1. Introduction

Irrigation engineering is the analysis and design of systems that optimally supply the right amount of water to the soil at the right time to meet the needs of the plant system.

2. Necessity

1- Less Rainfall
2- Non-uniform Rainfall
3- Growing a number of crops during a year.
4- Growing Perennial crops.
5- Commercial crops with additional water.
6- Controlled water supply.

3. Advantage of application of water by modern methods

1- It adds water to the soil to supply the moisture essential for the plant growth.
2- It saves the crops from drying during short duration droughts.
3- It cools the soil and the atmosphere, and thus makes more favorable environment for healthy plant growth.
4- It washes out or dilutes salts in the soil.
5- It reduces the hazard of soil piping.
6- It softens the tillage pans.

4. Scope of irrigation science

The scope of irrigation is not limited to the application of water to the soil. It deals with all aspects and problems extending from the watershed to the agricultural farms. It deals with the design and construction of all works, such as dams, weirs, head regulators etc. in connection with the storage or diversion of water, as well as he problems of subsoil drainage, soil reclamation and water-soil-crop relationships, An irrigation engineer is also required to have the knowledge of cultivation of various crops, their maturing and protection from pests. Briefly speaking, the scope of irrigation can be divided into two heads:

a-Engineering aspect

1- Storage, Diversion, or lifting of water
2- Conveyance of water to the Agricultural fields.
3- Application of water to Agricultural fields.
4- Drainage and relieving water-Logging.
5- Development of water power.

b-Agricultural Aspect

the agricultural aspect deals with the thorough study of the following points.
1- Proper depth of water necessary in single application of water for various crops.
2- Distribution of water uniformly and periodically.
3- Capacities of water uniformly and periodically.
4- Reclamation of waste and alkaline lands, where this can be carried out through the agency of water.

5. Benefits of irrigation

1- Increase in food production.
2- Protection from famine .
3- Cultivation of cash crops.
4- Elimination of mixed cropping.
5- Addition to the wealth of the country.
6- Increase in prosperity of people.
7- Generation of Hydro-Electric power.
8- Domestic and industrial water supply.
9- Inland navigation .
10- Improvements of communication.
11- Canal plantations.
12- Improvement in the Ground water storage.
13- Aid in Civilization.
14- General Development of the country .

6. Basic Design Factors

1- Consumptive Use (or Evapotranspiration) الاستهلاك المائي او التبخر-نتح
Consumptive use refers to the water needs of a crop in a specified time and is the sum of the volume of transported and evaporated water.
2- Root-zone soil water مياه التربة في المنطقة الجذرية
Water serves the following useful functions in the process of plant growth:
(i) Germination of seeds، إنبات البذور ,
(ii) All chemical reactions,
(iii) All biological processes,
(iv) Absorption of plant nutrients through their aqueous solution,
(v) Temperature control,
(vi) Tillage operationsعمليات الحرث , and
(vii) Washing out or dilution of salts.

Soil water can be divided into three categories:
(i) Gravity (or gravitational or free) water,
(ii) Capillary water, and
(iii) Hygroscopic water.

Gravity water is that water which drains away under the influence of gravity. Soon after irrigation (or rainfall) this water remains in the soil and saturates the soil, thus preventing circulation of air in void spaces.

The *capillary water* is held within soil pores due to the surface tension forces (against gravity) which act at the liquid-vapour (or water-air) interface.

Water attached to soil particles through loose chemical bonds is termed *hygroscopic water*. This water can be removed by heat only. But, the plant roots can use a very small fraction of this moisture under drought conditions.

مياه الجاذبية هي تلك المياه التي تستنزف تحت تأثير الجاذبية بعد الري (أو هطول الأمطار) يبقى هذا الماء في التربة ويشبع التربة ، وبالتالي يمنع دوران الهواء في فراغات.

يتم الاحتفاظ بالمياه الشعرية داخل مسام التربة بسبب قوى التوتر السطحي (ضد الجاذبية) التي تعمل على واجهة بخار السائل (أو الماء والهواء). المياه المرتبطة بجزيئات التربة من خلال الروابط الكيميائية السائبة تسمى الماء الاسترطابي. يمكن إزالة هذا الماء بالحرارة فقط. ولكن ، يمكن أن تستخدم جذور النباتات صغيرة جدا جزء من هذه الرطوبة تحت ظروف الجفاف.

The water remaining in the soil after the removal of gravitational water is called the field capacity. Field capacity of a soil is defined as the moisture content of a deep, permeable, and well-drained soil several days after a thorough wetting.

يتم تعريف السعة الحقلية للتربة على أنها محتوى الرطوبة العميقة ، ويسمى الماء المتبقي في التربة بعد إزالة المياه الجاذبية بـ السعة الحقلية ونفاذية ، والتربة جيدا استنزفت عدة أيام بعد ترطيب شامل.

Permanent wilting point is defined as the soil moisture fraction, Wwp at which the plant leaves wilt (or droop) permanently and applying additional water after this stage will not relieve the wilted condition.

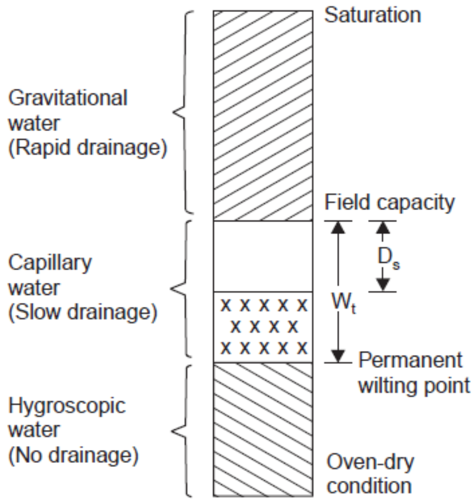


Fig 1. Different stages of soil moisture content in a soil

1- Crop and Crop seasons.

One of the primary drivers in irrigation system selection is crop type. For example, vegetable crops cannot be flooded. the crops was divided into four general categories .

Category 1. Row or bedded crops: sugar beets, sugarcane, potatoes, pineapple, cotton, soybeans, corn, sorghum, milo, vegetables, vegetable and flower seed, melons, tomatoes, and strawberries.

Category 2. Close-growing crops (sown, drilled, or sodded): small grain, alfalfa, pasture, and turf.

Category 3. Water flooded crops: rice and taro.

Category 4. Permanent crops: orchards of fruit and nuts, citrus groves, grapes, cane berries, blueberries, cranberries, bananas and papaya plantations, hops, and trees and shrubs for windbreaks, wildlife, landscape, and ornamentals.

1-1 Multiple Cropping

There are two forms of multiple cropping:

(i) intercropping, and (ii) sequential cropping. When two or more crops are grown simultaneously on the same field, it is termed **intercropping**. Crop intensification is in both time and space dimensions. There is, obviously, strong intercrop competition in this form of multiple cropping. On the other hand, when two or more crops are grown in sequence on the same field in a year, it is termed **sequential cropping**. The succeeding crop is planted after the preceding crop has been harvested. Crop intensification is only in time dimension and there is no intercrop competition in sequential cropping.

2- consumptive use (OR EVAPOTRANSPIRATION)

The combined loss of water from soil and crop by vaporisation is identified as evapotranspiration. Crops need water for transpiration and evaporation. During the growing period of a crop,there is a continuous movement of water from soil into the roots, up the stems and leaves, and out of the leaves to the atmosphere. This movement of water is essential for carrying plant food from the soil to various parts of the plant. Only a very small portion (less than 2 per cent)of water absorbed by the roots is retained in the plant and the rest of the absorbed water, after performing its tasks, gets evaporated to the atmosphere mainly through the leaves and stem. This process is called transpiration. In addition, some water gets evaporated to the atmosphere directly from the adjacent soil and water surfaces and from the surfaces of the plant leaves(i.e., the intercepted precipitation on the plant foliage). The water needs of a crop thus consists of transpiration and evaporation and is called evapotranspiration or consumptive use.

Consumptive use refers to the water needs of a crop in a specified time and is the sum of the volume of transpirated and evaporated water. **Consumptive use is defined as the amount of water needed to meet the water loss through evapotranspiration.** It generally applies to a crop but can be extended to a field, farm, project or even a valley. **Consumptive use is generally measured as volume per unit area or simply as the depth of water on the irrigated area.**

Knowledge of consumptive use helps determine irrigation requirement at the farm which should, obviously, be the difference between the consumptive use and the effective precipitation.

Evapotranspiration is dependent on climatic conditions like temperature, daylight hours,humidity, wind movement, type of crop, stage of growth of crop, soil moisture depletion, and other physical and chemical properties of soil. For example, in a sunny and hot climate, crops need more water per day than in a cloudy and cool climate. Similarly, crops like rice or sugarcane need more water than crops like beans and wheat. Also, fully grown crops need more water than crops which have been just planted.

Potential evapotranspiration from a cropped surface can be estimated either by correlating potential evapotranspiration with water loss from evaporation devices or by estimations based on various climatic parameters. Correlation of potential evapotranspiration assumes that the climatic conditions affecting crop water loss (Det) and vaporation from a free surface of water (Ep) are the same. Potential evapotranspiration Det can be correlated to the pan evaporation Ep as ,

$$D_{et} = KE_p \ldots 1$$

in which, K is the crop factor for that period. The crop factor K depends on the crop as well as its stage of growth (Table 1). The main limitations of this method are the differences in physical features of evaporation surfaces compared with those of a crop surface.

Table 1 Values of crop factor *K* from some major crops

Percentage of crop growing season since sowing	*Maize, cotton, potatoes, peas and sugarbeets*	*Wheat, barley and other small grains*	*Sugarcane*	*Rice*
0	0.20	0.08	0.50	0.80
10	0.36	0.15	0.60	0.95
25	0.75	0.33	0.75	1.10
50	1.00	0.65	1.00	1.30
75	0.85	0.90	0.85	1.15
100	0.20	0.20	0.50	0.20

In the absence of pan evaporation data, the consumptive use is generally computed as follows:

(i) Compute the seasonal (or monthly) distribution of potential evapotranspiration, which is defined as the evapotranspiration rate of a well-watered reference crop which completely **shades** the soil surface . It is thus an indication of the climatic evaporation demand of a vigorously growing crop.

Usually, grass and alfalfa (a plant with leaves like that of clover and purple flowers used as food for horses and cattle) are taken as reference crops.

(ii) Adjust the potential evapotranspiration for the type of crop and the stage of crop growth. Factors such as soil moisture depletion are ignored so that the estimated values of the consumptive use are conservative values to be used for design purposes.
Thus, evapotranspiration of a crop can be estimated by multiplying potential evapotranspiration by a factor known as crop coefficient.

Potential evapotranspiration can be computed by one of the several methods available for the purpose. These methods range in sophistication from simple temperature correlation (such as the Blaney-Criddle formula) to equations (such as Penman's equation) which account for radiation energy as well. Blaney-Criddle formula for the consumptive use has been used extensively and is expressed as (1)

$$u = kf \quad \dots 2$$

in which, u = consumptive use of crop in mm,
k = empirical crop consumptive use coefficient (Table 2), and
f = consumptive use factor.

The quantities u, k, and f are determined for the same period (annual, irrigation season,growing season or monthly). The consumptive use factor f is expressed as

$$f = \frac{p}{100}(1.8t + 32) \quad \dots..3$$

in which, t = mean temperature in °C for the chosen period, and
p = percentage of daylight hours of the year occurring during the period.
Table 3 lists the values of p for different months of a year for 0° north latitude. The value of the consumptive use is generally determined on a monthly basis and the irrigation system must be designed for the maximum monthly water needs. It should be noted that Eq.(2) was originally in FPS system with appropriate values of k. Similarly, Eq. (3) too had a different form with t in Fahrenheit.

Table 2 Consumptive use coefficient for some major crops (1)

Crop	*Lenght of normal growing season or period*	*Consumptive use coefficient, k*	
		*For the growing period**	*Monthly (maximum value)***
Corn (maize)	4 months	19.05 to 21.59	20.32 to 30.48
Cotton	7 months	15.24 to 17.78	19.05 to 27.94
Potatoes	3-5 months	16.51 to 19.05	21.59 to 25.40
Rice	3-5 months	25.40 to 27.94	27.94 to 33.02
Small grains	3 months	19.05 to 21.59	21.59 to 25.40
Sugarbeet	6 months	16.51 to 19.05	21.59 to 25.40
Sorghums	4-5 months	17.78 to 20.32	21.59 to 25.40
Orange and lemon	1 year	11.43 to 13.97	16.21 to 19.05

*The lower values are for more humid areas and the higher values are for more arid climates.
** Dependent upon mean monthly temperature and stage of growth of crop.

Table 3 Per cent daylight hours for northern hemispere (0-50° latitude)

Latitude North (in degr-ees)	*Jan.*	*Fab.*	*March*	*April*	*May*	*June*	*July*	*Aug.*	*Sep.*	*Oct.*	*Nov.*	*Dec.*
0	8.50	7.66	8.49	8.21	8.50	8.22	8.50	8.49	8.21	8.50	8.22	8.50
5	8.32	7.57	8.47	8.29	8.65	8.41	8.67	8.60	8.23	8.42	8.07	8.30
10	8.13	7.47	8.45	8.37	8.81	8.60	8.86	8.71	8.25	8.34	7.91	8.10
15	7.94	7.36	8.43	8.44	8.98	8.80	9.05	8.83	8.28	8.26	7.75	7.88
20	7.74	7.25	8.41	8.52	9.15	9.00	9.25	8.96	8.30	8.18	7.58	7.66
25	7.53	7.14	8.39	8.61	9.33	9.23	9.45	9.09	8.32	8.09	7.40	7.42
30	7.30	7.03	8.38	8.72	9.53	9.49	9.67	9.22	8.33	7.99	7.19	7.15
32	7.20	6.97	8.37	8.76	9.62	9.59	9.77	9.27	8.34	7.95	7.11	7.05
34	7.10	6.91	8.36	8.80	9.72	9.70	9.88	9.33	8.36	7.90	7.02	6.92
36	6.99	6.85	8.35	8.85	9.82	9.82	9.99	9.40	8.37	7.85	6.92	6.79
38	6.87	6.79	8.34	8.90	9.92	9.95	10.10	9.47	8.38	7.80	6.82	6.66
40	6.76	6.72	8.33	8.95	10.02	10.08	10.22	9.54	8.39	7.75	6.72	6.52
42	6.63	6.65	8.31	9.00	10.14	10.22	10.35	9.62	8.40	7.69	6.62	6.37
44	6.49	6.58	8.30	9.06	10.26	10.38	10.49	9.70	8.41	7.63	6.49	6.21
46	6.34	6.50	8.29	9.12	10.39	10.54	10.64	9.79	8.42	7.57	6.36	6.04
48	6.17	6.41	8.27	9.18	10.53	10.71	10.80	9.89	8.44	7.51	6.23	5.86
50	5.98	6.30	8.24	9.24	10.68	10.91	10.99	10.00	8.46	7.45	6.10	5.65

Table 4 gives typical values of the water needs of some major crops for the total growing period of some of the crops . This table also indicates the sensitivity of the crop to water shortages or drought. High sensitivity to drought means that the crop cannot withstand water shortages, and that such shortages should be avoided.

Table 4 Indicative values of crop water needs and sensitivity to drought

Crop	*Crop water need (mm / total growing period)*	*Sensitivity of drought*
Alfalfa	800 - 1600	low - medium
Banana	1200 - 2200	high
Barley/oats/wheat	450 - 650	low - medium
Bean	300 - 500	medium - high
Cabbage	350 - 500	medium - high
Citrus	900 - 1200	low - medium
Cotton	700 - 1300	low
Maize	500 - 800	medium - high
Melon	400 - 600	medium - high
Onion	350 - 550	medium - high
Peanut	500 - 700	low - medium
Pea	350 - 500	medium - high
Pepper	600 - 900	medium - high
Potato	500 - 700	high
Rice (paddy)	450 - 700	high
Sorghum/millet	450 - 650	low
Soybean	450 - 700	low - medium
Sugarbeet	550 - 750	low - medium
Sugarcane	1500 - 2500	high
Sunflower	600 - 1000	low - medium
Tomato	400 - 800	medium - high

Example 1: Using the Blaney-Criddle formula, **estimate the yearly consumptive use** of water for sugarcane for the data given in the first four columns of Table 5.

Solution:

According to Eqs 2 & 3

$$u = k\frac{p}{100}(1.8\,t + 32)$$

Values of monthly consumptive use calculated from the above formula have been tabulated in the last column of Table 5. Thus, yearly consumptive use = Σu = 1.75 m.

Table 5 Data and solution for Example 1

Month	*Mean monthly temperature, t°C*	*Monthly crop coefficient, k*	*Per cent sunshine hours, p*	*Monthly consumptive use, u (mm)*
January	13.10	19.05	7.38	78.14
February	15.70	20.32	7.02	85.96
March	20.70	21.59	8.39	125.46
April	27.00	21.59	8.69	151.22
May	31.10	22.86	9.48	190.66
June	33.50	24.13	9.41	209.58
July	30.60	25.40	9.60	212.34
August	29.00	25.40	9.60	205.31
September	28.20	24.13	8.33	166.35
October	24.70	22.86	8.01	140.01
November	18.80	21.59	7.25	103.06
December	13.70	19.05	7.24	78.15

1- DUTY OF WATER:

For proper planning of a canal system, the designer has to first decide the 'duty of water' in the locality under consideration. **Duty is defined as the area irrigated by a unit discharge of water flowing continuously for the duration of the base period of a crop**. The ***base period*** of **a crop is the time duration between the first watering at the time of sowing and the last watering before harvesting the crop**. Obviously, the base period of a crop is smaller than the crop period. **Duty is measured in hectares/m3/s**. **The duty of a canal depends on the crop, type of soil, irrigation and cultivation methods, climatic factors, and the channel conditions**.

By comparing the duty of a system with that of another system or by comparing it with the corresponding figures of the past on the same system, one can have an idea about the performance of the system. Larger areas can be irrigated if the duty of the irrigation system is improved. Duty can be improved by the following measures:

(i) The channel should not be in sandy soil and be as near the area to be irrigated as possible so that the seepage losses are minimum. Wherever justified, the channel may be lined.

(ii) The channel should run with full supply discharge as per the scheduled program so that farmers can draw the required amount of water in shorter duration and avoid the tendency of unnecessary over irrigation.

(iii) Proper maintenance of watercourses and outlet pipes will also help reduce losses, and thereby improve the duty.

(iv) Volumetric assessment of water makes the farmer to use water economically. This is, however, more feasible in well irrigation

Well irrigation has higher duty than canal irrigation due to the fact that water is used economically according to the needs. Open wells do not supply a fixed discharge and, hence, the average area irrigated from an open well is termed its duty. Between the head of the main canal and the outlet in the distributary, there are losses due to evaporation and percolation. As such, duty is different at different points of the canal system. The duty at the head of a canal system is less than that at an outlet or in the tail end region of the canal. Duty is usually calculated for the head discharge of the canal. Duty calculated on the basis of outlet discharge is called 'outlet discharge factor' or simply 'outlet factor' which excludes all losses in the canal system.

Imagine a field growing a single crop having a base period B days and a Delta Δ mm which is being supplied by a source located at the head (uppermost point) of the field. The water being supplied may be through the diversion of river water through a canal, or it could be using ground water by pumping. If the water supplied is just enough to raise the crop within D hectares of the field, then a relationship may it found out amongst all the variables as:

D = duty in hectares/cumec
Δ = total depth of water supplied (in metres)
B = base period in days.

(*i*) If we take a field of area D hectares, water supplied to the field corresponding to the water depth Δ metres will be $= \Delta \times D$ hectare-metres

$$= D \times \Delta \times 10^4 \text{ cubic-metres.} \quad ...(1)$$

(*ii*) Again for the same field of D hectares, one cumec of water is required to flow during the entire base period. Hence, water supplied to this field

$$= (1) \times (B \times 24 \times 60 \times 60)\ \text{m}^3 \quad ...(2)$$

Equating Equations (1) and (2), we get

$$D \times \Delta \times 10^4 = B \times 24 \times 60 \times 60$$

or

$$\Delta = \frac{B \times 24 \times 60 \times 60}{D \times 10^4} = 8.64 \frac{B}{D} \text{ metres}$$

Note : 1 hectare = 10^4 sq. metres
1 cumec-day = 8.64 hectare-metres.

Example *Find the delta for a crop if the duty for a base period of 110 days is 1400 hectares/cumec.*

Solution : Given : $B = 110$ days and $D = 1400$ hectares/cumec

$$\therefore \quad \Delta = 8.64 \frac{B}{D} = \frac{8.64 \times 110}{1400} \text{ m} = 0.68 \text{ m} = \mathbf{68 \text{ cm}}$$

Example *A crop requires a total depth of 92 cm of water for a base period of 120 days. Find the duty of water.*

Solution : Given : $B = 120$ days and $\Delta = 92$ cm $= 0.92$ m

$$D = \frac{8.64\,B}{\Delta} \text{ hectares/cumec} = \frac{8.64 \times 120}{0.92} = \mathbf{1127 \text{ hectares/cumec.}}$$

FACTORS AFFECTING DUTY

The duty of water of canal system depends upon a variety of the factors. The principal factors are :

1. Methods and system of irrigation ;
2. Mode of applying water to the crops ;
3. Method of cultivation ;
4. Time and frequency of tilling ;
5. Type of the crop ;
6. Base period of the crop ;
7. Climatic conditions of the area;
8. Quality of water ;
9. Method of assessment of irrigation method ;
10. Canal conditions ;
11. Character of soil and sub-soil of the canal ;
12. Character of soil and sub-soil of the irrigation fields.

IRRIGATION EFFICIENCIES

Efficient use of irrigation water is an obligation of each user as well as of the planners. Even under the best method of irrigation, not all the water applied during an irrigation is stored in the root zone. In general, efficiency is the ratio of water output to the water input and is expressed as percentage. The objective of efficiency concepts is to show when improvements can be made which will result in more efficient irrigation. The following are the various types of irrigation efficiencies : (*i*) water conveyance efficiency, (*ii*) water application efficiency, (*iii*) water use efficiency, (*iv*) water storage efficiency, (*v*) water distribution efficiency and (*vi*) consumptive use efficiency.

1. *Water Conveyance Efficiency* (η_c)

This takes into account the conveyance or transit losses and is determined from the following expression :

$$\eta_c = \frac{W_f}{W_r} \times 100$$

where η_c = water conveyance efficiency
W_f = water delivered to the farm or irrigation plot
W_r = water supplied or diverted from the river or reservoir.

2. *Water Application Efficiency* (η_a)

The water application efficiency is the ratio of the quantity of water stored into the root zone of the crops to the quantity of water delivered to the field. This focuses the attention of the suitability of the method of application of water to the crops. It is determined from the following expression :

$$\eta_a = \frac{W_s}{W_f} \times 100$$

where η_a = water application efficiency
W_s = Water *stored* in the root zone during the irrigation
W_f = water delivered to the farm.

The common sources of loss of irrigation water during water application are (*i*) surface run off R_f from the farm and (*ii*) deep percolation D_f below the farm root-zone soil. Hence

$$W_f = W_s + R_f + D_f$$

and

$$\eta_a = \frac{W_f - (R_f + D_f)}{W_f} \times 100$$

In a well designed surface irrigation system, the water application efficiency should be atleast 60% ; in the sprinkler irrigation system this efficiency is about 75%.

3. *Water Storage Efficiency* (η_s)

The concept of water storge efficiency gives an insight to how completely the required water has been stored in the root zone during irrigation. It is determined from the following expression:

$$\eta_s = \frac{W_s}{W_n} \times 100$$

where η_s = water storage efficiency

W_s = water *stored* in the root zone during irrigation

W_n = water *needed* in the root zone prior to irrigation

= (Field capacity – Available moisture).

4. *Water Distribution Efficiency* (η_d)

Water distribution efficiency evaluates the degree to which water is uniformly distributed throughout the root zone. Uneven distribution has many undesirable results. The more uniformly the water is distributed, the better will be the crop response. It is determined from the following expression :

$$\eta_d = 100\left[1 - \frac{y}{d}\right]$$

where η_d = water distribution efficiency

y = average numerical deviation in depth of water stored from average depth stored during irrigation.

d = average depth of water stored during irrigation.

The efficiency provides a measure for comparing various systems or methods of water application, *i.e.* sprinkler compared to surface, one sprinkler system compared to the other system or one surface method compared to other surface method.

Example *If the depths of water stored at 5 points in a field are 1.0, 0.9, 0.8, 0.7 and 0.60 m, determine the water distribution efficiency.*

Solution. Average depth $= \dfrac{1.0 + 0.9 + 0.8 + 0.7 + 0.6}{5} = 0.80$ m

Deviations from the mean = + 0.20 + 0.10, 0.0 − 0.10 − 0.2

Absolute values of these deviations from the mean = 0.2, 0.1, 0.0, 0.10, 0.2

Average of these absolute values of deviations $= \dfrac{0.2 + 0.10 + 0.0 + 0.10 + 0.20}{5} = 0.12$

Therefore, water distribution efficiency $= \left(1 - \dfrac{0.12}{0.80}\right) \times 100 = 85\%$

Example *Five cumecs of water is supplied to a field having an area of 30 ha for 6 hours. It is found that 25 cm of water depth has been stored in the root zone of the crop. Determine the water application efficiency.*

Solution. Quantity of water applied $= 5 \times 6 \times 3600 = 10.8 \times 10^4\ m^3 = 10.8$ ha–m

Quantity of water stored in the root zone $= 30 \times 0.25 = 7.5$ ha–m

Water application efficiency $= \dfrac{7.5}{10.8} \times 100 = \mathbf{69.44\%}$

Alternative Method Depth of water applied = 10·8/30·0 = 0·36 m

Depth of water stored = 0·25

Water application efficiency $= \dfrac{0\cdot25}{0\cdot36} \times 100 = 69\cdot44\%$

PLANNING OF IRRIGATION PROJECTS

These projects mainly consist of engineering (or hydraulic) structures which collect, convey, and deliver water to areas on which crops are grown. Irrigation projects may range from a small farm unit to those serving extensive areas of millions of hectares. A small irrigation project may consist of a low diversion weir or an inexpensive pumping plant along with small ditches (channels) and some minor control structures. A large irrigation project includes a large storage reservoir, a huge dam, hundreds of kilometers of canals, branches and distributaries, control structures, and other works. Assuming all other factors (such as enlightened and experienced farmers, availability of good seeds, etc.) reasonably favorable, the following can be listed as conditions essential for the success of any irrigation project.

