Chapter 6
Hydraulic structures

Hydraulic structures are installed in open canal irrigation networks to:

- ❖ Control and measure discharge
- ❖ Control water levels for command requirements
- ❖ Dissipate unwanted energy
- ❖ Deliver the right volume of water to meet crop water requirements
- ❖ Incorporate recycled tail water, if available

The most common structures are:

a. Headworks for river water offtake
b. Night storage reservoirs
c. Head regulators
d. Cross regulators
e. Drop structures
f. Tail-end structures
g. Canal outlets
h. Discharge measurement structures
i. Crossings, like bridges, culverts, inverted siphons

Depending on the size and complexity of the irrigation scheme, some or all of the above-mentioned structures could be incorporated in the design.

6.1. Headworks for river water offtake

Abstraction and/or diversion of water from its source to the scheme is often difficult and can be quite costly, depending on its complexity. Figure 37 presents a sketch of schemes irrigated from different water sources.

Figure 37
Schemes irrigated from different sources (Source: FAO, 1992)

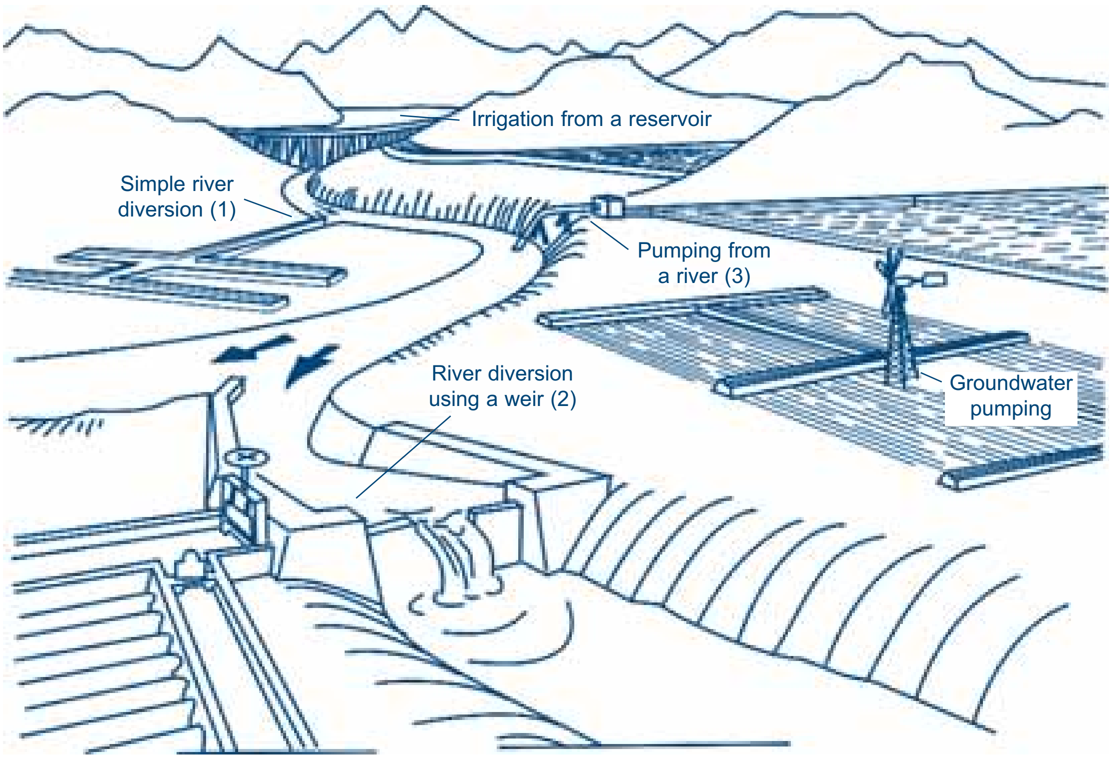
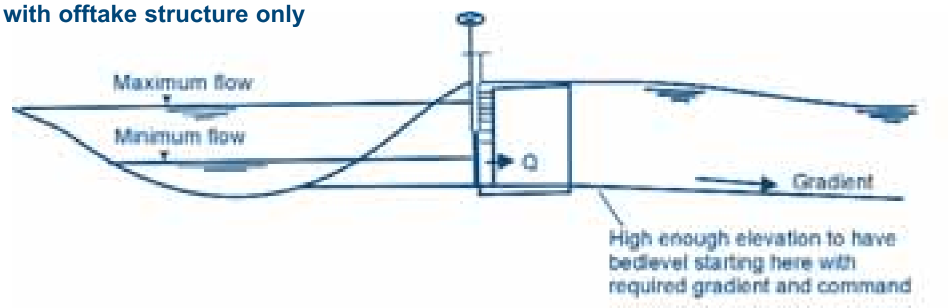


Figure 38
Headwork with offtake structure only



The function of a headwork is to divert the required amount of water at the correct head from the source into the conveyance system. It consists of one or more of the following structures:

- ❖ Offtake at the side of the river
- ❖ Regulating structure across the river or part of it
- ❖ Sediment flushing arrangement

This section concentrates mainly on the headworks for direct river offtake and offtakes using a weir. Some attention is paid to important dam and reservoir aspects, such as the outlet pipe diameter. However, for detailed dam design, the reader is referred to other specialized literature.

6.1.1. Headwork for direct river offtake

In rivers with a stable base flow and a high enough water level throughout the year in relation to the bed level of the intake canal, one can resort to run-off-river water supply (Figure 38 and Example 1 in Figure 37). A simple offtake structure to control the water diversion is sufficient.

The offtake should preferably be built in a straight reach of the river (Figure 39). When the water is free from silt, the centre line of the offtake canal could be at an angle to the centre line of the parent canal. When there is a lot of silt in the system, the offtake should have a scour sluice to discharge sediments or should be put at a 90° angle from the parent canal.

If it is not possible to build the offtake in a straight reach of the river, one should select a place on the outside of a bend, as silt tends to settle on the inside of bends. However, erosion usually takes place on the outside of the bend and therefore protection of the bank with, for example, concrete or gabions might be needed. The offtake can be perpendicular, at an angle or parallel to the riverbank, depending on site conditions, as illustrated in Figure 40.

Figure 39
Offtake possibilities in straight reach of river

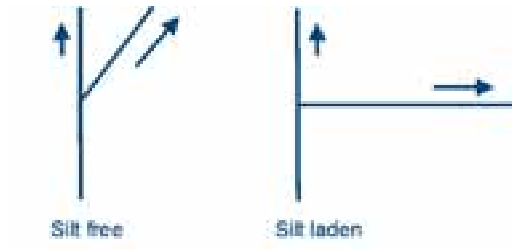


Figure 40
Possible arrangements for offtakes based on site conditions

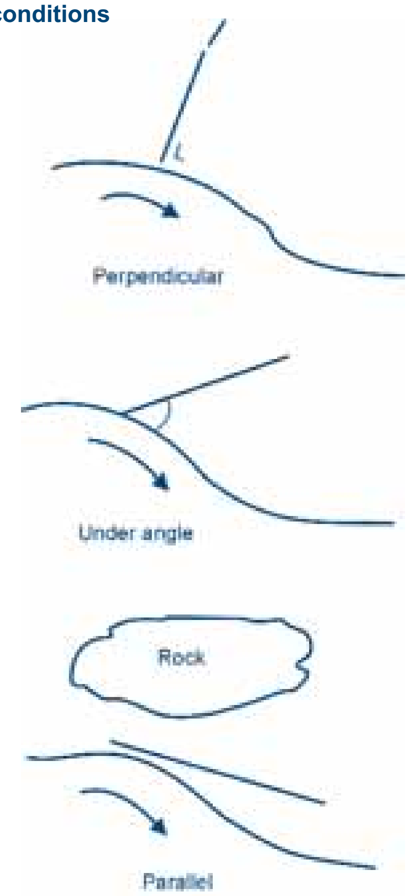
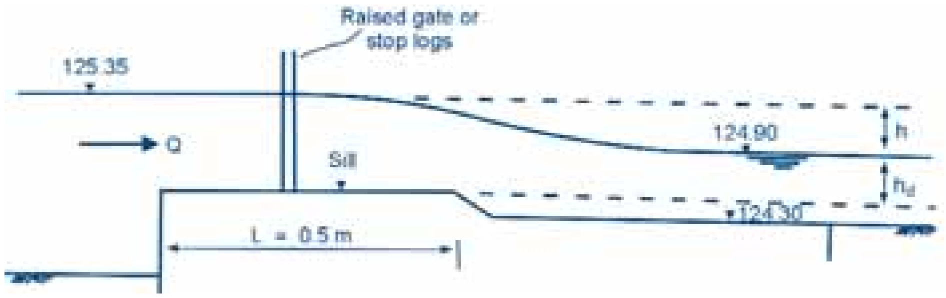


Figure 41
An example of an intake arrangement of a headwork



The functions of the offtake structures are:

- To pass the design discharge into the canal or pipeline
- To prevent excessive water from entering during flood

Considering these functions, the most important aspect of the structure is the control arrangement, which can be a gate, stop logs, or other structures. When the gate is fully opened, the intake behaves like a submerged weir (Figure 41) and its discharge is given by equation 23.

Equation 23

$$Q = C \times B(h + h_d)^{3/2}$$

Where:

Q = Discharge in intake (m^3/sec)
C = Weir coefficient
B = Width of the intake (m)
h = Difference between river water level and canal design water level (m)
h_d = Difference between canal design water level and sill level of the intake (m)

In some instances, the base flow water level fluctuates greatly over the year and the water level can become so low that the gate opening to the offtake structure will be at a higher elevation than the normal base flow water level. To abstract the required discharge in these situations, one could consider the options below:

- Select an offtake site further upstream. However, site conditions, the increased length of the conveyance canal, and other factors have to be considered carefully.
- Build a cheap temporary earthen dam and temporary diversion structure. This method is especially suitable in unstable rivers, where high expenses for a permanent structure are not warranted because of the danger of the river changing its course.
- Construct a permanent diversion dam or structure (weir or gate) across the river, where the design elevation of the weir should relate to the design water level in the conveyance canal, similar to the previous example.

6.1.2. River offtake using a weir

Figure 42 shows an example of a river diversion structure, in this case a weir (Example 2 in Figure 37).

Example 11

A discharge of 1.25 m^3/sec has to be abstracted from a river, into an open conveyance canal. The base flow water level of the river is 125.35 m. The design water level in the canal is 124.90 m and the water depth is 0.60 m. The weir coefficient is 1.60. The width of the intake is 1.50 m and the length of the weir is 0.50 m (Figure 41). What will be the sill level?

Q = 1.25 m^3/sec
C = 1.60
h = 125.35 - 124.90 = 0.45 m
B = 1.50 m

The next step would be to substitute these values in Equation 23:

$$1.25 = 1.60 \times 1.50(0.45 + h_d)^{2/3} \Rightarrow h_d = 0.20 \text{ m}$$

Thus the sill level should be at an elevation of 124.90 - 0.20 = 124.70 m

Figure 42
An example of a diversion structure

Structures constructed across rivers and streams with an objective of raising the water level are called cross regulators (see Section 6.4).

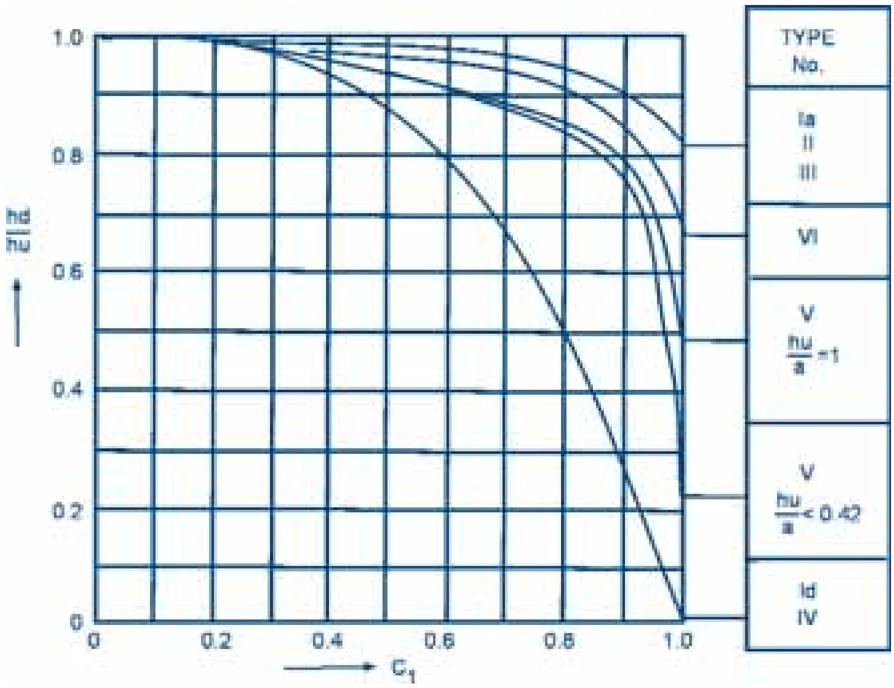
A weir should be located in a stable part of the river where the river is unlikely to change its course. The weir has to be built high enough to fulfil command requirements. During high floods, the river could overtop its embankments and change its course. Therefore, a location with firm, well-defined banks should be selected for the construction of the weir. Where possible, the site should have good bed conditions, such as rock outcrops. Alternatively, the weir should be kept as low as possible. Since weirs are the most common diversion structures, their design aspects will be discussed below.

Design of a weir for flood conditions

The weir height has to be designed to match the design water level in the conveyance canal. The weir length has to be designed to allow the design flood to safely discharge over the weir.

After deciding upon the location of the weir, the design flood, which is the maximum flood for which the weir has to be designed, has to be determined. If data are available, a flood with a return period of 50 or 100 years for example could be selected. If sufficient data are not available, flood marks could be checked, upon which the cross-sectional area can be determined and used, together with the gradient of the river, to calculate the flood discharge. Some formulae have been developed for this purpose, based on peak rainfall intensity and catchment characteristics.

Figure 43
C_1 coefficient for different types of weirs in relation to submergence, based on crest shape



The general equation for all weir types is:

Equation 24

$$Q = C_1 \times C_2 \times B \times H^{3/2}$$

Where:

Q	=	Discharge (m^3/sec)
C_1	=	Coefficient related to condition of submergence and crest shape (Figure 43)
C_2	=	Coefficient related to crest shape (Figure 44)
B	=	Weir length, i.e. the weir dimension across the river or stream (m)
H	=	Head of water over the weir crest (m)

Three general types of weirs are shown in Figure 45. The choice depends, among other aspects, on:

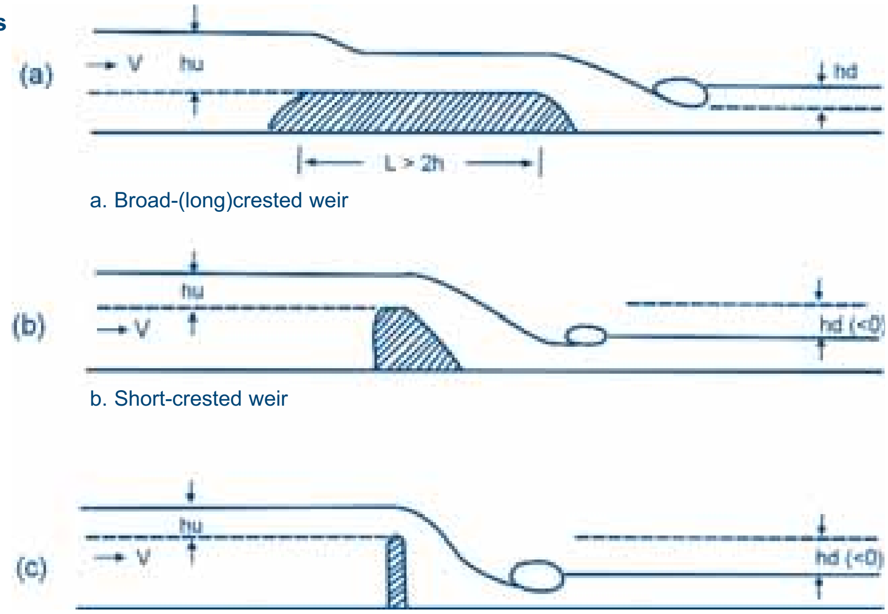
- ❖ Availability of local materials
- ❖ Available funds
- ❖ Local site conditions and floods

As an example, a broad-crested weir would be selected if gabion baskets were available as construction material (Figure 46). Gabion baskets are made of galvanized steel and look like pig netting (see Module 13). However, for the filling of gabion baskets large quantities of stones are required as well as plenty of cheap labour, since the construction method is labour intensive. Stone size is critical for a gabion weir, as large stones leave big spaces between them that allow water to quickly flow through, while too small stones may pass through the mesh.

Figure 44
C_2 coefficient for different types of weirs in relation to crest shape

TYPE No	SHAPE	$\frac{L}{hu}$	C_2
Ia Ib Ic Id	hu L	≥ 3 2 1 ≤ 0.6	1.4 1.5 1.7 1.9
II		≥ 3	1.6
III		≥ 3	2.0
IV			1.9
V	a		2.2
VI			2.3
VII	1:25 Romun		1.75

Figure 45
Types of weirs

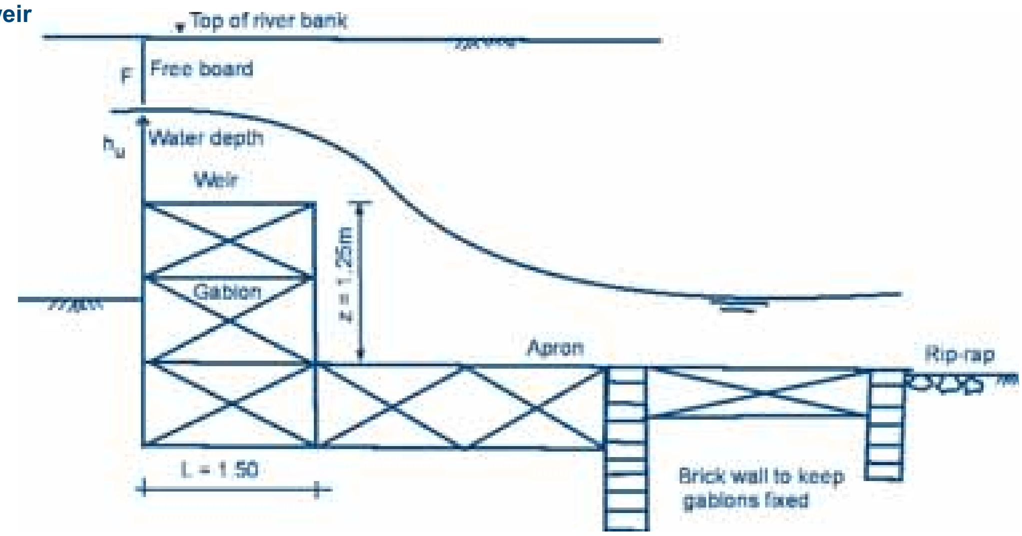


a. Broad-(long)crested weir

b. Short-crested weir

c. Sharp-crested weir

Figure 46
Gabion weir



Example 12

In Example 11 the weir coefficient C, which is the product of C_1 and C_2, was assumed to be 1.60. Can this be confirmed by calculating C_1 and C_2 respectively?

From Example 11 the difference between the water level in the river and the sill elevation can be calculated as follows:

$h_u = 125.35 - 124.70 = 0.65m$

The weir length L is 0.50 m, thus $\frac{L}{h_u}$ in Figure 44 is: $\frac{L}{h_u} = \frac{0.50}{0.65} = 0.77$

This relates to a weir type between 1c and 1d in Figure 44. By interpolation, C_2 is approximately 1.8.

The difference between the canal design water level and the sill elevation h_d = 0.20 m.

Thus $\frac{h_d}{h_u}$, which is the y-axis in Figure 43, is: $\frac{h_d}{h_u} = \frac{0.20}{0.65} = 0.31$

Using the curve for weir type 1b-d in Figure 43, gives a value for C_1 of approximately 0.9.

Thus $C = C_1 \times C_2 = 0.9 \times 1.8 = 1.62$, which is almost the same as the weir coefficient 1.60 used in Example 11.

Example 13

A broad-crested weir is to be constructed with gabion baskets. The top width L, which is the dimension of the weir in the direction of the river, is 1.50 m. There will be non-submerged conditions, which means that the water level downstream of the weir will be below the weir crest. The design discharge is 37 m^3/sec. Due to local site conditions, the head of water over the crest should not exceed 0.75 m. The freeboard (F), which is the distance between the design level of the water and the top of the river bank is 0.70 m (Figure 46). What should be the weir length or the dimension of the weir across the river?

The first step is to determine the values of C_1 and C_2 from Figures 43 and 44 respectively:

h_d is the distance from the crest of the weir to the design water level downstream of the weir (Figure 41). Since there will be non-submerged conditions, h_d will be below the crest of the weir. This means that h_d is 0.

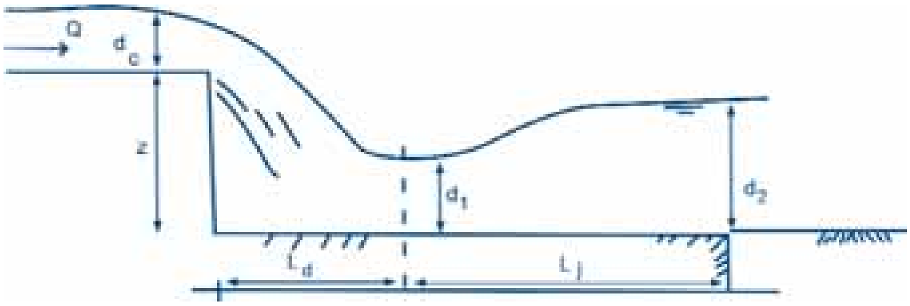
As a result $\frac{h_d}{h_u} = 0$, thus $C_1 = 1$ (Figure 43)

L = 1.50 m and h_u = 0.75 m, thus $\frac{L}{h_u} = 2$, which means that $C_2 = 1.5$, which is weir type 1b in Figure 44.

Substituting the above data in Equation 24 gives:

$37 = 1.0 \times 1.5 \times B \times 0.75^{3/2} \Rightarrow B = 38$ m.

Figure 47
Typical parameters used in the design of a stilling basin



The downstream side of the weir has to be protected using a stilling basin to dissipate the energy of the dropping water. It could be constructed using masonry, concrete, gabions or Reno mattresses.

Design of a stilling basin

The length of the stilling basin should be correctly determined in order to avoid bed scour and the subsequent undermining of the structure. The parameters used in the design of a stilling basin are shown in Figure 47.

