8 Orifices

A well defined opening in a plate or bulkhead, the top of which is placed well below the upstream water level, is classified here as an orifice.

8.1 Circular sharp-edged orifice

8.1.1 Description

A circular sharp-edged orifice is an opening in a (metal) plate or bulkhead, which is placed perpendicular to the sides and bottom of a straight approach channel. For true orifice flow to occur, the upstream water level must always be well above the top of the opening, such that vortex-flow with air entrainment is not evident. If the upstream water level drops below the top of the opening, it no longer performs as an orifice but as a weir (see Section 5.4).

This orifice is one of the older devices used for measuring water and formerly it was set up to discharge freely into the air, resulting in a considerable loss of head. To overcome this excessive head loss, the orifice is now arranged with the tailwater above the top of the opening. This 'submerged orifice' conserves head and can be used where there is insufficient fall for a sharp-crested weir. Circular orifices have the advantage that the opening can be turned and its edges bevelled with precision on a lathe. Another advantage is that during installation no levelling is required.

In practice, circular sharp-edged orifices are fully contracted so that the bed and sides of the approach channel and the free water surface should be sufficiently remote from the control section to have no influence on the contraction of the discharging jet. The fully contracted orifice may be placed in a non-rectangular approach channel, provided that the dimensions comply with those explained in Section 8.1.3.

A general disadvantage of submerged orifices is that debris, weeds and sediment can accumulate upstream of the orifice, and may prevent accurate measurements. In sediment-laden water, it is especially difficult for maintenance personnel to determine whether the orifice is obstructed or completely open to flow. To prevent the overtopping of the embankments in the case of a blocked orifice, the top of the orifice wall should only be to the maximum expected upstream water level so it can act as an overflow weir.

Orifice plates are simple, inexpensive and easy to install, which makes them suitable as a portable device to measure streamflow. An example of a portable orifice plate with three ranges of measurement is shown in Figure 8.1. The orifice plate shown contains three slots covered with clear vinyl plastic to permit the reading of the differential head from the downstream side of the plate. Since flow through this orifice must be submerged it may be necessary to restrict the downstream channel in order to raise the tailwater level above the top of the orifice.

8.1.2 Determination of discharge

The basic head-discharge equations for orifice flow, according to Section 1.12, are

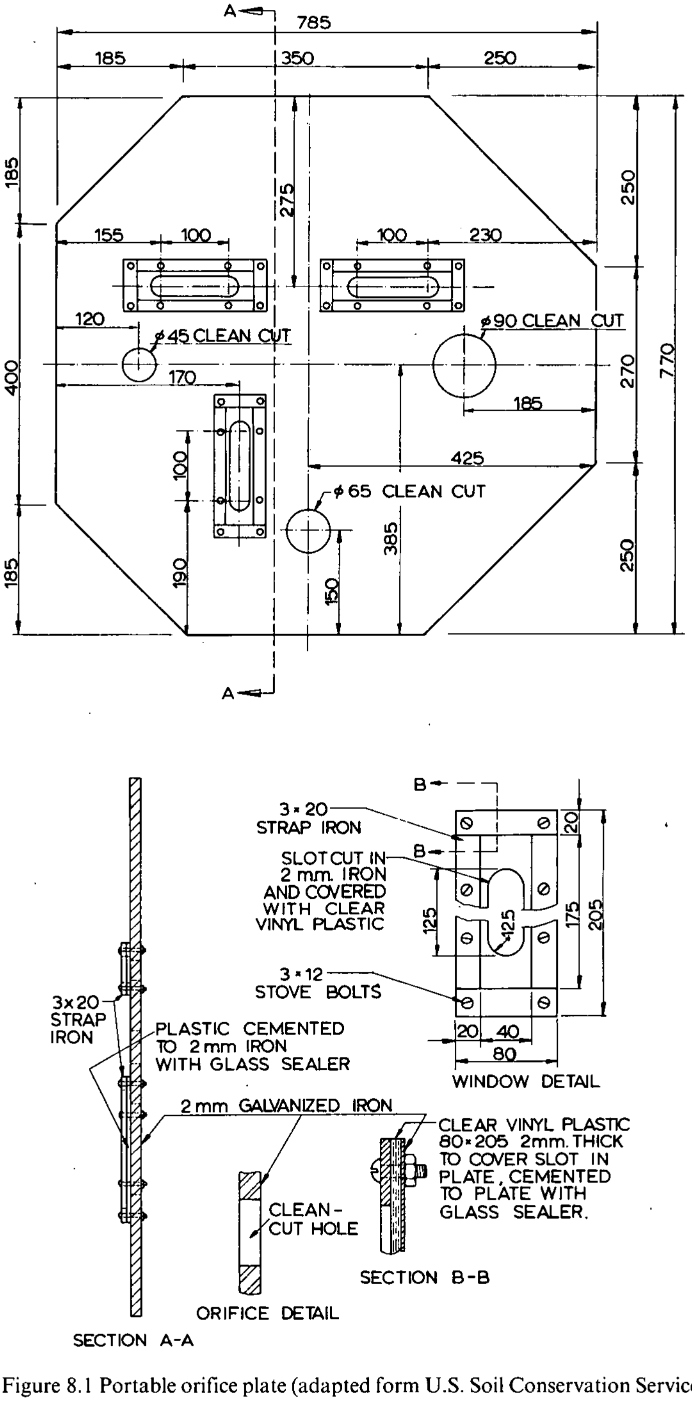


Figure 8.1 Portable orifice plate (adapted form U.S. Soil Conservation Service)

$$Q = C_d C_v A \sqrt{2g(h_1 - h_2)} \quad (8\text{-}1)$$

for submerged flow conditions, and

$$Q = C_d C_v A \sqrt{2g\Delta h} \quad (8\text{-}2)$$

if the orifice discharges freely into the air. In these two equations $h_1 - h_2$ equals the head differential across the orifice and Δh equals the upstream head above the centre of the orifice (see Figures 1.8 and 1.19). A is the area of the orifice and equals $\frac{1}{4}\pi d^2$, where d is the orifice diameter.

Orifices should be installed and maintained so that the approach velocity is negligible, thus ensuring that C_v approaches unity. Calibration studies performed by various research workers have produced the average C_d-values shown in Table 8.1.

The error in the discharge coefficient for a well-maintained circular sharp-crested orifice, constructed with reasonable care and skill, is expected to be of the order of 1%. The method by which the coefficient error is to be combined with other sources of error is shown in Annex 2.

