frac{6.8}{200} \times 1000 = 34 \text{ ml}$$

$$\text{Hence alkalinity of 1 litre sample till pH of 8.3} = 34 \text{ mg/l as CaCO}_3$$

$$\text{Therefore, carbonate alkalinity} = 2(34 - 1.58) = 64.84 \text{ mg/l as CaCO}_3$$

$$\text{Now, total acid used during titration in 200 ml sample} = 9.6 + 6.8 = 16.4 \text{ ml}$$

$$\text{So, Total acid used in 1 litre sample} = \frac{16.4}{200} \times 1000 = 82 \text{ ml}$$

$$\text{Therefore, total alkalinity of 1 litre sample} = 82 \text{ mg/l as CaCO}_3$$

$$\text{Hence, bicarbonate alkalinity} = 82 - (64.84 + 1.58)$$

$$= 15.58 \text{ mg/l as CaCO}_3$$

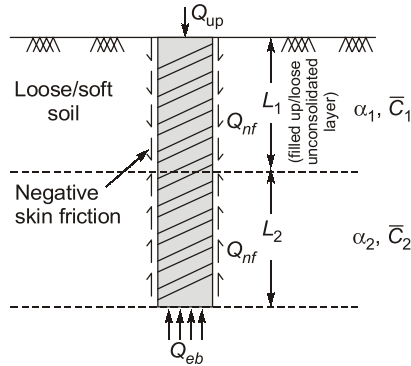
### 3. (b) (i) Solution:

Negative skin friction, also known as down-drag, is a downward force acting on a pile or pile group that occurs when the surrounding soil settles more than the pile itself. This phenomenon typically develops when piles are driven through soft, compressible soil layers (like soft clay or peat) into a firm bearing stratum, or when a recent fill has been placed over a soft soil layer, causing consolidation. Since the soil moves downward relative to the pile, the friction acts in the direction of the load, effectively increasing the total load on the pile instead of helping to support it.

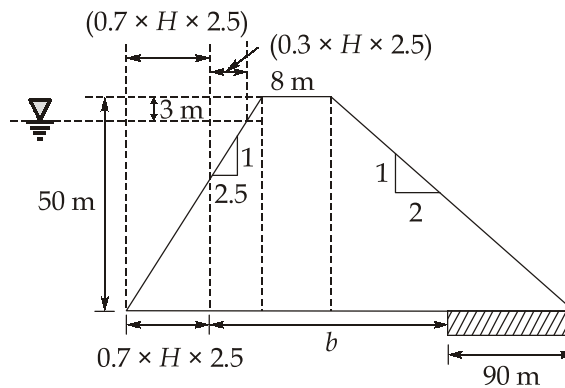
#### Key Factors Influencing Negative Skin Friction

- **Lowering of Water Table:** This increases effective stress and causes soil consolidation.
- **Surcharge Loading:** Placing a heavy fill on the ground surface around the piles.

- **Pile Groups:** In a group, the total negative skin friction is the maximum of either the sum of individual piles or the friction acting on the perimeter of the entire block of soil enclosed by the group.



3. (b) (ii) Solution:



Discharge through earthen dam,

$$q = K.S$$

where,

$$K = \sqrt{K_H \cdot K_V}$$

S = Focal length

$$S = \sqrt{b_T^2 + H^2} - b_T$$

where, H = Maximum depth of water = 50 - 3 = 47 m

From the diagram,

$$b = 8 + 2 \times [50] - 90 + 0.3 \times 47 \times 2.5 + 3 \times 2.5 = 60.75 \text{ m}$$

Now,

$$b_T = b \left( \sqrt{\frac{k_y}{k_x}} \right) = 60.75 \left( \sqrt{\frac{5 \times 10^{-7}}{7.25 \times 10^{-7}}} \right) = 50.450 \text{ m}$$

$$\therefore S = \sqrt{(50.450)^2 + (47)^2} - 50.450 = 18.501 \text{ m}$$

$$\begin{aligned} \therefore q &= \sqrt{K_H \cdot K_V} \times S = \sqrt{7.25 \times 10^{-7} \times 5 \times 10^{-7}} \times 18.501 \text{ m} \\ &= 1.114 \times 10^{-5} \text{ m}^3/\text{sec}/\text{m} \end{aligned}$$

### 3. (c) Solution:

Given Data

$$\text{Speed of overtaking vehicle (V)} = 80 \text{ kmph} = 80 \times \frac{5}{18} = 22.222 \text{ m/sec}$$

$$\text{Speed of overtaken vehicle (V}_b\text{)} = 60 \text{ kmph} = 60 \times \frac{5}{18} = 16.667 \text{ m/sec}$$

$$\text{Acceleration (a)} = 2.5 \text{ kmph/sec} = 2.5 \times \frac{5}{18} = 0.694 \text{ m/sec}^2$$

Reaction time ( $t$ ) = 2 sec

Length of vehicle ( $L$ ) = 6.5 m

#### 1. Length of Safe OSD

For a two-way traffic road:

$$OSD = d_1 + d_2 + d_3$$

**Calculation of  $d_1$  (Distance traveled during reaction time):**

$$d_1 = V_b \times t$$

$$\Rightarrow d_1 = 16.667 \times 2$$

$$\Rightarrow d_1 = 33.334 \text{ m}$$

**Calculation of  $d_2$  (Distance traveled during overtaking operation):**

First, calculate the spacing ( $s$ ):

$$s = 0.2 \times V_b + L$$

$$\Rightarrow s = 0.2 \times 60 + 6.5$$

$$\Rightarrow s = 18.5 \text{ m}$$

Next, calculate the time for the overtaking maneuver ( $T$ ):

$$T = \sqrt{\frac{4 \times s}{a}}$$

$$\Rightarrow T = \sqrt{\frac{4 \times 18.5}{0.694}}$$

$$\Rightarrow T = 10.326 \text{ sec}$$

$$\text{Now, calculate } d_2: \quad d_2 = V_b \times T + 2 \times s$$

$$\Rightarrow d_2 = 16.667 \times 10.326 + 2 \times 18.5$$

$$\Rightarrow d_2 = 209.103 \text{ m}$$

Calculation of  $d_3$  (Distance traveled by oncoming vehicle):

$$d_3 = V \times T$$

$$\Rightarrow d_3 = 22.222 \times 10.326$$

$$\Rightarrow d_3 = 229.464 \text{ m}$$

**Total Safe OSD:**

$$OSD = 33.334 + 209.103 + 229.464$$

$$\Rightarrow OSD = 471.901 \text{ m}$$

## 2. Minimum Length of Overtaking Zone

$$\text{Minimum Length} = 3 \times OSD$$

$$\Rightarrow \text{Minimum Length} = 3 \times 471.901$$

$$\Rightarrow \text{Minimum Length} = 1415.703 \text{ m}$$

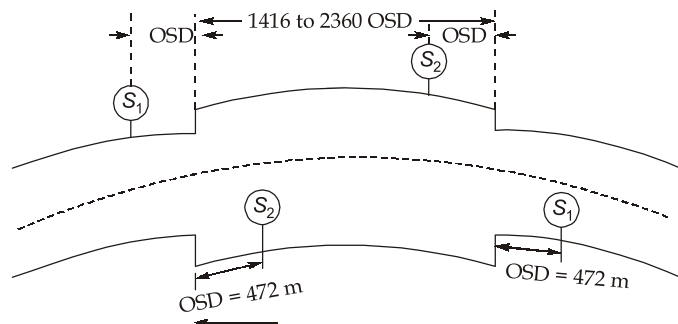
## 3. Desirable Length of Overtaking Zone

$$\text{Desirable Length} = 5 \times OSD$$

$$\Rightarrow \text{Desirable Length} = 5 \times 471.901$$

$$\Rightarrow \text{Desirable Length} = 2359.505 \text{ m}$$

Sketch of overtaking zone with the positions of the sign posts:



$S_1$  = Sign post "Overtaking zone Ahead"

$S_2$  = Sign post "End of Overtaking zone"

## 4. (a) Solution:

Given data

Rising gradient,  $n_1 = +\frac{1}{40} = +0.025$

Falling gradient,  $n_2 = -\frac{1}{200} = -0.005$

Deviation angle,  $N = n_1 - n_2 = 0.025 - (-0.005) = 0.03$

Sight distance,  $S = 250$  m

Height of driver's eye,  $h_1 = 1.2$  m

Height of obstacle,  $h_2 = 0.15$  m

Reduced level of PVI = 220 m

Length of summit curve is calculated assuming  $L > S$ .

$$L = \frac{N \times S^2}{(\sqrt{2h_1} + \sqrt{2h_2})^2}$$

$$\Rightarrow L = \frac{0.03 \times 250 \times 250}{(\sqrt{2 \times 1.2} + \sqrt{2 \times 0.15})^2}$$

$$\Rightarrow L = 426.421 \text{ m}$$

Since  $426.421 > 250$ , the assumption  $L > S$  is correct.

