FILTRATION UNIT

III. Design a rapid sand filter unit for 4 MLD of supply with all principal components.

Solution:

- water required per day = 4 ML/day
- Assuming that 4% of water is required for washing of the filters,

$= 4ML \times 0.04 = 0.16$

Therefore $4 + 0.16 = 4.16$ ML/day

- Assuming 0.5 hours is lost in washing filters everyday.

then $\frac{4.16}{23.5}$ ML/~~day~~ hour = 0.177 ML/Hour.

- Assuming the rate of filtration to be 5000 lt/hr/m², we have

the area required $= \frac{0.177 \times 10^6}{5000} = 35.46 m^2$

- Assuming Length = 1.5 times the width of the filter bed (B).

we have, $2A = 35.46 m^2$ {Since two beds are required}

$$2 \times (L \times B) = 35.46 m^2$$

$$2 \times (1.5B \times B) = 35.46 m^2$$

$$3B^2 = 35.46 m^2$$

$$\boxed{B = 3.44 m}$$

$$L = 1.5B \Rightarrow 1.5 \times 3.44 = 5.16 m$$

say $\boxed{L = 5.2 m}$

using length as 5.2 m,

$$B = \frac{35.46}{2 \times 5.2} = 3.4 m.$$

Therefore, adopt 2 filter units each of dimension.

5.2m X 3.4m

PUMPS & DIA OF RISING MAIN

Population = 1,00,000 ; Rate of Supply = 150 lpcd ;
Source of water = 2000 m away ;
The difference in elevation between the lowest water level in the sump & the reservoir is 36 m. If the demand has to be supplied in 8 hours, determine the size of the main & the BHP of the pump required. Assume max. daily demand as 1.5 times the average daily demand. Assume $f = 0.0075$; velocity in pipe = 2.4 m/s; efficiency of the pump = 80%.

Solution:

- avg daily demand = 1,00,000 × 150 lpcd
  = 15 MLD.
- max. daily demand
  = 1.5 times avg. daily demand
  = 1.5 × 15 = 22.5 MLD.
- Pump has to work for 8 hrs/day
  $22.5 \times \frac{24}{8} = 67.5$ MLD
- Discharge required / Second.
  $= \frac{67.5 \times 10^6}{10^3 \times 24 \times 60 \times 60} = 0.781\ m^3/sec$
- Area required $= \frac{Q}{V} = \frac{0.781}{2.4\ (\text{given})}$
  $= 0.325 m^2$
- Diameter of the main required
  $A = \frac{\pi d^2}{4}$
  $d = \sqrt{\frac{A \times 4}{\pi}} = \sqrt{\frac{0.325 \times 4}{\pi}}$
  $d = 0.643 m$
  $d \simeq 0.65 m$ {dia of Rising}

To calculate the BHP of Pump:

- Total lift including suction & delivery = 36 m (given)
- Head loss due to friction in pipe H_f or H_L
  $$H_f = \frac{4fLV^2}{2gd}$$
  $$= \frac{4 \times 0.0075 \times 2000 \times 2.4^2}{2 \times 9.81 \times 0.65}$$
  $H_f = 27.1 m$
- Total lift against which pump works
  = 36 + 27.1 = 63.1 m.
- $$BHP = \frac{\gamma_w \cdot QH}{\eta \times 0.735}$$
  $$= \frac{9.81 \times 0.781 \times 63.1}{0.8 \times 0.735}$$
  = 822 HP

SEDIMENTATION TANK :-

Data:

i) Volume of water to be treated = 3000000 liters per day
ii) Detention period = 4 hours
iii) Velocity of flow = 10 cm/min.

Solution:

- Detention time = 4 hours = 240 mins.
- velocity of flow = 10 cm/min
- Therefore Length of the tank = $0.10 \times 240 = 24\,m$
- Volume of water in 4 hours = $\frac{3 \times 10^6}{10^3} \times \frac{4}{24} = 500\,m^3$
- Cross-Sectional Area $A = \frac{V}{L} = \frac{500}{24} = 20.8\,m^2$
- Assume a working depth of 3m
- width of the tank = $\frac{20.8}{3} \simeq 7m$
- Provide an extra depth of 1m for Sludge Storage & 0.5 m for free board. ie $3 + 1.5 = 4.5m$.

