

Figure 6.8 Two-dimensional 1-to-2/1-to-5 weir, submerged flow product  $C_v f$  (after White 1971)

- c. The height of the weir crest should not be less than 0.06 m above the approach channel bottom ( $p_1 \geq 0.06$  m);
- d. To reduce the influence of boundary layer effects at the sides of the weir, the breadth of the weir  $b_c$  should not be less than 0.30 m and the ratio  $b_c/H_1$  should not be less than 2.0;
- e. To obtain a sensibly constant discharge coefficient for 1-to-2/1-to-2 profile weirs, the ratio  $H_1/p_2$  should not exceed 1.25. For 1-to-2/1-to-5 profile weirs, this ratio should be less than 3.0.

## 6.4 Triangular profile flat-V weir

### 6.4.1 Description

In natural streams where it is necessary to measure a wide range of discharges, a triangular control has the advantage of providing a wide opening at high flows so that it causes no excessive backwater effects, whereas at low flows its opening is reduced so that the sensitivity of the structure remains acceptable. The Hydraulics Research Station, Wallingford investigated the characteristics of a triangular profile flat-V weir with cross-slopes of 1-to-10 and 1-to-20. (For the two-dimensional triangular profile weir, see Section 6.3.) The profile in the direction of flow shows an upstream slope of 1-to-2 and a downstream slope of either 1-to-5 or 1-to-2 (Figure 6.9). The intersec-

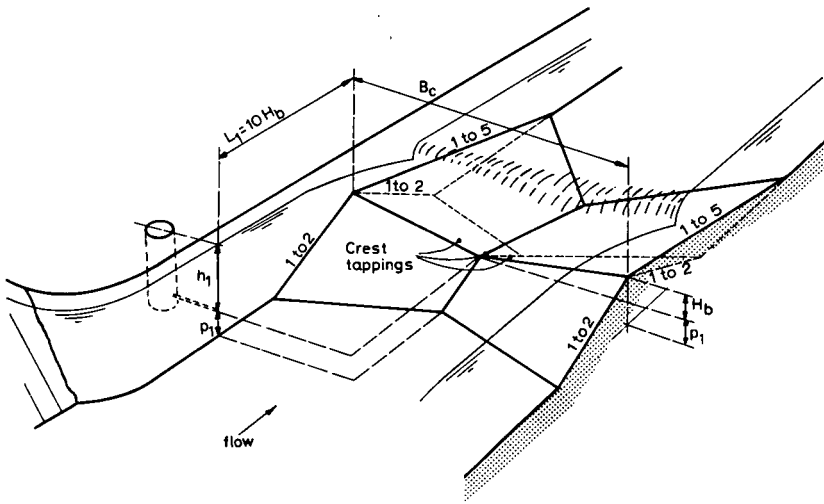


Figure 6.9 Triangular profile flat-V weir

tions of the upstream and downstream surfaces form a crest at right angles to the flow direction in the approach channel. Care should be taken that the crest has a well-defined corner made either of carefully aligned and joined precast concrete sections or of a cast-in non-corrodible metal profile.

The permissible truncation of the weir block is believed to be the same as that of the two-dimensional weir (see Section 6.3.1). Therefore the minimum horizontal distance from the weir crest to the point of truncation whereby the  $C_d$ -value is within 0.5% of its constant value, equals  $1.0 H_{1\max}$  for the upstream and  $2.0 H_{1\max}$  for the downstream slope of a 1-to-2/1-to-5 weir. For a 1-to-2/1-to-2 weir these minimum distances equal  $0.8 H_{1\max}$  for the upstream slope and  $1.2 H_{1\max}$  for the downstream slope.

The upstream head over the weir crest  $h_1$  should be measured in a rectangular approach channel at a distance of ten times the V-height upstream of the crest, i.e.  $L_1 = 10 H_b$ . At this location, differential drawdown across the width of the approach channel is negligible and a true upstream head can be measured accurately.

If a 1-to-2/1-to-5 weir is to be used for discharge measuring beyond its modular range, three crest tappings should be provided to measure the piezometric level in the separation pocket,  $h_p$ , immediately downstream (0.019 m) of the crest (see also Figure 6.5). One crest tapping should be at the centre line, the other two at a distance of  $0.1 B_c$  offset from the centre line.

#### 6.4.2 Evaluation of discharge

According to Section 1.10, the basic head-discharge equation for a short-crested flat-V weir with vertical side walls reads

$$Q = C_d C_v \frac{4}{15} \sqrt{2g} \frac{B_c}{H_b} [h_e^{2.5} - (h_e - H_b)^{2.5}] \quad (6-6)$$

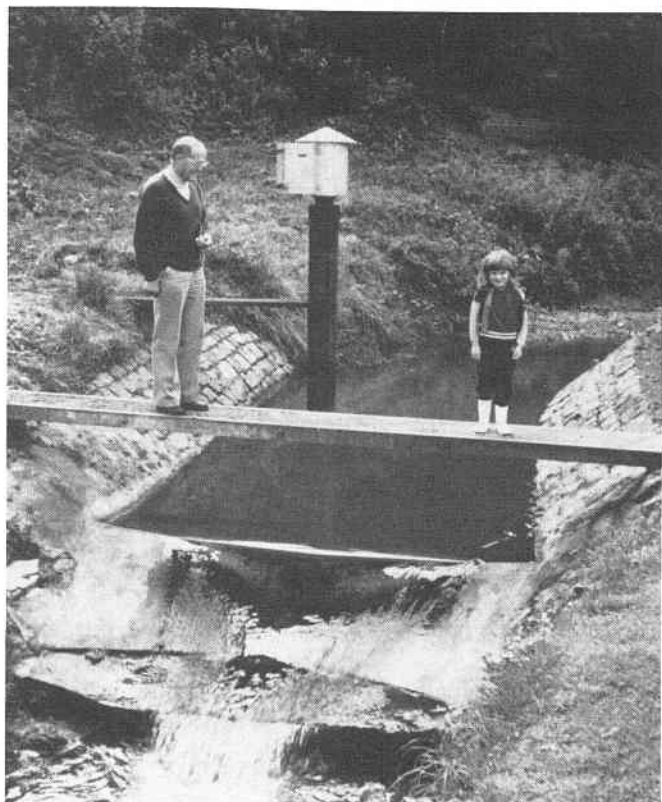


Photo 6.1 An 1-to-2/1-to-5 shaped weir on a natural stream

in which the term  $(h_e - H_b)^{2.5}$  should be deleted if  $h_e$  is less than  $H_b$ . The effective head over the weir crest  $h_e = h_1 - K_h$ ,  $K_h$  being an empirical quantity representing the combined effects of several phenomena attributed to viscosity and surface tension. Values for  $K_h$  are presented in Table 6.3.

Table 6.3  $K_h$ -values for triangular profile flat-V weirs (White 1971)

weir profile	1-to-20 cross slope	1-to-10 cross slope
1-to-2/1-to-2	0.0004 m	0.0006 m
1-to-2/1-to-5	0.0005 m	0.0008 m

For the 1-to-2/1-to-5 weir, an average  $C_d$ -value of 0.66 may be used for both cross slopes provided that the ratio  $h_e/p_2 < 3.0$ . The  $C_d$ -value of a 1-to-2/1-to-2 weir is more sensitive to the bottom level of the tailwater channel with regard to crest level. An average value of  $C_d = 0.71$  may be used provided that  $h_e/p_2$  does not exceed 1.25.