(i) Suitability of land (with respect to its soil, topography and drainage features) for continued agricultural production,

(ii) Favorable climatic conditions for proper growth and yield of the crops,

(iii) Adequate and economic supply of suitable quality of water, and

(iv) Good site conditions for the safe construction and uninterrupted operations of the engineering works.

Most of the irrigation projects divert stream flow into a canal system which carries water to the cropland by gravity and, hence, are called gravity projects. In pumping projects, water is obtained by pumping but delivered through a gravity system

A gravity type irrigation project mainly includes the following works:

(i) Storage (or intake) and diversion works,

(ii) Conveyance and distribution channels.

(iii) Conveyance, control, and other hydraulic structures,

(iv) Farm distribution, and

(v) Drainage works.

Development of an Irrigation Project

A small irrigation project can be developed in a relatively short time. Farmers having land suitable for agriculture and a source of adequate water supply can plan their own irrigation system, secure necessary finance from banks or other agencies, and get the engineering works constructed without any delay. On the other hand, development of a large irrigation project is more complicated and time-consuming. Complexity and the time required for completion of a large project increase with the size of the project. This is due to the organizational, legal, financial administrative, environmental, and engineering problems all of which must be given detailed consideration prior to the construction of the irrigation works. The principal stages of a large irrigation project are: (i) the promotional stage, (ii) the planning stage, (iii) the construction stage, and (iv) the settlement stage. The planning stage itself consists of three substages: (i) preliminary planning including feasibility studies, (ii) detailed planning of water and land use, and (iii) the design of irrigation structures and canals. Engineering activities are needed during all stages (including operation and maintenance) of development of an irrigation project. However, the planning and construction stages require most intensive engineering activities. A large irrigation project may take 10–30 years for completion depending upon the size of the project.

IRRIGATION METHODS

Irrigation water can be applied to croplands using one of the following irrigation methods:
(i) Surface irrigation which includes the following:
(a) Uncontrolled (or wild or free) flooding method,
(b) Border strip method,
(c) Check method,
(d) Basin method, and
(e) Furrow method.
(ii) Subsurface irrigation
(iii) Sprinkler irrigation
(iv) Trickle irrigation

Each of the above methods has some advantages and disadvantages, and the choice of the method depends on the following factors :
- Size, shape, and slope of the field,
- Soil characteristics,
- Nature and availability of the water supply subsystem,
- Types of crops being grown,
- Initial development costs and availability of funds, and
- Preferences and past experience of the farmer.

The design of an irrigation system for applying water to croplands is quite complex and not amenable to quantitative analysis. Principal criteria for the design of a suitable irrigation method are as follows :
(i) Store the required water in the root-zone of the soil,
(ii) Obtain reasonably uniform application of water,
(iii) Minimise soil erosion,
(iv) Minimise run-off of irrigation water from the field,
(v) Provide for beneficial use of the runoff water,
(vi) Minimise labour requirement for irrigation,
(vii) Minimise land use for ditches and other controls to distribute water,
(viii) Fit irrigation system to field boundaries,
(ix) Adopt the system to soil and topographic changes, and
(x) Facilitate use of machinery for land preparation, cultivating, furrowing, harvesting, and so on.

1. Well and Tube Well Irrigation

- There are various types of wells – shallow wells, deep wells, tube wells, artesian wells, etc. From the shallow wells water is not always available as the level of water goes down during the dry months. Deep wells are more suitable for the purpose of irrigation as water from them is available throughout the year.

- At places where ground water is available, a tube-well can be installed near the agricultural area. A deep tube well worked by electricity, can irrigate a much larger area (about 400 hectares) than a surface well (half hectares).

- **Merits**: Well is simplest, cheapest and independent source of irrigation and can be used as and when the necessity arises. Several chemicals such as nitrate, chloride, sulphate, etc. found in well water add to the fertility of soil. More reliable during periods of drought when surface water dries up

- **Demerits:** Only limited area can be irrigated. In the event of a drought, the ground water level falls and enough water is not available. Tubewells can draw a lot of groundwater from its neighbouring areas and make the ground dry and unfit for agriculture.

2. Canal Irrigation

- Canals can be an effective source of irrigation in areas of low- level relief, deep fertile soils, perennial source of water and extensive command area.

- The digging of canals in rocky and uneven areas is difficult and uneconomic.

- **Merits:** Most of the canals provide perennial irrigation and supply water as and when needed. This saves the crops from drought conditions and helps in increasing the farm production.

- **Demerits:** Many canals overflow during the rainy season and flood the surrounding areas. Canal irrigation is suitable in plain areas only.

3. Tank Irrigation

- A tank is developed by constructing a small bund of earth or stones built across a stream. The water impounded by the bund is used for irrigation and other purposes.

- **Merits:** Most of the tanks are natural and do not involve heavy cost for their construction and have longer life span. In many tanks, fishing is also carried on, which supplements both the food resources and income of the farmer.

- **Demerits:** Many tanks dry up during the dry season and fail to provide irrigation when it is required. Lifting of water from tanks and carrying it to the fields is a strenuous and costly exercise.

4. Drip Irrigation

- In drip irrigation, water is applied near the plant root through emitters or drippers, on or below the soil surface, at a low rate varying from 2-20 liters per hour. The soil moisture is kept at an optimum level with frequent irrigations.

- Among all irrigation methods, drip irrigation is the most efficient and can be practiced for a large variety of crops, especially in vegetables, orchard crops, flowers and plantation crops.
- **Merits:** Fertilizer and nutrient loss is minimized due to localized application and reduced leaching. Field leveling is not necessary. Recycled non-potable water can be used. Water application efficiency increases. Soil erosion and weed growth is lessened.
- **Demerits:** Initial cost can be more, can result in clogging, wastage of water, time and harvest, if not installed properly.

5. Sprinkler Irrigation

- In this method, water is sprayed into the air and allowed to fall on the ground surface somewhat resembling rainfall. The spray is developed by the flow of water under pressure through small orifices or nozzles. The sprinkler irrigation system is a very suitable method for irrigation on uneven lands and on shallow soils.
- Nearly all crops are suitable for sprinkler irrigation systems except crops like rice, etc. The dry crops, vegetables, flowering crops, orchards, plantation crops like tea, coffee are all suitable and can be irrigated through sprinklers.

- **Merits:** Suitable to all types of soil except heavy clay. Water saving. Increase in yield. Saves land as no bunds etc. are required.
- **Demerits:** Higher initial cost. Under high wind conditions and high temperature distribution and application efficiency is poor.

Definitions of Irrigation concepts

Net depth of irrigation (dn) : is the depth of water applied and stored in the root zone (It is only water available for plant growth).

Leaching requirements (L.R) : is defined as the fraction of irrigation water that must be leaching through the root zoon to control soil salinity at any specified level.

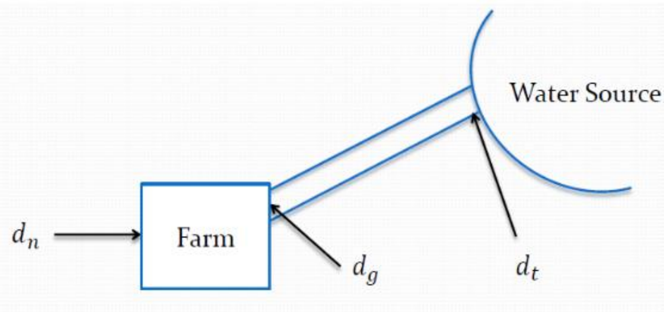
Gross depth of irrigation (dg) : is the depth of water delivered to the farms, which content the amount of net depth, water loss (run off + deep percolation), leaching requirements, and rain fall.

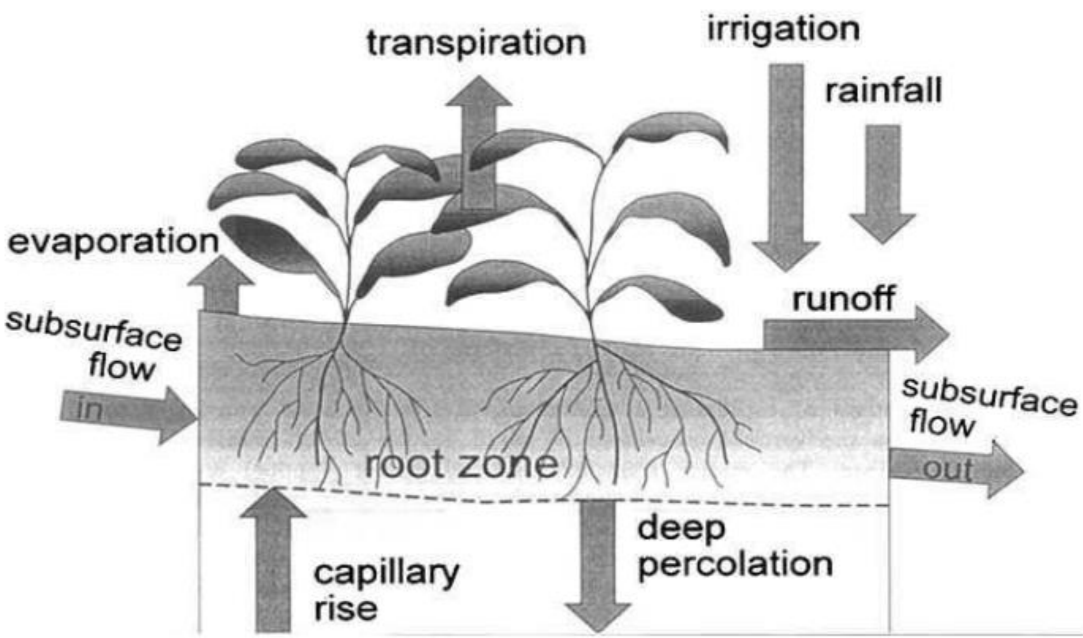
$$d_g = d_n + \text{water losses} + \text{LR} - \text{effective rainfall}$$

Total depth of water (dt) : is that depth of water delivered from the water source (reservoir, river, etc) for irrigation requirement.

$$d_t = d_g + canal\ seepage + evaporation$$

- Seepage losses from earth canals = 35%,
- Seepage losses from lining canals = 5.8%





Application efficiency (Ea): The ratio between water stored in the soil root zone during irrigation to water delivered to the farm.

$$E_a = \frac{d_n}{d_g} * 100\%$$

Conveyance Efficiency (CE): It is the percentage ratio between depth of water at the farm and depth of water delivered from irrigation source.

$$CE = \frac{d_g}{d_t} * 100\%$$

Overall Efficiency (OE) = Ea*CE

$$CE = \frac{d_n}{d_t} * 100\%$$

Example (1): given

✓Gross depth 150mm ,

✓Irrigation efficiency 80% ,

❖Find net depth of irrigation

❖ Find the depth of water lost during irrigation process.

Example (2): Given the following data of an irrigation project: Ea=85% and CE=75%.Find the percentage of water from the total depth that useful for plants.

1- CANAL IRRIGATION:

Irrigation conduits of a typical gravity project are usually open channels through earth or rock formations. These are called **canals**.

A ***canal*** **is defined as an artificial channel constructed on the ground to carry water from a river or another canal or a reservoir to the fields. Usually, canals have a trapezoidal crosssection.**

2- Classification of canals

Canals can be classified in many ways:

Based on the **nature of source of supply**, a canal can be either a **permanent** or **an inundation canal**. *A permanent canal* has a continuous source of water supply. Such canals are also called perennial canals. *An inundation canal* draws its supplies from a river only during the high stages of the river. Such canals do not have any headworks for diversion of river water to the canal, but are provided with a canal head regulator.

Depending **on their function**, canals can also be classified as: (i**) irrigation**, (ii**) navigation**, (iii) **power**, and (iv) **feeder canals**. An irrigation canal carries water from its source to agricultural fields. Canals used for transport of goods are known as navigation canals. Power canals are used to carry water for generation of hydroelectricity. A feeder canal feeds two or more canals.

An irrigation canal system consists of canals of different sizes and capacities (Fig.1). Accordingly, the canals are also classified as: (i) **main canal**, (ii) **branch canal**, (iii) **major distributary**, (iv) **minor distributary**, and (v) **watercourse**.

The main canal takes its supplies directly from the river through the head regulator and acts as a feeder canal supplying water to branch canals and major distributaries. Usually, direct irrigation is not carried out from the main canal.

Branch canals (also called 'branches') take their supplies from the main canal. Branch canals generally carry a discharge higher than 5 m3/s and act as feeder canals for major and minor distributaries. Large branches are rarely used for direct irrigation. However, outlets are provided on smaller branches for direct irrigation.

Major distributaries (also called 'distributaries' or rajbaha) carry 0.25 to 5 m3/s of discharge. These distributaries take their supplies generally from the branch canal and sometimes from the main canal. The distributaries feed either watercourses through outlets or minor distributaries.

Minor distributaries (also called 'minors') are small canals which carry a discharge less than 0.25 m3/s and feed the watercourses for irrigation. They generally take their supplies from major distributaries or branch canals and rarely from the main canals.

A watercourse is a small channel which takes its supplies from an irrigation channel (generally distributaries) through an outlet and carries water to the various parts of the area to be irrigated through the outlet.

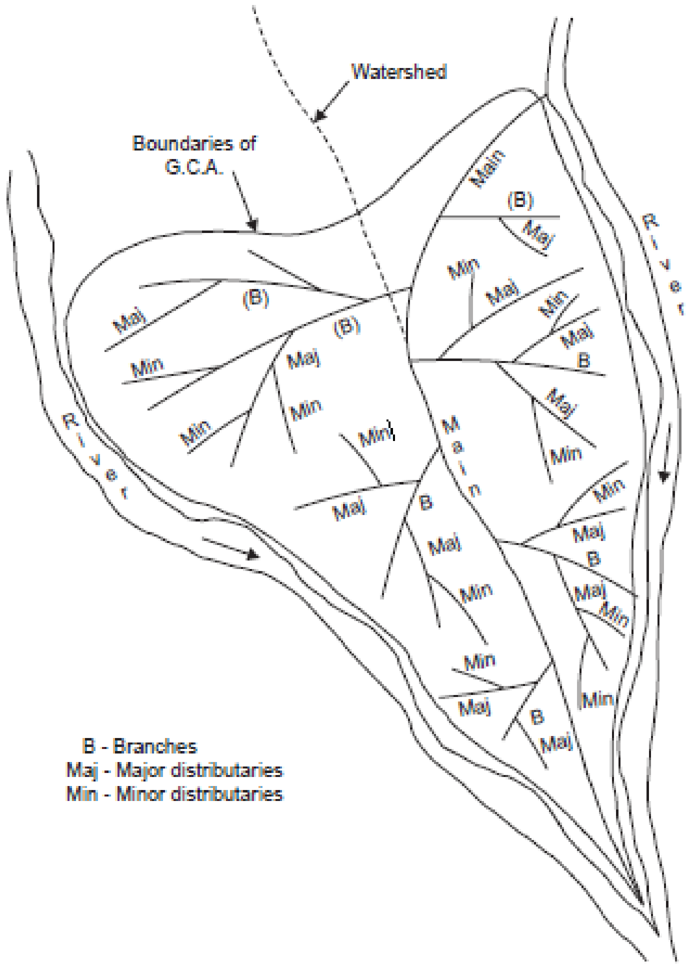


Fig (.1) Layout of an irrigation canal network

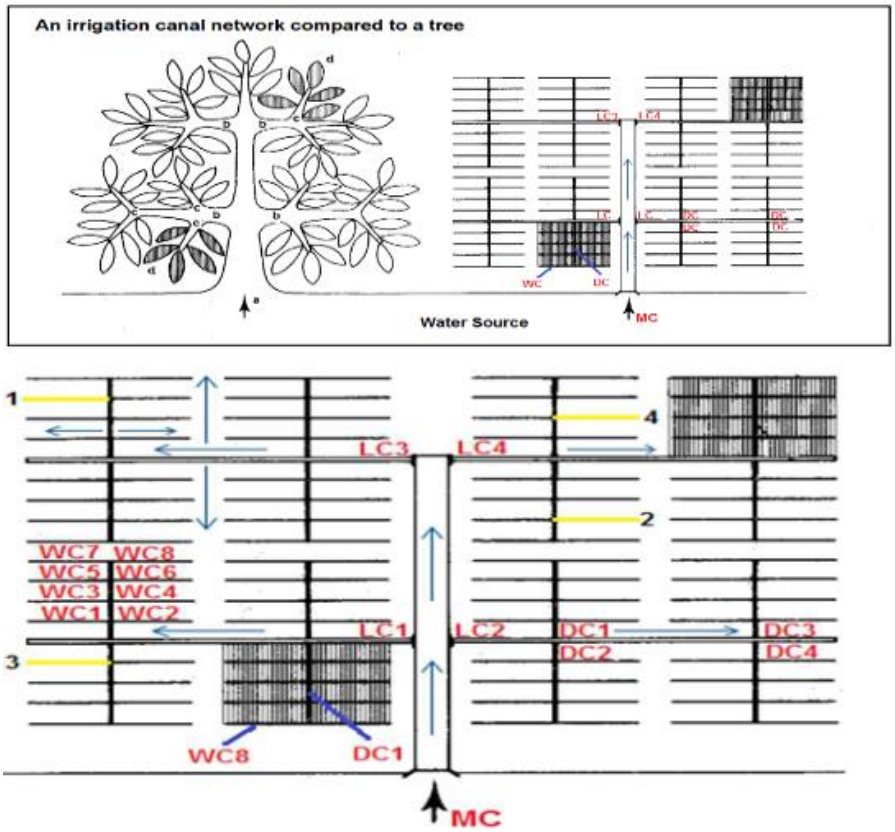
Classification of canals based on discharge:

1. Main Canal (MC): It is the largest canal in the system which takes off from a water source.

2. Lateral or branch canals (LC): the canals branches from MC to feed the distributaries canal.

3. Distributary canals (DC): Smaller canals take off from the branch canals and distribute their supply through outlets into water courses.

4. Water courses (WC): The smallest canals which feeds the water to the farm units.



3- ALIGNMENT OF IRRIGATION CANALS

Desirable locations for irrigation canals on any gravity project, their cross-sectional designs and construction costs are governed mainly by topographic and geologic conditions along different routes of the cultivable lands. Main canals must convey water to the higher elevations of the cultivable area. Branch canals and distributaries convey water to different parts of the irrigable areas.

On projects where land slopes are relatively flat and uniform, **it is advantageous to align channels on the watershed of the areas to be irrigated**. The natural limits of command of such irrigation channels would be the drainages on either side of the channel. Aligning a canal)main, branch as well as distributary) on the watershed ensures gravity irrigation on both sides of the canal. Besides, the drainage flows away from the watershed and, hence, no drainage can cross a canal aligned on the watershed. Thus, **a canal aligned on the watershed saves the cost of construction of cross-drainage structures.** However, the main canal has to be taken off from a river which is the lowest point in the cross-section, and this canal must mount the watershed in as short a distance as possible. Ground slope in the head reaches of a canal is much higher than the required canal bed slope and, hence, the canal needs only a short distance to mount the watershed. This can be illustrated by Fig.2 in which the main canal takes off from a river at P and mounts the watershed at Q. Let the canal bed level at P be 400 m and the elevation of the highest point N along the section MNP be 410 m. Assuming that the ground slope is 1 m per km, the distance of the point Q (395 m) on the watershed from N would be 15 km. If the required canal bed slope is 25 cm per km, the length PQ of the canal would be 20 km. Between P and Q, the canal would cross small streams and, hence, construction of cross-drainage structures would be necessary for this length. In fact, the alignment PQ is influenced considerably by the need of providing suitable locations for the cross-drainage structures. The exact location of Q would be determined by trial so that the alignment PQ results in an economic as well as efficient system. Further, on the watershed side of the canal PQ, the ground is higher than the ground on the valley side (i.e., the river side). Therefore, this part of the canal can irrigate only on one side (i.e., the river side) of the canal.

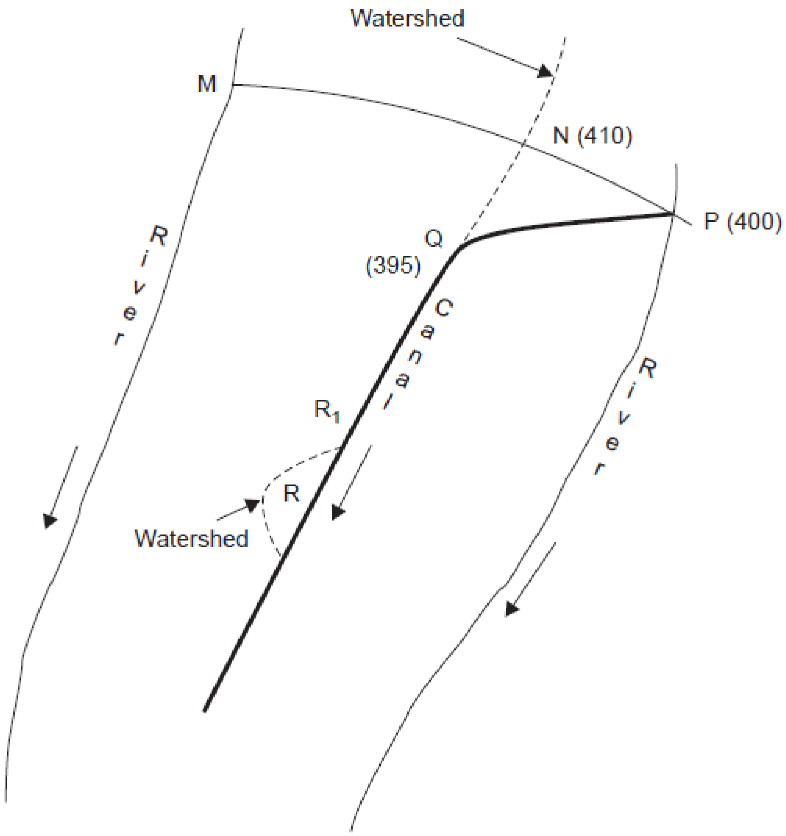


Fig. (.2) Head reach of a main canal in plains

Once a canal has reached the watershed, it is generally kept on the watershed, except in certain situations, such as the looping watershed at R in Fig.2. In an effort to keep the canal alignment straight, the canal may have to leave the watershed near R. The area between the canal and the watershed in the region R can be irrigated by a distributary which takes off at R1 and follows the watershed. Also, in the region R, the canal may cross some small streams and, hence, some cross-drainage structures may have to be constructed. If watershed is passing through villages or towns, the canal may have to leave the watershed for some distance.

In hilly areas, the conditions are vastly different compared to those of plains. Rivers flow in valleys well below the watershed or ridge, and it may not be economically feasible to take the channel on the watershed. In such situations, contour channels (Fig.3) are constructed. Contour channels follow a contour while maintaining the required longitudinal slope. It continues like this and as river slopes are much steeper than the required canal bed slope the canal encompasses more and more area between itself and the river. It should be noted that the more fertile areas in the hills are located at lower levels only.

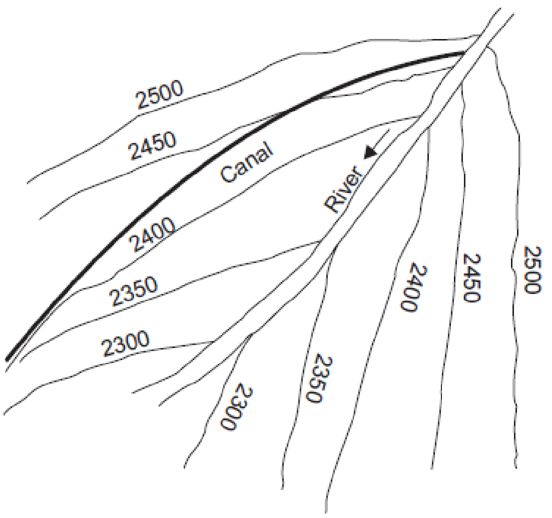


Fig.(.3) Alignment of main canal in hills

In order to finalise the channel network for a canal irrigation project, trial alignments of channels are marked on the map prepared during the detailed survey. A large-scale map is required to work out the details of individual channels. However, a small-scale map depicting the entire command of the irrigation project is also desirable. The alignments marked on the map are transferred on the field and adjusted wherever necessary. These adjustments are transferred on the map as well. The alignment on the field is marked by small masonry pillars at every 200 metres. The centre line on top of these pillars coincides with the exact alignment. In between the adjacent pillars, a small trench, excavated in the ground, marks the alignment.

4- CURVES IN CANALS

Because of economic and other considerations, the canal alignment does not remain straight all through the length of the canal, and curves or bends have to be provided. The curves cause disturbed flow conditions resulting in eddies or cross currents which increase the losses. In a curved channel portion, the water surface is not level in the transverse direction. There is a slight drop in the water surface at the inner edge of the curve and a slight rise at the outer edge of the curve. This results in slight increase in the velocity at the inner edge and slight decrease in the velocity at the outer edge. As a result of this, the low-velocity fluid particles near the bed move to the inner bank and the high-velocity fluid particles near the surface gradually cross to the outer bank. The cross currents tend to cause erosion along the outer bank. The changes in the velocity on account of cross currents depend on the approach flow condition and the characteristics of the curve. When separate curves follow in close succession, either in the same direction or in the reversed direction, the velocity changes become still more complicated.

Therefore, wherever possible, **curves in channels excavated through loose soil should be avoided. If it is unavoidable, the curves should have a long radius of curvature**. The permissible minimum radius of curvature for a channel curve depends on the type of channel, dimensions of cross-section, velocities during full-capacity operations, earth formation along channel alignment and dangers of erosion along the paths of curved channel. In general, the permissible

minimum radius of curvature is shorter for flumes or lined canals than earth canals, shorter for small cross-sections than for large cross-sections, shorter for low velocities than for high velocities, and shorter for tight soils than for loose soils. Table 1 indicates the values of minimum radii of channel curves for different channel capacities.