The empirical formulae to use for the design of a stilling basin (apron) are:

Equation 25

$$D = \frac{q^2}{(g \times z^3)}$$

Equation 26

$$\frac{L_d}{z} = 4.30 \times D^{0.27}$$

Equation 27

$$\frac{d_1}{z} = 0.54 \times D^{0.425}$$

Equation 28

$$\frac{d_2}{z} = 1.66 \times D^{0.27}$$

Equation 29

$$L_j = 6.9 \times (d_2 - d_1)$$

Where:

- D = Drop number (no limit)
- q = Discharge per metre length of the weir (m^2/sec)
- g = Gravitational force (9.81 m/sec^2)
- z = Drop (m)
- L_d = Length of apron from the drop to the point where the lowest water level d_1 will occur (hydraulic jump) (m)
- d_1 = Lowest water level after the drop (m)
- d_2 = Design water level after the apron (m)
- L_j = Length of apron from the point of lowest water level to the end of the apron (m)

Example 14

A weir with a length B of 38 m across the river and a design discharge Q of 37 m³/sec, has a design drop z of 1.25 m. What will be the apron length?

The unit discharge is $\frac{37}{38}$ = 0.974 m^3/sec per metre length of weir.

Substituting this value and the drop z in Equations 25 to 29 for the design of a stilling basin dimension gives:

$$D = \frac{0.974^2}{(9.81 \times 1.25^3)} = 0.05$$

$$\frac{L_d}{1.25} = 4.30 \times 0.05^{0.27} \Rightarrow L_d = 2.40 \text{ m}$$

$$\frac{d_1}{1.25} = 0.54 \times 0.05^{0.425} \Rightarrow d_1 = 0.19 \text{ m}$$

$$\frac{d_2}{1.25} = 1.66 \times 0.05^{0.27} \Rightarrow d_2 = 0.92 \text{ m}$$

$$L_j = 6.9 \times (0.92 - 0.19) = 5.04 \text{ m}$$

Thus the total apron length is $(L_d + L_j)$ = 2.40 + 5.04 = 7.44 m

Apron floors should have sufficient thickness to counter-balance the uplift hydrostatic pressure and should be sufficiently long to prevent piping action. This is responsible for the removal of the bed material from under the floor, thereby causing its collapse.

Bed material that allows uplift is liable to piping. Piping could be avoided by using sheet piling, which is a method whereby metal or wooden posts are driven vertically into the ground until they reach an impermeable sub-layer. However, this is expensive. Alternatively, horizontal, impermeable layers could be provided. By applying Lane's weighted-creep theory, which is an empirical, but simple and proven method, the length can be determined. This is defined by the following terms:

- The weighted-creep distance L_W of a cross-section of a weir or a dam is the sum of the vertical creep distances (steeper than 45°) plus one-third of the horizontal creep distances (Equation 30).
- The weighted-creep ratio is the weighted-creep distance (L_W) divided by the effective head on the structure, which in this case is the drop (z) (Equation 31).
- The upward pressure may be estimated by assuming that the drop in pressure from headwater to tail water along the line of contact of the foundation is proportional to the weighted-creep distance (Equation 32).

Figure 48 shows the different heights and lengths to be used in determining the weighted-creep ratios.

The weighted-creep distance is as follows:

Equation 30

$$L_w = h_1 + h_2 + h_3 + h_4 + h_5 + \frac{1}{3}(W_1 + L_1 + L_2 + W_2)$$

The weighted-creep ratio is formulated as follows:

Equation 31

$$\frac{L_w}{z}$$

For designing the floor thickness, the uplift pressure P has to be estimated. The uplift pressure at point B of Figure 48 is calculated as follows:

Equation 32

$$P_b = z - \frac{h_1 + h_2 + h_3 + \frac{1}{3}(W_1 + L_1)}{L_w}$$

The thickness of a floor can be determined using the following equation:

Equation 33

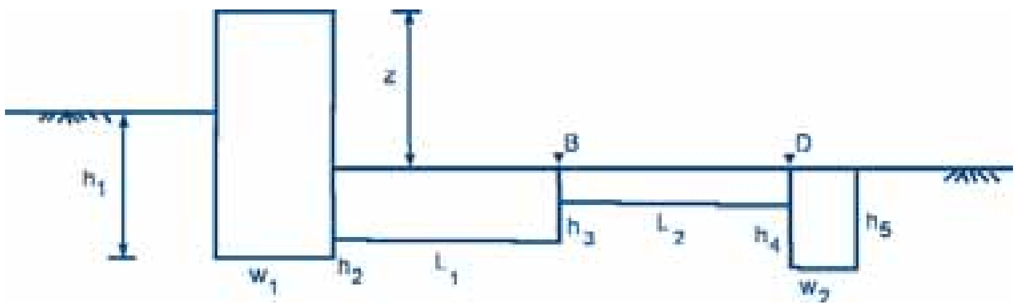
$$t = \text{Uplift pressure x } \frac{\text{Unit weight of water}}{\text{Unit weight of submerged mansonry}}$$

The recommended weighted-creep ratios are given in Table 25.

Table 25
Weighted-creep ratios for weirs, depending on soil type

Bed materials	Weighted-creep ratio
Medium sand	6
Coarse sand	5
Fine gravel	4
Medium gravel	3.5
Coarse gravel	3.0
Boulders with gravel	2.5
Medium clay	2

Figure 48
Schematic view of a weir and apron



Thus, for a given type of bed material, the weighted-creep ratio can be brought within the recommended value by selecting a suitable combination of the floor lengths and vertical cut-offs, as given in Table 25. The materials being used often determine the cut-off walls, though the apron length should also be long enough to dissipate the energy.

Example 15

A masonry weir (Figure 49) has to be built in a coarse sand bed material. The proposed dimensions are as follows (Figure 48):

z	=	1.25 m	h_5	=	1.00 m
h_1	=	1.00 m	W_1	=	1.50 m
h_2	=	0.25 m	L_1	=	3.50 m
h_3	=	0.50 m	L_2	=	3.00 m
h_4	=	0.50 m	W_2	=	1.00 m

Would this structure be safe against piping?

The weighted-creep distance L_w is:

$$L_w = 1.00 + 0.25 + 0.50 + 0.50 + 1.00 + \frac{1}{3}(1.50 + 2.50 + 1.00) = 6.25 \text{ m}$$

The weighted creep ratio is:

$$\frac{L_w}{z} = \frac{6.25}{1.25} = 5.0$$

Comparing this value to the recommended one given for coarse sand in Table 25 shows that the selection of the structure dimensions is acceptable and that no piping should be expected.

Example 16

Using the same data as given in the previous example, calculate the floor thickness at points B and D (Figure 48).

Assuming a masonry floor, the unit dry weight can be taken as 2 400 kg/m^3, while the unit weight of water is 1 000 kg/m^3. The submerged weight of masonry is the difference between the unit dry weight of the masonry minus the unit weight of water, thus: (2 400 – 1 000) = 1 400 kg/m^3.

The uplift pressure at point B is:

$$P_b = 1.25 - \frac{1.00 + 0.25 + 0.50 + \frac{1}{3}(1.50 + 3.50)}{6.25} = 0.70 \text{ m}$$

Thus the required floor thickness at point B is:

$$t = 0.70 \times \frac{1\,000}{1\,400} = 0.50 \text{ m}$$

Similarly for point D, the uplift pressure is:

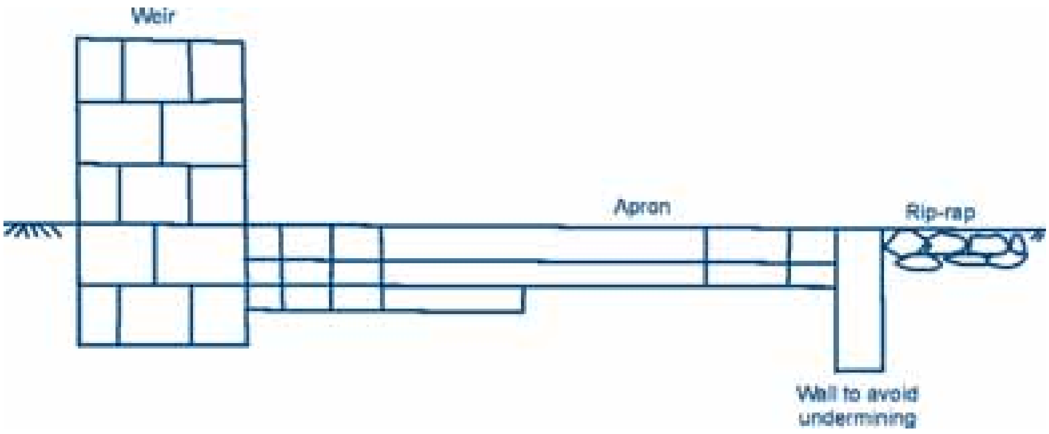
$$P_b = 1.25 - \frac{1.00 + 0.25 + 0.50 + 0.50 + \frac{1}{3}(1.50 + 3.50 + 3.00)}{6.25} = 0.46 \text{ m}$$

The required floor thickness at point D is:

$$t = 0.46 \times \frac{1\,000}{1\,400} = 0.33 \text{ m}$$

As there is a drop in pressure from the head to the tail of the structure, the floor thickness can be less at point D, which is further away from the head of the structure.

Figure 49
Masonry weir and apron



6.1.3. River offtake using a dam

Design of dams will not be discussed in this Irrigation Manual. For this the reader is referred to other specialized literature available. In this section only those aspects of dams that affect irrigation designs will be discussed. A typical dam cross-section is given in Figure 50.

Lowest drawdown level

The lowest drawdown level is the minimum water level in the reservoir that can be abstracted into the irrigation system. The water remaining below the lowest drawdown level is called dead storage. This could be used as drinking water for human beings and animals. The lowest drawdown level often coincides with the latter part of the dry season, when water requirements are high. Even at the lowest draw down level one would like to abstract the design discharge.

A simple relation between the discharge, the difference in height between lowest drawdown level and outlet pipe invert and outlet pipe diameter can be obtained using Equation 34:

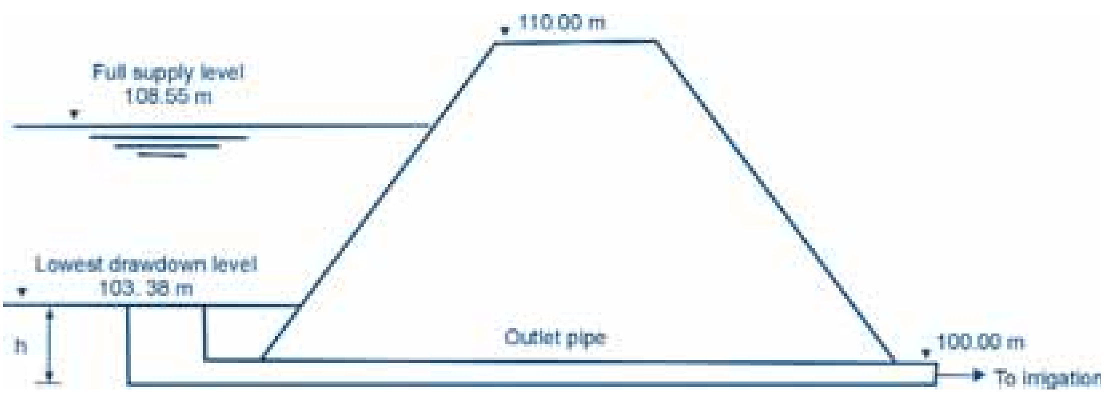
Equation 34

$$Q = C \times A \times \sqrt{2gh}$$

Where:

Q = Discharge (m^3 /sec)
C = Discharge coefficient, approximately 0.5
A = Cross-sectional area of pipe (m^2)
g = Gravitational force (9.81 m/sec^2)
h = Available head (m)

Figure 50
Dam cross-section at Nabusenga



Example 17

The lowest drawdown level of Nabusenga dam is 3.38 m above the outlet pipe (Figure 50). The pipe has a diameter of 225 mm. Would this outlet pipe be able to deliver a discharge of 78.3 l/sec?

Substituting these data in Equation 34 gives:

$$78.3 \times 10^{-3} = 0.5 \times \left(\frac{1}{4} \times \pi \times 0.225^2\right) \times (2 \times 9.81 \times h)^{1/2} \Rightarrow h_{required} = 0.79 \text{ m}$$

The minimum required head of 0.79 m is much less than the $h_{available}$ of 3.38 m. Thus no reduction in discharge should be expected when the water level in the dam reaches its minimum level.

Example 18

Using the data from Example 17, what would be the outlet pipe diameter if a discharge of 78.3 l/sec has to be abstracted at a minimum available head of 1.25 m instead of 3.38 m?

Substituting the data in Equation 34 gives:

$$78.3 \times 10^{-3} = 0.5 \left(\frac{1}{4} \times \pi \times d^2\right) \times (2 \times 9.81 \times 1.25)^{1/2} \Rightarrow d = 200 \text{ mm}$$

Friction losses in outlet pipe

There should be sufficient head available to overcome friction losses in the outlet pipe as well as in the conveyance pipeline in case the conveyance system is a pipe and not an open canal. The available head refers to the water height above the outlet pipe. The friction losses (HL) through a pipe can be calculated using the Hazen-Williams equation, which was given in Equation 21 (see Section 5.2.2):

$$Hf_{100} = \frac{K \times \left[\frac{Q}{C}\right]^{1.852}}{D^{4.87}}$$

The value of the material constant C depends on the smoothness of the material (Table 24). If the pipe size is small in relation to the discharge, high friction losses are expected, which means that the water head above the pipe outlet should be large. In such a case, the discharge would be reduced at a lower drawdown level than for a larger pipe.

6.1.4. Scour gates for sedimentation control

Many rivers carry substantial sediment loads, especially during the rainy season, in the form of sand, silt, weeds, moss and tree leaves. Approximately 70% of all suspended and bed load sediments travel in the lower 25% of the flow profile.

Example 19

The Nabusenga dam has a 70 m long AC outlet pipe with a diameter of 225 mm. What are the friction losses for discharges of 78.3 l/sec and 32.6 l/sec, including 20% extra for minor losses?

Substituting the above data in Equation 21 gives for Q = 78.3 l/sec:

$$Hf_{100} = \frac{1.22 \times \left[\frac{78.3}{140}\right]^{1.852}}{225^{4.87}} = 1.46 \text{ m per 100 m or HL} = 1.02 \text{ m per 70 m} + 20\% = 1.22 \text{ m}$$

For Q = 32.6 l/sec:

$$Hf_{100} = \frac{1.22 \times \left[\frac{32.6}{140}\right]^{1.852}}{225^{4.87}} = 0.29 \text{ m per 100 m or HL} = 0.20 \text{ m per 70 m} + 20\% = 0.24 \text{ m}$$

Since the minimum head available is 3.38 m (Example 17 and Figure 50) no reduction in discharge is expected, even if the full discharge of 78.3 l/sec has to be delivered.

Example 20

What are the friction losses for a discharge of 78.3 l/sec through a 70 m long galvanized steel pipeline with a 200 mm diameter? The minimum available head is 1.25 m.

$$Hf_{100} = \frac{1.22 \times \left[\frac{78.3}{140}\right]^{1.852}}{200^{4.87}} = 3.44 \text{ m per 100 m or HL} = 2.41 \text{ m per 70 m} + 20\% = 2.89 \text{ m}$$

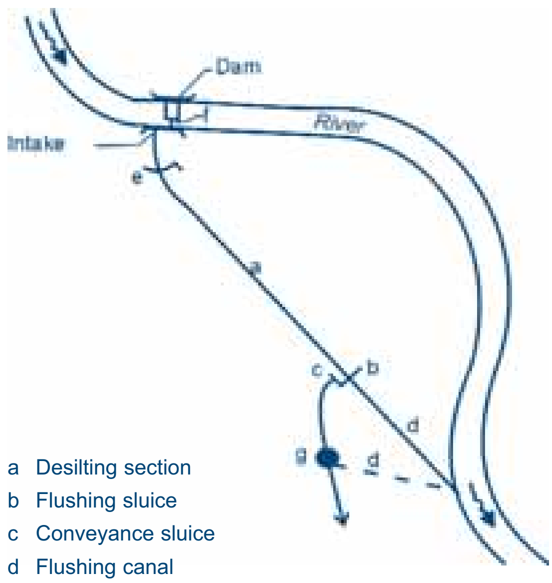
Already at 1.64 m (2.89 - 1.25 m) above the minimum drawdown level, the discharge will be reduced as the water head is insufficient to overcome the friction losses of the design discharge.

It should be noted that with aging the C for galvanized steel pipes drops to 80. This will further increase the head losses in the pipe.

While suspended silt can be beneficial to the scheme by adding nutrients to the farmland, coarse sediments usually cause problems once they are blocked by a weir or other diversion structure. Headworks have to be adapted to these sediment loads to avoid silting of canals and structures. A properly-designed intake should divert only the relatively clean upper part of the water flow into the canal and dispose of the lower part down the river. A sluice should therefore be incorporated into the diversion structure design. It should be placed in line with the weir near the canal intake (Figure 51). Its seal level is generally placed at the river bed level while the floor to the intake gate should be located higher (Figure 52).

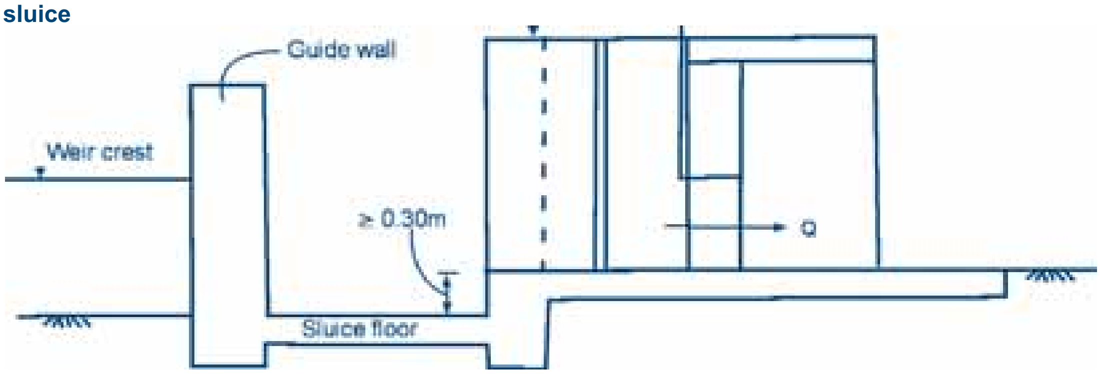
The control arrangement in the scour sluice generally consists of a series of stop logs (timber, concrete) or a sluice gate. This arrangement allows the water to be raised when there are very few or no sediments in the water. During the flood season, the sluice is permanently open or opened at regular intervals so that depositions of sediments can be flushed away. The guide wall prevents lateral movement of sediments deposited in front of the weir and separates the flow through the sluice and the flow over the weir.

Figure 51
Gravity offtake with diversion dam



- a Desilting section
- b Flushing sluice
- c Conveyance sluice
- d Flushing canal
- e Gravel and sand trap
- f Flushing gate
- g Regulation basin

Figure 52
Scour sluice



6.2. Night Storage Reservoirs (NSR)

Night storage reservoirs (NSR) store water during times when there is abstraction from the headwork but no irrigation. Depending on the size of the scheme one could construct either one reservoir, located at the top of the scheme as shown in Figure 19, or more than one to command sections of the scheme they are serving.

Night storage reservoirs could be incorporated in the design of a scheme when:

i. The distance from the water source to the field is very long, resulting in a long time lag between releasing water from the source and receiving it in the field.

ii The costs of constructing the conveyance canal or pipeline are very high because of the large discharge it has to convey without a NSR. Incorporating a reservoir means that a smaller size conveyance system can be built.

iii. The discharge of the source of the water is smaller than would be required for the area without storing the water during times of no irrigation.

The following examples illustrate scenarios i to iii.

Example 21

A discharge of 78.30 l/sec has to be delivered through a 7 km long canal with a wetted cross-section of 0.19 m^2. When should the headwork gate be opened, if water has to reach the field at 07.00 hours?

The water velocity (V) was given by the Continuity Equation 12:

$$V = \frac{Q}{A}$$

Substituting the values in Equation 12 gives:

$$V = \frac{0.0783}{0.19} = 0.41 \text{ m/sec}$$

The time (t) it takes for the water to reach the top of the field is given:

$$t = \frac{\text{distance}}{\text{velocity}} = \frac{7\ 000}{0.41} = 17\ 073 \text{ seconds or 4 hours and 45 minutes.}$$

This would mean that the head gate should be opened at 02.15 hours if irrigation is to start at 07:00 hours. If this is unsuitable for proper management, one should incorporate a night storage reservoir.

Example 22

Water is abstracted from a river with a base flow of less than 78.3 l/sec (required if the delivery period is 10 hours per day), but more than 32.6 l/sec (required if the delivery period can take place 24 hours per day). If abstraction only takes place during daytime the area under irrigation would have to be reduced. Determine the size of the reservoir for the scheme in order to be able to irrigate the whole area.

With a night storage reservoir, one could collect the required discharge of 32.6 l/sec from the water source. At an abstraction rate of 32.6 l/sec, the volume of water accumulated during the 14 hours when there is no irrigation should be stored in the night storage reservoir. Thus the volume (V) to be stored is:

$$V = \frac{32.6 \times 3\ 600 \times 14}{1\ 000} = 1\ 643 \text{ m}^3$$

If 20% is added to cater for evaporation and seepage losses, a night storage reservoir with a capacity of 1 970 m^3 could be proposed.

Example 23

If the friction losses in a conveyance or supply pipeline, delivering 78.3 l/sec for a period of 10 hours per day, are kept at around 0.30 m per 100 m, then a 300 mm diameter AC pipe (Class 18) could be used (Figure 35). What pipe size could be used, if a night storage reservoir were built allowing a water flow 24 hours per day?

If abstraction could take place for 24 hours per day, then the discharge would reduce to 32.6 l/sec (= 78.3/(24/10)) and subsequently a pipe size of 225 mm could be selected, considering the same friction loss of 0.30 m per 100 m.

The need for a night storage reservoir should be carefully considered, weighing advantages, such as money saving in water delivery works, against disadvantages, such as cost of reservoir construction, maintenance, seepage and evaporation losses and disease vector control costs.

6.2.1. Types of reservoirs

Reservoirs can be classified on the basis of:

- The material used in construction, such as bricks, concrete or earth
- Their shape, which can be circular, square or rectangular

Earthen reservoirs

Earthen reservoirs are the most common, as they are usually cheaper to construct. Figure 53 shows a design of a typical square earthen reservoir, including the inlet, the outlet and the spillway. The embankments should be well compacted. If the original soils are permeable, a core trench should be dug and filled up with less permeable soils.

Circular reservoirs

A circular reservoir is the common shape of a concrete or brick reservoir. It is the most economical, as the perimeter of a circle is smaller than the perimeter of a square or rectangle for the same area. It also does not need heavy corner reinforcement to resist the water pressure, as do square or rectangular reservoirs. The formula for the calculation of the volume (V) of a circular reservoir is:

Equation 35

$$V = \frac{1}{4}\pi d^2 h$$

Where:

V = Volume of reservoir (m^3)
d = Diameter (m)
h = Water depth (m)

Example 24

If the reservoir in Example 23 has water depth, h, of 2.0 m (the maximum recommended depth for brick reservoirs), what would be the required diameter for the reservoir?

Using Equation 35:

$$1\ 970 = \frac{1}{4} \times 3.14 \times d^2 \times 2 \Rightarrow d = 35.42\text{ m}$$

The best site for a reservoir is on a flat area with firm, uniform soils. It is not recommended to build a reservoir on made-up ground, unless the compaction is extremely well done.