Table 8.1 Average discharge coefficients for circular orifices (negligible approach velocity)

Orifice diameter 'd' in metres	C_d free flow	C_d submerged flow
0.020	0.61	0.57
0.025	0.62	0.58
0.035	0.64	0.61
0.045	0.63	0.61
0.050	0.62	0.61
0.065	0.61	0.60
$\geqslant$ 0.075	0.60	0.60

8.1.3 Limits of application

To ensure full contraction and accurate flow measurement, the limits of application of the circular orifice are:

a. The edge of the orifice should be sharp and smooth and be in accordance with the profile shown in Figure 5.1;
b. The distance from the edge of the orifice to the bed and side slopes of the approach and tailwater channel should not be less than the radius of the orifice. To prevent the entrainment of air, the upstream water level should be at a height above the top of the orifice which is at least equal to the diameter of the orifice;
c. The upstream face of the orifice plate should be vertical and smooth;
d. To make the approach velocity negligible, the wetted cross-sectional area at the upstream head-measurement station should be at least 10 times the area of the orifice;
e. The practical lower limit of the differential head, across the orifice is related to fluid properties and to the accuracy with which gauge readings can be made. The recommended lower limit is 0.03 m.

8.2 Rectangular sharp-edged orifice

8.2.1 Description

A rectangular sharp-edged orifice used as a discharge measuring device is a well-defined opening in a thin (metal) plate or bulkhead, which is placed perpendicular to the bounding surfaces of the approach channel. The top and the bottom edges should be horizontal and the sides vertical.

Since the ratio of depth to width of (irrigation) canals is generally small and because changes in depth of flow should not influence the discharge coefficient too rapidly, most (submerged) rectangular orifices have a height, w, which is considerably less than the breadth, b_c. The principal type of orifice for which the discharge coefficient has been carefully determined in laboratory tests is the submerged, fully contracted, sharp-edged orifice. Since the discharge coefficient is not so well defined where the contraction is partially suppressed, it is advisable to use a fully contracted orifice wherever conditions permit. Where sediment is transported it may be necessary to place the lower edge of the orifice at canal bed level to avoid the accumulation of sediments on the upstream side. If the discharge must be regulated it may even be desirable to suppress both bottom and side contractions so that the orifice becomes an opening below a sluice gate.

A submerged orifice structure is shown in Figure 8.2. A box is provided downstream from the orifice to protect unlined canals from erosion. Both the sides and the floor of this box should be set outward from the orifice a distance of at least two times the height of the orifice. To ensure that the orifice is submerged or to cut off the flow, an adjustable gate may be provided at the downstream end of the orifice box.

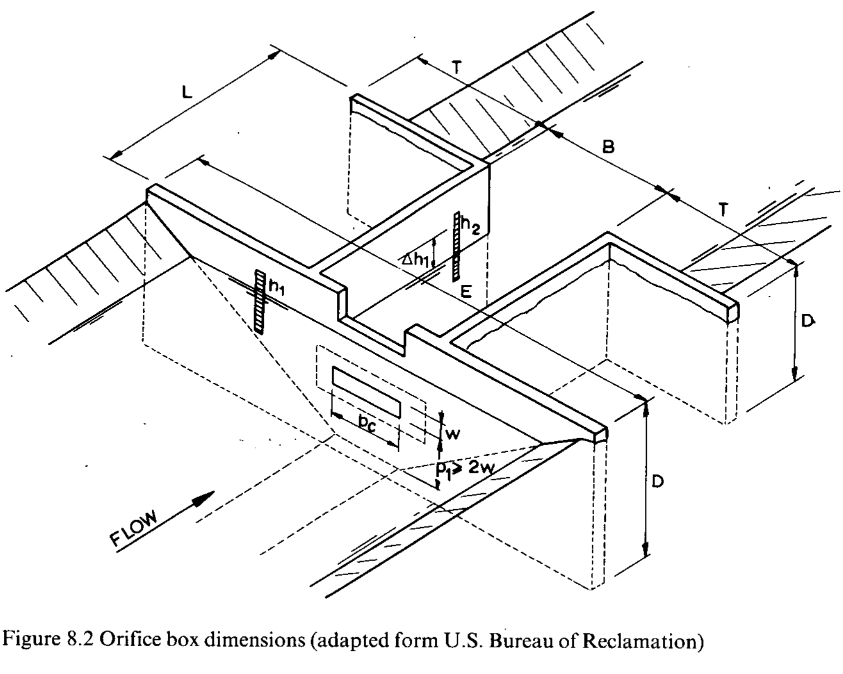


Figure 8.2 Orifice box dimensions (adapted form U.S. Bureau of Reclamation)

This gate should be a sufficient distance downstream from the orifice so as not to disturb the issuing jet.

The top of the vertical orifice wall should not be higher than the maximum expected water level in the canal, so that the wall may act as an overflow weir if the orifice should become blocked. Suitable submerged orifice-box dimensions for a concrete, masonry, or wooden structure as shown in Figure 8.2 are listed in Table 8.2.

Table 8.2 Recommended box sizes and dimensions for a submerged orifice (after U.S. Bureau of Reclamation 1967)

Orifice size		Height of structure	Width of head wall	Length	Width	Length of downstream head wall
height w	breadth b_c	D	E	L	B	T
0.08	0.30	1.20	3.00	0.90	0.75	0.60
0.08	0.60	1.20	3.60	0.90	1.05	0.60
0.15	0.30	1.50	3.60	1.05	0.75	0.90
0.15	0.45	1.50	4.25	1.05	0.90	0.90
0.15	0.60	1.50	4.25	1.05	1.05	0.90
0.23	0.40	1.80	4.25	1.05	0.90	0.90
0.23	0.60	1.80	4.90	1.05	1.05	0.90

8.2.2 Determination of discharge

The basic head-discharge equation for submerged orifice flow, according to Section 1.12 is

$$Q = C_d C_v A\sqrt{2g(h_1 - h_2)} \qquad (8\text{-}3)$$

where $h_1 - h_2$ equals the head differential across the orifice, and A is the area of the orifice and equals the product wb_c. In general, the submerged orifice should be designed and maintained so that the approach velocity is negligible and the coefficient C_v approaches unity. Where this is impractical, the area ratio A^*/A_1 may be calculated and a value for C_v obtained from Figure 1.12.

For a fully contracted, submerged, rectangular orifice, the discharge coefficient $C_d = 0.61$. If the contraction is suppressed along part of the orifice perimeter, then the following approximate discharge coefficient may be used in Equation 8-3, regardless of whether the orifice bottom only or both orifice bottom and sides are suppressed

$$C_d = 0.61\,(1 + 0.15\,r) \qquad (8\text{-}4)$$

where r equals the ratio of the suppressed portion of the orifice perimeter to the total perimeter.