Table 1 Radius of curvature for channel curves

Channel capacity (m^3/s)	*Minimum radius of curvature (metres)*
Less than 0.3	100
0.3 to 3.0	150
0.3 to 15.0	300
15.0 to 30.0	600
30.0 to 85.0	900
More than 85	1500

5- CANAL LOSSES

When water comes in contact with an earthen surface, whether artificial or natural, the surface absorbs water. This absorbed water percolates deep into the ground and is the main cause of the loss of water carried by a canal. In addition, some canal water is also lost due to evaporation.

The loss due to evaporation is about 10 percent of the quantity lost due to seepage. The seepage loss varies with the type of the material through which the canal runs. Obviously, the loss is greater in coarse sand and gravel, less in loam, and still less in clay soil. If the canal carries silt-laden water, the pores of the soil are sealed in course of time and the canal seepage reduces with time. In almost all cases, the seepage loss constitutes an important factor which must be accounted for in determining the water requirements of a canal.

Between the headworks of a canal and the watercourses, the loss of water on account of seepage and evaporation is considerable. **This loss may be of the order of 20 to 50 percent of water diverted** at the headworks depending upon the type of soil through which canal runs and the climatic conditions of the region.

For the purpose of estimating the water requirements of a canal, the total loss due to evaporation and seepage, also known as conveyance loss, is expressed as m3/s per million square metres of either wetted perimeter or the exposed water surface area. Conveyance loss can be calculated using the values given in Table.2., the total loss (due to seepage and evaporation) per million square metres of water surface varies from 2.5 m3/s for ordinary clay loam to 5.0 m3/s for sandy loam. The following empirical relation has also been found to give comparable results .

$$q_l = (1/200)\,(B + h)^{2/3}$$

Table.2 Conveyance losses in canals

Material	*Loss in m^3/s per million square metres of wetted perimeter (or water surface)*
Impervious clay loam	0.88 to 1.24
Medium clay loam underlaid with hard pan at depth of not over 0.60 to 0.90 m below bed	1.24 to 1.76
Ordinary clay loam, silty soil or lava ash loam	1.76 to 2.65
Gravelly or sandy clay loam, cemented gravel, sand and clay	2.65 to 3.53
Sandy loam	3.53 to 5.29
Loose sand	5.29 to 6.17
Gravel sand	7.06 to 8.82
Porous gravel soil	8.82 to 10.58
Gravels	10.58 to 21.17

In this relation, *ql* is the loss expressed in m3/s per kilometre length of canal and *B* and *h* are, respectively, canal bed width and depth of flow in metres.

1- Canal drops:

Definition: Whenever the available natural ground slope is steeper than the designed bed slope of the channel, the difference is adjusted by constructing vertical 'falls' or 'drops' in the canal bed at suitable intervals, as shown in Fig..1. Such a drop in a natural canal bed will not be stable and, therefore, in order to retain this drop, a masonry structure is constructed. Such a pucca structure is called a canal fall or a canal drop.

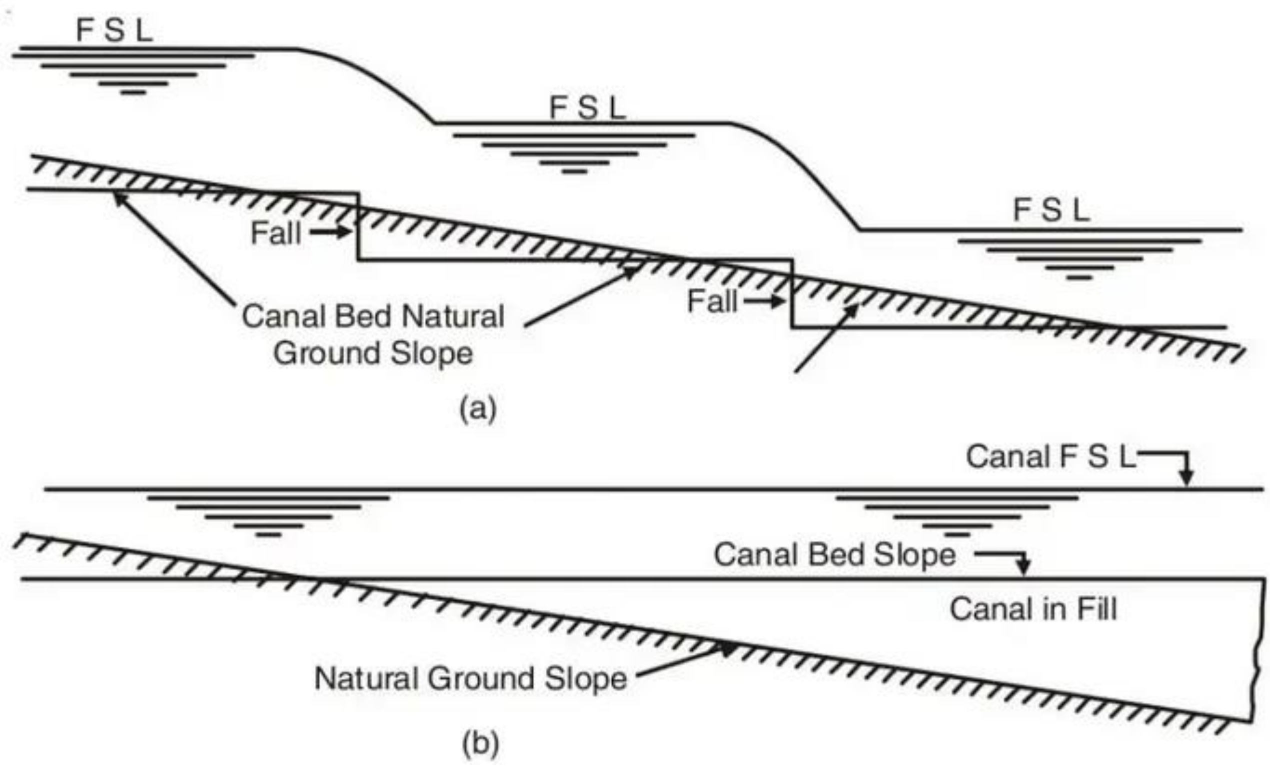


Fig.1(a) *Canal with falls* (b) *Canal without falls.*

Proper location: The location of a fall in a canal depends upon the topography of the country through which the canal is passing. In case of the main canal, which does not directly irrigate any area, the site of a fall is determined by considerations of economy in 'cost of excavation and filling' versus 'cost of fall'. The excavation and filling on two sides of a fall should be tried to be balanced, because the unbalanced earthwork is quite costly. An economy between these two factors has to be worked out before deciding the locations and extent of falls.

2- Types of drops:

Various types of falls have been designed and tried since the inception of the idea of 'falls construction' came into being. The important types of such falls, which were used in olden days and those which are being used in modern days, are described below

1- **Ogee Falls**: The 'Ogee type fall' was constructed in olden days on projects. The water was gradually led down. by providing convex and cincave curves , as shown in fig.2

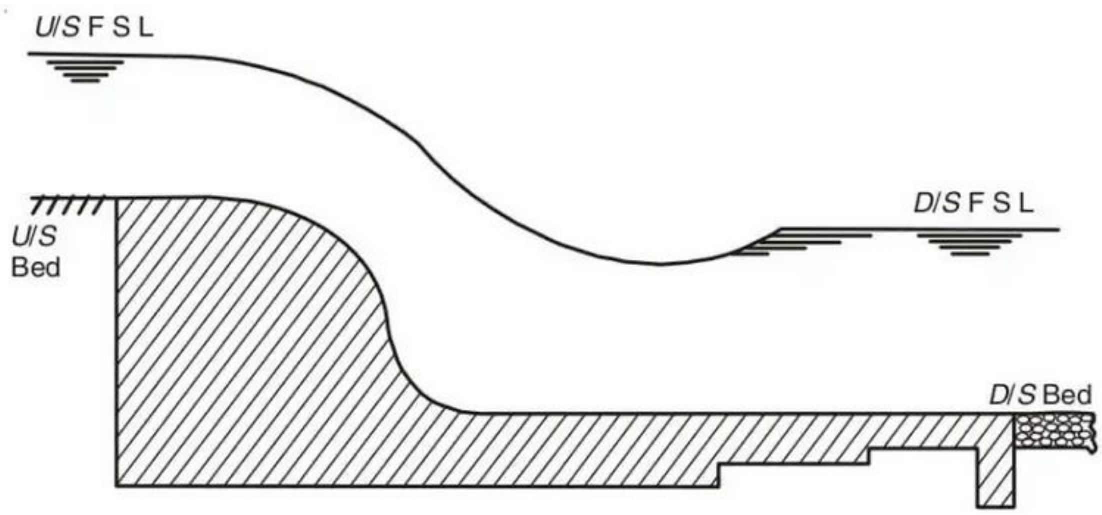
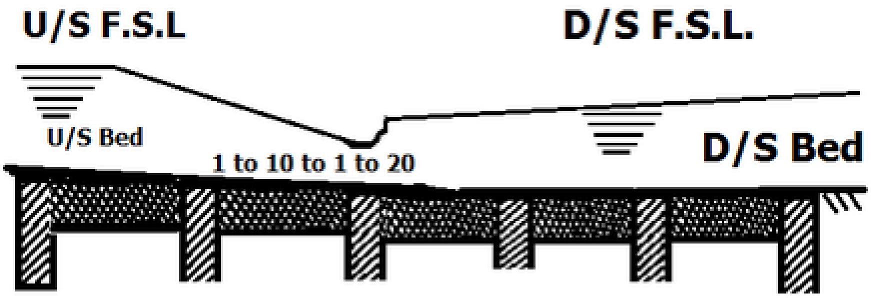


fig.2 Ogee Falls

2- **Rapids**: long rapids at slopes of 1 : 15 to 1 : 20.



Rapid Fall or Rapid

3- **Trapezoidal Notch Falls**. The trapezoidal notch fall was designed by Ried in 1894. It consists of a number of trapezoidal notches constructed in a high crested wall across the channel with a smooth entrance and a flat.circular lip projecting downstream from each notch to spread out the falling jet. Fig.3

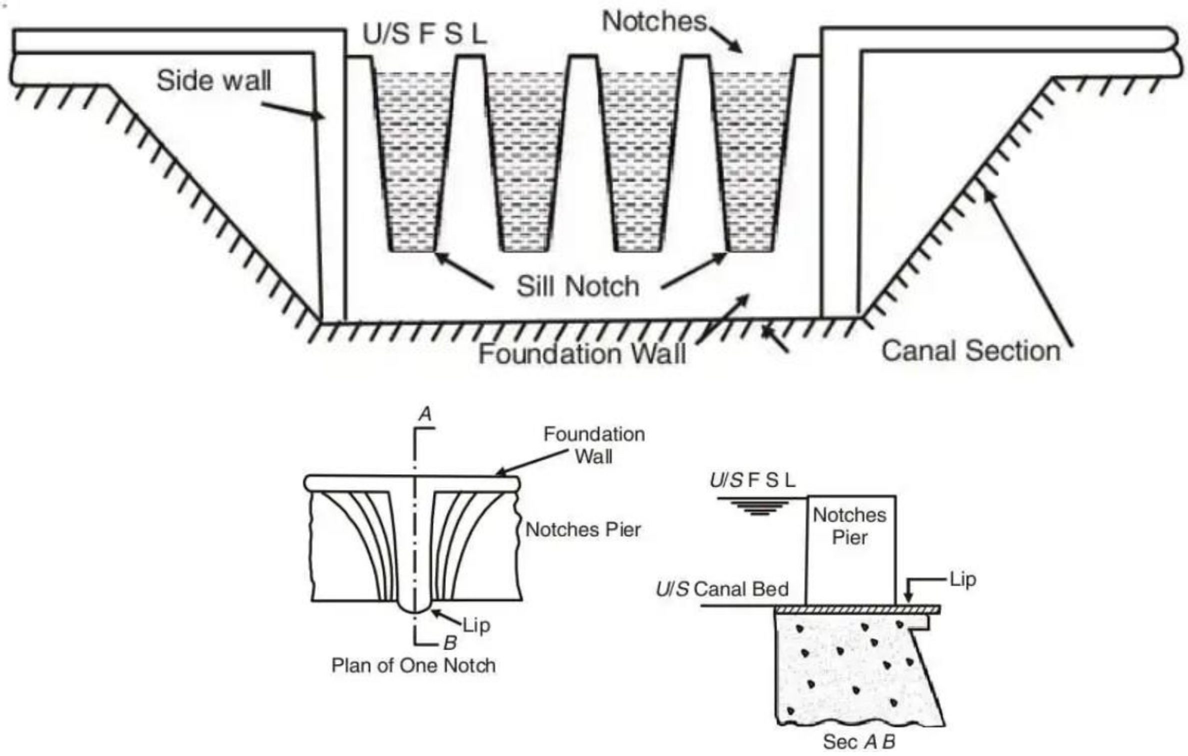


Fig.3 **Trapezoidal Notch Falls**

4- **Well Type Falls or Cylinder Falls, or Syphon Well Drops**. This type of a fall consists of an inlet well with a pipe at its bottom, carrying water from the inlet well to downstream well or a cistern. The downstream well is necessary in the case of falls greater than 1.8 m and for discharges greater than 0.29 cumecs. The waterfalls into the inlet well, through a trapezoidal notch constructed in the steining of the well, from where it emerges near the bottom, dissipating its energy in turbulence inside the well fig .4

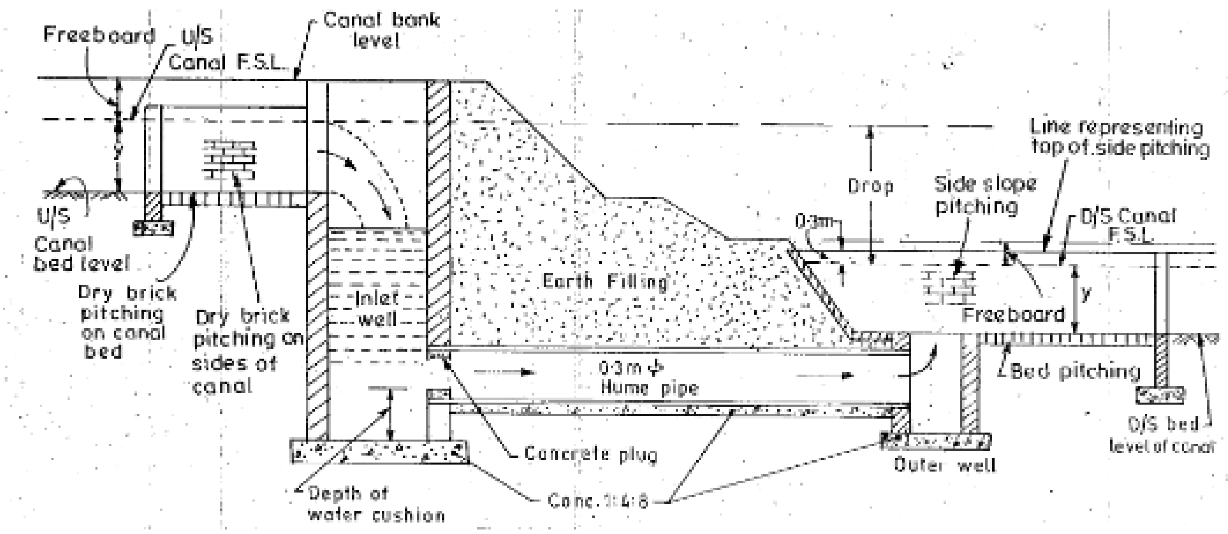


Fig.4 Syphon Well

5- **Simple Vertical Drop Type and Sarda Type Falls**. In the vertical drop type fall, the clear nappe leaving the crest makes to impinge into a cistern below. When the cistern provides a water cushion and helps to dissipate the surplus energy of the falling jet. (Fig. 5).

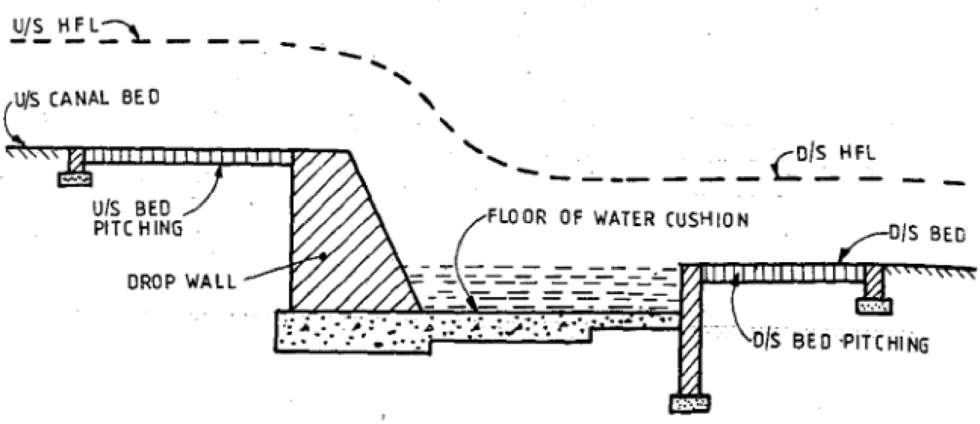


Fig.5 Simple Vertical Drop

6- **Straight Glacis Falls in this type of a modern fall**: a 'straight glacis' (generally sloping 2 : 1) .is provided after a 'raised crest' (see Fig .6). The hydraulic jump is made to occur on the glacis, causing sufficient energy dissipation. This type of falls gives very good performance if not flumed, although they may be flumed for economy. They are suitable for up to 60 cumecs discharge and 1.5 m drop.

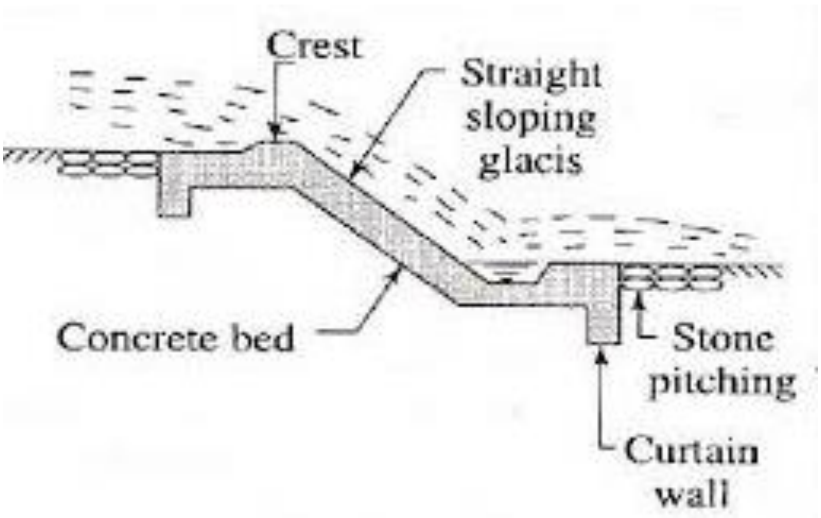


Fig .6 'Straight Glacis fall (without fuming), without Regulator and Bridge Details.

7- **Montague Type Falls**. The energy dissipation on a straight glacis remains incomplete due to vertical component of velocity remaining unaffected. An improvement in energy dissipation may be brought about in this type of fall [see Fig. .7 (a)], by replacing the straight glacis with a parabolic glacis', commonly known as 'Montague Profile'

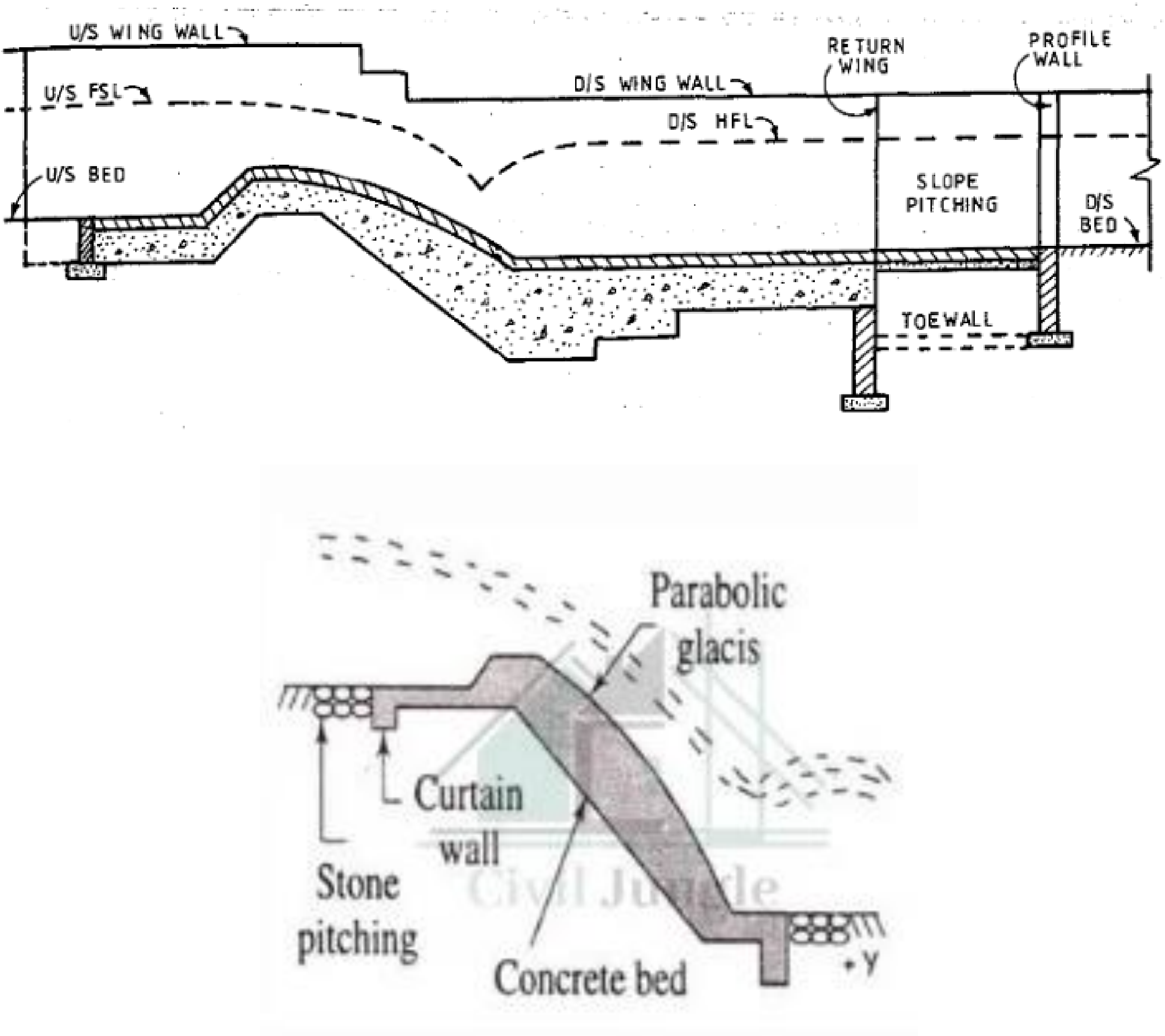


Fig.7. Montague Type fall.

8- **Inglis Falls or Baffle Falls:** A straight glacis type fall when added with a baffle platform and a baffle wall as shown in Fig. 12.8, was developed by Englis, and is called 'Englis Fall' or 'Baffle Fall'. They are quite suitable for all discharges and for drops of more than 1.5 m. They can be flumed easily as to affect economy. The baffle wall is provided at a calculated height and a calculated distance from the toe of the glacis, so as to ensure the formation of the . jump on the baffle platform, as shown in Fig.8

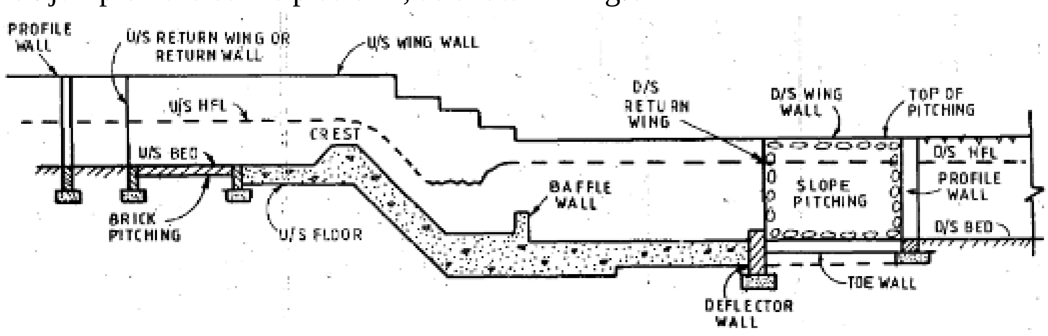
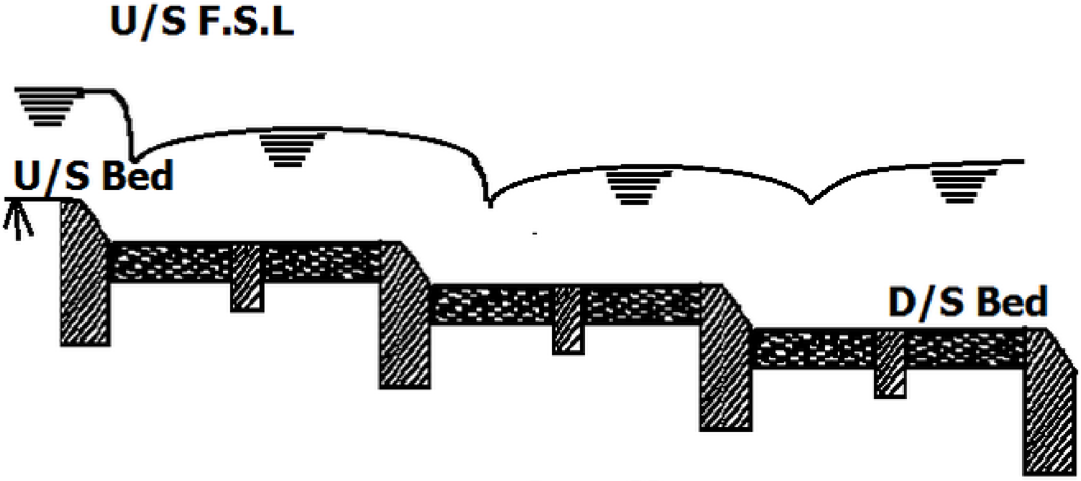


Fig.8 Inglis Falls or Baffle Falls

9- **Stepped Fall :** The stepped falls were the modified form of rapid falls in this respect that the long glacis of the rapid falls was replaced by floors in steps in the stepped falls. However, the cost of construction of the steeped falls was also very high.



