Department of Civil Engineering

Institute of Technology, GGV

B.Tech. Fourth Year [VIIth Sem.]

Subject: Water Resources Engineering II

Subject Code: 41CE03T

Maximum Marks: 60

Note: (i) Section-A, all questions carry equal marks. 02 Marks allotted for each question.
(ii) Section-B, Attempt any one question from each Unit. All question carry equal Marks.

SECTION – A

Q (1 Gravity dam is most suitable when foundation is
(a) strong (b) rocky but cracked (c) with heavy overburden (d) weak
ANSWER: (a)

Q (2) Embankment dam is a type of dam
(b) Rigid (b) Non Rigid (c) Concrete (d) Masonry
ANSWER: (b)

Q (3) In ogee-shaped spillway for Pointed nose pier Value of K_p is
(a) 0.2 (b) 0 (c) 0.02 (d) 0.01
ANSWER: (b)

Q (4) In a chute spillway, the flow is generally
(a) laminar (b) turbulent (c) critical (d) super critical
ANSWER: (d)

Q (5) For Bligh's creep theory for safety against piping H/L
(a)>1/C (b) <1/C (c) $\leq$ 1/C (d) $\geq$ 1/C
ANSWER: (c)

Q (6) According to the Khosla's theory, the exit gradient in the absence of a downstream cut off is
(a) zero (b) unity (c) infinity (d) None
ANSWER: (c)

Q (7) In Rapid Fall masonry surface is finished with rich cement mortar
(a) 1:5 (b) 1:4 (c) 1:3 (d) 1:6
ANSWER: (c)

Q (8) A fall which consists series of vertical drops
(a) Rapid fall (b) Stepped fall
(c) Rectangle notch fall (d) Trapezoidal notch fall
ANSWER: (b)

Q (9) Cut off wall is provided in
(a) An Aqueduct (b) Supper Passage (c) Siphon Aqueduct (d) Level Crossing
ANSWER: (c)

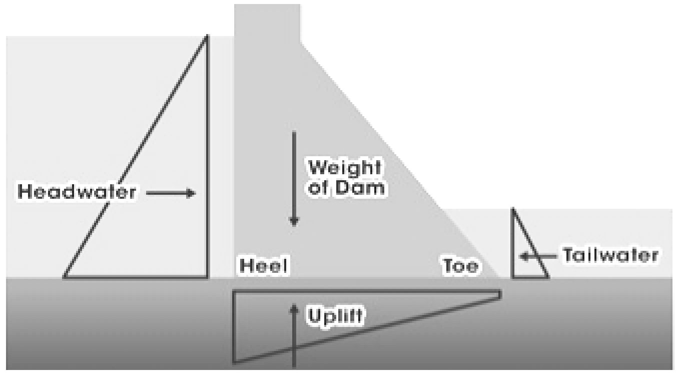
Q (10) When a canal and a drainage approach each other at the same level, the structure so provided, is
(a) An Aqueduct (b) Siphon Aqueduct (c) Inlet and Outlet (d) Level crossing
ANSWER: (d)

SECTION – B

Unit-I

Q (1) (a) Explain in brief the forces which affect the stability of gravity dam.

Forces Acting on Gravity Dam



A gravity dam is subjected to the following main forces:

1. Weight of the dam 2. Water pressure 3. Uplift pressure 4. Wave pressure 5. Earth and Silt pressure 6. Ice pressure 7. Wind pressure 8. Earthquake forces

These forces fall into two categories as:
a) Forces, such as weight of the dam and water pressure, which are directly calculable from the unit weights of the materials and properties of fluid pressures
b) Forces, such as uplift, earthquake loads, silt pressure and ice pressure, which can only be assumed on the basis of assumption of varying degree of reliability. It is in the estimating of the second category of the forces that special care has to be taken and reliance placed on available data, experience, and judgment. It is convenient to compute all the forces per unit length of the dam.

Modes of failure of a gravity dam

a. Overturning

FSR =Overturning Moments / Righting Moments = $\sum M_O / \sum M_R$

b. Sliding

Sliding Factor = $\frac{\mu \Sigma W}{\Sigma P}$

c. Crushing

d. Tension

e. Shear Friction factor (S.F.F.S) = $\frac{\mu \Sigma W + S_a \cdot A}{\Sigma P}$

Gravity dam stability analysis

The most common failure mode for gravity dams is sliding or overturning along or beneath the dam/foundation interface. Stability analysis for gravity dams often is simplified into a two-dimensional rigid body analysis of a cross section of the structure and is focused on stability against sliding. In this analysis, overturning of the dam is considered within the context of its potential influence on sliding. Overturning tendencies express themselves through development of tensile stresses at the heel of the dam. In these cases, sliding stability is analyzed considering a cracked base, which reduces sliding resistance. While the gravity dam stability analysis often is simplified to evaluate failure along the base, it is important to consider kinematically feasible failure mechanisms along joints, foliations and bedding planes or within the rock mass.

