MODELLING ANCILLARIES: WEIR COEFFICIENTS

David Balmforth, MWH

1. SCOPE

This user note gives advice on the choice of coefficient for overflow weirs and orifices when modelling storm sewage overflows and bifurcations.

2. BACKGROUND

Before modelling any ancillary the user must understand:

- the hydraulic performance of the prototype;
- the workings of the algorithm in the computer model.

A combined sewer overflow is typically represented as a small on-line tank with a horizontal base, horizontal weir, and horizontal water surface. Some software can also accommodate an overflow as a hole in the wall of a manhole chamber. In each case the flow in the chamber is assumed to be subcritical and the water level regulated by a throttle on the continuation pipe (WaPUG User Note No 2).

The discharge over the weir (Q_w) is determined by the head above the crest (H_w) using the equation:

$$Q_w = C_w L_w \sqrt{g h_w^n} \qquad (1)$$

where L_w length of weir (or weirs if more than one) (m)
g gravitational acceleration (9.81 m/s^2)
n index, normally 1.5
C_w weir coefficient

Alternatively the user can specify an orifice overflow, such that the discharge through the orifice overflow (Q,) is determined by the equation:

$$Q_0 = C_0 A_0 \sqrt{g H_0^n} \qquad (2)$$

where A_0 area of orifice (m^2)
H_0 head across the orifice (m)
n index, normally 0.5
C_0 orifice coefficient

The user also has the option to specify whether the overspill goes to waste, or to the head of a specified branch.

A bifurcation may be modelled as a special case of an overflow orifice, but with the crest level of the overflow placed a small vertical distance above the chamber floor (e.g. 100 mm).

3. TRANSVERSE WEIRS

3.1 Introduction

Five different flow cases can occur and it is important to establish which case is appropriate in each ancillary. Sometimes the flow case will change during the operation of an overflow and the flow case that occurs during a verification event may be different from that which occurs in a more extreme event (e.g. a design storm).

3.2 Case 1: Free discharge over a weir

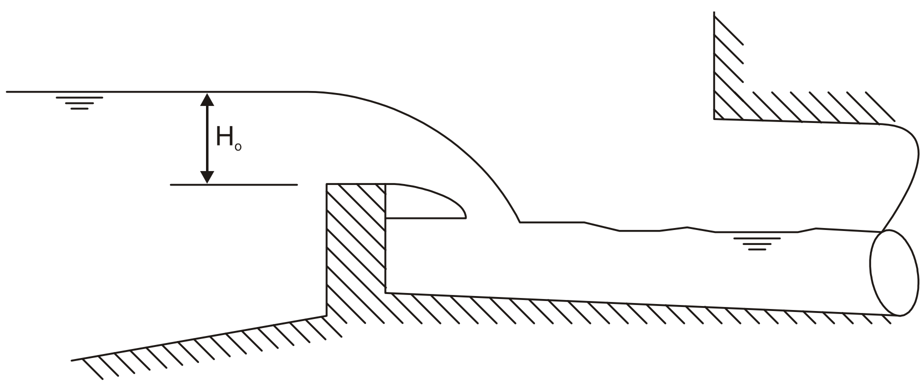


Figure 1a Free discharge over a weir

The discharge is solely dependent upon the head above the crest, and is calculated by Equation 1. The weir control should be selected and the weir coefficient C_w, depends on the geometry of the weir crest, and Table 1 gives suitable values.

