2019

RANK improvement WORKBOOK

Answer key and Hint of
Objective & Conventional *Questions*

Civil Engineering

Irrigation Engineering

MADE EASY Publications

MADE EASY
Publications

1 Water Requirements of Crops

LEVEL 1 Objective Questions

1.	(d)
2.	(c)
3.	(b)
4.	(d)
5.	(d)
6.	(a)
7.	(b)
8.	(c)
9.	(c)
10.	(b)
11.	(a)
12.	(d)

LEVEL 2 Objective Questions

13.	(5200)
14.	(c)
15.	(a)
16.	(b)
17.	(b)
18.	(d)
19.	(c)
20.	(a)
21.	(a)
22.	(c)
23.	(d)
24.	(b)

LEVEL 3 Conventional Questions

Solution : 1

$Q = 11.703924\ m^3/s$

Solution : 2

Evapotranspiration, $C_u = 34.1806$ cm

$FIR = 35.8163$ cm

Solution : 3

(i) Gross storage for reservoir = 13.35 Mm^3

(ii) Full supply discharge = 66.47 cumec

Solution : 4

$t = 39.06$ minutes

$A_{max} = 0.144$ ha

Solution : 5

$C_u = 34.16$ cm

$FIR = 35.8$ cm

Solution : 6

$Q = 4.595$ cumecs

Solution : 7

The favourable conditions to adopt Sprinkler Irrigation method are:

(i) When the topography is irregular.
(ii) When the gradient of the land is steep and the soil is easily erodible.
(iii) When the permeability of the soil is high.
(iv) When there is scarcity of water.
(v) When crops to be grown are such:
  (a) as to require humidity control, e.g., tobacco
  (b) crops having shallow roots
  (c) crops requiring high and frequent irrigation

A sprinkler system can be classified as:

(a) Permanent system : In this system, pipes are permanently buried in way which does not obstruct farming activity.
(b) Semi-permanent System : In this system the main lines are buried in the ground and the laterals are portable.
(c) Portable system : In this system, the mains as well as laterals are portable.

Solution : 8

The duty of canal water by using the water in an economic manner. There are certain measures which can be adopted during field preparation, sewing and handling irrigation supplier, which will improve the duty of water.

A. During field preparation:
   (i) levelling of the land should be done.
   (ii) Crop rotation should be adopted.
   (iii) Leaching of alkaline soil.
   (iv) Improved cultivation method should be adopted.
   (v) Treatment of porous soil should be done to reduce seepage.

B. During hadling irrigation supplies:
   (i) lining of the canals should be done.
   (ii) Free flooding methods can be replaced by furrow irrigation methods.
   (iii) Subsurface and Drip irrigation should be adopted.
   (iv) Irrigation supplies can be used economically by providing proper education to farmers.
   (v) Source of water should be nearer and should be capable of delivering required amount of water.

Solution : 9

105.93 Mm^3

Solution : 10

$$\eta_a = \left(\frac{t_d}{T}\right)^{n+1} \times 100$$

Solution : 11

(a) 0.08 or 8%

(b) 92.24%

(c) 31.97 minutes

■■■■

2 Design of Stable Channels

LEVEL 1 Objective Questions

1. (2.6425)
2. (1/3371)
3. (b)
4. (c)
5. (a)
6. (c)
7. (c)
8. (b)
9. (d)
10. (b)
11. (a)
12. (d)
13. (c)

LEVEL 2 Objective Questions

14. (c)
15. (c)
16. (d)
17. (c)
18. (a)
19. (d)
20. (b)
21. (b)
22. (b)
23. (b)
24. (d)
25. (a)
26. (b)

LEVEL 3 Conventional Questions

Solution : 1

Shield proposed a dimensionally homogeneous equation for sediment having uniform size taking into account specific gravity of sediment.

$$\frac{qS_s}{q_b} = 10\left[\frac{\tau_0 - \tau_c}{\nu_w \cdot d(S_s - 1)}\right]$$

q_b – bed load transported in m^3/sec per m width of channel.

S_s = sp. gravity of bed grain

d = dia. of bed grain

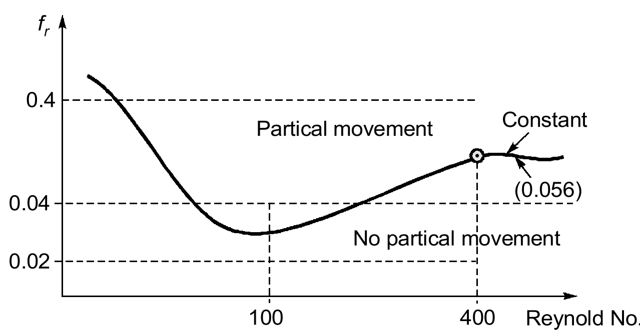
q – discharge per unit width

y_w – unit wt. of fluid

τ_0, τ_c – Shear stresses.

$$\tau_c = \rho_w (V^*)^2$$

$$f_s = \frac{\tau_c}{\nu_w d(S_s - 1)} \qquad V^* = \text{Shear velocity} = \sqrt{\frac{\tau_0}{\rho_w}}$$



$$\text{Boundary Reynold No.} = \frac{V^* d}{\nu}$$

for Re > 400, $f_s = 0.056$

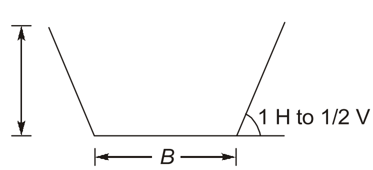
Hence for non scouring coarse alluvium ($d > 6$ mm)

$$\text{resisting shear stress,} \qquad \frac{\tau_c}{\nu_w d(S_s - 1)} = 0.056$$

Solution : 2

$H_L = 0.0754$ m

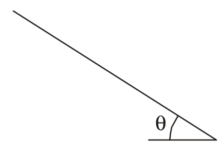
Solution : 3



$$B = 3 \text{ m}$$
$$Y = 1.2 \text{ m}$$
$$\text{Side slope} = 1 \text{ H to } 1/2 \text{ V}$$

slope 1V : mH
2

$$P = B + 2y\sqrt{1+m^2} = B + 2y\sqrt{1+2^2}$$
$$= B + 2\sqrt{5}y = 3 + 2\sqrt{5} \times 1.2 = 8.366 \text{ m}$$
$$A = (B + my)y = (3 + 2 \times 1.2) \times 1.2 = \mathbf{6.48 \ m^2}$$



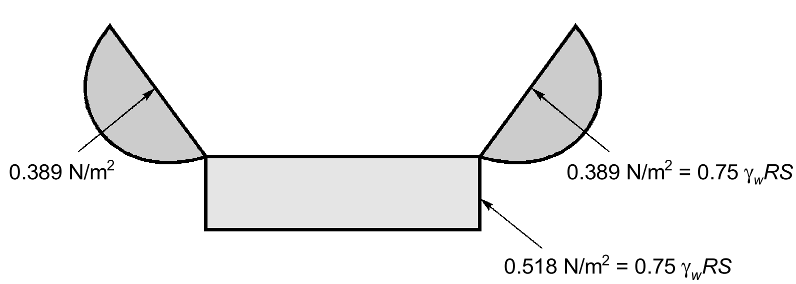
$$\mathbf{S = \tan\theta} \quad \text{as } \theta \text{ is very small } \sin\theta \simeq \tan\theta = S$$
$$\tau = \text{Shear stress} = \frac{(y_w AL)\sin\theta}{\text{wetted area}}$$
$$\tau = \frac{(y_w AL)S}{PL} \Rightarrow \tau = y_w RS$$
$$n = 0.012$$