OR

(b) A masonry dam 10 m high is trapezoidal in section with a top width of 1 m and bottom width of 8.25 m. The face exposed to water has a better of 1:10. Calculate

(i) Factor of safety against overturning

(ii) Factor of safety against sliding

(iii) Shear friction factor

(iv) Principal stresses at the tot and heel of the dam

Assume coefficient of friction as 0.75 unit weight of masonry as 22.40 kN/m^3, and permissible stress of joint = 1400 kN/m^2, unit wt. of water 9.81 kN/m^3

Marks 08

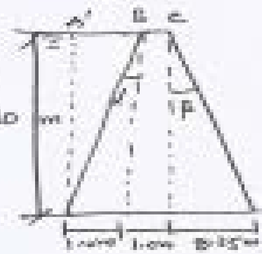
1. Vertical forces

(a) Self wt. of the dam $= \frac{8.25+1}{2} \times 10 \times 1 \times 22.4 = 1036$ kN

(b) Wt. of water column AA'B $= \frac{10 \times 1}{2} \times 9.81 = 49.05$ kN

(c) Up lift force $= \frac{1}{2} \times 8.25 \times 9.81 \times 10 = 404.66$ kN

$\Sigma V = 1036 + 49.05 - 404.66 = 680.39$ kN



2. Horizontal water pressure

$\Sigma H = \frac{wh^2}{2} = \frac{9.81 \times 100}{2} = 490.05$ kN

Moment due to various forces at toe

(a) Due to self wt. of dam $= (\frac{1}{2} \times 1 \times 10 \times 22.4) \times (7.25 + \frac{1}{3})$

$+ \{(1 \times 10 \times 22.4)(6.25 + 0.5)$

$+ (\frac{1}{2}(6.25 \times 10 \times 22.4)(\frac{2}{3} \times 6.25)\} = 5278$ kN-m (+ve)

(b) Due to column of water in AA'B $= (\frac{1}{2} \times 10 \times 1 \times 9.81) \times (8.25 - \frac{1}{3}) = 388.31$ kN-m (+ve)

(c) Due to up lift force $= 404.66 \times \frac{2}{3} \times 8.25 = 2225.63$ kN-m (-ve)

(d) Due to Horizontal water Pressure $=$ ~~490~~ $490.5 \times \frac{10}{3} = 1635$ kN-m (-ve)

$\Sigma M = 5278 + 388.31 - 2225.63 - 1635 = 5666.31 - 3860.63 = +1805.68$ kN-m

Calculation of factor of safety

(i) Factor of safety against overturning $= \frac{M_R}{M_O} = \frac{5666.31}{3860.63} = 1.47 < 1.5$ Unsafe

(ii) Factor of safety against sliding $= \frac{\mu \Sigma V}{\Sigma H} = \frac{0.75 \times 680.39}{490.50} = 1.04 > 1$

(iii) Shear friction factor $= \frac{\mu \Sigma V + bs}{\Sigma H} = \frac{0.75 \times 680.39 + 8.25 \times 1400}{490.50} = 24.60$

Calculation of stresses

$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{1805.68}{680.39} = 2.65$ m But $e = \frac{b}{2} - \bar{x} = \frac{8.25}{2} - 2.65 = 1.475$ m

$\frac{1}{10}$ Compressive stress at toe $= \frac{\Sigma V}{b}[1 + \frac{6e}{b}] = \frac{680.39}{8.25}[1 + \frac{6 \times 1.475}{8.25}] = 167.80$ kN/m^2

$\sqrt{1.01}$

$\frac{6.25}{10}$ Compressive stress at heel $= \frac{\Sigma V}{b}[1 - \frac{6e}{b}] = \frac{680.39}{8.25}[1 - \frac{6 \times 1.475}{8.25}] = -5.55$ kN/m^2

1.391 Principal stress at toe of dam $= p_n \sec^2\beta = 167.60 \times 1.391 = 233.4$ kN/m^2

Principal stress at heel of dam $= p_n \sec^2\alpha - p \tan^2\alpha = -5.49 \times 1.01 - 9.81 \times 10 \times 0.01 = -7.03$ kN/m^2

Unit-II

Q (2) (a) An ogee spillway with vertical upstream face has design discharge of 3000 m^3/sec, crest length of 150m, normal reservoir level is 700m average river bed level is 650m and coefficient of discharge of is 2.2. Determine the crest level. (Assume Kp= 0.01, Ka= 0.1)

Marks 04

$Q = C L H_e^{3/2}$ $\quad H_e = \left(\frac{Q}{CL}\right)^{2/3}$ $\quad H_e = \left(\frac{3000}{2.2 \times 150}\right)^{2/3} = 4.36m.$