If water discharges freely through an orifice with both bottom and side contractions suppressed, the flow pattern equals that of the free outflow below a vertical sluice gate as shown in Figure 8.3. The free discharge below a sluice gate is a function of the upstream water depth and the gate opening:

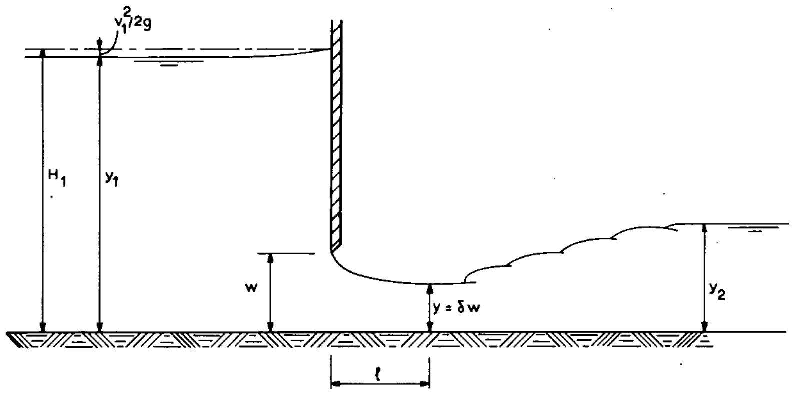


Figure 8.3 Flow below a sluice gate

$$Q = C_d C_v b_c w \sqrt{2g(y_1 - y)} \tag{8-5}$$

If we introduce the ratios $n = y_1/w$ and $\delta = y/w$, where δ is the contraction coefficient, Equation 8-5 may be written as

$$Q = C_d C_v b_c w^{1.5} \sqrt{2g(n - \delta)} \tag{8-6}$$

which may be simplified to

$$Q = K\, b_c w^{1.5} \sqrt{2g} = A\, w^{0.5}\, K \sqrt{2g} \tag{8-7}$$

where the coefficient K is a function of the ratio $n = y_1/w$ as shown in Table 8.3.

Table 8.3 Coefficients for free flow below a sluice gate

Ratio	Contraction coefficient	Discharge coefficient Eq. 8-6	Coefficient Eq. 8-7	$K\sqrt{2g}$ Eq. 8-7
$n = y_1/w$	δ	C_d	K	$m^{1/2}\ s^{-1}$
1.50	0.648	0.600	0.614	2.720
1.60	0.642	0.599	0.641	2.838
1.70	0.637	0.598	0.665	2.946
1.80	0.634	0.597	0.689	3.052
1.90	0.632	0.597	0.713	3.159
2.00	0.630	0.596	0.735	3.255
2.20	0.628	0.596	0.780	3.453
2.40	0.626	0.596	0.823	3.643
2.80	0.625	0.598	0.905	4.010
3.00	0.625	0.599	0.944	4.183
3.50	0.625	0.602	1.038	4.597
4.00	0.624	0.604	1.124	4.977
4.50	0.624	0.605	1.204	5.331
5.00	0.624	0.607	1.279	5.664

Adapted from Franke 1968

Some authors prefer to describe a sluice gate as a half-model of a two-dimensional jet as shown in Figure 1.20, the bottom of the channel being the substitute for the plane of symmetry of the jet. Hence a discharge equation similar to Equation 1-67 is used to determine the free flow below the gate. This is

$$Q = C_e A\sqrt{2gy_1} \tag{8-8}$$

where C_e also expresses the influence of the approach velocity, since it is a function of the ratio y_1/w. The results of experiments by Henry (1950) are plotted in Figure 8.4, which show values of C_e as a function of y_1/w and y_2/w for both free and submerged flow below the sluice gate. The C_e-values read from Figure 8.4 will result in considerable errors if the difference between y_1/w and y_2/w becomes small (< 1.0). This condition will generally be satisfied with small differential heads and thus we recommend that the submerged discharge be evaluated by the use of Equations 8-3 and 8-4.

The results obtained from experiments by Henry, Franke and the U.S. Bureau of Reclamation are in good agreement. In this context it should be noted that the velocity $\sqrt{2gy_1}$ does not occur anywhere in the flow system; it simply serves as a convenient reference velocity for use in Equation 8-8.

The discharge coefficients given for the fully contracted submerged orifice (C_d = 0.61) and for free flow below a sluice gate in Table 8.3 can be expected to have an error of the order of 2%. The coefficient given in Equation 8-4 for flow through a submerged partially suppressed orifice can be expected to have an error of about 3%.

The method by which the coefficient error is to be combined with other sources of error is shown in Annex 2.

8.2.3 Modular limit

Free flow below a sluice gate occurs as long as the roller of the hydraulic jump does not submerge the section of minimum depth of the jet, which is located at a distance of

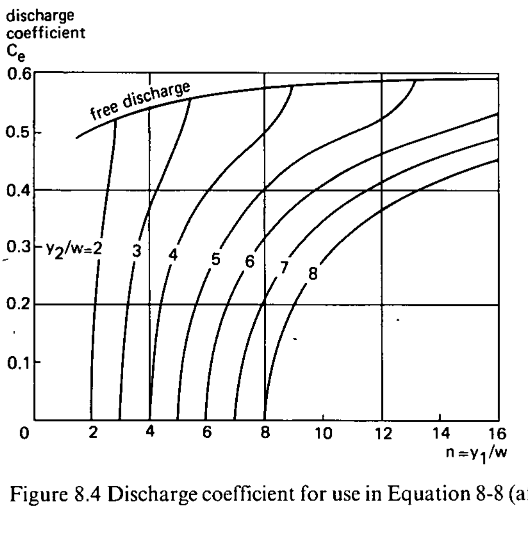


Figure 8.4 Discharge coefficient for use in Equation 8-8 (after Henry 1950)

$$\ell = w/\delta = y_1/n\delta \qquad (8\text{-}10)$$

downstream of the face of the vertical gate. To ensure such free flow, the water depth, y_2, downstream of the hydraulic jump should not exceed the alternate depth to $y = \delta w$, or according to the equation

$$\frac{y_2}{w} < \frac{\delta}{2}\left[\sqrt{1 + 16\left(\frac{H_1}{\delta w} - 1\right)} - 1\right] \qquad (8\text{-}11)$$

Relative numbers y_2/w worked out with the theoretical minimum contraction coefficient $\delta = 0.611$, corresponding to high values of the ratio n, are given in Figure 8.5 as a function of y_1/w.