Table 1 Values of C_w for Case 1 flow

Weir crest	**C_w**
Sharp edged	0.60
Square crest	0.70
Round crest	0.80

3.3 Case 2: Freely discharging orifice

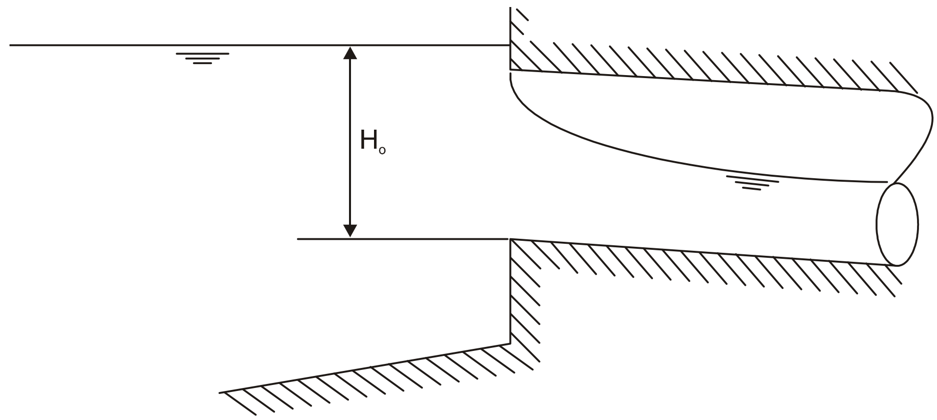


Figure 1b Freely discharging orifice

The discharge is unaffected by water levels in the overflow pipe. The upper surface of the jet springs free from the upper edge of the pipe entry, and is vented to the atmosphere. H_0 is defined as the head above the vertex of the overflow orifice, and Q_0 is calculated from Equation 2. The orifice control should be selected and C_0 given a value of 0.85.

3.4 Case 3: Drowned Overflow – overflow pipe not specified in the data file

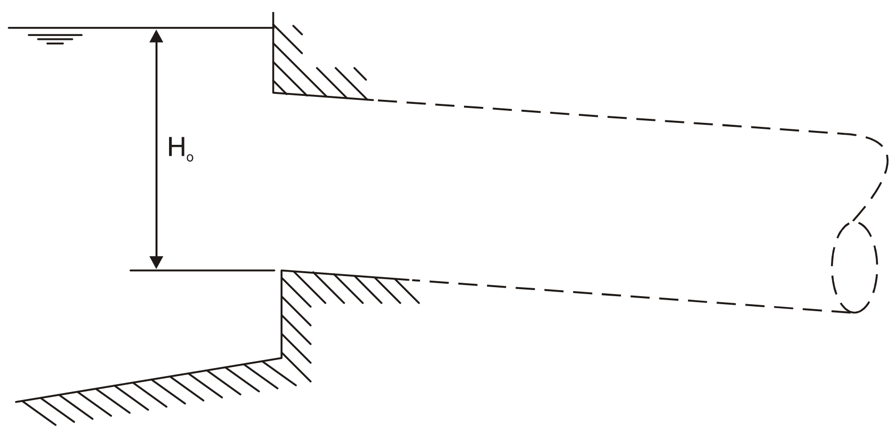


Figure 1c Drowned overflow with overflow pipe not included in model

In this case H_0 is taken as the head above the vertex of the overflow pipe, but energy losses in the overflow pipe (bends, flap valves etc.) can only be accounted for in the value C_0. In this case the orifice control should be selected and C_0 calculated from:

$$C_0 = \frac{2}{\sqrt{\sum K}} \qquad (3)$$

where ΣK is the sum of the loss factors for fittings in the overflow pipe.

Values of appropriate loss factors may be taken from Table 2 which is based on energy loss factors for fittings in water mains. Note that if Equation 3 gives a value in excess of 0.85 then 0.85 should be used.

Table 2 Energy loss factors for pipe fittings

Fitting	K
Sharp entry	0.5
Sharp exit	1.0
90^{o} bend	1.0
Empty silt trap	3.0
Flap valve	3.0 to 6.0

3.5 Case 4: Drowned orifice, overflow pipe specified in the data file

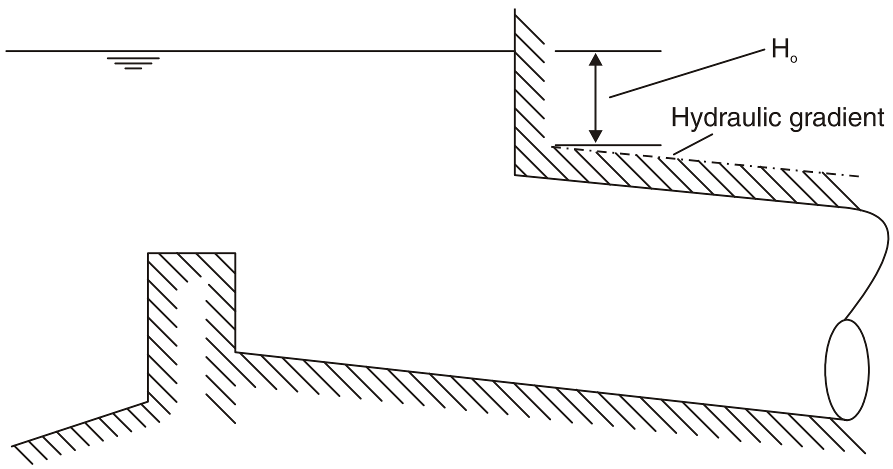


Figure 1d Drowned overflow with overflow pipe included in model

In recent software the overflow pipe is normally included as a second control pipe.