Design principles of various types of falls

1- **Design of a Trapezoidal Notch Fall.**

As pointed out earlier, a notch fall provides a proportionate fall, in the sense that there is no heading up or drawdown of water level in the canal near the fall. The whole width of the channel is divided into several notches. The crest (i.e. the sill level or the level of the bottom of the notch) may be kept higher than the bed level of the canal, which will tend to increase the length of the weir, but in no case, the total length of the weir openings should exceed the bed "width of the canal upstream, and may well be reduced to about 7/8th of the bed width

Discharge Formula. The discharge passing through one notch of a notch fall can be obtained by adding the discharge of a rectangular notch and a V-notch

The discharge passing .through a trapezoidal notch such as shown in Fig..9 is given by

$$Q=\frac{2}{3}C_d\cdot\sqrt{2g}\;l\,.H^{3/2}+\frac{8}{15}\cdot C_d\cdot\sqrt{2g}\tan\frac{\alpha}{2}H^{5/2}$$

$$=\frac{2}{3}C_d\cdot\sqrt{2g}\left[lH^{3/2}+\frac{4}{5}\tan\frac{\alpha}{2}H^{5/2}\right]$$

$$=\frac{2}{3}C_d\cdot\sqrt{2g}\left[lH^{3/2}+\frac{2}{5}\left(2\tan\frac{\alpha}{2}\right)H^{5/2}\right]$$

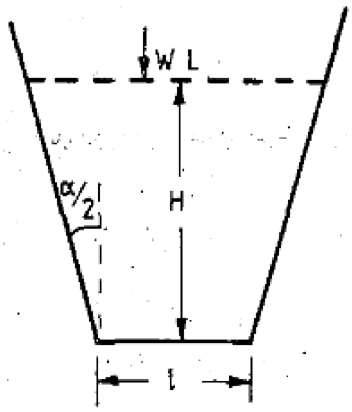


fig 9

If $2\tan\frac{\alpha}{2}$ is represented by n, then

$$Q = \frac{2}{3} C_d \cdot \sqrt{2g}\left[lH^{3/2} + 0.4 \, . \, nH^{5/2}\right]$$

where, C_d = Coefficient of discharge ≈ 0.75

$$Q = \frac{2}{3} \times 0.75\sqrt{2 \times 9.81}\left[lH^{3/2} + 0.4\, nH^{5/2}\right]$$

$$Q = 2.22H^{3/2}\left[l + 0.4nH\right]$$

The above discharge equation contains two unknowns l and n. For solving this equation,two values of Q and corresponding values of H must be assumed. It is a common practice to design notches for full supply discharge (Q100) and half supply discharge (Q50) with values of H equal to the normal water depths in the channel in the 'respective cases. Let the normal water depths in the channel at full discharge and half discharge be represented by Y100 andy50 respectively. Then H100 =Y100, and H50 = Y50·

The depth of water in the channel at 50% discharge (i.e. y50 can be approximately evaluated in terms of full supply depth (y100) as follows :

Let $V = C \cdot y^{0.64}$ ...(Kennedy's Eq. for Vel. in channels)

Now $Q = A.V.$

$\therefore$ $Q = B \, . \, y \, . \, C \, . \, y^{0.64}$ Using $A \approx B.y$ (neglecting sy^2)

or $Q = C \, . \, B \, . \, y^{1.64}$

$\therefore$ $Q_{100} = C \, . \, B \, . \, y_{100}^{1.64}$

and $Q_{50} = C \, . \, B \, . \, y_{50}^{1.64}$

or $$\frac{Q_{50}}{Q_{100}} = \left(\frac{y_{50}}{y_{100}}\right)^{1.64}$$

or $$\frac{y_{50}}{y_{100}} = \left(\frac{Q_{50}}{Q_{100}}\right)^{\frac{1}{1.64}} = (0.5)^{\frac{1}{1.64}} = 0.66$$

$\therefore$ $y_{50} = 0.66 \cdot y_{100}$

Number of Notches. The number. of notches should be so adjusted by the hit and trial method that the top width of the notch lies between to full water depth above the ill of the notch. This hit and trial procedure would become clear when we solve a numerical example.

Notch Piers. The thickness of notch piers should not be less than half the water depth and maybe kept more if they have to carry a heavy super structure. The top length of piers should not be less than their thickness. In plan, the notch profile is set back by 0.5 m from the downstream face of the notch . fall for larger canals, and by 0.25 m for distributaries. All curves are circular arcs, and all centers lie in the plane of the profile. The splay upstream from the notch section is 45°, and the downstream splay is kept at 22.5°. The lip is circular and is corbelled 'out by 0.8 m on larger canals, and by 0.6 m on distributaries.

Example 1. Design the size and number of notches required for a canal drop with the following particulars.

Full supply discharge =4cumecs

Bed width =6.0m

F.S. depth. =I.5m

Half supply depth =I.0m

Assume any other data if required.

Solution. The bed width of the canal is 6 m. Each potch at top should be roughly equal to F.S. depth i.e. 1.5 m. So let us, in the first trial, provide 3 notches.

Full supply discharge through e.ach notch=4/3= 1.33 cμmecs

$$Q = 2.22 H^{3/2} [l + 0.4\, nH]$$

Using $Q_{100} = 2.22\,(y_{100})^{3/2}\,[l + 0.4n\, y_{100}]$

where $Q_{100} = 1.33$ cumecs

$y_{100} = 1.5$ m

$\therefore$ We have $1.33 = 2.22\,.\,(1.5)^{3/2}\,[l + 0.4n \times 1.5]$

or $1.33 = 2.22 \times 1.84\,[l + 0.6n]$

or $l + 0.6n = 0.326$ ...(*i*)

Now, using

$$Q_{50} = 2.22\,.\,(y_{50})^{3/2}\,[l + 0.4n\,.\,y_{50}]$$

where $Q_{50} = \frac{1.33}{2} = 0.67$ cumecs

$y_{50} = 1.0$ m

$\therefore \quad 0.67 = 2.22 \,.\, (1.0)^{3/2} \,[l + 0.4n \times 1]$

or $\quad l + 0.4n = 0.3 \quad \ldots(ii)$

Subtracting (*ii*) from (*i*) we get

$$0.2n = 0.026$$

or $\quad n = 0.13.$

Putting the value of n in (*ii*) we get

$$l + 0.4 \times 0.13 = 0.3$$

or $\quad l = 0.248$; say **0.25 m.**

By this trial, we find the top width

$$= 0.25 + 2 \tan\alpha \cdot H = 0.25 + n.\,H$$
$$= 0.25 + 0.13 \times 1.5 = 0.25 + 0.195$$

$= 0.445$; say 0.45 m, which is much less than the full depth of 1.5 m.

To increase the top width, and to make it near 1 to 3/4th FSD, it is necessary to increase l and n which can be done by reducing the number of notches. The values of l and n obtained for 3 notches will increase in direct proportion, when number of notches are reduced. In other words, the values l of and n will become 3 times, when number of notches are reduced 3 times. Thus, when we provide only one notch instead of 3 notches, the values of n and l will triple. Similarly, when we use 2 notches against 3, i.e., 1/5 times the values n and l will become 1.5 times of those obtained for 3 notches.

Hence when we use 2 notches, values will be:

$$n = 1.5 \times 0.13 = 0.20$$
$$l = 1.5 \times 0.25 = 0.3 \text{ m}$$

and $\quad \text{Top width} = 1.5 \times 0.45 = \mathbf{0.68\ m.}$

Since the width is still quite low, we may use only one notch .

When we use only one notch, the values will be:

$$n = 3 \times 0.13 = 0.39$$
$$l = 3 \times 0.25 = 0.75 \text{ m}$$
$$\text{Top width} = 3 \times 0.45 = 1.35 \approx \text{FSD (O.K.)}$$
$$\frac{\alpha}{2} = \tan^{-1}\frac{n}{2} = \tan^{-1}\frac{0.39}{2} = 11°$$

Since this condition gives us top width = 1.35 m, which is O.K., we may pro-vide one notch, centrally placed in the given channel of 6 m width. The section of the. notch to be adopted is also shown in Fig.10

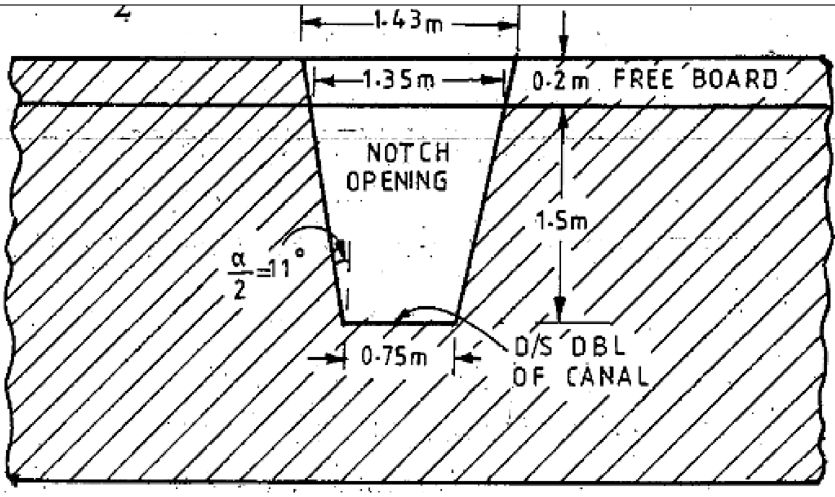


Fig 10

Check for raised crest if possible. It has also been noticed that when lesser number of notches are Provided, with their. bottoms kept at U/S DBL of canal the concentration of flow gets increased considerably.to avoid such an eventuality,its preferable to increase the number of notches, and this may sometimes be. achieved by providing the notches in the raised crest. In other words, the bottom of notch opening will be kept higher than U/S DBL of canal. This raising may be between 10% to 30% of full depth. The design calculations are hence to be repeated to compute n and l with a raised crest, whenever a detailed designing is being done, and number of notches determined are low.

These calculations for the above question will be as follows :

Let us assume a raised crest equal to 20% of FSD

$$= 20\% \times 1.5 \text{ m} = 0.3 \text{ m}.$$

$$Q = 2.22\, H^{3/2}\, [l + 0.4nH], \text{ we have}$$

$$Q_{100} = 2.22\,(1.5 - 0.3)^{3/2}\,[l + 0.4n\,(1.5 - 0.3)]$$

or $$Q_{100} = 2.92\,(l + 0.48n) \quad \text{...(iii)}$$

Also $$Q_{50} = 2.22\,(1.0 - 0.3)^{3/2}\,[l + 0.4n\,(1.0 - 0.3)]$$

$$[\because \text{FSD at } \tfrac{1}{2} \text{ discharge} = 1.0 \text{ m (given)}]$$

or $$Q_{50} = 1.3\,(l + 0.28\,n) \quad \text{...(iv)}$$

But $$Q_{100} = 2Q_{50}$$

$\therefore$ $$2.92\,(l + 0.48\,n) = 2 \times 1.3\,(l + 0.28\,n)$$

This gives a negative value of n, which is not feasible, and hence such a raised crest may not be feasible in this particular case. Hence, the design made earlier, and shown in Fig. 10, holds good.

2- Design of a Syphon Well Drop

A syphon well drop, such as shown in Fig.4, is generally adopted for smaller discharges and larger drops. The main features of the design involve determining the size of the inlet well and that of the pipe. Suitable size for the outer well, a proper provision of water cushion at the bottom of the inlet well, the bed and side slope pitchings in the canal upstream as well as downstream for suitable lengths, are also provided. The size of the inlet well and that of the syphon pipe are determined on the following considerations w .r. to Fig. 11.

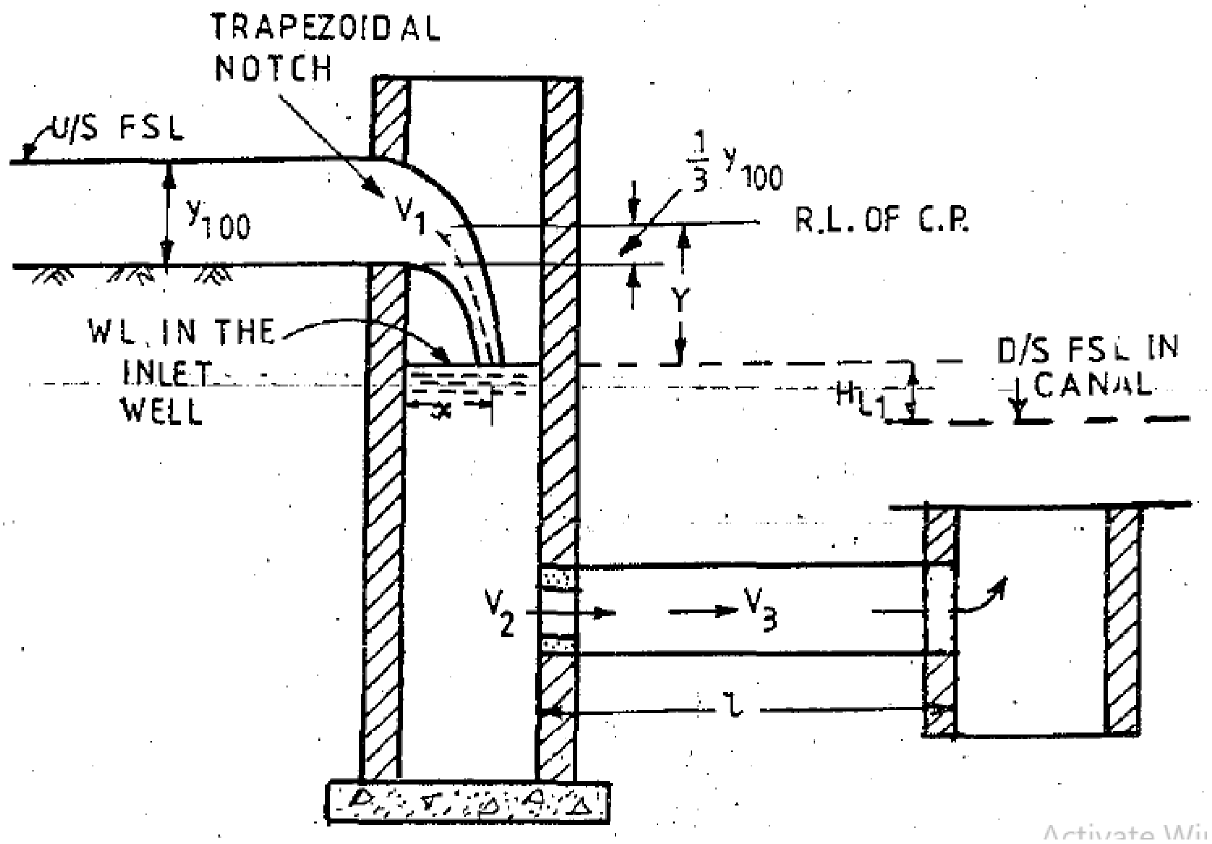


Fig.11

First of all, the size of the trapezoidal notch is determined to pass the designed discharge by using eq. (12.4) in the same way, as is done for a trapezoidal notch. Then let V1 be the velocity over the notch, V2 be the velocity of entry in the pipe, and V3 be the velocity through the pipe. All these values of velocities can be determined easily as below :

$$V_1 = \frac{\text{Full supply discharge}}{\text{Area of flow over the notch}}$$

$$V_2 = \frac{\text{Full supply discharge}}{\text{Area of opening at entry (for assumed dia of opening)}}$$

$$V_3 = \frac{\text{Full supply discharge}}{\text{Area of pipe (for assumed dia)}}$$

The head loss between the inlet well and the d/s FSL is then given by HL_1 as

$$H_{L_1} = 0.5\frac{V_2^2}{2g}\text{ (i.e. loss due to entry)} + \frac{(V_2 - V_3)^2}{2g}\text{ (i.e. the loss due to sudden enlargement)} + \frac{f' L V_3^2}{2gd}\text{ (i.e. the loss in the assumed pipe length } L\text{)}$$

$$+\frac{V_3^2}{2g}\text{ (loss due to exit).}$$

Knowing all the above values, HL_1 can be determined, and thus the R.L. of water surface inlet well *(i.e. d/s* FSL + *HL)* can be determined.

Now, approximate *R.L.* of the centre of pressure (C.P,) of the trapezoidal waterway through the notch

$$= \text{u/s canal bed level} + \frac{1}{3}\text{FSD.}$$

$$= \text{(which can be calculated)}$$

Then, the height *(Y)* of the centre of pressure above the water level in the inlet well

$$= \text{R.L. of C.P.} - \text{R.L. of water level in inlet well}$$

$$= \text{(Known)}$$

Now using the eq.

$$V_1 = \sqrt{\frac{gX^2}{2.Y}}$$

where *X* and *Y* are the coordinates· of the jet· (issuing from centre of pressure) . t. the water surface level in the inlet well *as fig.12*

$$X = V_1\,t \text{ (after a time } t\text{)}$$

$$Y = \frac{1}{2}gt^2$$

or

$$Y = \frac{1}{2}g\cdot\left(\frac{X}{V_1}\right)^2$$

or

$$V_1 = \sqrt{\frac{gX^2}{2y}}$$

$$\left[\because \text{ using } S = ut + \frac{1}{2}gt^2 \text{ and } u = 0\text{, we have } S = \frac{1}{2}gt^2\right]$$

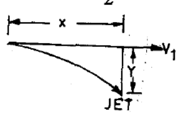


Fig .12

The value of X can be determined. Finally, the dia of the inlet well may be kept at about 1.5 times the value of X. The entire procedure will become more clear when we solve. the following numerical example.

Example .2. Design the salient dimensions of a syphon well drop for the following particulars :
Fall =3.8m , General ground level = + 163.36 m , Full supply depth = 75 cm , Bed level upstream = + 162.83 , Discharge = 1 cumec , Bed width upstream and downstream = 2.4 m

Solution. For a trapezoidal notch, we have the discharge eq. as

$Q = 2.22 \cdot H^{3/2}\,[l + 0.4\,n\,H]$

At full supply discharge, we have

$Q_{100} = 2.22\,(y_{100})^{3/2}\,[l + 0.4\,n\,Y_{100}]$

where y_{100} = F.S.D. = 0.75 m , Q_{100}= F.S.Q= 1 cumec

1 = 2.22 $(0.75)^{3/2}$ [l + *0.4n* (0.75)]

0.71=l+0.3n ……(i)

At 50% full discharge, we have

$Q_{50} = 2.22\, y_{50}\, [l + 0.4n\, y_{50}]$

where y_{50}= 0.66 y_{100}

=0.66x0.75

=0.5 m

Q_{50} = 0.5 cumec

0.5 = 2.22 (0.5)312 [*l* + 0.4n (0.5)]

0.64= *l* + 0.2 n ….(ii)

Subtracting (ii) from (i) we get

0.07=0.l n

n=0.7

$$2 \tan \frac{\alpha}{2} = 0.7, \text{ or } \frac{\alpha}{2} = 19.3°$$

Substituting this value of n in (ii), we get

l=0.64-0.14=0.50

Hence, provide a trapezoidal notch in the staining of the inlet well, with 0.5 m bottom width and each side inclined to an angle of 19.3° with the vertical.

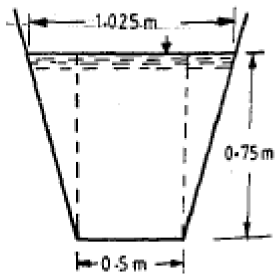
Now, the width of water (at FSL) flowing over notch

= 0.5 + 0.7 x (0.75) = 0.5 + 0.525 = 1.025 m.

Velocity (V_1) over the notch

$$= \frac{\text{F.S.Q.}}{\text{Area of flow over the notch}}$$

$$= \frac{1}{\frac{0.5+1.025}{2} \times 0.75} \text{ m/sec} = \frac{1}{0.76 \times 0.75} \text{ m/sec} = 1.75 \text{ m/sec}$$



Let us now assume that the diameter of the pipe used to be 1 m

fig .13

Velocity V_3 through the pipe

$$= \frac{1}{\frac{\pi}{4}(1)^2} \text{ m/sec.} = 1.27 \text{ m/sec.}$$

Let us assume that the diameter of the opening at the inlet of pipe be 0.5 m

The velocity of entry into the pipe (V2)

$$= \frac{1}{\frac{\pi}{4}(0.5)^2} \text{ m/s} = 5.1 \text{ m/sec.}$$

Loss of head between the inlet well and the dis FSL is given by Eq.

$$= 0.5 \cdot \frac{V_2^2}{2g} + \frac{(V_2 - V_3)^2}{2g} + \frac{f' L V_3^2}{2gd} + \frac{V_3^2}{2g}$$

Let us assume that the length of the pipe is kept as 12m and f' =Darcey's coefficient of friction be taken as equal to 0.012, we than have

$$H_{L_1} = 0.5 \times \frac{(5.1)^2}{2 \times 9.81} + \frac{(5.10-1.27)^2}{2 \times 9.81} + \frac{0.012 \times 12 \times (1.27)^2}{2 \times 9.81 \times 1.0} + \frac{(1.27)^2}{2 \times 9.81}$$

$$= 0.66 + 0.77 + 0.01 + 0.08 = \mathbf{1.52\ m.}$$

R.L. of the water surface in the inlet well

$$= \text{d/s FSL} + 1.52$$

$$\left[\begin{aligned} \text{d/s FSL} &= \text{u/s FSL} - \text{fall} \\ &= (162.83 + 0.75) - 3.8 = 159.78 \end{aligned}\right]$$

$$= 159.78 + 1.52 = \mathbf{161.30}.$$

Approximate R.L. of the centre of pressure (C.P.) of the trapezoidal waterway through a notch

$$= \text{u/s canal bed level} + \frac{1}{3}\text{FSD}$$

$$= 162.83 + \frac{0.75}{3} = 162.83 + 0.25 = \mathbf{163.08}$$

Height Y of C.P .. above the water level in the inlet well
= 163.08-161.30= 1.78 m.

$$V_1 = \sqrt{\frac{g \cdot X^2}{2Y}}$$

or $$X = \sqrt{\frac{V_1^2\, 2Y}{g}}$$

$$= \sqrt{\frac{(1.75)^2 \times 2 \times 1.78}{9.81}} = 1.75 \times 0.6 = 1.05\ \text{m.}$$

Now, the dia. of the inlet well may be kept at about 1.5 X, i.e. l.5 x 1.05 = 1.575 m, say 1.6 m. Keep the dia. of the d/s outlet well, as say 1.2 m. Also, provide a water cushion at the bottom of the inlet well. Bed and sides of the channel for suitable lengths on the u/s as well as d/s side are protected by dry brick pitching. The complete details are shown in Fig.14.

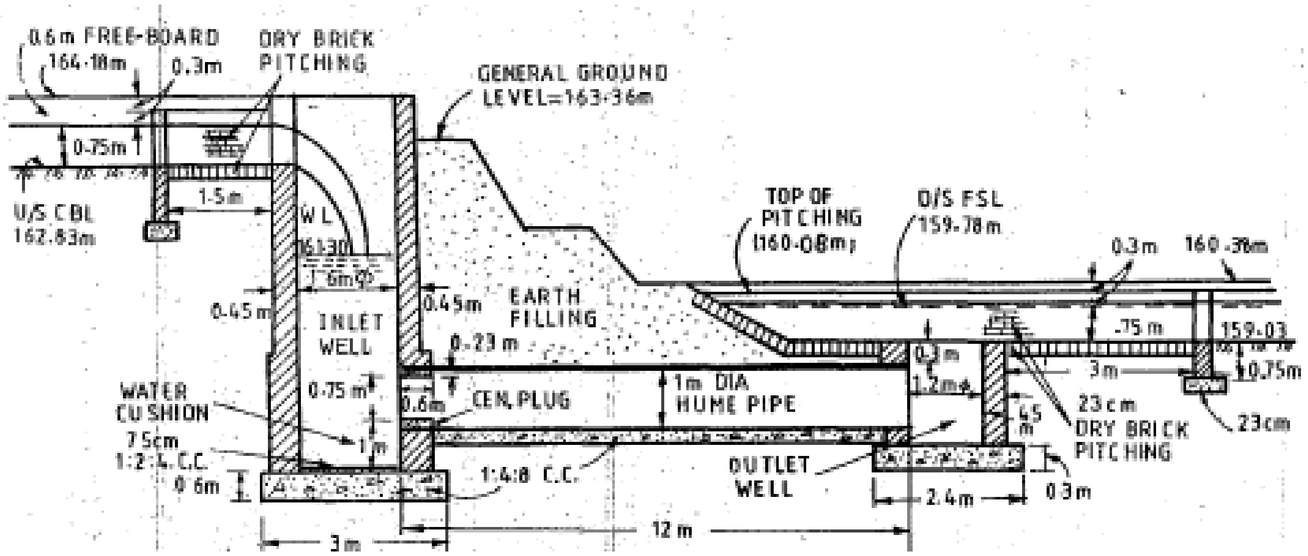


Fig.14

3- Design of Simple Vertical drop Fall

In a vertical drop fall, the energy of the flowing water is dissipated by means of impact and by sudden deflection of velocity from the vertical to the horizontal direction. A water cushion is provided at the toe of the drop, so as to reduce the impact of falling jet and thus to save the downstream floor from scour. The water cushion is formed by depressing the floor below the downstream bed of the canal, as shown in Fig 1

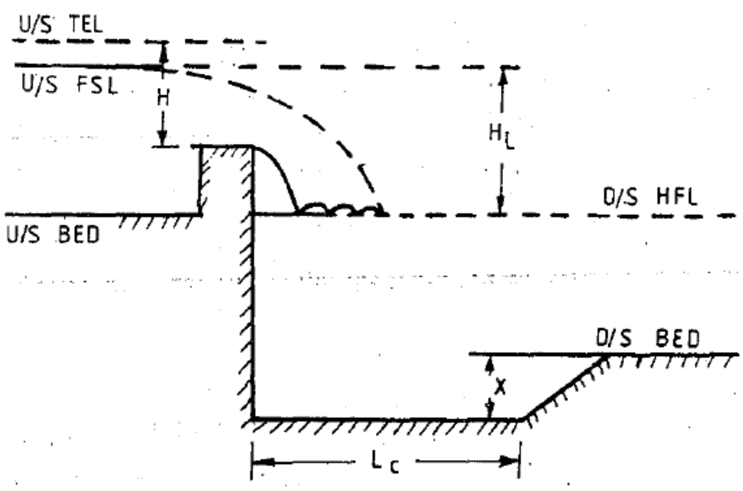


Fig. 1

The following dimensions for the cistern have been suggested by U.P. Irrigation Research Institute:

$$L_C = 5 \cdot \sqrt{H \,.\, H_L}$$

$$X = \frac{1}{4} \cdot (H \,.\, H_L)^{2/3}$$ ……………..eq.1&2

where Lc= The length of the cistern in metres.