1. Equation of summit curve with origin at VPC is given by

$$y = ax^2 + n_1x$$

where  $a = -\frac{N}{2L}$

$$\Rightarrow a = -\frac{0.03}{2 \times 426.421} = -0.000035177$$

$$\therefore y = -0.000035177 x^2 + 0.025x$$

2. Position of summit point from VPC is obtained using

$$x = \frac{n_1 L}{N}$$

$$\Rightarrow x = \frac{0.025 \times 426.421}{0.03}$$

$$\Rightarrow x = 355.351 \text{ m}$$

3. Reduced level of vertical point of curve (*VPC*) is determined as follows.  
The *VPC* is located at a horizontal distance of  $L/2$  before the *PVI*.

$$\frac{L}{2} = \frac{426.421}{2} = 213.211 \text{ m}$$

$$\Rightarrow \text{R.L. of } VPC = \text{R.L. of } PVI - (n_1 \times 213.211)$$

$$\Rightarrow \text{R.L. of } VPC = 220 - (0.025 \times 213.211)$$

$$\Rightarrow \text{R.L. of } VPC = 214.67 \text{ m}$$

4. Reduced level of vertical point of tangency (*VPT*) is determined as

$$\text{R.L. of } VPT = \text{R.L. of } PVI + (n_2 \times 213.211)$$

$$\Rightarrow \text{R.L. of } VPT = 220 + (-0.005 \times 213.211)$$

$$\Rightarrow \text{R.L. of } VPT = 218.934 \text{ m}$$

5. Reduced level of point on curve directly below the *PVI* occurs at

$$x = \frac{L}{2} = 213.211 \text{ m}$$

Substituting in the curve equation

$$y = -0.000035177(213.211)^2 + 0.025(213.211)$$

$$y = 3.731 \text{ m}$$

$\therefore$  R.L. of the point on the curve below *VPI*.

$$\text{R.L. of point} = \text{R.L. of } VPC + y$$

$$\Rightarrow \text{R.L. of point} = 214.67 + 3.731$$

$$\Rightarrow \text{R.L. of point} = 218.401 \text{ m}$$

#### 4. (b) (i) Solution:

##### Hardy Cross Theorem

The Hardy Cross method is an iterative technique used to determine the flow distribution in a complex network of pipes where the inlets and outlets are known, but the internal flows are unknown. It is based on two fundamental physical principles.

##### 1. Law of Continuity (Node Law)

The algebraic sum of flows at any junction (node) must be zero.

$$\Sigma Q = 0$$

##### 2. Law of Conservation of Energy (Loop Law)

The algebraic sum of head losses around any closed loop must be zero.

$$\Sigma h_f = 0$$

### Derivation of the Correction Expression

The head loss in a pipe is generally expressed by the relation

$$h_f = r \times Q^n$$

where  $r$  is the pipe resistance and  $n$  is a constant (usually 2 for the Darcy-Weisbach or Manning equation).

Let  $Q_a$  be the assumed flow in a pipe and  $\Delta Q$  be the required correction to obtain the actual flow  $Q$ .

$$Q = Q_a + \Delta Q$$

For any closed loop in the network, the sum of head losses must satisfy

$$\Sigma r \times Q^n = 0$$

Substituting  $Q = Q_a + \Delta Q$

$$\Sigma r \times (Q_a + \Delta Q)^n = 0$$

Expanding the expression using the Binomial Theorem and neglecting higher order terms (since  $\Delta Q$  is small compared to  $Q_a$ )

$$\Sigma r \times (Q_a^n + n \times Q_a^{n-1} \times \Delta Q) = 0$$

Distributing the summation

$$\Sigma r \times Q_a^n + \Sigma n \times r \times Q_a^{n-1} \times \Delta Q = 0$$

Since  $\Delta Q$  is constant for all pipes in a loop during one iteration, it can be taken outside the summation

$$\Sigma r \times Q_a^n + \Delta Q \Sigma n \times r \times Q_a^{n-1} = 0$$

Rearranging to obtain the correction term

$$\Delta Q = \frac{-\Sigma r \times Q_a^n}{\Sigma n \times r \times Q_a^{n-1}}$$

For the common case where  $n = 2$

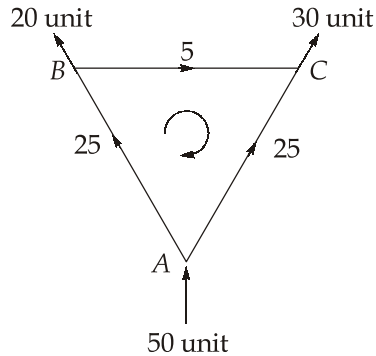
$$\Delta Q = \frac{-\Sigma r \times Q_a |Q_a|}{\Sigma 2 \times r \times |Q_a|}$$

This expression is used to compute the correction in flow during each iteration of the Hardy Cross method until the loop head loss becomes approximately zero.

## 4. (b) (ii) Solution:

Initial Flow Distribution (Trial 0)

Based on continuity at each node:



Clockwise flows are taken as positive and counter-clockwise flows as negative.

**Iteration 1**

Pipe	$h = r \times Q^2$	$ 2 \times r \times Q $
AB	$12 \times 25^2$	$2 \times 12 \times 25$
BC	$15 \times 5^2$	$2 \times 15 \times 5$
CA	$-10 \times (25)^2$	$2 \times 10 \times (25)$
<b>Sum</b>	<b>1625</b>	<b>1250</b>

$$\Delta Q_1 = -\frac{\Sigma r \times Q^2}{\Sigma |2 \times r \times Q|} = -\frac{1625}{1250}$$

$$\Delta Q_1 = -1.3$$

**Iteration 2**

Updated flows

$$Q_{AB} = 25 + (-1.3) = 23.7 \text{ unit}$$

$$Q_{BC} = 5 + (-1.3) = 3.7 \text{ unit}$$

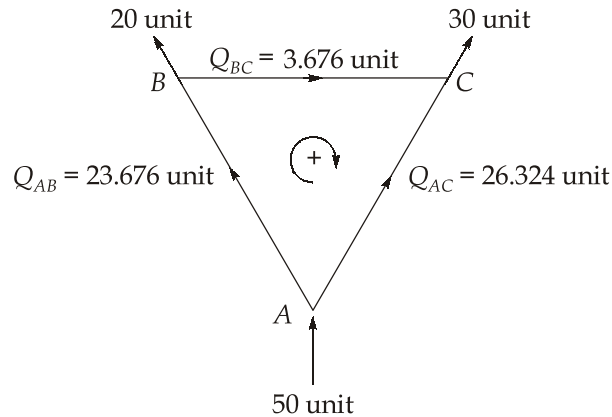
$$Q_{CA} = -25 + (-1.3) = -26.3 \text{ unit}$$

Pipe	$h = r \times Q^2$	$ 2 \times r \times Q $
AB	$12 \times 23.7^2$	$2 \times 12 \times 23.7$
BC	$15 \times 3.7^2$	$2 \times 15 \times 3.7$
CA	$-10 \times (26.3)^2$	$2 \times 10 \times (26.3)$
<b>Sum</b>	<b>28.73</b>	<b>1205.8</b>

$$\Delta Q_2 = -\frac{28.73}{1205.8}$$

$$\Delta Q_2 = -0.024$$

### Final Flow Distribution



#### 4. (c) (i) Solution:

##### Drift

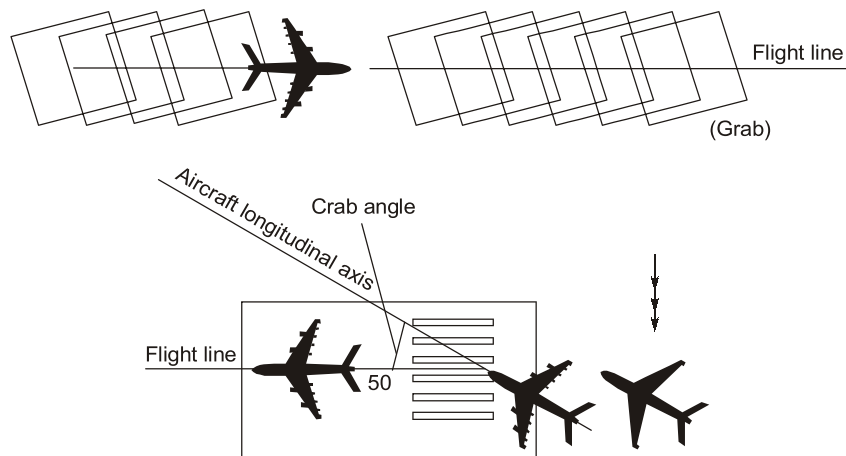
Drift is the lateral displacement of an aircraft from its intended course caused by the action of the wind. When an aircraft flies through a moving air mass, it is carried along with that air. If the wind is blowing from the side (crosswind), the aircraft will be pushed away from its desired track over the ground.