8.2.4 Limits of application

To ensure accurate flow measurements, the limits of application of the rectangular sharp-edged orifice are:

a. The upstream edge of the orifice should be sharp and smooth and be in accordance with the profile shown in Figure 5.1;
b. The upstream face of the orifice should be truly vertical;
c. The top and bottom edges of the orifice should be horizontal;
d. The sides of the orifice should be vertical and smooth;
e. The distance from the edge of the orifice to the bed and side slopes of the approach and tailwater channel should be greater than twice the least dimension of the orifice if full contraction is required;
f. The wetted cross-sectional area at the upstream head-measurement station should be at least 10 times the area of the orifice so as to make the approach velocity negligible; this is particularly recommended for fully contracted orifices;

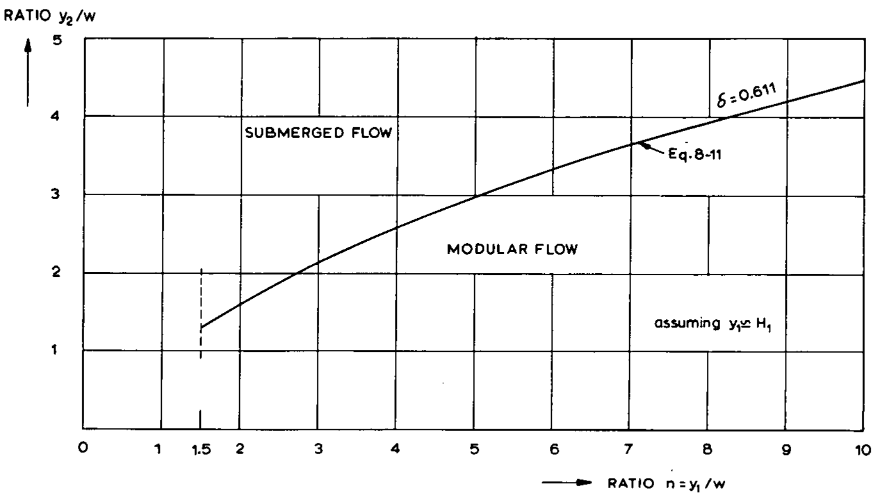


Figure 8.5 Limiting tail-water level for modular flow below a sluice gate

g. If the contraction is suppressed along the bottom or sides of the orifice, or along both the bottom and sides, the edge of the orifice should coincide with the bounding surface of the approach channel;
h. The practical lower limit of the differential head across the submerged orifice is related to fluid properties and to the accuracy to which gauge readings can be made. The recommended lower limit is 0.03 m;
i. If the contraction along bottom and sides is suppressed, the upstream head should be measured in the rectangular approach channel;
j. The upper edge of the orifice should have an upstream submergence of 1.0w or more to prevent the formation of air-entraining vortices;
k. A practical lower limit of $w = 0.02$ m and of $y_1 = 0.15$ m should be observed.

8.3 Constant-head-orifice

8.3.1 Description

The constant-head-orifice farm turnout (CHO) is a combination of a regulating and measuring structure that uses an adjustable submerged orifice for measuring the flow and a (downstream) adjustable turnout gate for regulation. The turnouts are used to measure and regulate flows from main canals and laterals into smaller ditches and are usually placed at 90° angle to the direction of flow in the main canal. The CHO was developed by the United States Bureau of Reclamation and is so named because its operation is based upon setting and maintaining a constant head differential, Δh, across the orifice. Discharges are varied by changing the area of the orifice. A typical constant head-orifice turnout installation is shown in Figure 8.6.

To set a given flow, the orifice opening A required to pass the given discharge is determined from a graph or table, and the orifice gate is set at this opening. The downstream turnout gate is then adjusted until the head differential as measured over the orifice gate equals the required constant-head, which usually equals 0.06 m. The discharge will then be at the required value. The rather small differential head used is one of the factors contributing to the inaccuracy of discharge measurements made by the CHO. For instance, errors of the order of 0.005 m in reading each staff gauge may cause a maximum cumulative error of 0.01 m or about 16% in Δh, which is equivalent to 8% error in the discharge. Introducing a larger differential head would reduce this type of error, but larger flow disturbances would be created in the stilling basin between the two gates. Furthermore, it is usually desirable to keep head losses in an irrigation system as low as possible.

Since the downstream gate merely serves the purpose of setting a constant head differential across the orifice gate, its shape is rather arbitrary. In fact, the turnout gate shown in Figure 8.6 may be replaced by a movable weir or flap-gate if desired. If the CHO is connected to a culvert pipe that is flowing full, the air pocket immediately downstream of the turnout gate should be aerated by means of a ventilation pipe. The diameter of this pipe should be 1/6 of the culvert diameter to provide a stable flow pattern below the turnout gate.

If the flow through the downstream gate is submerged, a change of tailwater level of the order of a few centimetres will cause an equivalent change of water level in the basin between the two gates. Under field conditions, the discharge in the main

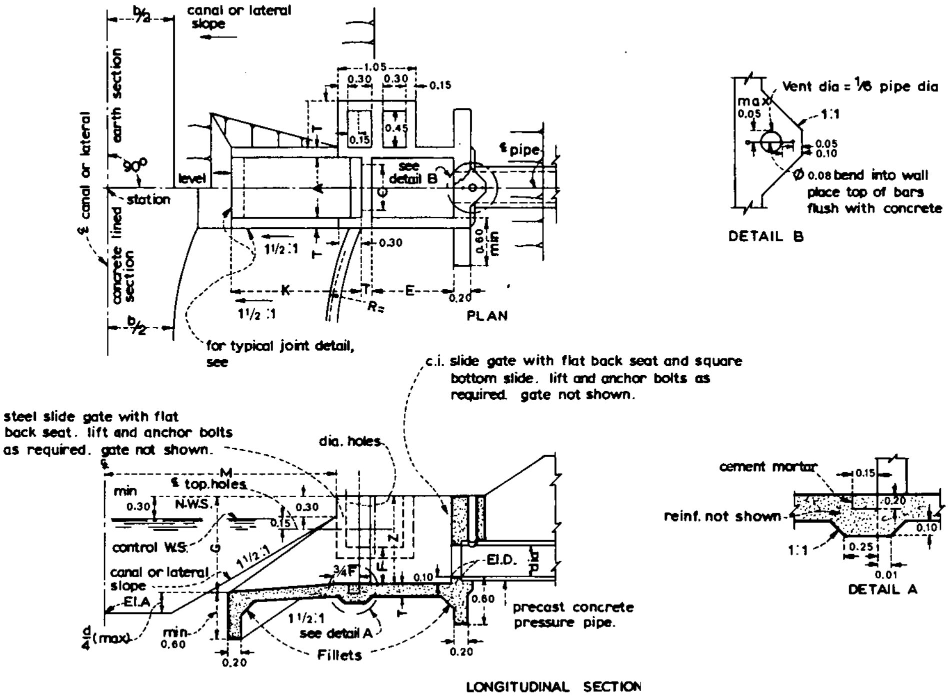


Figure 8.6 Example of a constant-head-orifice (adapted form USBR 1970)

canal is likely to be large compared with the discharge through the turnout. Hence the head differential over the orifice gate will change with any change in tailwater level, resulting in a considerable error in the diverted flow. The reader will note that if reasonable accuracy is required in discharge measurement, the flow below the turnout gate should be supercritical at all tailwater levels. For this to occur, the turnout gate requires a minimum loss of head which may be calculated as explained in Section 8.2.2 and with the aid of Figure 8.5. The combined loss of head over the orifice gate (usually 0.06 m) and over the turnout gate (variable) to produce modular flow is considerable.