6.2.2. Reservoir components

Foundation and floor

A foundation of 450-600 mm in width and 225-300 mm in depth for a 250 mm wall thickness should be adequate for circular reservoirs on firm, solid ground. Normally, foundations do not need reinforcement, except when placed on unstable soils.

A floor thickness of 100 mm should be adequate. Often a reinforcement grid with 200-300 mm interval is placed in the concrete floor. Joints, meant to control cracking, are placed in the reservoir floor and the reinforcement grid should not cut across these joints. The concrete panels should not exceed 6 m in either length or width. Long narrow panels should be avoided.

Reinforcement

The pressure of the water in a circular reservoir produces tension in the wall. Pressure exerted by the water is directly proportional to the water head (depth) from the surface to the depth considered. Tension produced in the wall of a circular reservoir is directly proportional to the water depth and the diameter of the reservoir.

The tension is taken up, to some degree, by the material of the wall. However, concrete and bricks are weak in tension, therefore reinforcement should be provided. In a

Figure 53
Design of a typical earthen night storage reservoir

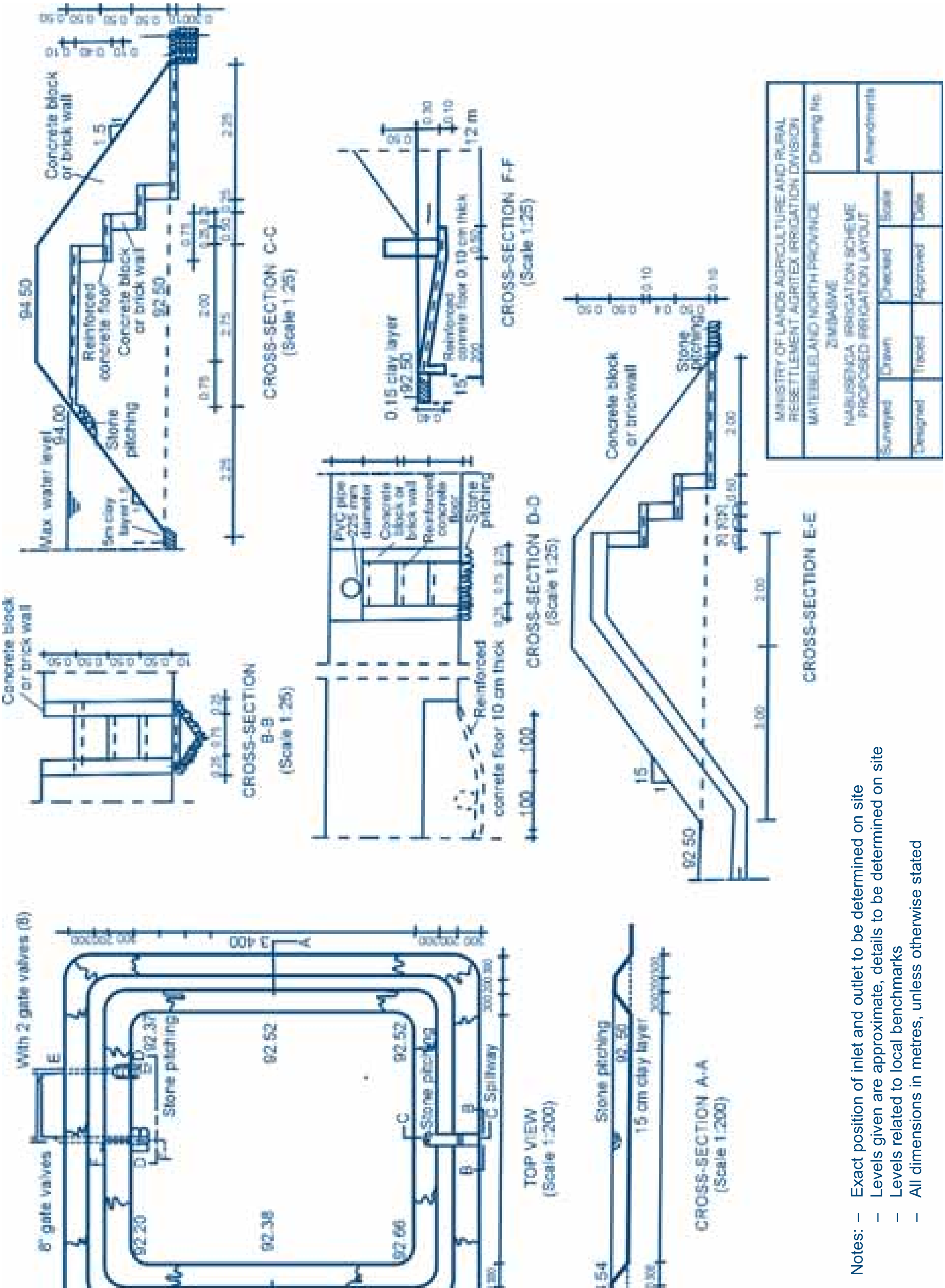
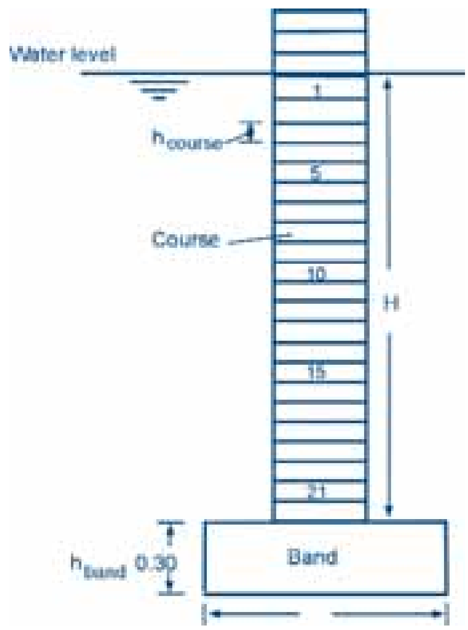


Figure 54
Courses in brick wall of a reservoir



brick or concrete block wall the reinforcement rods are best placed within the mortar of the horizontal joints between courses. A course is a continuous level line of bricks or stones in a wall (Figure 54). A simple formula to calculate the cross-sectional area (A) of reinforcement needed per course is:

Equation 36

$$A = 44.6 \times d \times H \times h$$

Where:

A	=	Cross-sectional area of reinforcement needed for the course or band under consideration (mm^2)
d	=	Diameter of the reservoir (m)
H	=	Distance down from the design water level to the bottom of the course (m)
h_{course}	=	Height of a course (m)
h_{band}	=	Height of band (m)

The band for a concrete wall is assumed to be 300 mm. The course for a brick wall is 90 mm (including 15 mm for the mortar) and the course for a concrete block is assumed to

Example 25

What are the reinforcement requirements for a brick reservoir of 2 m high and 36 m in diameter, with a wall as shown in Figure 54?

The calculations of the steel rod requirements for courses 1, 5 and 21 are given below. Calculations for all other courses are similar and summarized in Table 26. Cross-sectional areas of the different rod sizes are given in Table 27.

Using Equation 36:

Course 1: $A = 44.6 \times 36 \times (1 \times 0.09) \times 0.09 = 13\ mm^2$ ⇒ 1 steel rod of 4 mm diameter is required

Course 5: $A = 44.6 \times 36 \times (5 \times 0.09) \times 0.09 = 65\ mm^2$ ⇒ 3 steel rods of 6 mm diameter are required

Course 21: $A = 44.6 \times 36 \times (21 \times 0.09) \times 0.09 = 273\ mm^2$ ⇒ 6 steel rods of 8 mm diameter or 4 steel rods of 10 mm diameter are required

Table 26
Reinforcement requirements in a clay brick wall of a reservoir

Course	Reinforcement (mm^2)	Number of rods and diameter	Course	Reinforcement (mm^2)	Number of rods and diameter
1	13.0	1 x 4 mm	12	156.1	3 x 8 mm
2	26.0	1 x 6 mm	13	169.1	4 x 8 mm
3	39.0	2 x 6 mm	14	182.1	4 x 8 mm
4	52.0	2 x 6 mm	15	195.1	4 x 8 mm
5	65.0	3 x 6 mm	16	208.1	5 x 8 mm
6	78.0	3 x 6 mm	17	221.1	5 x 8 mm
7	91.0	4 x 6 mm	18	234.1	5 x 8 mm
8	104.0	4 x 6 mm	19	247.1	5 x 8 mm
9	117.0	5 x 6 mm	20	260.1	6 x 8 mm
10	130.1	5 x 6 mm	21	273.1	6 x 8 mm
11	143.1	5 x 6 mm			

Note :It is not recommended to have more than 8 steel bars in a course for a wall thickness of 250 mm. If need be, then it would be necessary to use smaller diameter bars.

be 160 mm high (including 20 mm for the mortar). The height could differ, depending on the actual block or brick used and should be confirmed on site.

Table 27
Cross-sectional areas of reinforcement steel rods

Diameter of rod (mm)	Cross-sectional area $= \frac{1}{4} \pi d^2$ (mm^2)
4	12.6
6	28.3
8	50.3
10	78.5
12	113.1

Pipe requirements

The main pipe requirements are:

- A supply pipe for filling the reservoir
- An outlet pipe
- An overflow pipe
- A scour pipe

The supply pipe generally discharges into the reservoir over the wall from the outside, although it could also be brought under the foundation and up through the floor. The pipe should have a gate valve so that supplies can be shut off when necessary. The diameter depends on the design discharge.

The outlet pipe may be installed into the reservoir wall about 150 mm above floor level or under the foundation and up through the floor. By positioning the pipe above floor level, sludge and sediments are prevented from entering the delivery system. The pipe should be fitted with a screen as a precaution against blockage. The diameter of the outlet pipe depends on the design discharge and the available head. This pipe should also have a gate valve to be able to shut it off.

An overflow pipe should be installed in the wall with its bottom at the same height as the full supply level of the reservoir. A 100 mm diameter pipe usually suffices.

A scour pipe should be provided at a level slightly below floor level, so that the reservoir can be regularly cleaned of sediments. It could have a 100 mm diameter and should be fitted with a gate valve.

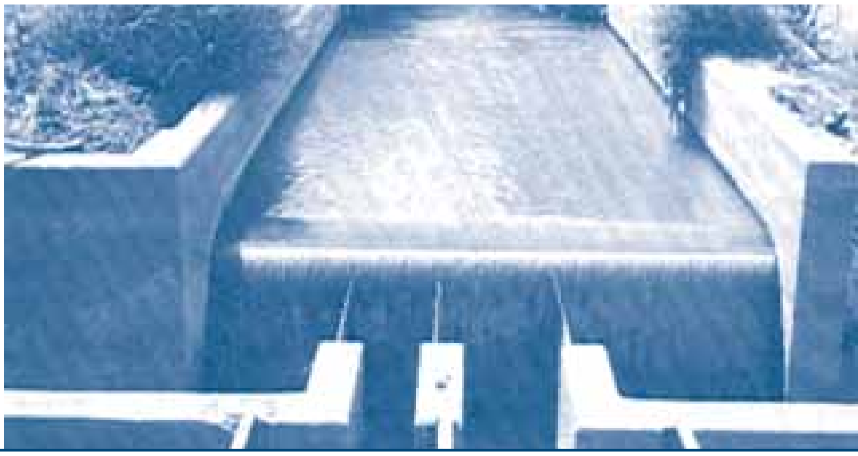
6.3. Head regulators

A head regulator is a structure used to control, and usually also to measure, the discharge of water into the irrigation system. It should be designed in such a way that head losses are kept as low as possible.

On large schemes needing large quantities of water, head regulators can be very large and would usually be built in concrete. The use of concrete will result in strong structures, but can be expensive. The thickness of the floor and the walls should be between 10 cm and 15 cm. A cheaper structure could be a concrete block or brick structure, which would be suitable for smaller structures with low walls. Wooden diversion structures could be used where discharges are less than 200 l/sec. In small schemes, concrete blocks, bricks or even stones could be used to build the regulators. In this case, they have manual lifting gates or moveable weirs.

In addition to the headwork described in Section 6.1, head regulators could also be located at the top of any canal in the scheme, for example a secondary canal or even a

Figure 55
A simple in-situ concrete proportional flow division structure (Source: Jensen, 1983)

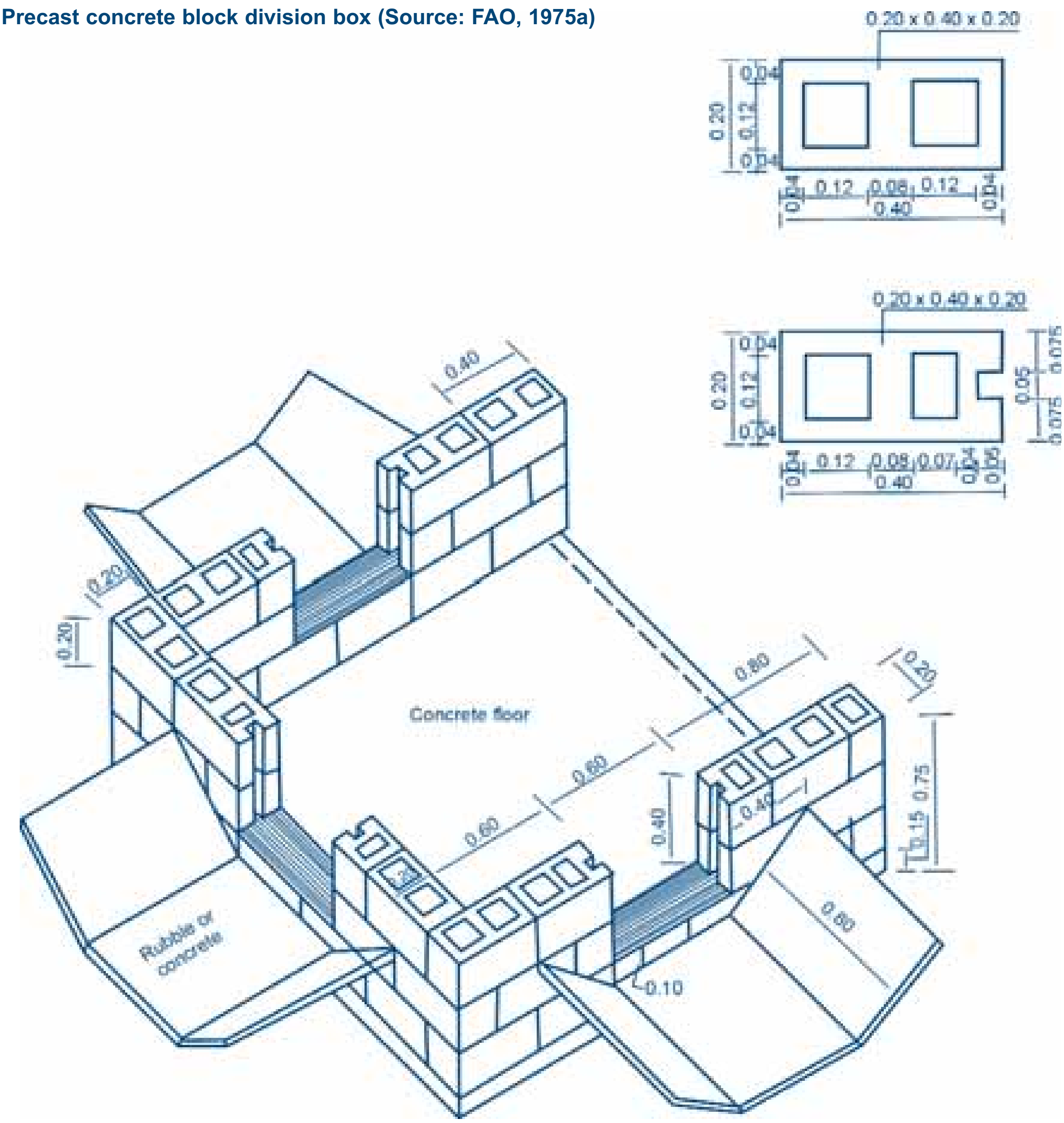


tertiary canal. In these cases, the head regulator is usually called a diversion structure. Weir-type diversion structures have been discussed in Section 6.1. Below diversion structures as regulating structures in general will be discussed.

A diversion structure regulates the flow from one canal into one or more other canals. It normally consists of a box with vertical walls in which controllable openings are provided. The minimum dimensions of the structure depend on its performance in the fully open position. The width of the outlet is usually proportional to the division of water flow to be made. Figures 55-57 show some examples of diversion structures (in-situ concrete, pre-cast concrete and timber structures respectively). The walls can be made either of concrete (10-15 cm thick precast or in-situ) or masonry or even wood.

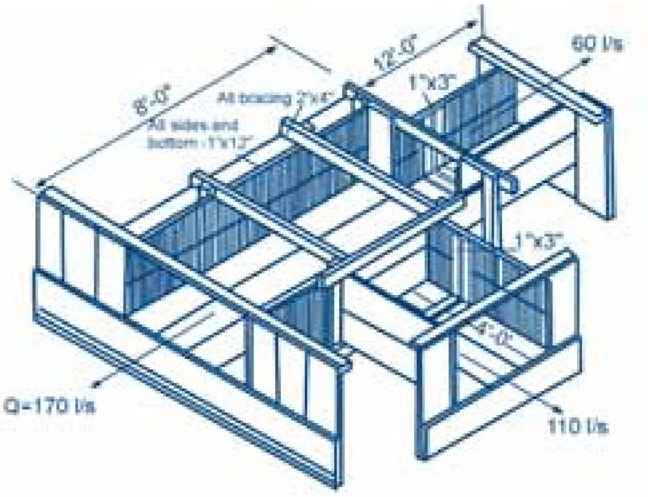
Large backup of water upstream of the structure, which would result in overtopping of the canal, should be avoided. Since a lined canal is designed to carry water at relatively high velocities, a full gate-opening at the intake to the box, covering approximately the same area as the canal section, should be provided. In earthen canals, gate-opening dimensions can be based on assuming velocities of less than 1.0-1.5 m/sec.

Figure 56
Precast concrete block division box (Source: FAO, 1975a)

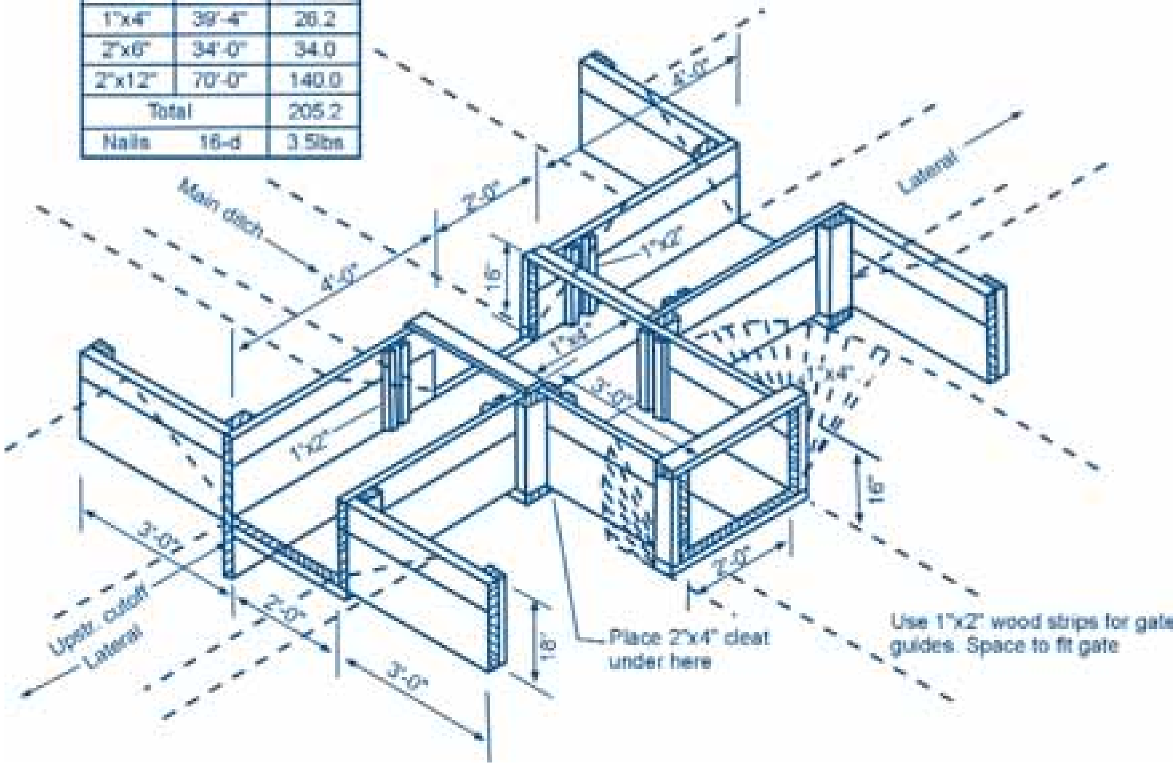


All dimensions are in metres

Figure 57
Timber division structures (Source: FAO, 1975a)



Bill of Materials		
Size	L-F	B-F
1"x4"	15'-4"	5.0
1"x4"	39'-4"	26.2
2"x6"	34'-0"	34.0
2"x12"	70'-0"	140.0
Total		205.2
Nails	16-d	3.5lbs



Example 26

What should be the minimum width of the opening b of a diversion structure, if the discharge Q is equal to 78.3 l/sec and if the water depth in the opening is not to exceed 0.30 m?

Using Equation 12:

$Q = A \times V = 0.30 \times b \times V$

Assuming a $V_{max} = 1.50$ m/sec for concrete lining and substituting it into Equation 12 gives:

$0.00783 = 0.30 \times b \times 1.50 \Rightarrow b_{min} = 0.18$ m

Example 27

What should be the water depth over a weir crest, if the discharge Q is equal to 78.3 l/sec, the weir length B is equal to 0.40 m and C is equal to 1.75?

Using Equation 24:

$Q = C \times B \times H^{3/2} \Rightarrow H = 0.23$ m

The structure should be designed in such a way that the water velocity will not cause erosion in the earthen canal. Thus, the water velocity should reduce to its canal design value after the opening and before the water re-enters the earthen canal.

There is a relationship between the width of the opening of the gate and the head loss. Hydraulic losses through a properly designed structure are small. When the ground slope is very gentle, head losses should be kept to a minimum so as to maintain command in the canals. This would be achieved by making a wider and larger diversion structure.

6.4. Cross regulators

A cross regulator is a structure built across the canal to maintain the water level at the command level required to irrigate the fields. Cross regulators could be simple timber stop logs, check plates, weirs or expensive automatically operated gates, which automatically control a constant water level.

In Section 6.1, weirs have been discussed as headwork structures. In the context of cross regulation, examples of common weirs are duckbill and diagonal weirs, which control the water level at a given height, (Figure 58, 59 and 60). Detailed explanations of weirs as discharge measurement structures are given in Section 6.6.

