International Standard ISO 4377

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Liquid flow measurement in open channels — Flat-V weirs

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Foreword

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It has been approved by the member bodies of the following countries :

Australia	India	Spain
China	Italy	Switzerland
Czechoslovakia	Netherlands	United Kingdom
France	Romania	USA
Germany, F.R.	South Africa, Rep. of	USSR

The member body of the following country expressed disapproval of the document on technical grounds :

Belgium

Liquid flow measurement in open channels — Flat-V weirs

1 Scope and field of application

1.1 This International Standard deals with the measurement of flow in rivers and artificial channels using flat-V weirs under steady or slowly varying flow conditions. The standard flat-V weir is a control structure, the crest of which takes the form of a shallow "V" when viewed in the direction of flow.

1.2 The standard weir is of triangular profile with an upstream slope of 1 (vertical):2 (horizontal) and a downstream slope of 1:5. The cross-slope of the crest line must not be steeper than 1:10. The cross-slope shall be in the range 0 to 1:10 and at the limit, when the cross-slope is zero, the weir becomes a two dimensional triangular profile weir (see ISO 4360).

1.3 The weir can be used in both the modular and drowned ranges of flow. In the modular flow range, discharges depend solely on upstream water levels and a single measurement of upstream head will suffice. In the drowned flow range, discharges depend on both upstream and downstream water levels and two independent head measurements are required. For the standard flat-V weir, these are

a) the upstream head, and

b) the head developed within the separation pocket which forms just downstream of the crest.

1.4 The flat-V weir will measure a wide range of flows and has the advantage of high sensitivity to low flows. Operation in the drowned flow range minimises afflux at very high flows. Flat-V weirs should not be used in steep rivers, particularly where there is a high sediment load.

1.5 Annex A gives the guidelines for the selection of weirs and flumes for the measurement of the discharge of water in open channels.

1.6 There is no specified upper limit for the size of this structure. The following table gives the ranges of discharges for three typical weirs :

Table 1

Elevation of crest above bed m	Crest cross-slope	Width m	Range of discharge m^3/s
0,2	1:10	4	0,015 to 5
0,5	1:20	20	0,030 to 180 (within maximum head of 3 m)
1,0	1:40	80	0,055 to 630 (within maximum head of 3 m)

2 Definitions and symbols

For the purpose of this International Standard, the definitions given in ISO 772[1)] apply. A full list of symbols with the corresponding units of measurement is given in Annex B.

3 Units of measurement

The units of measurement used in this International Standard are SI units.

4 Installation

4.1 Selection of site

4.1.1 The weir shall be located in a straight section of the channel, avoiding local obstructions, roughness or unevenness of the bed.

4.1.2 A preliminary study should be made of the physical and hydraulic features of the proposed site, to check that it conforms (or may be constructed or modified so as to conform) to the requirements necessary for measurement of discharge by the weir. Particular attention should be paid to the following features in selecting the site :

a) The adequacy of the length of channel of regular cross-section available (see 4.2.2.2).

b) The uniformity of the existing velocity distribution (see Annex C).

1) ISO 772, *Liquid flow measurement in open channels — Vocabulary and symbols.*

c) The avoidance of a steep channel (but see 4.2.2.6).

d) The effects of increased upstream water levels due to the measuring structure.

e) The conditions downstream (including such influences as tides, confluences with other streams, sluice gates, mill dams and other controlling features, including seasonal weed growth, which might cause drowning).

f) The impermeability of the ground on which the structure is to be founded and the necessity for piling, grouting or other means of controlling seepage.

g) The necessity for flood banks, to confine the maximum discharge to the channel.

h) The stability of the banks, and the necessity for trimming and/or revetment.

j) Uniformity of section of the approach channel.

k) The effect of wind on the flow over the weir or flume, especially when it is wide and the head is small and when the prevailing wind is in a transverse direction.

4.1.3 If the site does not possess the characteristics necessary for satisfactory measurements, or if an inspection of the stream shows that the velocity distribution in the approach channel deviates appreciably from the examples shown in Annex C, the site should not be used unless suitable improvements are practicable. Alternatively, the performance of the installation should be checked by independent flow measurement.

4.2 Installation conditions

4.2.1 General requirements

4.2.1.1 The complete measuring installation consists of an approach channel, a weir structure and a downstream channel. The condition of each of these three components affects the overall accuracy of the measurements. Installation requirements include such features as the surface finish of the weir, the cross-sectional shape of the channel, channel roughness and the influence of control devices upstream or downstream of the gauging structure.

4.2.1.2 The distribution and direction of velocity may have an important influence on the performance of a weir (see 4.2.2 and Annex C).