X = Cistern depression below the downstream bed in metres.

H= Head of water over the crest, including velocity head, in metres, i.e. = (u/s TEL - Crest level).

4- Design of a Sarda Type Fall

The design criteria for various components of such a fall, based on. the recommendations of Bahadarabad Research Station, are given below :

Length of the Crest: Since fluming is not permissible in this type of falls, the length of the crest is kept equal to the bed width of the canal. Sometimes, for future expansion, the crest length may be kept equal to (bed width +depth).

Shape of the Crest: A rectangular crest with both faces vertical has been suggested for discharges under 14 cumecs . the top width is kept equal to $0.55\sqrt{d}$ and the minimum base width is kept equal to $\frac{h+d}{G}$ (Take $G = 2$ for masonry) where d is the height of the crest above the downstream bed level and h is the head over the crest [see fig.2-a]

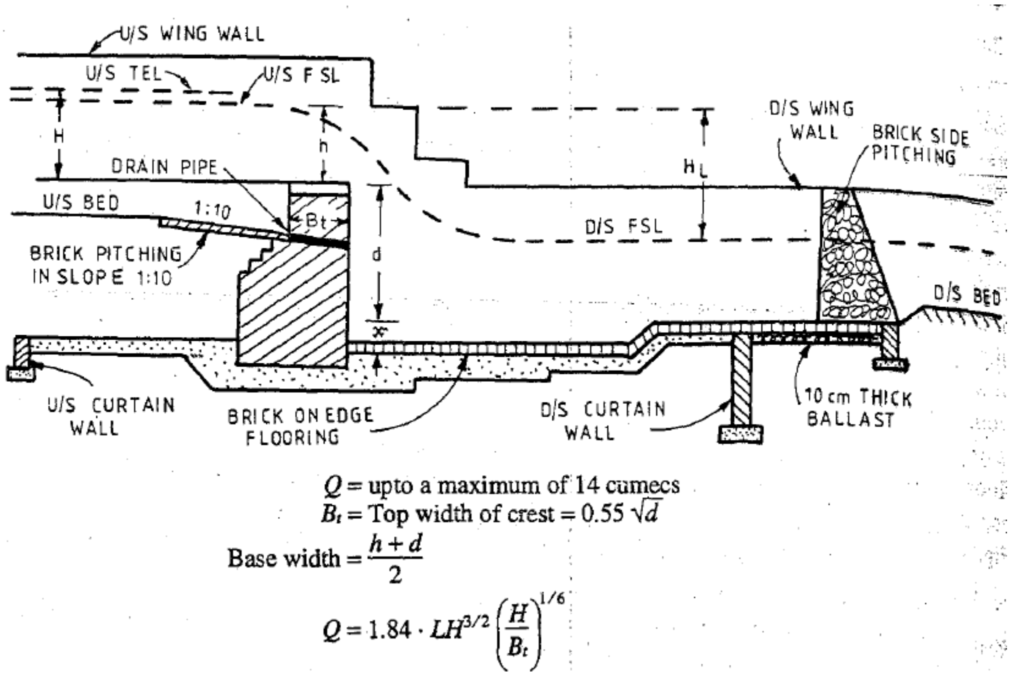


Fig.2-a: Rectangular Crest for Sarda Type fall

For discharges over 14 cumecs. a trapezoidal crest with top width equal to $0.55\sqrt{H+d}$ with upstream side slope of 1:3 and downstream side slope of 1:8 is adopted [see fig.2-b]

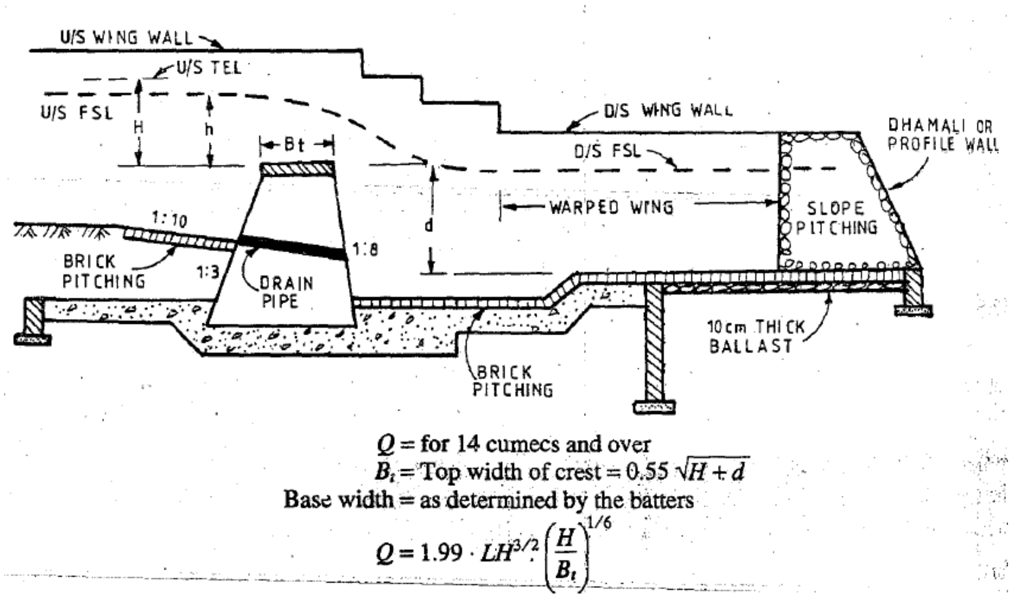


Fig.2-b: Trapezoidal crest for Sarda Type fall

Crest level: The. following discharge formula is used to determine the height of the crest .

$$Q = C_d \,.\, \sqrt{2g} \,.\, L \,.\, H^{3/2}\left(\frac{H}{B_t}\right)^{1/6}$$

where C_d= 0.415 for rectangular crest ,

= 0.45 for trapezoidal crest

L = Length of the crest

B_t = Top width of crest

Height of the crest above bed = y – h

≈ y-H (assuming h≈ H i.e. neglecting velocity of approach)

where y is the normal depth of channel(upstream).

Upstream Wing Wall. For trapezoidal crest, the upstream wing walls are kept segmental with radius equal to 5 to 6 times H and subtending an angle of 60° at centre, and then carried tangential into the berm as shown fig.3 The foundations of the wing walls are laid on the impervious concrete floor itself. For rectangular crest (i.e. discharge Jess than 14 cumecs), the approach wings may be splayed straight at an angle of 45°.

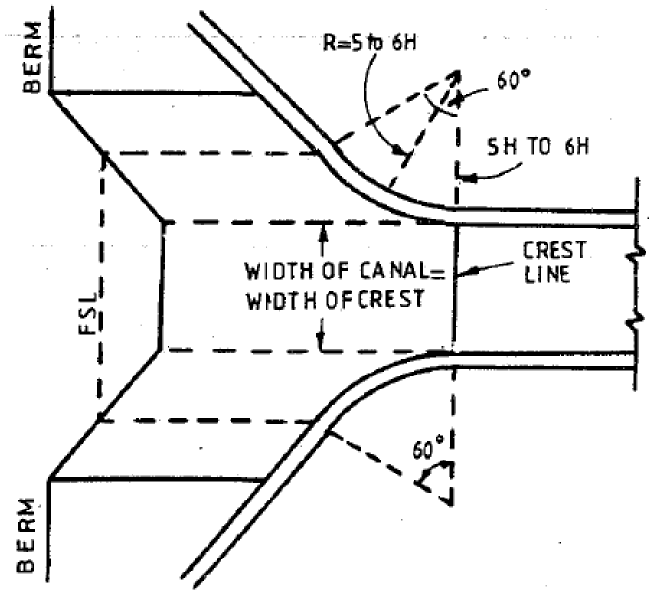


Fig.3 : Upstream wing walls for Trapezoidal crest of Sarda Type fall

Upstream Protection: Brick pitching in a length equal to upstream water depth may be laid on the upstream bed, sloping towards the crest at a slope of 1 : 10. Drain pipes should also be provided at the u/s bed level in-the cresr so as to drain out the·u/s bed during the closer of the channel.

Upstream Curtain Wall: $1\frac{1}{2}$ brick thick upstream curtain wall is provided, having

a depth equal to $\frac{1}{3}^{rd}$ of water depth.

Impervious Concrete Floor. The total length of impervious floor can be. Determined by Bligh's theory for small works and by Khosla's theory for large works. The minimum length of floor on d/s of the. toe of the crest wall should be = [2(water depth + 1.2 m) + drop]. The balance can be provided under the crest and on upstream.

The floor thickness required on the downstream side can be worked out for uplift pressures (using minimum thickness of 0.4 m to 0.6 metre) and only a nominal thickness of 0.3 metre is provided on the upstream side. The maximum seepage head will occur when water is stored upto top of crest on u/s side and there is no flow on the downstream side.

Cistern. The length and depth of cistern can be worked out from equations 1&2

Downstream Protection. The dis bed may be protected with dry brick pitching, about 20 cm thick resting on 10 cm thick ballast. The length of the d/s pitching is given by the values of Table.1; or 3 times the depth of downstream water, whichever is more. The pitching may be provided between two or three curtain walls. The curtain walls may be $1\frac{1}{2}$ brick thick and of depth equal to $\frac{1}{2}$ the downstream depth; or as given in Table .1 (minimum= 0.5 m).

Table.1

Head over the crest H (metres)	*Total length of d/s pitching (metres)*	*Remarks*	*Curtain walls*	
			No.	*Depth in metres*
Upto 0.3 m	3.0	All sloping down at 1 in 10	1	0.30
0.3 to 0.45	3.0 + Twice H_L	Horizontal up to end of masonry wings and then sloping down at 1 : 10	1	0.30
0.45 to 0.60	4.5 + Twice H_L		1	0.45
0.60 to 0.75	6.0 + Twice H_L		1	0.60
0.75 to 0.90	9.0 + Twice H_L	"	1	0.75
0.90 to 1.05	13.5 + Twice H_L	"	2	0.90
1.05 to 1.20	18.0 + Twice H_L	"	2	1.05
1.20 to 1.50	22.5 + Twice H_L	"	3	1.35

Slope Pitching. After the return wing, the sides of the channel are pitched with one brick on edge. The pitching should rest on a toe wall $1\frac{1}{2}$ brick thick and of depth equal to half the downstream water depth. The side pitching may be curtailed at an angle of 45 ° from the end of the bed pitching, or extended straight from the end of the bed pitching.

Downstream Wings. Downstream wings are kept straight for a length of 5 to 8 time $\sqrt{H \cdot H_L}$ and may then be gradually wrapped. They should be taken upto. the end of the pucca floor.

All wing walls must be designed as retaining walls, subjected to full pressure of submerged soil at their back when the channel is closed. Such a wall generally has base width equal to $\frac{1}{3}$rd its height.

Example. *Design a 1.5 metres Sarda type fall for a canal having a discharge of 12 cumecs, with the following data:*

Bed level upstream = 103.0 m, Side slopes of channel = 1 : 1 m, Bed level downstream= 101.5 m, Full supply level upstream= 104.5 m, Bed width u/s and d/s= 1.0 m, Soil= Good loam, Assume Bligh's Coefficient=6.

Solution:

Length of crest. Same as d/s bed width = 10 m

Crest level. A rectangular crest is provided, since the discharge is less than 14cumecs. The discharge formula is given by

$$Q = 1.84 \, . \, L \, . \, H^{3/2} \left[\frac{H}{B_t} \right]^{1/6}$$

Assume top width of the crest as 0.8 m.

$$12 = 1.84 \times 10 \times H^{3/2} \times \frac{H^{1/6}}{(0.8)^{1/6}}$$

$$H^{5/3} = \frac{12 \times 0.964}{1.84 \times 10} = 0.628$$

$$H = (0.628)^{3/5} = 0.755 \text{ m} \; ; \text{ Say } \mathbf{H = 0.76 \, m.}$$

Velocity of approach:

$$= V_a = \frac{\text{Discharge}}{\text{Area}} = \frac{12}{(10 + 1.5)\, 1.5} \qquad (\because \text{Depth of water } = 1.5 \text{ m})$$

$$= \frac{12}{11.5 \times 1.5} = 0.696 \text{ m/sec.}$$

$$\text{Velocity head} \quad = \frac{V_a^2}{2g} = 0.025 \text{ m.}$$

$$\text{u/s TEL} = \text{u/s FSL} + \text{Velocity Head}$$
$$= 104.5 + 0.025 = 104.525 \text{ m}$$

R.L. of the crest

$$= (\text{u/s TEL} - H)$$
$$= 104.525 - 0.755 = \mathbf{103.77 \, m.}$$

Use crest level of 103.77 metres

Height of the crest above d/s floor

$$= 103.77 - 103.0 = \mathbf{0.77 \, m.}$$

Shape of the crest.

Width of the crest (B_t) $= 0.55 \, . \, \sqrt{d}$

where d =Height of the crest above d/s bed

$$= 103.77 - 101.5 = 2.27 \text{ m}$$
$$B_t = 0.55 \cdot \sqrt{d} = 0.55 \cdot \sqrt{2.27} = \mathbf{0.825 \, m.}$$

Keep 0.85 m width of the crest

$$\text{Thickness at base} = \frac{h+d}{2}$$

$$= \frac{(0.755-0.025)+2.27}{2} = \frac{0.73+2.27}{2} = 1.5\text{ m.}$$

The top shall be capped with 20 cm thick C.C. 1 : 2: 4

Upstream wing. It shall be splayed straight at an angle of 45° from the u/s edge of the crest and shall be embedded by 1.0m into the berm. On the d/s side, wing walls are kept straight and parallel up to the end of floor and joined to return walls, as shown in fig .4

Upstream protection 1.5m long brick pitching (equal to u/s water depth) is laid on the u/s bed, sloping down towards the crest at 1:10, and three drain pipes of 15cm diameter at the u/s bed level should be provided in the crest so as to drain out the u/s bed during the closure of the canal.

Upstream curtain wall. Maximum depth of u/s curtain wall.

$$= \frac{y_u}{3} = \frac{1.5}{3} = 0.5\text{ m.}$$

Provide 0.4 m x 0.8 m deep curtain wall on the u/s.

Cistern. Depth of cistern, is given by:

$$X = \frac{1}{4}[H \cdot H_L)^{2/3} = \frac{1}{4}[0.76 \times 1.5]^{2/3} = \frac{1}{4} \times (1.14)^{0.667}$$

$$= \frac{1}{4} \times 1.091 = 0.273\text{ m ; } \textbf{Say 0.3 m deep.}$$

$$\therefore \text{ R.L. of cistern} = 101.5 - 0.3 = \mathbf{101.2\ m.}$$

$$\text{Length of cistern} = 5\sqrt{H \cdot H_L}$$

$$= 5 \times \sqrt{0.76 \times 1.5} = 5 \times \sqrt{1.14} = 5.34\text{ m ; say } \mathbf{5.5\ m.}$$

Provide 5.5 m long cistern at R.L. 101.2 m.

Impervious floor.

Maximum Static Head

$$= (\text{Crest level} - \text{d/s bed level})$$

$$= 103.77 - 101.5 = 2.27\text{ m.}$$

Total floor length required

$$= C.H.\text{; where } C \text{ is Bligh's coefficient}$$

$$= 6 \times 2.27 = 13.62\text{ m. ; say } \mathbf{13.7\ m.}$$

Minimum d/s floor length required

$$= [2\,(\text{Water depth } + 1.2) + H_L)$$
$$= 2\,(1.5 + 1.2) + 1.5 = 2\,(2.7) + 1.5 = 5.4 + 1.5 = 6.9 \text{ m} \text{ ; say } \mathbf{7\ m.}$$

Provide 7m d/s floor and the balance 6.7m under and upstream of crest ,as shown in fig 5.

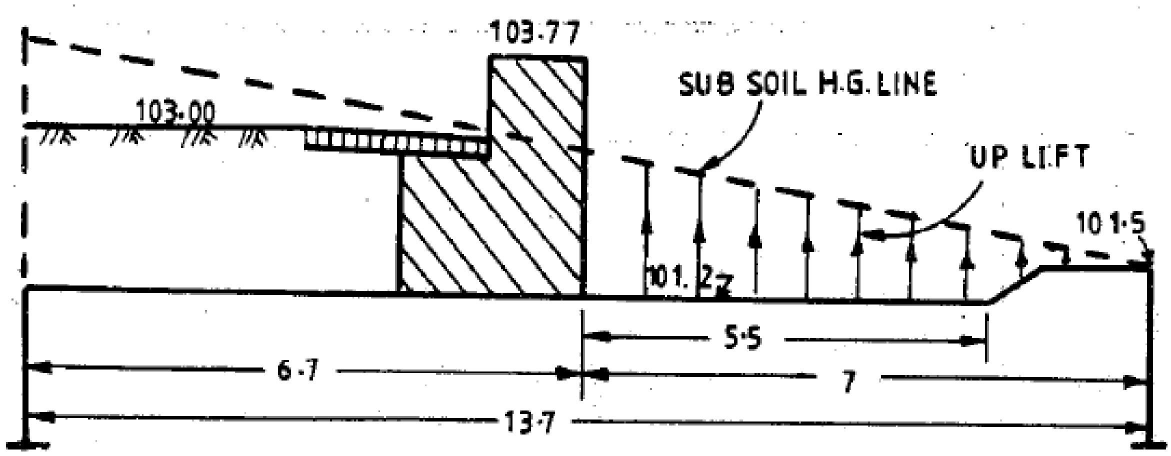


Fig.5

Floor Thicknesses.H.G.1ine for the maximum static head is shown in Fig.5

Maximum unbalanced uplift at the dis toe of the crest

$$= 0.3 + \frac{(103.77 - 101.3)}{13.7} \times 7 = 0.3 + 1.16 = 1.46 \text{ m}$$

$$\text{Thickness required } \frac{1.46}{1.24} = 1.29 \text{ m ; say } 1.3 \text{ m.}$$

Provide 1.1 m thick concrete overlain with .0.2 m thick brick pitching.

Unbalanced head at 3 m from the toe of the crest.

$$= 0.3 + \frac{2.27}{13.7} \times 4 = 0.3 + 0.67 = 0.97$$

$$\text{Thickness required} = \frac{0.97}{1.24} = 0.78 \text{ m ; say } 0.8 \text{ m.}$$

Use 0.6 m thick concrete with 0.2 m brick layer.

Unbalanced head at 5 m from the toe

$$= 0.3 + \frac{2.27}{13.7} \times 2 = 0.3 + 0.33 = 0.63 \text{ m.}$$

Thickness required

$$= \frac{0.63}{1.24} = 0.51 \text{ m} \text{ ; Say } 0.55 \text{ m.}$$

Use 0.35 m thick concrete with 20 cm thick brick layer, as shown in Fig.4

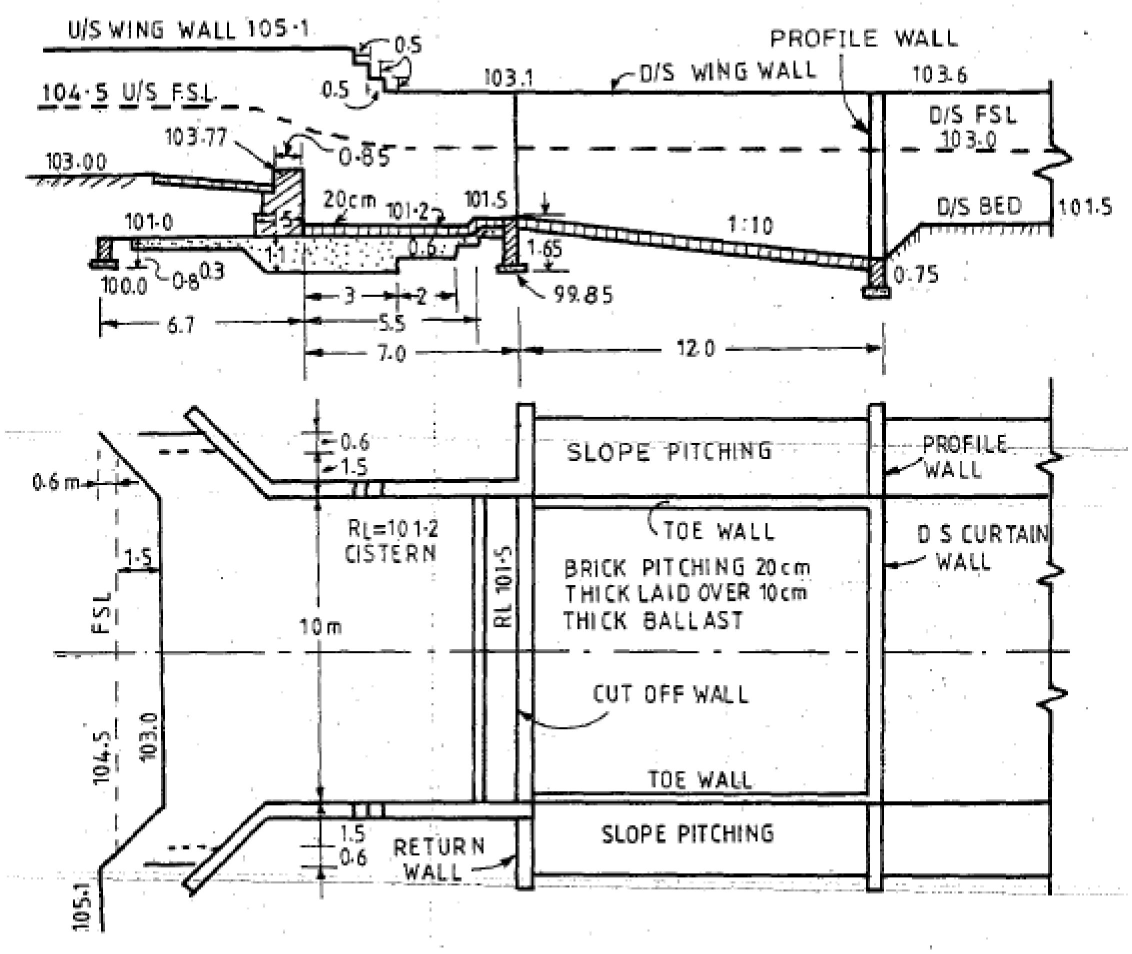


Fig.4 Details of the Sarda Type fall(rectangular crest)of example

D/S Curtain Wall. The curtain wall at the d/s end of the floor should be 0.75 m deep (for H = 0.76 m in Table .1)

Provide 0.4 m x 1.65 m deep curtain wall at d/s end of floor, i.e. upto a level of 101.5- 1.65 = 99.85 metres, i.e. the deepest foundation level.

Downstream pitching. From 'Fable .1

Total length of d/s pitching

=9+2x 1.5= 12metres.

Pitching is kept sloped at 1 : 10. A curtain wall of 0.4 m x 0.75 m shall be provided at the end of the pitching, as shown in Fig.4

5- Design of a Baffle Fall or Inglis Fall

Certain flumed type ordinary straight Glacis falls constructed in Punjab were later found to give some serious troubles, which gave rise to the conclusion that considerable surplus energy might remain in water even after the jump formation. One major cause of these troubles was found to be, too, rapid expansion after fluming, which may generate cork screw eddies causing deep scours. Research was carried out to eliminate these defects and Baffle fall was evolved.

A baffle fall makes use of the principle of horizontal impact for energy dissipation. The jump is held stable on a horizontal platform by means of a baffle wall (called baffle).

Baffle Platform. The horizontal platform. Is provided at the level at which the jump would normally form. This can be determined. by Blench curves in case them is no expansion of the wings in the region of supercritical jet. If the supercritical jet is splayed, the optimum level at which the baffle platform should be provided, can be determined by designer's curves (given in C.B.I. publication No, 10). In the absence of curves, the values can be determined by using the formulas given below.

Subcritical depth (y2) required. for jump formation in ordinary cases without fluming is very nearly given by:

$$y_2 = 0.98\, q^{0.52}\, H_L^{0.21}$$

where q = the discharge intensity in cumecs/metre

H_L= Drop in metres.

y_2 = the subcritical depth in metres.

The subcritical depth [y_2 (flumed)] required for jump formation, in case there is a fluming, is given by:

$$y_2\,(\text{flumed}) = y_2 + (H_X - H_L)$$

where H_x is the calculated drop in metres given by

$$H_X = \frac{H_L}{K^{0.152}}$$

where K is the fluming ratio (more than 1),*i.e.*

$$= \frac{\text{Actual width of canal before fluming}}{\text{Flumed width of canal}}$$

The R.L of the baffle platform then be given by

(i) for unflumed fall= d/s FSL- y_2
(ii) For flumed fall =d/s FSL -y_2 (flumed)

Baffle Wall. Height of the baffle wall = $h_b = y_c - y_1$
where y_c is the critical depth, given by

$$y_c = \left[\frac{q^2}{g}\right]^{1/3}$$

and y_1 is prejump super critical depth, given by:

$$y_1 = 0.183\, q^{0.89} \cdot H_X^{-0.35}$$

$H_x = H_L$ when there is no fluming

Thickness of the baffle wall = $\frac{2}{3} h_b$

Length of baffle platform = $5.25\, h_b$.