- **Drift Angle:** This is the angular difference between the aircraft's heading (the direction the nose is pointing) and its track (the actual path followed over the ground).
- **Correction:** Pilots must calculate the drift angle to determine how much they need to compensate to reach their destination.

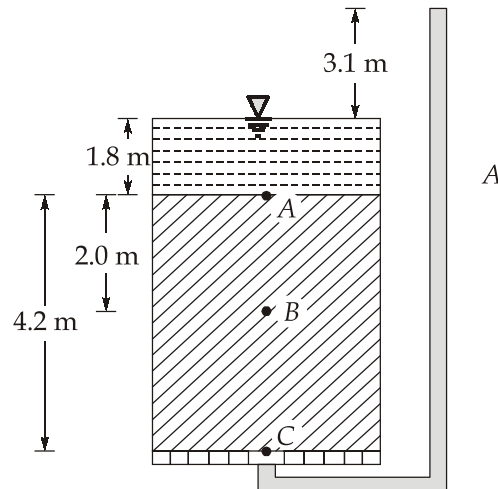
##### Crab

Crab (or Crabbing) is the maneuver used to compensate for drift. To maintain a desired track in a crosswind, the pilot turns the aircraft's nose into the wind. This creates an angle between the aircraft's longitudinal axis and the track.

- **Crab Angle:** The specific angle at which the aircraft is turned into the wind so that the resulting "diagonal" movement aligns perfectly with the intended ground track.
- **The Result:** Even though the nose is pointed slightly to the left or right of the destination, the aircraft moves "sideways" along the correct path, much like a crab walking on a beach.



## 4. (c) (ii) Solution:



Given data

Specific gravity,  $G_s = 2.68$

Void ratio  $e = 0.74$

Head causing upward flow  $h = 3.1 \text{ m}$

Saturated unit weight of sand

$$\gamma_{\text{sat}} = \frac{(G_s + e)\gamma_w}{1 + e}$$

$$\gamma_{\text{sat}} = \frac{(2.68 + 0.74) \times 9.81}{1 + 0.74}$$

$$\gamma_{\text{sat}} = 19.282 \text{ kN/m}^3$$

Hydraulic gradient

$$i = \frac{h}{L} = \frac{3.1}{4.2}$$

$$i = 0.738$$

(i) Stresses at points A, B and C

At point A (top of sand)

$$\begin{aligned} \text{Total vertical stress} & \quad \sigma_A = H_w \times \gamma_w \\ \Rightarrow & \quad \sigma_A = 1.8 \times 9.81 \\ \Rightarrow & \quad \sigma_A = 17.658 \text{ kN/m}^2 \\ \text{Pore water pressure} & \quad u_A = 1.8 \times 9.81 \\ \Rightarrow & \quad u_A = 17.658 \text{ kN/m}^2 \\ \text{Effective stress} & \quad \sigma'_A = \sigma_A - u_A \\ \Rightarrow & \quad \sigma'_A = 0 \text{ kN/m}^2 \end{aligned}$$

At point B (2.0 m below A)

$$\begin{aligned} \text{Total vertical stress} & \quad \sigma_B = 2 \times 19.282 + 1.8 \times 9.81 \\ \Rightarrow & \quad \sigma_B = 56.222 \text{ kN/m}^2 \\ \text{Pore water pressure} & \quad u_B = (1.8 + 2.0) \times 9.81 + (0.738 \times 2.0 \times 9.81) \\ \Rightarrow & \quad u_B = 51.758 \text{ kN/m}^2 \\ \text{Effective stress} & \quad \sigma'_B = \sigma_B - u_B \\ \Rightarrow & \quad \sigma'_B = 4.464 \text{ kN/m}^2 \end{aligned}$$

At point C (bottom of sand)

$$\begin{aligned} \text{Total vertical stress} & \quad \sigma_C = (1.8 \times 9.81) + (4.2 \times 19.282) \\ \Rightarrow & \quad \sigma_C = 98.642 \text{ kN/m}^2 \\ \text{Pore water pressure} & \quad u_C = (1.8 + 4.2 + 3.1) \times 9.81 \\ \Rightarrow & \quad u_C = 89.271 \text{ kN/m}^2 \\ \text{Effective stress} & \quad \sigma'_C = \sigma_C - u_C \\ \Rightarrow & \quad \sigma'_C = 9.371 \text{ kN/m}^2 \end{aligned}$$

Stress summary table

Point	Total stress $\sigma$ (kN/m <sup>2</sup> )	Pore water pressure $u$ (kN/m <sup>2</sup> )	Effective stress $\sigma'$ (kN/m <sup>2</sup> )
A	17.658	17.658	0
B	56.222	51.758	4.464
C	98.642	89.271	9.371

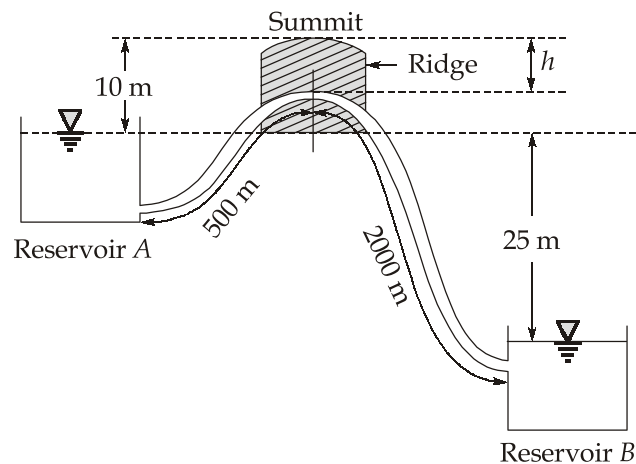
(ii) Upward seepage force per unit volume

Seepage force per unit volume

$$\begin{aligned}
 &= i \times \gamma_w \\
 &= 0.738 \times 9.81 \\
 &= 7.240 \text{ kN/m}^3
 \end{aligned}$$

### Section B

5. (a) Solution:



Velocity of flow in pipeline

Applying Bernoulli's equation between the surfaces of the two reservoirs. Since entry and exit losses are neglected, the entire head loss is due to friction.

$$H = \frac{4fLV^2}{2gD}$$

$$\Rightarrow 25 = \frac{4 \times 0.007 \times 2500 \times V^2}{2 \times 9.81 \times 0.25}$$

$$\Rightarrow V = 1.324 \text{ m/s}$$

Discharge through the pipeline

$$Q = \frac{\pi}{4} D^2 V$$

$$\Rightarrow Q = \frac{\pi}{4} (0.25)^2 \times 1.324$$

$$\Rightarrow Q = 0.065 \text{ m}^3/\text{s}$$

Minimum depth of pipe below the summit of ridge

Applying Bernoulli's equation between the surface of higher reservoir (A) and the apex of the pipeline (S). Taking the water surface of reservoir A as datum.

Pressure head at apex  $\frac{P_S}{\gamma} = H_{atm} - H_{vac}$

Elevation of pipe at apex  $Z_S = Z_{ridge} - h$

Bernoulli equation

$$\frac{P_A}{\gamma} + Z_A + \frac{V_A^2}{2g} = \frac{P_S}{\gamma} + Z_S + \frac{V_S^2}{2g} + h_{f(A-S)}$$

$$\Rightarrow 10.3 + 0 + 0 = (10.3 - 7.5) + (10 - h) + \frac{V^2}{2g} + \frac{4fL_s V^2}{2gD}$$

$$\Rightarrow 10.3 = 12.8 - h + \frac{1.324^2}{2 \times 9.81} + \frac{4 \times 0.007 \times 500 \times 1.324^2}{2 \times 9.81 \times 0.25}$$

$$\Rightarrow h = 7.593 \text{ m}$$

Minimum depth of pipe below ridge summit

$$h = 7.593 \text{ m}$$

Discharge through pipeline

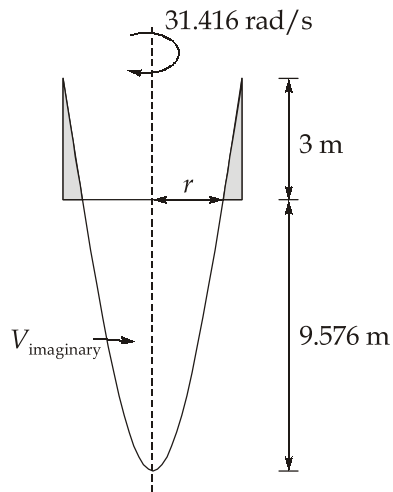
$$Q = 0.065 \text{ m}^3/\text{s} = 65 \text{ l/sec}$$