6.5. Drop structures and tail-end structures

Drop structures and chutes are flow control structures that are installed in canals when the natural land slope is too steep compared to the design canal gradient (see Section 5.1) to convey water down steep slopes without erosive velocities. If a canal were allowed to follow a steep natural gradient, the velocities would be too high. This in turn would cause erosion and make water management difficult. For this, the canal is divided into different reaches over its length. Each reach follows the design canal gradient. When the bottom level of the canal

Figure 58
Duckbill weir photograph (Source: FAO, 1975b)

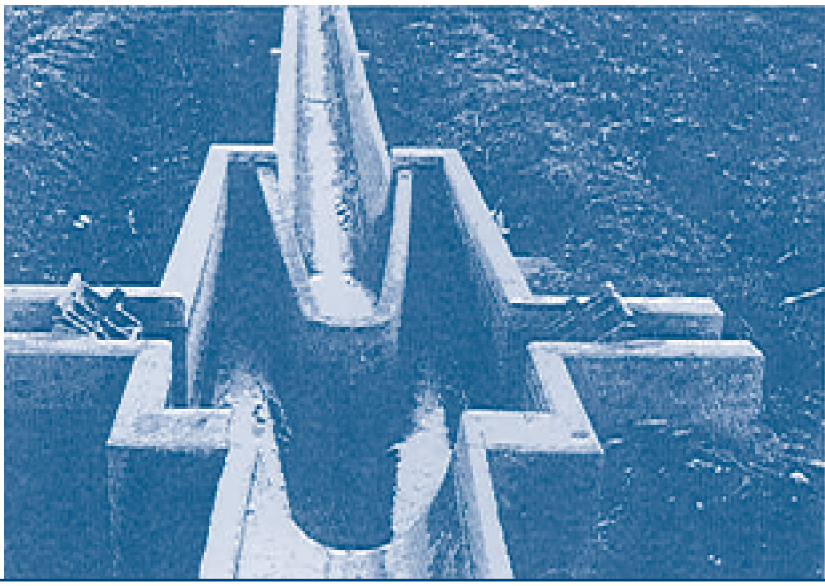
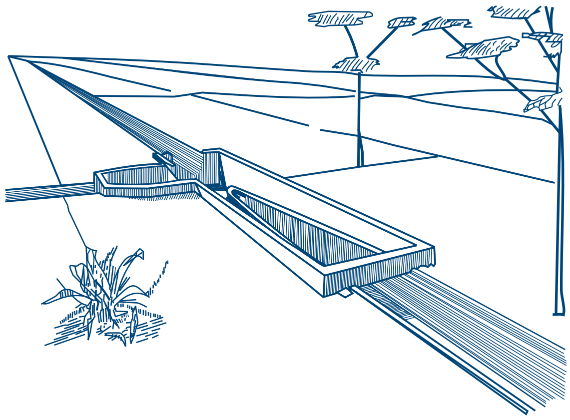


Figure 59
Duckbill weir design (Source: FAO, 1975b)

Basic characteristics

Canal capacity upstream of weir l/s	Dimensions in cm					
	a	b	c	d	e	f
110	7	150	300	13	10	0.08
160	7	220	370	12	9	0.08
210	7	290	440	12	9	0.10
270	8	310	460	11	9	0.10
320	8	370	520	11	9	0.12
370	8	430	580	11	9	0.12



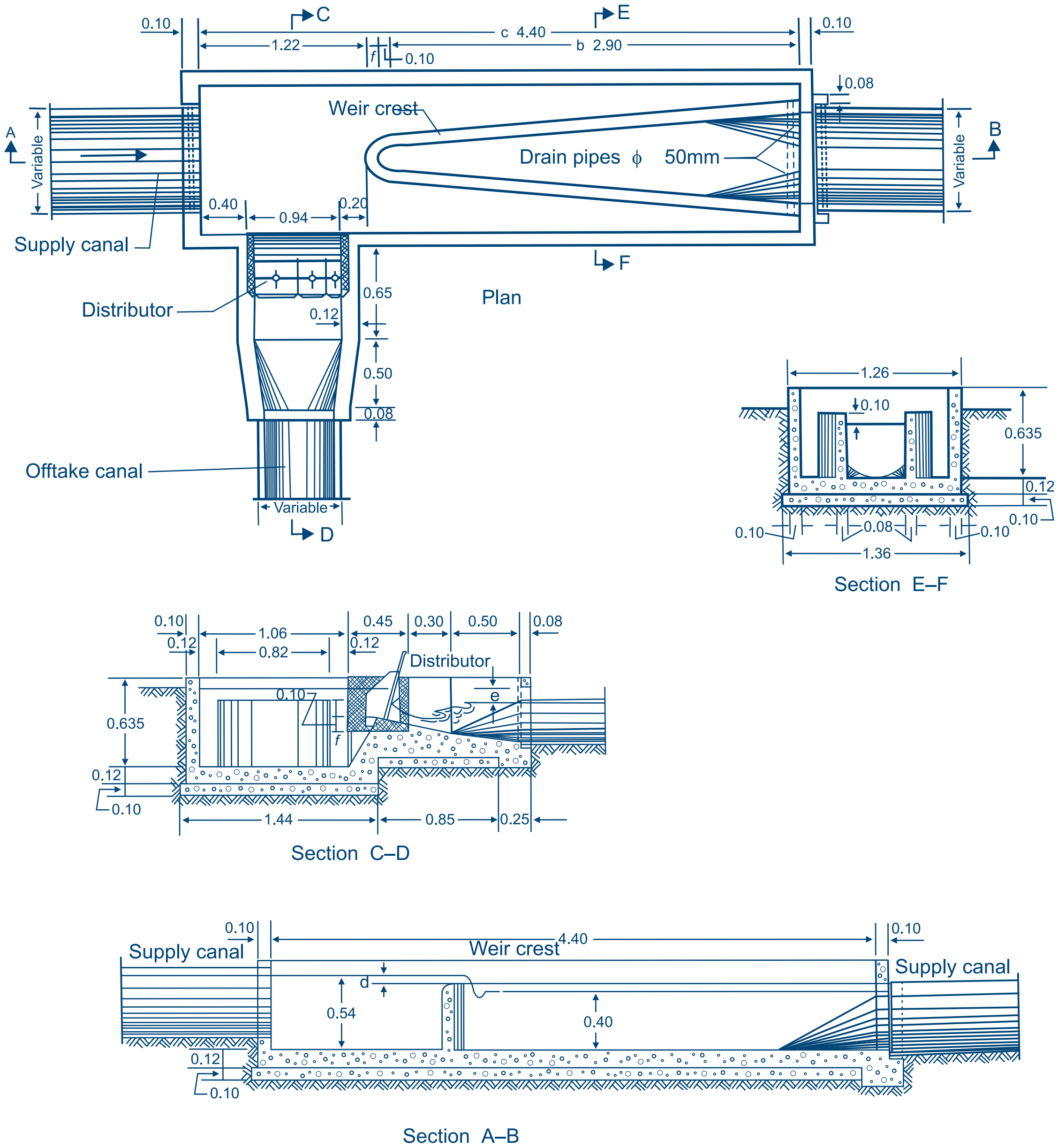
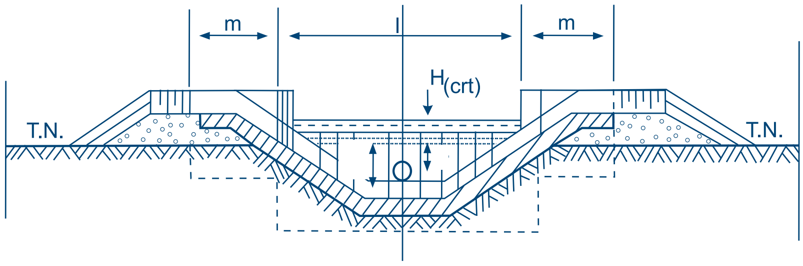
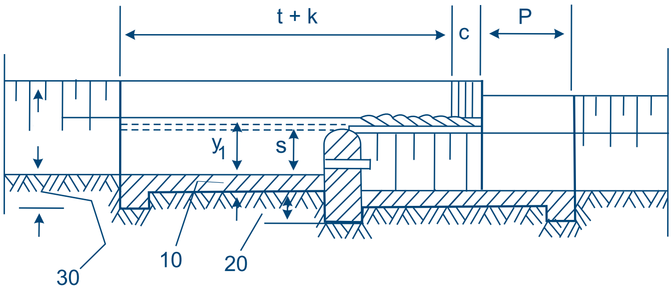


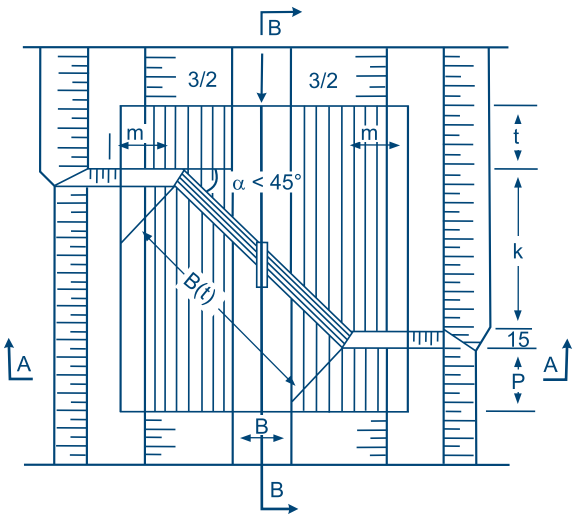
Figure 60
Diagonal weir (Source: FAO, 1975b)



Section A–A



Section B–B



Plan

Range of suitable dimensions for capacities up to 500 l/s

B	=	0.20 to 1.00
f	=	0.20 to 1.00
y_1	=	0.10 to 0.70 (upstream water depth)
H(crt)	=	0.05 to 0.15 (difference between upstream water level and crest level)
s	=	0.10 to 0.60
c	=	0.15 (thickness of weir)
l	=	(width of available upstream water surface)
B(t)	=	(crest length) = $l \times \frac{1}{\cos\alpha}$
α	=	(angle between weir crest and cross-section of channel)
m	=	1.5f - 1.5s + 0.20
k	=	$L\sin\alpha$
p	=	f_n
t	=	f

becomes too high compared to the natural ground level, drop structures are installed. Vertical drops are normally used for the dissipation of up to 1 m head for unlined canals and up to 2 m head for lined canals. For larger drops, chutes are usually used.

For canals that do not require command, the position of drops is determined by considering the cost of canal construction, including balancing the cuts and fills and the cost of the structure. Where there is need for command, the drops should be located in such a way that the canal banks are not too high, but still keeping enough command at the same time.

Figures 61 and 62 show examples of drop structures built with different materials.

6.5.1. Vertical drop structure

An important aspect of a drop is the stilling basin, required to avoid downstream erosion. The floor of the stilling basin should be set at such a level that the hydraulic jump occurs at the upstream end of the basin floor in order to avoid erosion at the unprotected canal bed downstream. A common straight drop structure is shown in Figure 63.

Figure 61
Some drop structures used in open canals (Source: James, 1988)

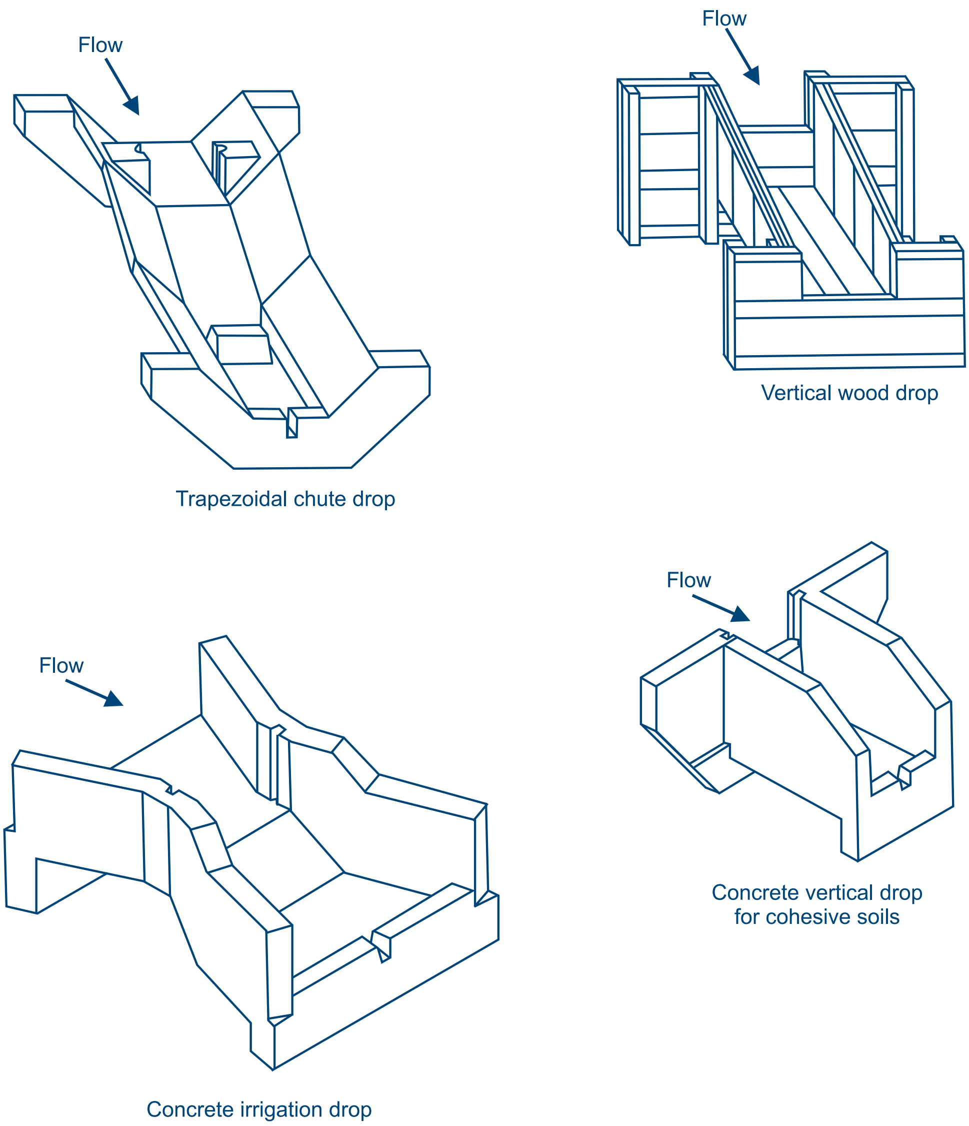


Figure 62
Standard drop structure without stilling basin

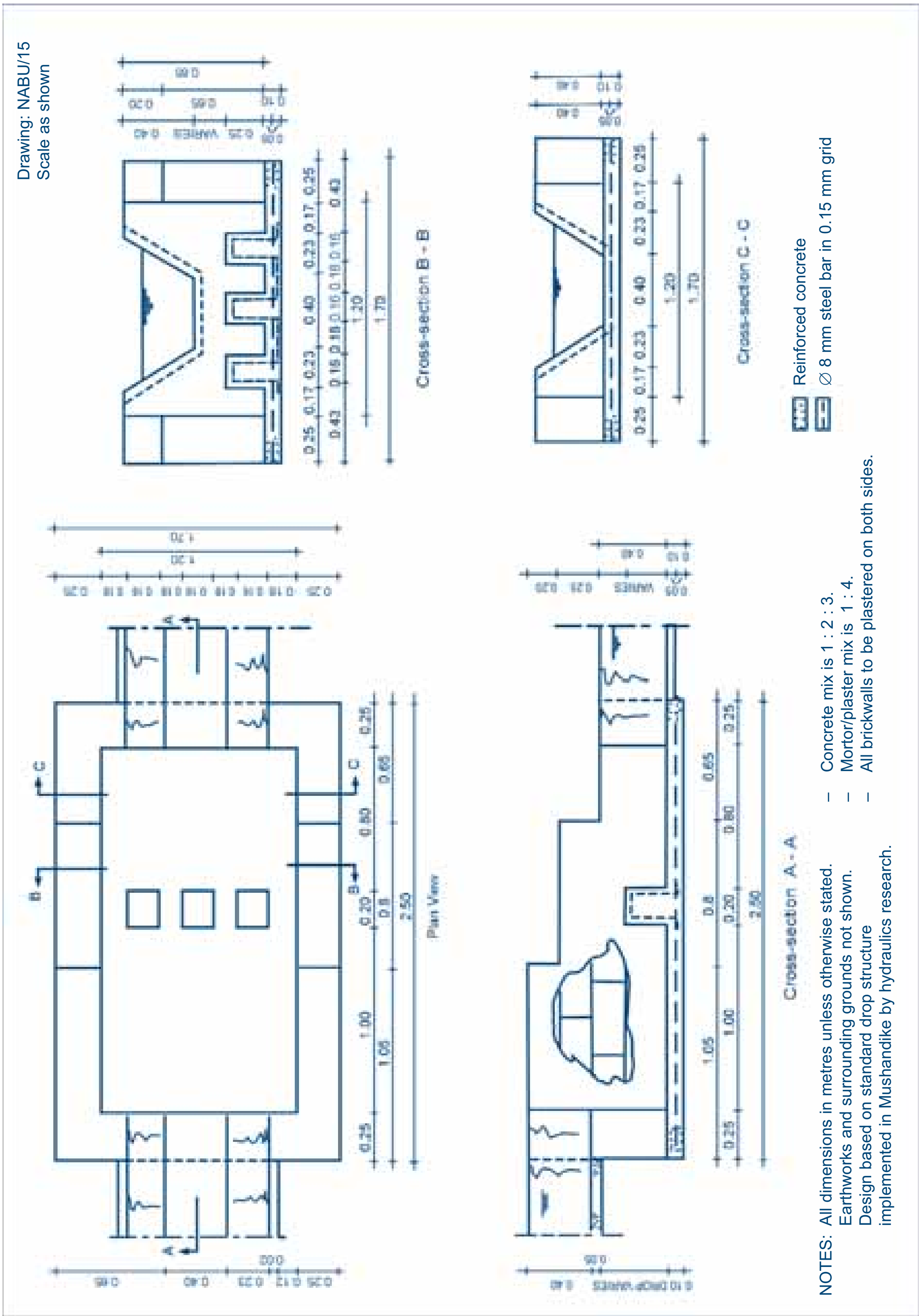
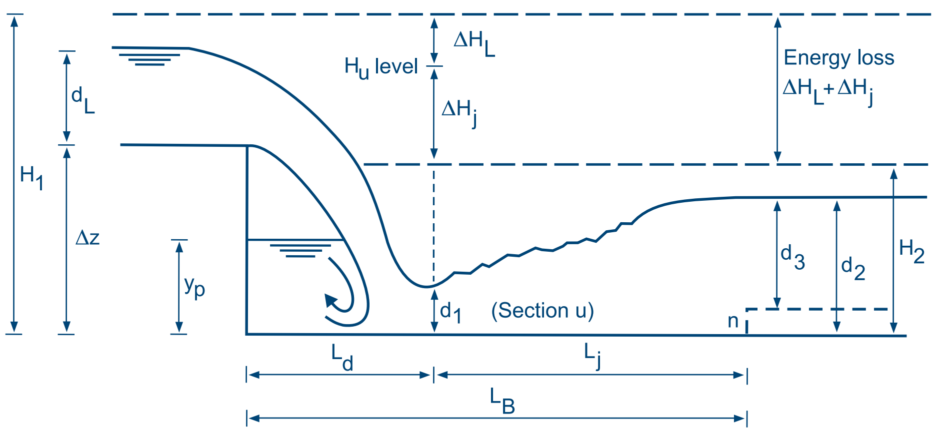


Figure 63
A vertical drop structure



Example 28

Given a discharge of 0.0783 m^3/sec, a drop height of 0.50 m and a drop width of 0.30 m, what would be (Figure 63 and 47):

- *The length of the apron from the drop to the hydraulic jump, where the lowest water level will occur (L_d)?*
- *The height of the jump or the lowest water level after the drop (d_1)?*
- *The design water level after the drop (d_2)?*
- *The total length of the apron (L_B)?*

Q = 0.0783 m^3/sec ⇒ Q = 0.261 m^3/sec per 0.30 m width

Using Equation 25:

$$D = \frac{0.261^2}{(9.81 \times 0.05^3)} \Rightarrow D = 0.0556$$

Substituting the data in Equations 26 to 29 respectively gives:

L_d = 0.50 x 4.30 x $0.0556^{0.27}$ = 0.99 m

d_1 = 0.50 x 0.54 x $0.0556^{0.425}$ = 0.15 m

d_2 = 0.50 x 1.86 x $0.0556^{0.27}$ = 0.38 m

L_j = 6.9 x (0.38 - 0.15) = 1.59 m ⇒ L_B = 0.99 + 1.59 = 2.58 m

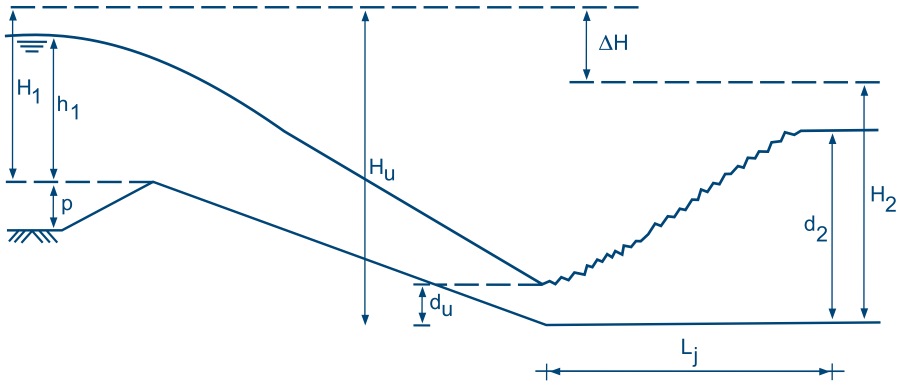
Due to the impact of the water flow on the basin floor and the turbulent circulation, an amount of energy (ΔH_L) is lost. Further energy is lost in the hydraulic jump downstream of the section U in Figure 63. Experiments have shown that the energy head (H_2) is equal to about 2.5 x d_1 (that is 2.5 times the critical depth). This provides a satisfactory basis for design.

An upward step is often added at the end of the basin floor in order to be sure that the hydraulic jump occurs immediately below the drop. This step has the disadvantage of retaining standing water when the canal is not in use, thereby posing a danger to health.

6.5.2. Chutes

An example of a chute structure is given in Figure 64. Chutes are normally rectangular, although they are also made in a trapezoidal shape. They have an inlet, a steep-sloped section of a lined canal, a stilling basin or some other energy dissipating devices, baffle blocks, and an outlet. The energy dissipation is usually effected by the creation of a

Figure 64
A chute structure



hydraulic jump at the toe of the steep-sloped section of the structure. Baffle blocks could be used to facilitate the creation of a hydraulic jump.

The slope of the downstream face (steep-sloped section) usually varies between 1 in 4 and 1 in 6. The length of the stilling basin (often called cistern), L_j, can be estimated with the following equation:

Equation 37

$$L_j = 5 \times d_2$$

6.5.3. Tail-end structures

Most canals need some way of getting rid of water. A tail-end structure should be provided at the end of the canal so that excess water can flow safely into the drain. It normally consists of a drop structure to bring the water level from a command canal level to the drain level from where it will be taken to the main drainage system of the project.