$L_e = L - 2(N.K_p + K_a) H_e$

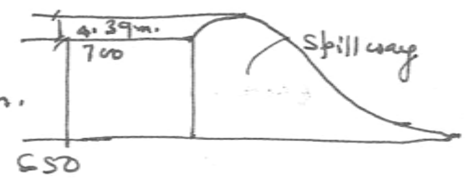
$L_e = 150 - 2(9 \times 0.01 + 0.1) \times 4.36$

$= 148.34m$

$3000 = 2.2 \times 148.34\, H^{3/2}$

$H = 4.39m$ $\quad \therefore$

Crest Level $= 700 + 4.39 = 704.39m.$



(b) Discuss in brief Drop type of spillway. **Marks 04**

Drop structures are commonly used for flow control and energy dissipation. Changing the channel slope from steep to mild, by placing drop structures at intervals along the channel reach, changes a continuous steep slope into a series of gentle slopes and vertical drops. Instead of slowing down and transferring high erosion producing velocities into low non-erosive velocities, drop structures control the slope of the channel in such a way that the high, erosive velocities never develop. The kinetic energy or velocity gained by the water as it drops over the crest of each structure is dissipated by a specially designed apron or stilling basin.

The drop structures discussed here (Figure 2(b)) require an aerated nappe and are, in general, for subcritical flow in the upstream as well as downstream channel. The effect of upstream supercritical flow on drop structure design is discussed in a later section. The stilling basin protects the channel against erosion below the drop and dissipates energy. This is accomplished through the impact of the falling water on the floor, redirection of the flow, and turbulence. The stilling basin used to dissipate the excess energy can vary from a simple concrete apron to an apron with flow obstructions such as baffle blocks, sills, or abrupt rises. The length of the concrete apron required can be shortened by the addition of these appurtenances.

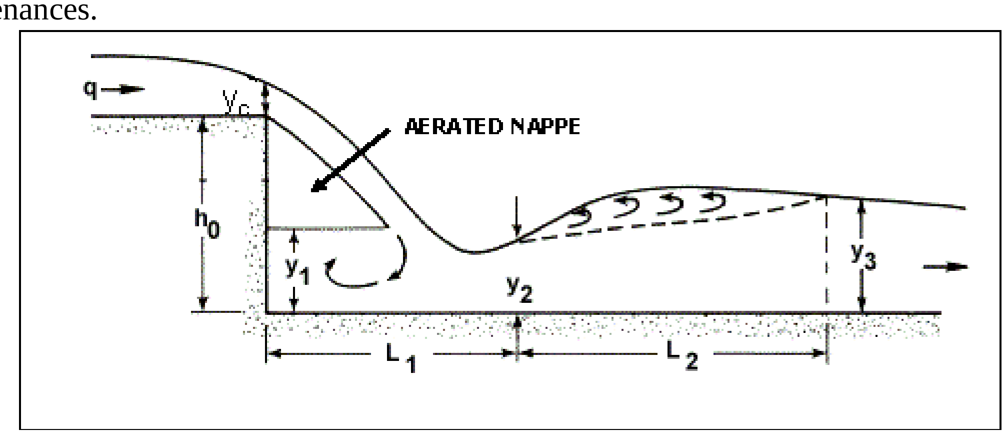
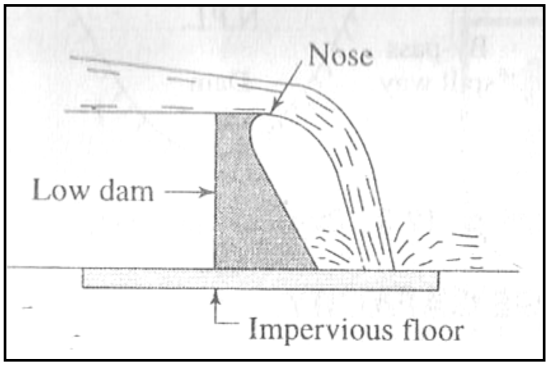


Figure 2(a). Flow Geometry of a Straight Drop Spillway

Drop Spillway

In drop spillway, the over flowing water falls freely and almost vertically on the downstream side of the hydraulic structure. This type of spillway is suitable for weirs or low dams. The crest of the spillway is provided with nose so that the water jet may not strike the downstream base of the structure. To protect the structure from the effect of scouring horizontal impervious apron should be provided on the downstream side. Sometimes a basin is constructed on the downstream side to form a small artificial pool which is known as water cushion. This cushion serves the purpose of energy dissipater.



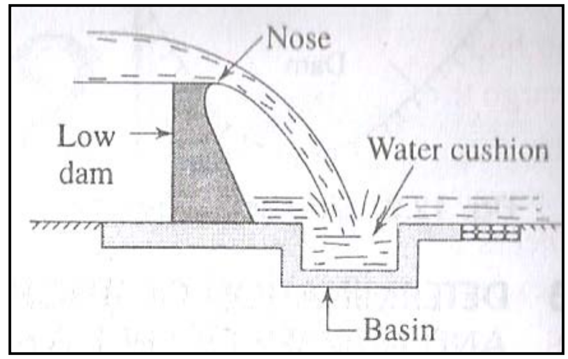


Fig. 2(b) Drop Spillway with Impervious Floor Fig. 2(c) Drop Spillway with Water Cushion

OR

(c) Describe in brief IS type stilling basin, with neat sketches.