Where an orifice entry is assumed in place of the normal entry losses, the orifice equation 2 has to represent only the energy loss at entry, and C, should be specified as 2.0. In some early software (e.g. WASSP) H was taken as the head above the vertex until the overflow pipe surcharged. Thereafter the difference was used. The sudden change in H, when the pipe surcharged could lead to instabilities in computation if the vertex was low (as in a bifurcation). To overcome such problems overflow pipes were often oversized.

3.6 Case 5: Weir drowned by flow backing up from overflow pipe

This flow condition is quite common. Once the overflow operates and flow builds up in the overflow pipe the weir ceases to become the dominant control. The flow is effectively Case 3 or Case 4 and should be treated as such, but with the invert level of the overflow pipe reset to the weir crest level to give the correct overflow setting.

3.7 Allowance for velocity of approach

Allowance for the velocity energy in flow approaching a transverse weir may be made by multiplying the weir coefficient by a factor F, where

$$F = \frac{1 + \sqrt{1 - 2C_w^2 r^2}}{C_w^2 r^2} \qquad (5)$$

where $r = \dfrac{H_w}{H_w + P}$

P height of weir crest above the chamber floor

Similarly for an orifice overflow:

$$F = \frac{1}{\sqrt{1 - \frac{1}{2} C_0^2 r^2}} \qquad (6)$$

where $r = \dfrac{A_0}{A_u}$

A_u cross sectional area of flow approaching the orifice

These corrections should only be used where there is an appreciable velocity of flow directly approaching the overflow.

4. MODELLING SIDE WEIRS

4.1 Introduction

With side weirs the main issue that arises is the potential variation in water level along the length of the weir due to the reduction in flow in the channel. With low side weirs and where the incoming flow is supercritical there can also be a hydraulic jump in the channel leading to a large and unpredictable change in water level.

Low side weirs normally have a crest below the centre line level of the upstream sewer, and this flow type occurs when the energy at inlet exceeds twice the weir height.

$$d_1 + \frac{V_1^2}{2g} \geq 2C_1 \qquad (7)$$

where d_1 = depth at the upstream end of the weir

V_1 = velocity at the upstream end of the weir

C_1 = height of weir crest at the upstream end of the weir

Fraser (Ref 2) identified five possible flow types of side weir flow, as illustrated in Figure 2.

Type I Flow:

Occurs with low side weirs in mild sloping sewers. A mild sloping sewer is defined as a sewer where uniform flow is subcritical.

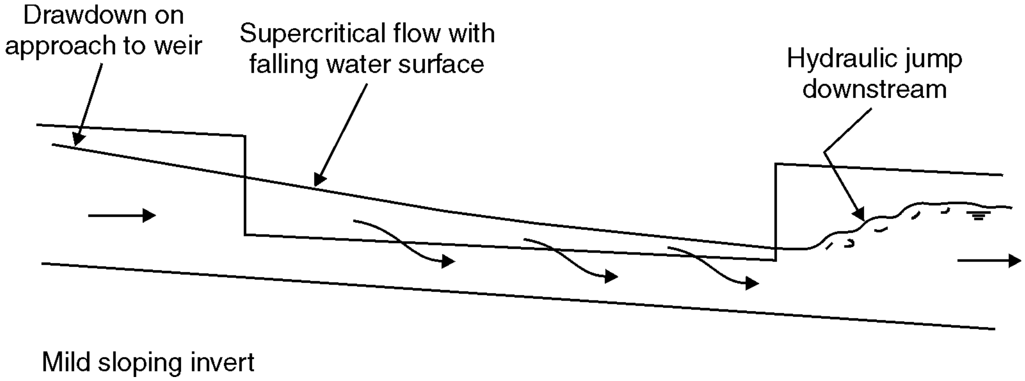


Figure 2a Low side weir

The head over the weir decreases along the length of the chamber and water levels (and hence weir discharge) are governed by conditions at inlet to the chamber, where the depth will be between 0.85 and 0.90 of the critical depth. When this flow type occurs, conditions in the downstream sewer have no influence on the weir discharge nor on the level of the hydraulic gradient in the upstream sewer. Flow in the chamber is supercritical.