4.2.1.3 Once a weir has been installed, any physical changes in the installation will change the discharge characteristics; recalibration will then be necessary.

4.2.2 Approach channel

4.2.2.1 If the flow in the approach channel is disturbed by irregularities in the boundary, for example large boulders or rock outcrops, or by a bend, sluice gate or other feature which causes asymmetry of discharge across the channel, the accuracy of gauging may be significantly affected. The flow in the approach channel should have a symmetrical velocity distribution (see Annex C) and this can most readily be measured by providing a long straight approach channel of uniform cross-section.

4.2.2.2 A length of straight approach channel five times the water-surface width at maximum flow will usually suffice, provided flow does not enter the approach channel with high velocity via a sharp bend or angled sluice gate. However, a greater length of uniform approach channel is desirable if it can readily be provided.

4.2.2.3 The length of uniform approach channel suggested in 4.2.2.2 refers to the distance upstream of the head measuring position. However, in a natural channel it could be uneconomic to line the bed and banks with concrete for this distance, and it could be necessary to provide a contraction in plan if the width between the vertical walls of the lined approach to the weir is less than the width of the natural channel. The unlined channel upstream of the contraction should nevertheless comply with the requirements of 4.2.2.1 and 4.2.2.2.

4.2.2.4 Vertical side walls constructed to effect a narrowing of the natural channel shall be symmetrically disposed with respect to the centre line of the channel and should preferably be curved with a radius not less than 2 H_{max} as shown in figure 1. The tangent point of this radius nearest to the weir shall be at least H_{max} upstream of the head measurement section. The height of the side walls should be chosen to contain the design maximum discharge.

4.2.2.5 In a channel where the flow is free from floating and suspended debris, good approach conditions can also be provided by suitably placed baffles formed from vertical laths, but no baffle should be nearer to the point at which head is measured than 10 times H_{max}.

4.2.2.6 Under certain conditions a hydraulic jump may occur upstream of the measuring structure, for example if the approach channel is steep. Provided this wave is at a distance upstream of not less than about 30 times H_{max}, flow measurement will be feasible, subject to confirmation that an even velocity distribution exists at the gauging station.

4.2.2.7 Conditions in the approach channel can be verified by inspection or measurement for which several methods are available such as current meters, floats, velocity rods, or concentrations of dye, the latter being useful in checking conditions at the bottom of the channel. A complete and quantitative assessment of velocity distribution may be made by means of a current meter. The velocity distribution should then be assessed by reference to Annex C.

4.3 Weir structure

4.3.1 The structure shall be rigid and watertight and capable of withstanding flood flow conditions without damage from outflanking or from downstream erosion. The weir crest should be straight in plan and at right angles to the direction of flow in the upstream channel, and the geometry should conform to the dimensions given in relevant clauses.

The weir must be contained within vertical side walls and the crest width should not exceed the width of the approach channel (see figure 1). Weir blocks may be truncated but not so as to reduce their horizontal dimensions in the direction of flow to less than H_{max} and 2 H_{max} upstream and downstream of the crest line respectively.

4.3.2 The weir and the immediate approach channel (the part with vertical side walls) may be constructed in concrete with a smooth cement finish or surfaced with a smooth non-corrodible material. In laboratory installations, the finish should be equivalent to rolled sheet metal or planed, sanded and painted timber. The surface finish is of particular importance near the crest but may be relaxed a distance along the profile 1/2 H_{max} upstream and downstream of the crest line.

4.3.3 In order to minimize uncertainty in the discharge, the following tolerances should be aimed at during construction :

— On the crest width, 0,2 % of this width with a maximum of 0,01 m.

— On the upstream and downstream slopes, 0,5 %.

— On the crest cross-slope, 0,1 %.

— On point deviations from the mean crest line, 0,05 % of crest width.

Laboratory installations will normally require higher accuracy.

4.3.4 The structure should be measured on completion and average values of relevant dimensions and their standard deviations at 95 % confidence limits computed. The former are used for computation of discharge and the latter are used to obtain the overall uncertainty of a single determination of discharge (see 9.6).

4.4 Downstream of the structure

4.4.1 Conditions downstream of the structure are important in that they control the tailwater level. This level is one of the factors which determines whether modular or drowned flow conditions will occur at the weir. It is essential, therefore, to calculate or observe tailwater levels over the full discharge range and make decisions regarding the type of the weir and its required geometry in the light of this evidence.