Marks 08

Indian Standards Stilling Basins

The stilling basins recommended according to Indian standards may be classified into the following two types.

(1) Stilling basins with horizontal apron (or floor).

(2) Stilling basins with sloping apron (or floor).

(*i*) **Stilling basins with horizontal apron (or floor).** The stilling basins with horizontal apron (or floor) may be provided when the jump height curve coincides with the tail water rating curve ; or the former is slightly above or below the later. As such in this case the requisite depth for the development of the jump may be obtained on an apron near or at the ground level. The stilling basins with horizontal apron (or floor) may be further classified into the following two types.

(*i*) Indian standards stilling basin Type I.

(*ii*) Indian standards stilling basin Type II.

(*i*) *Indian standards stilling basin Type I.* It may be provided when the Froude number F_1 of the incoming flow is less than 4·5. This case is generally encountered on weirs and barrages. This stilling basin is provided with chute blocks, basin blocks (or baffle blocks) and a dentated end sill. The size, spacing and location of the chute blocks for this basin are exactly same as those for Type I stilling basin of U.S.B.R [illegible]. The height and top thickness of the basin blocks for this stilling basin are same as those for Type II stilling basin of U.S.B.R ([illegible]). However, the width and spacing of the basin blocks for this case is equal to the height of the blocks. Further the size of the dentated end sill for this stilling basin is exactly same as the one for Type III stilling basin of U.S.B.R [illegible] The length L of the basin may be obtained for different value of F_1 from the following table :

F_1	2	3	4	4·5
(L/y_2)	3·15	4·0	4·75	5·0

where y_2 = sequent depth

A comparison of the basin lengths of this stilling basin with those of U.S.B.R. Type I stilling basin shows that the former are shorter than the later, which is obviously due to the use of basin blocks in the former.

(*ii*) *Indian standards stilling basin Type II.* It may be provided when the Froude number F_1 of the incoming flow is greater than 4·5. This case is a general feature for medium and high dams. This stilling basin is also provided with chute blocks, basin blocks (or baffle blocks) and a dentated end sill. However, when the velocity of flow at the location of the basin blocks exceeds 15 m/s, no basin blocks are provided and in that case the floor of the basin should be kept at a depth equal to the sequent depth y_2 below the tail water level. The size, spacing and location of the chute blocks for this stilling basin are exactly same as those for Type II or Type III stilling basins of U.S.B.R. Further the size, spacing and location of the basin blocks if provided for this stilling basin are exactly same as those for Type II stilling basin of U.S.B.R. Also the dentated end sill provided for this stilling basin is exactly same as Type III stilling basin of U.S.B.R. (or Type I stilling basin of Indian Standards). The length L of this stilling basin may be obtained for different values of F_1 from the following table :

F_1	5	6	8	10	12
(L/y_2)	2·9	3·2	3·7	3·9	4·0

where y_2 = sequent depth.

(2) **Stilling basin with sloping apron (or floor).** When the tail water depth is too large as compared to the sequent depth y_2, a drowned jump will develop which is not desirable. In such a case a stilling basin with a sloping apron (or floor) may be provided as it will alow a clear jump to be developed at suitable level on the sloping apron (or floor). The stilling basins with sloping apron (or floor) may be further classified into the following two types.

(*i*) Indian standards stilling basin Type III.

(*ii*) Indian standards stilling basin Type IV.

Indian standards stilling basin Type III is recommended for the case where tail water rating curve is above the jump height curve at all discharges. It is usually provided with a sloping apron (or floor) for its entire length.

Indian standards stilling basin Type IV is suitable for the case where the tail water depth at maximum discharge exceeds the sequent y_2 considerably but is equal to or slightly greater than y_2 at lower discharges. It is provided with a partly sloping and partly horizontal apron (or floor).

For both these stilling basins except a solid or dentated end sill no other accessories are provided.