Usually the CHO is placed at an angle of 90° from the centre line of the main canal, and no approach channel is provided to the orifice gate. As a result, the flow in the main canal will cause an eddy and other flow disturbances immediately upstream of the orifice gate opening, thus affecting the flow below the orifice gate. Such detrimental effects increase as the flow velocity in the main canal increases and are greater if the CHO is working at full capacity. Full-scale tests showed a deviation of the discharge coefficient of as much as 12% about the mean C_d-values with high flow velocities (1.0 m/s) and with larger orifice gate openings. The approach flow conditions, and thus the accuracy of the CHO can be improved significantly by introducing an approach channel upstream of the orifice gate. For example, if the CHO is used in combination with a culvert under an inspection road, the CHO could be placed at the downstream end of the culvert, provided that the culvert has a free water surface.

Since the CHO is usually operated at a differential head of 0.06 m (0.20 foot) it is clear that extreme care should be taken in reading heads. Fluctuations of the water surfaces just upstream of the orifice gate and in the stilling basin downstream of the orifice can easily result in head-reading errors of one or more centimetres if the heads are read from staff gauges. This is particularly true if the CHO is working at full capacity. Tests have revealed that, with larger orifice-gate openings, staff gauge readings may show a negative differential head while piezometers show a real differential head of 0.06 m. Head-reading errors can be significantly reduced if outside stilling wells are connected to 0.01 m piezometers placed in the exact positions shown in Figure 8.6. Two staff gauges may be installed in the stilling wells, but more accurate readings will be obtained by using a differential head meter as described in Section 2.12. Head-reading errors on existing structures equipped with outside staff gauges can be reduced by the use of a small wooden or metal baffle-type stilling basin and an anti-vortex baffle. The dimensions and position of these stilling devices, which have been developed by the U.S. Agricultural Research Service, are shown in Figure 8.7.

Because of the above described error in discharge measurement, the construction of a new CHO is not recommended.

8.3.2 Determination of discharge

The basic head-discharge equation for a submerged orifice, according to Section 1.13 reads

$$Q = CA\sqrt{2g\Delta h} \tag{8-12}$$

where, for the CHO, the differential head Δh usually equals 0.06 m. The discharge coefficient C is a function of the upstream water depth, y_1, and the height of the orifice

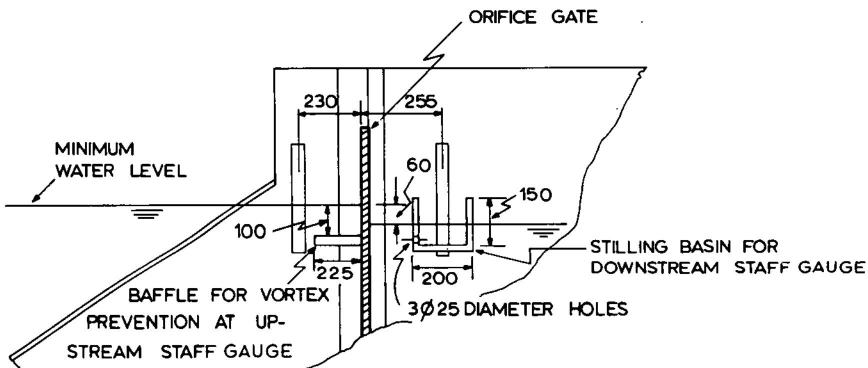


Figure 8.7 Device to reduce water level fluctuations at CHO staff gauges (after U.S. Agricultural Research Service, SCS 1962)

w. Experimental values of C as a function of the ratio y_1/w are shown in Figure 8.8. The reader should note that the coefficient C also expresses the influence of the approach velocity head on the flow.

From Figure 8.8 it appears that the discharge coefficient, C, is approximately 0.66 for normal operating conditions, i.e. where the water depth upstream from the orifice gate is 2.5 or more times the maximum height of the gate opening, w. Substitution of the values $C_d = 0.66$, $\Delta h = 0.06$ m, and $g = 9.81$ m/s^2 into Equation 8.1 gives the following simple area-discharge relationship for the CHO:

$$Q = 0.716\,A = 0.716\,b_c w \qquad (8\text{-}13)$$

If the breadth of the orifice is known, a straight-line relationship between the orifice gate opening and the flow may be plotted for field use.

The error in the discharge coefficient given for the Constant-Head-Orifice (C = 0.66) can be expected to be of the order of 7%. This coefficient error applies for structures that have an even velocity distribution in the approach section. If an eddy is formed upstream of the orifice gate, however, an additional error of up to 12% may occur (see also Section 8.3.1).

The method by which the coefficient error is to be combined with other sources of error, which have a considerable effect on the accuracy with which flow can be measured, is shown in Annex 2. In this context, the reader should note that if the upstream gate is constructed with uninterrupted bottom and side walls and a sharp-edged gate, Equations 8-3 and 8-4 can be used to determine the discharge through the orifice with an error of about 3%.

8.3.3 Limits of application

The limits of application of the Constant-Head-Orifice turnout are:

a. The upstream edge of the orifice gate should be sharp and smooth and be in accordance with the profile shown in Figure 5.1;
b. The sides of the orifice should have a groove arrangement as shown in Figure 8.6;

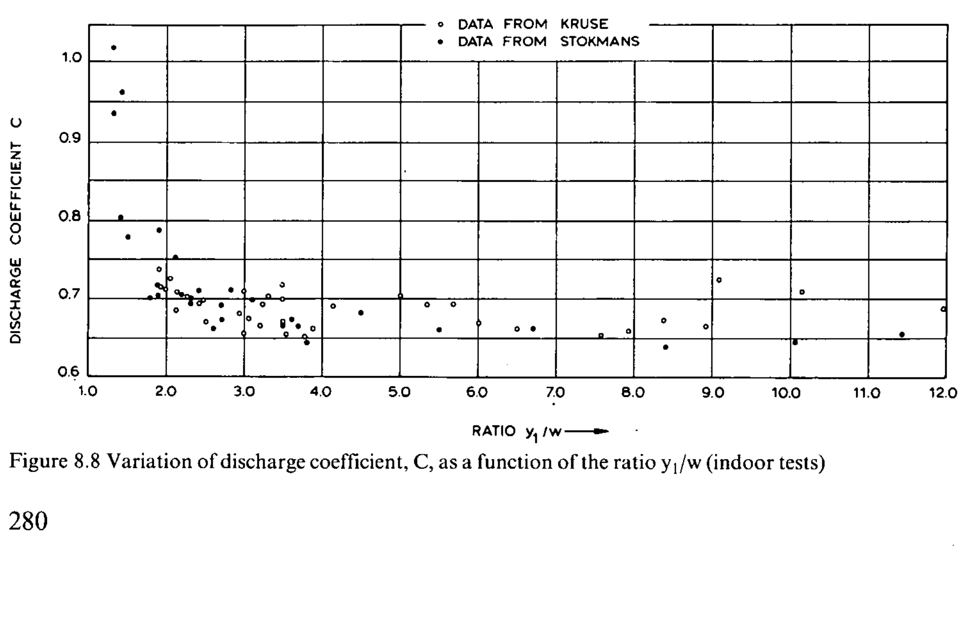


Figure 8.8 Variation of discharge coefficient, C, as a function of the ratio y_1/w (indoor tests)