Hence, provide a settling tank of size

$24m \times 7m \times 4.5m$.

I) POPULATION FORECASTING-

Population forecasting is done by the methods of arithmetic ~~pro~~ Increase method since the village has reached its saturation population as there is no development activities undergone.

Year	Population.	Increment Per decade
1988	250	
		75
1998	325	
		75
2008	400	
		100
2018	500	

$I(avg) = 83.33 \simeq 84$

$P_n = P + nI$

P = Population in 2018 = 500

n = number of decades.

I = average increase per decade = 84

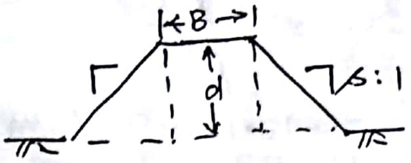
~~for~~ Population forecast for the year: 2028.

$P_n = 500 + (1)(84) = 584$

Population forecast for the Year 2038

$P_n = 500 + 2(84) = 668$

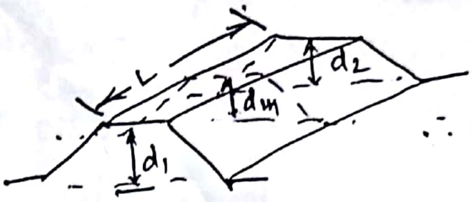
Quantity, Q = Area at mid-section × Length.



C/S area = (Area of Rectangular Portion) + 2 (Area of side Δ^{lar} portion)

$= Bd + 2 \times \frac{1}{2} \times sd \cdot d$

$= Bd + sd^2$



∴ Area at mid-section = $(Bd_m + sd_m^2) L$.

Problem:

1. Estimate the Quantity of Earthwork for a portion of the road for a length of 400m. Formation width of the road is 10m. Side slopes is 2:1 in banking and 1.5:1 in cutting.

Station:	25	26	27	28	29	30	31	32	33	34	35
Distance:	1000	1040	1080	1120	1160	1200	1240	1280	1320	1360	1400
RL of GL:	51.00	50.90	50.50	50.80	50.60	50.70	51.20	51.40	51.30	51.00	50.60

RL of Formation level: 52.00 ←—— Downward gradient of 1 in 2000 ——→

Soln: Given Downward gradient 1 in 200

∴ For every 200m — 1m fall

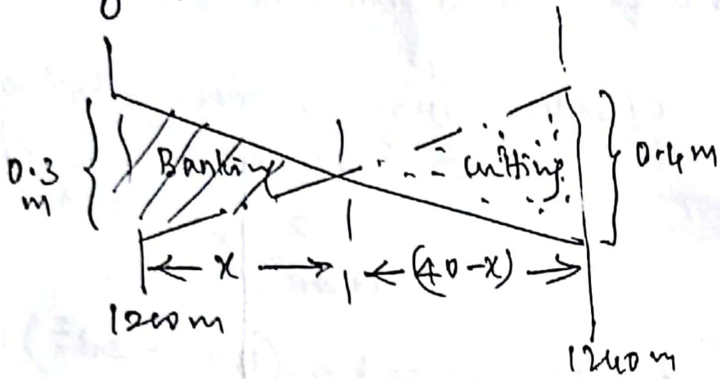
∴ For every 40m : $\frac{1}{200} \times 40 = 0.2m$.

Deduct 0.2m from previous RL to get RL of formation level.