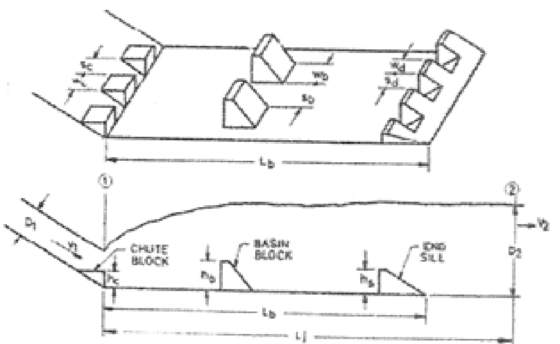


Figure 2(i): Dimension Sketch for Basin I and II

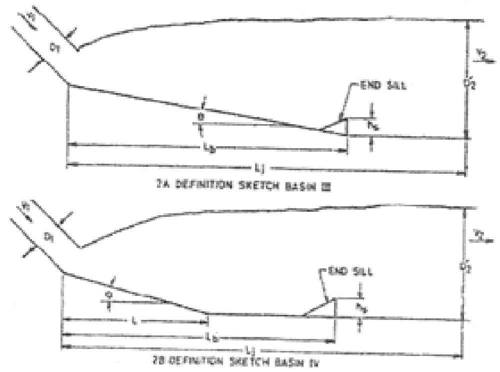
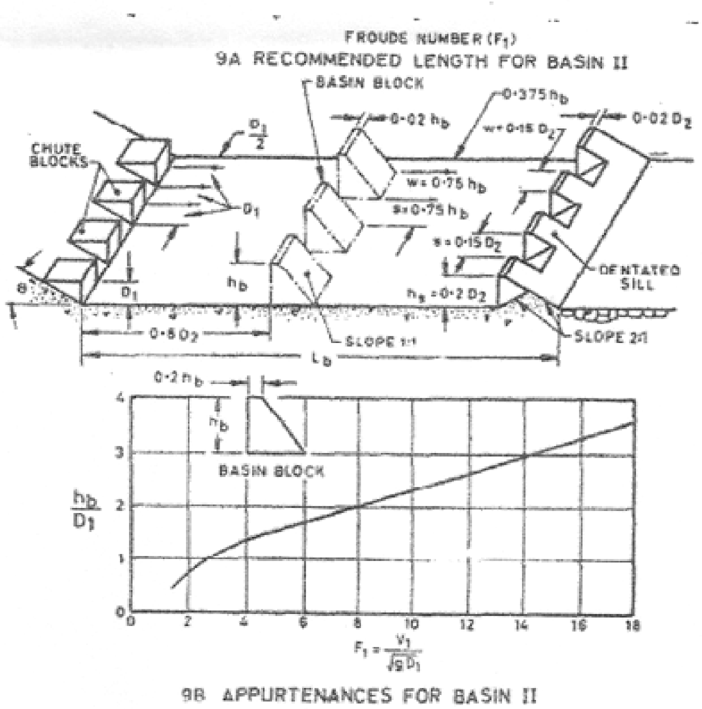


Figure 2(ii): Dimension Sketch for Basin III and IV



Unit-III

Q (3) (a) What is importance of exit gradient? How would you check it? **Marks 08**

Correct estimation of uplift force, seepage discharge and exit gradient is very important in stability analysis of hydraulic structures. In this research, by carrying out a set of experiments on a laboratory model, the application of various methods for estimating uplift pressure and seepage discharge under hydraulic structures, and the exit gradient have been investigated. The results show that for the soil type used in the experiments, the method of Khosla *et al.* gives a better estimation of seepage effects than creep theories. Moreover, compared with Lane's method, it is found that Bligh's theory has better agreement with the observed data of uplift pressure, seepage discharge and piping. By using the finite-element method, the magnitude of pressure head upon a dam foundation in different conditions is calculated, and a relationship between the anisotropic conductivity ratio and the ratio of coefficients in Lane's creep theory is introduced. It is found that while the accuracy of Lane's theory is reduced by increasing the anisotropic conductivity ratio, the new relation estimates the amount of uplift force very well. Based on the finite-element method, a set of graphs is presented to estimate the exit gradient in different conditions with the presence of a cutoff wall at the downstream end or without any cutoff wall.

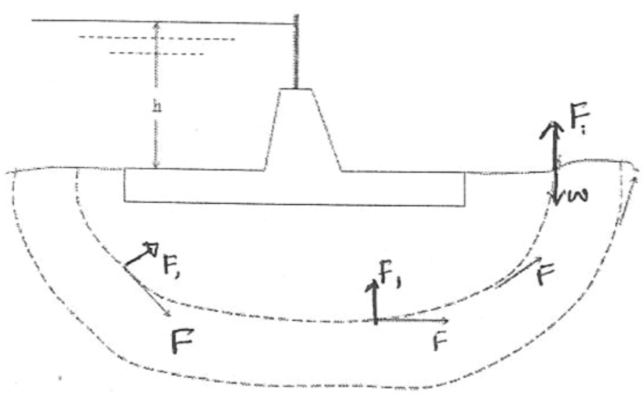


Figure 3(a) Seepage below Weir

Every particle of water while seeping through the sub-soil, at any position will exert a force **F**, which will be tangential to the streamline at any point. As the streamlines bend upward, the tangential force f will be having a vertical component $\mathbf{F_1}$. Also at that point, there will be a downward force W due to the submerged weight of the soil particle. Thus at that point there will be two forces on the particle; one upward vertical component of **F**, and the other, the submerged weight. It is evident that if the soil particle is not to be dislodged, then the submerged weight must be greater than the upward vertical component of **F**. The upward vertical component force at any point is proportional to the water pressure gradient dp/dx.