6.6. Discharge measurement in canals

Discharge measurement in irrigation schemes is important for the following reasons:

- To ensure the maintenance of proper delivery schedules
- To determine the amount of water delivered for water pricing, where it is applicable
- To detect the origin of water losses and to estimate the quantity
- To ensure efficient water distribution
- To conduct applied research

Almost any kind of obstacle that partially restricts the flow of water in an irrigation canal and provides a free fall, to ensure that upstream and downstream flow are independent, can be used as a measuring device, provided that it can be calibrated. Standard structures, which have already been accurately described and calibrated, exist. Weirs, flumes and orifices are the devices that are normally used for discharge measurement.

6.6.1. Discharge measurement equations

The three fundamental equations used to solve discharge problems in canals are based on the principles of conservation of mass, energy and momentum. For our purposes, only the conservation of mass and energy equations will be dealt with.

Conservation of mass

Conservation of mass leads to the Continuity Equation 12 to be constant:

$$Q = A \times V = \text{Constant}$$

Conservation of energy

Conservation of energy applied along a streamline results in the Bernoulli Equation:

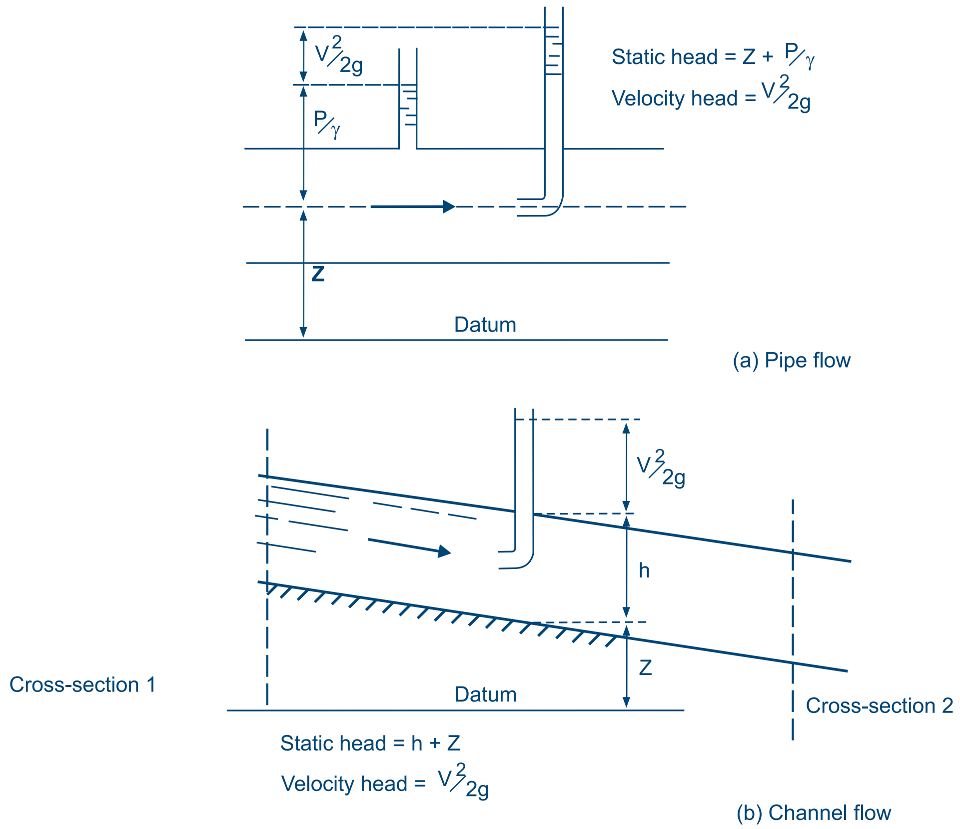
Equation 38

$$\frac{P}{\gamma} + \frac{V^2}{2g} + z = \text{Constant}$$

Where:

P = Pressure (kgf/m^2)
γ = Density of water (kg/m^3)
V = Water velocity (m/sec)
g = Gravitational force (9.81 m/sec^2)
z = Elevation above reference line (m)

Figure 65
Static and velocity heads



Equation 38 sums up the pressure head, velocity head and gravitational head to give the total head. For an open canal, the pressure head equals the water depth h (Figure 65).

When there is frictional loss along the flow path, an expression for frictional head loss must be included. Thus applying the Bernoulli Equation to two successive cross-sections along a flow path results in:

Equation 39

$$h_1 = \frac{V_1^2}{2g} + z_1 = h^2 + \frac{V_2^2}{2g} + z_2 + HL$$

The numbers 1 and 2 refer to the first and second cross-section in Figure 65. HL is the frictional head loss.

Specific energy

The concept of specific energy is used in the analysis of critical flow. At any cross-section of a canal, the energy with respect to the canal bed is referred to as specific energy. It is derived from the Bernoulli Equation according to the following equation:

Equation 40

$$E = \frac{h}{2g} + V^2$$

Where:

E = Specific energy (m)
h = Depth of flow (m)
g = Gravitational force (9.81 m/sec^2)
V = Water velocity (m/sec)

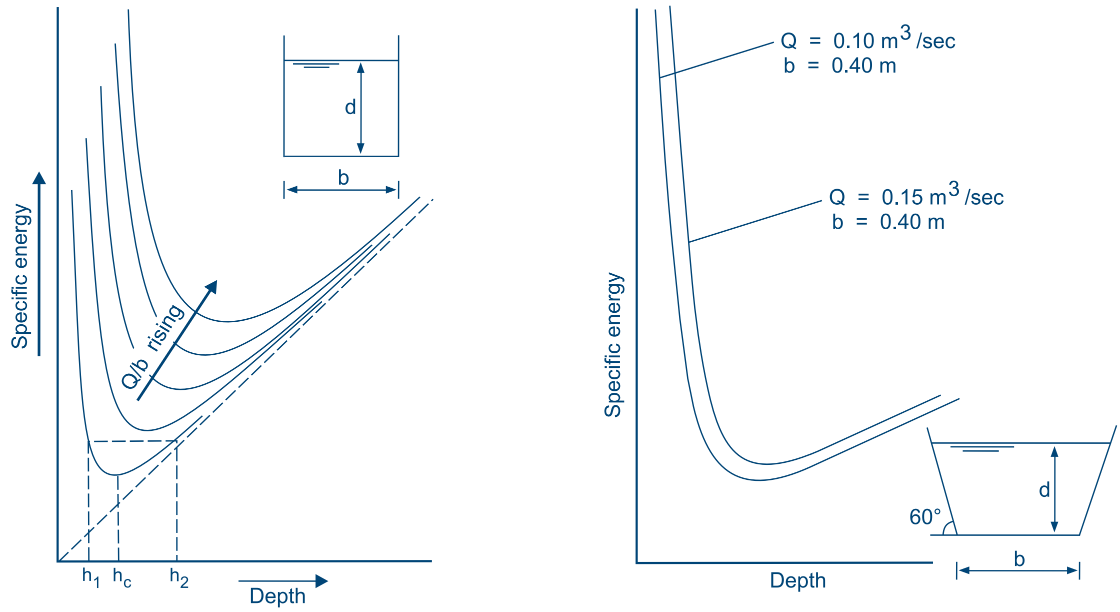
Assuming a uniform velocity distribution, the specific energy is constant across the section. Combining the above equation and the Continuity Equation gives:

Equation 41

$$E = \frac{h}{2g} + \left[\frac{Q}{A}\right]^2$$

The cross-sectional area varies with the depth of flow only if the geometry of the canal is constant. Therefore, for a given discharge the specific energy is a function of depth alone.

Figure 66
Variation of specific energy with depth of flow for different canal shapes



Specific energy can be determined for different structures:

Rectangular canal

$$A = b \times h$$

$$E = \frac{h}{2g} + \left[\frac{Q}{b \times h}\right]^2$$

Trapezoidal canal (with a side slope of 60°)

$$A = b \times h + 0.58 \times h^2$$

$$E = \frac{h}{2g} + \left[\frac{Q}{b \times h + 0.58\ h^2}\right]^2$$

Plotting E against h for different values of (Q/b) gives curves as shown in Figure 66.

The curves show that, for a given discharge and specific energy, there are two alternate depths of flow, which coincide at a point where the specific energy is a minimum for a given discharge. Below this point, flow is physically not possible. At this point flow is critical and it occurs at critical depth and velocity. At a greater depth, the velocity is low and flow is sub-critical. At the lesser depth, the velocity is high and flow is super-critical.

For sub-critical flow, the mean velocity is less than the velocity of propagation of stream disturbances such as waves. Thus, stream effects can be propagated both upstream and downstream. This means that downstream conditions affect the behaviour of flow. When flow is super-critical, the velocity of flow exceeds the velocity of propagation. Consequently, stream effects (for example, waves) cannot be transmitted upstream, and downstream conditions do not affect the behaviour of the flow.

For critical flow, the specific energy is a minimum for a given discharge. In this case, a relationship exists between the minimum specific energy and the critical depth. This relationship is found by differentiating Equation 41 with respect to h, while Q remains constant. This gives:

Equation 42

$$V_c = \left[\frac{g \times A_c}{b_c}\right]^{1/2}$$

Froude Number

The Froude Number is calculated according to Equation 19 (see Section 5.1.2):

$$Fr = \frac{v}{(g \times h)^{1/2}}$$

Where:

Fr = 1 for critical flow

Fr = > 1 for super-critical flow

Fr = < 1 for sub-critical flow

If a structure is built in a canal which has sub-critical flow, it may cause the flow to pass through the critical to the super-critical state. This means that the state upstream of the structure becomes independent of the state downstream. This can either be achieved if the structure narrows the canal, which means increasing the (Q/b)-ratio without altering the specific energy, or if it raises the canal bed, which means reducing the specific energy without altering the discharge per unit width. That is how critical flow is obtained with a measuring device. A control section in a canal is a section that produces a definitive relationship between water depth and discharge.

Hydraulic jump

If, through a structure, super-critical flow is introduced in a canal where the normal flow is sub-critical, flow adjusts back to the sub-critical state through a hydraulic jump in which the water level rises over a short distance with much visible turbulence. This situation occurs, for example, downstream of a sluice gate or a flume. It is undesirable to have a hydraulic jump in an unlined canal because of the risk of scour. In such cases, a jump is usually induced over a concrete apron by means of a sill or baffle blocks set in the floor, as shown in Figure 67.

The relationship between depths just upstream and downstream of a hydraulic jump is found by the application of the momentum theory to the simplified situation shown in Figure 68. It is assumed that boundary frictions are negligible over the length of the jump. For a rectangular canal it can be shown that:

Equation 43

$$h_2 = -\frac{h_1}{2} + 0.5 \times \left[h_1^2 + \left(8 \times V_1^2 \times \frac{h_1}{9} \right) \right]^{1/2}$$

6.6.2. Weirs

The weir is the most practical and economical device for water measurement. Weirs are simple to construct, easy to inspect, robust and reliable. Discharge measurement weirs can either be sharp-crested (Figure 69, 70, 71) or broad-crested (Figure 72).

Sharp-crested weirs

Sharp-crested weirs, also called thin plate weirs, consist of a smooth, vertical, flat plate installed across the channel and perpendicular to the flow (Figure 69). The plate obstructs flow, causing water to back up behind the weir plate and to

Figure 67
Hydraulic jump over a concrete apron

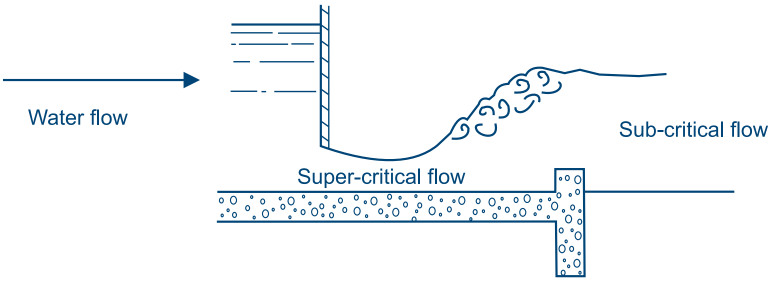


Figure 68
The form of a hydraulic jump postulated in the momentum theory

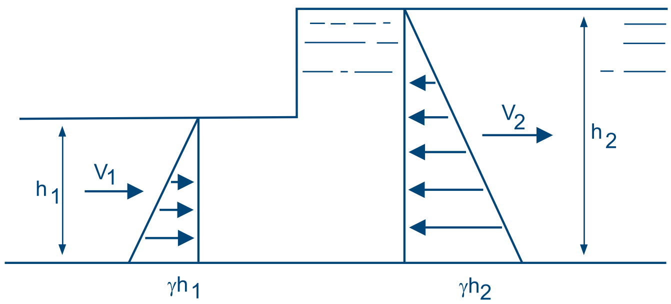
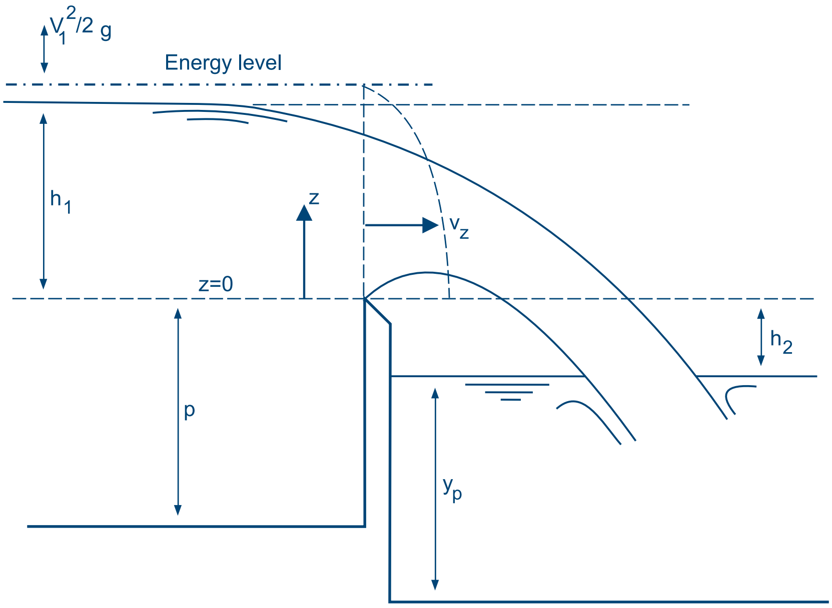


Figure 69
Parameters of a sharp-crested weir



flow over the weir crest. The distance from the bottom of the canal to the weir crest, p, is the crest height. The depth of flow over the weir crest, measured at a specified distance upstream of the weir plate (about four times the maximum h_1), is called the head h_1. The overflowing sheet of water is known as the nappe.

Thin plate weirs are most accurate when the nappe springs completely free of the upstream edge of the weir crest and air is able to pass freely around the nappe. The crest of a sharp-crested weir can extend across the full width of channel or it can be notched. The most commonly used notched ones are:

- ❖ Rectangular contracted weir
- ❖ Trapezoidal (Cipoletti) weir (Figure 70)
- ❖ Sharp sided 90° V-notch weir (Figure 71)

The type and dimensions of the weir chosen are based on the expected discharge and the limits of its fluctuation. For example, a V-notch weir gives the most accurate results when measuring small discharges and is particularly adapted to the measurement of fluctuating discharges. Calibration curves and tables have been developed for standard weir types.

The conditions and settings for standard weirs are as follows:

i. The height of the crest from the bottom of the approach canal (p) should preferably be at least twice the depth of water above the crest and should in no case be less than 30 cm. This will allow the water to fall freely, leaving an airspace under and around the jets.

ii. At a distance upstream of about four times the maximum head a staff gauge is installed on the crest with the zero placed at the crest elevation, to measure the head h1.

iii. For the expected discharge, the head (h1) should not be less than 6 cm and should not exceed 60 cm.

iv. For rectangular and trapezoidal weirs, the head (h_1) should not exceed 1/3 of the weir length.

v. The weir length should be selected so that the head for the design discharge will be near the maximum, subject to the limitations given in (ii) and (iii).

vi. The thickness of the crest for sharp-crest weirs should be between 1-2 mm.

In sediment-laden canals, a main disadvantage of using weirs is that silt is deposited against the upstream face of the weir, altering the discharge characteristics. Weirs also cannot be used in canals with almost no longitudinal slopes, since the required difference in elevation between the water levels upstream and downstream side of the weir is not available.

Discharge equations for weirs are derived by the application of the Continuity and Bernoulli Equations (Equation 12 and 38 respectively). In each case, a discharge coefficient is used in order to adjust the theoretical discharge found by laboratory measurements.

Rectangular contracted weir

A rectangular contracted weir is a thin-plate weir of rectangular shape, located perpendicular to the flow. To allow full horizontal contraction of the nappe, the bed and sides of the canal must be sufficiently far from the weir crest and sides.

Many practical formulae have been developed for computing the discharge, amongst which are the following:

Equation 44

Hamilton-Smith formula:

$$Q = \left[0.616 \times \left(1 - \frac{0.1h}{b}\right)\right] \times \frac{2}{3} \times (2g)^{1/2} \times b \times h^{3/2}$$

Equation 45

Francis formula:

$$Q = 1.838 \times (b - 2h) \times h^{3/2}$$

Where:

Q = Design discharge over weir (m^3/sec)
b = Length of weir crest (m)
h = Design water depth measured from the top of the weir crest (m)

Table 28 gives discharge data related to length of crest, b, and water head, h, over a weir.

Trapezoidal (Cipoletti) weir

The trapezoidal weir has a trapezoidal opening, the base being horizontal. The Cipoletti weir is a trapezoidal weir

Example 29

A rectangular contracted weir has to be placed in a lined canal. The design discharge is 0.0783 m^3/sec and the maximum allowable water depth, h, at the measuring gauge can be 0.15 m. What should be the minimum weir crest length, b, calculated using the Francis formula?

Using Equation 45:

$$Q = 0.0783 = 1.838 \times (b - 0.2 \times 0.15) \times 0.15^{3/2} = 0.1068 \times b - 0.0032 \Rightarrow b = 0.76 \text{ m.}$$

Table 28
Discharge Q (m^3/sec) for contracted rectangular weir, depending on h and b

Head h (m)	Length of crest b (m)						
	0.30	0.40	0.50	0.75	1.00	1.25	1.50
0.0025	0.0001	0.0001	0.0001	0.0002	0.0002	0.0003	0.0003
0.015	0.0010	0.0013	0.0017	0.0025	0.0034	0.0042	0.0051
0.030	0.0028	0.0038	0.0047	0.0071	0.0095	0.0119	0.0143
0.045	0.0051	0.0069	0.0086	0.0130	0.0174	0.0218	0.0262
0.060	0.0078	0.0105	0.0132	0.0199	0.0267	0.0335	0.0402
0.075	0.0108	0.0145	0.0183	0.0278	0.0372	0.0466	0.0561
0.090	0.0140	0.0190	0.0239	0.0363	0.0487	0.0612	0.0736
0.105	0.0175	0.0237	0.0300	0.0456	0.0612	0.0769	0.0925
0.12	0.0211	0.0287	0.0364	0.0555	0.0746	0.0937	0.1128
0.15	0.0288	0.0395	0.0502	0.0769	0.1036	0.1303	0.1570
0.18		0.0511	0.0651	0.1002	0.1353	0.1704	0.2055
0.21			0.0810	0.1253	0.1695	0.2137	0.2580
0.24			0.0977	0.1517	0.2058	0.2598	0.3139
0.27				0.1795	0.2440	0.3085	0.3730
0.30				0.2084	0.2840	0.3595	0.4350
0.36				0.2692	0.3685	0.4678	0.5671
0.42					0.4584	0.5835	0.7086
0.48					0.5527	0.7055	0.8584
0.54						0.8331	1.0155
0.60						0.9655	1.1791

with the sides having an outward sloping inclination of 1 horizontal to 4 vertical (Figure 70). This side slope is such that the water depth-discharge relationship is the same as that of a full width rectangular weir.

The discharge equation for a Cipoletti weir is:

Equation 46

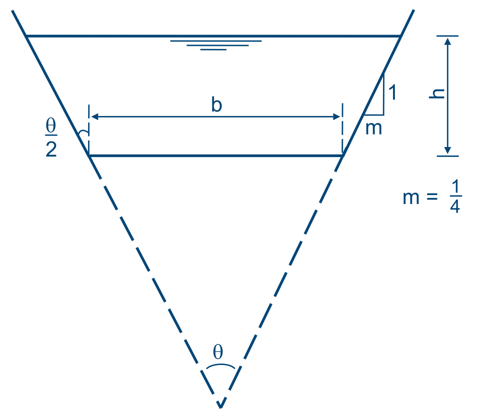
$$Q = 1.859 \times b \times h^{3/2}$$

Where:

Q = Design discharge over weir (m^3/sec)
b = Length of weir crest (m)
h = Design water depth measured from the top of the weir crest (m)

Table 29 shows discharge data, related to the design water depth, h, and weir length, b.

Figure 70
Trapezoidal (Cipoletti) weir



Example 30

A Cipoletti weir has to be placed in a lined canal. The design discharge is 0.0783 m^3/sec and the maximum allowable head, h, at the measuring gauge is 0.15 m. What should be the minimum weir crest length, b?

Using Equation 46:

$$0.0783 = 1.859 \times b \times 0.15^{3/2} = 1.108b \Rightarrow b = 0.73 \text{ m}$$

Table 29
Discharge Q (m^3/sec) for Cipoletti weir, depending on h and b

Head h	Length of crest b (m)						
(m)	0.30	0.40	0.50	0.75	1.00	1.25	1.50
0.0025	0.0001	0.0001	0.0001	0.0002	0.0002	0.0003	0.0003
0.015	0.0010	0.0014	0.0017	0.0026	0.0034	0.0043	0.0051
0.030	0.0029	0.0039	0.0048	0.0072	0.0097	0.0121	0.0145
0.045	0.0053	0.0071	0.0089	0.0133	0.0177	0.0222	0.0266
0.060	0.0082	0.0109	0.0137	0.0205	0.0273	0.0341	0.0410
0.075	0.0115	0.0153	0.0191	0.0286	0.0382	0.0477	0.0573
0.090	0.0151	0.0201	0.0251	0.0376	0.0502	0.0627	0.0753
0.105	0.0199	0.0253	0.0316	0.0474	0.0632	0.0791	0.0949
0.12	0.0232	0.0309	0.0386	0.0580	0.0773	0.0966	0.1159
0.15	0.0324	0.0432	0.0540	0.0810	0.1080	0.1350	0.1620
0.18		0.0568	0.0710	0.1065	0.1420	0.1774	0.2129
0.21			0.0894	0.1342	0.1789	0.2236	0.2683
0.24			0.1093	0.1639	0.2186	0.2732	0.3278
0.27				0.1956	0.2608	0.3260	0.3912
0.30				0.2291	0.3054	0.3818	0.4582
0.36				0.3011	0.4015	0.5019	0.6023
0.48					0.3060	0.6325	0.7590
0.54					0.6182	0.7727	0.9273
0.60						0.9220	1.1065
						1.0799	1.2959

V-notch weir

A V-notch weir has two edges that are symmetrically inclined to the vertical to form a notch in the plane perpendicular to the direction of flow. The most commonly used V-notch weir is the one with a 90° angle. Other common V-notches are the ones where the top width is equal to the vertical depth (1/2 x 90° V-notch) and the one where the top width is half of the vertical depth (1/4 x 90° V-notch) (Figure 71). The V-notch weir is an accurate discharge-measuring device, particularly for discharges less than 30 l/sec, and it is as accurate as other types of sharp-crested weirs for discharges from 30 to 300 l/sec (U.S. Department of Interior, 1975).