B = 10m; s = 2 in banking s = 1.5 in cutting*

Station	Distance	RL of GL	RL of FL	Ht. or depth (FL~GL)	Mean depth (d_m) (m)	Central Area Bd_m m^2	Area of sides sd_m^2 m^2	Distance (L) m	Quantity $Q = (Bd_m + sd_m^2)L$	
									Banking	Cutting

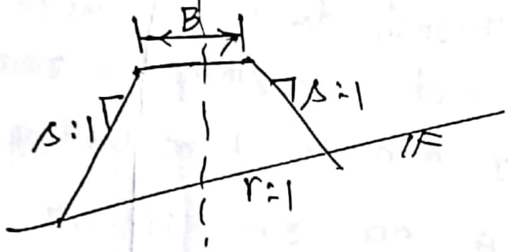
As seen from the table road passes from banking to Cutting b/w stations 30 (1200m) and stations 31 (1240m)



$$\frac{x}{0.3} = \frac{40-x}{0.4}$$

$\therefore x = 17.14m$ say 17m.

Earth work in hill roads will have cross slopes



<u>Use Mean-Sectional area method</u>

(i) Find A_1 & A_2 at both ends using:

$$A = \frac{sb^2 + r^2(2bd + sd^2)}{r^2 - s^2}$$

(ii) $A_m = \frac{A_1 + A_2}{2}$

(iii) $Q = A_m \times L$.

At zero point, one half of the road will be in cutting and one-half will be in banking.

Calculate Area of banking / cutting $= \left(\frac{1}{2} \times \frac{b^2}{r-s}\right)$

where r = mean harmonic slope $= \frac{2r_1r_2}{r_1+r_2}$.

...timate the volume of Earth work for a highway using following data:

The formation level at chainage 0m is 908 and has a raising gradient of 1 in 250 upto 1st 400m beyond which a falling gradient of 1 in 250 is provided. The formation width is 9m. Side slope is 1:1 for cutting and 2:1 is banking.

Chainage :	0	100	200	300	400	500	600	700
RL :	900	904.60	902.30	907.00	910	911	910.50	911.50
Cross slope :	level	level	level	level	level	level	1 in 8 (1 in r)	1 in 10 (1 in r)

Soln: Raising gradient upto 400m is 1 in 250

∴ For every 100m, $\frac{1}{250} \times 100 = 0.4$ m rise.

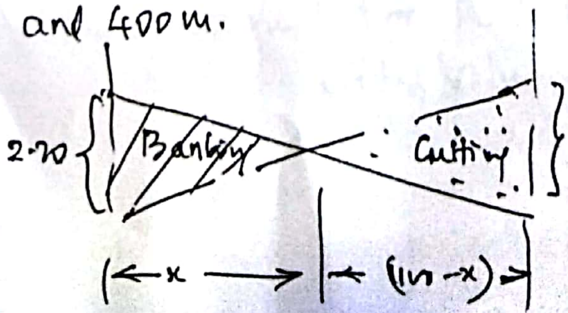
falling gradient of 1 in 250 beyond 400m

∴ For every 100m, $\frac{1}{250} \times 100 = 0.4$m fall.

Given, B = 9m; s = 1 for cutting, s = 2 for banking, $b = B/2 = 4.5$m $r_{600} = 8$, $r_{700} = 10$

Chainage	RL of GL	RL of FL	FL ~ GL = depth (m)	Total sectional area (m²)	Mean Area (A_m) (m²)	Distance (L) (m)	Quantity $Q = A_m \times L$ Banking	Quantity $Q = A_m \times L$ Cutting
0	900	908.00	8.00	200.00	—	—	—	
100	904.60	908.40	3.800	63.08	131.540	100	13154	
200	902.30	908.80	6.500	143.00	103.040	100	10304	
300	907.00	909.200	2.200	29.48	86.240	100	8624	
384.62	—	—	0.00	0.00	14.740	84.62	1247.30	
400	910.00	909.600	(−) 0.400	(−) 3.76	(−) 1.88	15.38		28.91
500	911	909.200	(−) 1.80	(−) 19.04	(−) 11.60	100.00		1160.00
600	910.50	908.800	(−) 1.70	(−) 18.80	(−) 19.12	100.00		1912.00
700	911.50	908.400	(−) 3.10	(−) 38.09	(−) 28.45	100.00		2845.00
							33329.30	5945.91

* The road passes from banking to cutting b/w chainage 300 and 400m.



$$\frac{x}{2.20} = \frac{100-x}{0.40}$$

$\therefore x = 84.62$m.