Hence for stability of the soil and for the prevention of erosion and piping, the seeping water when it emerges at the downstream side, at the exit position, the force f1 should be less than the submerged weight W. In other words the exit gradient at the downstream end must be safe.

If at the exit point at the downstream side, the exit gradient is such that the force f1is just equal to the submerged weight of the soil particle, then that gradient is called critical gradient. Safe exit gradients = 0.2 to 0.25 of the critical exit gradient.

Values of safe exit gradient may be taken as:

0.14 to 0.17 for fine sand

0.17 to 0.20 for coarse sand

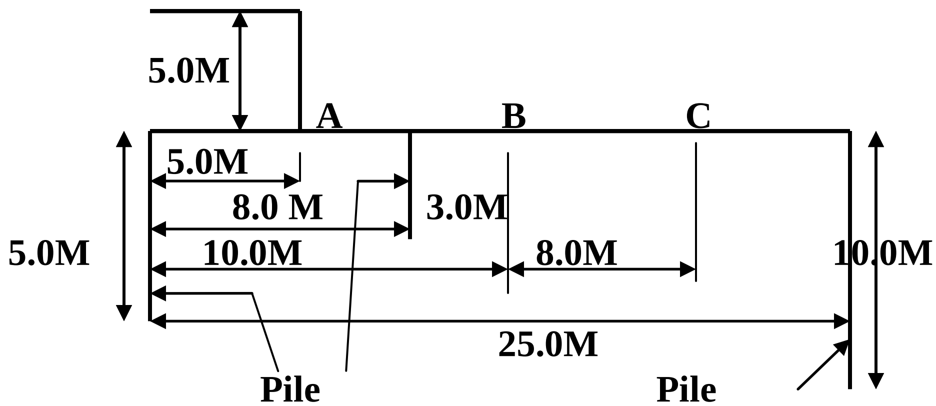
0.20 to 0.25 for shingle

For the standard form consisting of a floor of a length b, and a vertical cut-off of depth d, the exit gradient at its downstream end is given by:

Exit gradient GE = (H/d) x 1/л*√λ

OR

(b) The figure shows the section of a hydraulic structure on permeable foundation, Calculate the average hydraulic gradient according to Bligh's creep theory find uplift pressure at points A,B and C as shown in figure and the floor thickness required at these points.



Marks 08

Total Length of Creep $= 25 + 2(d_1 + d_2 + d_3)$

$= 25 + 2(5 + 3 + 10)$

$= 25 + 36 = 61\ m$

Hydraulic Gradient $= \frac{5}{61} = 0.0819$

(i) Up lift Pressure at a point A, 5 m from U/s

Length of creep up to A $= 5 + 2 \times 5 = 5 + 10 = 15\ m$

$\because$ Unbalanced head $h_A = 5\left(1 - \frac{15}{61}\right) = 3.77\ m$

$\therefore$ up lift Pressure $= \gamma h_A = 9.81 \times 1000 \times 3.77$

$= 36.98\ kN/m^2$

Thickness $t_A = \frac{4}{3} \frac{h_A}{2.24-1} = \frac{4}{3} \times \frac{3.77}{1.24} = 4.053\ m$

(ii) up lift Pressure at a point B, 10m from U/s

Length of creep up to B $= 10 + 2(5+3) = 10 + 16 = 26\ m$

$\because$ Unbalanced head $h_B = 5\left(1 - \frac{26}{61}\right) = 2.86\ m$

$\therefore$ Up lift Pressure $= \gamma h_B = 9.81 \times 1000 \times 2.86 = 28.14\ kN/m^2$

Thickness $t_B = \frac{4}{3} \frac{h_B}{2.24-1} = \frac{4}{3} \times \frac{2.86}{1.24} = 3.075\ m$

(iii) Up lift Pressure at a point C, 18 m from U/s

Length of Creep up to C $= 18 + 2(5+3) = 18 + 16 = 34\ m$

$\because$ Unbalanced head $= h_c = 5\left(1 - \frac{34}{61}\right) = 2.21\ m.$

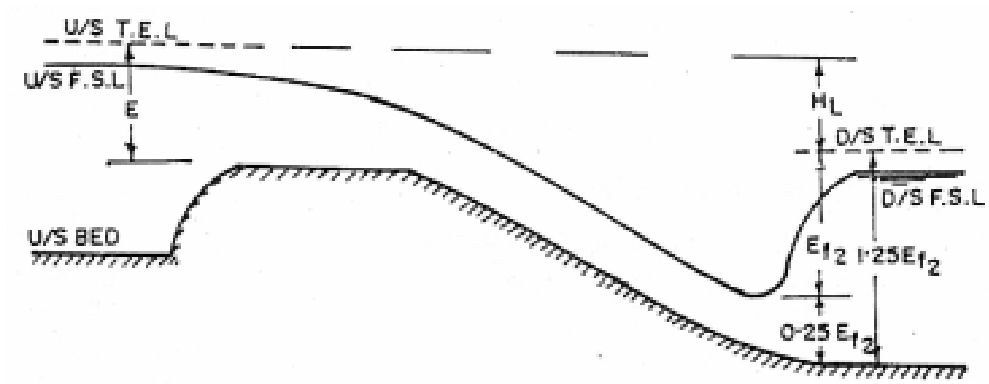
$\therefore$ up lift Pressure $= \gamma h_c = 9.81 \times 1000 \times 2.21 = 21.71\ kN/m^2$