For rigid boundary channel, $n = \frac{1}{24} d^{1/6}$ **(d in m)** (Stickler formula)

$$0.012 = \frac{1}{24} d^{1/6}$$
$$d = 5.71 \times 10^{-4} \text{ m}$$
$$d \geq 10.8 \text{ RS}$$
$$5.71 \times 10^{-4} = 10.8 \times \left(\frac{6.48}{8.366}\right) \times 5$$

$\Rightarrow$ $$S = 6.82 \times 10^{-5}$$

$$\tau_{\text{required at bed}} = y_w \text{RS}$$
$$= 9.81 \times 1000 \times \left[\frac{6.48}{8.366}\right] \times 6.82 \times 10^{-5}$$
$$\tau = 0.518 \text{ N/m}^2$$
$$\tau_{av \text{ on sides}} = 0.75 \, \nu_w \text{RS} = 0.75 \times 0.518 = 0.389 \text{ N/m}^2$$



- Shear stress required to move a grain on side slopes is less than that shear stress required to move grain on canal bed.
- Shear stress will be zero at junction of bed and sides of section because it will be a sharp joint.

Solution : 4

For

Balancing Depth : In construction of a canal section, if the quantity of excavated earth can be fully utilized for making the banks on both sides, then that canal section is known as economical section. The depth of cutting for that ideal condition is known as balancing depth. In this case, no borrow pit on spoil banks need to be constructed. This condition may not occur in all the cases. It happen only when the canal section is partially in cutting and partially in banking. The cost of earth work will also be balanced.

Assume,

(i) Base width of canal = B

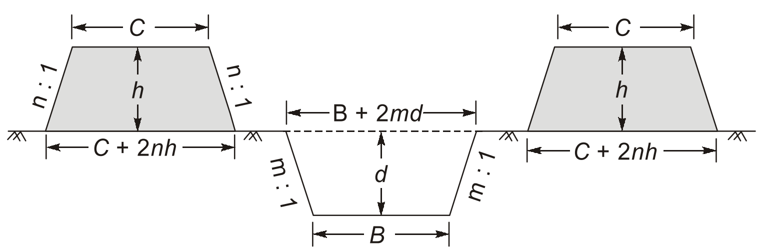
(ii) Side slope of cutting = $m : 1$

(iii) Side slope in Banking = $n : 1$

(iv) Top width of Bank = C

(v) Height of bank above $G.L.$ = h

Let 'd' be the balancing depth of cutting.



$$\text{Area of banking} = 2 \times \frac{[C + (C + 2nh)]}{2} \times h$$

$$= 2\,(C + nh)h$$

$$\text{Area of cutting} = \frac{B + (B + 2md)}{2} \times d$$

$$= (B + md)d$$

For economical section,

$$\text{Area of banking} = \text{Area of cutting}$$

$$2\,(C + nh)h = (B + md)d$$

$$2(C + nh)h = Bd + md^2$$

$$md^2 + Bd - 2(C + nh)\,h = 0$$

$$d = \frac{-B \pm \sqrt{B^2 + 4 \times m \times 2(C + nh)h}}{2m}$$

$$d = \frac{-B \pm \sqrt{B^2 + 8mh(C + nh)}}{2m}$$

$$d = \frac{\sqrt{B^2 + 8mh(C + nh)} - B}{2m} \quad \text{(Neglecting the Negative value)}$$

Solution : 5

390.70 tonnes per day

Solution : 6

$C = 16.81 \times 10^{-6}$ ppm

Solution : 7

$b_{min} = 3$ m

Solution : 8

(a) Average boundary stress $\tau_0 = 2.918$ N/m^2

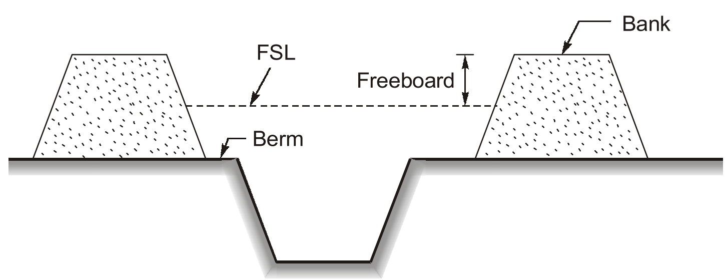
(b) Percentage saving in earthwork = 34.55%

Solution : 9

c = 290 ppm

Solution : 10

(a) Berms :

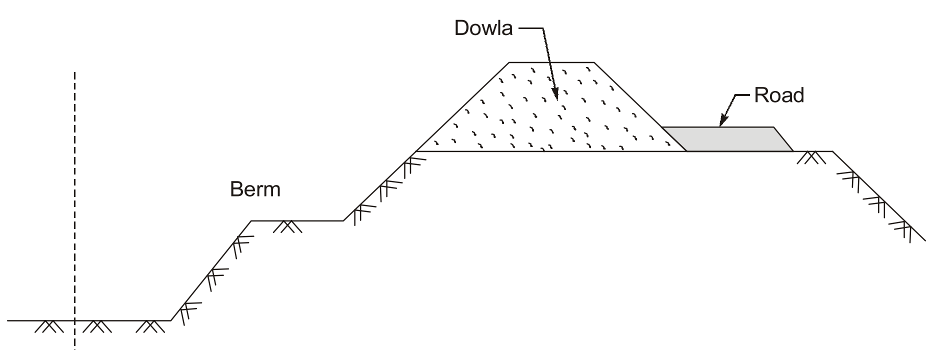


Berm is the horizontal distance left at ground level between the toe of the bank and top edge of the cutting.

Silt deposited on the sides (Berm) is very fine and impervious and thus reduces seepage and leakage.

Berm helps the channel to attain regime condition and give additional strength to the banks.

(b) Dowlar :

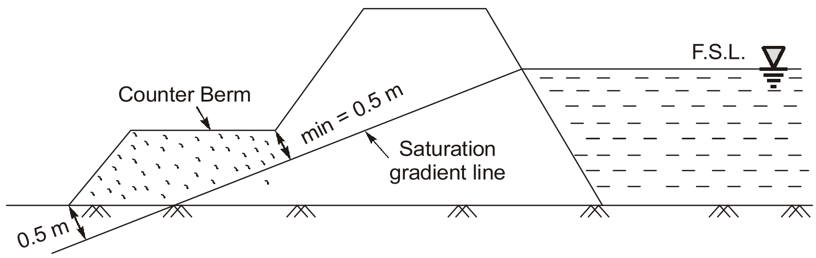


As shown in figure, a Dowel or Dowla is provided on the side of a service road between the service road and channel.

They acts as a kerb on the side of the roadway and as a measure of safety in driving.

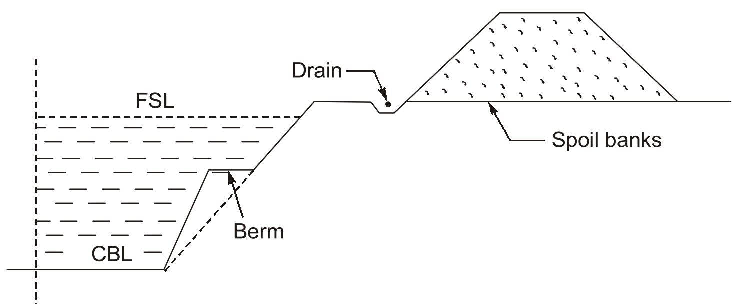
They also help in preventing slope erosion due to rains.