Use For level cross slopes:

$A = Bd + sd^2$

For GL having cross slopes:

Use $A = sb^2 + r^2(2bd + sd^2)$

Canal Design

An irrigation canal has a cultivable command Area of 80% of total command area of 1,20,000 hectares. The intensity of irrigation is 35% for kharif season and 45% Rabi season. The duty at the field is 800 hect/cumec for kharif crop and 1400 hect/cumec for Rabi crop. If the canal losses are 15%, compute the capacity of canal.

Soln: Given GCA = 1,20,000 hectares

∴ CCA = 80% of GCA = 0.8×1,20,000
= 96000 hectares

∴ Area to be Irrigated in kharif season
= CCA × I.I
= 96000 × 35/100 = 33,600 hectares

Area to be Irrigated in Rabi season
= CCA × I.I
= $96000 \times \frac{45}{100}$ = 43,200 hectares.

∴ Water required for kharif crops
= $\frac{33600}{\text{Duty}} = \frac{33600}{800}$ = 42 cumecs (or m^3/sec)

∴ Water required for Rabi crops
= $\frac{43,200}{1400}$ = 31 cumecs.

∴ Capacity of Canal

Crop type	Duty (Hect/cumec)	A.I (hectares)	Capacity reqd (cumecs)	Total Capacity (cumecs)
Kharif	800	33600	42	73 cumecs.
Rabi	1400	43200	31	

To account for 15% losses,
Total volume reqd. in the Canal = 73×1.15 = 86 cumecs.

A SH passing through a plain terrain has a horizontal curve of radius equal to the ruling minimum radius.
Design all the geometric features of this horizontal curve, assuming suitable data

Solution: The various geometric elements of the horizontal curve to be designed are:

Ruling minimum radius, R_{ruling}

Superelevation, e

Extra widening of pavement, W_e

Length of transition curve, L_s

Ruling minimum radius of curve:

Ruling design speed of SH on plain terrain, $V = 80$ kmph

$$R_{ruling} = \frac{V^2}{127(e+f)} = \frac{80^2}{127(0.07+0.15)} = 229m \simeq 230m$$

$R_{ruling} = 230m$

Design of superelevation, e (for mixed traffic):

As the value of 0.124 is higher than the maximum SE of 0.07, limit the value of e to 0.07.

Check for transverse skid resistance developed:

$$f = \frac{V^2}{127R} - e = \frac{80^2}{127 \times 230} - 0.07 = 0.149$$

As the value of lateral friction co-efficient, f developed is less than 0.15, the SE design of 0.07 is safe.

c) Extra widening of pavement:

Assume two lane pavement, i.e. W = 7.0m, number of lanes, n = 2 & wheel base, l = 6m

Extra widening of pavement, $W_e = \frac{nl^2}{2R} + \frac{V}{9.5\sqrt{R}}$

$$= \frac{2 \times 6^2}{2 \times 230} + \frac{80}{9.5\sqrt{230}}$$

$$= 0.157 + 0.555 = 0.712 m$$

Provide an extra width of 0.71m & a total width of pavement, $B = W + W_e$

$$= 7 + 0.71 = 7.71 m$$

d) Length of transition curve, L_s:

Transition curve length, L_s is to be designed considering i) Rate of change of centrifugal acceleration

ii) Rate of introduction of the amount of SE, E &

iii) Minimum length formula, the highest of three values is adopted @ the design length, L_s

i) Design of L_s based on rate of change of CF, C

$$C = \frac{80}{75+V} = \frac{80}{75+80} = 0.52$$

As this value of C is within the range 0.5 - 0.8, the value is acceptable for design

$$L_s = \frac{0.0215V^3}{CR} = \frac{0.0215 \times 80^3}{0.52 \times 230} = 92m$$

ii) Total amount of SE, E i.e. raising of the outer edge of the pavement w.r.t inner edge = B x e