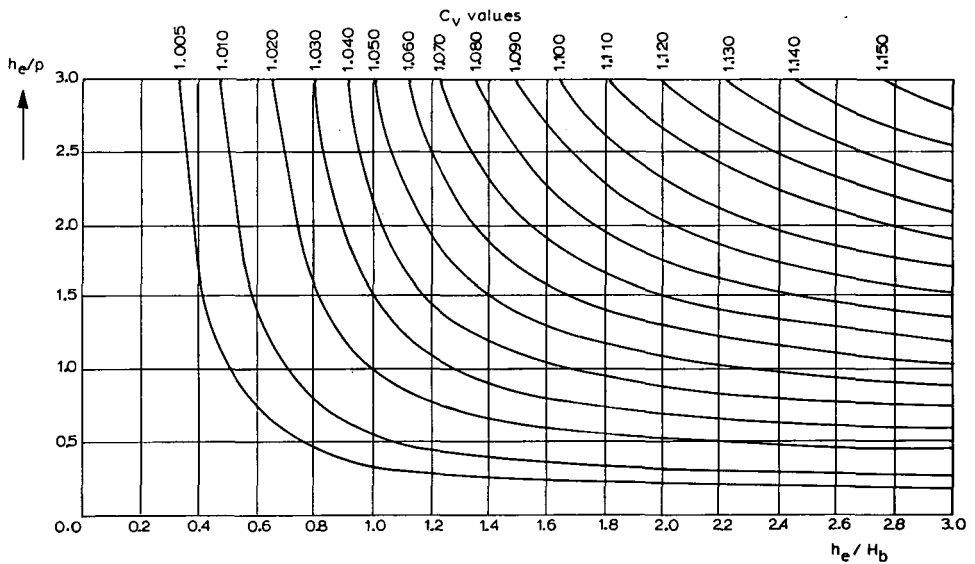


Figure 6.10  $C_v$ -values as a function of  $h_e/p_1$  and  $h_e/H_b$  (after White 1971)

The approach velocity coefficient  $C_v$  can be read as a function of the ratios  $h_e/p_1$  and  $h_e/H_b$  in Figure 6.10.

The error in the product  $C_d C_v$  of a well-maintained triangular profile weir with modular flow, constructed with reasonable care and skill may be expected to be

$$X_c = \pm (10 C_v - 8) \text{ per cent} \quad (6-7)$$

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

#### 6.4.3 Modular limit and non-modular discharge

The modular limit again is defined as that submergence ratio  $H_2/H_1$  which produces a 1% reduction from the equivalent modular discharge as calculated by Equation 6-6. Results of various tests have shown that for a 1-to-2/1-to-2 weir the drowned flow reduction factor,  $f$ , and thus the modular limit, are functions of the dimensionless ratios  $H_2/H_1$ ,  $H_1/H_b$ ,  $H_1/p_1$ ,  $H_1/p_2$ , and the cross slope of the weir crest.

Because of these variables, the modular limit characteristics of a 1-to-2/1-to-2 weir are rather complex and sufficient data are not available to predict the influence of the variables. A limited series of tests in which only discharge, cross-slope, and downstream bed level ( $p_2$ ) were varied was undertaken at Wallingford. The results of these tests, which are shown in Figure 6.11, are presented mainly to illustrate the difficulties.

For a 1-to-2/1-to-5 profile weir, the drowned flow reduction factor is a less complex phenomenon, and it appears that the  $f$ -value is a function of the ratios  $H_2/H_1$  and  $H_1/H_b$  only (Figure 6.12). Tests showed that there is no significant difference between the modular flow characteristics of the weirs with either 1-to-10 or 1-to-20 cross slopes. As illustrated in Figure 6.12, the drowned flow reduction factor  $f$  equals 0.99 for

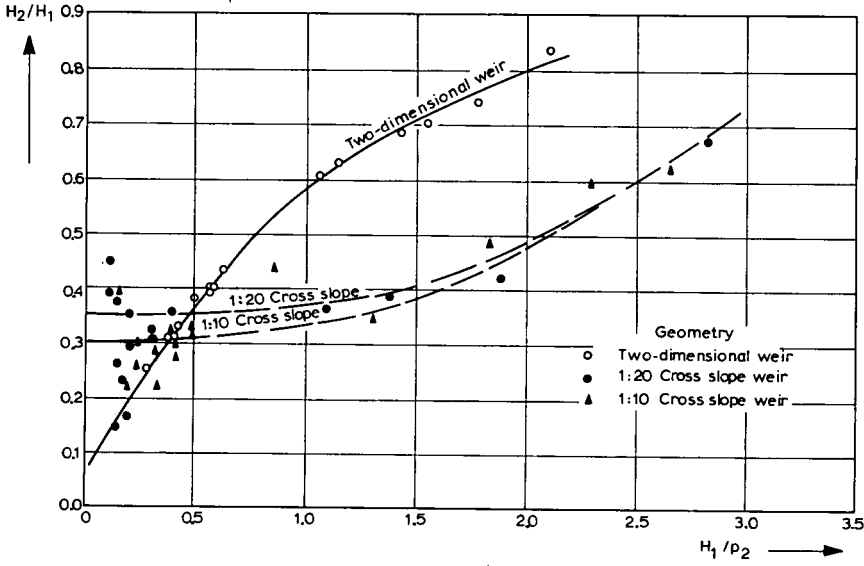


Figure 6.11 Modular limit conditions, triangular profile 1-to-2/1-to-2 flat-V (after White 1971)

modular limit values between 0.67 and 0.78, depending on the modular value of  $H_1/H_b$ . For non-modular flow conditions, the discharge over the weir is reduced because of high tailwater levels, and the weir discharge can be calculated from Equation 6-8, which reads

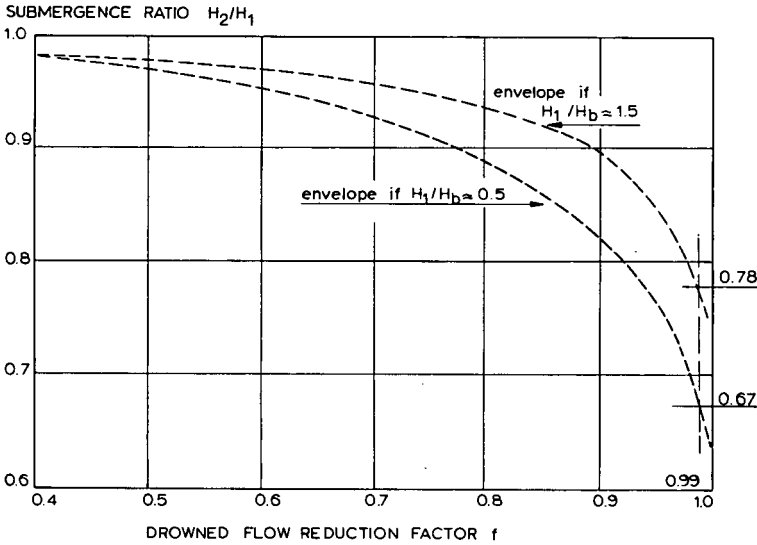


Figure 6.12 Modular limit conditions, 1-to-2/1-to-5 flat-V weir (adapted from White 1971)

$$Q = C_d C_v f \frac{4}{15} (2g)^{0.5} \frac{B_c}{H_b} [h_e^{2.5} - (h_e - H_b)^{2.5}] \quad (6-8)$$

This equation is similar to Equation 6-6 except that a drowned flow reduction factor  $f$  has been introduced. For 1-to-2/1-to-5 profile weirs,  $f$ -values have been determined and, in order to eliminate an intermediate step in the computation of discharge, they have also been combined with the approach velocity coefficient as a product  $C_v f$ . This product is a function of  $h_e/H_b$ ,  $h_p/h_e$ , and  $H_b/p_1$  and as such is presented in Figure 6.13. To find the proper  $C_v f$ -value, one enters the figure by values of  $h_e/H_b$  and  $h_p/h_e$  and by use of interpolation in terms of  $H_b/p_1$  a value of the product  $C_v f$  is obtained. Substitution of all values into Equation 6-8 gives the non-modular discharge.