Cistern. A cistern of length equal to 5 y_2 (flumed) (equal to 5 y_2, for without fluming) may be provided after. the baffle wall. The depth of the cistern below the downstream bed should be 10% of the downstream water depth (y_d), subject to a minimum of 15 cm for distributaries and minors and 30 cm for main canals and branches.

Upstream Wings. The upstream wings can be curved with a radius equal to $3.6\, H^{3/2}$ (where His the total head over the crest) when His more than 0.3 m, and equal to 2H when His less than 0.3 m. The circular wings are continued till they subtend an angle of 60° at the centre, and afterwards they can be extended tangentially for the required length into the berm or bank.

Downstream Wings. The downstream divergence for flumed meter falls can be provided at a slope of 1 in 3 to 1 in 4.

A milder divergence is preferable for straight Glacis fall as well as for Baffle fall (1 in 4 to 1 in 10 depending upon the bed width depth ratio of the downstream channel) but that will make the structure costlier. Hence, a divergence of 1 in 3 is generally used for meter falls of both types. The dis wings of the unflumed baffle fall are kept as are kept for unflumed straight Glacis fall.

Downstream Glacis. The dis glacis for unflumed baffle falls is kept at 2/3 : 1; but for flumed meter falls, it is kept as 2 : 1. The glacis is joined to crest at the u/s end and to the floor at the dis end with a radius equal to H, as was done in the case of a glacis fall.

All other details of pitching, etc. remain the same as for straight glacis fall.

The friction blocks are either not provided (upto 2 m fall); or provided as explained earlier (for drops of more than 2 m.)

Example .Design an unflumed non-meter baffle fall for the· canal having the following data :

Full supply discharge = 30 cumecs , Bed level u/s = 203.0 m , Bed level d/s = 201.2 m , FSL u/s= 204.3 m , FSL d/s= 202.5 m , Bed width= 28m , Drop (H_L)= 1.8m , Side slopes of channel = 1: 1.

Solution.

Crest Length. Equal to bed width : Provide 28 m. crest length

Crest level. A sharp narrow crest is provided, for which

$$Q = 1.84\, L H^{3/2}$$

$$30 = 1.84 \times 28\, H^{3/2}$$

Or

Or $$H^{3/2} = \frac{30}{1.84 \times 28} = 0.582$$

$$H = (0.582)^{2/3} = 0.697 \text{ m; say } \mathbf{0.7\ m.}$$

Velocity of approach

$$= V_a = \frac{30}{(28 + 1.3)\,1.3} = 0.787 \text{ m/sec.}$$

$$\text{Velocity Head} = \frac{V_a^2}{2g} = \frac{(0.787)^2}{2 \times 9.81} = 0.0315 \text{ m; say } \mathbf{0.03\ m.}$$

Now, u/s TEL = u/s FSL + Velocity head

$$= 204.3 + 0.03 = 204.33 \text{ m.}$$

$$\text{Crest Level} = \text{u/s TEL} - H$$

$$= 204.33 - 0.7 = 203.63 \text{ m.}$$

Adopt crest level = 203.6 m.

Width of the crest is kept equal to $\frac{2}{3}H = \frac{2}{3} \times 0.7 = 0.47 \text{ m}$

Provide crest width = 0.47 m.

U/s Glacis. Glacis of 1/2 : 1 joined tangentially to the crest with a radius equal to H/2 =0.35m shall be provided.

D/s Glacis. Glacis of 2/3: 1 joined tangentially to the baffle platform with a radius equal to H = 0.70 m. shall be provided.

Upstream Wings. The u/s wing walls shall be splayed at an angle of 45° from the u/s end of the floor and shall be embedded into the bank by 1.0 m beyond FSL line.

Downstream Wings. Parallel vertical sides up to the end of pucca floor shall be provided, which shall be connected with the return walls at 90°.

Upstream Protection. No pitching is required in bed and· on sides. Depth of u/s curtain wall required is.

$$= \frac{y_u}{3} = \frac{1.3}{3} = 0.43 \text{ m}$$

Provide 0.4 m x 0.6 m deep curtain wall over 0.3 m foundation concrete, thus making its overall depth as 0.9 m.

Baffle Platform and Raffle Wall

Baffle Platform. Since there is no fluming,

$H_L = H_x = 1.8$ m

Now $y_2 = 0.98 \cdot q^{0.52} \cdot H_L^{0.21}$

where $q = \frac{30}{28} = 1.07$ cumecs/metre

$y_2 = 0.98 \cdot (1.07)^{0.52} \cdot (1.8)^{0.21}$

$= 0.98 \times 1.0358 \times 1.133 = 1.15$ m.

R.L. of baffle platform

$=$ d/s FSL $- y_2 = 202.5 - 1.15 = 201.35$ m.

Provide Baffle platform at R.L. = 201.35 in

Baffle Wall. Height of the baffle wall

$= h_b = y_c - y_1$

Where $y_c = \left[\frac{q^2}{g}\right]^{1/3} = \left[\frac{(1.07)^2}{9.81}\right]^{1/3} = (0.117)^{0.33} = 0.49$ m

$y_1 = 0.183 \cdot q^{0.89} \cdot (H_x)^{-0.35}$

$= 0.183\,(1.07)^{0.89}\,(1.8)^{-0.35} = \mathbf{0.16\ m}$

:. Height of the baffle wall

$h_b = y_c - y_1$

$= 0.49 - 0.16 = 0.33$m.

Thickness of baffle wall =2/3*h_b =0.22m.

Length of baffle platform

= 5.25 h_b = 5.25 x 0.33 = l.73 m. ; say 1.8 m.

Cistern. Depth of cistern below d/is bed

= 0.1y_d=0.1x1.3 = 0:13 m.

R.L. of cistern= 201.2- 0.13 =.20l.07m

Length of cistern = 5 · y_2

=5 X l.15=5.75m; say 5.8m.

D/s Curtain Wall. Depth of the downstream curtain wall required

$$=\frac{y_d}{2}=\frac{1.3}{2}=0.65\text{ m};$$

or from Table .1, the depth for the curtain wall is equal to 0.6 m.

Provide 0.4 m x 1.0 m deep .curtain wall, over 0.3 m thick foundation concrete, thus making a total depth of curtain wall= 1.3 m

Height of deflector above d/s bed

$$=\frac{y_d}{10}=\frac{1.3}{10}=0.13\text{ m}.$$

Hence, the d/s curtain wall shall be raised by 0.13 m above d/s bed.

D/s Bed Pitching. No pitching is required.

D/s Side Slope Pitching. Is required in a length equal to

3 . y_d= 3 x 1.3 =.3.9 m.

Provide 0.2 m thick dry brick pitching over 0.1 m thick brick ballast in a length equal to 3.9 m. The slope pitching shall rest on a toe wall 0.4 m thick and 0.8 m deep (overall) constructed in the bed at the junction of bed and sides. A solid profile wall called 'Dhamali' shall be constructed at the end of pitching. It shall be 0.4 m thick and plastered n cement mortar.

Friction Blocks. Not required

Total Floor Length from Exit Gradient Considerations

Safe exit gradient = 1/5

Maximum static head(H) is exerted when water is stored upto crest level on u/s and there is no water on d/s.

H = 203.6- 201.2 = 2.4 m.

Depth of dis curtain wall = d = 1.3 m

Now, $$G_E=\frac{H}{d}\cdot\frac{1}{\pi\sqrt{\lambda}}$$

$$\frac{1}{5}=\frac{2.4}{1.3}\cdot\frac{1}{\pi\sqrt{\lambda}}$$

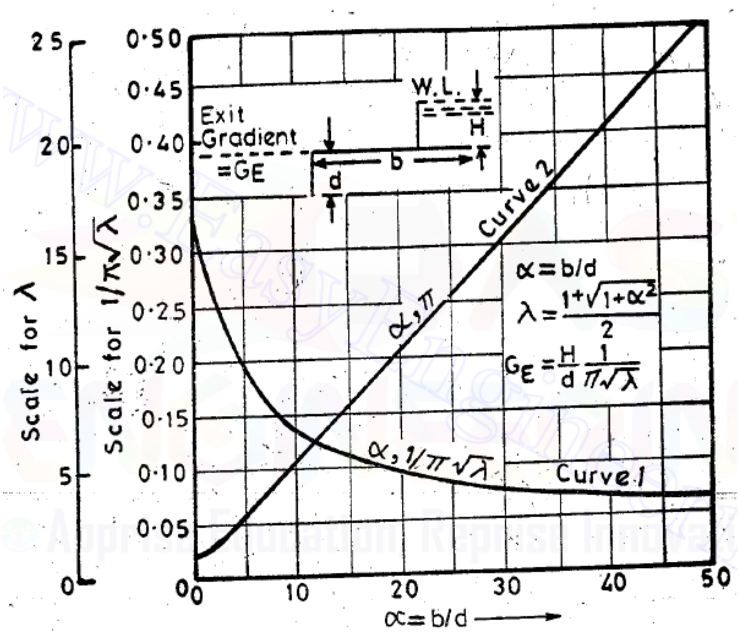


Plate 1

Or $$\frac{1}{\pi\sqrt{\lambda}}=\frac{1}{5}\times\frac{1.3}{2.4}=0.108$$

From Plate 1 $$\alpha=16.5$$

:. Total floor length required= α.d= 16.5 x 1.3 = 21.45 m;

Provide 22 m overall length

The floor length already provided (12.365 m) is shown in Fig. 1 ; the balance, i.e. 9.635 m is now provided .on the u/s, as shown in Fig. 1 ·

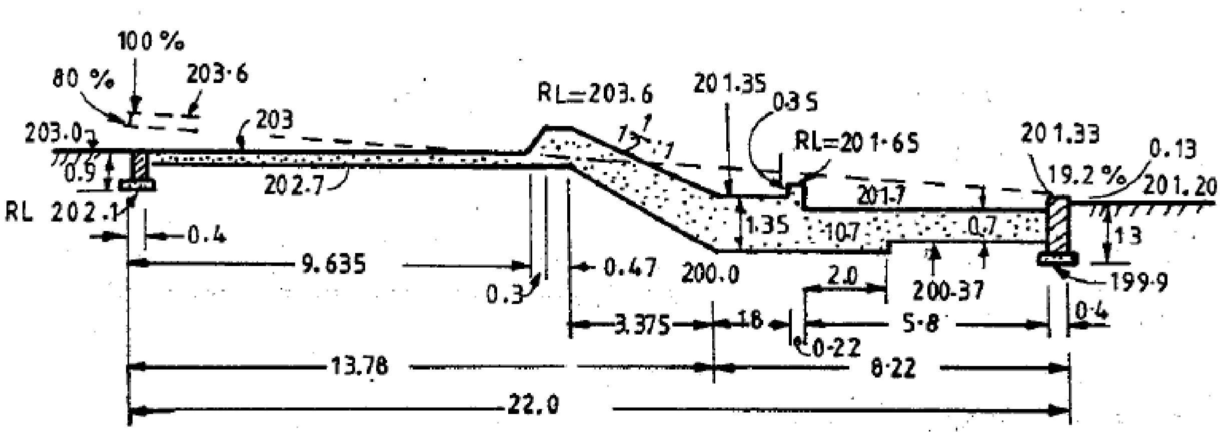
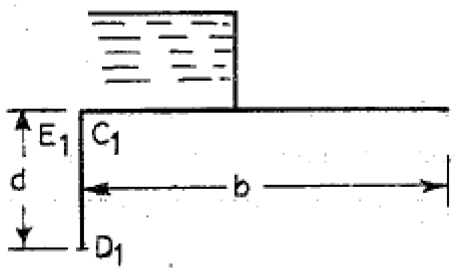


Fig.1: Dimensions of the baffle fall of example

Calculations for Uplift Pressures

(i) Upstream curtain wall

$$b=22\text{ m}$$
$$d=0.9\text{ m}$$
$$\frac{1}{\alpha}=\frac{d}{b}=\frac{0.9}{22}=0.041$$



$$\phi_{C_1}=100-\phi_E$$
$$\phi_{D_1}=100-\phi_D$$

From Plate .2

$$\phi_{E_1}=100\%$$
$$\phi_{D_1}=100-\phi_D=100\%-14\%=86\%$$
$$\phi_{C_1}=100-\phi_E=100\%-20\%=80\%$$

Assume upstream floor thickness.= 0.3 m.

Correction to ϕ_{C_1} *for depth*

$$=\frac{86\%-80\%}{0.9}\times0.3=2.0\%\ (+\text{ve})$$

ϕ_{C_1} **(Corrected)** $=80\%+2\%=$ **82%**

(ii) Downstream curtain wall

$$b=22\text{ m}$$
$$d=1.3\text{ m}$$
$$\frac{1}{\alpha}=\frac{d}{b}=\frac{1.3}{22}=0.059$$

From Plate .2

$\phi_{E_2} = \phi_E = 23\%$

$\phi_{D_2} = \phi_D = 16\%$
$\phi_{C_2} = 0\%$

Assume dis floor thickness near d/s curtain wall= 0.7 m

Correction to ϕ_{E_2} for floor thickness

$$= \frac{23\% - 16\%}{1.3} \times 0.7 = 3.8\% \ (-\text{ve})$$

ϕ_{E_2} **(Corrected)** $= 23\% - 3.8\% = \mathbf{19.2\%}$.

Uplift pressure at the toe of the glacis

$$= 19.2\% + \frac{80\% - 19.2\%}{22} \times 8.22$$
$$= 19.2\% + 22.7\% = 41.9\%$$

Floor Thicknesses

U/s Floor. Provide a nominal thickness .of 0.3 m on the upstream side and extend it up to .dis end of crest. Its bottom level shall be at R.L. 202.7m

Toe of Glacis

Level of H.G. line at toe of glacis

= 201.2+ 41.9% x 2.4

= 201.2 + 1.0 = 202.2 m

:. Unbalanced. head due to this maximum static head of 2.4.m

= 202.2 - 201.35 = 0.85m

Unbalanced head due to dynamic condition may be taken as

$$= 50\% \ (y_2 - y_1) + \% \text{ pressure} \times H_L$$
$$= 50\% \ (1.15 - 0.16) + 41.9\% \times 1.8$$
$$= 0.495 + 0.754 = 1.249 \text{ m} \ ; \ \textbf{say 1.25 m.}$$

Thus, at toe of glacis, the head due to dynamic condition is more than that due to static condition. Hence, minimum thickness required at toe $= \frac{1.25}{1.24} = 1.01 \text{ m}$. Provide 1.35 m thickness in the entire length of baffle platform, thus keeping its bottom at R.L. 201.0m.

Thickness at the start of cistern: Percentage pressure at 2.02 m from toe of glacis (i.e. start point of cistern)

$$= 19.2\% + \frac{80\% - 19.2\%}{22} \times 6.2$$

$$= 19.2\% + 17.2 = 36.4\%$$

Maximum unbalanced head at this point

= 2 02.07 - 201.07 = 1.0 m

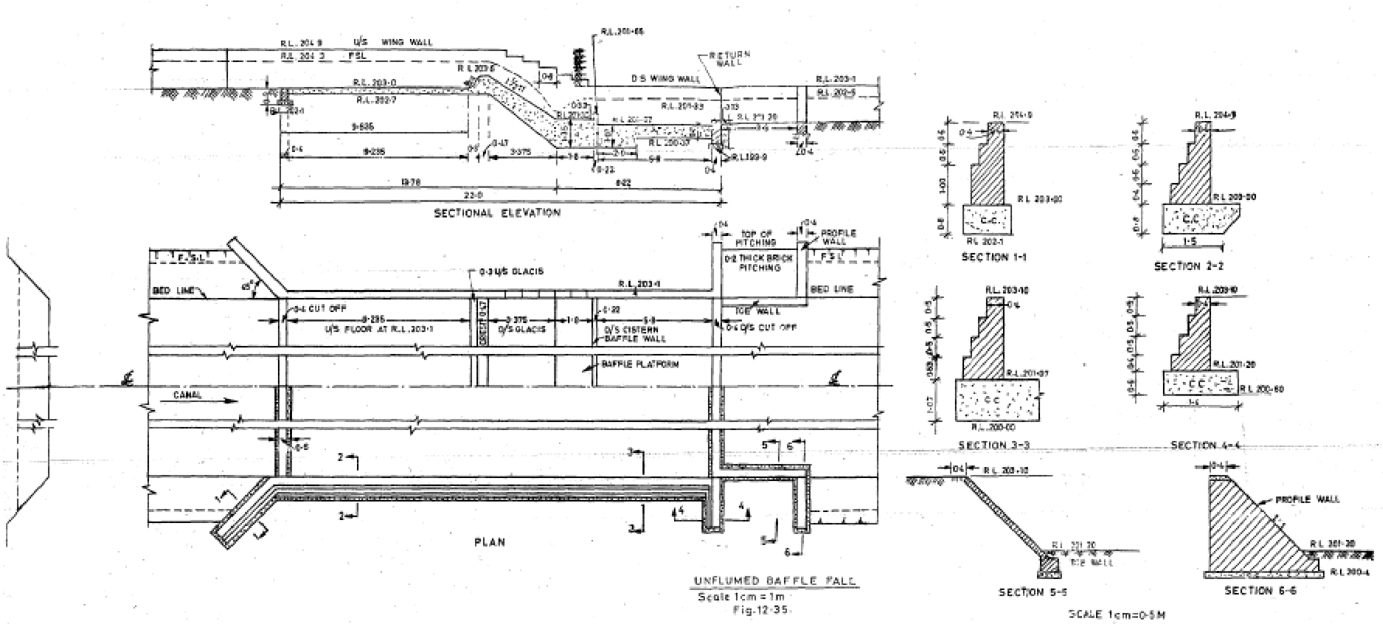


Fig.2

Floor thickness required at this point

$$= \frac{1.0}{1.24} = 0.81 \text{ m}$$

Provided thickness = 201.07 - 200.0 = 1.07 m.

Thickness required at 3 m beyond the baffle wall

$$= 19.2\% + \frac{80\% - 19.2\%}{22} \times 3.2$$
$$= 19.2\% + 8.85\% = 28.05\%$$

Level of H.G. line at this point

= 201.2+ 28.05% (2.4)

= 201.2+ 0.67 = 201.87m

Maximum unbalanced head at this point

= 201.87 - 201.07 = 0.80 m

Floor thickness required at this point

$$= \frac{0.80}{1.24} = 0.645 \text{ m}$$

Hence, provide 0.7 m thickness in the remaining portion, as shown in Fig.1

Full details of the fall are shown in the attached chart Fig.2

cross drainage works

Definition: A cross drainage work is a structure carrying the discharge from a natural stream across a canal intercepting the stream. The canal comes across obstructions like rivers, natural drains, and other canals .The various types of structures that are built to carry the canal water across the above-mentioned obstructions or vice versa are called cross drainage works.

Types of cross drainage works

Depending upon levels and discharge, it may be of the following types:

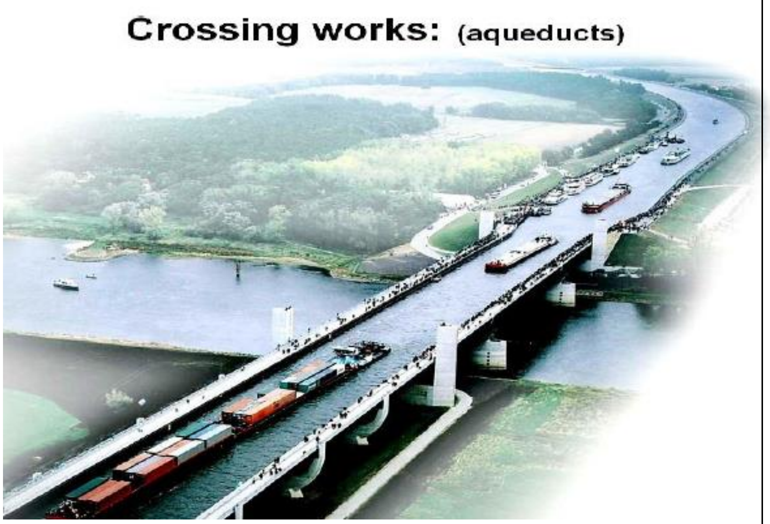
a. Cross drainage works carry canals across the drainage.[HFL < FSL]

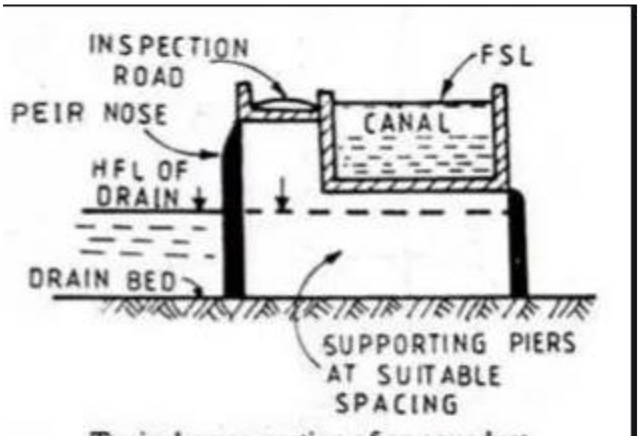
the structures that fall under this type are:

1. Aqueduct

2. Siphon Aqueduct

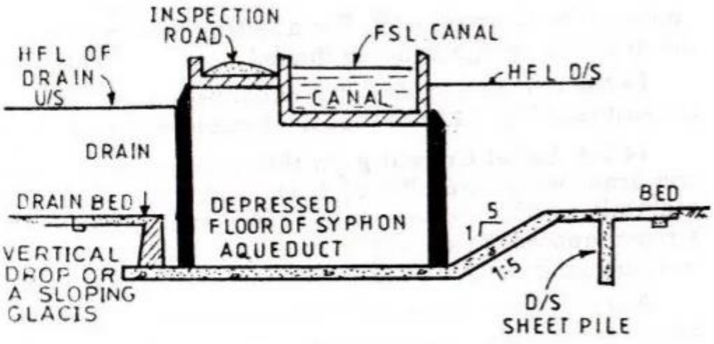
1) Aqueduct: - When the HFL of the drain is sufficiently below the bottom of the canal such that the drainage water flows freely under gravity, the structure is known as an Aqueduct





Typical cross-section of an aqueduct.

2) Siphon Aqueduct: In a hydraulic structure where the canal is taken over the drainage, but the drainage water cannot pass clearly below the canal. It flows under siphon action. So, it is known as a siphon aqueduct. This structure is suitable when the bed level of the canal is below the highest flood level



Typical cross-section of a Syphon Aqueduct.

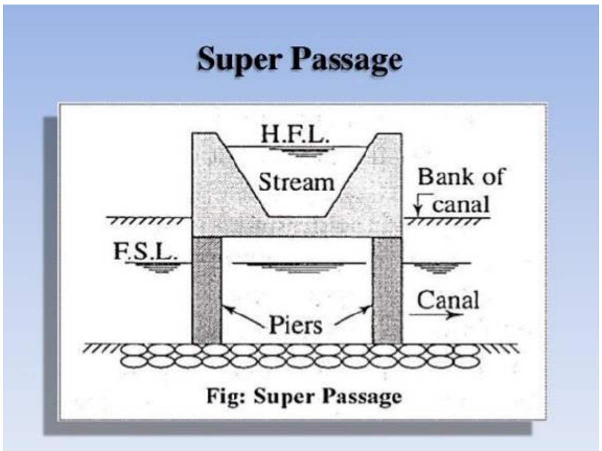
Fig (b) Syphon Aqueduct

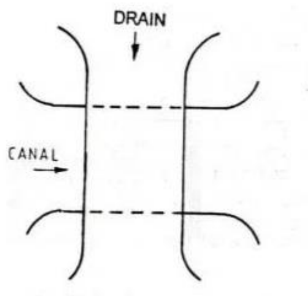
b) Cross drainage works carrying drainage over a canal.[HFL > FSL]

The structures that fall under this type are:

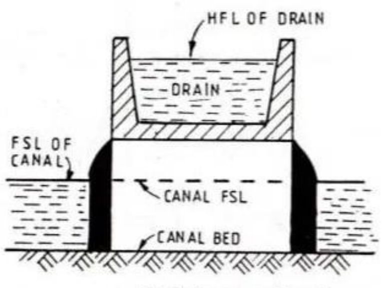
1. Super passage
2. Canal siphon

1) Super passage: -if the bed level of drainage is sufficiently above the F.S.L of the canal the structure is known as super passage.





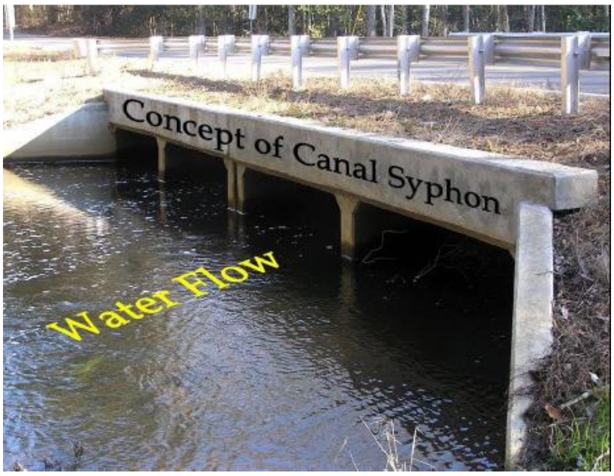
Drain taken over the canal in a Superpassage or in a Syphon. (Line Plan of Crossing)



Typical cross-section of a Superpassage.

Fig (c) Super passage

2) canal syphon: -if the F.S.L of the canal is much above the bed level of the drainage through the structure is known as canal syphon.