### 5. (b) (i) Solution:

Angular rotation,  $\omega = \frac{2\pi N}{60} = \frac{2\pi \times 300}{60} = 31.416 \text{ rad/s}$

Depth of cavity formed when cylinder is rotated about its axis

$$Z = \frac{R^2 \omega^2}{2g} = \frac{(0.5)^2 \times (31.416)^2}{2 \times 9.81} = 12.576 \text{ m}$$



Let  $r$  be the radius upto which bottom will be exposed

$$Z = \frac{\omega^2 r^2}{2g}$$

$$\Rightarrow 9.576 = \frac{(31.416)^2 \times r^2}{2g}$$

$$\Rightarrow r = 0.436 \text{ m}$$

Initial volume,  $V_i = \pi \times 0.5^2 \times 3 = 2.356 \text{ m}^3$

Final volume,  $V_f = V_i - [V_{\text{cavity}} - V_{\text{imaginary}}]$

$$V_{\text{cavity}} = \frac{1}{2} \times \pi \times 0.5^2 \times 12.576 = 4.939 \text{ m}^3$$

$$V_{\text{imaginary}} = \frac{1}{2} \times \pi \times 0.436^2 \times 9.576 = 2.86 \text{ m}^3$$

$$\therefore V_f = 2.356 - [4.939 - 2.86] = 0.277 \text{ m}^3$$

Spilled volume,  $V_{\text{spilled}} = V_i - V_f = 2.356 - 0.277 = 2.079 \text{ m}^3$

**Q.5 (b) (ii) Solution:**

For pure water and clean glass, angle of contact:

$$\theta = 0^\circ$$

$$\therefore \text{Capillary rise, } h = \frac{4\sigma \cos \theta}{\gamma d}$$

$$\Rightarrow h = \frac{4\sigma}{\gamma d} = \frac{4 \times 0.075}{9810 \times 2.5 \times 10^{-3}}$$

$$= 0.01223 \text{ m} = 12.23 \text{ mm}$$

So, the correction of 12.23 mm should be applied to the manometric reading of tube.

**5. (c) Solution:**

Given data

Porosity,  $n = 0.3$

Degree of saturation,  $S = 0.9$

Initial wetted front distance,  $x_0 = 3 \text{ cm}$

Applied head in stage 1,  $h_1 = 100 \text{ cm}$

Time for stage 1,  $t_1 = 10 \text{ min}$

Final wetted front distance in stage 1,  $x_1 = 15 \text{ cm}$

Applied head in stage 2,  $h_2 = 250 \text{ cm}$

Time for stage 2,  $t_2 = 30 \text{ min}$

Final wetted front distance in stage 2,  $x_2 = 40 \text{ cm}$

The governing equation for horizontal capillary permeability test is,

$$x_f^2 - x_i^2 = \frac{2k(h + h_c)}{nS} t$$

Rearranged as,  $h + h_c = \frac{nS}{2kt} (x_f^2 - x_i^2)$

For stage 1,  $100 + h_c = \frac{0.3 \times 0.9}{2 \times k \times 10} (15^2 - 3^2)$

$$\Rightarrow 100 + h_c = \frac{2.916}{k} \quad \dots(i)$$

For stage 2,  $250 + h_c = \frac{0.3 \times 0.9}{2 \times k \times 30} (40^2 - 15^2)$

$$\Rightarrow 250 + h_c = \frac{6.1875}{k} \quad \dots(ii)$$

Subtract Equation 1 from Equation 2,

$$(250 + h_c) - (100 + h_c) = \frac{6.1875}{k} - \frac{2.916}{k}$$

$$\Rightarrow 150 = \frac{3.2715}{k}$$

$$\Rightarrow k = 0.02181 \text{ cm/min}$$

Convert to cm/sec,

$$k = \frac{0.02181}{60} = 0.000364 \text{ cm/sec}$$

Find the capillary suction head  $h_c$  using Equation 1,

$$100 + h_c = \frac{2.916}{0.02181}$$

$$\Rightarrow h_c = 33.7 \text{ cm}$$

## 5. (d) (i) Solution:

### 1. Sewage:

Sewage refers to the actual liquid waste or wastewater produced by a community. It includes discharge from residential, commercial, and industrial buildings, as well as surface runoff and groundwater that enters the pipes. It is roughly 99.9% water and 0.1% solid matter (organic and inorganic).

### 2. Sewer:

A sewer is the physical pipe or underground conduit used for carrying the sewage from the point of origin to the point of disposal or treatment. These pipes are typically designed to flow under gravity and are made of materials like concrete, PVC, or cast iron.

### 3. Sewerage System:

The sewerage system is the complete infrastructure required to collect, transport, treat, and dispose of sewage. This includes:

- The entire network of sewers (lateral, branch, and main sewers).
- Appurtenances such as manholes, catch basins, and pumping stations.
- The wastewater treatment plant (WWTP) where the sewage is processed before being discharged into a natural water body.

## 5. (d) (ii) Solution:

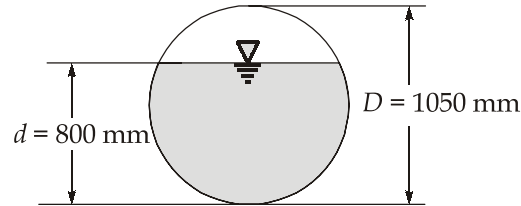
Given data

Diameter,  $D = 1050 \text{ mm} = 1.05 \text{ m}$

Flow depth,  $d = 800 \text{ mm} = 0.8 \text{ m}$

Invert slope,  $S = 0.0002$

Manning's coefficient,  $n = 0.013$



Since,

$$\frac{d}{D} = \frac{1 - \cos \frac{\alpha}{2}}{2}$$

$$\Rightarrow \frac{800}{1050} = \frac{1 - \cos \frac{\alpha}{2}}{2}$$

$$\Rightarrow \alpha = 243.176^\circ$$

Since

$$\frac{a}{A} = \frac{\alpha}{360} - \frac{\sin \alpha}{2\pi}$$

$$\Rightarrow a = \frac{\pi}{4} (1.05)^2 \left[ \frac{243.176^\circ}{360^\circ} - \frac{\sin(243.176^\circ)}{2\pi} \right]$$

$$\Rightarrow a = 0.7079 \text{ m}^2$$

Since

$$\frac{p}{P} = \frac{\alpha}{360^\circ}$$

$$\Rightarrow p = \pi \times 1.050 \times \frac{243.176^\circ}{360^\circ} = 2.228$$

Hydraulic radius,

$$r = \frac{0.7079}{2.228} = 0.3177 \text{ m}$$

Calculate the discharge  $q$  using Manning's equation:

$$q = \frac{1}{n} ar^{2/3} S^{1/2}$$

Substitute values,  $q = \frac{1}{0.013} \times 0.7079 \times (0.3177)^{2/3} \times (0.0002)^{1/2} = 0.358 \text{ m}^3/\text{s}$

So, The discharge through the sewer pipe is approximately

$$Q = 0.358 \text{ m}^3/\text{s}$$

**5. (e) Solution:**

Given data

Population,  $P = 4,50,000$

Annual Average Daily water demand,

$$ADD = 320 \text{ liters/head/day}$$

Kuichling's formula for fire demand,

$$Q = 3182\sqrt{P_{\text{thousands}}} \text{ liters/min}$$

Multiplying factors for peak demands:

$$MDD = 1.8 \times ADD, MHD = 1.5 \times MDD$$

Calculate the Average Daily Draft (ADD):

$$ADD = P \times \text{average demand} = 4,50,000 \times 320 = 144,000,000 \text{ liters/day} = 144 \text{ MLD}$$

Maximum Daily Draft (MDD):

$$MDD = 1.8 \times ADD = 1.8 \times 144 = 259.2 \text{ MLD}$$

Maximum Hourly Draft (MHD):

$$MHD = 1.5 \times MDD = 1.5 \times 259.2 = 388.8 \text{ MLD}$$

Fire Demand Calculation using Kuichling's formula:

$$P_{\text{thousands}} = \frac{450,000}{1,000} = 450$$

$$Q = 3182 \times \sqrt{450} = 67,500.41 \text{ liters/min}$$

Convert to MLD:

$$Q_{\text{fire}} = \frac{67500.41 \times 60 \times 24}{1,000,000} = 97.2 \text{ MLD}$$

Coincident Draft:

$$\text{Coincident Draft} = MDD + Q_{\text{fire}} = 259.2 + 97.2 = 356.4 \text{ MLD}$$