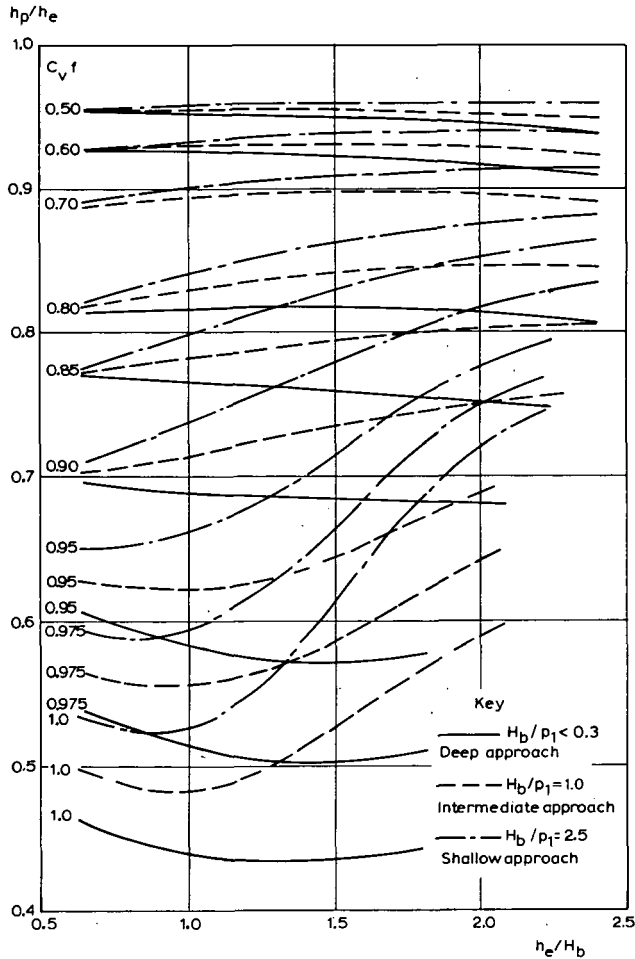


Figure 6.13 Values of  $C_v f$  for a 1-to-2/1-to-5 flat-V weir as a function of  $h_e/H_b$ ,  $h_p/h_e$  and  $H_b/p_1$  (after White 1971)

#### 6.4.4 Limits of application

For reasonable accuracy, the limits of application of a triangular profile flat-V weir are:

- a. For a well-maintained weir with a non-corrodible metal insert at its crest, the recommended lower limit of  $h_1 = 0.03$  m. For a crest made of pre-cast concrete sections or similar materials,  $h_1$  should not be less than 0.06 m;
- b. To prevent water surface instability in the approach channel in the form of stationary waves, the ratio  $h_1/p_1$  should not exceed 3.0;
- c. The height of the vertex of the weir crest should not be less than 0.06 m above the approach channel bottom;
- d. To reduce the influence of boundary layer effects at the sides of the weir, the width of the weir  $B_c$  should not be less than 0.30 m and the ratio  $B_c/H_1$  should not be less than 2.0;
- e. To obtain a sensibly constant discharge coefficient for 1-to-2/1-to-2 profile weirs, the ratio  $H_1/p_2$  should not exceed 1.25. For 1-to-2/1-to-5 profile weirs, this ratio should be less than 3.0;
- f. The upstream head over the weir crest should be measured a distance of  $10 H_b$  upstream from the weir crest in a rectangular approach channel;
- g. To obtain modular weir flow, the submergence ratio  $H_2/H_1$  should not exceed 0.30 for 1-to-2/1-to-2 profile weirs and should be less than 0.67 for 1-to-2/1-to-5 profile weirs. For the latter weir profile, however, non-modular flows may be calculated by using Equation 6-8 and Figure 6.13.

### 6.5 Butcher's movable standing wave weir

#### 6.5.1 Description

Butcher's weir was developed to meet the particular irrigation requirements in the Sudan, where the water supplied to the fields varies because of different requirements during the growing season and because of crop rotation. A description of the weir was published for the first time in 1922 by Butcher, after whom the structure has been named.\* The weir consists of a round-crested movable gate with guiding grooves and a self-sustaining hand gear for raising and lowering it. The cylindrical crest is horizontal perpendicular to the flow direction. The profile in the direction of flow shows a vertical upstream face connected to a 1-to-5 downward sloping face by a  $0.25 h_{1\max}$  radius circle, where  $h_{1\max}$  is the upper limit of the range of heads to be expected at the gauge located at a distance  $0.75 h_{1\max}$  upstream from the weir face.

The side walls are vertical and are rounded at the upstream end in such a way that flow separation does not occur. Thus a rectangular approach channel is formed to assure two-dimensional weir flow. The upstream water depth over the weir crest  $h_1$  is measured in this approach channel by a movable gauge mounted on two supports. The lower support is connected to the movable gate and the upper support is bolted to the hoisting beam. The gauge must be adjusted so that its zero corresponds exactly

\* Nowadays the structure is manufactured commercially by Boving Newton Chambers Ltd., Rotherham, SG0 1TF, U.K.





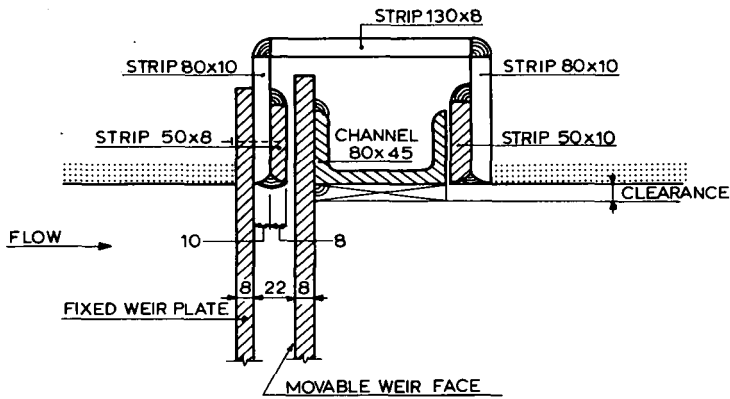
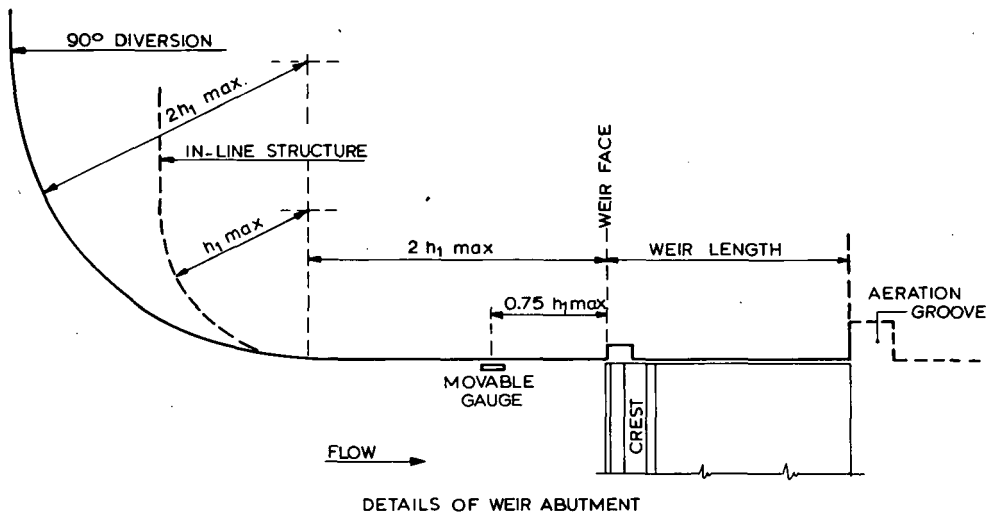


Figure 6.14 (cont.)

crete or masonry sill whose top width is about 0.10 m and whose upstream face is not flatter than 2-to-1.