(c) Back Berm or Counter Berm



In order to avoid saturation gradient line cutting downstream end of the bank, counter berm is provided and is ensured that the saturation line remains covered at least 0.5 m.

(d) Spoil Banks



The deposits of earthwork (which is in excess of balancing depth requirement) in form of heaps on both the banks or only on bank is called as spoil banks. The heaps are discontinued at suitable intervals and longitudinal drains are excavated for the disposal of rain water.

It may be noted that the disposal of the excess earth by mechanical transport may become very costly and thus deposition of these earth in form of spoil bank proves to be economical.

■■■■

3 Design and Construction of Gravity Dams

LEVEL 1 Objective Questions

1.	(d)
2.	(a)
3.	(d)
4.	(c)
5.	(b)
6.	(b)
7.	(d)
8.	(a)
9.	(b)
10.	(c)
11.	(d)
12.	(a)

LEVEL 2 Objective Questions

13.	(c)
14.	(d)
15.	(a)
16.	(b)
17.	(d)
18.	(d)
19.	(b)
20.	(a)
21.	(b)
22.	(b)
23.	(c)

LEVEL 3 Conventional Questions

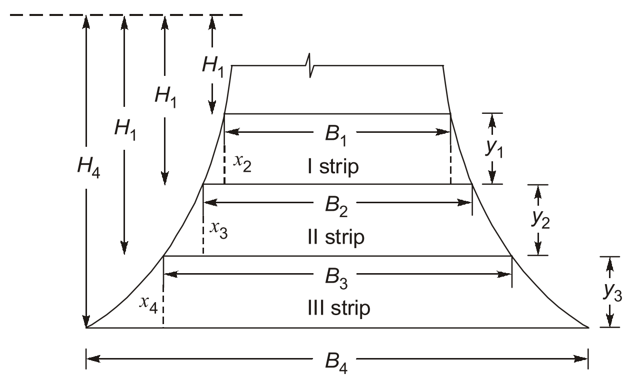
Solution : 1

$q = 3.29 \times 10^{-6}$ m^3/s/m

Solution : 2

Zone method of design of high gravity dams:

When height of dam exceeds H_1 is given by $\dfrac{f}{\gamma_w(S_c - 1)}$; then its upper height equal to H_1 can be designed as low gravity dam and its remaining height is designed by dividing it into number of suitable strips. The design of each strip can be carried out as formulas given below. The basis of these formula is that maximum normal stress. Should not exceed the allowable value (f) and at same time, the section should be economical.



Design of I strip:

The total base width required at the bottom of the 1st strip (B_2) is given by:

$$B_2 = \sqrt{\frac{\gamma_w H_2^3}{f}\left[1 + \frac{\gamma_w{}^2 H_2^4}{4W_2^2}\right]}$$

where, f = allowable compressive stress of the dam material

W_1 = Total vertical height of dam and water above top of I strip

W_2 = Total wt. of dam portion and water above the bottom of II strip

Design of II strip:

$$B_3 = \sqrt{\frac{\gamma_3 H_3^3}{f}\left[1 + \frac{\gamma_w{}^2 H_3^4}{4W_3^2}\right]}$$

where, H_3 = Height of dam portion from M.W.L. to the bottom of II strip

W_3 = Total vertical wt. of dam and water above the bottom of II strip

Solution : 3

$F = 1.63$ (Safe)

Solution : 4

11.11 years

Solution : 5

$\sigma_{toe} = 1586.66$ kN/m^2

$\sigma_{heel} = -49.2$ kN/m^2

Solution : 6

$Q_{min} = 25.14$ m^3/s

Solution : 7

For No Tension condition, the base width may be expressed as

$$b = \frac{H}{\sqrt{S-1}}, \qquad S = \text{specific gravity of concrete (or masonary)}$$

The compressive stress at the upstream face of the dam is given by

$$\sigma_v = \frac{\Sigma v}{b}\left(1-\frac{6e}{b}\right)$$

where, Σv = sum of all verticle forces acting on dam

e = eccentricity

According to Maurice Levy criterion

$$\frac{\Sigma v}{b}\left(1-\frac{6e}{b}\right) = \gamma_w \cdot H \qquad \text{...(i)}$$

γ_w = specific weight of water

Since uplift pressure is not acting in accordance with Maurice criterion, the other forces acting on the dam

water pressure, $P = \frac{\gamma_w \cdot H^2}{2}$

$$\text{Weight of the dam, } W = \frac{1}{2} bH \cdot S\gamma_w$$

Thus, in present case, $\Sigma v = W = \frac{1}{2} bH \cdot S \cdot \gamma_w$

Substituting this value in equation (i), we get

$$\frac{W}{b}\left(1-\frac{6e}{b}\right) = \gamma_w \cdot H$$

$$\frac{\frac{1}{2} bH \cdot S \cdot \gamma_w}{b}\left(1-\frac{6e}{b}\right) = \gamma_w \times H$$

$$e = \frac{b}{6} - \frac{b}{3S} \qquad \text{...(ii)}$$

Moreover, if 0 is the point of intersection of the resultant with base of the dam, then taking the moment of the forces about point 0, we get

$$\frac{\gamma_w H^3}{2} \times \frac{H}{3} = \frac{1}{2} bHS\gamma_w\left(e+\frac{b}{6}\right)$$

$$e = \left(\frac{H^2}{3Sb} - \frac{b}{6}\right) \quad \text{...(iii)}$$

Equating equation (ii) and (iii) we get

$$\frac{b}{6} - \frac{b}{3S} = \frac{H^2}{3Sb} - \frac{b}{6}$$

$$b = \frac{H}{\sqrt{S-1}}$$

Which is same as the base width required for No Tension with unit intensity factor.

Solution : 8

The earthquake waves are capable of shaking the earth in every possible direction. For design purpose the earthquake wave can be resolved in vertical and horizontal components. Hence, two accelerations, i.e., horizontal acceleration (α_n) and vertical acceleration (α_v) are induced by an earthquake.