To operate properly, the weir should be installed so that the minimum distance from the canal bank to the weir edge is at least twice the head on the weir. In addition, the distance from the bottom of the approach canal to the point of the weir notch should also be at least twice the head on the weir (U.S. Department of Interior, 1975).

The general and simple discharge equation for a V-notch weir is:

Equation 47

$$Q = 1.38 \times \tan\left(\frac{1}{2} \times \theta\right) \times h^{5/2}$$

Where:

- Q = Design discharge over the weir (m^3/sec)
- θ = Angle included between the sides of the notch (degrees)
- h = Design water depth (m)

Table 30 gives discharge data for the three common V-notches related to water depth (head) and angle°.

Example 31

A design discharge of 0.0783 m^3/sec has to pass through a V-notch weir with an angle θ of 90°. What will be the water depth over the weir?

Substituting the above data in Equation 47:

$$0.0783 = 1.38 \times \tan\left(\frac{1}{2} \times 90\right) \times h^{5/2} \Rightarrow h^{5/2} = 0.0783 \Rightarrow h = 0.317 \text{ m.}$$

Figure 71
V-notch weirs

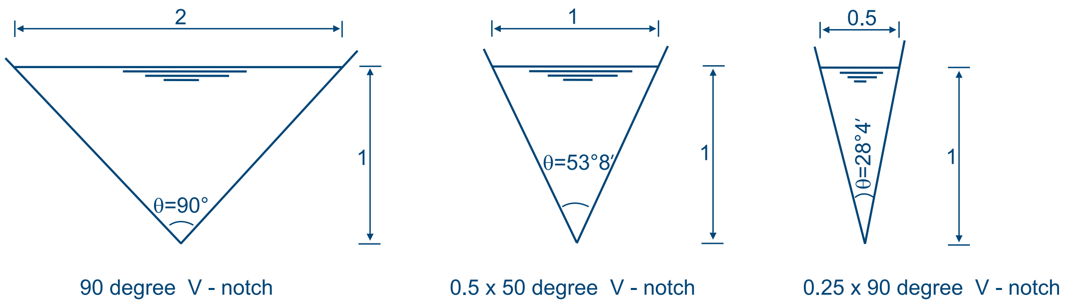


Table 30
Discharge Q (m^3/sec x 10) for a 90° V-notch weir, depending on h

Head (m)	Discharge (m^3/sec x 10)	Head (m)	Discharge (m^3/sec x 10)	Head (m)	Discharge (m^3/sec x 10)
0.050	0.008	0.160	0.142	0.270	0.523
0.055	0.010	0.165	0.153	0.275	0.548
0.060	0.012	0.170	0.165	0.280	0.573
0.065	0.015	0.175	0.177	0.285	0.599
0.070	0.018	0.180	0.190	0.290	0.626
0.075	0.022	0.185	0.203	0.295	0.653
0.080	0.025	0.190	0.217	0.300	0.681
0.085	0.029	0.195	0.232	0.305	0.710
0.090	0.034	0.200	0.247	0.310	0.739
0.095	0.039	0.205	0.263	0.315	0.770
0.100	0.044	0.210	0.279	0.320	0.801
0.102	0.050	0.215	0.296	0.325	0.832
0.110	0.056	0.220	0.313	0.330	0.865
0.115	0.062	0.225	0.332	0.335	0.898
0.120	0.069	0.230	0.350	0.340	0.932
0.125	0.077	0.235	0.370	0.345	0.966
0.130	0.084	0.240	0.390	0.350	1.002
0.135	0.093	0.245	0.410	0.355	1.038
0.140	0.102	0.250	0.432	0.360	1.075
0.145	0.111	0.255	0.454	0.365	1.113
0.150	0.121	0.260	0.476	0.370	1.152
0.155	0.131	0.265	0.499	0.375	1.191
				0.380	1.231

Broad-crested weir

A broad-crested weir is a broad wall set across the canal bed. The way it functions is to lower the specific energy and thus induce a critical flow (Figure 72).

One of the most commonly used broad-crested weirs for discharge measurements is the Romijn broad-crested weir, which was developed in Indonesia for use in relatively flat areas and where the water demand is variable because of different requirements during the growing season (FAO, 1975b). It is a weir with a rectangular control section, as shown in Figure 73.

The Romijn weir consists of two sliding blades and a movable weir crest, which are mounted in one steel guide frame (Figure 74). The bottom blade, which is locked under operational conditions, acts as the bottom terminal for the movable weir. The upper blade, which is connected to the bottom blade by means of two steel strips placed in the frame grooves, acts as the top terminal for the movable weir. Two steel strips connect the movable weir to a horizontal lifting beam. The horizontal weir crest is perpendicular to the water flow and slopes 1:25 upward in the direction of the flow. Its upstream nose is rounded off in such a way that flow separation does not occur. The operating range of the weir equals the maximum upstream head (H_{crt}) which has been selected for dimensioning the regulating structure.

Figure 72
Broad-crested weir

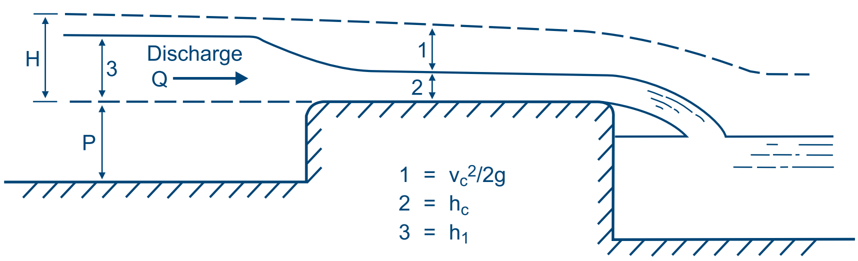


Figure 73

Romijn broad-crested weir, hydraulic dimensions of weir abutments (Source: FAO, 1975b)

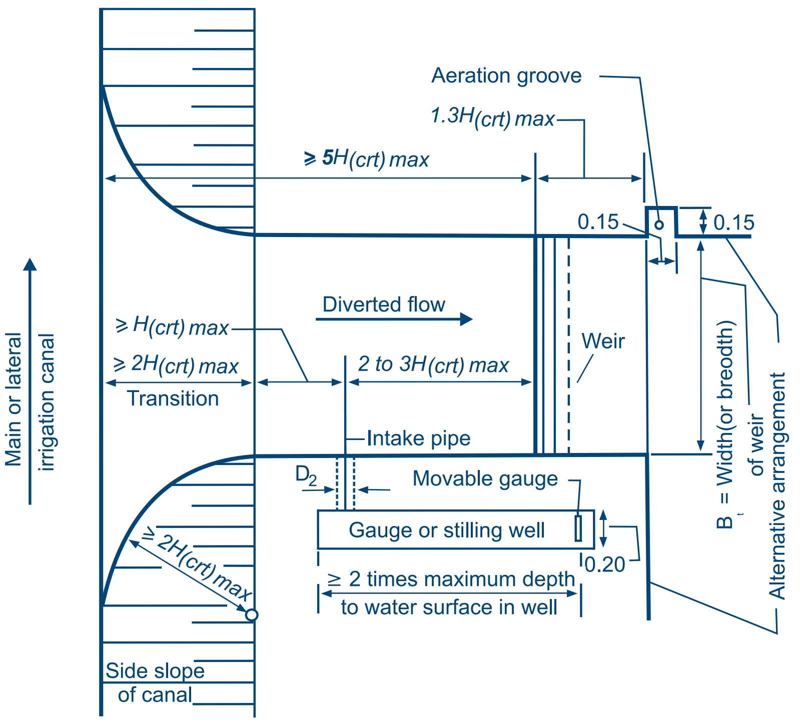
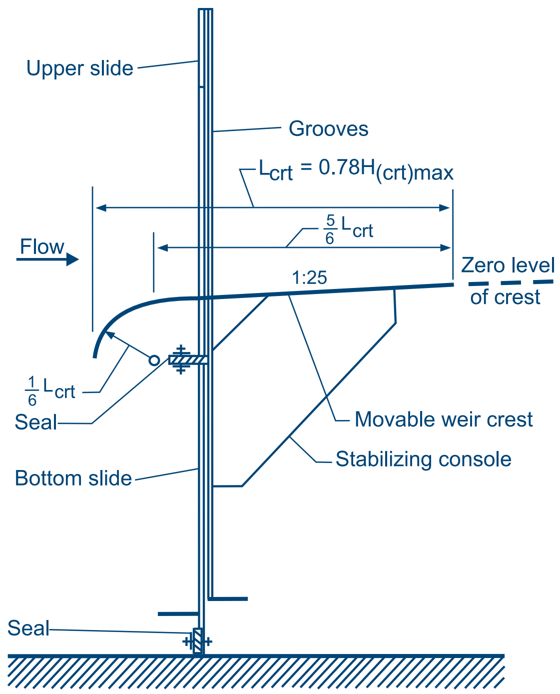


Figure 74

Romijn broad-crested weir, sliding blades and movable weir crest (Source: FAO, 1975b)



The discharge equation for the Romijn broad-crested weir is written as:

Equation 48

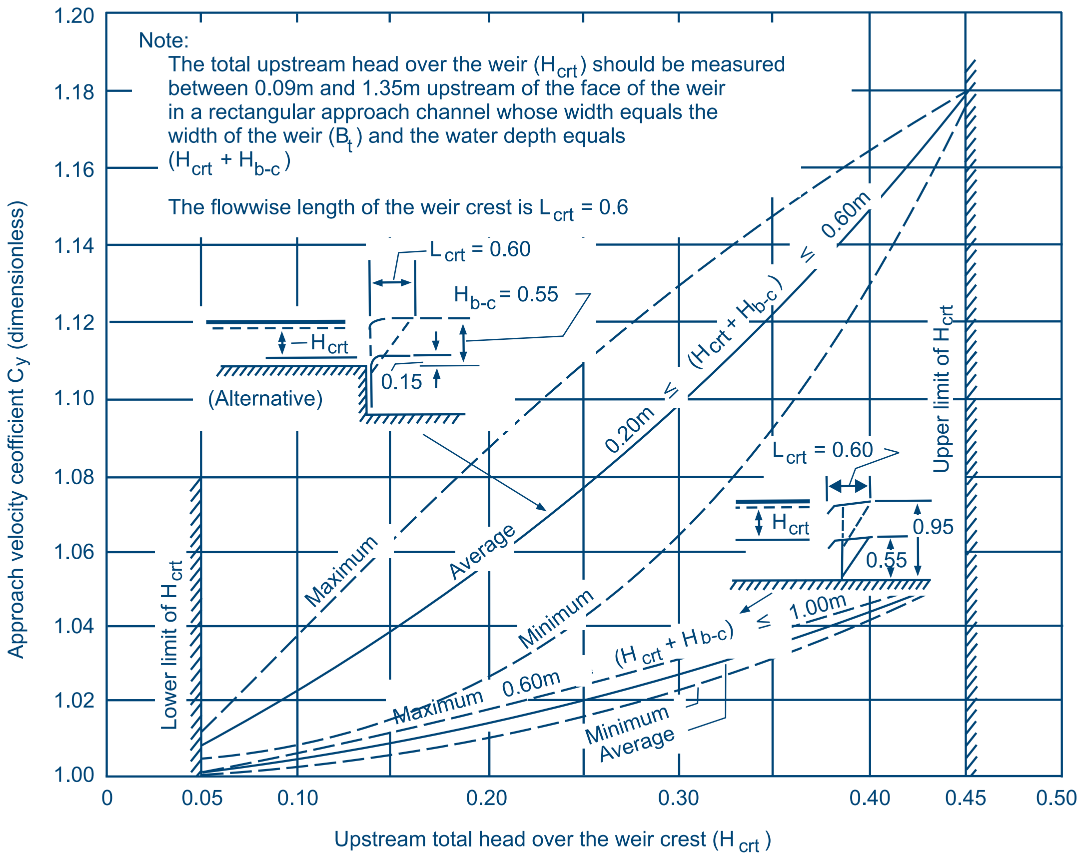
$$Q = \frac{2}{3} \times C_d \times C_v \times \left[\frac{2}{3} \times g\right]^{1/2} \times B_t \times H_{crt}^{3/2}$$

Where:

Q	=	Design discharge over the weir (m^3/sec)
C_d	=	Discharge coefficient
C_v	=	Approach velocity coefficient
g	=	Acceleration due to gravity (= 9.81 m/sec^2)
B_t	=	Width (or breadth) of the weir across the direction of flow (m)
H_{crt}	=	Design upstream water depth over the weir (m)

The value of the discharge coefficient, C_d, has been determined in laboratory tests. For field structures with concrete abutments, it is advisable to use an average value of C_d = 1.00. The value of the approach velocity coefficient, C_v, ranges between 1.00 and 1.18, depending on H_{crt} (Figure 75).

Figure 75
Approach velocity coefficient, C_v, as a function of the total head over the movable weir crest, H_{crt} (Source: FAO, 1975b)



Where both C_d and C_v are considered to be 1.00, substituting these values and the value for g in Equation 48 gives Equation 49:

Equation 49

$$q = 1.7 \times B_t \times H_{crt}^{3/2}$$

More details on the Romijn weir can be found in FAO (1975b).

6.6.3. Flumes

Discharge measurement flumes are extensively used in irrigation schemes mainly because they:

- Can be used under almost any flow condition
- Have smaller head-losses than weirs, thus are more accurate over a large flow range
- Are insensitive to the velocity of approach
- Are relatively less susceptible to sediment and debris transport

Example 32

A Romijn broad-crested weir has to discharge 0.0783 m³/sec. The maximum allowable water depth over the weir can be 0.15 m. What should be the minimum width of weir?

Considering a C_d value of 1.00 and an average C_v value of 1.04 (Figure 75), Equation 48 gives:

$$0.0783 = \frac{2}{3} \times 1.00 \times 1.04 \times \left[\frac{2}{3} \times 9.81\right]^{1/2} \times B_t \times 0.15^{3/2} \Rightarrow Bt = 0.76 \text{ m}$$

Using the simplified Equation 49 would give:

$$0.0783 = 1.7 \times B_t \times 0.15^{3/2} \Rightarrow B_t = 0.79 \text{ m}$$

However, major disadvantages of flumes include the relative large sizes and the accurate manufacturing/construction workmanship required for optimum performance (James, 1988).

A canal section that causes flow to pass from sub-critical through critical to the super-critical state forms a control and the discharge is a single valued function of the upstream water level. Critical flow can be achieved by raising the canal bed, thereby reducing the specific energy, or by decreasing the canal width, thereby increasing the discharge per unit width (see Section 6.6.1). This latter technique is the one used by flumes.

A flume has:

- A convergent section, in which the flow accelerates
- A throat, in which critical flow occurs
- A divergent section, in which the flow returns to normal

Super-critical flow passing from the throat will return to sub-critical flow downstream of the flume. This occurs due to the development of a hydraulic jump, which is induced within the divergent section by a sill or other barrier. Where there is sufficient head available, the divergent section of the flume could be avoided as the flow could fall freely in a stilling basin. In this case, weirs could also be used. However, if canals are expected to carry a lot of sediment, the flume should be the better choice.

Flumes are most commonly rectangular or trapezoidal in cross-section. The former type is the most simple to construct, but if the canal cross-section is not rectangular there is a risk that unpredictable flow patterns will result from an abrupt change of cross-section.

The most commonly used flumes are:

- Parshall flume
- Trapezoidal flume
- Cut-throat flume

Figure 76
Parshall flume

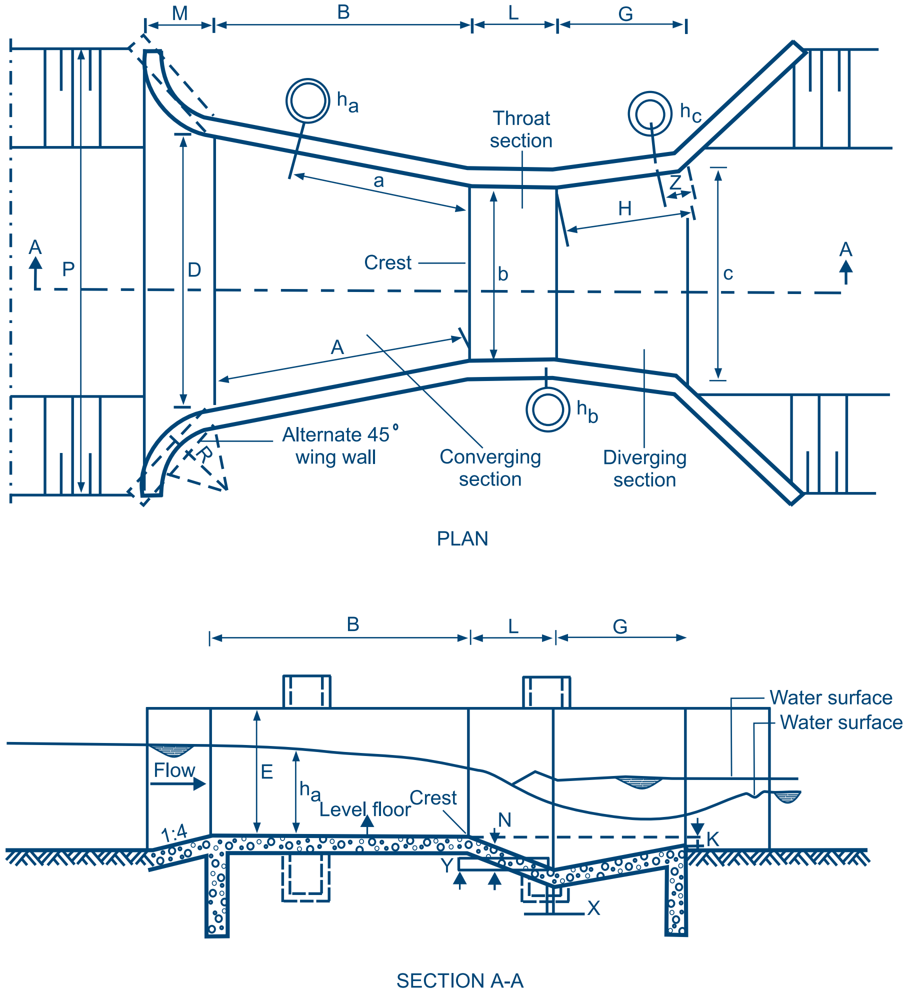


Table 31

Standard dimensions of Parshall flumes (the letters are shown in Figure 76) (Adapted from: FAO, 1975b)

b		A	a	B	C	D	E	L	G	H	K	M	N	P	R	X	Y	Z
‘ + “	mm	mm																
1”	25.4	363	242	356	93	167	229	76	203	206	19	-	29	-	-	8	13	3
2”	50.8	414	276	406	135	214	254	114	254	257	22	-	43	-	-	16	25	6
3”	76.2	467	311	457	178	259	457	152	305	309	25	-	57	-	-	25	38	9
6”	152.4	621	414	610	394	397	610	305	610	-	76	305	114	902	406	51	76	-
9”	228.6	879	587	864	381	575	762	305		-	76	305	114	1080	406	51	76	-
1”	304.8	1372	914	134	610	845	914	610	914	-	76	381	229	1492	508	51	76	-
1’6”	457.2	1448	965	1419	762	1026	914	610	914	-	76	381	229	1676	508	51	76	-
2’	609.6	1524	1016	1495	914	1206	914	610	914	-	76	381	229	1854	508	51	76	-
3’	914.4	1676	1118	1645	1219	1572	914	610	914	-	76	381	229	2222	508	51	76	-
4’	1219.2	1829	1219	1794	1524	1937	914	610	914	-	76	457	229	2711	610	51	76	-
5’	1524.0	1981	1321	1943	1829	2302	914	610	914	-	76	457	229	3080	610	51	76	-
6’	1828.8	2134	1422	2092	2134	2667	914	610	914	-	76	457	229	3442	610	51	76	-
7’	2133.6	2286	1524	2242	2438	3032	914	610	914	-	76	457	229	3810	610	51	76	-
8’	2438.4	2438	1626	2391	2743	3397	914	610	914	-	76	457	229	4172	610	51	76	-
10’	3048	-	1829	4267	3658	4756	1219	914	1829	-	76	-	343	-	-	305	229	-
12’	3658	-	2032	4877	4470	5607	1542	914	2438	-	152	-	343	-	-	305	229	-
15’	4572	-	2337	7620	5588	7620	1829	1219	3048	-	152	-	457	-	-	305	229	-
20’	6096	-	2845	7620	7315	9144	2134	1829	3658	-	305	-	686	-	-	305	229	-
25’	7620	-	3353	7620	8941	10668	2134	1829	3962	-	305	-	686	-	-	305	229	-
30’	9144	-	3861	7925	10566	12313	2134	1829	4267	-	305	-	686	-	-	305	229	-
40’	12192	-	4877	8230	13818	15481	2134	1829	4877	-	305	-	686	-		305	229	-
50’	15240	-	5893	8230	17272	18529	2134	1829	6096	-	305	-	686	-	-	305	229	-

Parshall flume

The Parshall flume is a widely-used discharge measurement structure. Figure 76 shows its general form. The characteristics of Parshall flumes are:

- Small head losses
- Free passage of sediments
- Reliable measurements even when partially submerged
- Low sensitivity to velocity of approach

The Parshall flume consists of a converging section with a level floor, a throat section with a downward sloping floor and a diverging section with an upward sloping floor. Flume sizes are known by their throat width.

Care must be taken to construct the flumes accurately if the calibration curves have to be used. Each size has its own characteristics, as the flumes are not hydraulic scale models of each other. In other words, each flume is an entirely different device (see Table 31).

The flow through the Parshall flume can occur either under free flow or under submerged flow conditions. Under free flow the rate of discharge is solely dependent on the throat width and the measured water depth, h_a. The water depth is measured at a fixed point in the converging section.

The upstream water depth-discharge relationship, according to empirical calibrations, has the following general form:

Equation 50

$$Q = K \times (h_a)^u$$

Where:

Q = Discharge (m^3/sec)

h_a = Water depth in converging section (m)

K = A fraction, which is a function of the throat width

u = Variable, lying between 1.522 and 1.60.

Table 32 gives the values for K and u for each flume size.

When the ratio of gauge reading h_b to h_a exceeds 60% for flumes up to 9 inches, 70% for flumes between 9 inches and 8 feet and 80% for larger flume sizes, the discharge is reduced due to submergence. The upper limit of submergence is 95%, after which the flume ceases to be an effective measuring device because the head difference between h_a and h_b becomes too small, such that a slight inaccuracy in either head reading results in a large discharge measurement error.

