8
OPEN CHANNEL FLOW MEASUREMENT STRUCTURES

8.1 Introduction

Open channel flow measurement structures are so designed that the flow rate can be reliably determined from measurement of the upstream head relative to a reference level in the 'control section' of the structure. The control section may incorporate a weir, orifice plate or critical depth flume.

To insure that there is a unique relationship between the upstream head and flow rate it is essential that the upstream head is not influenced by variations in the downstream or 'tailwater' level. When flow is not influenced by the tailwater level, conditions are said to be "modular" and the upstream head is entirely determined by the control section of the measuring structure.

The following measuring structures are discussed in this chapter:

(1) broad-crested weirs

(2) sharp-crested weirs

(3) long-throated flumes

(4) sharp-edged orifices

8.2 The broad-crested weir

The broad-crested weir has a raised horizontal cill of sufficient length in the flow direction to effect an horizontal surface and hydrostatic pressure distribution for at least a short distance, as shown on Fig 8.1. Neglecting any energy losses between the upstream and control sections:

$$H_1 = 1.5h_c = 1.5\left(\frac{Q^2}{gb^2}\right)^{0.33} \qquad (8.1)$$

hence

$$Q = \frac{2}{3}b\sqrt{\frac{2}{3}g}\ H_1^{1.5} \qquad (8.2)$$

For application to practical flow measurement, this equation is written in the form

$$Q = C_d C_v \frac{2}{3}b\sqrt{\frac{2}{3}g}\ \ h_1^{1.5} \qquad (8.3)$$

where C_d is an empirically determined discharge coefficient and C_v is an approach velocity coefficient that allows for the replacement of H_1 by h_1 in the discharge equation.

The discharge coefficient C_d is a function of the upstream head over the cill h_1, the cill length L in the flow direction, the crest width b and the roughness of the flow surface. The following expression for C_d has been proposed by Bos (1976):

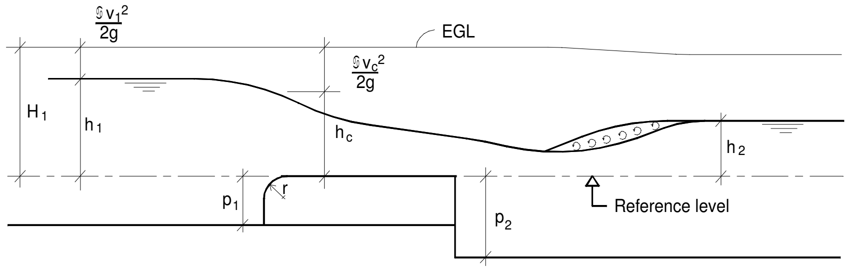

Fig 8.1 **Broad-crested weir**

$$C_d = \left[1 - \frac{2x(L-r)}{b}\right]\left[1 - \frac{x(L-r)}{h_1}\right]^{1.5} \qquad (8.4)$$

where x is a surface roughness parameter, which for well-finished concrete may be taken as 0.005 and smooth surfaces as 0.003.

The approach velocity coefficient for discharge measurement structures in general is given by the relation

$$C_v = \left(\frac{H_1}{h_1}\right)^n \qquad (8.5)$$

where n is the head parameter exponent in the discharge equation, in this instance having the value 1.5.

The following practical design recommendations have been proposed by Bos (1976):

(l) $h_1 \geq 0.06$m or $\geq 0.05L$, whichever is greater
(2) radius of cill nose $r = 0.2\ H_{1(max)}$
(3) $p \geq 0.15$m or $\geq 0.67H_1$, whichever is greater
(4) $20H_1 \geq L \geq 2H_1$, to insure parallel flow while avoiding undulations over the cill.
(5) $b \geq 0.3$m or $\geq H_1$ or $\geq L/5$, whichever is greater.
(6) to insure modular flow conditions the downstream depth and step height p should comply with the modular limit values set out in Table 8.1.