Effect of Vertical Acceleration :

When vertical acceleration is in upward direction :

- foundation of the dam will be lifted upward and becomes closer to body of the dam.
- effective weight of the dam will increase
- stress developed will increase

When vertical acceleration is in downward direction :

- foundation will tend to more downward away from body
- effective weight and stability of dams reduces
- it is worst case of design
- in this case inertia force, exerted by acceleration is given by

$$\frac{W}{g}\cdot\alpha_v \qquad \text{(i.e., Force = Mass × Acceleration)}$$

$$\text{Net effective weight of dam} = W - \frac{W}{g}\cdot\alpha_v$$

If $\alpha_v = k_v \times g$, where k_v is the fraction of gravity adopted for vertical acceleration such as 0.1 or 0.2 etc

$$\therefore \quad \text{Net effective weight of dam} = W - \frac{W}{g}\cdot k_v \cdot g = W[1 - k_v]$$

Effect of Horizontal Acceleration (α_n):

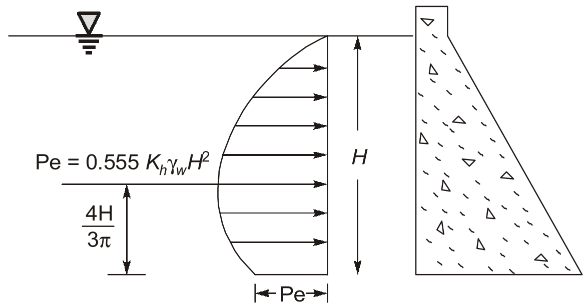
The following two forces are caused by the Horizontal acceleration :

(i) Hydrodynamic pressure

(ii) Horizontal Inertia force

(i) **Hydrodynamic Pressure :** When the horizontal acceleration is acts towards the reservoir, it causes a momentary increase in the water pressure as the foundation and dam accelerate towards the reservoir and water resists the movement owing to its inertia. The extra pressure developed is known as Hydrodynamic Pressure. According to Von-Karman the Hydrodynamic Pressure (Pe) is given by

$$\text{Pe} = 0.555 \times k_w \times \gamma_w \cdot H^2$$



(ii) **Horizontal Inertia Force :** This force is generated in order to keep the body and foundation of the dam together as one piece the direction of the produced force will be opposite to the acceleration imparted by the earthquake.

$$\text{Horizontal Inertia force} = \left(\frac{W}{g}\right)\cdot \alpha_n = \frac{W}{g}\cdot k_n \cdot g$$
$$= W \cdot k_n$$

■■■■

4 River Training and Diversion Headworks

LEVEL 1 Objective Questions

1.	(a)
2.	(a)
3.	(d)
4.	(c)
5.	(e)
6.	(a)
7.	(c)
8.	(b)
9.	(b)
10.	(a)
11.	(b)
12.	(c)
13.	(c)

LEVEL 2 Objective Questions

14.	(a)
15.	(d)
16.	(c)
17.	(a)
18.	(d)
19.	(b)
20.	(a)
21.	(b)
22.	(a)
23.	(a)
24.	(c)
25.	(b)
26.	(c)

LEVEL 3 Conventional Questions

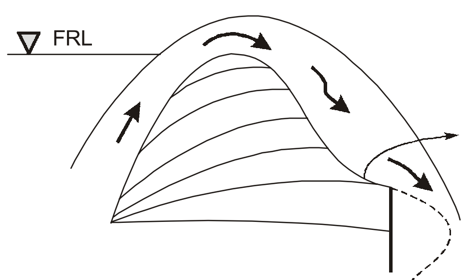
Solution : 1

Priming and depriming: Priming is the process of removal of air in the bend in order for them to function and most siphon spillways are designed with a system that makes use of water to remove the air and automatically prime the siphon.

The time period between the instant the water just starts to spill from the crest to instance that siphon runs in full is called priming when the water level of the reservoir decreases and reaches just to inlet of the depriming hood, air will be passed to the throat section and negative pressure developed will be released and siphon will stop running. If deprimers is not provided then water level will decrease just to be upper level of the inlet pipe. Which will cause the loss of water.

Devices for early priming in saddle syphon spillway: Various devices are used to intace early priming of the syphon. The most commonly used are as follows:

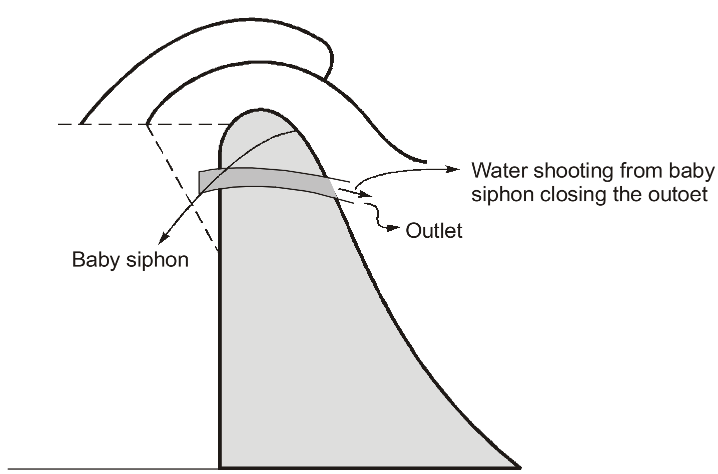
(i) **Step/deflector:** The priming is accomplished by means of a step called joggle which deflects sheet of water to strike against the lower end of cover or hood and thus sealing lower end from the atmosphere.



Siphon with step

(ii) **Baby siphon:** In addition to above, baby siphon may be installed as priming device.

When water level reaches strightly above the crest, the baby siphon which is an additional siphon, starts running full. The sheet of water issuing fram it, is arranged to shoot across the lower end of the main siphon so as to seal it from the atmosphere.



Baby siphon installed as primary device

Solution : 2

(i) 5 m mho/cm

(ii) L.F> = 12.8%

Solution : 3

Ogee spillway downstream profile, $x^n = K(H_d)^{n-1} \cdot y$

Hd – design head I/c velocity head

Here upstream head of water = 20 m (given)

Ha-velocity head (very small so can be neglected)

O-origin at highest point C of crest, $n = 1.81$

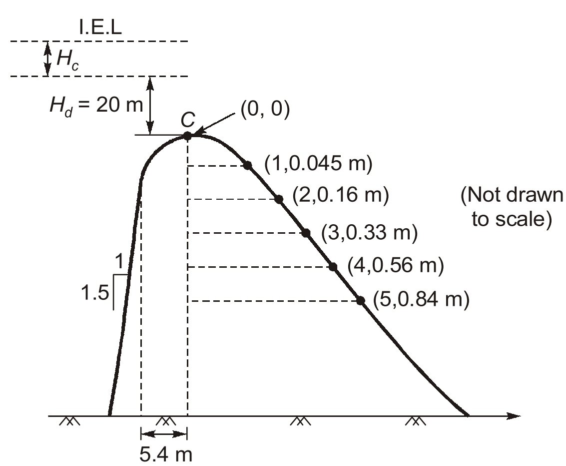
$K = 1.939$

$x^{1.81} = 1.939 \times (20)^{(1.81-1)} y = 1.939 \times 11.32\, y = \mathbf{21.95\, y}$

upstream profile can extend upto $x = -0.27\, H_d$

$x = -0.27 \times 20 = \mathbf{-5.4\ m}$

x	y	x	y
1 m	0.045 m	6 m	1.17 m
2 m	0.16 m	7 m	1.54 m
3 m	0.33 m	8 m	1.96 m
4 m	0.56 m	9 m	2.43 m
5 m	0.84 m	10 m	2.94 m



Solution : 4

Necessity of a spillway: A 'spillway' is a structure constructed at the dam site for effectively disposing off the surplus water from upstream to down stream.

- The height of the dam is always fixed according to the maximum reservoir capacity. The normal pool level indicates the maximum capacity of reservoir. The water is never stored in the reservoir above this level. The dam may fail by overturning, so, for the safety of the dam the proper designing of spillway are essential.
- The top of the dam is generally utilized by making road. The surplus water is not be allowed to flow over the top of the dam, so to stop the over tapping by the surplus water, the spillways become extremely essential.