Thickness $t_c = \frac{4}{3} \times \frac{h_c}{2.24-1} = \frac{4}{3} \times \frac{2.21}{1.24} = 2.376\ m.$

Unit-IV

Q (4) (a) Explain in brief Inglis type fall. **Marks 08**

The efficiency of the hydraulic jump as a very potent means of destroying the energy of canal falls is used in glacis falls. The glacis type of fall utilizes the standing wave phenomenon for dissipation of energy. The glacis fall may be (i) straight glacis type, or (ii) parabolic glacis type, commonly known as the Montague type. The straight glacis fall may be with baffle platform and baffle wall. In such a case, the formation of jump takes place on the baffle platform. This type was first developed by Inglis and is called Inglis fall.



OR

(b) Explain in brief operation of Drum gate. **Marks 08**

Several dam projects include spillway drum gates. These gates are unique in that the gates are lowered to release water. Water flows over the top of the gates. Figure 4(b) shows a cross section through a drum gate. The gate is a hollow steel structure, hinged on the upstream side that floats in a bath of water contained within a float chamber bounded by reinforced concrete walls. Water is let into the float chamber to raise the gate (raising the reservoir level and shutting off spillway flow) and water is released from the chamber to lower the gate (initiating spillway flows and lowering the reservoir level). Gate seals prevent water from going around the gates, and gate stops or seats keep the gates from rotating too far up and out of the float chamber. Since the gates are hollow and relatively water tight in order to allow them to float, a drain usually connected from the interior of the gate by a flexible hose to an outlet through the concrete at some location to allow any water that gets into the gate to exit.