Table 8.1
Modular limit values for broad-crested weirs

	H_2/H_1	
H_1/p_2	Vertical back face	Sloping back face (1:4)
0.1	0.71	0.74
0.2	0.74	0.79
0.4	0.78	0.85
0.6	0.82	0.88
0.8	0.84	0.91
1.0	0.86	0.92
2.0	0.90	0.96
4.0	0.94	0.97
7.0	0.96	0.98
10.0	0.98	0.99

8.3 The sharp-crested weir

Sharp-crested or thin plate weirs are widely used for measurement of small to medium discharges. The control section opening may be of rectangular, triangular (V-notch) or exponential shape (Sutro). The thickness of the plate crest in the direction of flow is generally less than 2mm. If the plate thickness exceeds 2mm, a bevelled edge is formed, as illustrated on Fig 8.2. Sharp-crested weirs are placed vertically, the weir plate being normal to the direction of flow.

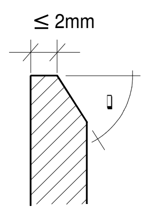

Fig 8.2 Profile of sharp-crested weir edge
$\theta = 45^o$ for rectangular weirs and 60^o for non-rectangular weirs

8.3.1 Rectangular sharp-edged weirs

Fig 8.3 defines the dimensional parameters for rectangular sharp-edged weirs. Such weirs can be treated as simple orifices to give the theoretical discharge equation:

$$Q = \frac{2}{3}\sqrt{2g}\ bh_1^{\ 1.5} \tag{8.6}$$

where the velocity of approach is considered negligible. For practical use in flow measurement this equation can be written in modified form as proposed by Kindswater and Carter (1957):

$$Q = C_e\ \frac{2}{3}\sqrt{2g}\ b_e h_e^{\ 1.5} \tag{8.7}$$

where the discharge coefficient $C_e = K_1 + K_2(h_1/p_1)$; the effective weir width $b_e = b + K_b$; the effective weir head $h_e = h + 0.001$m. Numerical values for the empirical coefficients K_1, K_2 and K_b are given in Table 8.2.

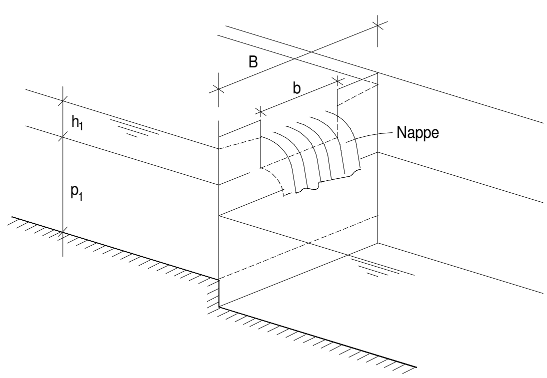

Fig 8.3 **Rectangular thin plate weir**

The following design limits for practical application are suggested by Bos (1976):

(1) $h_1 \geq 0.03m$;

(2) $h_1 / p_1 \geq 2$; $p_1 \geq 0.10m$;

(3) $b \geq 0.15m$;

(4) to allow unobstructed weir overspill (aerated nappe) the tailwater level should be at least 0.05m below the weir crest level.

Table 8.2
Coefficient values for rectangular sharp-crested weirs
(Kindsvater and Carter 1957)

b/B	K_1	K_2	K_b
1.0	0.602	0.075	-0.0009
0.9	0.599	0.064	0.0037
0.8	0.597	0.045	0.0043
0.7	0.595	0.030	0.0041
0.6	0.593	0.018	0.0037
0.5	0.592	0.011	0.0030
0.4	0.591	0.0058	0.0027
0.3	0.590	0.0020	0.0025
0.2	0.589	-0.0018	0.0024
0.1	0.588	-0.0021	0.0024
0	0.587	-0.0023	0.0024