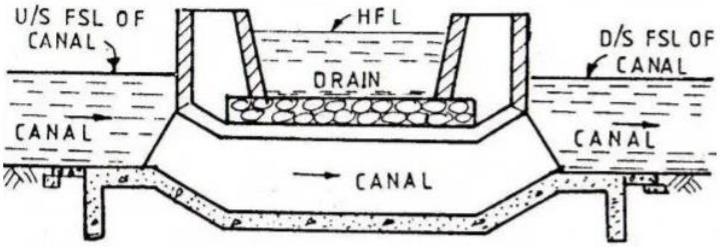


Fig (d) Canal Syphon

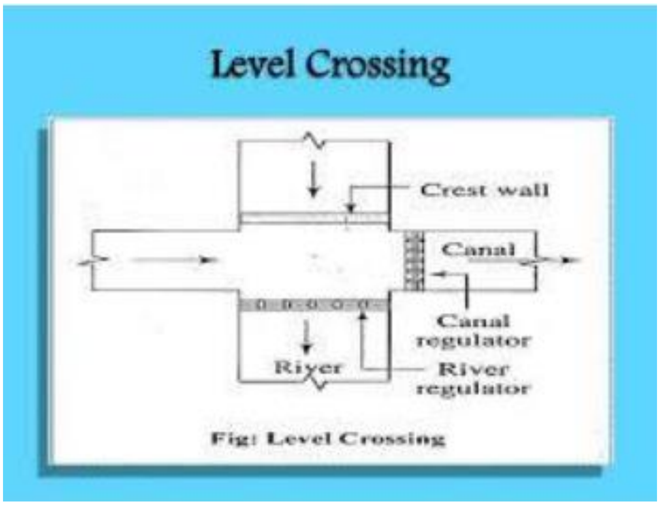
c) canal and normal drain intersecting each other.[HFL = FSL]

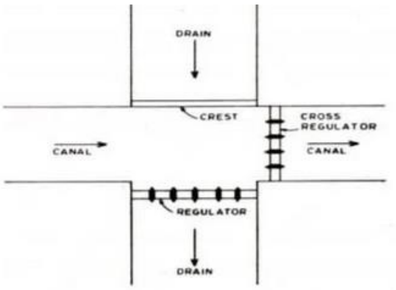
The structures that fall under this type are:

1. level crossing

2. inlets and outlets

1) level crossing: - When the bed level of canal and the stream are approximately the same and quality of water in canal and stream is not much different, the cross-drainage work constructed is called level crossing where water of canal and stream is allowed to mix.





Level Crossing

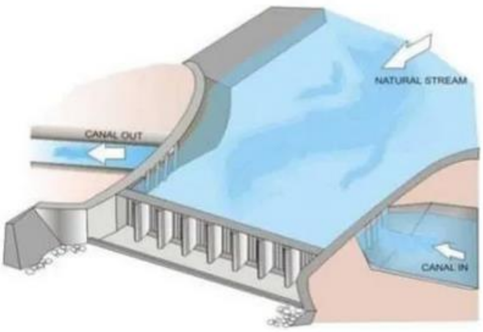
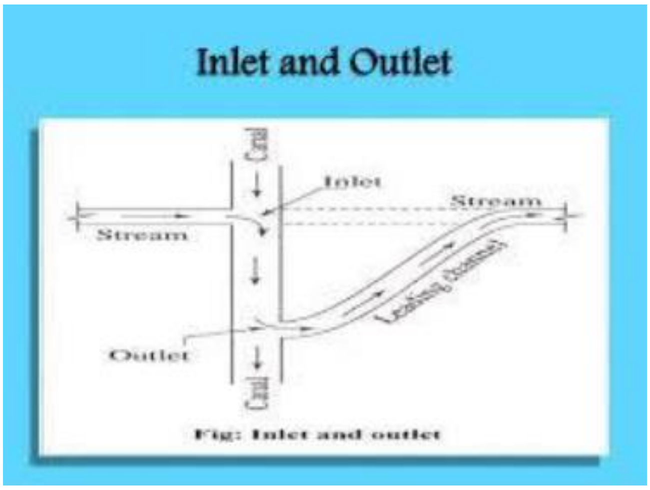


Figure (e) Level crossing

3. inlets and outlets: -when irrigation canal meets a small stream or drain at same level, drain is allowed to enter the canal as in inlet.at some distance from this inlet point a part of water is allowed to drain as outlet which eventually meets the original stream. Stone pitching is required at the inlet and outlet



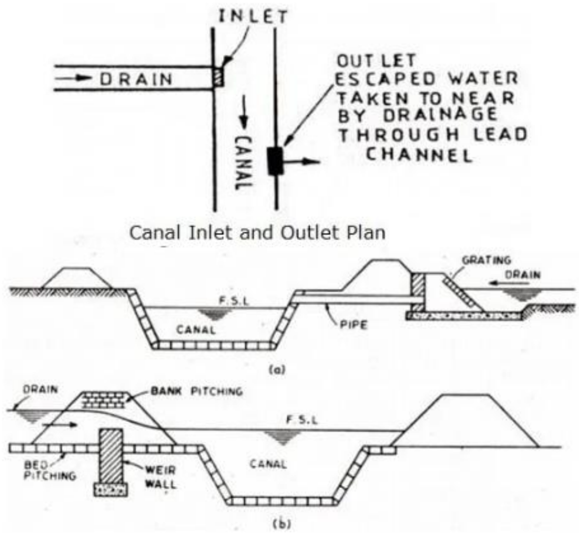


Figure (f) Canal inlet plan and cross sectional views (a) pipe type (b) open cut type

Selection of suitable site for cross drainage works- :

The factors which affect the selection of suitable type of cross drainage works are:

• Relative bed levels and water levels of canal and drainage

• Size of the canal and drainage.

• The following considerations are important

• When the bed level of the canal is much above the HFL of the drainage, an aqueduct is the obvious choice.

• When the bed level of the drain is well above FSL of canal, super passage is provided.

• The necessary headway between the canal bed level and the drainage HFL can be increased by shifting the crossing to the downstream of drainage. If, however, it is not possible to change the canal alignment, a siphon aqueduct may be provided.

• When canal bed level is much lower, but the FSL of canal is higher than the bed level of drainage, a canal siphon is preferred.

• When the drainage and canal cross each other practically at same level, a level crossing may be preferred. This type of work is avoided as far as possible.

Notes:- high flood level (HFL)

- full supply level (FSL)

Design Considerations for Cross Drainage Works:

The following steps may be involved in the design of an aqueduct or a siphon-aqueduct. The design of a super passage and a siphon is done on the same lines as for aqueducts and siphon aqueducts, respectively, since hydraulically there is not much difference between them, except that the canal and the drainage are interchanged by each other.

- **Determination of Maximum Flood Discharge**

The high flood discharge for smaller drains may be worked out by using empirical formulas; and for large drains, other reliable methods Such as Hydrograph analysis, Rational formula, etc. may be used.

- **Fixing the Waterway Requirements for Aqueducts and Syphon-Aqueducts**

An approximate value of the required waterway for the drain may be obtained by using Lacey's equation, given by

$$P = 4.75\sqrt{Q}$$

where P is the wetted perimeter in meters and Q is the Total discharge in cumecs.

Example1. Design a suitable cross-drainage work, given the following data at the crossing of a canal and drainage.

Canal:	Drainage:
Full supply discharge = 32 cumecs	High flood discharge = 300 cumecs
Full supply level = R.L. 213.5	High flood level = 210.0 m
Canal bed level = R.L. 212.0	High flood depth = 2.5 m
Canal bed width = 20 m	General ground level = 212.5 m.

Trapezoidal canal section with 1.5 H: 1 V slopes.

Canal water depth = 1.5 m

Solution: Since the drainage is of large size, work of Type III will be adopted. Also, because the canal bed level (212.0) is much above the HFL of drainage (210.0), an aqueduct will be constructed. To affect economy, the canal shall be flumed.

Step 1: Design of Drainage Waterway:

Lacey's regime perimeter = $P = 4.75\sqrt{Q} = 4.75\sqrt{300} = 82.3$ m

Let the clear span between piers be 9 m and the pier thickness be 1.5 m.

Using 8 bays of 9 m each, clear waterway = 8×9=72

Using 7 piers of 1.5 m each, length occupied by piers = 7×1.5=10.5

Total length of waterway = 72 + 10.5 = 82.5 m

Step 2: Design of Canal Waterway:

Bed width of canal = 20.0 m

Let the width be flumed to 10.0 m

Providing a splay of 2: 1 in contraction, the length of contraction transition $= \frac{20-10}{2} \times 2 =$ 10.0m

Providing a splay of 3: 1 in expansion, the length of expansion transition $= \frac{20-10}{2} \times 3 = 15.0\ m$

Length of the flumed rectangular portion of the canal between abutments = 82.5 m

In transitions, the side slopes of the canal section will be warped in plan from the original slope of 1.5: 1 to vertical.

Step 3: Head loss and bed levels at different sections(fig .1):

At section 4-4

Area of trapezoidal canal section = (B + 1.5 y) y

$$= (20 + 1.5 \times 1.5)\ 1.5 = 22.5 \times 1.5 = 33.75\ m^2$$

Velocity = $V_4 = \left(\frac{Q}{A}\right) = \frac{32}{33.75} = 0.947$ m/sec

Velocity head = $\frac{V_4^2}{2g} = \frac{(0.947)^2}{2 \times 9{,}81} = 0.046$ m

R.L of bed at 4-4 = 212.0 m (given)

R.L of water surface at 4-4 = 212.0 + 1.5 = 213.5 m

R.L of TEL at 4-4 = 213.5 + 0.046 = 213.546 m

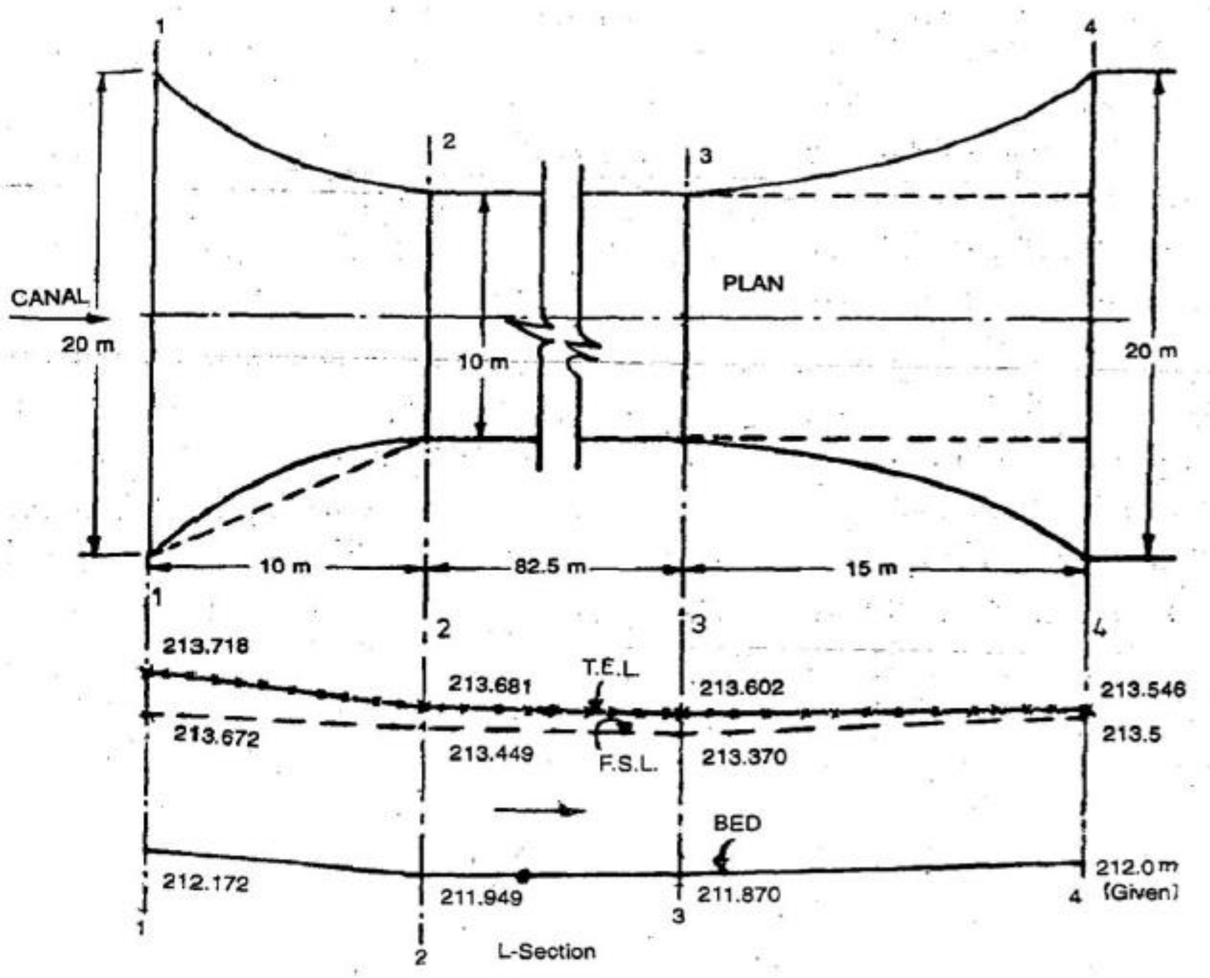


Figure 1. Plan and section of canal trough

At section 3-3

Keeping the same depth of 1.5 m throughout the channel, we have

Bed width = 10 m

Area of channel = $10 \times 1.5 = 15\ m^2$

Velocity = $V_3 = \left(\frac{Q}{A}\right) = \frac{32}{15} = 2.13\ m/sec$

Velocity head = $\frac{V_3^2}{2g} = \frac{(2.13)^2}{2 \times 9{,}81} = 0.232\ m$

Assuming that the loss of head in expansion from section 3-3 to 4-4 is taken = $0.3\left(\frac{V_3^2 - V_4^2}{2g}\right)$

= 0.3 [0.232 – 0.046] = 0.056 m

R.L of TEL at section 3-3 = R.L of TEL at 4-4 + loss in expansion

= 213.546 + 0.056 = 213.602 m

R.L of water surface at 3-3 = R.L of TEL at 3-3 – velocity head

= 213.602 – 0.232 = 213.370 m

R.L of bed at 3-3 = 213.370 – 1.5 = 211.87 m

At section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are same as at 3-3, there is a friction loss between 2-2 and 3-3 which is given by manning's formula

$$H_L = \frac{n^2.V^2.L}{R^{2/3}} = \frac{(0.016)^2 \times (2.13)^2 \times 82.5}{(1.16)^{2/3}} = 0.079 \text{ m}$$

R.L of TEL at section 2-2 = R.L of TEL at 3-3 + friction loss

=213.602 + 0.079 = 213.681m

R.L of water surface at 2-2 = R.L of TEL at 2-2 – velocity head

= 213.681 – 0.232 = 213.449 m

R.L of bed at 2-2 = 213.449 – 1.5 = 211.949 m

At section 1-1

Loss of head in contraction transition from 1-1 to 2-2 = $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

$$= 0.2\left(\frac{(2.13)^2-(0.947)^2}{2\times9.81}\right) = 0.037 \text{ m}$$

R.L of TEL at section 1-1 = R.L of TEL at 2-2 + Loss in contraction

= 213.681 + 0.037 = 213.718 m

R.L of water surface at 1-1 = R.L of TEL at 2-2 – velocity head

= 213.718 – 0.046 = 213.672 m

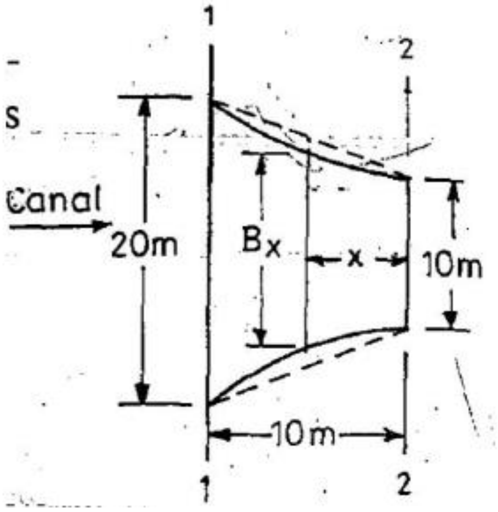
R.L of bed at 1-1 = 213.672 – 1.5 = 212. 172 m

Step 4: Design of Transitions:

(a) Contraction transition: Since the depth is kept constant, the transition can be designed on the basis of Mitra's method.

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - (B_n - B_f)x}$$

$$= \frac{20 \times 10 \times 15}{15 \times 20 - x\,(20-10)} = \frac{3000}{300 - 10\,x}$$



For various values of x lying between 0 to 10 m, various values of B_x are worked out by using the above equation as:

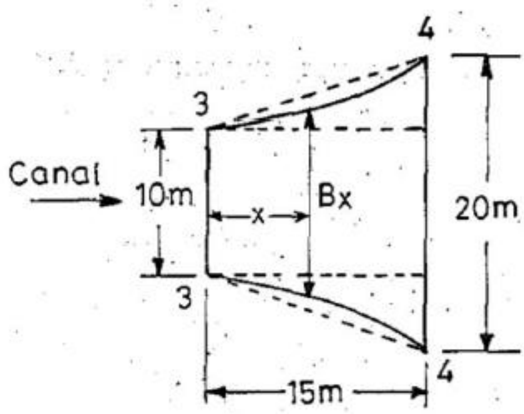
x in metres	0	2	4	6	8	10
$B_x = \frac{3000}{300 - 10\,x}$	10.0	11.11	12.5	14.29	16.67	20.0

The contraction transition can be plotted with these values.

(b) Expansion transition:

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - (B_n - B_f)x}$$

$$= \frac{20 \times 10 \times 15}{15 \times 20 - x\,(20-10)} = \frac{3000}{300 - 10\,x}$$



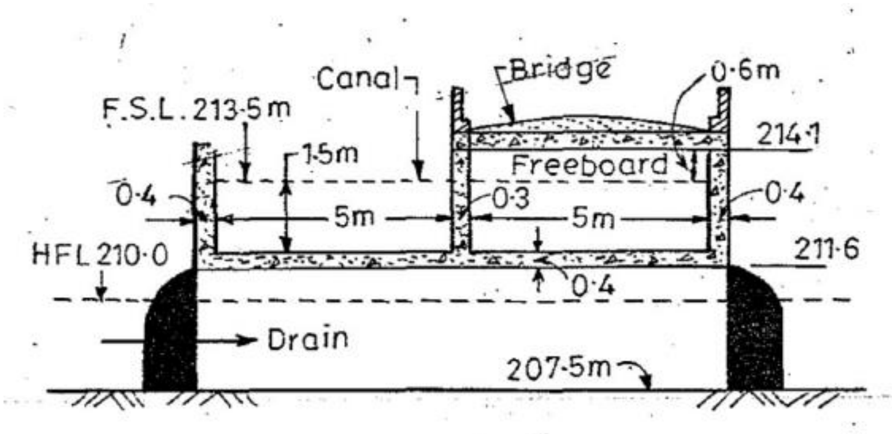
For various values of x lying between 0 to 15 m, various values of B_x are worked out by using the above equation as:

x in metres	0	2	4	6	8	10	12	14	15
$B_x = \frac{3000}{300 - 10\,x}$	10.0	10.71	11.54	12.5	13.64	15.0	16.67	18.75	20.0

The expansion transition can be plotted with these values.

Step 5: Design of Trough:

The trough shall be divided into two compartments of 5 m each and separated by an intermediate wall of 0.3 m thickness. The inspection road shall be carried on the top of left compartment as shown in figure below.



A freeboard of 0.6 m above the normal water depth of 1.5 m is sufficient, and hence the bottom level of bridge slab over the left compartment can be kept at 1.5 + 0.6 = 2.1 m above the bed level of trough. The entire trough section can be constructed in monolithic reinforced concrete and can be designed by usual structural methods.

Example1. Design a suitable cross-drainage work, given the following data at the crossing of two streams of water

Irrigation channel:

Full supply discharge = 350 cumecs

Full supply level = R.L. 202.5

Canal bed level = R.L. 197.0

Canal bed width = 35 m

Trapezoidal canal section with 0.5 H: 1 V slopes.

Canal water depth = 4.7 m

Natural Drainage:

Drainage bed level = 203.9 m

High flood level = 205.2 m

Catchment area of drainage up to crossing= 14.3 sq. km

The dickens formula may be used for computing H.F.Q. with its coefficient as 18.

Solution: The high flood discharge of the drainage at the point of crossing may be obtained by using the Dickens formula:

$$Q = C.\ A^{3/2} = 18.\ (14.3)^{3/2} = 132 \text{ cumecs}$$

Since the bed level of drainage (203.9) is much above the canal FSL (202.5), the canal water can be taken below the drainage. Hence the CD work to be constructed at the crossing will be a super-passage. The design of the super-passage is to be done on the same lines as other of an aqueduct

Step 1: Design of canal waterway:

$$\text{Flow velocity in the canal} = \frac{Discharge}{Area} = \frac{Q}{(B+0.5\ y)y} = \frac{350}{(35+0.5\times 4.7)\ 4.7} = 1.99 \text{ m/sec.}$$

This high velocity shows that the canal is already a lined canal, and much more lining cannot be affected. Hence the original bed width of 35 m can be continued as canal barrels below the drainage trough or slight fluming may be done. Let us adopt a clear waterway of 30 m in two spans, each of 15 m, with a central pier of width say 1.5 m, thus providing an overall linear waterway of 31.5 m between abutments, and this will be the length of drainage troughs.

Providing a splay of 2: 1 in contraction, the length of contraction transition= $\frac{35-31.5}{2}\times 2 = 5\ m$

Providing a splay of 3: 1 in expansion, the length of expansion transition = $\frac{35-31.5}{2}\times 3 = 7.5\ m$

Length of the flumed rectangular portion of the canal will be equal to the width of the drainage troughs = 50.5 m.

The piers, abutments, wing walls and return walls of the canal will be designed as those of a bridge taking the load of drainage trough (including the load of water and inspection road, etc) instead of a bridge deck slab.

Step 2: Design of drainage waterway:

Lacey's regime perimeter = $P = 4.75\sqrt{Q} = 4.75\sqrt{132} = 54.5\ m$

The total length of the waterway is generally chosen equal to P, although it can be slightly reduced to affect economy, but too much contraction of the drainage poses problems, and hence too much fluming is never done.

Let us provide 6 RCC compartments, each of clear width equal to 8 m, thus giving

Clear waterway = 6×8=48m

Using 5 partition walls of 0.3 m thick each, length occupied by walls = 5×0.3=1.5m

Therefore, total length of waterway provided = 48 + 1.5 = 49.5 m

The two side walls of the RCC drainage trough may be kept 0.4 m thick each, with 49.5 m as aggregate waterway between them. Thus,

End to end length of drainage trough = 49.5 + 0.8 ≈50.5 m.

Thus, the length of the rectangular portion of the canal will also be equal to 50.5 m

Since the drainage has also been slightly flumed and kept lesser than P, so let's design its contraction and expansion lengths

Assuming 2: 1 convergence, the length of contraction transition= $\frac{54.5-49.5}{2} \times 2 = 5\ m$

Providing a splay of 3: 1 in expansion, the length of expansion transition= $\frac{54.5-49.5}{2} \times 3 =$ 7.5m

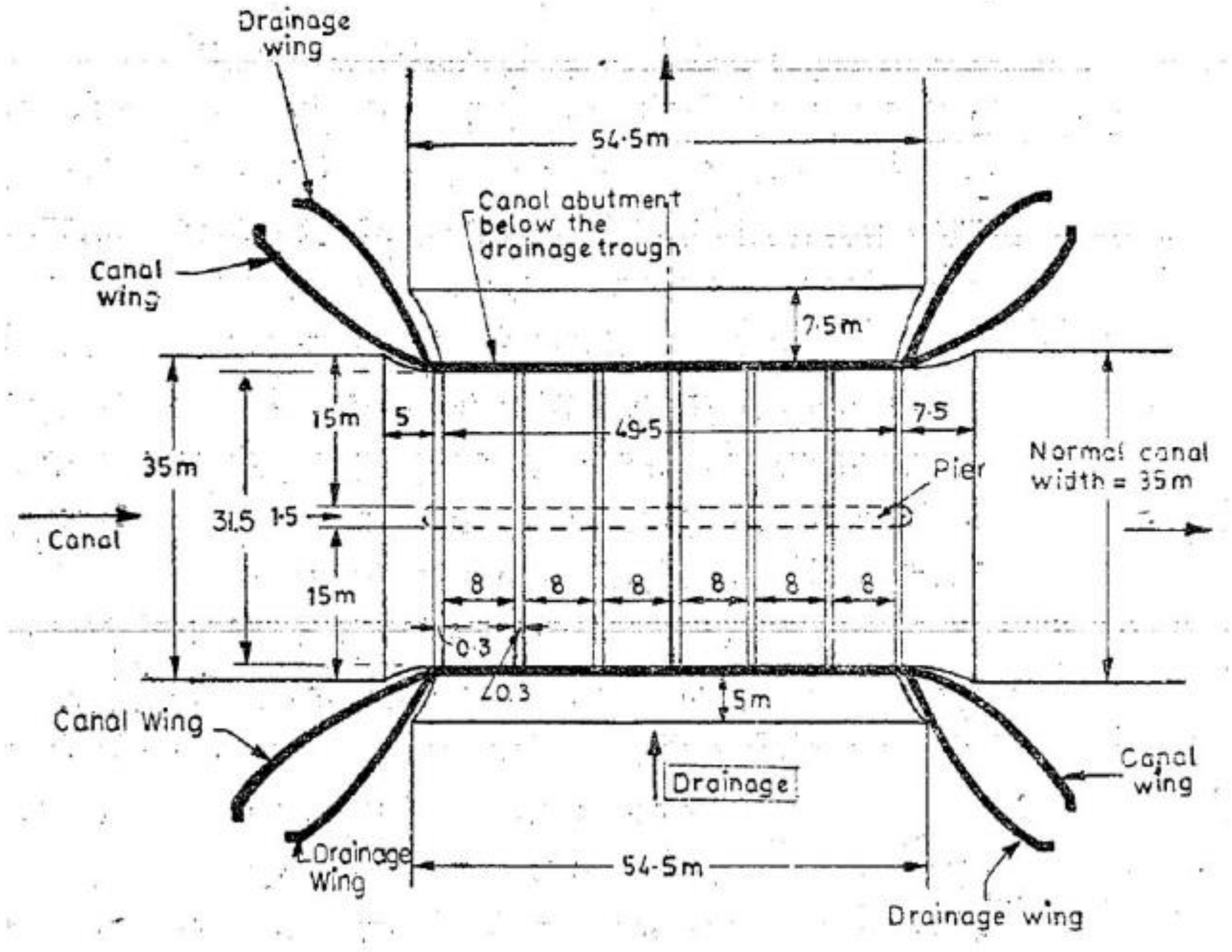


Figure 1. Inductive plan of super passage crossing

The wing walls will be constructed to reduce the drainage waterway width from 54.5 to 49.5 m on u/s and return walls will be constructed to expand the drainage waterway from 49.5 to 54.5 m on d/s. These wings will be extended so as to enter the berms of the drain.