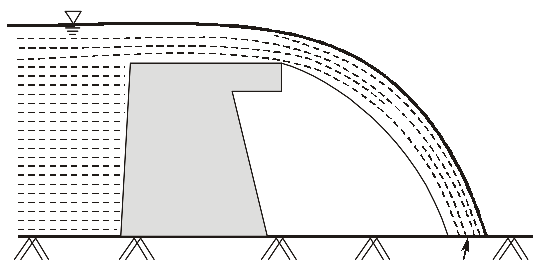
- To protect the downstream base and floor of the dam from the effect of scouring and erosion, the spillway are provided so that the excess water flows smoothly.

Types of spillway:

(1) Straight drop spillway
(2) Ogee spillway
(3) Chute spillway
(4) Side Channel spillway
(5) Shaft spillway
(6) Syphon spillway

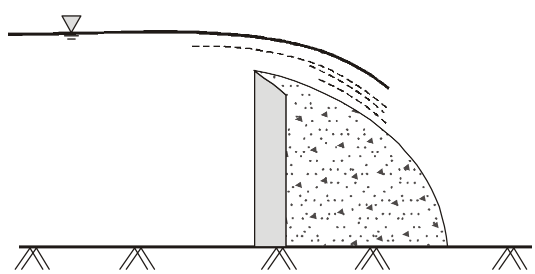
(1) Straight drop spillway:

Choice: May be constructed on small bunds or on thin Arch dams.



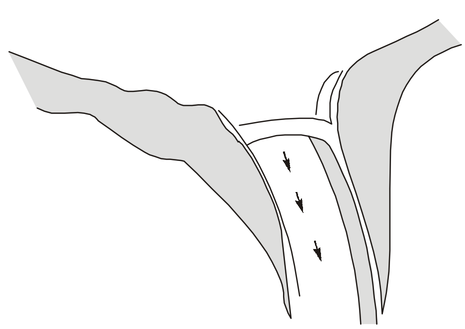
(2) Ogee spillway:

Choice: Used in masonry, arch and buttress dam.



(3) Chute Spillway: An ogee spillway is mostly suitable for concrete gravity dam especially when spillway is located within the dam body. But for earthen dam or Rock fill dam a separate spillway is generally constructed in a saddle, away from the main valley.

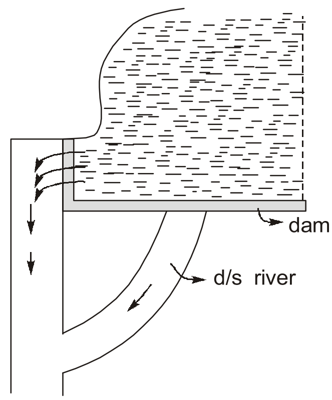
Choice: Earthern and Rockfill dam.



Earthern and Rock fill dam

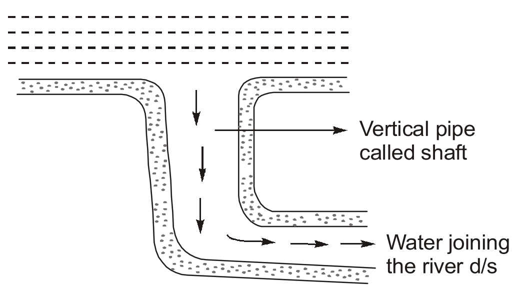
(4) Side Channel Spillway: In side channel spillway the crest is turned by 90° such that it flow parallel to weir crest.

Choice: Provided in narrow valley of Earthern and Rockfall dam.



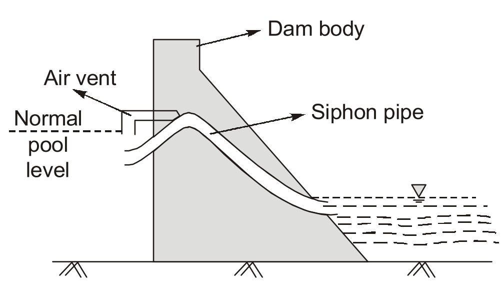
(5) **Shaft Spillway:** The water from the reservoir enters into a vertical shaft which conveys this water into a horizontal tunnel which finally discharge the water into the river downstream.

Choice: Adopted when space required for ogee and chute spillway is not available.



(6) **Syphon Spillway:** Level of air vent kept at normal pool level and entry point of siphon kept below normal pool lane so When water is below normal level, air goes into the vent and no flow takes place through as to prevent the entry of debris etc in siphon. When water level is above normal pool level then under siphonic action surplus water dispose into downstream side.

Choice: No space for separate ogee or chute spillway.



Solution : 5

2.39 m below the ground.

Solution : 6

Not Safe.

Solution : 7

$LR = 6\%$

$D_i = 85.1$ mm

Solution : 8

Advantages:

(i) No mechanical device is needed for its operation.

(ii) A siphon spillway have larger discharge in capacity fro same rise in reservoir level than normal spillway. This is because of the fast that in siphon spillway discharge occurs at much larger head.

(iii) Since it can deliver high discharge at comparatively lower head, the required height of dam and area to be acquired reduces.

Disadvantages:

(i) Cost of construction is high.

(ii) Special measures are required to avoid blockage of outlet by flood debris.

(iii) Priming of siphon results in excessive vibration which may cause expansion problems in joints.

(iv) It causes sudden appearance of flow water downstream.

(v) There is a possibility of cavitation for negative pressure.

(vi) Maintenance cost is high.

(vii) A siphon spillway has to be constructed in form of multiple units as development of crack in siphon of a single unit doesn't affect the siphonic action of the whole spillway.

(viii) Like other types of closed conduct spillways, a siphon spillway too is incapable to handling flow appreciably larger than the designed capacity.

■■■■

5 Miscellaneous

LEVEL 1 Objective Questions

1. (Superpassage)
2. (b)
3. (c)
4. (c)
5. (b)
6. (b)
7. (a)
8. (a)
9. (b)
10. (d)
11. (c)
12. (a)
13. (b)
14. (c)

LEVEL 2 Objective Questions

15. (a)
16. (d)
17. (a)
18. (a)
19. (c)
20. (d)
21. (c)
22. (c)
23. (b)
24. (a)
25. (d)
26. (c)
27. (d)

LEVEL 3 Conventional Questions

Solution : 1

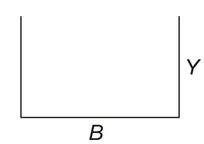
$$\text{Canal Bed level} = \text{RL } 300.000 \text{ m}$$
$$\text{Drain HFL} = \text{RL } 300.500 \text{ m}$$

Here Drain HFL is below canal bed level.