Table 32
Discharge characteristics of Parshall flumes

Throat width b feet + inches	Discharge range Minimum (m³/sec x 10⁻³)	Discharge range Maximum (m³/sec x 10⁻³)	Equation $Q = K \times h_a^u$ (m³/sec)	Head range Minimum (m)	Head range Maximum (m)	Modular limit h_b/h_a (m)
1"	0.09	5.4	$0.0604\ h_a^{1.55}$	0.015	0.21	0.50
2"	0.18	13.2	$0.1207\ h_a^{1.55}$	0.015	0.24	0.50
3"	0.77	32.1	$0.1771\ h_a^{1.55}$	0.030	0.33	0.50
6"	1.50	111	$0.3812\ h_a^{1.58}$	0.030	0.45	0.60
9"	2.50	251	$0.5354\ h_a^{1.53}$	0.030	0.61	0.60
1'	3.32	457	$0.6909\ h_a^{1.522}$	0.030	0.76	0.70
1'6"	4.80	695	$1.056\ h_a^{1.538}$	0.030	0.76	0.70
2'	12.1	937	$1.428\ h_a^{1.550}$	0.046	0.76	0.70
3'	17.6	1 427	$2.184\ h_a^{1.566}$	0.046	0.76	0.70
4'	35.8	1 923	$2.953\ h_a^{1.578}$	0.060	0.76	0.70
5'	44.1	2 424	$3.732\ h_a^{1.587}$	0.060	0.76	0.70
6'	74.1	2 929	$4.519\ h_a^{1.595}$	0.076	0.76	0.70
7'	85.8	3 438	$5.312\ h_a^{1.601}$	0.076	0.76	0.70
8'	97.2	3 949	$6.112\ h_a^{1.607}$	0.076	0.76	0.70
	m³/sec					
10'	0.16	8.28	$7.463\ h_a^{1.60}$	0.09	1.07	0.80
12'	0.19	14.68	$8.859\ h_a^{1.60}$	0.09	1.37	0.80
15'	0.23	25.04	$10.96\ h_a^{1.60}$	0.09	1.67	0.80
20'	0.31	37.97	$14.45\ h_a^{1.60}$	0.09	1.83	0.80
25'	0.38	47.14	$17.94\ h_a^{1.60}$	0.09	1.83	0.80
30'	0.46	56.33	$21.44\ h_a^{1.60}$	0.09	1.83	0.80
40'	0.60	74.70	$28.43\ h_a^{1.60}$	0.09	1.83	0.80
50'	0.75	93.04	$35.41\ h_a^{1.60}$	0.09	1.83	0.80

The discharge under submerged conditions is:

Equation 51

$$Q_s = Q - Q_c$$

Where:

Q_c = Reduction of the modular discharge due to submergence.

Figure 77 gives the corrections Q_c for submergence for flumes with 6 inch, 9 inch and 1 foot throat width. The correction for the 1 foot flume is made applicable to other sizes by multiplying the correction Q_c for the 1 foot by the factors given in Figure 77 (1 foot flume).

Usually the smallest practical size of flume is selected because of economical reasons. In general the width should vary between 1/3 to 1/2 of the canal width. Often the head loss across the flume is the limiting factor.

The procedure for selecting the appropriate flume is as follows:

Step 1: Collect site information: maximum and minimum canal discharges, corresponding normal flow depths and canal dimensions.

Step 2: List flumes capable of taking the given discharge, using Table 32.

Then, for free flow at the maximum canal discharge:

i) List values of h_a for the maximum canal discharge passing through the flumes.

ii) Apply the submergence limit appropriate to the flume to find the value of h_b corresponding to the values of h_a (Table 32).

iii) Subtract h_b from the normal flow depth at maximum discharge to give the vertical distance from the canal bed to the flume crest level. This assumes that at maximum submergence the downstream stage is the same as that at h_b, and that the flow downstream of the flume is not affected by it.

iv) Find the head loss across the flume at maximum discharge (Figure 78). Add this to the downstream water depth to obtain the water depth upstream of the flume.

v) Select the smallest size of flume for which the upstream stage is acceptable.

Figure 77
Discharge corrections due to submergence for Parshall flumes with different throat width

a. Parshall flume with a throat width b of 6 inch or 15.2 cm

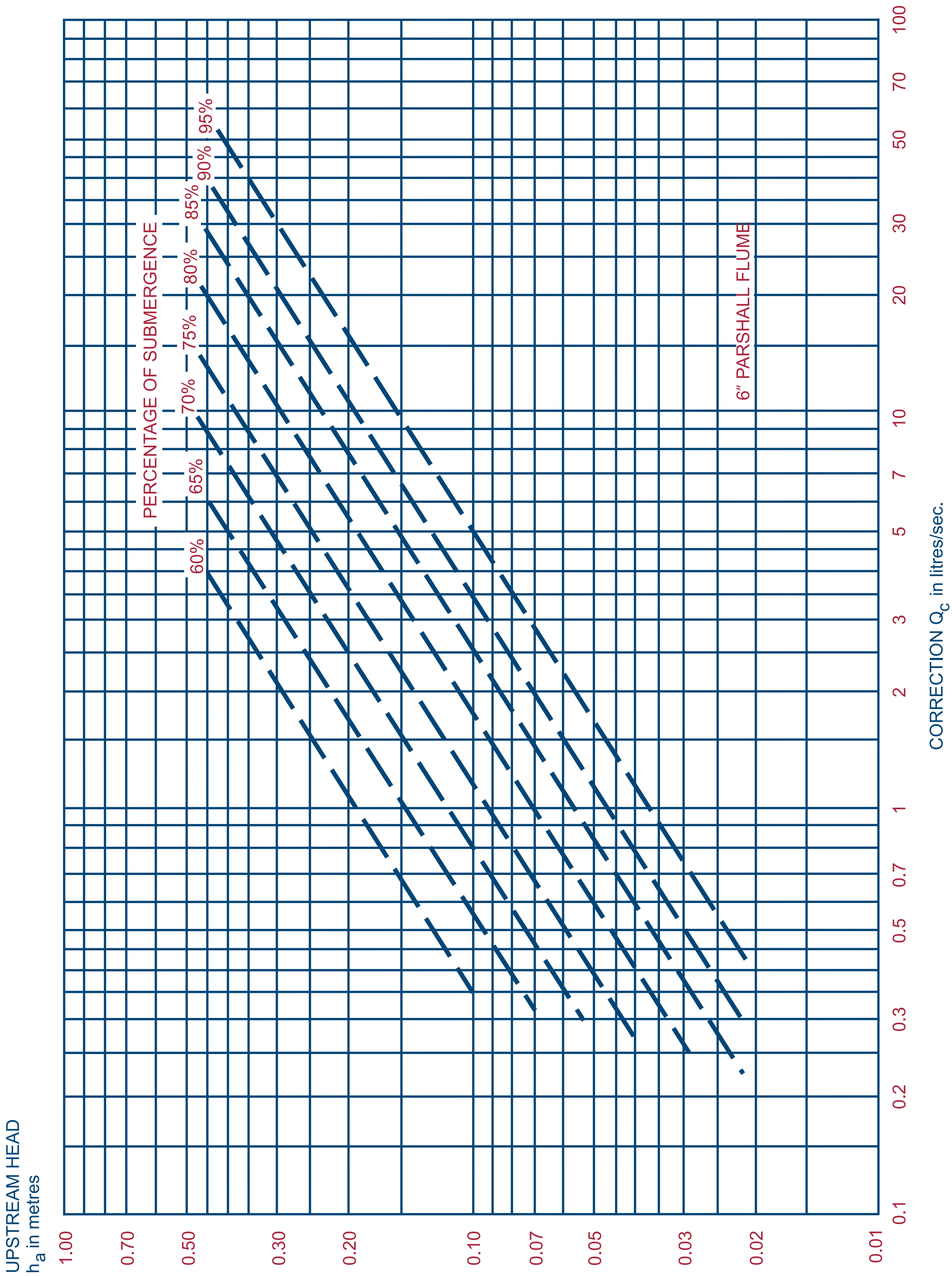


Figure 77

Discharge corrections due to submergence for Parshall flumes with different throat width

b. Parshall flume with a throat width b of 9 inch or 22.9 cm

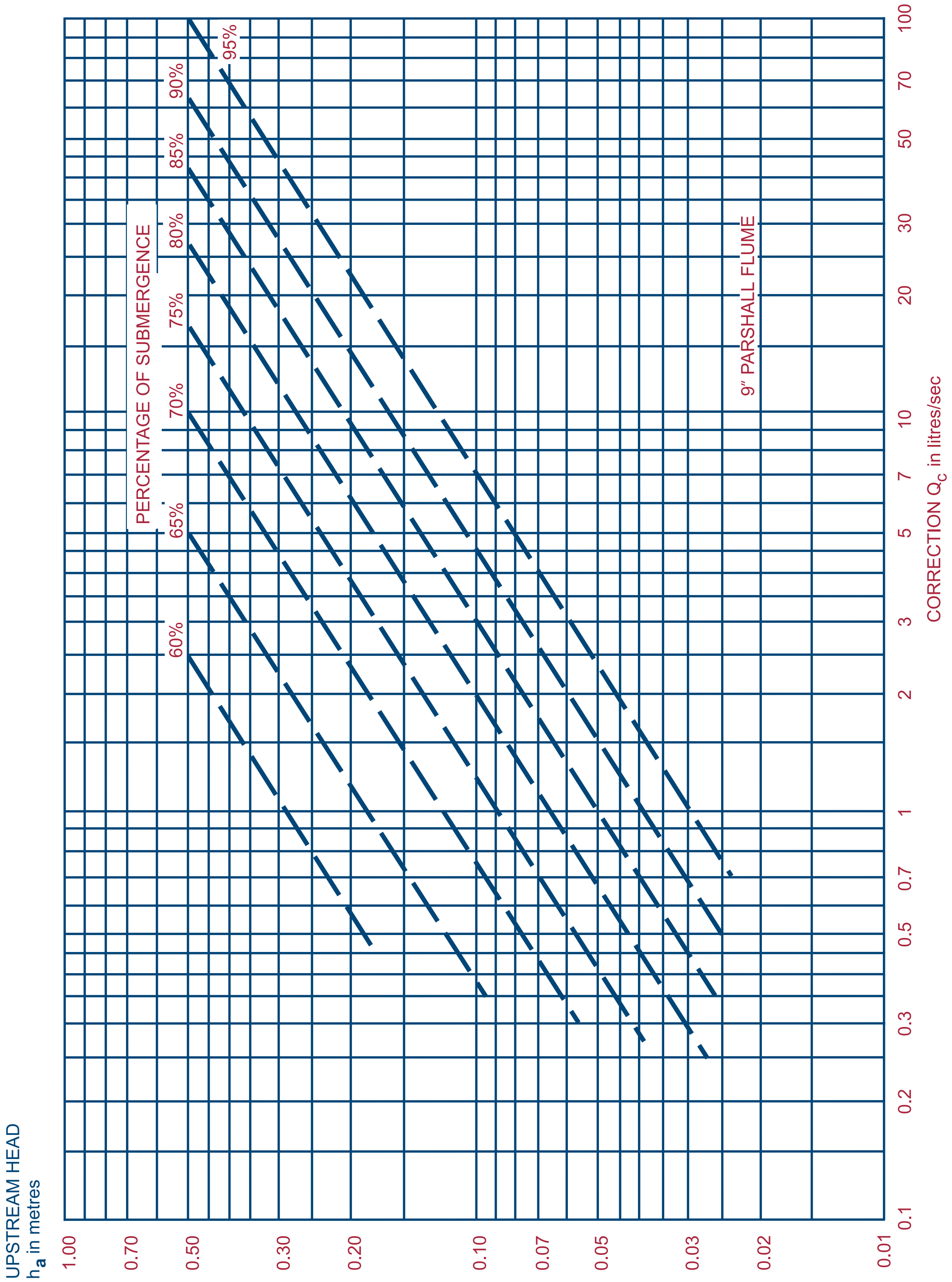


Figure 77
Discharge corrections due to submergence for Parshall flumes with different throat width

c. Parshall flume with a throat width b of 1 foot or 30.5 cm

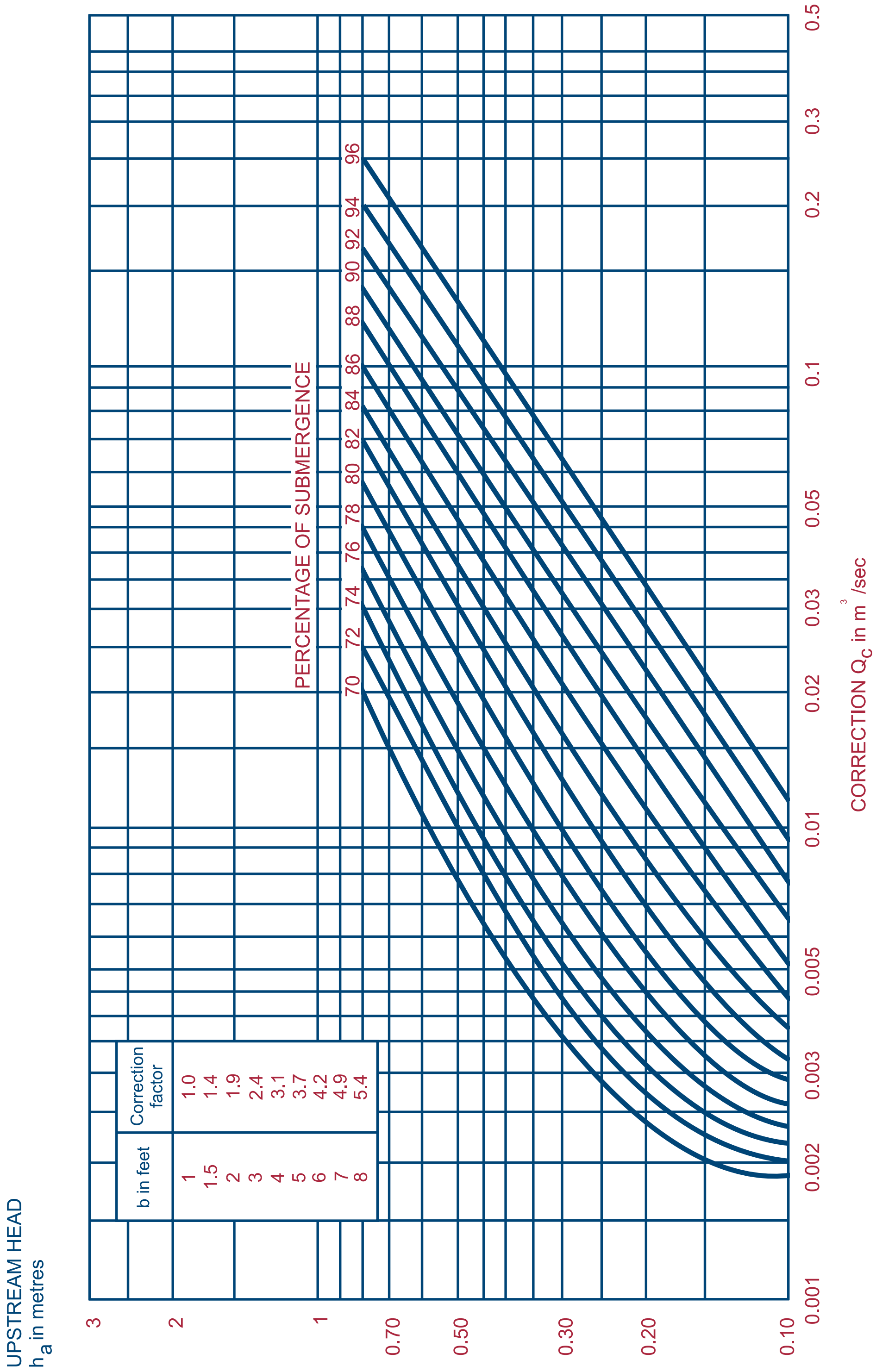
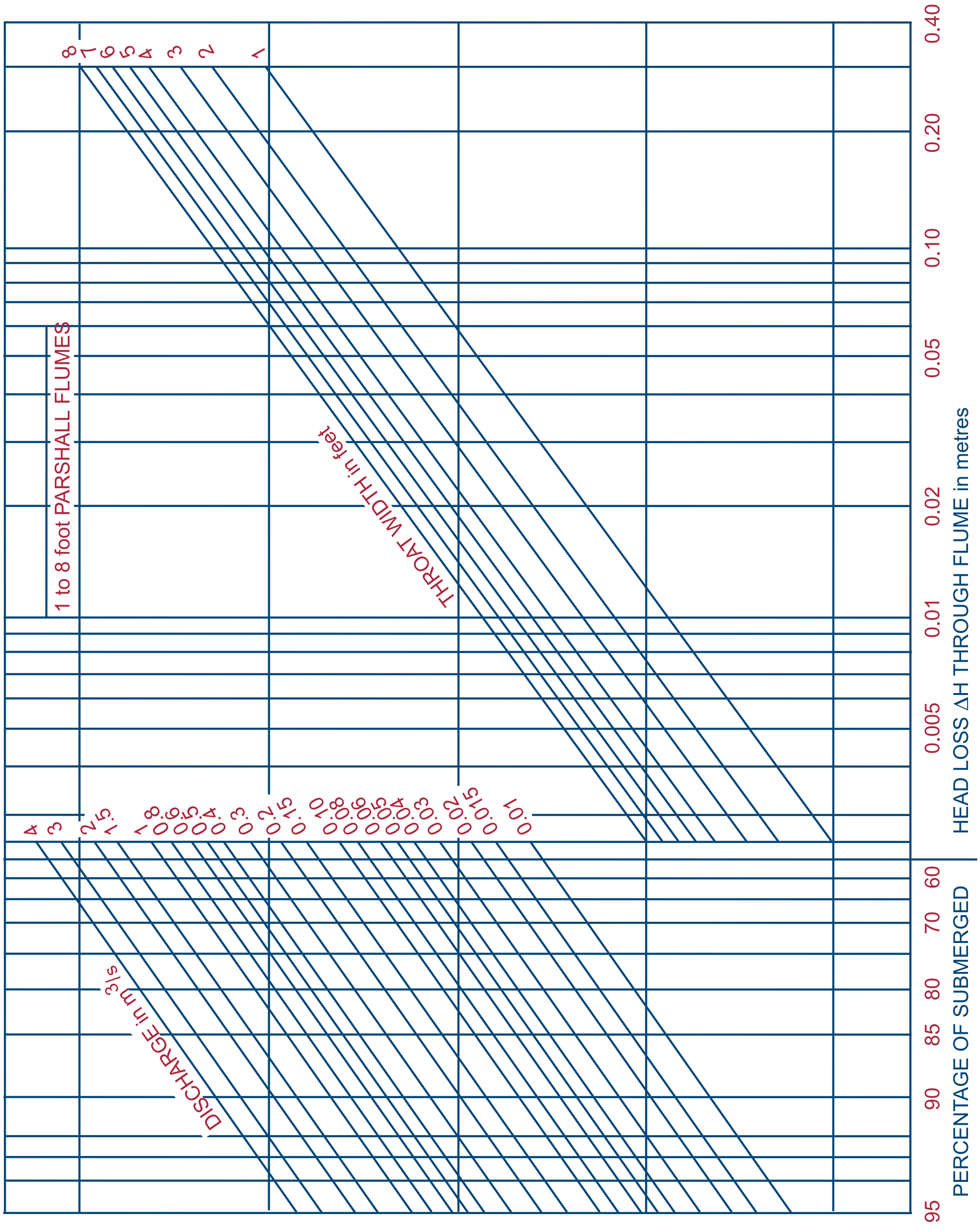


Figure 78
Head loss through Parshall flumes



Example 33

Select the most appropriate flume to be placed in a canal with the following characteristics:

Maximum discharge = 0.566 m^3/sec

Canal water depth = 0.77 m

Canal banks at 3 m apart

The freeboard of the canal = 0.15 m

a. Consider the flumes with a throat width of 3 and 4 foot. Table 32 gives discharge equations for the different flume sizes.

The discharge equation for the 3 foot flume is:

$$Q = 2.184 \times h_a^{1.566} \Rightarrow 0.566 = 2.184 \times h_a^{1.566} \Rightarrow h_a = 0.43 \text{ m}$$

The discharge equation for the 4 foot flume is:

$$Q = 2.953 \times h_a^{1.578} \Rightarrow 0.566 = 2.953 \times h_a^{1.578} \Rightarrow h_a = 0.35 \text{ m}$$

b. Assume that the submerging of 70% must not be exceeded. This means that $h_b = 0.70 \times h_a$ (Table 32). Thus for the 3 foot flume the water depth h_b = 0.30 m and for the 4 foot flume h_b = 0.25m.

c. The elevation of the crest above the bottom of the canal (K in Figure 76) equals the design water depth minus h_b. Thus K = 0.77 m - 0.30 m = 0.47 m for the 3 foot flume and K = 0.77 m - 0.25 m = 0.52 m for the 4 foot flume.

d. From Figure 78 it can be seen that the head loss is 0.16 m for the 3 foot flume and 0.13 m for the 4 foot flume. Thus the upstream water depth becomes 0.77 m + 0.16 m = 0.93 m and 0.77 m + 0.13 m = 0.90 m for the 3 foot and 4 foot flume respectively.

e. The upstream water depth of the 3 foot flume just exceeds the sum of the normal water depth and freeboard, thus overtopping would result. The 4 foot flume is just within the available limit of depth. Thus this flume could be selected for implementation. If there was sufficient freeboard available for either of the flumes, considering the rise in water level upstream of the flume, one should select the 3 foot flume because this is cheaper.

Trapezoidal flume

Whenever the canal section is not rectangular, trapezoidal flumes such as those shown in Figure 79, are often preferred, especially for measuring smaller discharges. A typical trapezoidal flume has an approach, a converging section, a throat, a diverging and an exit section. A minimum transition will be required. An additional advantage is the flat bottom, which allows sediment to pass through fairly easily. Furthermore, the loss in head may be less for comparable discharges.

Trapezoidal flumes are particularly suited for installation in concrete-lined canals. The flume should normally be put on top of the lining, thus constricting the flow section to the extent required for free flow conditions over a whole range of discharges up to the canal design discharge. As a rule of thumb, one can say that the lower the canal gradient the higher the elevation of the flume above the canal bed level.

The flow characteristics of the flume can be determined experimentally. This allows for the calibrations of the flume. As an example, a flume with dimensions such as those given in Figure 79 can be located in a canal with a bed width of 0.30 m (1 foot), having side slopes of 1:1. The range of calibrated water depth is 6-37 cm and the range of calibrated discharge is 1.4-169 l/sec. This will suit most conditions in a typical small-scale irrigation canal.

Cut-throat flume

The cut-throat flume has a converging inlet section, throat and diverging outlet section. The flume has a flat bottom and vertical walls (Figure 80).