**(i) Design capacity of Water Distribution System:**

The system is designed for the higher of Maximum Hourly Draft or Coincident Draft,

$$MHD = 388.8 \text{ MLD}, \text{ Coincident Draft} = 356.4 \text{ MLD}$$

$$\text{Design Capacity} = 388.8 \text{ MLD}$$

**(ii) Design capacity of the pipe from Treatment Unit to the storage tank:**

The main supply conduit is designed for Maximum Daily Draft,

$$\text{Design Capacity} = \text{MDD} = 259.2 \text{ MLD}$$

**(iii) Design capacity of Filter Unit:**

Filter units and treatment plant components are also designed for Maximum Daily Draft,

$$\text{Design Capacity} = \text{MDD} = 259.2 \text{ MLD}$$

**6. (a) Solution:**

Given data

Area of survey region, 200 km × 120 km

Photograph size,  $l = w = 23 \text{ cm} = 0.23 \text{ m}$

Focal length,  $f = 150 \text{ mm} = 0.15 \text{ m}$

Photo scale,  $S = 1: 20,000$

Average terrain elevation,  $h = 450 \text{ m}$

Longitudinal overlap,  $p_l = 0.65$

Side overlap,  $p_s = 0.35$

Aircraft ground speed,  $V = 300 \text{ km/hr} = 83.333 \text{ m/s}$

Intervalometer least count = 1 s

1. Flying height above mean sea level is obtained using the scale relation

$$S = \frac{f}{H - h}$$

$$\Rightarrow \frac{1}{20,000} = \frac{0.15}{H - 450}$$

$$\Rightarrow H = 3,450 \text{ m}$$

2. Ground dimension of one photograph

$$L = W = 0.23 \times 20,000 = 4,600 \text{ m} = 4.6 \text{ km}$$

Spacing between flight lines

$$W_1 = W(1 - p_s)$$

$$\Rightarrow W_1 = 4.6(1 - 0.35)$$

$$\Rightarrow W_1 = 2.99 \text{ km}$$

3. Theoretical ground exposure distance

$$L_1 = L(1 - p_l)$$

$$\Rightarrow L_1 = 4.6(1 - 0.65)$$

$$\Rightarrow L_1 = 1.61 \text{ km} = 1,610 \text{ m}$$

Theoretical exposure interval

$$t = \frac{L_1}{V} = \frac{1610}{83.333}$$

$$\Rightarrow t = 19.32 \text{ s}$$

Adjusted exposure interval considering intervalometer least count

$$t_{adj} = 19 \text{ s}$$

Actual ground exposure distance

$$L_{1 \text{ revised}} = V \times t_{adj}$$

$$\Rightarrow L_{1 \text{ revised}} = 83.333 \times 19$$

$$\Rightarrow L_{1 \text{ revised}} = 1,583.327 \text{ m}$$

4. Number of flight lines

$$N_1 = \frac{120}{2.99} + 1$$

$$\Rightarrow N_1 = 41.133 \approx 42 \text{ lines}$$

Number of photographs per flight line

$$N_2 = \frac{200000}{1583.327} + 1$$

$$\Rightarrow N_2 = 127.317 \approx 128 \text{ photographs}$$

Total number of photographs

$$\text{Total} = N_1 \times N_2$$

$$\Rightarrow \text{Total} = 42 \times 128 = 5,376$$

6. (b) (i) Solution:

Given data

Thickness of clay layer,  $H = 6 \text{ m}$

Drainage condition = Double drainage

Drainage path,  $d = \frac{H}{2} = \frac{6}{2} = 3 \text{ m}$

Initial load,  $\sigma'_0 = 140 \text{ kN/m}^2$

Final load,  $\sigma'_f = 210 \text{ kN/m}^2$

Increase in load,  $\Delta\sigma' = 210 - 140 = 70 \text{ kN/m}^2$

Coefficient of volume compressibility,

$$m_v = 6.25 \times 10^{-4} \text{ m}^2/\text{kN}$$

Coefficient of permeability,  $k = 1.8 \times 10^{-8} \text{ m/min}$

Unit weight of water,  $\gamma_w = 9.81 \text{ kN/m}^3$

Ultimate settlement is calculated using

$$S_f = m_v \times \Delta\sigma' \times H$$

$$\Rightarrow S_f = (6.25 \times 10^{-4}) \times 70 \times 6$$

$$\Rightarrow S_f = 0.2625 \text{ m}$$

Settlement at 50% consolidation is

$$S_{50} = U \times S_f$$

$$\Rightarrow S_{50} = 0.5 \times 0.2625$$

$$\Rightarrow S_{50} = 0.1313 \text{ m}$$

Coefficient of consolidation is calculated as

$$C_v = \frac{k}{m_v \times \gamma_w}$$

$$\Rightarrow C_v = \frac{1.8 \times 10^{-8}}{(6.25 \times 10^{-4}) \times 9.81}$$

$$\Rightarrow C_v = 2.936 \times 10^{-6} \text{ m}^2/\text{min}$$

For 90% consolidation,  $T_{90} = 1.781 - 0.933 \log_{10} (100 - 90)$

Time factor  $T_{90} = 0.848$

Time required for 90% consolidation is

$$t_{90} = \frac{T_{90} \times d^2}{C_v} = \frac{0.848 \times 3^2}{2.936 \times 10^{-6}}$$

$$\Rightarrow t_{90} = 2599455.041 \text{ min}$$

$$\text{Convert to days, } t_{90} = \frac{2599455.041}{60 \times 24 \times 365}$$

$$\Rightarrow t_{90} = 4.946 \text{ years}$$

## 6. (b) (ii) Solution:

Given data:

$$\text{Slab thickness } (h) = 25 \text{ cm}$$

$$\text{Joint spacing in } x\text{-direction } (L_x) = 4.8 \text{ m} = 480 \text{ cm}$$

$$\text{Joint spacing in } y\text{-direction } (L_y) = 3.6 \text{ m} = 360 \text{ cm}$$

$$\text{Modulus of subgrade reaction } (k) = 10 \text{ kg/cm}^3$$

$$\text{Radius of loaded area } (a) = 15 \text{ cm}$$

$$\text{Thermal expansion coefficient } (\alpha) = 10 \times 10^{-6}/^\circ\text{C}$$

$$\text{Modulus of elasticity } (E) = 3 \times 10^5 \text{ kg/cm}^2$$

$$\text{Poisson's ratio } (\mu) = 0.15$$

$$\text{Max temperature differential during day } (T) = 0.62 \times 25 = 15.5 \text{ }^\circ\text{C}$$

$$\text{Max temperature differential during night } (T') = 0.42 \times 25 = 10.5 \text{ }^\circ\text{C}$$

The radius of relative stiffness is calculated using the relation

$$l = \left[ \frac{Eh^3}{12k(1-\mu^2)} \right]^{1/4}$$

$$\Rightarrow l = \left[ \frac{3 \times 10^5 \times 25^3}{12 \times 10 \times (1-0.15^2)} \right]^{1/4}$$

$$\Rightarrow l = 79.508 \text{ cm}$$

The ratios required for Bradbury's coefficients are calculated as

$$\frac{L_x}{l} = \frac{480}{79.508} = 6.037$$

and 
$$\frac{L_y}{l} = \frac{360}{79.508} = 4.528$$

From Bradbury's coefficient table and using linear interpolation

$$C_x = 0.920 + \frac{0.98 - 0.920}{7 - 6} (6.037 - 6)$$

$$C_x = 0.922$$

and 
$$C_y = 0.52 + \left( \frac{0.78 - 0.52}{5 - 4} \right) \times (4.528 - 4)$$

$$C_y = 0.657$$

The interior warping stress is calculated using

$$\sigma_{wi} = \frac{E\alpha T}{2} \left[ \frac{C_x + \mu C_y}{1 - \mu^2} \right]$$

$$\Rightarrow \sigma_{wi} = \frac{3 \times 10^5 \times 10 \times 10^{-6} \times 15.5}{2} \left[ \frac{0.922 + 0.15 \times 0.657}{1 - 0.15^2} \right]$$

$$\Rightarrow \sigma_{wi} = 24.274 \text{ kg/cm}^2$$

The edge warping stress is calculated using

$$\sigma_{we} = \frac{C_x E \alpha T}{2}$$

$$\Rightarrow \sigma_{we} = \frac{0.922 \times 3 \times 10^5 \times 10 \times 10^{-6} \times 15.5}{2}$$

$$\Rightarrow \sigma_{we} = 21.436 \text{ kg/cm}^2$$

The corner warping stress is calculated using

$$\sigma_{wc} = \frac{E\alpha T'}{3(1-\mu)} \sqrt{\frac{a}{l}}$$

$$\Rightarrow \sigma_{wc} = \frac{3 \times 10^5 \times 10 \times 10^{-6} \times 10.5}{3(1-0.15)} \sqrt{\frac{15}{79.508}}$$

$$\Rightarrow \sigma_{wc} = 5.366 \text{ kg/cm}^2$$

#### 6. (c) (i) Solution:

##### Primary and Secondary Air Pollutants

Air pollutants are categorized based on how they are introduced into the atmosphere.