The maximum water depth over the weir crest, and thus the maximum permissible discharge per metre weir crest, influences the weir dimensions. Used in the Sudan are two standard types with maximum values of  $h_1 = 0.50$  m and  $h_1 = 0.80$  m respectively. It is recommended that 1.00 m be the upper limit for  $h_1$ . The breadth of the weir varies from 0.30 m to as much as 4.00 m, the larger breadths used in conjunction with high  $h_{1max}$ -values. As shown in Figure 6.14,  $p_1 = 1.4 h_{1max}$ , which results in low approach velocities.

The modular limit is defined as the submergence ratio  $h_2/h_1$  which produces a 1% reduction from the equivalent modular discharge. Results of various tests showed that the modular limit is  $h_2/h_1 = 0.70$ . The average rate of reduction from the equivalent modular discharge is shown in percentages in Figure 6.15.

### 6.5.2 Evaluation of discharge

Since the water depth over the weir crest is measured in the area of water surface drawdown at a distance of  $0.75 h_{1max}$  upstream from the weir face, i.e.  $h_{1max}$  upstream from the weir crest, the stage-discharge relationship of the weir has the following empirical shape

$$Q = c b_c h_1^{1.6} \quad (6-9)$$

where  $h_1$  equals the water depth at a well-prescribed distance  $L_1 = 0.75 h_{1max}$  upstream from the weir face. It should be noted that this water depth is somewhat lower than the real head over the weir crest. For weirs that are constructed in accordance with the dimensions shown in Figure 6.14, the effective discharge coefficient equals  $c = 2.30 \text{ m}^{0.4} \text{ s}^{-1}$ . The influence of the approach velocity on the weir flow is included in this coefficient value and in the exponent value 1.6.

The error in the discharge coefficient  $c$  of a well-maintained Butcher movable weir which has been constructed with reasonable care and skill may be expected to be less than 3%. The method by which this error is to be combined with other sources of error is shown in Annex 2.

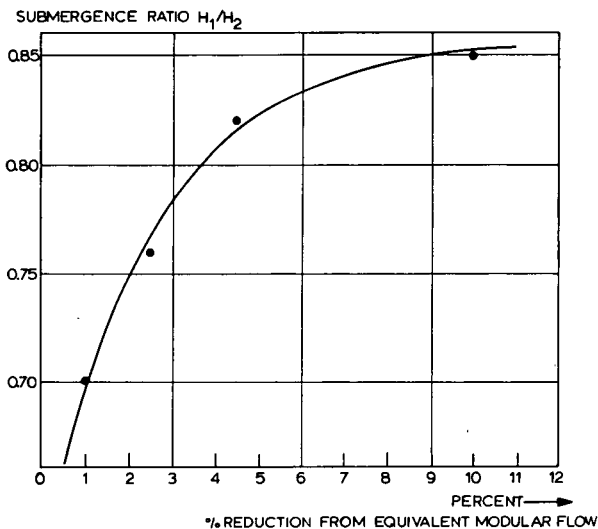


Figure 6.15 Modular flow condition

### 6.5.3 Limits of application

For reasonable accuracy, the limits of application of Equation 6-9 for Butcher's movable weir are:

- All dimensions of both the weir and the approach channel should be strictly in accordance with the dimensions shown in Figure 6.14;
- The width of the weir  $b_c$  should not be less than 0.30 m and the ratio  $b_c/h_1$  should not be less than 2.0;
- The upstream water depth should be measured with a movable gauge at a distance of  $0.75 h_{1max}$  upstream from the weir face;
- To obtain modular flow, the submergence ratio  $h_2/h_1$  should not exceed 0.70;
- The recommended lower limit of  $h_1 = 0.05$  m, while  $h_1$  should preferably not exceed 1.00 m.

## 6.6 WES-Standard spillway

### 6.6.1 Description

From an economic point of view, spillways must safely discharge a peak flow under the smallest possible head, while on the other hand the negative pressures on the crest must be limited to avoid the danger of cavitation. Engineers therefore usually select a spillway crest shape that approximates the lower nappe surface of an aerated sharp crested weir as shown in Figure 6.16.

Theoretically, there should be atmospheric pressure on the crest. In practice, however, friction between the surface of the spillway and the nappe will introduce some negative pressures. If the spillway is operating under a head lower than its design head, the nappe will normally have a lower trajectory so that positive pressures occur throughout the crest region and the discharge coefficient is reduced. A greater head will cause negative pressures at all points of the crest profile and will increase the discharge coefficient.

The magnitude of the local minimum pressure at the crest  $(P/\rho g)_{min}$  has been measured by various investigators. Figure 6.17 shows this minimum pressure as a function of the ratio of actual head over design head as given by Rouse & Reid (1935) and Dillman (1933).

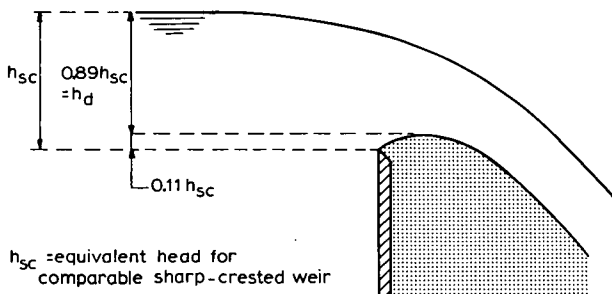


Figure 6.16 Spillway crest and equivalent sharp-crested weir

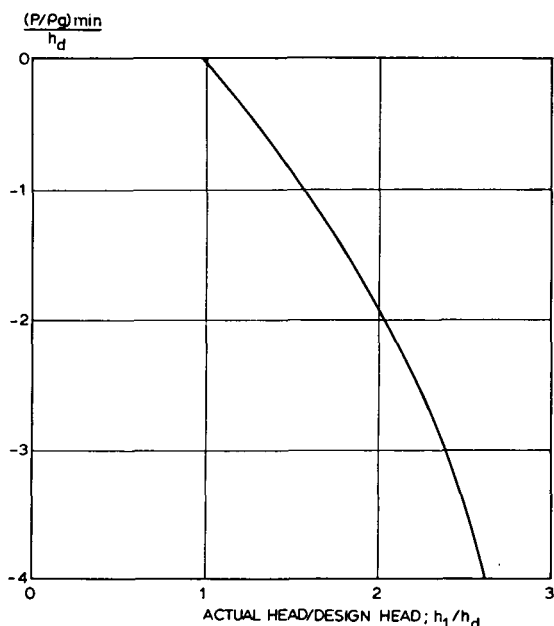


Figure 6.17 Negative pressure on spillway crest (after Rouse & Reid 1935 and Dillman 1933)

The avoidance of severe negative pressures on the crest, which may cause cavitation on the crest or vibration of the structure, should be considered an important design criterion on high-head spillways. In this context it is recommended that the minimum pressure on the weir crest be  $-4$  m water column. This recommendation, used in combination with Figure 6.17, gives an upper limit for the actual head over the crest of a spillway.