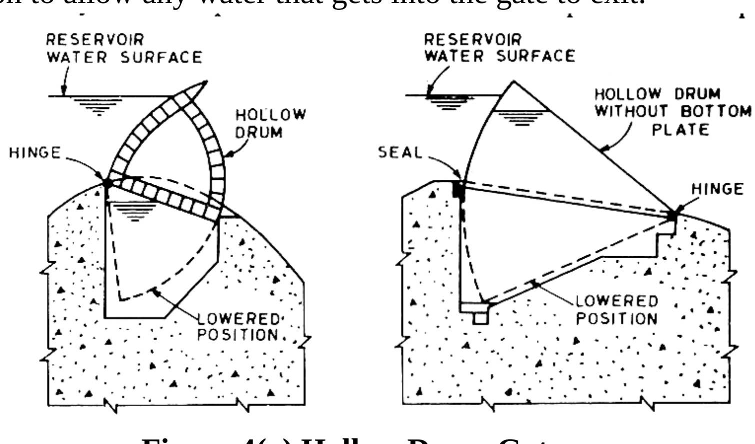


Figure 4(a) Hollow Drum Gate

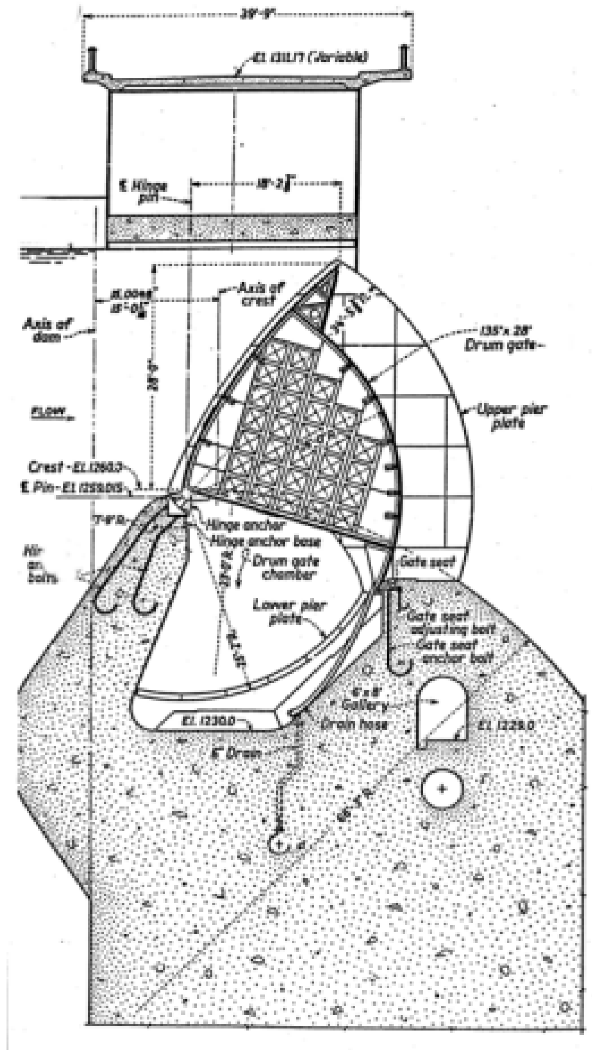


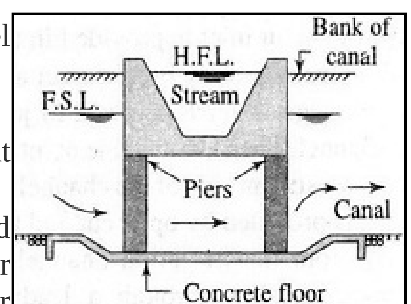
Figure 4(b) Cross Section of Drum Gate

Unit-V

Q (5) (a) What is siphon super passage? Explain it with neat sketch. **Marks 08**

When the canal is in full cutting and there is sufficient head way for the canal water a Super Passage may be proposed. That is if the Full supply level (F.S.L) of canal is below a depth of free board required for canal from bed level of drain crossing, at this point a super passage may be constructed to let the discharge of drain without causing any damage to the canal.

If two canals cross each other and one of the canals is siphoned under the other, then the hydraulic structure at crossing is called "canal siphon". For example, lower Jhelum canal is siphoned under the Rasul-Qadirabad (Punjab, Pakistan) link canal and the crossing structure is called "L.J.C siphon"



Siphon super passage

In case of siphon the FSL of the canal is much above the bed level of the drainage trough, so that the canal runs under the siphonic action.

The canal bed is lowered and a ramp is provided at the exit so that the trouble of silting is minimized. Reverse of an aqueduct siphon

In the above two types, the inspection road cannot be provided along the canal and a separate bridge is required for roadway. For economy, the canal may be flumed but the drainage trough is never flumed.

OR

(b) Explain in brief Inlet and outlet CD works with neat sketch. **Marks 08**

Inlet and outlet

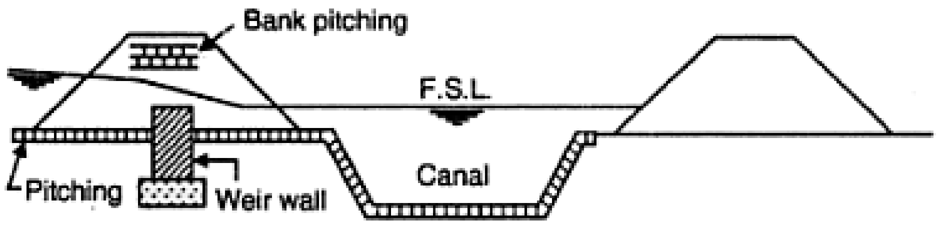


Figure 5(a)Bank Pitching in Inlet and Outlet

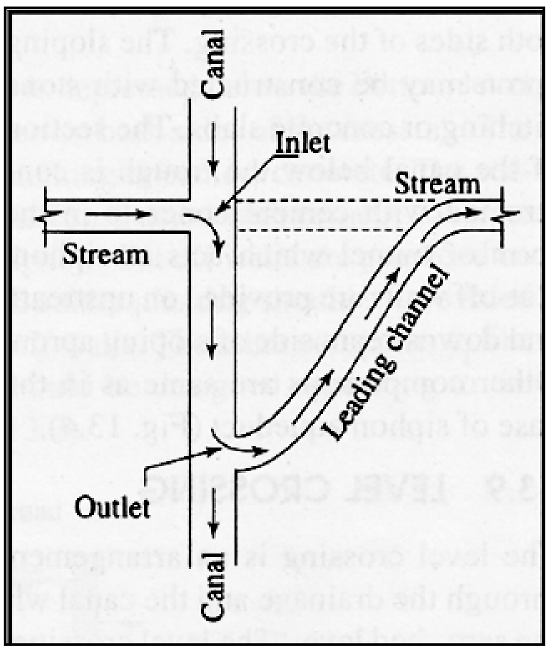


Figure 5(b) Inlet and outlet

When the drainage flow is small it may be absorbed into the canal through inlets. The flow in the canal may be balanced, if necessary (in the case of small canals), by providing suitable outlets (or escapes). The inlet and outlet structures must also be provided with energy dissipators wherever necessary.

In the crossing of small drainage with small channel no hydraulic structure is constructed. Simple openings are provided for the flow of water in their respective directions. This arrangement is known as inlet and outlet.

In this system, an inlet is provided in the channel bank simply by open cut and the drainage water is allowed to join the channel. At the points of inlet and outlet, the bed and banks of the drainage are protected by stone pitching.