5 Maintenance — General requirements

5.1 Maintenance of the measuring structure and the approach channel is important to secure accurate measurements. It is essential that the approach channel should be kept clean and free from silt and vegetation as far as practicable for at least the distance specified in 4.2.2.2. The float-well, and the entry from the approach channel shall also be kept clean and free from deposits.

The weir structure shall be kept clean and free from clinging debris and care shall be taken in the process of cleaning to avoid damage to the weir crest.

6 Measurement of head(s)

6.1 General requirements

6.1.1 Where spot measurements are required, the heads can be measured by vertical gauges, hooks, points, wires or tape gauges. Where continuous records are required, recording gauges should be used. Locations for the head measurements are dealt with in 6.4.1, 6.4.2 and 6.4.3.

6.1.2 As the size of the weir and the head on it reduces, small discrepancies in construction and in the zero setting and reading of the head measuring device become of greater relative importance.

6.2 Gauge wells

6.2.1 It is preferable to measure the upstream head in a gauge well to reduce the effects of water surface irregularities. When this is done, it is also desirable to measure the head in the approach channel as a check from time to time. Where the weir is designed to operate in the drowned flow range, a separate gauge well is required to record the piezometric head developed within the separation pocket which forms immediately downstream of the crest.

6.2.2 Gauge wells should be vertical and of sufficient height and depth to cover the full range of water levels. In field installations they should have a minimum height of 0,3 m above the maximum water levels expected. Gauge wells should be connected to the appropriate head measurement positions by means of pipes.

6.2.3 Both the well and the connecting pipe should be watertight, and where the well is provided for the accommodation of the float of a level recorder, it should be of adequate size and depth to give clearance around and beneath the float at all stages. The float should not be nearer than 0,075 m to the wall of the well.

6.2.4 The pipe should have its invert not less than 0,06 m below the lowest level to be gauged.

6.2.5 The pipe connection to the upstream head measurement position should terminate flush with the boundary of the approach channel and at right angles thereto. The approach channel boundary should be plain and smooth (equivalent to carefully finished concrete) within a distance of 10 times the diameter of the pipe from the centreline of the connection. The pipe may be oblique to the wall only if it is fitted with a removable cap or plate, set flush with the wall, through which a number of holes are drilled. The edges of these holes shall not be rounded or burred. Perforated cover plates are not recommended where weed or silt are likely to be present.

6.2.6 The pipe connection to the measurement position for the separation pocket head should terminate in a manifold set within the crest of the weir. For large field installations, the outlet from this manifold should consist of ten 10 mm diameter holes spaced at 50 mm intervals along a line parallel to, and 20 mm downstream of, the crest line. For laboratory and small field installations (b < 2,5 m), these dimensions should be halved. The holes should be flush with the downstream face of the weir block, preferably in a metal cover plate which can be removed to facilitate maintenance of the system. It is important that this cover plate should have an efficient seal around its perimeter. Locations for the head measurement positions are given in 6.4.

6.2.7 Adequate additional depth should be provided in wells to avoid the danger of floats grounding either on the bottom or on any accumulation of silt or debris. The gauge well arrangement may include an intermediate chamber of similar size and proportions between it and the approach channel to enable silt and other debris to settle out where they may be readily seen and removed.

6.2.8 The diameter of the connecting pipe or width of slot should be sufficient to permit the water level in the well to follow the rise and fall of head without appreciable delay, but on the other hand it should be as small as possible, consistent with ease of maintenance, to damp out oscillations due to short period waves.

6.2.9 No firm rule can be laid down for determining the size of the connecting pipe, because this is dependent on the circumstances of the particular installation, for example, whether the site is exposed and thus subject to waves, and whether a large diameter well is required to house the floats of recorders. It is preferable to make the connection too large rather than too small, because a restriction can easily be added later if short period waves are not adequately damped out. A 100 mm diameter pipe is usually suitable for a flow measurement in the field. A diameter of 3 mm may be appropriate for precision head measurement with steady flows in the laboratory.

6.2.10 Isolating valves with extended spindles shall preferably be fitted to connecting pipes inside the gauge wells so that the wells can be drained or pumped out and cleaned. If possible the well shall be connected to a drainage system via a sludge plug valve and pipe line.

6.3 Zero setting

6.3.1 Accurate initial setting of the zeros of the head measuring devices with reference to the level of the crest, and regular checking of these settings thereafter, is essential if overall accuracy is to be attained.

6.3.2 An accurate means of checking the zero at frequent intervals should be provided. Bench marks, in the form of horizontal metal plates, should be set up on the top of the vertical side walls and in the gauge wells. These should be accurately levelled such that their elevation relative to crest level is known. Instrument zeros can then be checked relative to these bench marks without the necessity of re-surveying the crest each time. Any settlement of the structure may, however, affect the relationships between crest and bench mark levels and hence it is advisable to make occasional checks on these relationships.