$$= 7.71 \times 0.07 = 0.54m$$

As the terrain is plain, assume the pavement to be rotated about the centre @ a rate of 1 in 150,

$$L_s = \frac{E}{2} \times N = \frac{0.54 \times 150}{2} = 40.5m$$

iii) Minimum value of L_s, as per IRC is given by:

$$L_s = \frac{2.7V^2}{R} = \frac{2.7 \times 80^2}{230} = 75.1m$$

Adopting the highest of three, $L_s = 92m$

* Length of Vertical curve < Summit curve / Valley curve

1) A vertical summit curve is formed @ the intersection of two gradients, +3.0 & −5.0%. Design the length of summit curve to provide a SSD for a design speed of 80 kmph. Assume other data.

Soln: Given, design speed V = 80 kmph, gradients $n_1 = +3.0\%$ & $n_2 = -5.0\%$.

a) Determination of SSD

As there is ascending gradient on one side of the summit & descending gradient on the other side, the effects of gradients on the SSD is assumed to get compensated & hence ignored.

$$SSD = 0.278\,Vt + \frac{V^2}{254f}$$

Assuming t = 2.5s, f = 0.35 for V = 80 kmph

$$SSD = 0.278 \times 80 \times 2.5 + \frac{80^2}{254 \times 0.35} = 128\text{m}$$

b) Det of length of summit curve

Deviation angle $N = 0.03 - (-0.05) = 0.08$

Assuming L > SSD, $L = \frac{NS^2}{4.4} = \frac{0.08 \times 128^2}{4.4} = 298\text{m}$

The value of summit curve length 'L' is greater than SSD of 128 m as per assumption & ∴ the calculated length may be accepted.

The formula for OSD: $L = \frac{NS^2}{9.6}$ (L > OSD)

$$L = 2S - \frac{9.6}{N} \quad (L < OSD)$$

2) A valley curve is formed by a descending grad 1in 25 meeting an ascending grade of 1in 30. Design the length of valley curve to fulfill both comfort condition & HSD requirements for a design speed of 80 kmph. Assume C = 0.6 m/s³.

<u>Soln</u>: Given, V = 80 kmph, $n_1 = -1/25$ & $n_2 = +1/30$

Dev angle, $N = \frac{-1}{25} - \frac{1}{30} = \frac{-11}{150}$

V = 80 kmph, $v = 80/3.6 = 22.2 m/s$

b) Valley curve length, L for comfort condition

$$L = 2\left[\frac{Nv^3}{C}\right]^{1/2} = 2\left[\frac{11}{150} \times \frac{22.2^3}{0.6}\right]^{1/2} = 73.1 m$$

c) Valley curve length for HSD:

t = 2.5 s, f = 0.35, SSD = HSD = 127.3 m

Assuming L > SSD = 127.3 m

$$L = \frac{NS^2}{(1.5 + 0.035S)}$$

$$= \frac{11 \times 127.3^3}{150(1.5 + 0.035 \times 127.3)} = 199.5m$$

As the value of 'L' is higher than the SSD of 127.3m; the assumption is correct.

If L < SSD, $L = 2S - \frac{(1.5 + 0.035S)}{N}$

SHEET - 1 SURPLUS WEIR WITH STEPPED APRON

Introduction

A Weir is a structure constructed across a river or a natural stream to raise its water level.

In a minor irrigation (a tank) the waste weir or surplus weir is constructed to dispose off the flood waters entering the reservoir from the catchment on the upstream side.

Design data required

1. Catchment details : (i.e., the area of the catchment, both the combined catchment area M, and the intercepted catchment area, m) along with the catchment constant C. These data facilitate in calculating the discharge Q, expected to flow over the weir.
2. Details of the tank : (i.e., full tank level FTL, and maximum water level MWL. The difference of these two values gives the head over the weir). These data facilitate in calculating the length of the weir, L.
3. Foundation and downstream details : These data help in designing the cross section of the weir and other necessary protection works, particularly on the downstream side of the structure.