Suggested/proposed cross drainage structure = Syphon aqueduct

$$\text{Required waterway for drain } (P) = 4.75\sqrt{Q}$$
$$Q = 6 \text{ m}^3/\text{s}^{-1}$$
$$P = 11.635 \text{ m}$$
$$\text{For smaller drains } P^1 = 0.8P = 0.8 \times 11.635 = 9.31 \text{ m}$$
$$\text{Let velocity through barrels} = 2.5 \text{ m/s [generally varies from 2 to 3 ms}^{-1}]$$
$$A = \frac{6}{2.5} = 2.4 \text{ m}^2$$

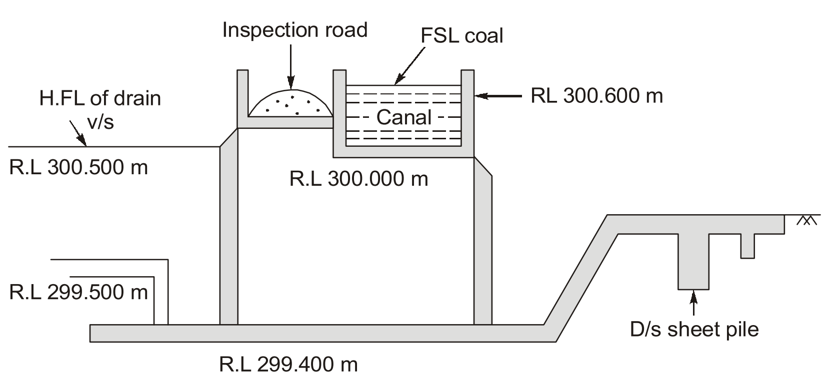
So, $B + 2y = 9.31$ ...(1)

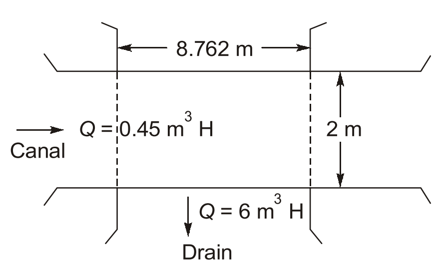


and $By = 2.4$

$\therefore$ $(9.31 - 2y)y = 2.4$

$$y = 4.381 \text{ m}$$
$$y = 0.274 \text{ m} \quad \text{(Acceptable)}$$
$$B = 8.762 \text{ m}$$





Solution : 2

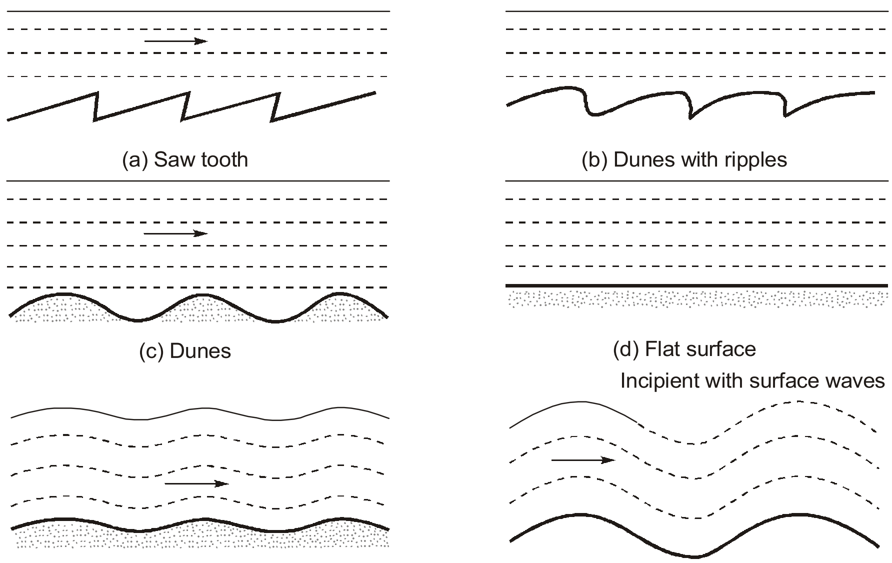
(i) Bed forms in alluvial channels:

(a) Saw tooth ripples - when velocity is gradually increased, a stage is reached, when sediment load comes just at the point of motion. This stage of is known as ripples.

(b) **Dunes** - such ripples can also be seen in sand on any beach. As the velocity in increased further, larger periodic irregularities appear and are called dunes.

(c) **Flat surface** - when velocity is increased beyond the formation of dunes, the dunes are erased by the flow, leaving very small undulations or virtually a flat surface with sediment particles in motion.

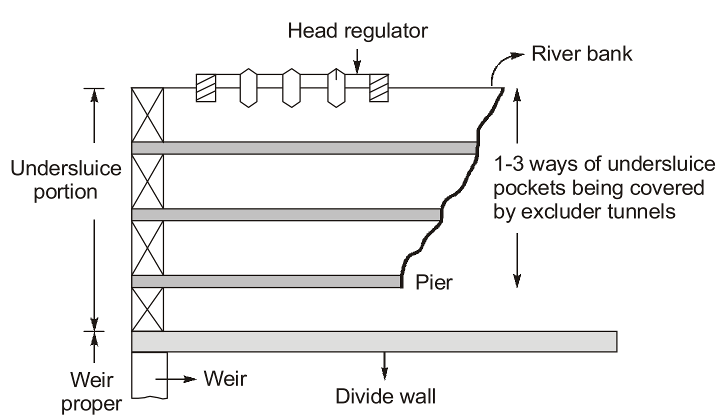
(d) **Antidunes** - further increase in velocity results in formation of sand waves in association with surface waves. As the velocity is further increased so as to make froude number exceeding unity, the flow becomes supercritical.



(a) Saw tooth
(b) Dunes with ripples
(c) Dunes
(d) Flat surface
(e) Sand waves
(f) Antidunes

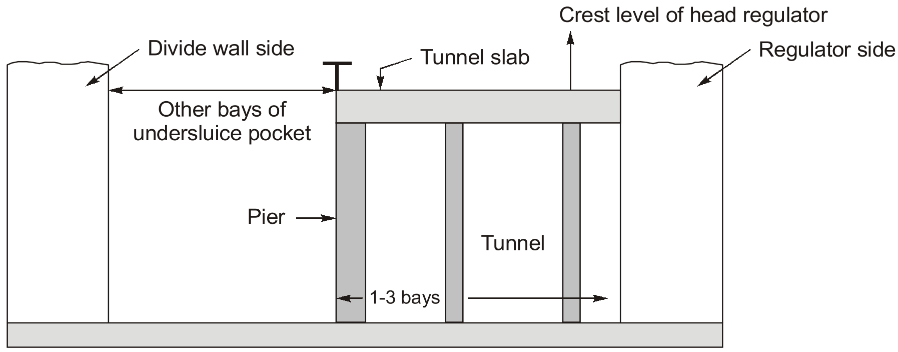
(ii) **Silt excluder:** Silt excluder are those works which are constructed on the bed of the river upstream of the head regulator. The clearer water enters the head regulator and the silted water enters the silt excluder. In this type of works, the silt is therefore removed from the water before it enter the canals.

Working and design basis of silt excluder: A silt excluder consists of a number of rectangular tunnels running parallel to axis of the head regulator and terminating near the under-sluiced weir. The tunnel nearest to the crest of the head regulator has to atleast of the same length as the head regulator. The roof slab of the excluder tunnels is kept at the same level as that of regulator crest. The bottom layer of water which is highly charged with silt and sediment will pass down the tunnels and escape over the floors of the under sluice ways since the gates of the under sluices shall be kept upon top of the tunnels.



(Plan of silt excluder)

Usually two or three bays of the undersluices of the weir or barrage are covered by the excluder. However, excluder covering only one bay has been designed and is usually adopted for sandy rivers.



Section of silt excluder

(iii) **Divided wall and fish ladder:** A divide wall is along masonary or concrete wall protected on all sides by stone or concrete blocks, constructed at right angles to the axis of the weir to separate the weir proper section and under sluice section.

Function of divide wall:

(a) It separates the floor of scouring sluices from that of weir proper which is at higher level.

(b) It provides silt content in front of the canal head regulator so that silt gets deposited in it.

(c) It isolated the pocket upstream of the head regulator to facilitate scouring operation.