c. The bottom of the approach section upstream of the orifice gate should be horizontal over a distance of at least four times the upstream water depth.
d. To obtain a somewhat constant value for the discharge coefficient, C, the ratio y_1/w should be greater than 2.5;
e. The approach section should be such that no velocity concentrations are visible upstream of the orifice gate.

8.4 Radial or tainter gate

8.4.1 Description

The radial or tainter gate is a movable control; it is commonly used in a rectangular canal section. It has the structural advantage of not requiring a complicated groove arrangement to transmit the hydraulic thrust to the side walls, because this thrust is concentrated at the hinges. In fact, the radial gate does not require grooves at all, but has rubber seals in direct contact with the undisturbed sides of the rectangular canal section.

Figure 8.9 shows two methods by which the radial gate can be installed, either with the gate sill at stream bed elevation or with its sill raised.

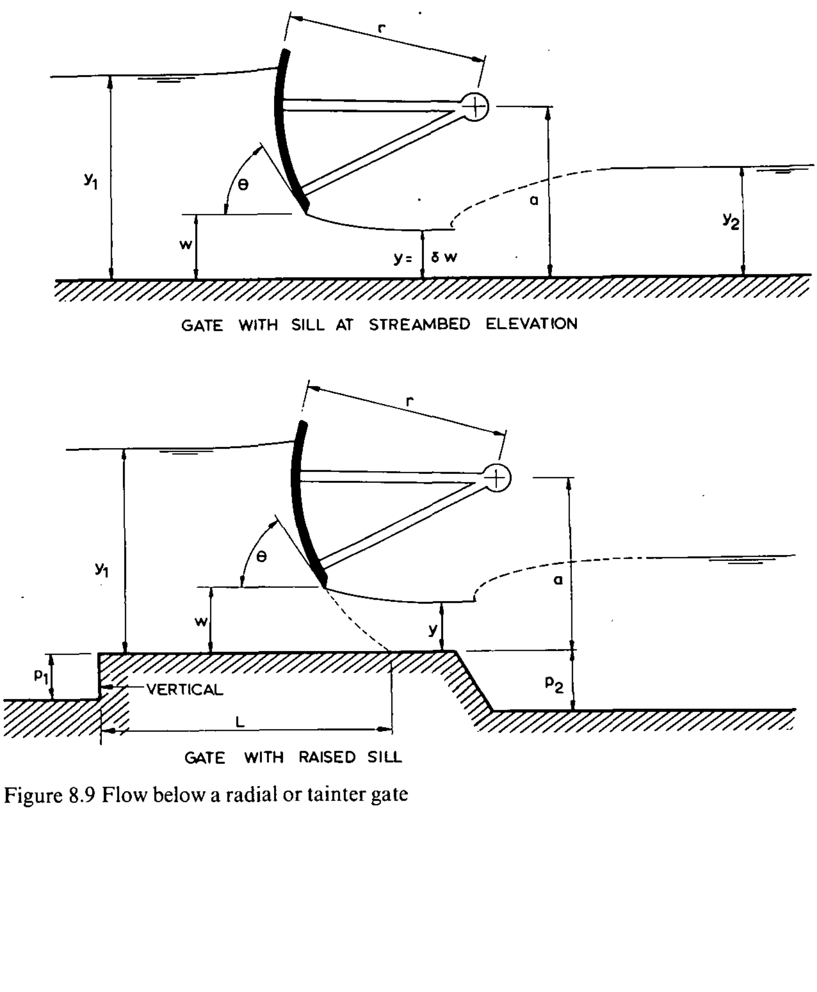


Figure 8.9 Flow below a radial or tainter gate

8.4.2 Evaluation of discharge

Free flow through a partially open radial gate is commonly computed with the following equation:

$$Q = C_o C_1 \, w b_c \sqrt{2 g y_1} \qquad (8\text{-}15)$$

The coefficient, C_o, depends on the contraction of the jet below the gate and may be expressed as a function of the gate opening w, gate radius r, trunnion height a, and upstream water depth y_1, for a gate sill at streambed elevation. Figure 8.10 gives C_o-values for a/r ratios of 0.1, 0.5, and 0.9. Coefficient values for other a/r-values may be obtained by linear interpolation between the values presented.

The coefficient C_1 is a correction to C_o for gate sills above streambed elevation and depends upon sill height p_1 and the distance between the step and the gate seat L, as shown in Figure 8.11. Insufficient information is available to determine the effects, if any, of the parameter p_1/r.

It should be noted that the velocity $\sqrt{2gy_1}$ in Equation 8-15 does not occur anywhere in the flow system, but simply serves as a convenient reference velocity.

The experiments on which Figure 8.10 is based showed that the contraction coefficient, δ, of the jet below the gate is mainly determined by the angle θ and to a much lesser extent by the ratio y_1/w. For preliminary design purposes, Henderson (1966) proposed Equation 8-16 to evaluate δ-values.

$$\delta = 1 - 0.75\,(\theta/90°) + 0.36\,(\theta/90°)^2 \qquad (8\text{-}16)$$

where θ equals the angle of inclination in degrees.

Equation 8-16 was obtained by fitting a parabola as closely as possible to Toch's results (1952, 1955) and data obtained by Von Mises (1917) for non-gravity, two-dimensional flow through an orifice with inclined side walls. Values of δ given by Equation 8-16 and shown in Figure 8.12 can be expected to have an error of less than 5%, provided that $\theta < 90°$.

If the discharge coefficient C_o in Equation 8-15 is to be evaluated from the contraction coefficient, we may write, according to continuity and Bernoulli:

$$C_o = \frac{\delta}{\sqrt{1 + \delta w / y_1}} \qquad (8\text{-}17)$$

The discharge coefficient, C_o, given in Figure 8.10 and Equation 8-17 for free flow below a radial gate can be expected to have errors of less than 5% and between 5 and 10% respectively. The error in the correction coefficient C_1, given in Figure 8.11 can be expected to have an error of less than 5%. The method by which these errors have to be combined with other sources of error is shown in Annex 2.