8.3.2 V-notch weirs

Fig 8.4 defines the dimensional parameters for thin plate V-notch weirs. The basic theoretical discharge equation for such weirs is

$$Q = \frac{8}{15}\sqrt{2g}\,\tan(\theta/2)h_e^{2.5} \tag{8.8}$$

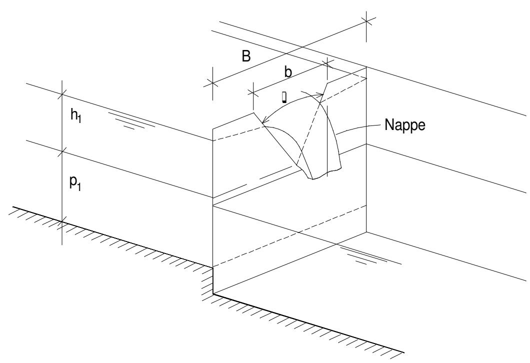

Fig 8.4 Thin plate V-notch weir

Kindswater and Carter (1957) proposed the following practical form of this equation:

$$Q = C_e\frac{8}{15}\sqrt{2g}\,\tan(\theta/2)h_e^{2.5} \tag{8.9}$$

where the discharge coefficient C_e is a function of the notch angle θ, as given in Table 8.3. The effective head $h_e = h_1 + K_h$, where K_h is an empirical head correction factor, being a function of the notch angle θ, as given in Table 8.3.

Table 8.3
V-notch sharp-crested weir coefficients
(Kindsvater and Carter, 1957)

Notch angle θ (deg)	20	40	60	80	100
C_e	0.595	0.581	0.577	0.577	0.580
K_h(mm)	2.8	1.8	1.2	0.85	0.80

The following design limits for practical application of sharp-crested v-notch weirs are recommended by Bos (1976):

(1) $h_1/p_1 \leq 1.2$
(2) $h_1/B \leq 0.4$; $B \geq 0.60$m
(3) $0.60 \geq h_1 \geq 0.05$m
(4) $p_1 \geq 0.10$m
(5) $100^o \geq \theta \geq 25^o$
(5) tailwater level ≥ 0.05m below vertex of V-notch.

8.3.3 The proportional-flow (Sutro) weir

When installed in a rectangular channel, the proportional-flow weir regulates flow such that the discharge is linearly related to the upstream depth and in consequence the mean upstream velocity remains constant. Hence, it is sometimes used as an outlet control device on grit-separation channels in sewage treatment plants. The geometric outline of the weir profile is given on Fig 8.5, which also defines its dimensional parameters.

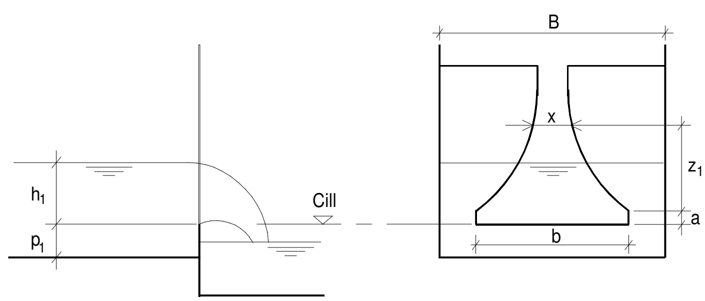

Fig 8.5 **Proportional-flow weir**

The weir opening has a lower rectangular portion connected to an upper curved portion, the width of which reduces according to the relation

$$\frac{x}{b} = 1 - \frac{2}{\pi} \tan^{-1} \sqrt{\frac{z_1}{a}} \tag{8.10}$$

The discharge equation for this weir type may be written as follows (Bos, 1976):

$$Q = C_d b \sqrt{2ga} \left(h_1 - a/3 \right) \tag{8.11}$$

Recommended values for the discharge coefficient C_d, as a function of a and b, are given in Table 8.4.

The values given in Table 8.4 may also be used for crestless weirs provided the weir width b is not less than 0.15m (Singer and Lewis, 1966).

The following design limits for practical application are recommended by Bos (1976):

(1) $h1 \geq 2a$ or ≥ 0.03m, whichever is greater;
(2) $a \geq 0.005$m;
(3) $b \geq 0.15$m;
(4) $b/p_1 \geq 1$;
(5) $B/b \geq 3$
(6) tailwater level ≥ 0.05m below weir crest.