It is preferable to have the cut-throat flume operating under free flow conditions. This facilitates measurements and ensures a high degree of accuracy. Free flow conditions through the cutthroat flume are described by the following equations:

Equation 52

$$Q = C \times (h_a)^n$$

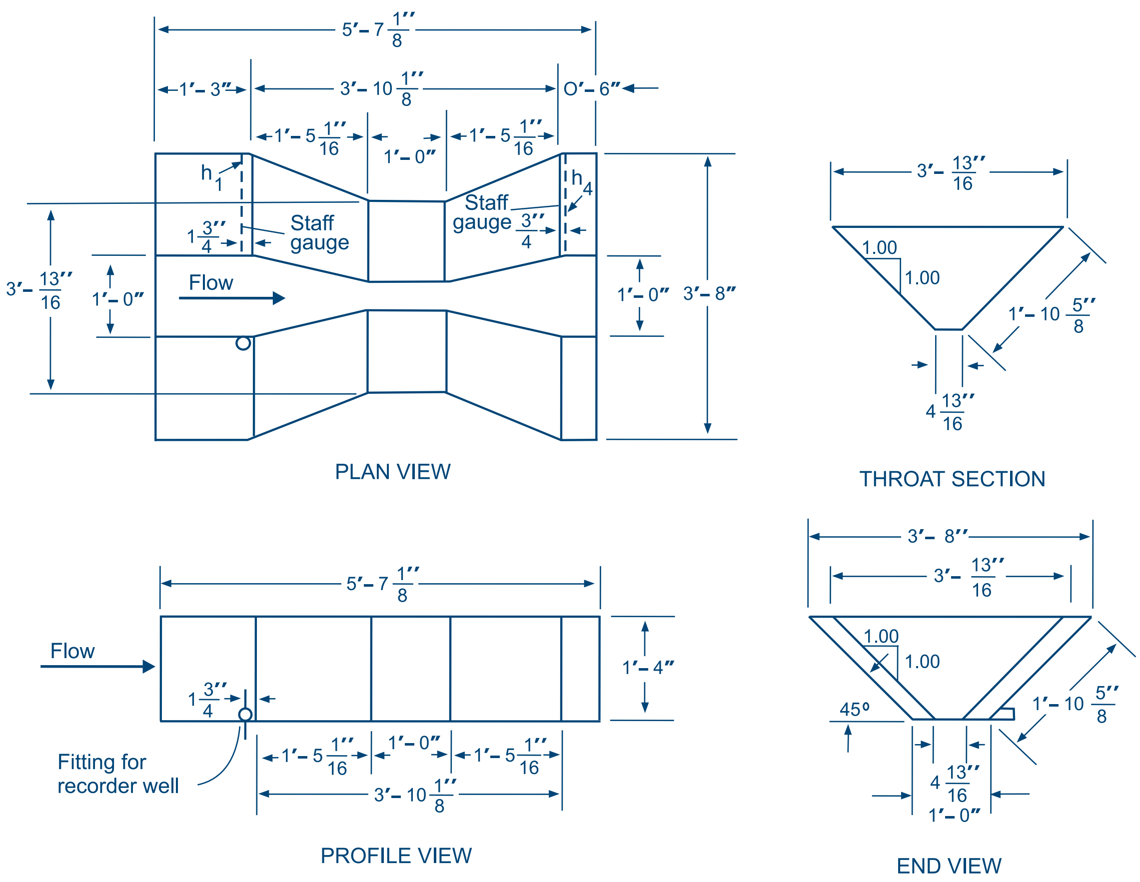
Equation 53

$$C = K \times W^{1.025}$$

Where:

Q = Discharge (m^3/sec)
C = Free flow coefficient
h_a = Upstream water depth (m)
K = Flume length coefficient
W = Throat width (m)

Figure 79
Trapezoidal flume (Source: FAO, 1975b)



For a given flume length, the values of n and K are obtained from Figure 81. In order to ensure free flow conditions, the ratio between the water depths h_a and h_b should not exceed a certain limit, which is called the transition submergence, S_t.

Example 34

A cut-throat flume is to be installed with a length L = 1.22 m and throat width W = 0.36 m. The maximum discharge through the structure is 0.20 m³/sec. How should it be installed in order to operate under free flow conditions?

From Figure 81, it follows that for a flume length L = 1.22 m:

S_t = 68.2%

K = 3.1

n = 1.75

Using Equations 53 and 52 respectively:

$$C = 3.1 \times 0.36^{1.025} = 1.088$$

$$Q = 1.088 \times h_a^{1.75} = 0.200 \Rightarrow h_a = 0.38 \text{ m}$$

$$S_t = \frac{h_b}{h_a} = 0.682 \Rightarrow h_b = 0.682 \times 0.38 = 0.26 \text{ m}$$

Therefore the floor of the flume should be placed not lower than 0.26 m below the normal water depth, in order to let pass the maximum discharge of 0.20 m³/sec. under free flow conditions.

Figure 80

Cut-throat flume (Source: FAO, 1975b)

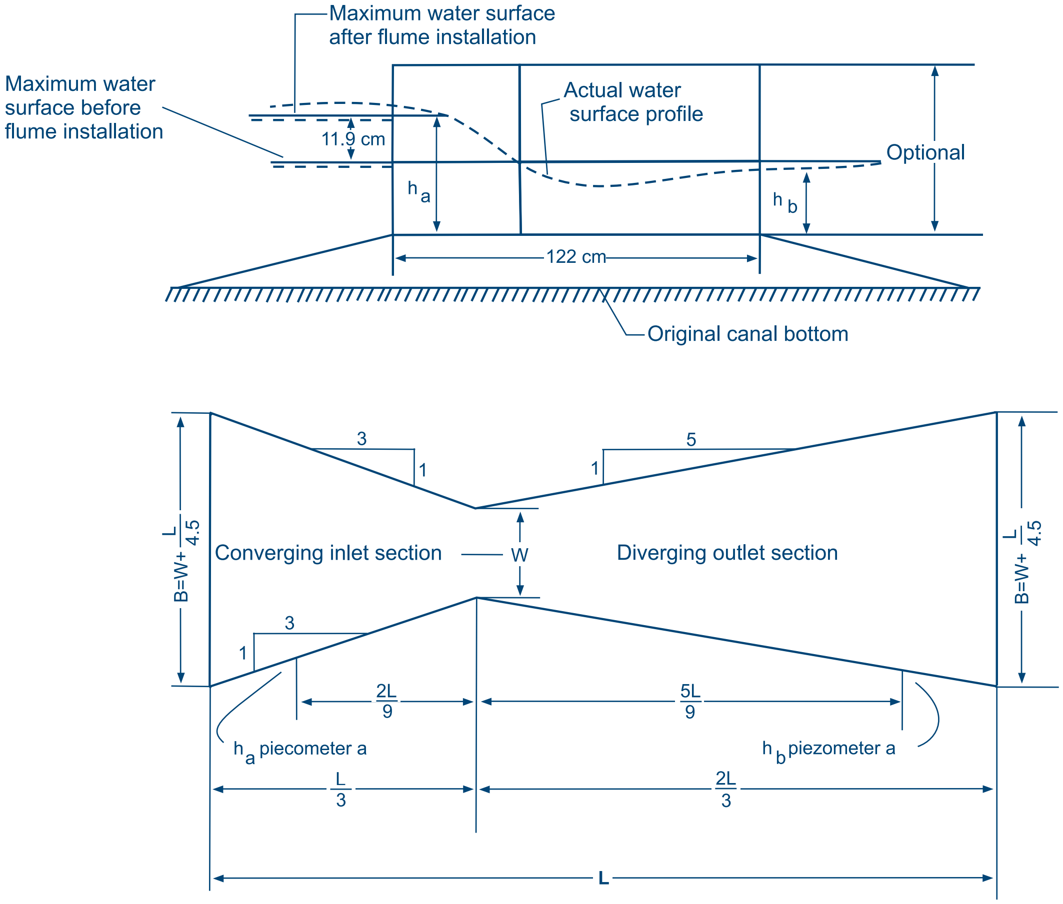
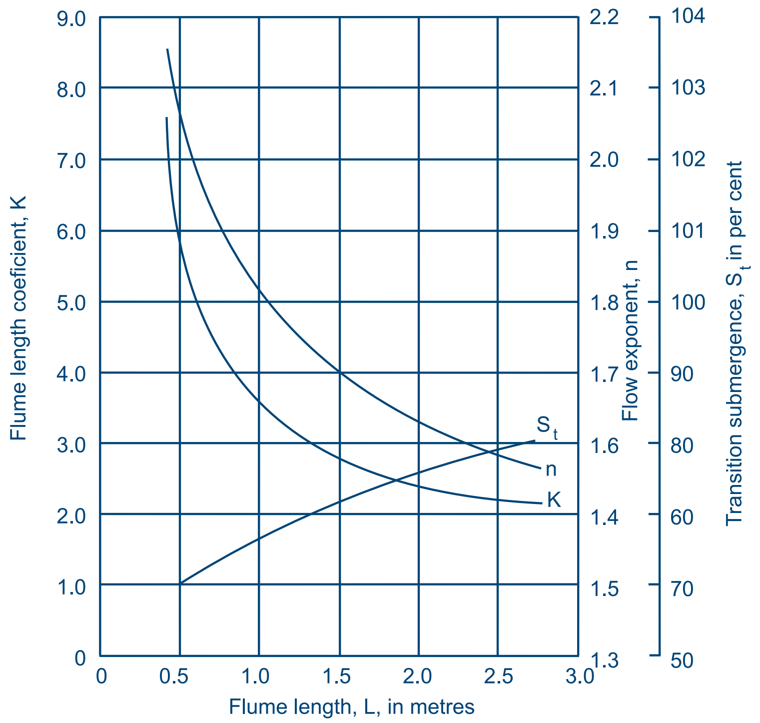


Figure 81

Cut-throat flume coefficients (Source: FAO, 1975b)



6.4.4. Orifices

Orifices, such as gates and short pipes, are also used as water measuring devices (Figure 82). However, they do not offer any advantage over the use of weirs or flumes. Furthermore, their calibrations are not as accurate nor as stable as other types of measuring devices.

For weirs the discharge is proportional to the head above the crest raised to the power 3/2 (Equations 44, 45, 46, 48). Therefore, they are sensitive to the fluctuations in the upstream water level. For orifices, including gates and short pipes, the discharge is proportional to the head of water above the crest raised to the power 1/2, as shown by Equation 34 (see Section 6.1.3). Therefore, they are less sensitive to small fluctuations of the upstream water level.

Under submerged conditions both the upstream and downstream sides of the structure need water level recordings. For free flow conditions, the discharge is a function of the upstream water depth alone.

Figure 82
Examples of orifices

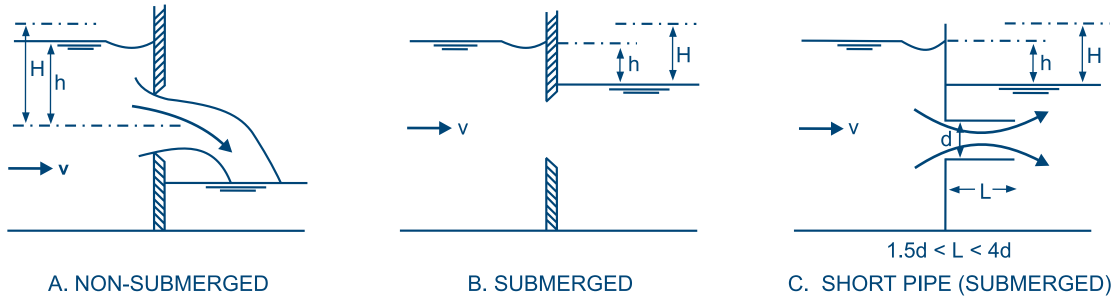
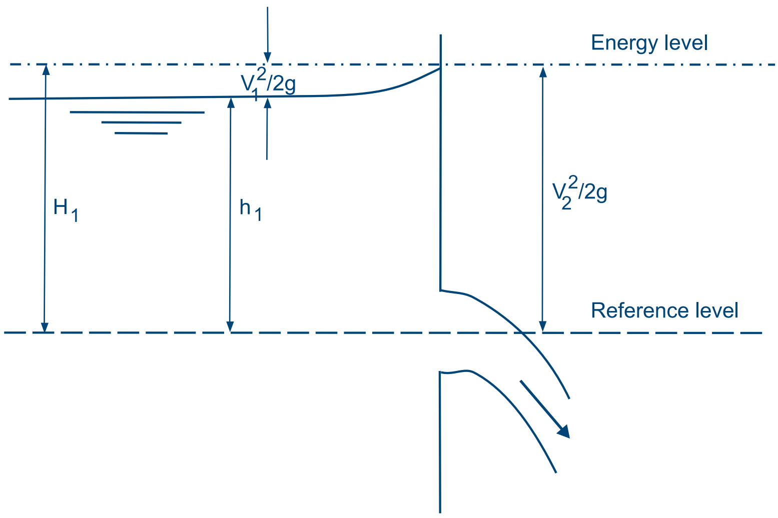


Figure 83
Free discharging flow through an orifice



Example 35

A circular orifice is placed in a canal, which discharges 0.0783 m^3/sec. The maximum allowable water depth over the centre of the orifice is 0.25 m. What should be the opening of the orifice?

Substituting the above data in Equation 34 gives:

$$0.0783 = 0.6 \times \left(\frac{1}{4} \times \pi \times d^2\right) \times (2 \times 9.81 \times 0.25)^{1/2} \Rightarrow d^2 = \frac{0.0783}{1.0437} = 0.075 \Rightarrow d = 0.27 \text{ m.}$$

Thus the diameter of the orifice should be 0.27 m

The general discharge equation for a free flow orifice is (Equation 34):

$$Q = C \times A \times (2gh_1)^{1/2}$$

Where:

Q = Design discharge through orifice (m^3/sec)
C = Design coefficient (approximately 0.60)
A = Cross-sectional area of the orifice (m^2)
g = Gravitational force (9.81 m/sec^2)
h_1 = Water depth upstream of orifice over reference level (m) (Figure 83)

Partially-opened sluice gates could be used for discharge measurements, in which case they will be acting like submerged orifices (Figure 84).

For partially-opened sluice gates and submerged orifices the discharge equation reads:

Equation 54

$$Q = C \times A\ (2g[h_1 - h_2])^{1/2}$$

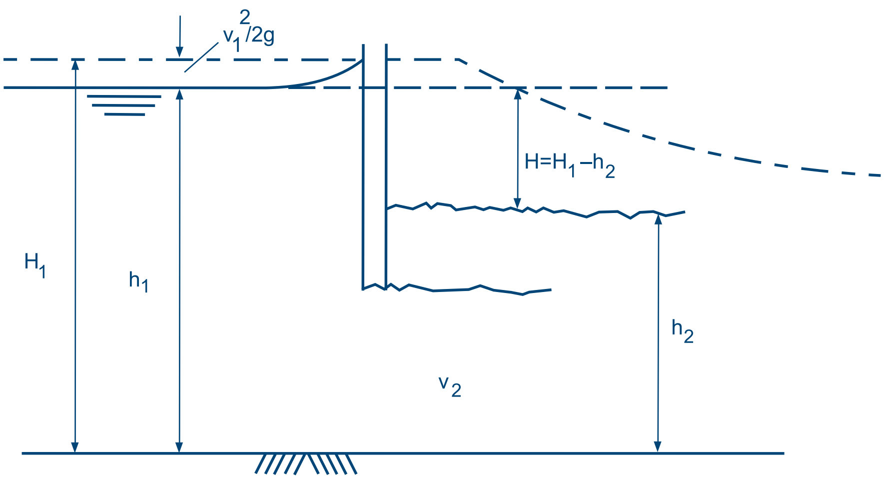
Where:

Q = Design discharge through orifice (m^3/sec)
C = Discharge coefficient, which is 0.63 for sluice gates and submerged orifices and 0.85 for short pipes
A = Cross-sectional area of the orifice (m^2)
g = Gravitational force (9.81 m/sec^2)
h_1 = Water depth upstream of orifice over reference level (m)
h_2 = Water depth downstream of the structure (m)

6.6.5. Current meters

Current meters are used to measure the velocity in a canal, from where the discharge can be calculated using the Continuity Equation 12 (see Section 5.1). Most current meters have a propeller axis in the direction of the current. The flowing water sets the propeller turning. On a meter,

Figure 84
Sluice gate under submerged conditions



Example 36

A sluice gate is installed in a canal with a design water depth of 0.30 m. The canal discharges 0.0783 m^3/sec. The maximum allowable rise in water level upstream of the sluice gate is 0.25 m. The width of the gate opening is 0.40 m. What should be the height d of the opening?

h_2 being 0.30 m and the allowable rise in water level upstream of the gate being 0.25 means that:

h_1 = 0.30 m + 0.25 m = 0.55 m.

Substituting the above data in Equation 54 gives:

$0.0783 = 0.63 \times (0.40 \times d) \times (2 \times 9.81 \times [0.55 - 0.30])^{1/2} \Rightarrow d = 0.14$ m.

forming part of the equipment, the number of revolutions per time unit can be read and, by means of a calibrated graph or table, the velocity can be determined. A well-known type of current meter is the Ott instrument C31 for velocities up to 10 m (Figure 85). Propeller meters are reliable and accurate, but rather expensive.

In measuring the velocities, the number of points per vertical and the number of verticals per cross-section should be determined. For this purpose, the quantity of work and the time required should be weighed against the degree of accuracy (Euroconsult, 1989). For example, measurements can be taken at 10 cm horizontal distance over the cross-section and at 0.2h and 0.8h depth at each 10 cm (h is the water depth). The velocity is the average of the velocity at 0.2h and 0.8h depth. If the water depth is less than 0.5-0.6 m, one reading can be done at 0.6h. Then, for each vertical the flow per unit width can be calculated according to $q = v_{average} \times h$ (Figure 86a). These q_s are distributed over the total width (Figure 86b) and the area between the q-line and the water surface gives the total discharge. It is also possible to establish the discharge per section and to consider the sum of the discharges in the sections as the total discharge.

Figure 85
Ott C31 propeller instrument

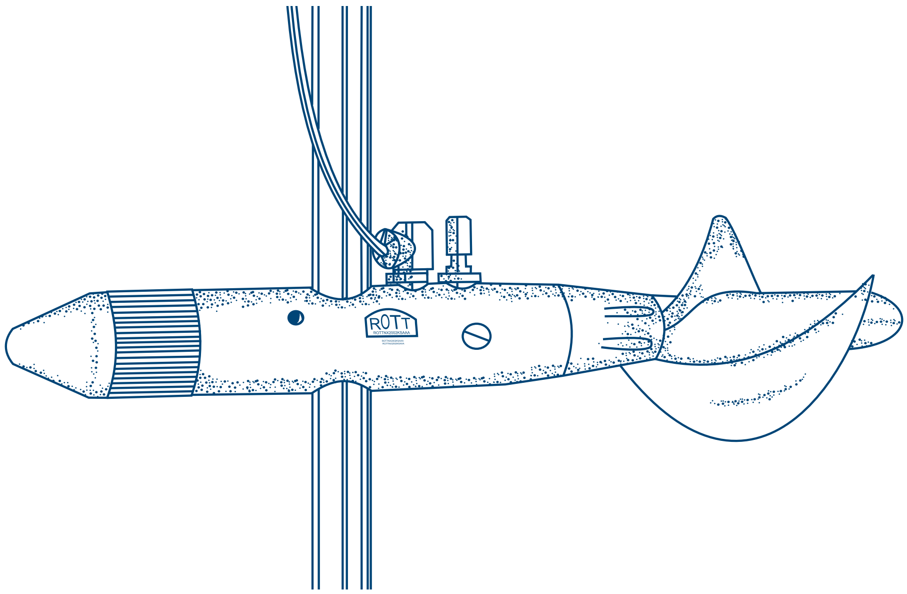
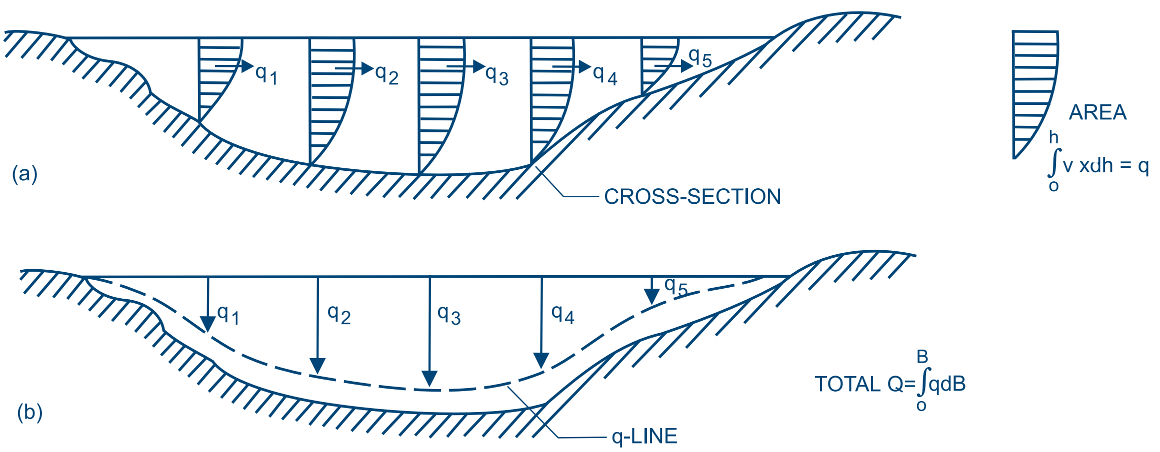


Figure 86
Depth-velocity integration method



6.7. Discharge measurement in pipelines

Several types of devices can be used to measure the discharge in pipelines. This section will discuss differential pressure and rotating mechanical meters, as they are the ones commonly used.

6.7.1. Differential pressure flow meters

Differential pressure flow meters create a pressure difference that is proportional to the square of the discharge. The pressure difference is created by causing flow to pass through a contraction. Manometers, bourdon gauges, or pressure transducers are normally utilized to measure the pressure difference. One good example of a differential pressure flow meter is the Venturi tube (Figure 87).

Venturi tube

The pressure drop between the inlet and throat is created as water passes through the throat. In the section downstream of the throat, the gradual increase in cross-sectional area causes the velocity to decrease and the pressure to increase. The pressure drop between the Venturi inlet and the throat is related to the discharge, as follows:

Equation 55

$$Q = \frac{Cd^2K(P_1 - P_2)^{1/2}}{[1 - (d/D)^2]^{1/2}}$$

Where:

Q = Discharge (l/min)
C = Flow coefficient
D = Diameter of upstream section (cm)
d = Diameter of contraction (cm)
P_1 = Pressure in upstream section (kPa)
P_2 = Pressure in contraction (kPa)
K = Unit constant (K is 6.66 for Q in l/min, d and D in cm, and P_1 and P_2 in kPa)

The flow coefficient C for a Venturi metre is 0.97.

6.7.2. Rotating mechanical flow meters

There are many types of rotating mechanical flow meters used in pipelines. These flow meters normally have a rotor that revolves at a speed roughly proportional to the discharge and a device for recording and displaying the discharge and total volume. The rotor may be a propeller or axial flow turbine, or a vane-wheel with the flow impinging tangentially at one or more points.

Calibration tests are usually needed to accurately relate rotor revolutions to the flow. The lowest discharge that can be accurately measured by a rotating mechanical flow meter depends on the amount of bearing friction that can be tolerated while the occurrence of cavitation often establishes the largest flow rate that can be measured (see Module 5). Head loss through most rotating mechanical discharge meters is moderate.

Figure 87
Venturi flow meter