Design procedure

The main steps in the design of the waste weir are :

a. Discharge calculation

b. Fixing the dimensions of the weir

c. Apron design

d. Design of protection works.

a. Discharge (Q)

The discharge Q, expected to flow over the surplus weir is calculated by an emperical equation of the form :

$$Q = [CM^n - cm^n] \quad \text{--- (1)}$$

Where,
C = Combined catchment constant
c = Intercepted catchment constant
M = Combined catchment area in km^2
m = Intercepted catchment area in km^2

The above formula is valid when the tanks are in **Series**. For an isolated tank

$$Q = CM^{n} \qquad \text{--- (2)}$$

Generally, Ryve's or Dicken's formula is used for calculating the discharge.

RYVE'S formula will be $Q = \left[CM^{2/3} - cm^{2/3}\right]$ --- (3)

DICKEN'S formula will be $Q = \left[CM^{3/4} - cm^{3/4}\right]$ --- (4)

Between the two, **Ryve's** formula is more popular.

The combined catchment constant C varies from 6.8 to 15 and is known as RYVE'S constant.

The Smaller value C indicates that the catchment is rough or rugged

The value of the intercepted constant c generally varies from $\frac{1}{6}$ to $\frac{1}{3}$ of C.

b. Weir

It is generally designed as a broad coasted weir, capable of discharging Q, when working under a head h = (MWL – FTL) Various parameters for the weir are :

(i) Length (L)

The length (L) of the surplus weir is calculated from the weir equation

$$Q = \frac{2}{3} c_d \sqrt{2g}\, L\, h^{3/2} \qquad \text{--- (5)}$$

c_d = Coefficient of discharge, whose values is generally tank as 0.6.

g = Acceleration due to gravity (= 9.81 m/s²)

The length (L) calculated above is the clear length of the weir as measured from one abutment to the other. Sometimes it may be necessary to store the surpluss water due to the fact that at higher levels (i.e., between FTL and MWL). The volume of water available is considerably large, as the contours at these levels extend to very large areal extent. Hence, temporary arrangement in the form of wooden planks between dam stones that are fixed on top of the weir should serve this purpose.

From the above discussions we see that by providing number of dam stones the clear length (L) of the weir reduces. Therefore, the effective length of weir (L') should be equal to

L = {Clear length L + number of dam stones × width of each stone}

(ii) Top width (a)

Top width of the weir is calculated from the equation

$$a = 0.55\left(\sqrt{H} + \sqrt{h}\right) \qquad \text{--- (6)}$$

Where,

h = head over the weir = (MWL – FTL)

H = Height of weir = (FTL - Top of foundation)

(iii) Bottom width (b)

Bottom width of the weir is calculated from the stability criterion, i.e., Restoring moment ≡ maximum overturning moment.

The above condition satisfies the middle third rule, i.e., the resultant of all the forces acting on the weir is well within the middle third of its base.

The above condition in a mathematical form would be,

$$\frac{1}{12}\left[\{(S+1.5)H+2\cdot 5h\}\,b^2+a(SH-H-h)b-\frac{1}{2}a^2(H+3h)\right]=\left[\frac{(h+H)^3}{6}\right] \quad \text{--- (7)}$$

By substituting the values of

S = Specific gravity of the masonry or concrete

H = Height of weir, h = Head over the weir

a = crest or top width of the weir.

Simplifying, we can calculate the base width (b) of the weir.

It is assumed that both the sides of the weir have the same slope.

c. Apron

It is a concrete slab / slabs provided on the downstream side of the weir and in continuation with it. If the ground level on the downstream of the weir is sloping a **stepped apron** is preferred. Apron serves two functions, viz

(i) Reduces the erosion on the downstream side of the weir due to the kinetic energy of the falling jet or nappe of water.

(ii) Increases the length of the seepage line beneath the structure and hence reduces the under mining of the structure.

The thickness and length of the apron has to be calculated considering the maximum or the **worst uplift** conditions, which generally occurs when the up stream water level is at MWL and no water on the downstream side (when gates are provided) or difference of FTL and no water on the downstream side (when there are no gates).

d. Protection works

Protection works consist of (i) Abutment, (ii) Wing walls and return wing walls, (iii) upstream and downstream revetments, bed pitchings, talus etc. (iv) Bank connections.