(d) It prevents formation of cross-currents and flow parallel to weir axis.

(e) It serves as one side of fish ladder.

Fish ladder: A fish ladder is a passage provided adjacent to the divide wall on the weir side for the fish to travel from u/s to d/s of the weir and also in the reverse direction.

- In has been established that most types of fish can travel u/s against a flow velocity of about 3 to 3.5 m.sec. If no fish ladder is provided at the weir site and only a gap is left, even the strongest fish will not above to travel upstream.
- In a fish ladder, the head is gradually dissipated so as to provide smooth flow at sufficient low velocity. Suitable baffers are provided in fish passage to reduce the flow velocity.

Solution : 3

Discharge intensity, q = 24.6 cumec/m

Depth of water above spillway crest = 100 – 95 = 5 m

$\therefore$ Velocity of flow just leaving the crest,

$$V = \frac{q}{1\times 5} = \frac{24.6}{1\times 5} = 4.92 \text{ m/s}$$

$$\text{Velocity head above spillway crest} = \frac{(4.92)^2}{2\times 9.81} = 1.23 \text{ m}$$

$$\therefore \quad T.E.L. = HFL + \frac{V^2}{2g} = 100 + 1.23 = 101.23 \text{ m}$$

$$\text{Velocity of flow before jump, } V_1 = C_V\sqrt{2gH} = 0.9\sqrt{2\times 9.81\times 100} = 39.86 \text{ m/s}$$

$$\text{Pre-jump depth, } y_1 = \frac{q}{V_1} = \frac{24.6}{39.86} = 0.62 \text{ m}$$

The post jump depth, (y_2) can be give as

$$y_2 = -\frac{y_1}{2} + \sqrt{\frac{y_1^2}{4} + \frac{2q^2}{gy_1}} = -\frac{0.62}{2} + \sqrt{\frac{(0.62)^2}{4} + \frac{2\times(24.6)^2}{9.81\times 0.62}}$$

$$= 13.79 \text{ m}$$

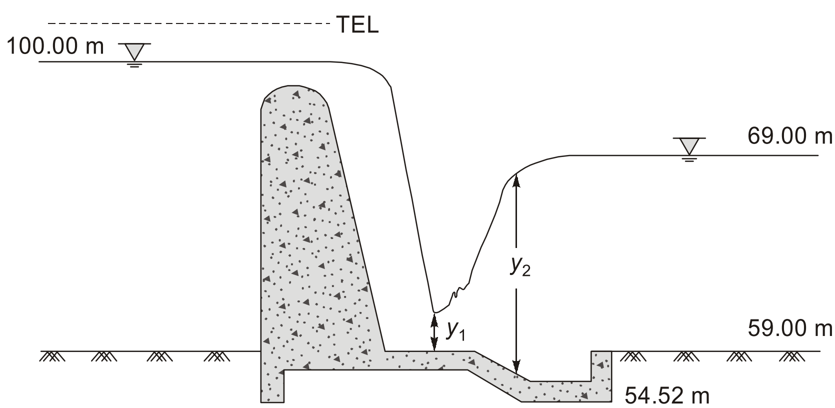
Specific energy before starting of Hydraulic jump,

$$E = y_1 + \frac{V_1^2}{2g} = 0.62 + \frac{(39.86)^2}{2\times 9.81} = 81.59 \text{ m}$$

Fraude No. $$F_1 = \frac{V}{\sqrt{gy_1}} = \frac{39.86}{\sqrt{9.81\times 0.62}} = 16.16 > 4.5$$

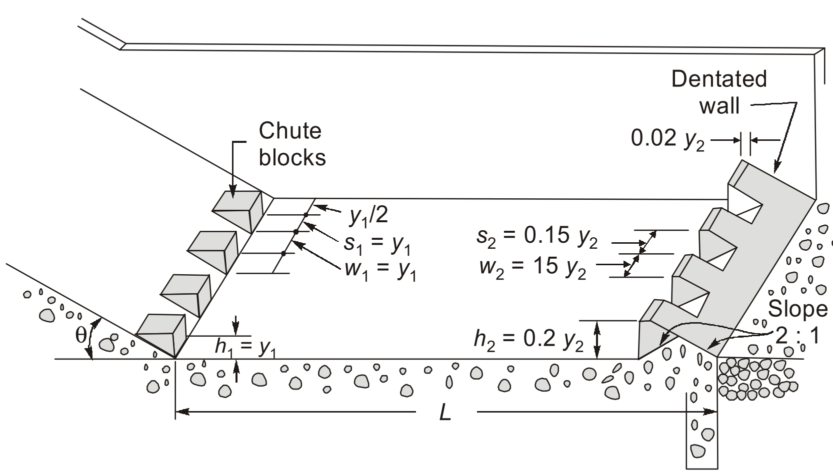
∴ Adopt U.S.B.R. Stilling basin - II

$$\text{Elevation of basin floor at tail water} = 69 - \frac{105\times y_2}{100} = 69 - 1.05 \times 13.79 = 54.52 \text{ m}$$



$$\text{Length of basin, } L_b = 4.30 \times y_2 = 4.30 \times 13.79 = 59.30 \text{ m}$$

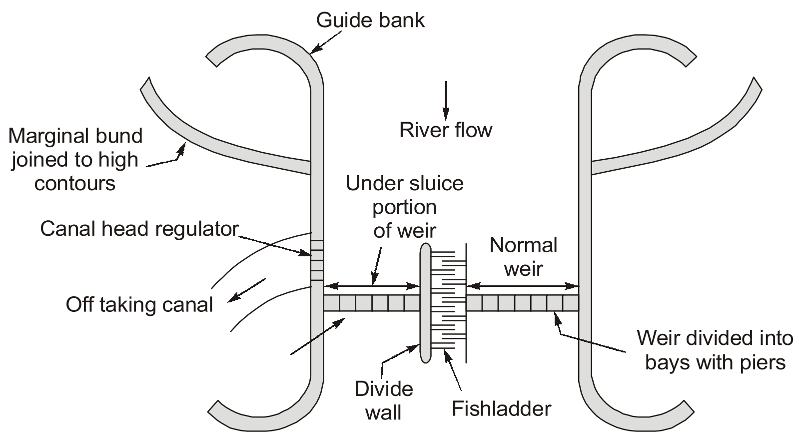
Detailed of chute block and dentated wall are shown in fig.



U.S.B.R. Stilling basin II (F_1 > 4.5).