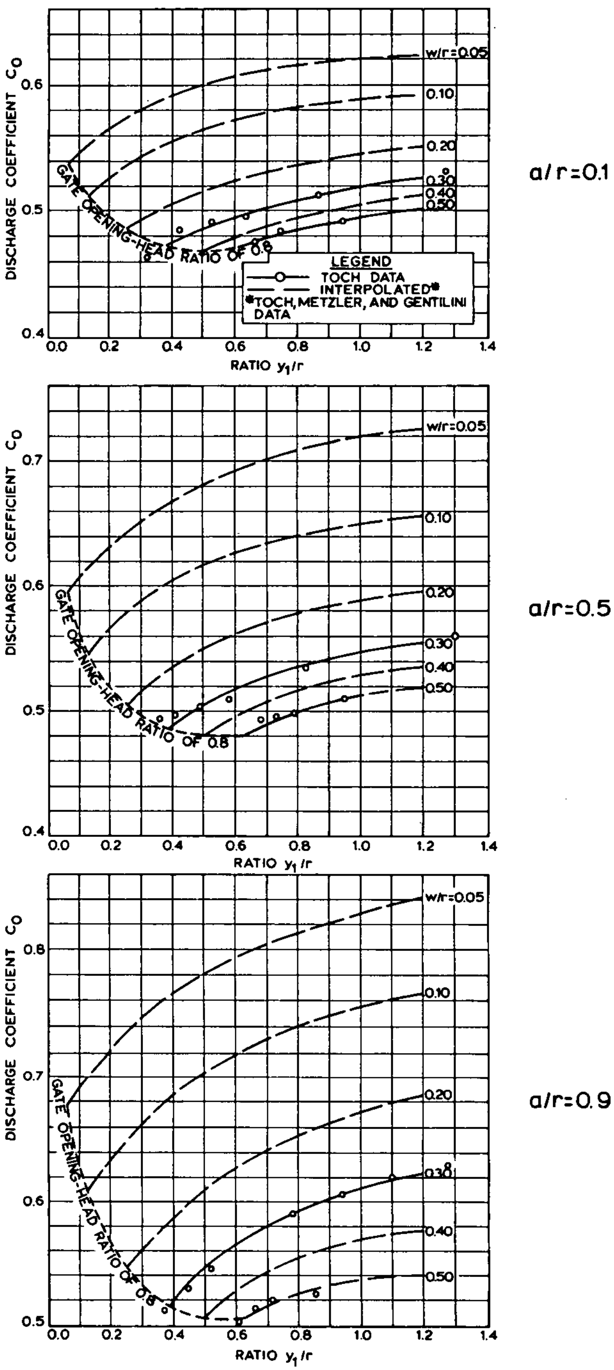


Figure 8.10 C_0-values as a function of a/r, y_1/r and w/r (from U.S. Army Engineer Waterways Experiment Station 1960)

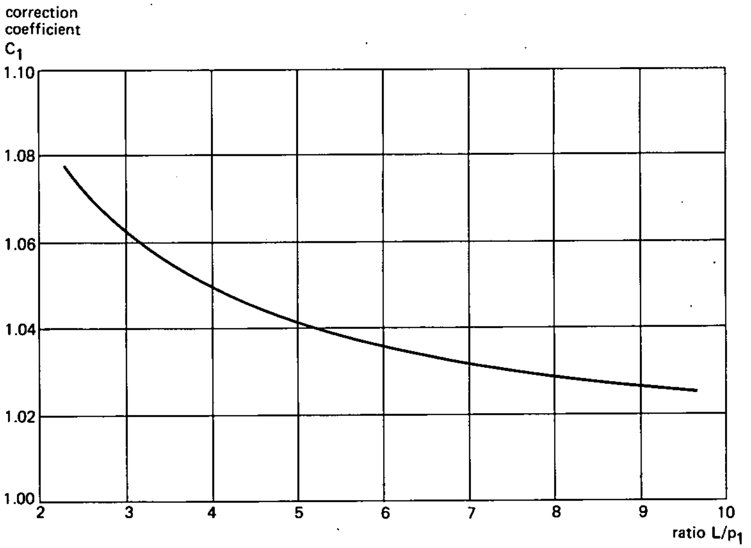


Figure 8.11 C_1-values for radial gates with raised sill (from U.S. Army Engineer Waterways Experiment Station)

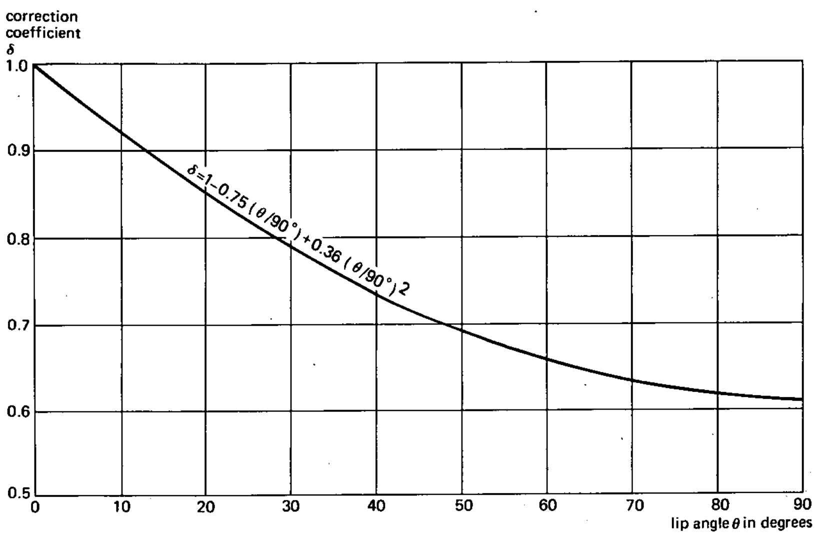


Figure 8.12 Effect of lip angle on contraction coefficient

8.4.3 Modular limit

Modular flow below a radial gate occurs as long as the roller of the hydraulic jump does not submerge the section of minimum depth of the jet (vena contracta). To prevent such submergence, the water depth, y_2, downstream of the hydraulic jump should not exceed the alternate depth to $y = \delta w$ or according to the equation

$$\frac{y_2}{w} < \frac{\delta}{2}\left[\sqrt{1 + 16\left(\frac{H}{\delta w} - 1\right)} - 1\right] \tag{8-18}$$

For each radial gate the modular limit may be obtained by combining Equation 8-16 (or Figure 8.12) and Equation 8-18.

If flow below the gate is submerged, Equation 1-73 as derived in Section 1.12 may be used as a head-discharge relationship. It reads

$$Q = C_e\, b_c w \sqrt{2g(y_1 - y_2)} \tag{8-19}$$

Photo 1 Radial gates are suitable flow control structures

Insufficient experimental data are available to present reasonably accurate C_e-values. For design purposes, however, the coefficient C_e may be evaluated from the contraction coefficient δ for free flow conditions (Figure 8.12).