Abutments, wingwalls and return wing walls are designed to satisfy the middle third rule.

Generally the top width of these structures is taken as 0.5m and the bottom width as 0.4 times the height of the structure.

Revetments, bed pitchings, bank connections etc. are provided as per field requirement.

SHEET - 3 DIRECT SLUICE OR GATE SLUICE

Introduction

A Sluice is an outlet provided in a dam, or a bund to supply water from the reservair to the channel on the downstream side.

A sluice can be provided at the junction of a distributary canal with the main canal.

Date required

* **Command area :** It is the area that can be supplied with water by the canal, flowing under gravitational conditions. The command area of a canal is expressed in Hectares or metre^2

  (1 Hectare = 100 m × 100 m = 10^4 m^2)

* **Duty :** It is the irrigating capacity of an unit of water flowing throughout the base period of the crop.

  It is also depend as the area that can be irrigated by one cumec of water flowing continuously throughout the base period of the crop.

* **Water levels :** Maximum water level (MWL), full tank level (FTL), and average low water level (LWL) of the above three values, LWL is important as it decides the head acting over the sluice.

* **Foundation and downstream details:** The data helps in the design of necessary protection

If the conveyance losses and other losses are to be accounted, the increased discharge would be

$$\left.\begin{array}{c}\text{Modified discharge}\\ \text{or}\\ \text{Discharge at head works}\end{array}\right\} = \left\{\frac{Q}{(1-\text{losses})}\right\} \qquad \text{--- (ii)}$$

* **Sluice Ventway**

The sluice is designed as a vertical circular orifice capable of meeting the downstream discharge requirements, when working under the least possible head.

Hence, using the discharge equation for an orifice.

$$Q = c_d\, a \sqrt{2gh} \qquad \text{--- (iii)}$$

Where, Q = Downstream discharge requirement $\left(\frac{m^3}{s}\right)$

c_d = Coefficient of discharge of the orifice $\cong 0.6$

a = area of the orifice

h = minimum driving head or head acting over the centre of the orifice corresponding to low water level (LWL)

$$\therefore \quad a = \frac{Q}{c_d \sqrt{2gh}} \qquad \text{--- (iv)}$$

But, for a circular --- $a = \frac{\pi d^2}{4}$

$$\therefore \quad d = \sqrt{\frac{4a}{\pi}} \qquad \text{--- (v)}$$

Where, d = diameter of the orifice opening.

* **Sluice barrel**

It will be a rectangular tunnel with side walls; foundation and a roof slab, constructed across the section of the dam. Its size should be such that there should be sufficient space for carrying out repair works inside the barrel.

* **Gate**

If will be fabricated of mild steel sheets and connected to a regulating arrangement on its top.

* **Protection works**

* Head wall : It is a retaining wall, built perpendicular to the flow on the upstream side of the dam. It will rest on top of the sluice barrel; with its top being slightly above the maximum water level.

Head wall acts as a retaining wall retaining the embankment from top bund level (TBL) upto the top of the roof slab. It also helps by providing space for the person operating the gate mechanism.

The top width of the head wall is generally taken as 0.5m while its bottom width will be 0.4 times the height of embankment retained.

* Guide walls : These are walls built just upstream of the head wall, parallel to the direction of flow. The top of the guide walls will be at the same level as the top of the head wall. The bottom would rest directly on the foundation.

Guide walls guide the flow of water into the sluice barrel as well as guides the movement of the gate.

* U/s wing walls : They are generally built upstream of the guide walls at an angle to the flow (generally 14°) or a splay of 4 : 1.

Wing walls retain the embankment and also provide smooth transition for the flow of water into the barrel.

* D/s return wing walls : These are built perpendicular to the direction of flow. The d/s return wing walls act as a junction between the rectangular barrel and the trapezoidal channel, as well as retain the embankment.

* Revetments, bed pitchings etc. : These are provided as per the field requirement.