Type II Flow:
Occurs with high weirs, which usually have their crest above the centreline level of the upstream sewer. Water levels in the chamber, and the weir discharge, are determined by the throttle at entry to the downstream sewer, and the discharge and level of the hydraulic gradient in that sewer. In this case the head on the weir increases along the length of the chamber. Flow in the chamber is subcritical.

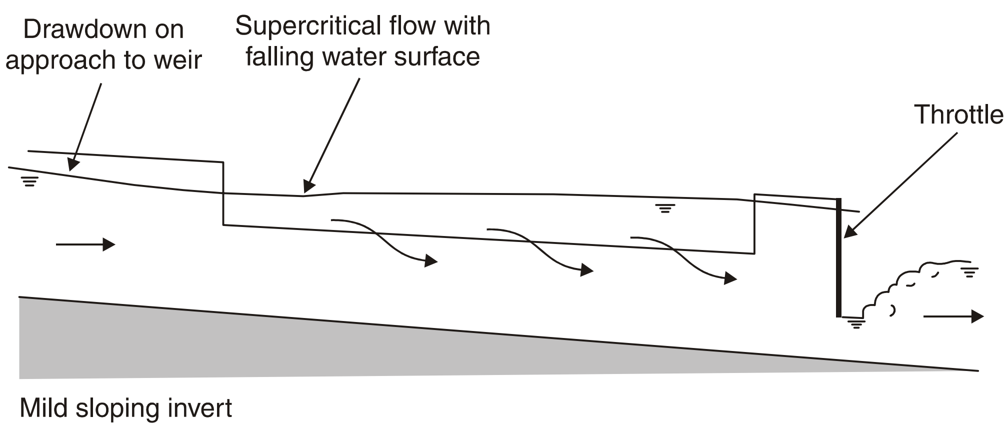
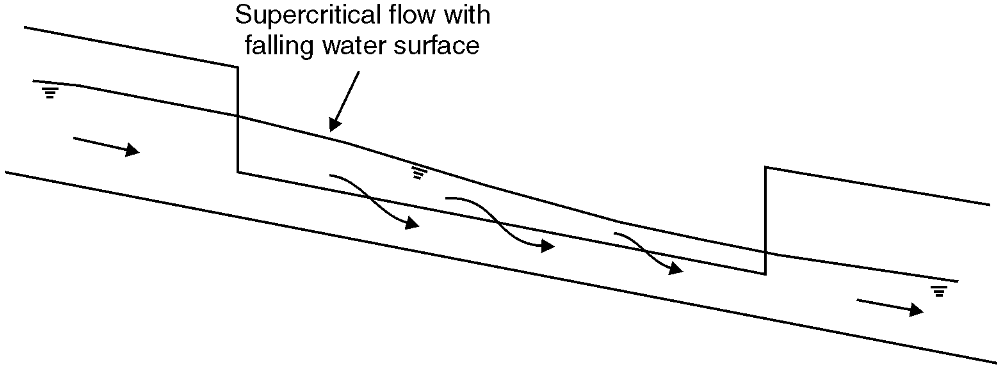


Figure 2b High side weir with throttle

Type III Flow:

Occurs in mild sloping sewers with low side weirs fitted with a throttle at the downstream end of the chamber, or with low side weirs where the downstream sewer is surcharged. Conditions in the downstream sewer influence the weir discharge but do not determine the water level in the upstream sewer. Flow in the chamber is a combination of Type I and II with a hydraulic jump forming.

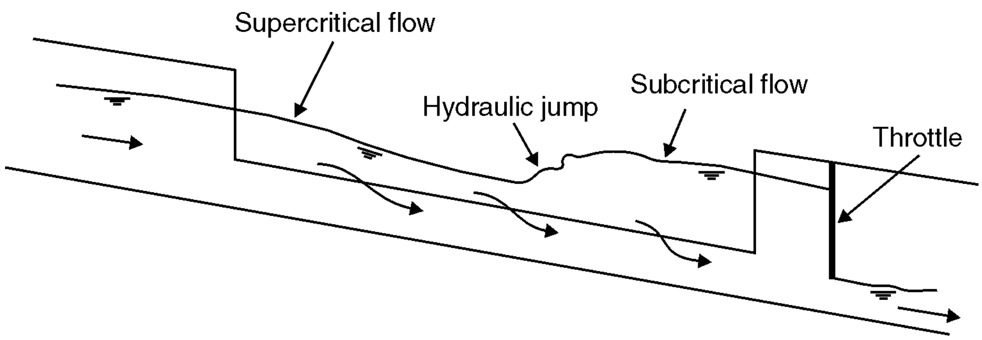


Steep sloping invert

Figure 2c Low side weir on steep slope

Type IV Flow:

Occurs with low weirs in steep sewers. It is similar to Type I flow but the approaching flow is uniform and d_1 approximates to the depth of uniform flow in the upstream sewer.