A combination of the Bernoulli and the continuity equations gives for C_e

$$C_e = \frac{\delta}{\sqrt{1-\left(\frac{\delta w}{y_1}\right)^2}} \tag{8-20}$$

It should be noted that the assumption that the contraction coefficient is the same for free flow as for submerged flow is not completely correct.

8.4.4 Limits of application

The limits of application of the radial or tainter gate are:

a. The bottom edge of the gate should be sharp and horizontal from end to end;
b. The upstream head should be measured in a rectangular approach channel that has the same width as the gate;
c. The gate opening over water depth ratio should not exceed 0.8 ($w/y_1 \leqslant 0.8$);
d. The downstream water level should be such that modular flow occurs (see Equation 8-18).

8.5 Crump-De Gruyter adjustable orifice

8.5.1 Description

The Crump-De Gruyter adjustable orifice is a short-throated flume fitted with a vertically movable streamlined gate. It is a modification of the 'adjustable proportional module', introduced by Crump in 1922. De Gruyter (1926) modified the flume alignment and replaced the fixed 'roof-block' with an adjustable sliding gate and so obtained an adjustable flume that can be used for both the measurement and regulation of irrigation water (see Figure 8.13).

Usually the orifice is placed at an angle of 90° from the centre line of the main canal which may cause eddies upstream of the orifice gate if canal velocities are high. For normal flow velocities in earthen canals, the approach section shown in Figure 8.13 is adequate. If canal velocities are high, of the order of those that may occur in lined canals, the approach section should have a greater length so that no velocity concentrations are visible upstream of the orifice gate. The structural dimensions in Figure 8.13 are shown as a function of the throat width b_c and head h_1.

Provided that the gate opening (w) is less than about $^2/_3\, H_1$ – in practice one takes $w \leqslant 0.63\, h_1$ – and the downstream water level is sufficiently low, supercritical flow will occur in the throat of the structure so that the discharge depends on the upstream water level (h_1) and the gate opening (w) only.

With the use of Equation 1-33, the discharge through the non-submerged (modular) structure can be expressed by

$$Q = C_d C_v b_c w \sqrt{2g(h_1 - w)} \tag{8-21}$$

where b_c equals the breadth of the flume throat and w is the gate opening which equals the 'water depth' at the control section of the flume. To obtain modular flow, a minimal loss of head over the structure is required. This fall, Δh, is a function of both h_1 and w, and may be read from Figure 8.14, provided that the downstream transition is in accordance with Figure 8.13.

From Figure 8.14 we may read that for a gate opening $w = 0.2\ h_1$ the minimal fall required for modular flow is $0.41\ h_1$, and that if $w = 0.4\ h_1$ the minimal fall equals $0.23\ h_1$. This shows that, if h_1 remains about constant, the adjustable orifice requires a maximum loss of head to remain modular when the discharge is minimal. Therefore, the required value of the ratio $\gamma = Q_{max}/Q_{min}$ is an important design criterion for the

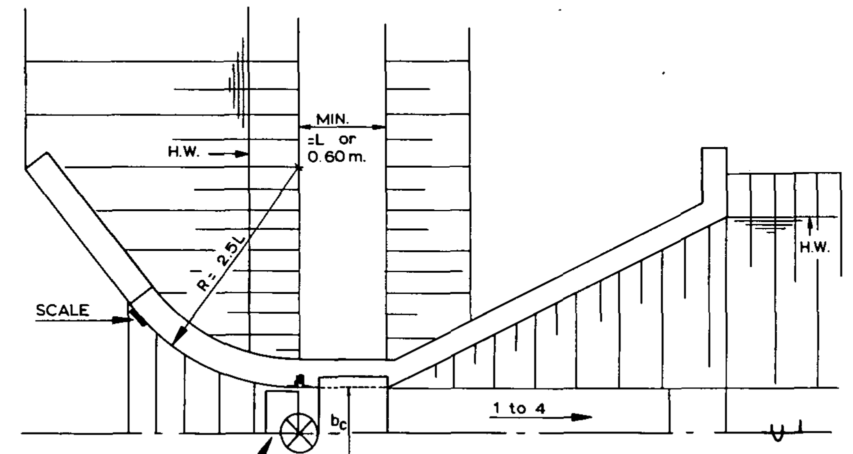
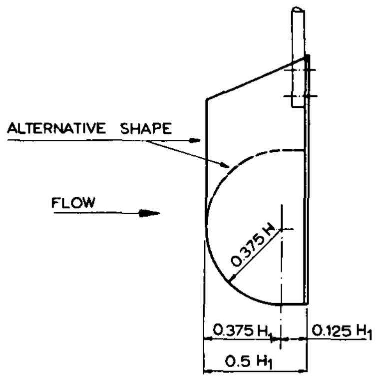
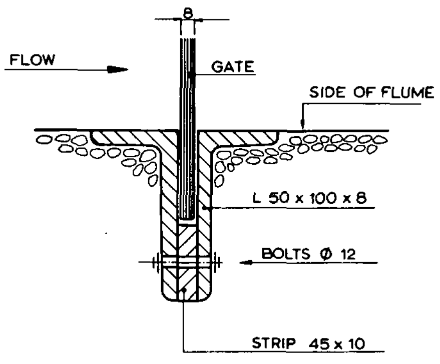


Figure 8.13 The Crump-De Gruyter adjustabel orifice dimensions as a function of h_1 and b_c



SECTION OVER ADJUSTABLE GATE



DETAIL OF GROOVE ARRANGEMENT.

Figure 8.13 cont.

elevation of the flume crest. If, for example, both γ and h_1 are known, the minimum loss of head, Δh, required to pass the range of discharges can be calculated from Figure 8.14. On the other hand, if both γ and Δh are known, the minimum h_1-value, and thus the flume elevation with regard to the upstream (design) water level, is known.

When a design value for h_1 has been selected, the minimum throat width, b_c, required to pass the required range of discharges under modular conditions can be calculated from the head-discharge equation and the limitation on the gate opening, which is $w \leqslant 0.63\, h_1$. Anticipating Section 8.5.2 we can write

$$Q_{max} \leqslant 0.94\, b_c(0.63\, h_1)\sqrt{2g(h_1 - 0.63\, h_1)} \qquad (8\text{-}22)$$

which results in a minimum value of b_c, being

$$b_c \geqslant \frac{Q_{max}}{1.60\, h_1^{3/2}} \qquad (8\text{-}23)$$

With the use of Figures 8.13 and 8.14 and Equation 8-23, all hydraulic dimensions of the adjustable orifice can be determined.