##### Primary Air Pollutants

These are pollutants emitted directly into the atmosphere from a source (like a tail pipe or a smokestack) in the same form as they were produced.

##### Examples:

1. Carbon Monoxide (CO): From incomplete combustion of fuels.
2. Sulfur Dioxide (SO<sub>2</sub>): From burning coal and oil.
3. Nitric Oxide (NO): From high-temperature combustion in engines.
4. Particulate Matter (PM): Such as dust, ash, or soot.

##### Secondary Air Pollutants

These are not emitted directly. Instead, they form in the air when primary pollutants react with each other or with basic atmospheric components like water vapor or sunlight).

**Examples:**

1. Ozone ( $O_3$ ): Formed by reactions between  $NO_x$  and VOCs in sunlight.
2. Sulfuric Acid ( $H_2SO_2$ ): Formed when  $SO_2$  reacts with water vapor (a component of acid rain)
3. Nitric Acid ( $HNO_3$ ): Formed from nitrogen oxides reacting with moisture
4. Peroxyacetyl Nitrate (PAN): A product of reactions in photochemical smog.

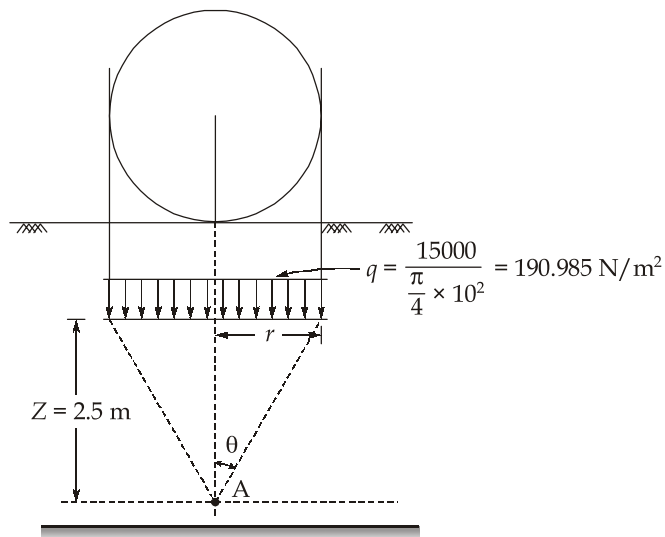
**6. (c) (ii) Solution:****Smog and Photochemical Smog****Definition of Smog**

The term Smog is a combination of the words “smoke” and “fog.” it refers to a type of intense air pollution that reduces visibility and poses health risks.

**Formation of Photochemical Smog**

Photochemical smog (also known as “Los Angeles-type smog”) occurs in warm, sunny climates. Its formation follows a specific process:

1. Primary Emission: Vehicles and industries release Nitrogen Oxides ( $NO_x$ ) and Volatile Organic Compounds (VOCs).
2. Photolytic Reaction: Sunlight breaks down Nitrogen Dioxide ( $NO_2$ ) into Nitric Oxide (NO) and free Oxygen atoms (O).
3. Ozone Creation: The free Oxygen atom reacts with atmospheric  $O_2$  to form Ground-level Ozone ( $O_3$ ).
4. Complex Mixture: This Ozone then reacts with VOCs to create a brownish haze containing secondary pollutants like PAN, aldehydes, and ketones.

**6. (c) (iii) Solution:**

$$\Delta\sigma_z = q(1 - \cos^3\theta)$$

$$\cos \theta = \frac{z}{\sqrt{r^2 + z^2}} = \frac{2.5}{\sqrt{2.5^2 + 5^2}} = 0.447$$

$$\begin{aligned} \therefore \Delta\sigma_z &= 190.985 \times (1 - 0.447^3) \\ &= 173.903 \text{ kPa} \end{aligned}$$

## 7. (a) (i) Solution:

1. **Full Face Method:** In this method, the entire cross-section of the tunnel is excavated in a single operation. It is most suitable for tunnels with a relatively small cross-sectional area (usually up to 3 meters in diameter) and where the rock is stable enough to support itself without immediate lining.

- **Process:** A series of holes are drilled across the entire face, loaded with explosives, and blasted. The resulting "muck" (loose rock) is then cleared before the next round of drilling begins
- **Advantages:** Simple and straightforward sequence of operations.
  - Allows for the use of heavy machinery like jumbo drills.
- **Disadvantages:**
  - Requires very stable rock conditions.
  - If a collapse occurs, it involves the whole face, making it more dangerous in weak zones.

## 2. Heading and Bench Method

This method involves dividing the tunnel face into two parts: the top portion (Heading) and the lower portion (Bench). The heading is driven ahead of the bench by a certain distance.

- **Process:**
  1. **Heading:** The top half of the tunnel is excavated first. This allows engineers to inspect the rock quality before the full section is opened.
  2. **Bench:** Once the heading has advanced (typically 3 to 12 meters), the lower portion is excavated.
- **Advantages:**
  - **Stability:** The bench provides a working platform to support the roof of the heading if needed.
  - **Efficiency:** Drilling for the bench can happen simultaneously while muck is being removed from the heading.
  - **Versatility:** It is better suited for larger tunnels or where the rock quality is slightly unpredictable.

## 7. (a) (ii) Solution:

Correction for elevation is calculated as per FAA guidelines which specify that the basic runway length should be increased at the rate of 7% per 300 m rise in elevation.

$$\begin{aligned}\text{Correction} &= \frac{7}{100} \times \frac{E}{300} \times L_b \\ \text{Correction} &= \frac{7}{100} \times \frac{300}{300} \times 1800 = 126 \text{ m} \\ L_1 &= 1800 + 126 = 1926 \text{ m}\end{aligned}$$

Correction for temperature is calculated by first determining the airport reference temperature.

$$\begin{aligned}T_R &= T_a + \frac{T_{\max} - T_a}{3} \\ \Rightarrow T_R &= 28 + \frac{38 - 28}{3} \\ \Rightarrow T_R &= 31.333^\circ\text{C}\end{aligned}$$

The standard temperature at the given elevation is

$$\begin{aligned}T_s &= 15 - 0.0065E \\ \Rightarrow T_s &= 15 - 0.0065 \times 300 \\ \Rightarrow T_s &= 13.05^\circ\text{C}\end{aligned}$$

The rise in temperature above the standard temperature is

$$\begin{aligned}\Delta T &= T_R - T_s \\ \Rightarrow \Delta T &= 31.333 - 13.05 \\ \Rightarrow \Delta T &= 18.283^\circ\text{C}\end{aligned}$$

According to FAA guidelines, the runway length is increased by 1% for every  $1^\circ\text{C}$  rise in temperature above the standard temperature.

$$\begin{aligned}\text{Correction} &= \frac{1}{100} \times \Delta T \times L_1 \\ \Rightarrow \text{Correction} &= \frac{1}{100} \times 18.283 \times 1926 = 352.131 \text{ m} \\ \therefore L_2 &= 2278.131 \text{ m}\end{aligned}$$

The total percentage increase due to elevation and temperature is checked.

$$\begin{aligned}\text{Total percentage increase} &= \frac{2278.131 - 1800}{1800} \times 100 \\ \text{Total percentage increase} &= 26.563\%\end{aligned}$$

Since the total increase is less than 35%, it is acceptable.

Correction for gradient is calculated using the effective gradient of the runway. The effective gradient is defined as the maximum difference in elevation divided by the total runway length.

Elevation changes along the runway are calculated from the grade data.