Table 8.4
Proportional-flow weir discharge coefficient C_d

a	b (m)				
(m)	0.15	0.23	0.30	0.38	0.46
0.006	0.608	0.613	0.617	0.618	0.619
0.015	0.606	0.611	0.615	0.617	0.617
0.030	0.603	0.608	0.612	0.613	0.614
0.046	0.601	0.606	0.610	0.612	0.612
0.061	0.599	0.604	0.608	0.610	0.610
0.076	0.598	0.603	0.607	0.608	0.609
0.091	0.597	0.602	0.606	0.608	0.608

8.4 The critical depth flume

The critical depth flume is created by reducing the cross-sectional area of a channel sufficiently for critical flow to occur at the constricted section or throat of the flume. In 'long-throated' flumes the prismatic throat section has a sufficient length in the flow direction to achieve parallel flow and associated hydrostatic pressure distribution, at least over a short length. The flume has a horizontal invert, which constitutes the reference level for flow measurement.

The following analysis relates to the general case of a trapezoidal log-throated flume in a trapezoidal channel, as illustrated on Fig 8.6. Rectangular long-throated flumes in rectangular or trapezoidal channels may be regarded as particular examples of this category of flow-measuring structure.

Critical flow conditions are created in the throat (control) section. Under such conditions, the discharge can be expressed, as shown in chapter 7, as follows:

$$Q = \left(\frac{gA_c^{\ 3}}{\alpha W_c}\right)^{0.5} \tag{8.12}$$

$$H_1 = y_c + \frac{A_c}{2W_c} \tag{8.13}$$

where y_c, A_c and W_c represent the flow, flow cross-sectional area and water surface width at the critical section, respectively. Combining these two relations, assuming α=1, we get

$$Q = A_c\sqrt{2g(H_1 - y_c)} \tag{8.14}$$

For practical computational purposes, this relation is written in the form:

$$Q = C_d C_v A_c\sqrt{2g(h_1 - y_c)} \tag{8.15}$$

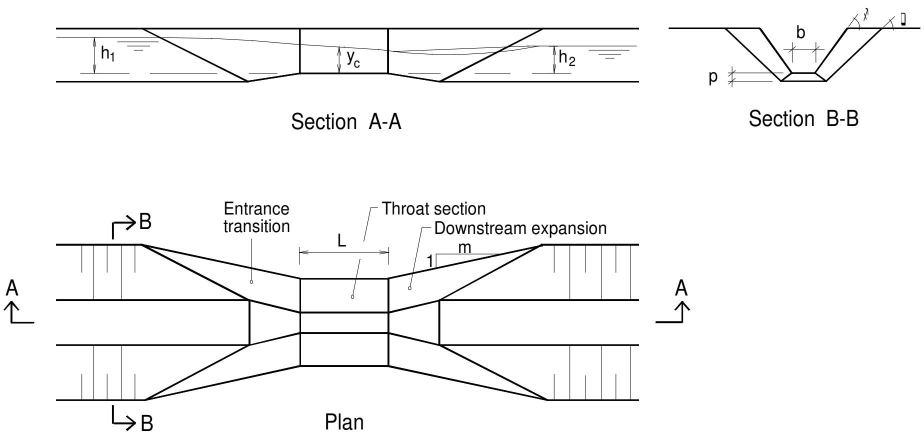

Fig 8.6 **Trapezoidal long-throated flume**

The discharge coefficient C_d is a measure of the variation of the discharge from its theoretical value as expressed by equation (8.14). This variation is due to friction head loss between the point at which the upstream head is measured and the throat and is also influenced by boundary layer separation in the throat (Ackers et al., 1979). Based on experimental data (Bos, 1976) C_d can be empirically correlated with H_1/L using the following linear approximations:

$$0.1 < H_1/L < 0.3 \qquad C_d = 0.89 + 0.20(H_1/L)$$
$$0.3 < H_1/L < 1.0 \qquad C_d = 0.93 + 0.07(H_1/L)$$

The velocity coefficient $C_v = (H_1/h_1)^{1.5}$ allows the substitution of the measured head h_1 for the total head H_1.