The length of the drainage pucca rectangular trough will be equal to 31.5 m (i.e. equal to the rectangular waterway of canal).

Step 3: Head loss and Bed levels of different sections along the length of drainage trough:

At section 4-4

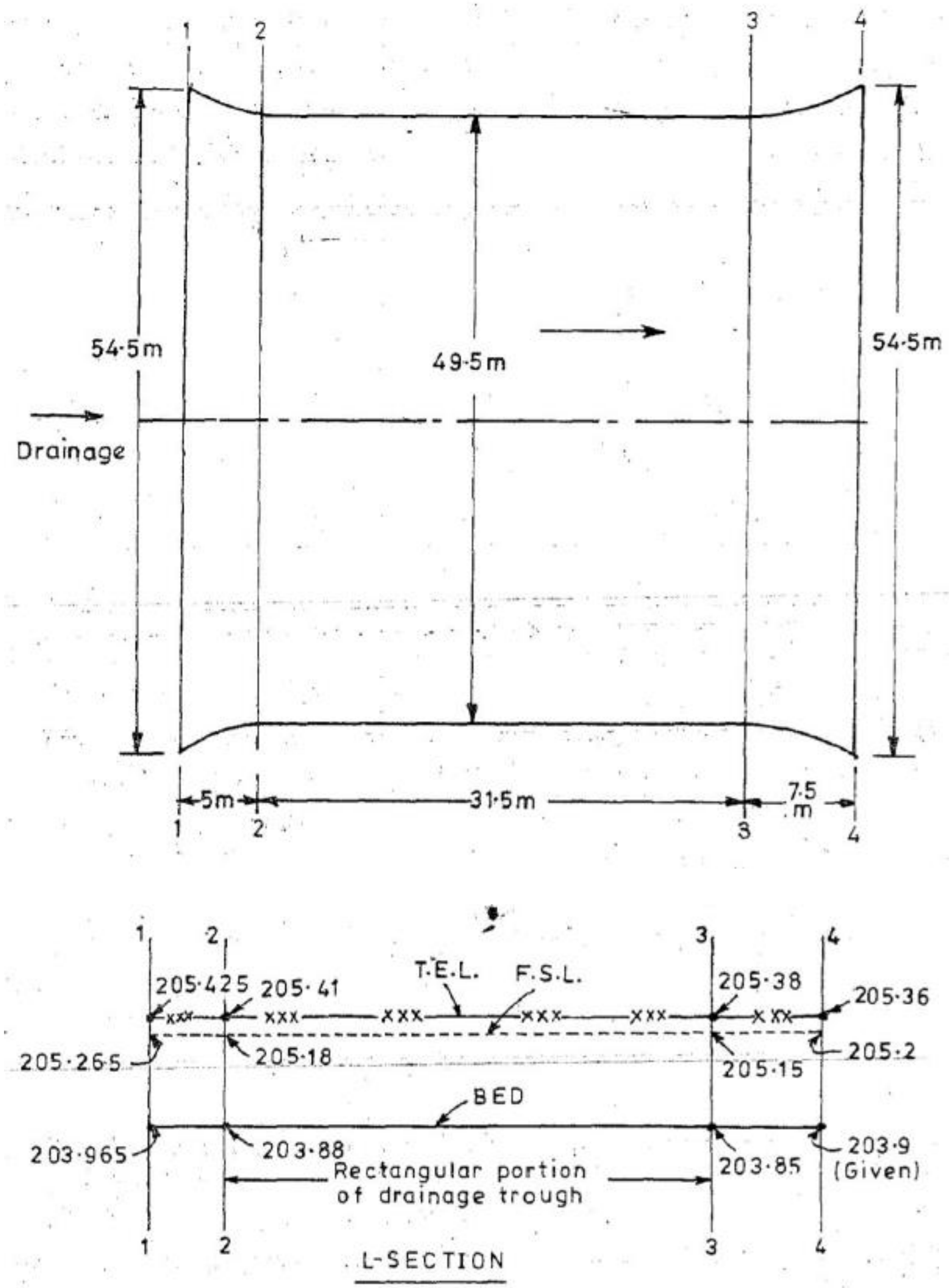


Figure 2. Plan and L-section of drainage trough carried over the canal

Area of natural drainage section = width (= perimeter)× depth

= 54.5× (205.2-203.9)=70.85m^2

$$\text{Velocity} = V_4 = \frac{Q}{A} = \frac{132}{70.85} = 1.86 \text{ m/sec, say } 1.9 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_4^2}{2g} = \frac{(1.9)^2}{2 \times 9.81} = 0.16 \text{ m}$$

R.L of bed at 4-4 = 203.9 m (given)

R.L of water surface at 4-4 = 203.9 + 1.3 = 205.2 m

R.L of TEL at 4-4 = 205.2 + 0.16 = 205.36 m

At section 3-3

Keeping the same depth of 1.3 m throughout the channel, we have

Clear waterway = 8 × 6 = 48 m

Area of flow = 48 × 1.3 = 62.4 m^2

$$\text{Velocity} = V_3 = \left(\frac{Q}{A}\right) = \frac{132}{62.4} = 2.12 \text{ m/sec}$$

$$\text{Velocity head} = \frac{V_3^2}{2g} = \frac{(2.12)^2}{2 \times 9.81} = 0.23 \text{ m}$$

Assuming that the loss of head in expansion from section 3-3 to 4-4 is taken = 0.3 $\left(\frac{V_3^2 - V_4^2}{2g}\right)$

= 0.3 [0.23 – 0.16] = 0.02 m

R.L of TEL at section 3-3 = R.L of TEL at 4-4 + loss in expansion

= 205.36 + 0.02 = 205.38 m

R.L of water surface at 3-3 = R.L of TEL at 3-3 – velocity head

= 205.38 – 0.23 = 205.15 m

R.L of bed at 3-3 = 205.15 – 1.3 = 203.85 m

At section 2-2

From section 2-2 to 3-3, the trough section is constant. Therefore, area and velocity at 2-2 are same as at 3-3, there is a friction loss between 2-2 and 3-3 which is given by manning's formula.

$$H_L = \frac{n^2.V^2.L}{R^{2/3}} = \frac{(0.016)^2 \times (2.12)^2 \times 31.5}{(1.23)^{2/3}} = 0.0275 \text{ m, say } 0.03 \text{ m}$$

R.L of TEL at section 2-2 = R.L of TEL at 3-3 + friction loss

= 205.38 + 0.03 = 205.41 m

R.L of water surface at 2-2 = R.L of TEL at 2-2 – velocity head

= 205.41 – 0.23 = 205.18 m

R.L of bed at 2-2 = 205.18 – 1.3 = 203.88 m

At section 1-1

Loss of head in contraction transition from 1-1 to 2-2 = $0.2\left(\frac{V_2^2 - V_1^2}{2g}\right)$

= 0.2 [0.23 – 0.16] = 0.015 m

R.L of TEL at section 1-1 = R.L of TEL at 2-2 + Loss in contraction

= 205.41 + 0.015 = 205.425 m

R.L of water surface at 1-1 = R.L of TEL at 2-2 – velocity head

= 205.425 – 0.16 = 205.265 m

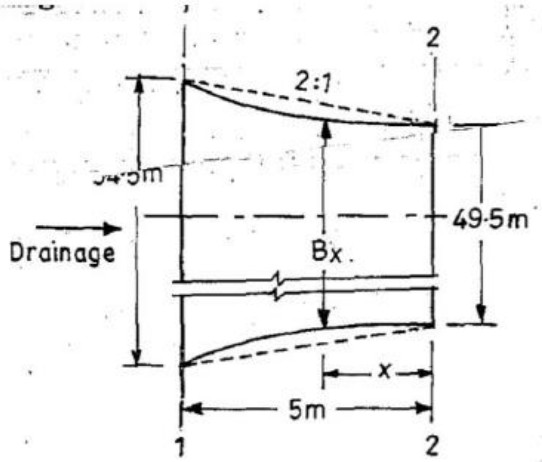
R.L of bed at 1-1 = 205.265 – 1.3 = 203.965 m

Step 4: Design of Transitions:

(a) Contraction transition: Since the depth is kept constant, the transition can be designed on the basis of Mitra's method.

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - (B_n - B_f) x}$$

$$= \frac{54.5 \times 49.5 \times 5}{5 \times 54.5 - x\,(54.5 - 49.5)} = \frac{2700}{54.5 - x}$$



For various values of x lying between 0 to 5 m, various values of B_x are worked out by using the above equation as:

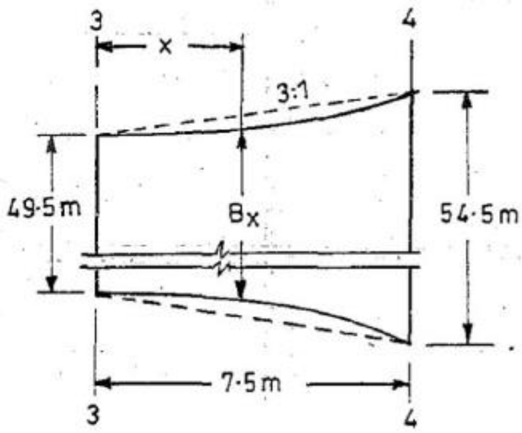
x in metres	0	1	2	3	4	5
$B_x = \dfrac{2700}{54.4 - x}$	49.5	50.5	51.5	52.5	53.5	54.5

The contraction transition can be plotted with these values.

(b) Expansion transition:

$$B_x = \frac{B_n . B_f . L_f}{L_f B_n - (B_n - B_f) x}$$

$$= \frac{54.5 \times 49.5 \times 7.5}{7.5 \times 54.5 - x\,(54.5 - 49.5)} = \frac{4050}{81.7 - x}$$



For various values of x lying between 0 to 7.5 m, various values of B_x are worked out by using the above equation as:

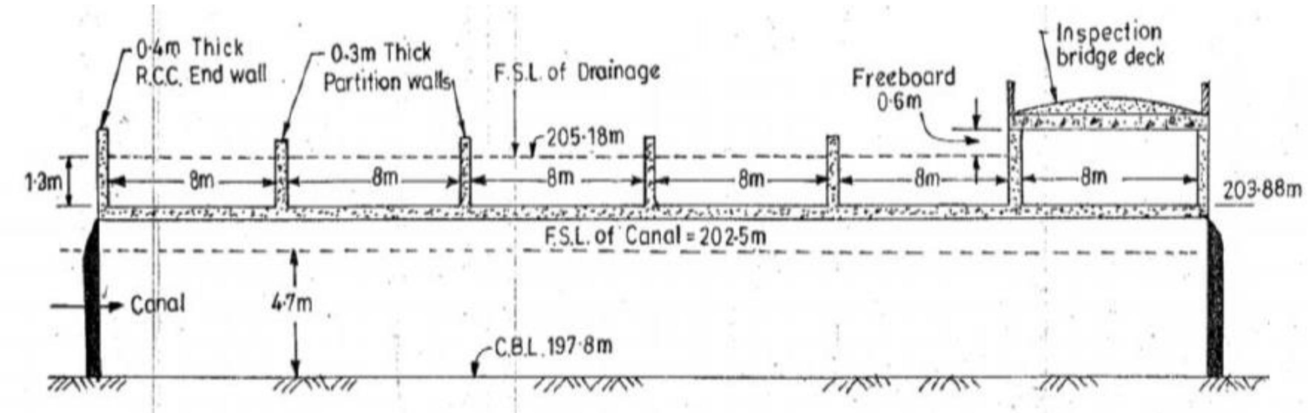
x in metres	0	2	4	6	7.5
$B_x = \dfrac{4050}{81.7 - x}$	49.5	50.7	52.0	53.3	54.5

The expansion transition can be plotted with these values.

Step 5: Design of Drainage Trough:

The RCC drainage trough has been divided into six compartments of 8 m each and separated by intermediate walls (5 in no.) of 0.3 m thickness. The end walls of the trough have tentatively been kept as 0.4 m wide each. The inspection road shall be carried on the top of end compartment as shown in fig.

A freeboard of 0.6 m above the normal water depth of 1.3 m is sufficient, and hence the bottom level of bridge slab over the end compartment can be kept at 1.3 + 0.6 = 1.9 m above the bed level of trough. The entire trough section can be constructed in monolithic reinforced concrete and can be designed by usual structural methods



CANAL HEADWORKS

An irrigation channel takes its supplies from its source which can be either a river (in the case of the main canal) or a channel (in the case of branch canals and distributaries). The structures constructed across a river source at the head of an offtaking main canal are termed “canal headworks” or “headworks”. The headworks can be either **diversion headworks** or **storage headworks**.

Diversion headworks divert the required supply from the source channel to the offtaking channel. The water level in the source channel is raised to the required level so as to divert the required supplies into the offtaking channel. The diversion headworks should be capable of regulating the supplies into the offtaking channel. If required, it should be possible to divert all the supplies (at times of keen demand and low supplies) into the offtaking channel. The headworks must have an arrangement for controlling the sediment entry into the channel offtaking from a river. By raising the water level, the need of excavation in the head reaches of the offtaking channel is reduced and the command area can be served easily by flow irrigation.

Storage headworks, besides fulfilling all the requirements of diversion headworks, store excess water when available and release it during periods when demand exceeds supplies.

Fig1: DIVERSION HEADWORK

Fig 2: STORAGE HEADWORK

LOCATION OF HEADWORKS ON RIVERS

Larger rivers, generally, have four stages, viz., the rocky, boulder, trough (or alluvial) and delta stages. Of these, the rocky and delta stages are generally unsuitable for siting headworks.

Usually, the command area is away from the hilly stage, and it would, therefore, involve avoidable expenditure to construct a channel from headworks located in the hilly stage to its command area. In the delta stage, the irrigation requirements are generally less and also the nature of the river at this stage poses other problems

The boulder and alluvial stages of a river are relatively more suitable sites for locating headworks. The choice between the boulder stage and the alluvial stage is mainly governed by the command area. If both stages are equally suitable for siting the headworks from command area considerations, the selection of the site should be made such that it results in the most economical alternative. The following features of the two stages should be considered while selecting the site for headworks.

1- The initial cost of headworks in the boulder stage is generally smaller than that in the alluvial stage because of: (a) local availability of stones, (b) smaller width of river (requiring smaller length of weir), (c) smaller scour depths which reduce the requirements of cutoffs and other protection works, and (d) close proximity of higher banks which requires less extensive training works.
2- An irrigation canal offtaking from a river in the boulder region will have a number of falls which may be utilised for generation of electricity. There is almost no scope for the generation of electricity in this manner in the alluvial reach of a river.
3- If the existing irrigation demand is less but is likely to develop with the provision of irrigation facilities, it is desirable to divert the river water into an irrigation channel by constructing a temporary boulder bund across the river. This bund will be washed away every year during the floods and will be reconstructed every year. This will, no doubt, delay the Rabi crop irrigation, but it is worthwhile to use temporary bunds for a certain period; when the irrigation demand grows, permanent headworks may be constructed. In this manner, it would be possible to get returns proportional to expenditures incurred on the headworks. Construction of temporary bunds is generally not possible in the alluvial stage of the river.
4- An irrigation channel offtaking in the boulder stage of a river will normally require a large number of cross-drainage structures.
5- Because of the nature of the boulder region, there is always a strong subsoil flow in the river bed. This causes considerable loss of water and is of concern during the periods of short supply. Similarly, there will be considerable loss of water from the head reach of the offtaking channel. In alluvial reach of the river this loss of water is much less.
6- The regions close to the hills usually have a wet climate and grow good crops. The irrigation demand in the head reach of the channel offtaking in the boulder stage is, therefore, generally small. However, this demand would increase with the provision of irrigation facilities. In alluvial regions, the demand for irrigation is high right from the beginning.

Selection of Site for Head work

DIFFERENT UNITS OF HEADWORKS

Diversion headworks (Fig. 1) mainly consist of a weir (or barrage) and a canal head regulator. A weir has a deep pocket of undersluice portion upstream of itself and in front of the canal head regulator on one or both sides. The undersluice bays are separated from other weir bays by means of a divide wall. In addition, river training structures on the upstream and downstream of weir, and sediment excluding devices near the canal head regulator are provided. Detailed model investigations are desirable to decide the location and layout of headworks and its component units. A typical layout of diversion head works is shown in Fig.1

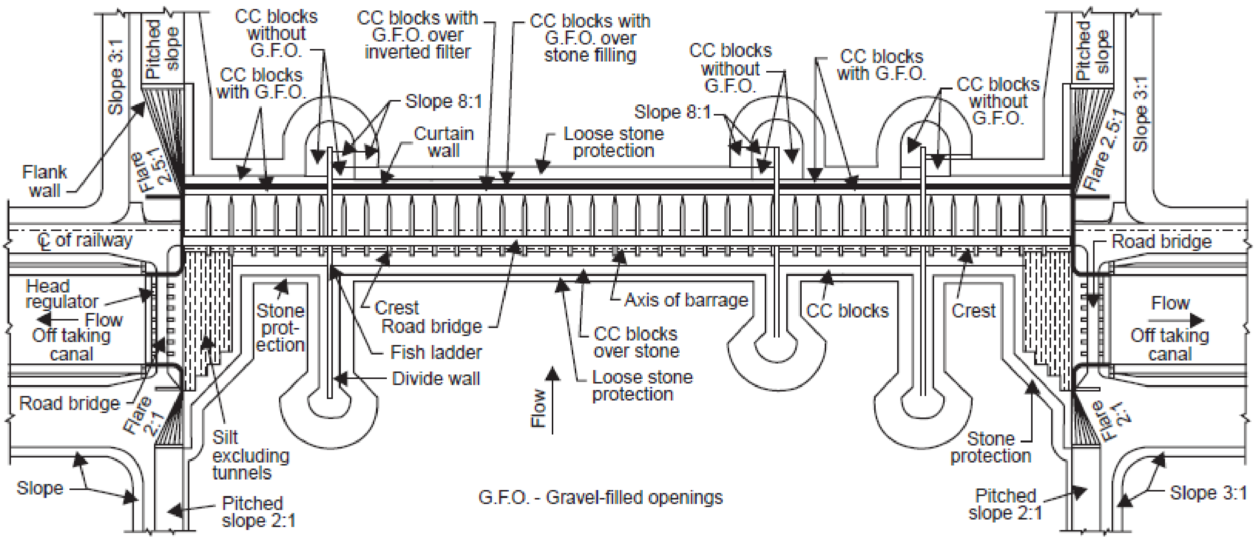


Fig. 1 Typical layout of headworks

WEIR (OR BARRAGE)

A weir is an ungated barrier across a river to raise the water level in the river. It raises the water level in the river and diverts the water into the offtaking canal situated on one or both of the river banks just upstream of the weir. Weirs are usually aligned at right angles to the direction of flow in the river. Such weirs will have minimum length and normal uniform flow through all the weir bays thereby minimising the chances of shoal formation and oblique flow.

The procedure of design of a barrage is similar to that of a weir. Weirs are of the following three types:

(i) Masonry weirs with vertical downstream face,

(ii) Rockfill weirs with sloping apron, and

(iii) Concrete weirs with glacis.

UNDER SLUICES

The construction of weir across a river results in ponding up of water and causes considerable sediment deposition just upstream of the canal head regulator. This sediment must be flushed downstream of the weir. This is done by means of under sluices (also called sluice ways or scouring sluices). A weir generally requires deep pockets of under sluices in front of the head regulator of the offtaking canal, and long divide wall to separate the remaining weir bays from the under sluices. The under sluices are the gate-controlled openings in continuation of the weir with their crests at a level lower than the level of the weir crest. The under sluices are located on the same side as the offtaking canal. If there are two canals each of which offtakes from one of the banks of the river, under sluices are provided at both ends of the weir.

River Training works

River training works: -Any work constructed to contain the rivers in their specified path of flow.

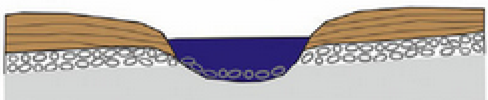
Classification of rivers:

The rivers on alluvial soils may be classified into three types:

1 .The meandering type

2. The aggrading type

3. The degrading



Objects of river training:

1. To achieve safe and expeditious passage of flow through the river.

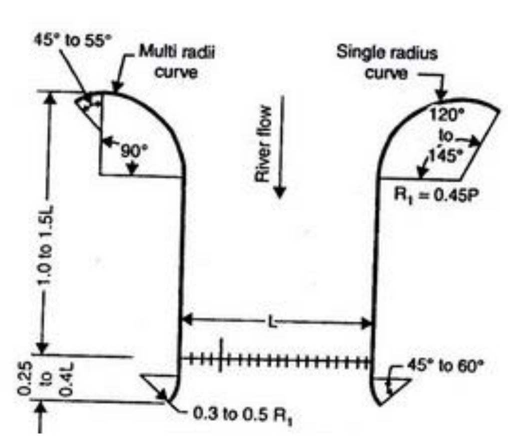
2. To achieve efficient transport of bed silt and suspended silt.

3. To achieve stable stream course with minimum bank erosion.

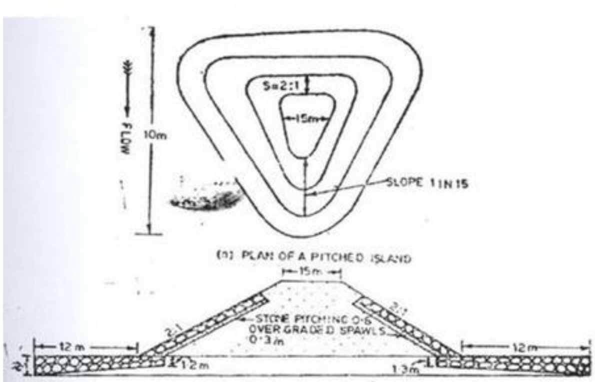
4. To achieve sufficient depth of flow for navigation.

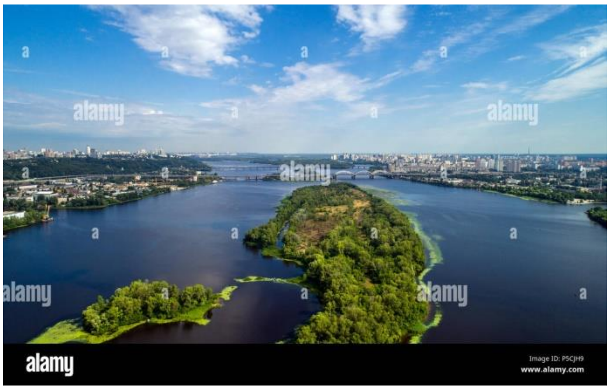
Methods of river training works :

1. Guide banks: they are used to guide the river to pass through the constrained width of the river at the structure as bridge or diversion structures.



2. Pitched Islands: it is an artificially created in the river. It may be made of masonry or earth embankment. but pitched all-round.

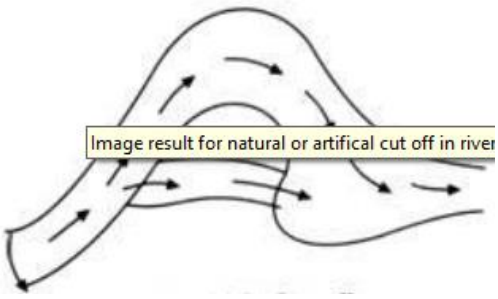




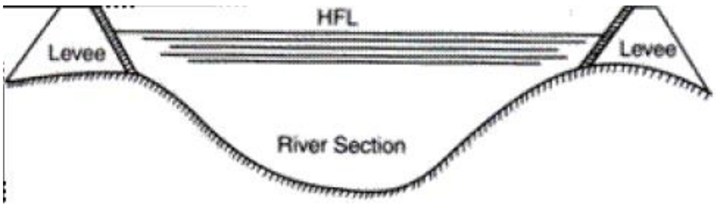
3. Spurs or Groynes : they are structures-built transverse to the river flow, extending from the bank towards the river, they perform many functions as:
   - Increase silting
   - Cause scouring
   - Deflecting the flow of water

4. Artificial and Natural cut-off: when meandering river develops very sharp horse-shoe bends, a small cut is given to connect the peaks with slope more than the slope of the river ,then

less Q will pass through the curved river and silted ,then the shape of the river will change after many years to straight river.

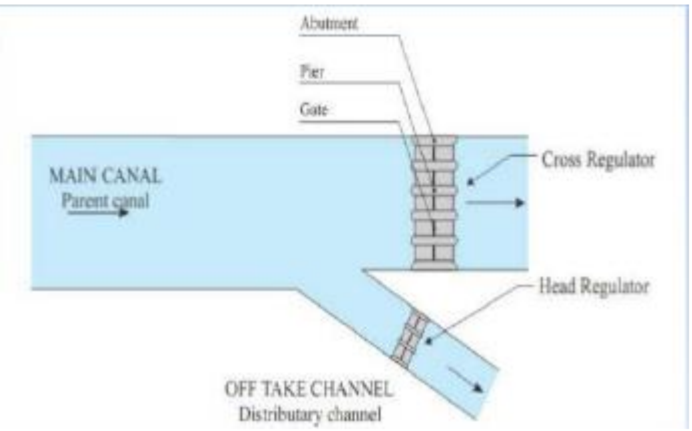


5. Retired embankments: Retired embankments are constructed at a distance from the river banks. Thus, retired embankments are the intermediate type between the case of marginal embankments and river with no embankments. Retired embankments are generally constructed on a lower ground away from the bank.



Cross and head regulator:

The supplies passing down the parent canal and off take channel are controlled by cross regulator and head regulator respectively:



Functions of Cross Regulators: -

1.Regulation of the canal system.

2.Raising the water level in the main canal in order to feed the off-take channels.

3.To facilitate communication by building a road over the cross regulator with little extra cost.

4.To absorb the fluctuations in the canal system.

Functions of Head Regulators: -

1.To regulate and control supplies entering the off-take channel (distributary) from the main(parent) canal.

2.To control silt entering into the distributary.

3.To serve for measurement of discharge.