Steep sloping invert

Figure 2d Low side weir with throttle, on steep slope

Type V Flow:

Occurs with low side weirs in steep sewers where the chamber has a throttle at outlet, or where the downstream sewer is surcharged. It is a combination of Types IV and II and is similar in character to Type III.

A number of alternative methods exist for analysing side weir flow and calculating weir discharges. Balmforth and Sarginson (Ref 1) have reviewed the various methods and explain how the discharge capacity of side weirs can be calculated.

4.2 Modelling requirements - High Side Weirs

Where the user is confident that the flow in the chamber is subcritical, the following approach is recommended.

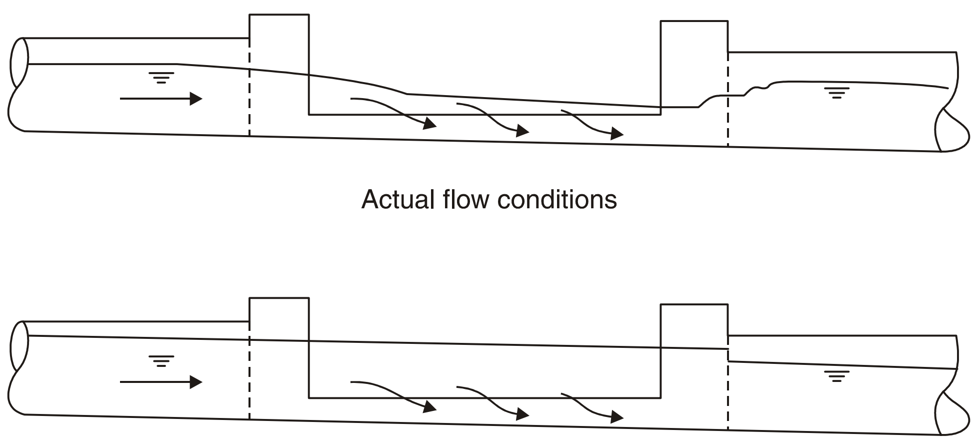
Where there is a significant variation in the head above the crest of a weir, along its length, then an allowance for this should be made in the value of the weir coefficient C_w. For a particular head at the **downstream** end of the weir the discharge over the weir should be calculated (Ref 1) or measured in situ. Denoting this head by H_w, and the corresponding weir discharge Q_w, these values should be substituted into Equation 1 and the corresponding value of C_w calculated.

Where the flow in the chamber can be supercritical (i.e for low side weirs). The method below should be used.

4.3 Modelling requirements - Low Side Weirs

In models it is assumed that the flow in the chamber is governed by the continuation throttle and conditions in the downstream sewer. However, with low side weirs, water levels and weir discharges, are governed by conditions in the upstream sewer at entry to the chamber.

If the actual chamber dimensions and weir coefficients are used directly in the model then the model will over-predict the weir discharge and the water levels in the upstream sewer, as Figure 3 demonstrates.



Model

Figure 3

For a particular inflow, the correct weir discharge can be simulated simply by reducing the weir coefficient to give the desired result. However, the water level in the upstream sewer will be over-predicted and this can affect the discharge capability of the upstream system and flooding may be predicted where it does not occur in practice.

Over-prediction of upstream water levels can be avoided by artificially over-sizing the first sewer length immediately downstream of the chamber, and if necessary reducing its gradient to maintain the correct first spill value. For Type I and IV flows the procedure is best summarised as follows:

(i) using proportional depth-discharge calculations for the upstream and downstream sewers, determine the setting of the overflow;

(ii) upsize the downstream sewer. An increase to twice the actual diameter is normally sufficient, though both higher and lower multiples have proved necessary at times.