Section (m)	Grade	Elevation change (m)	Cumulative elevation (m)
0 - 400	+1.20%	$400 \times 0.012 = 4.8$	4.8
400 - 1000	-0.50%	$600 \times -0.005 = -3.0$	1.8
1000 - 1600	+0.40%	$600 \times 0.004 = 2.4$	4.2
1600 - 2200	+0.80%	$600 \times 0.008 = 4.8$	9.0
2200 - 2278.13	-0.20%	$78.131 \times -0.002 = -0.156$	8.844

Maximum elevation = 9 m

Minimum elevation = 0 m

Difference in elevation = 9 m

The effective gradient is,

$$g_{eff} = \frac{9}{2278.131} \times 100$$

$$g_{eff} = 0.3951\%$$

According to FAA guidelines, the runway length should be increased by 20% for every 1% of effective gradient.

$$\text{Correction} = \frac{20}{100} \times g_{eff} \times L_2$$

$$\text{Correction} = \frac{20}{100} \times 0.3951 \times 2278.131 = 180 \text{ m}$$

$$\therefore L_3 = 2278.131 + 180 = 2458.131 \text{ m}$$

The actual length of runway to be provided is

$$\text{Actual runway length} = 2458.131 \text{ m} \simeq 2460 \text{ m}$$



**7. (b) (ii) Solution:****(i) Regime Channel according to Lacey's Theory**

According to Gerald Lacey, a channel is said to be in regime when it has reached a state of equilibrium such that it neither deposits silt (silting) nor erodes its bed and banks (scouring). Lacey's theory suggests that a channel is not merely a "stable" entity but a dynamic one that adjusts its dimensions—width, depth, and slope—to achieve a balance between the water discharge and the silt load it carries.

Lacey stated that even a channel with rigid boundaries cannot be in a true regime; only a channel flowing through an incoherent alluvium (soil of the same grade as that being transported) can achieve this state.

**(ii) Types of Regime**

Lacey identified three distinct stages or types of regime conditions:

**1. Initial Regime**

This is the first stage of stability achieved by a channel. When only the bed slope of the channel varies to adjust for the discharge and silt grade, but the width remains restricted (either by stiff soil or artificial boundaries), the channel is in an initial regime.

- It is a temporary state of stability.
- The channel has not yet adjusted its wetted perimeter or shape.

**2. Final Regime**

A channel is said to have reached a final regime when it is free to adjust its width, depth, and slope to reach ultimate equilibrium. This occurs when the channel flows through a deep layer of the same alluvium it carries.

- The channel adopts a semi-elliptical shape.
- This is the condition for which Lacey's regime equations are primarily derived.

**3. True (Permanent) Regime**

In practice, a "True Regime" is an ideal or theoretical state that is rarely, if ever, achieved. For a channel to be in a true regime, the following conditions must be met simultaneously:

- Discharge ( $Q$ ) must be constant.
- Flow must be uniform.
- Silt charge (amount of silt) and silt grade (type of silt) must be constant.
- The channel must flow through an infinite depth of incoherent alluvium of the same grade as the silt load.

## 7. (c) Solution:

Given data

$$\text{Duration of given U.H. } (D) = 3\text{hr}$$

$$\text{Duration of required U.H. } (T) = 9\text{ hr}$$

$$\text{Lag intervals } (n) = \frac{9}{3} = 3\text{ units}$$

$$\text{Effective Rainfall Intensity } (i) = 1.5\text{ cm/hr}$$

$$\text{Effective Rainfall Depth } (R) = 1.5 \times 9 = 13.5\text{ cm}$$

$$\text{Base Flow} = 20\text{ m}^3/\text{s}$$

The 9-hr unit hydrograph is derived from the 3-hr unit hydrograph by lagging the hydrograph by 3 hr and 6 hr, summing the ordinates, and dividing the sum by 3 because ( $n = 3$ ).

Time (hr)	3-hr U.H. (m <sup>3</sup> /s)	Lagged 3 hr (m <sup>3</sup> /s)	Lagged 6 hr (m <sup>3</sup> /s)	Sum of ordinate (m <sup>3</sup> /s)	9-hr U.H. (Sum ÷ 3) (m <sup>3</sup> /s)	DRH (U.H. × 13.5) (m <sup>3</sup> /s)	FH (DRH + 20) (m <sup>3</sup> /s)
0	0	-	-	0	0	0	20
3	15	0	-	15	5	67.5	87.5
6	50	15	0	65	21.667	292.505	312.505
9	95	50	15	160	53.333	719.996	739.996
12	120	95	50	265	88.333	1192.496	1212.496
15	100	120	95	315	105	1417.5	1437.5
18	70	100	120	290	96.667	1305.005	1325.005
21	45	70	100	215	71.667	967.505	987.505
24	20	45	70	135	45	607.5	627.5
27	8	20	45	73	24.333	328.496	348.496
30	0	8	20	28	9.333	125.996	145.996
33	-	0	8	8	2.667	36.005	56.005
36	-	-	0	0	0	0	20

## 8. (a) (i) Solution:

Processing techniques are used in solid waste managements system to serve the following purposes:

- To improve the efficiency of solid-waste disposal systems
- To recover resources (usable materials)
- To prepare materials for the recovery of conversion product and energy.

Important processing techniques used routinely in municipal solid-waste system include compaction, thermal volume reduction (incineration) and manual separation of waste components.

1. **Mechanical Volume Reduction:** It is the most important factor in the development and operation of solid-waste management systems. Vehicles equipped with compaction mechanisms are used for the collection of most municipal solid wastes. Wastes are compacted to increase the useful life of landfills.
2. **Thermal Volume Reduction:** The volume of municipal wastes can be reduced by more than 90 percent by incineration. Incineration was quite common in the past, however, with more restrictive air pollution control requirements only a limited number of municipal incinerators are currently in operations.
3. **Manual Component Separation:** The manual separation of solid waste components can be accomplished at the source where solid wastes are generated, at a transfer station, at a centralized processing station, or at the disposal site. Manual sorting at the source of generation is the most positive way to achieve the recovery and reuse of materials. The number and types of components salvaged or sorted (e.g., cardboard and high quality paper, metals and wood) depend on the location, the opportunities for recycling and the resale market.

8. (a) (ii) **Solution:**

**Indore Method:** It uses manual turning of piled up mass (refuse + night soil) for its decomposition under aerobic conditions. In this method, layers of vegetable wastes and night soil are alternatively piled in depths of about 7.5 to 10 cm each, to a total depth of about 1.5 m in a trench or above the ground to form a mound called a windrow. The mixture is kept aerobic by turning regularly for 2 to 3 months. This 1 compost mass is

then left for another about  $1$  to  $1\frac{1}{2}$  months without any turning 2 after which, the compost becomes ready for use. The entire process thus takes about 4 months. This method is primarily aerobic in nature.

**Bangalore Method:** The Bangalore method of composting, involves anaerobic decomposition of wastes, and does not involve any turning or handling of the mass, and is, hence, cleaner than the Indore method. This method is, therefore, widely adopted by municipal authorities throughout India. The refuse and night soil, in this method, are therefore piled up in layers in an underground earthen trench of about (10 m × 1.5 m × 1.5 m).

This mass is covered at its top by layer of earth of about 15 cm depth, and is finally left over for decomposition. Within 2 to 3 days of burial, intensive biological action starts taking place and organic matter begins to disintegrate. Considerable heat gets evolved in this process which raises the temperature of the decomposing mass to about 75°C. This heat prevents the breeding of flies by destroying the larvae. After about 4 to 5 months (depending upon the season), the refuse gets fully stabilised and changes into a brown coloured odourless innocuous powdery mass, called as humus. This humus is removed from the trenches, sieved on 12.5 mm sieves to remove stones, broken glass, brickbats, etc. and then sold out in the market as manure. The empty trenches can again be used for receiving further batches of refuse.

## 8. (b) Solution:

$$D_2 = 0.50 \text{ m}, b_2 = 0.03 \text{ m}, h_s = 4.0 \text{ m}$$

$$h_d = 36 \text{ m}, h_{fs} = 2.0 \text{ m}, h_{fd} = 6 \text{ m}, N = 1200 \text{ rpm}$$

$$\beta_2 = 20^\circ, \eta_{\text{ma}} = 0.85, D_p = 0.35 \text{ m}$$

$$\text{Net head} = \text{Static lift} + \text{Friction loss} = 40.0 + 2.0 + 6.0 = 48.0 \text{ m}$$

$$u_2 = \frac{\pi D_2 N}{60} = \frac{\pi \times 0.5 \times 1200}{60} = 31.42 \text{ m/s}$$

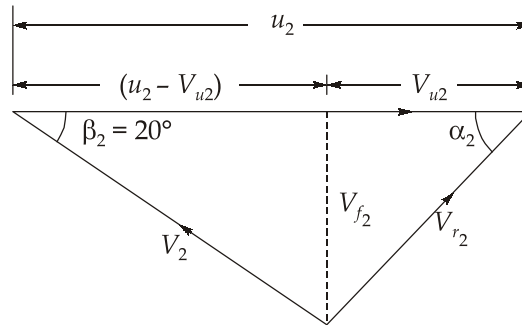
By assuming radial flow at inlet, manometric efficiency.

$$\eta_{\text{ma}} = \frac{gH_m}{u_2 V_{u2}}$$

$$\Rightarrow 0.85 = \frac{9.81 \times 48.0}{31.42 \times V_{u2}}$$

$$\Rightarrow V_{u2} = 17.63 \text{ m/s}$$

From outlet velocity triangle, figure.