On the basis of experiments by the U.S. Bureau of Reclamation the U.S. Army Corps of Engineers conducted additional tests at their Waterways Experimental Station and produced curves which can be described by the following equation

$$X^n = K h_d^{n-1} Y \quad (6-10)$$

which equation may also be written as

$$\frac{Y}{h_d} = \frac{1}{K} \left( \frac{X}{h_d} \right)^n \quad (6-11)$$

where  $X$  and  $Y$  are coordinates of the downstream crest slope as indicated in Figure 6.18 and  $h_d$  is the design head over the spillway crest.  $K$  and  $n$  are parameters, the values of which depend on the approach velocity and the inclination of the upstream spillway face. For low approach velocities,  $K$  and  $n$ -values for various upstream slopes are as follows:

Slope of upstream face	K	n
vertical	2,000	1,850
3 to 1	1,936	1,836
3 to 2	1,939	1,810
1 to 1	1,873	1,776

The upstream surface of the crest profile varies with the slope of the upstream spillway face as shown in Figure 6.18.

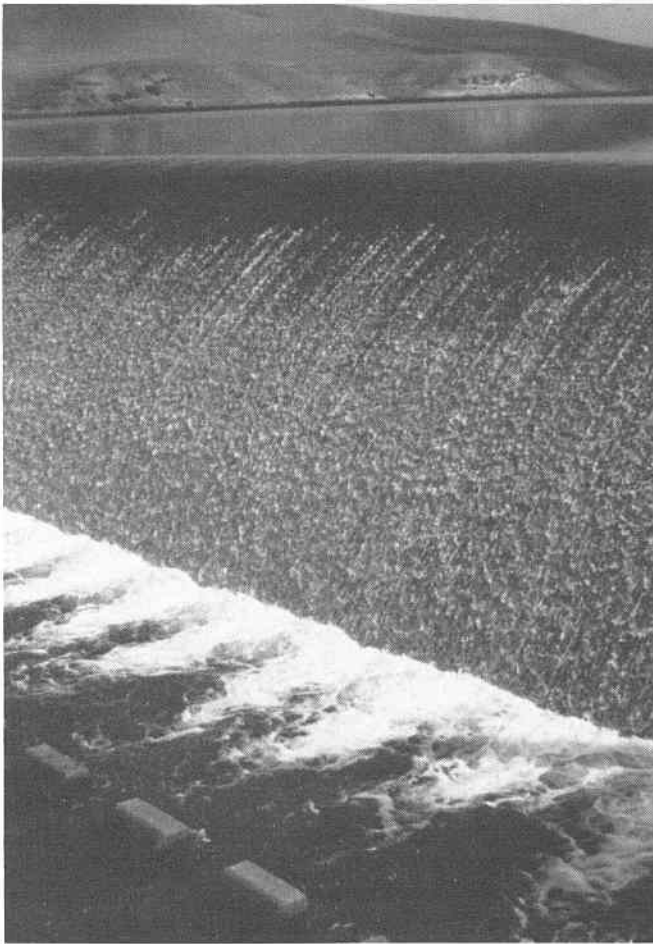


Photo 6.2 WES-spillway operating under low head

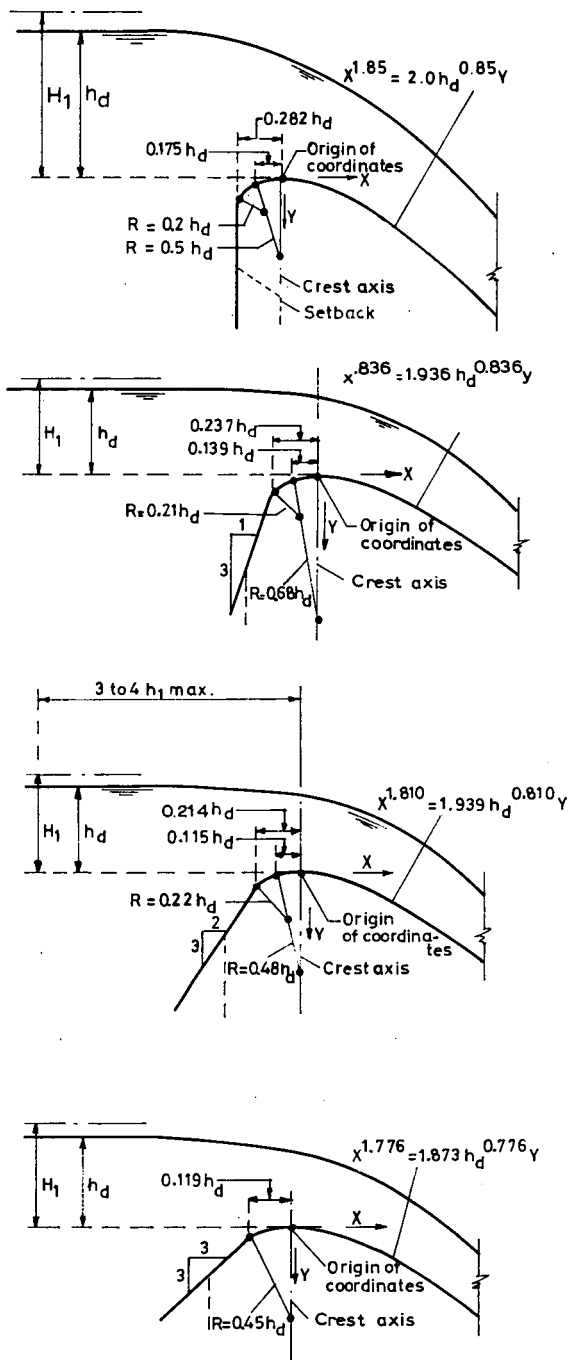


Figure 6.18 WES-standard spillway shapes (U.S. Army Corps of Engineers 1952)

## 6.6.2 Evaluation of discharge

The basic head discharge equation for a short-crested weir with a rectangular control section reads

$$Q = C_c \frac{2}{3} \sqrt{\frac{2}{3} g} b_c H_1^{1.5} \quad (6-12)$$

Since the WES-standard spillway evolved from the sharp-crested weir, we might also use an equation similar to that derived in Section 1.13.1, being

$$Q = C_c^* \frac{2}{3} \sqrt{2g} b_c H_1^{1.5} \quad (6-13)$$

A comparison of the two equations shows that  $C_c^* = C_c/\sqrt{3}$ , so that it is possible to use whichever equation suits one's purpose best.

In these two equations the effective discharge coefficient  $C_c$  (or  $C_c^*$ ) equals the product of  $C_o$  (or  $C_o^*$ ),  $C_1$  and  $C_2$  ( $C_c = C_o C_1 C_2$ ).  $C_o$  (or  $C_o^*$ ) is a constant,  $C_1$  is a function of  $p_1/h_d$  and  $H_1/h_d$ , and  $C_2$  is a function of  $p_1/h_1$  and the slope of the upstream weir face.