The modular limit requirement is satisfied if the available head difference between the upstream and downstream water levels can accommodate the head losses through the structure. A major component of this head loss is that due to flow expansion from the throat cross-section to the downstream channel section. The influence of the expansion rate on the modular limit is indicated in Table 8.5.

Where the head loss is not too critical, an expansion rate in the range 1:3 to 1:6 is recommended. Where the head loss is critical, the designer may select a more gradual downstream expansion to reduce the head loss through the structure.

On the inflow side, a convergence rate of about 1:3 is typically recommended for the transition from the upstream channel section to the throat section.

Table 8.5
Effect of downstream expansion rate on modular limit
(Ackers et al. 1979)

Expansion rate (1:m, Fig 8.6)	Modular limit (H_2/H_1)
1:20	0.91
1:10	0.83
1:6	0.80
1:3	0.74

For accurate flow measurement application, Bos (1976) recommends the following design limits:

(1) $h_1 \geq 0.06$m or ≥ 0.1L, whichever is greater;
(2) Froude number $F_r = v_1/(gA_1/W_1)$ in approach channel to be ≤ 0.5;
(3) $1.0 \geq H_1/L \geq 0.1$
(4) $W_t \geq 0.3$m or $\geq H_{1(max)}$ or $\geq L/5$, whichever is greater, where Wt is the width of the water surface in the throat at maximum discharge.

8.5 Sharp-edged orifices

Fig 8.7 shows the typical installation configuration for an orifice plate in an open channel. Such an orifice may have a free discharge to air or may be submerged on the downstream side. The basic discharge equation for orifices is:

$$Q = C_d C_v A\sqrt{2g\ \Delta h} \tag{8.16}$$

where Δh is the differential head for submerged orifices and is the upstream head relative to the centre of the orifice for freely discharging orifices; A is the orifice area. Orifices are preferably installed where the velocity of approach is negligible. The edge profile for orifice plates should comply with the specification recommended for thin plate weirs (Fig 8.2).

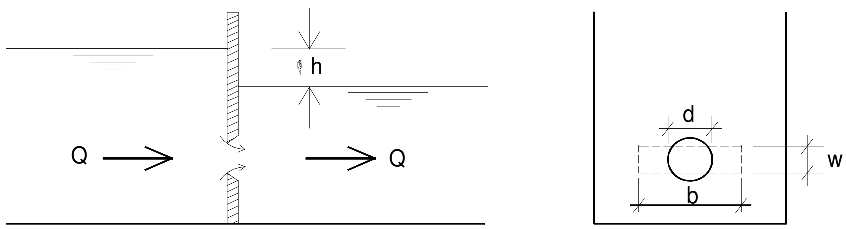

Fig 8.7 Sharp-edged orifice with a section showing circular and rectangular orifices

8.5.1 The circular sharp-edged orifice

Values for C_d for free flow through submerged sharp-edged circular orifices are given in Table 8.6. The following practical design limits are recommended:

(1) edge distance to channel boundaries $\geq d/2$.

(2) upstream channel cross-sectional area ≥ 10 x orifice area.

(3) upstream submergence of top of orifice ≥ d.

(4) $\Delta h \geq 0.03$m.

Table 8.6
Discharge coefficient C_d for sharp-edged circular orifices
(negligible approach velocity)

Orifice diameter d (m)	C_d free flow	C_d submerged flow
0.02	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
≥0.075	0.60	0.60

8.5.2 The rectangular sharp-edged orifice

The rectangular orifice has a width b and depth w, A = bw. Under fully contracted, submerged conditions, where the velocity of approach is negligible, the discharge coefficient C_d may be taken as 0.61. If the contraction is suppressed along part of the orifice perimeter, the coefficient of discharge is modified (Bos, 1976) as follows:

$$C_d = 0.61(1 + 0.15r) \tag{8.17}$$

where r is the ratio of suppressed length to the total perimeter length.