(iii) adjust the gradient of the oversize pipe to give the correct overflow setting by raising the downstream invert level (the ground level at this point may also have to be raised);

(iv) for the upstream sewer running approximately three-quarters full, calculate the weir discharge and continuation discharge using an established method of calculating side weir flow (Ref 1). Scumboards fitted to the weir reduce the discharge by 10-20%, and this should be allowed for in estimating the weir flow;

(v) using proportional depth discharge calculations, determine the depth of flow in the oversize downstream sewer at the calculated continuation discharge. Calculate the drop in the level of the hydraulic gradient at the throttle using the software's orifice equation. Add this drop to the depth in the oversize sewer to give the depth at the downstream end of the chamber;

(vi) the software typically assumes a horizontal water level and weir crest in the chamber, so that the downstream depth may now be used to determine the head on the weir and the water level in the upstream sewer. Use the former, together with the actual weir length and the calculated weir discharge, in the software's weir equation to obtain an equivalent weir coefficient for use in the model. This may be much smaller than traditional values;

(vii) review the calculated water level in the upstream sewer and if it is significantly higher or lower than that which actually occurs, after the diameter of the oversize downstream sewer and adjust the other parameters accordingly.

When test running the model, particular attention should be paid to the conditions adjacent to the low side weir overflow. In particular, the sewer immediately downstream of the oversize pipe surcharges to any extent then Type III or V flow conditions will probably occur in practice. In this case the downstream sewer should not be oversized.

For Type III and V flow the following procedure should be adopted:

(i) model the downstream sewer as built;

(ii) assume the hydraulic jump forms half way along the weir that the head along the downstream half of the weir is constant, and that the discharge over the upstream half of the weir is negligible;

(iii) model the weir as a transverse weir. Use the actual weir length. If there is only one side weir, take the transverse weir coefficient (reduced by 10-20% if a scumboard is fitted), and halve it. If there are two weirs, do not halve the coefficient.

With smaller side weir chambers in particular it is possible that the whole chamber becomes drowned so that the weir has little effect, and the weir discharge and continuation flow are determined by the size of their respective outlets. In this case it is better to model the chamber as a ‘hole-in-manhole’ overflow, but with the invert of the overflow orifice set level with the crest of the actual weir.

Note:
Adoption of the above procedures will greatly improve the simulation of the sewer system containing a number of low side weir overflows. During verification, it is permissible to make minor changes to the ancillary data provided they can be justified in the way the ancillary has been modelled, and not purely as a means of force fitting the data. Care should be taken to identify possible Type III and V flow conditions. This is particularly true when running a verified model with design storms where greater surcharging may cause the flow case to change from I to II or IV to V in practice. It may be necessary therefore to amend ancillary data between verification and running with design storms. Time-series rainfall should be in with the verification data however.

Often the data used to model low side weirs appears strange, bearing little resemblance to actual values. This is because the on-line tank model in the software behaves differently from the physical performance of low side weirs, and this has to be compensated for when specifying the ancillary data.

5. OTHER ALLOWANCES

5.1 Allowance for scumboards

Where fitted, scumboards tend to reduce the discharge capacity of weirs. They can have little or no effect, or where their supports are weak they can distort and press against the weir shutting off the flow almost completely. In most cases the vertical distance between the underside of the scumboard and the floor of the chamber will be much greater than the horizontal opening between the scumboard and the face of the weir. It is therefore the latter that usually determines how much the flow is restricted.

If the horizontal opening is greater than the maximum head on the weir then no allowance for the scumboard is needed. At other times a reduction in C, of 10% to 20% is recommended. Where ragging around scumboard supports is excessive a greater reduction should be made.

5.2 Allowance for screens

Screens fitted to the crest of the weirs also tend to reduce the discharge capacity. In this situation it is better to use an equivalent weir length, shorter than the actual weir length rather than adjusting the weir coefficient.

$$L_w = \text{Actual weir length} \quad \frac{h_b}{h_b + t_b}$$

where h_b is the clear spacing between the bars (mm)
t_b the bar thickness (mm)

If the screens are not mechanically raked, a further allowance may be necessary for ragging. For widely spaced bars rags tend to accumulate around the bars, reducing the effective dimension h_b. For finer screens, rags will completely obstruct the openings between the bars immediately above the weir crest. The weir crest height should therefore be increased to allow for this.

6. REFERENCES

1. Balmforth D J and Sarginson E J; A comparison of methods of analysis of side weir flow, Chartered Municipal Engineer, Vol 105, No 10 pp 273-279, October 1978.

2. Fraser W, ‘The Behaviour of Side Weirs in Prismatic Rectangular Channels’, proceedings of the Institution of Civil Engineers, Vol 6, February 1957.

AMENDMENTS

Ver	Description	Date
1.	First Published	August 1993
2.	Revision incorporating material from user note 14	March 2009