As illustrated in Figure 6.16 the high point of the nappe, being the spillway crest, is  $0.11 h_{sc}$  above the crest of the alternative sharp-crested weir (see also Figure 1.23). As a result, the spillway discharge coefficient at design head,  $h_d$  is about 1.2 times that of a sharp-crested weir discharging under the same head, provided that the approach channel is sufficiently deep so as not to influence the nappe profile. Model tests of spillways have shown that the effect of the approach velocity on  $C_c$  is negligible when the height,  $p_1$ , of the weir is equal to or greater than  $1.33 h_d$ , where  $h_d$  is the design head excluding the approach velocity head. Under this condition and with an actual head,  $H_1$ , over the spillway crest equal to design head  $h_d$ , the basic discharge coefficient equals  $C_c = 1.30$  in Equation 6-12 and  $C_c^* = 0.75$  in Equation 6-13.

$C_1$  can be determined from a dimensionless plot by Chow (1959), which is based on data of the U.S. Bureau of Reclamation and of the Waterways Experimental Station (1952), and is shown in Figure 6.19.

The values of  $C_1$  in Figure 6.19 are valid for WES-spillways with a vertical upstream face. If the upstream weir face is sloping, a second dimensionless correction coefficient  $C_2$  on the basic coefficient should be introduced; this is a function of both the weir face slope and the ratio  $p_1/H_1$ . Values of  $C_2$  can be obtained from Figure 6.20.

By use of the product  $C_c = C_o C_1 C_2$  an energy head-discharge relationship can now be determined provided that the weir flow is modular. After calculation of the approximate approach velocity,  $v_1$ , this  $Q$ - $H_1$  relationship can be transformed to a  $Q$ - $h_1$  curve.

To allow the WES-spillway to function as a high capacity overflow weir, the height  $p_2$  of the weir crest above the downstream channel bed should be such that this channel bed does not interfere with the formation of the overflowing jet. It is evident that when  $p_2$  approaches zero the weir will act as a broad-crested weir, which results in a reduction of the effective discharge coefficient by about 23 percent. This feature is shown in Figure 6.21. This figure also shows that in order to obtain a high  $C_c$ -value, the ratio  $p_2/H_1$  should exceed 0.75.

Figure 6.21 also shows that, provided  $p_2/H_1 \geq 0.75$ , the modular discharge as calcu-

lated by Equation 6-12 is decreased to about 99% of its theoretical value if the submergence ratio  $H_2/H_1$  equals 0.3. Values of the drowned flow reduction factor  $f$ , by which the theoretical discharge is reduced under the influence of both  $p_2/H_1$  and  $H_2/H_1$ , can be read from Figure 6.21.

The accuracy of the discharge coefficient  $C_e = C_0 C_1 C_2$  of a WES-spillway which has been constructed with care and skill and is regularly maintained will be sufficient

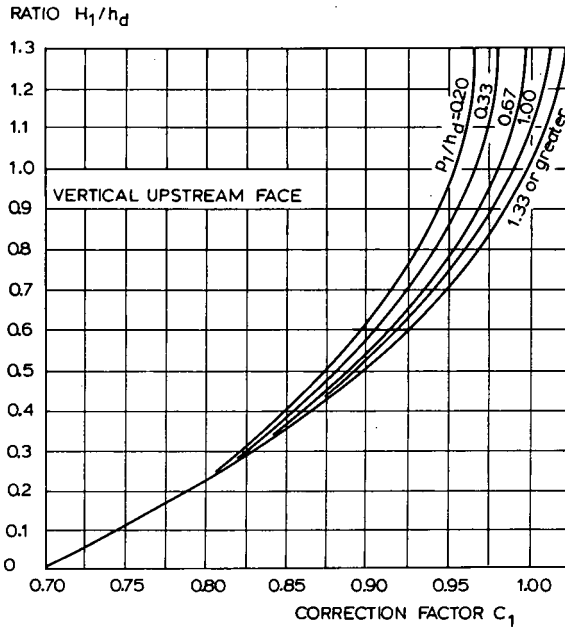


Figure 6.19 Correction factor for other than design head on WES-spillway (after Chow 1959, based on data of USBR and WES 1952)

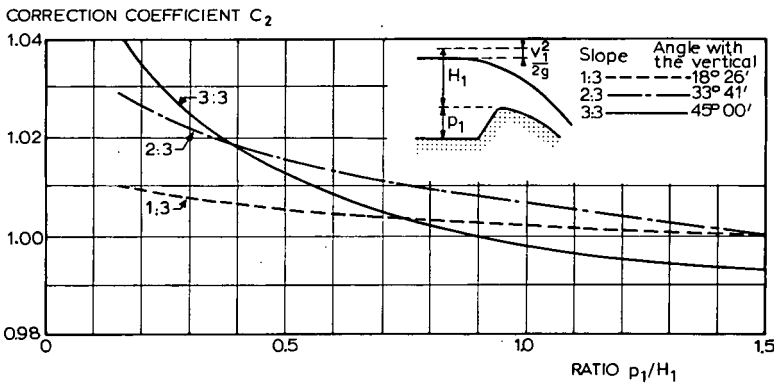


Figure 6.20 Correction factor for WES-spillway with sloping upstream face (after U.S. Bureau of Reclamation 1960)



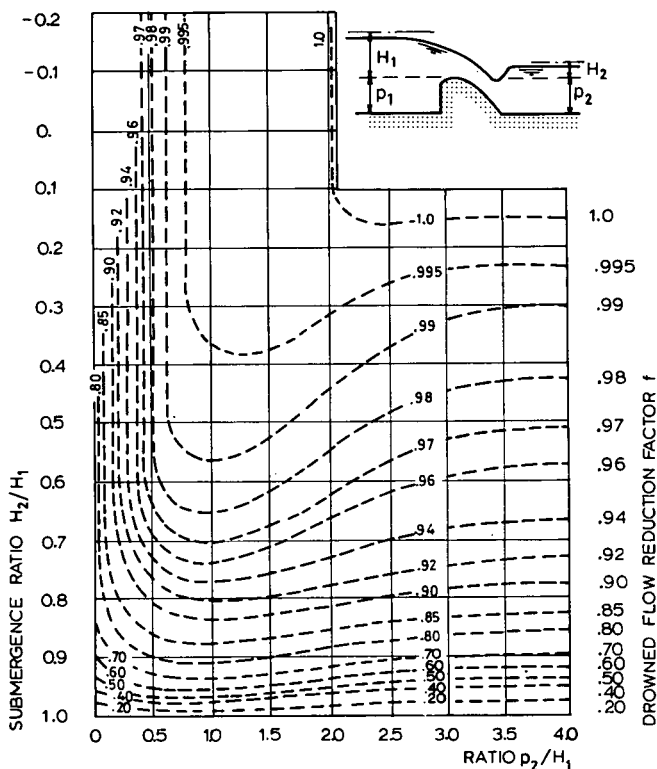


Figure 6.21 Drowned flow reduction factor as a function of  $p_2/H_1$  and  $H_2/H_1$  (Adapted from U.S. Army Corps of Engineers, Waterways Experimental Station 1952)

for field conditions. The error of  $C_c$  may be expected to be less than 5%. The method by which this error is to be combined with other sources of error is shown in Annex 2.

### 6.6.3 Limits of application

For reasonable accuracy, the limits of application of a weir with a WES-spillway crest are:

- The upstream head over the weir crest  $h_1$  should be measured a distance of 2 to 3 times  $h_{1max}$  upstream from the weir face. The recommended lower limit of  $h_1$  is 0.06 m;
- To prevent water surface instability in the approach channel, the ratio  $p_1/h_1$  should not be less than 0.20;
- To reduce the influence of boundary layer effects at the side walls of the weir, the ratio  $b_c/H_1$  should not be less than 2.0;
- To obtain a high  $C_c$ -value, the ratio  $p_2/H_1$  should not be less than about 0.75;
- The modular limit  $H_2/H_1 = 0.3$ , provided that the tailwater channel bottom does not interfere with the flow pattern over the weir ( $p_2/H_1 \geq 0.75$ );

- f. The minimum allowable pressure at the weir crest equals  $-4.0$  m water column ( $P/\rho g \geq -4.0$  m).