$$\tan \beta_2 = \frac{V_{f2}}{(u_2 - V_{u2})}$$

$$\Rightarrow \tan 20^\circ = \frac{V_{f2}}{(31.42 - 17.63)} = 0.3639$$

$$\Rightarrow V_{f2} = 5.02 \text{ m/s}$$

$$\text{Discharge, } Q = \pi D_2 b_2 V_{f2} = \pi \times 0.5 \times 0.03 \times 5.022 = 0.2366 \text{ m}^3/\text{s}$$

$$\text{Velocity in delivery pipe} = V_d = \text{Velocity of suction pipe} = V_s$$

$$V_s = \frac{Q}{\frac{\pi}{4} D_p^2} = \frac{0.2366}{\frac{\pi}{4} \times (0.35)^2} = 2.459 \text{ m/s}$$

$$\frac{V_s^2}{2g} = \frac{V_d^2}{2g} = \frac{(2.459)^2}{2 \times 9.81} = 0.308 \text{ m}$$

Delivery side of pump. Let the pressure on delivery side =  $P_d$

$$\frac{P_d}{\gamma} + \frac{V_d^2}{2g} = h_d + h_{fd} + \frac{V_d^2}{2g}$$

$$\Rightarrow \frac{P_d}{\gamma} = 36 + 6 = 42.0 \text{ m}$$

$$\Rightarrow P_d = 42.0 \times 9.81 = 412.02 \text{ kPa (gauge)}$$

and

$$\frac{P_{\text{atm}}}{\gamma} = h_s + h_{fs} + \frac{P_s}{\gamma} + \frac{V_{cd}^2}{2g}$$

Taking atmospheric pressure at datum pressure,

$$0 = 4 + 2 + \frac{P_s}{\gamma} + 0.308$$

$$\Rightarrow \frac{P_s}{\gamma} = -6.308 \text{ m (Vacuum pressure)}$$

$$\Rightarrow P_s = -(6.308 \times 9.81) = -61.88 \text{ kPa (vacuum)}$$

### 8. (c) (i) Solution:

Given: Safe load,  $Q_{\text{safe}} = 1500 \text{ kN}$

Factor of safety F.O.S. = 3

Unconfined compressive strength

$$q_u = 180 \text{ kN/m}^2$$

Cohesion  $c = \frac{q_u}{2} = 90 \text{ kN/m}^2$

Adhesion factor  $\alpha = 0.6$

Diameter of pile  $d = 0.4 \text{ m}$

Length of pile  $L = 10 \text{ m}$

The ultimate load for the pile group is calculated using

$$Q_{ug} = Q_{\text{safe}} \times F.O.S.$$

$$\Rightarrow Q_{ug} = 1500 \times 3$$

$$\Rightarrow Q_{ug} = 4500 \text{ kN}$$

The ultimate load for a single pile is calculated considering only skin friction because end bearing is neglected.

$$Q_{up} = \alpha \times c \times (\pi dL)$$

$$\Rightarrow Q_{up} = 0.6 \times 90 \times (\pi \times 0.4 \times 10)$$

$$\Rightarrow Q_{up} = 678.584 \text{ kN}$$

The number of piles required in the group is

$$n = \frac{Q_{ug}}{Q_{up}} = \frac{4500}{678.584}$$

$$\Rightarrow n = 6.631 \text{ nos.}$$

To maintain symmetry in a square pile group, adopt

$$n = 9 \text{ piles } (3 \times 3)$$

The spacing of piles for friction piles in clay is generally taken as three times the diameter.

$$s = 3d$$

$$\Rightarrow s = 3 \times 0.4$$

$$\Rightarrow s = 1.2 \text{ m}$$

The width of the pile group is calculated as

$$B = (2s) + d$$

$$\Rightarrow B = (2 \times 1.2) + 0.4$$

$$\Rightarrow B = 2.8 \text{ m}$$

The ultimate capacity of the group acting as a block is

$$Q_{ug(\text{block})} = \text{Perimeter} \times L \times c$$

$$\Rightarrow Q_{ug(\text{block})} = (4B) \times L \times c$$

$$\Rightarrow Q_{ug(\text{block})} = (4 \times 2.8) \times 10 \times 90$$

$$\Rightarrow Q_{ug(\text{block})} = 10080 \text{ kN}$$

The group efficiency is calculated using

$$\eta_g = \frac{Q_{ug(\text{block})}}{n \times Q_{up}} = \frac{10080}{9 \times 678.584}$$

$$\Rightarrow \eta_g = 1.65$$

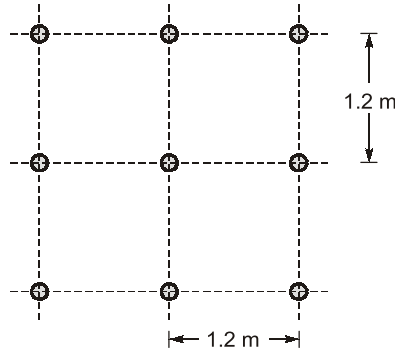
Since  $\eta_g > 1$ , the design is governed by individual pile action.

The total capacity of the pile group is

$$9 \times 678.584 = 6107.256 \text{ kN}$$

This value is greater than the required ultimate load of 4500 kN.

Provide 9 circular piles of 0.4 m diameter and 10 m length arranged in a 3 × 3 square pattern with 1.2 m center-to-center spacing.



**8. (c) (ii) Solution:**

Given:

Parameter	Symbol	Value
Original scale		1 cm: 50 m
Original map length	$L_{map}$	20 cm
Shrunk map length	$L'_{map}$	19.2 cm
Measured map area	$A_{measured}$	145.50 cm <sup>2</sup>
Nominal chain length	$L$	20 m
Chain error	$e$	8 cm = 0.08 m
Actual chain length	$L'$	20.08 m

The shrinkage factor of the map is calculated as

$$S_F = \frac{L'_{map}}{L_{map}} = \frac{19.2}{20}$$

$$\Rightarrow S_F = 0.96$$

The correct map area is obtained by removing the effect of shrinkage.

$$A_{correct\ map} = \frac{A_{measured}}{(SF)^2} = \frac{145.50}{(0.96)^2}$$

$$\Rightarrow A_{correct\ map} = 157.878\text{ cm}^2$$

The ground area corresponding to the corrected map area is determined using the given scale. Since 1 cm = 50 m, therefore

$$1 \text{ cm}^2 = 50^2 = 2500 \text{ m}^2$$

$$A_{\text{ground}} = 157.878 \times 2500$$

$$\Rightarrow A_{\text{ground}} = 394695 \text{ m}^2$$

The correction due to the chain error is applied as

$$A_{\text{true}} = A_{\text{ground}} \times \left(\frac{L'}{L}\right)^2$$

$$\Rightarrow A_{\text{true}} = 394695 \times \left(\frac{20.08}{20}\right)^2$$

$$\Rightarrow A_{\text{true}} = 397858.875 \text{ m}^2$$

### 8. (c) (iii) Solution:

Given:

Parameter	Symbol	Value
Velocity after jump	$v_2$	1.2 m/s
Depth after jump	$y_2$	2.15 m
Acceleration due to gravity	$g$	9.81 m/s <sup>2</sup>
Unit weight of water	$\gamma$	9.81 kN/m <sup>3</sup>

The Froude number after the jump is calculated using

$$F_{r2} = \frac{v_2}{\sqrt{g y_2}} = \frac{1.2}{\sqrt{9.81 \times 2.15}}$$

$$\Rightarrow F_{r2} = 0.261$$

The depth before the jump is obtained from the conjugate depth relation.

$$y_1 = \frac{y_2}{2} \left( \sqrt{1 + 8F_{r2}^2} - 1 \right)$$

$$\Rightarrow y_1 = \frac{2.15}{2} \left( \sqrt{1 + 8(0.261)^2} - 1 \right)$$

$$\Rightarrow y_1 = 0.261$$

The energy loss in the hydraulic jump is calculated using

$$E_L = \frac{(y_2 - y_1)^3}{4y_1y_2}$$

$$\Rightarrow E_L = \frac{(2.15 - 0.261)^3}{4 \times 0.261 \times 2.15}$$

$$\Rightarrow E_L = 3.003 \text{ m}$$

The discharge per meter width is

$$q = y_2 \times v_2$$

$$\Rightarrow q = 2.15 \times 1.2$$

$$\Rightarrow q = 2.58 \text{ m}^2/\text{s}$$

The power dissipated per meter width is calculated as

$$P = \gamma \times q \times E_L$$

$$\Rightarrow P = 9.81 \times 2.58 \times 3.003$$

$$\Rightarrow P = 76 \text{ kW/m}$$