Solution : 4

The typical layout of a diversion headwork is given below:



Diversion head works consists of:

1. **Weir proper**
2. **Under sluice or scouring sluices:** A comparatively less turbulent pocket of water is created near canal head regulator by constructing under sluices portion of the weir. The divide wall separates the main weir portion from the under sluice portion of the weir. The crest of the under sluice portion of the weir is kept at a lower level than the crest of normal portion of the weir. Under sluices help in bypassing the excess supplies to the downstream side of the river. These also help in scouring and removing the deposited silt from the under sluiced pocket and hence are also called scouring sluices.
3. **Divide wall:** The divide wall is a masonry or concrete wall constructed at right angles to the axis of the weir and separates the weir proper from the under sluices. It helps in providing a comparatively less turbulent pocket near the canal head regulator, resulting in deposition of silt in this pocket and thus to help in the entry of silt free water into the canal. Divide wall may keep the cross currents, if at all they are formed away from the weir.
4. **River training works such as marginal bunds, guide banks, groynes etc:** River training works are required near the weir site in order to ensure a smooth and an axial flow of water, and thus, to prevent the river from out flanking the works due to a change in its course. The guide banks force the river into a restricted channel and thus ensuring a smooth and an almost axial flow near the weir site. Marginal bunds are provided on the upstream side of the works in order to protect the area from submergence due to rise in HFL caused by afflux. These bunds are therefore, continued till they join contours higher than the new HFL.
5. **Fish ladder:** Large rivers are generally inhabited by several types of fish, many of which are migratory. Such migratory type of fishes are called anadromous fish which move from one part of the river to another part, according to season. Thus, a structure which enables the fishes to pass upstream is called a fish ladder. It is a device by which the flow energy can be dissipated in such a manner as to provide smooth flow at sufficiently low velocity.
6. **Canal head regulator:** A canal head regulator serves the following functions:
   (i) It regulates the supply of water entering the canal.
   (ii) It controls the entry of silt in the canal.
   (iii) It prevents the river floods from entering the canal.

7. **Silt regulation works:** The entry of silt into a canal which takes off from a headworks can be reduced by constructing certain special works, called silt control works. These works can be of two types:
   (i) **Silt Excluder :** These are constructed on the bed of river upstream of head regulator. The clear water enters the head regulator and the silted water enters the silt excluder. In this type of work, the silt is therefore removed from water before it enters canal.
   (ii) **Silt ejector :** These devices extract the silt from the canal water after the silted water has travelled a certain distance in the off-take canal. These works are therefore constructed on the bed of the canal, and a little distance downstream from the head regulator.

Solution : 5

1. Waterway for Barrage

High flood discharge of 3500 m^3/sec has to pass through the gated openings provided at the barrage site, with the following data:

Maximum water level at high flood u/s of barrage = 212.00 m

Crest level of Barrage = 207.00 m

∴ Opening available above the crest of barrage = 212.00 – 207.00 = 5.0 m

Now, we know that

$$Q = C L H^{3/2}$$

where C = coefficient of discharge through barrage = 2.10

L = effective waterway of barrage

H = 5.00 m

Q = 3500 m^3/sec

$$\therefore \quad 3500 = 2.10 \times L \times (5)^{3/2}$$

$$\Rightarrow \quad L = \frac{3500}{2.10 \times (5)^{3/2}} = 149.1 \text{ m say } 150 \text{ m}$$

Effective waterway = Total waterway (ignoring end contractions)

Hence providing 15 bays, each of 10 m clear span.

∴ Number of gates required for barrage = 15

2. Waterway for canal head regulator

Discharge through main canal = 500 m^3/s

Coefficient of discharge for head regulator = 1.50

Head over the crest of regulator at pond level = 211 – 208 = 3 m

We know that $Q_1 = C_1 L_1 H_1^{3/2}$

$$\Rightarrow \quad 500 = 1.5 \times L_1 \times (3)^{3/2}$$

$$\Rightarrow \quad L_1 = \frac{500}{1.5 \times (3)^{3/2}} = 64.15 \text{ m say } 64 \text{ m}$$

Hence, providing 7 bays each of 10 m clear span, thereby providing 70 m waterway. So, number of gates required for canal head regulator each of 10 m span = 7.

3. Design of stilling basin

U/S HFL = 212.00

D/S HFL = 210.00

U/S TEL = 212.00 (Ignoring velocity of approach)

$$\text{D/S TEL} = 210.00 \text{ (Ignoring velocity of approach)}$$
$$H_L = 212 - 210 = 2 \text{ m}$$

Discharge intensity through barrage bays $= \dfrac{3500}{150} = 23.33 \text{ m}^3\text{/sec/m}$

$$\text{Critical depth, } y_c = \left(\frac{q^2}{g}\right)^{1/3} = \left[\frac{(23.33)^2}{9.81}\right]^{1/3} = 3.81 \text{ m}$$

Now let $Y = y_2/y_c$

and $z = \dfrac{H_L}{y_c}$

$\therefore$ $z = \dfrac{2}{3.81} = 0.5249$

Computing value of Y using

$$Y = 1 + 0.93556\, z^{0.368} \text{ for } z < 1$$

$\Rightarrow$ $Y = 1 + 0.93556\,(0.5249)^{0.368}$

$\Rightarrow$ $Y = 1.738$

Now, $Y = \dfrac{y_2}{y_c}$

$\Rightarrow$ $y_2 = Y \times y_c = 1.738 \times 3.81 = 6.62 \text{ m}$

$\therefore$ $5y_2 = \text{length of cistern} = 5 \times 6.62 = 33.10 \text{ m}$

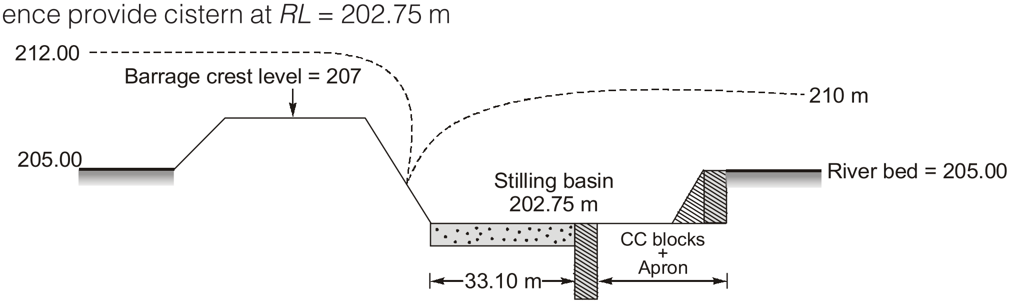
Now E_{f_2} can be calculated as

$$\frac{E_{f_2}}{y_c} = Y + \frac{1}{2Y^2}$$

$$E_{f_2} = 3.81 \times \left[1.738 + \frac{1}{2 \times (1.738)^2}\right] = 7.25 \text{ m}$$

Hence level at which jump will form = D/S TEL – E_{f_2} = 210 – 7.25 = 202.75 m

Hence provide cistern at RL = 202.75 m



Solution : 6

When a cut-off is induced artificially to avoid danger to flood in valuable land or property, this is called Artificial cut-off.

For developing an artificial cut-off, only a pilot channel is required to be excavated because for easily erodiable beds the flood water will gradually enlarge the pilot cut to the required cross-section and will abandon the old curved channel.

The pilot cut should be made as deep as possible. As tractive force is directly proportional to depth a deeper cut would be helpful for rapid development. Moreover, the alignment of the cut should be tangential to the main direction of flow approaching and leaving the cut.

Solution : 7

0.17 m

Solution : 8

Principles of Silt control :

Silt has a tendency to settle down at bed but they are kept in suspension by the force of verticle eddies. These eddies are developed due to friction of flowing water against the bed. Hence, if the friction of bed can be reduced more settlement of silt and its consequent settlement is possible.

Silt Excluders:

The are those works which are constructed on the bed of the river, upstream of the head regulator. The clearer water enter the headwork and further to canal but the silted water enters the silt excluders.

Silt Ejectors (Extractors)

These are those devices which are installed on the bed of canal and a little distance downstream of head regulator.

As we know, that less disturbance and smooth channel bed are two basic principles of silt control. Both of these two can be achieved more effectively in the canal as compared to river bed. Hence silt extractors is definitely better than silt excluder.

■■■■