## 6.7 Cylindrical crested weir

### 6.7.1 Description

A cylindrical crested weir is an overflow structure with a rather high discharge coefficient and is, as such, very useful as a spillway. The weir consists of a vertical upstream face, a cylindrical crest which is horizontal perpendicular to the direction of flow, and a downstream face under a slope 1-to-1 ( $\alpha = 45^\circ$ ) as shown in Figure 6.22. The abutments are vertical and should be rounded in such a manner that flow separation does not occur.

If the energy head over the weir crest as a function of the radius of the crest is small ( $H_1/r$  is small), the pressure on the weir crest is positive; if, however, the ratio  $H_1/r$  becomes large, the position of the overfalling nappe is depressed below that of a free falling nappe and the pressure of the crest becomes negative (sub-atmospheric) and at the same time causes an increase of the discharge coefficient. The magnitude of the local minimum pressure at the crest  $(P/\rho g)_{\min}$  was measured by Escande & Sannes (1959), who established the following equation from which  $P/\rho g$  minimum can be calculated

$$P/\rho g = H_1 - (H_1 - y) \left\{ (r + ny)/r \right\}^{2/n} \quad (6-14)$$

where  $n = 1.6 + 0.35 \cot \alpha$  and  $y$  equals the water depth above the weir crest, which approximates  $0.7 H_1$ , provided that the approach velocity is negligible. For a weir with a 1-to-1 sloping downstream face ( $\cot \alpha = 1$ ) the minimum pressure at the weir crest in metres water column  $(P/\rho g)_{\min}$  with regard to the energy head  $H_1$  is given as a function of the ratio  $h_1/r$  in Figure 6.23. To avoid the danger of local cavitation, the minimum pressure at the weir crest should be limited to  $-4$  m water column. This limitation,

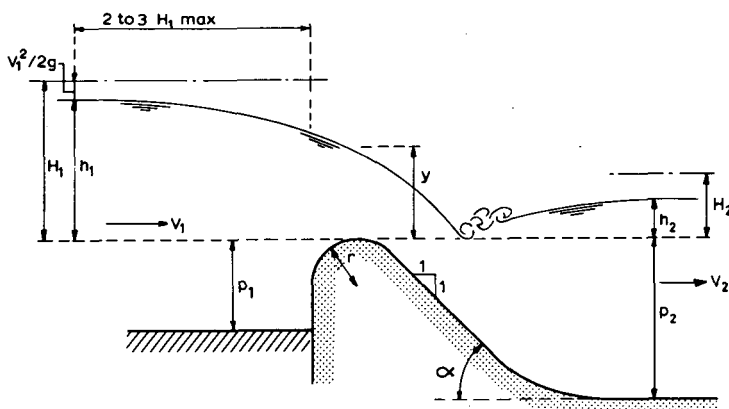


Figure 6.22 The cylindrical crested weir

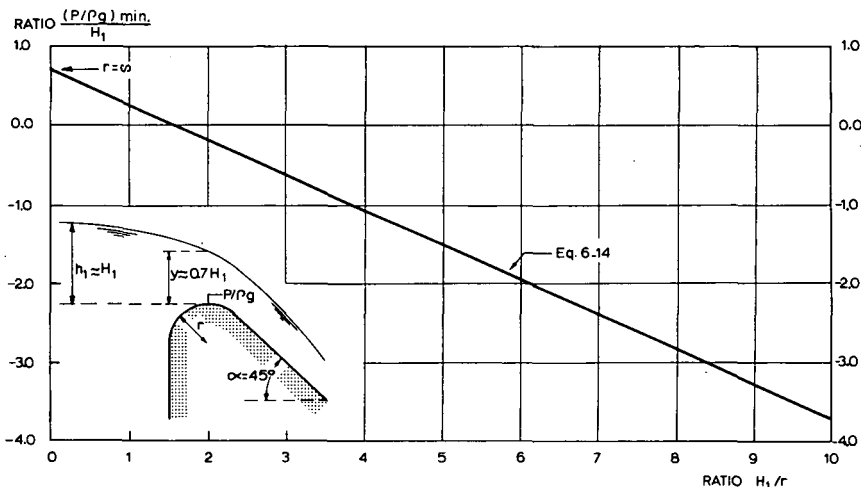


Figure 6.23 Minimum pressure at cylindrical weir crest as a function of the ratio  $H_1/r$

together with the maximum energy head over the weir crest, will give a limitation on the ratio  $H_1/r$  which can be obtained from Figure 6.23.

To allow the cylindrical-crested weir to function as a high capacity overflow weir, the crest height above the downstream channel bed should be such that this channel bed does not interfere with the formation of the overflowing nappe. Therefore the ratio  $p_2/H_1$  should not be less than unity.

## 6.7.2 Evaluation of discharge

The basic head-discharge equation for a short-crested weir with a rectangular control section reads, according to Section 1.10

$$Q = C_e \frac{2}{3} \sqrt{\frac{2}{3}} g b_c H_1^{1.5} \quad (6-15)$$

where the effective discharge coefficient  $C_e$  equals the product of  $C_0$  (which is a function of  $H_1/r$ ), of  $C_2$  (which is a function of  $p_1/H_1$ ) and of  $C_2$  (which is a function of  $p_1/H_1$  and the slope of the upstream weir face) ( $C_e = C_0 C_1 C_2$ ). The basic discharge coefficient is a function of the ratio  $H_1/r$  and has a maximum value of  $C_0 = 1.49$  if  $H_1/r$  exceeds 5.0 as shown in Figure 6.24.

The  $C_0$ -values in Figure 6.24 are valid if the weir crest is sufficiently high above the average bed of the approach channel ( $p_1/H_1 \geq$  about 1.5). If, on the other hand,  $p_1$  approaches zero, the weir will perform as a broad-crested weir and have a  $C_e$ -value of about 0.98, which corresponds with a discharge coefficient reduction factor,  $C_1$ , of  $0.98/1.49 \approx 0.66$ . Values of the reduction factor as a function of the ratio  $p_1/H_1$  can be read from Figure 6.25.

No results of laboratory tests on the influence of an upstream sloping weir face

are available. It may be expected, however, that the correction factor on the basic discharge coefficient,  $C_2$ , will be about equal to those given in Figure 6.20 for WES-spillway shapes.