Where both side and bottom contractions are suppressed, for example, as in flow under a sluice gate, the discharge equation may be written in the form

$$Q = C_d C_v bw\sqrt{2g(y_1 - y)} \tag{8.18}$$

where y_1 is the depth of flow upstream of the sluice gate and y is the contracted downstream depth. Introducing the parameters $n = y_1/w$ and $\delta = y/w$ (δ is the coefficient of contraction), eqn (8.18) becomes

$$Q = C_d C_v bw^{1.5}\sqrt{2g(n - \delta)} \tag{8.19}$$

or

$$Q = KA\sqrt{2gw} \tag{8.20}$$

where K is a function of n, δ, C_d, and C_v; K-values are given in Table 8.7.

To insure free flow below a sluice gate (modular flow conditions), the downstream water depth should not exceed the sequent hydraulic jump depth, as computed from the known supercritical depth y on the discharge side of the sluice gate.
For accurate flow measurement, the following design limits are recommended by Bos (1976) for sharp-edged rectangular orifices:

(1) To insure fully contracted flow, the edge distance to the channel boundaries should be at least twice the least dimension of the orifice.
(2) $\Delta h \geq 0.03m$
(3) submergence of top edge $\geq$ w
(4) $w \geq 0.02m$

Table 8.7
Sluice gate discharge coefficients
(Bos 1976)

y_1/w	δ	C_d	K
1.5	0.648	0.600	0.614
1.7	0.637	0.598	0.665
1.9	0.632	0.597	0.713
2.2	0.628	0.596	0.780
2.6	0.626	0.597	0.865
3.0	0.625	0.599	0.944
4.0	0.624	0.604	1.124
5.0	0.624	0.607	1.279

8.6 Selection and design of flow measurement structures

The installation of a flow measurement structure will in most instances cause an increase in upstream water levels throughout the flow range. This characteristic is often critically significant in the selection and design of such structures and is an aspect of selection and design that must always be carefully considered. It is also important to check that modular flow conditions prevail over the full flow range.

It is desirable to have an unobstructed approach channel of reasonably uniform cross-section and straight for a distance of 10 to 20 times the channel width. The selected location for head measurement should be at a distance of $3h_1$ to $4h_1$ upstream of the control section.

Manual head-measuring devices include the staff gauge; the point gauge and the hook gauge, while a variety of water level-sensing devices are available for automatic recording of head. Stilling chambers are normally used in conjunction with automatic head-measuring systems. They offer the advantage of eliminating water surface ripples and similar transient level variations.

8.7 Flume and weir design using ARTS software

The ARTS software package developed by Aquavarra Research Limited facilitates the design of flume and weir flow measurement structures, based on the flow equations and compliant with the application limits outlined in this chapter. The output includes a calibration curve and fitted head/discharge equation.

References

Ackers, P., White, W. R., Perkins, J. A. and Harrison, A. J. M. (1979) Weirs and flumes for flow measurement, Wiley, New York

Bos, G. (ed.) (1976) Discharge measurement structures. International Institute for Land Reclamation and improvement, Wageningen, The Netherlands.

Harrison, A. J. M. (1967). The streamlined broad-crested weir, Proc. ICE, 38, 657-678.

Kindsvater, C. E. and Carter, R. W. C. (1957) Discharge characteristics of rectangular thin-plate weirs, J. Hyd. Div. ASCE, HY 6, 83.

Singer, J. and Lewis, D. C. G. (1966) Proportional-flow weirs for automatic sampling or dosing. Water and Water Engineering, 70, 105-111.

Related reading

Bos, M. G., Replogle, J. A. and Clemmens, A. S. (1984) Flow measuring flumes for open channel systems, Wiley, New York.

Cheremisinoff, N. P. (1979) Applied fluid flow measurements, Marcel Dekker, Basel.

Scott, R. W. W. (1982) Developments in flow measurement, Applied Science Publishers, London.