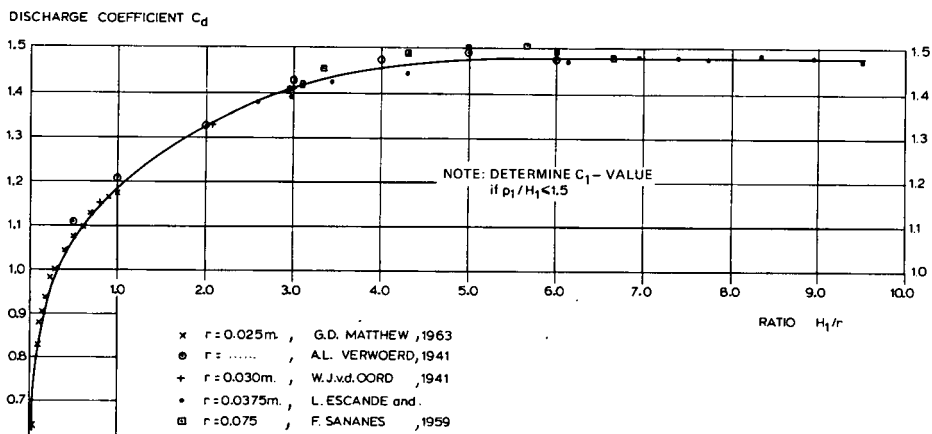


Figure 6.24 Discharge coefficient for cylindrical crested weir as a function of the ratio  $H_1/r$

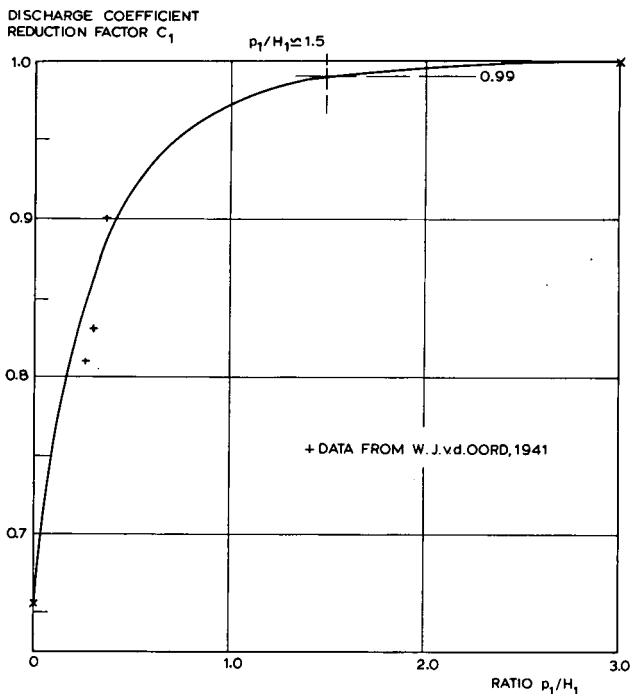


Figure 6.25 Reduction factor  $C_1$  as a function of the ratio  $p_1/H_1$

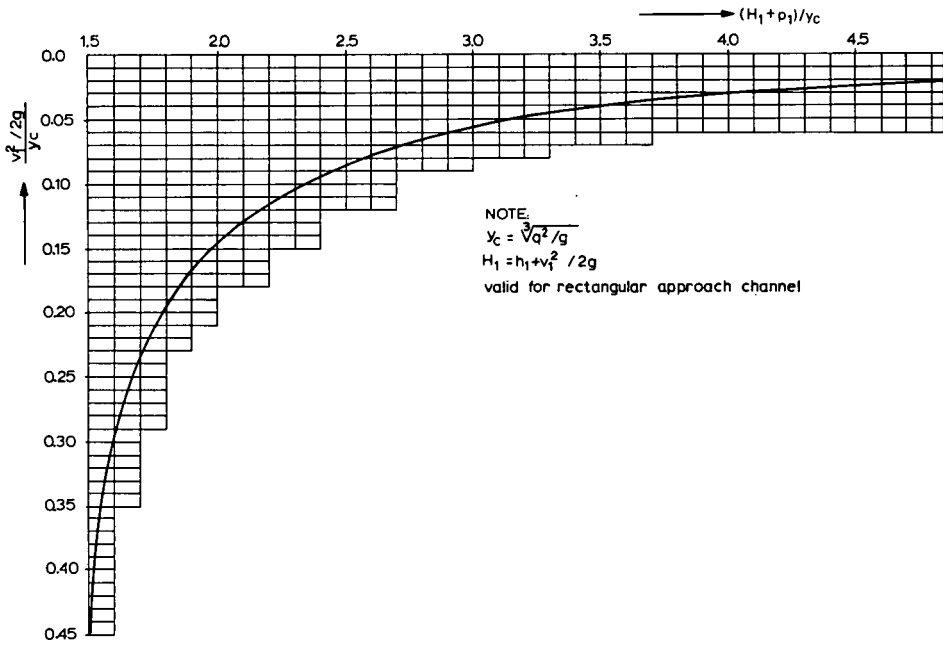


Figure 6.26 Graph for the conversion of  $H_1$  into  $h_1$  (after Van der Oord 1941)

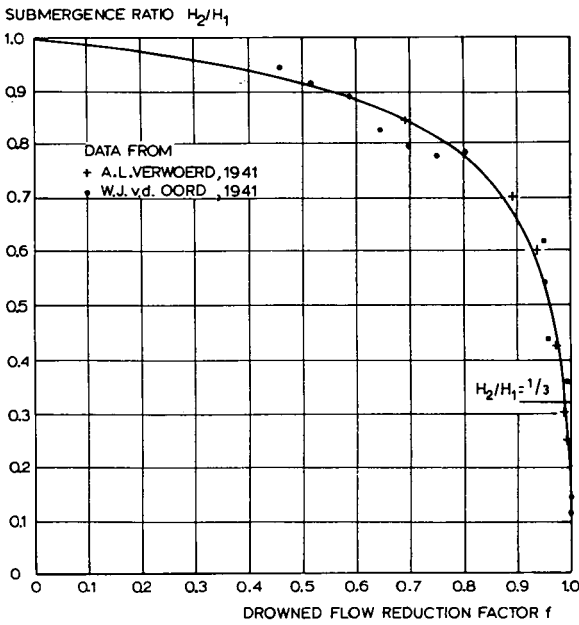


Figure 6.27 Drowned flow reduction factor as a function of  $H_2/H_1$

For each energy head over the weir crest, a matching discharge can be calculated with the available data, resulting in a  $Q-H_1$  curve. With the aid of Figure 6.26, this  $Q-H_1$  relationship can be changed rather simply into a  $Q-h_1$  relationship. For each value of the ratio  $(H_1 + p_1)/y_c$  a corresponding value of  $(v_1^2/2g)/y_c$  can be obtained, where  $y_c$  is the critical depth over the weir crest, so that  $h_1 = H_1 - v_1^2/2g$  can be calculated.

If we define the modular limit as that submergence ratio  $H_2/H_1$  which produces a 1% reduction from the equivalent discharge ( $f = 0.99$ ), we see in Figure 6.27 that the modular limit equals about 0.33. Values of the drowned flow reduction factor as a function of the submergence ratio can be obtained from Figure 6.27.

The accuracy of the effective discharge coefficient of a well-maintained cylindrical-crested weir which has been constructed with reasonable care and skill will be sufficient for field conditions. It can be expected that the error of  $C_c = C_0C_1C_2$  will be less than 5%. The method by which this error is to be combined with other sources of error is shown in Annex 2.

### 6.7.3 Limits of application

For reasonable accuracy, the limits of application of a cylindrical-crested weir are:

- a. The upstream head over the weir crest  $h_1$  should be measured a distance of 2 to 3 times  $h_{1\max}$  upstream from the weir face. The recommended lower limit of  $h_1 = 0.06$  m;
- b. To prevent water surface instability in the approach channel, the ratio  $p_1/h_1$  should not be less than 0.33;
- c. To reduce the boundary layer effects of the vertical side walls, the ratio  $b_c/H_1$  should not be less than 2.0;
- d. On high head installations, the ratio  $h_1/r$  should be such that the local pressure at the crest is not less than  $-4$  m water column;
- e. To prevent the tailwater channel bottom from influencing the flow pattern over the weir, the ratio  $p_2/H_1$  should not be less than unity;
- f. The modular limit  $H_2/H_1 = 0.33$ .

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