6.3.3 A zero check based on the water level (either when the flow ceases or just begins) is liable to serious errors due to surface tension effects and should not be used.

6.3.4 Values for the crest cross-slope m, and the gauge zero can be obtained by measuring the crest elevation at regular intervals along the crest line. A best fit straight line is then fitted through the measured points for each side of the weir and the intersection of these lines is the gauge zero level. The average of the two side slopes is used for m in the discharge formulae. For field installations, the use of standard levelling techniques is recommended but precise micrometer or vernier gauges are required for laboratory installation.

6.4 Location of head measurement sections

6.4.1 The approach flow to a flat-V weir is three-dimensional. Drawdown in the approach to the lowest crest elevation is more pronounced than to other positions across the width of the approach channel and this results in a depression in the water surface immediately upstream of the lowest crest position. Further upstream, this depression is less pronounced and at a distance of 10 times the V-height, 10 h', the water surface elevation across the width of the channel is sensibly constant. Thus, to achieve an accurate assessment of the upstream head, the tapping should be set 10 h' upstream of the crest line. $h' = b/2\,n$ = difference between the lowest and the highest crest elevation in metres. However, if this distance is less than 3 H_{max} the tapping should be set 3 H_{max} upstream of the crest to avoid drawdown effects.

6.4.2 If other considerations necessitate siting the tapping closer to the weir, then corrections to the discharge coefficients will be necessary if $H_1/P_1 > 1$. In all cases an increase in coefficient is applicable and the percentage increase will depend on the tapping point location and the value of H_1/P_1 as follows :

Table 2

L_1	H_1/P_1		
	1	2	3
$10h'$	0,0	0,0	0,0
$8h'$	0,0	0,3	0,6
$6h'$	0,0	0,6	0,9
$4h'$	0,0	0,8	1,2

where

H_1 is the upstream total head relative to lowest crest elevation;

P_1 is the height of lowest crest elevation relative to upstream bed level;

L_1 is the distance of upstream head measurement position from crest line.

6.4.3 Flat-V weirs can be used for gauging purposes in the drowned flow range if a tapping is incorporated at the crest. The centre position of the ten crest tapping holes (see 6.2.6) should be offset laterally from the position of the lowest crest elevation a distance of 0,1 times the total crest width, see figure 1.

7 Discharge relationships

7.1 Equations of discharge

7.1.1 In terms of total head the basic discharge equation for a flat-V operating under modular flow conditions is :

$$Q = 0{,}8\, C_{De} \sqrt{g}\, m\, Z_H\, H_{le}^{5/2} \quad \ldots (1)$$

where

Q is the total discharge;

C_{De} is the effective coefficient of discharge;

g is the acceleration due to gravity;

m is the crest cross-slope (1 vertical/m horizontal);

Z_H is the shape factor;

H_{le} is the effective upstream total head relative to lowest crest elevation.

Alternatively, the discharge equation may be expressed in terms of gauged head by introducing a coefficient of velocity dependent upon the weir and flow geometries.

$$Q = 0{,}8\, C_{De}\, C_v \sqrt{g}\, m\, Z_h\, h_{le}^{5/2} \quad \ldots (2)$$

where

C_v is the coefficient of velocity;

Z_h is the shape factor;

h_{le} is the effective upstream gauged head relative to lowest crest elevation.

7.1.2 In terms of total head, the basic discharge equation for a flat-V weir operating under drowned flow conditions is :

$$Q = 0{,}8\, C_{De} f_v \sqrt{g}\, m\, Z_H\, H_{le}^{5/2} \quad \ldots (3)$$

where f_v is the drowned flow reduction factor.

The corresponding gauged head equation is :

$$Q = 0{,}8\, C_{De}\, C_v f_v \sqrt{g}\, m\, Z_h\, h_{le}^{5/2} \quad \ldots (4)$$

Values for the modular coefficient of discharge, C_{De}, are given in table 5.

7.2 Effective heads

7.2.1 Effective heads are obtained by reducing observed values by a small constant amount which corrects for fluid property effects. Thus :

$$h_{le} = h_1 - k_h \quad \ldots (5)$$

$$\text{and } H_{le} = H_1 - k_h = h_1 + \frac{\alpha v^2}{2g} - k_h \quad \ldots (6)$$

Values for the head correction factor, k_h, are given in table 5.

The value of the Coriolis energy coefficient, α, should be checked on site by measuring the velocity distribution at the section where the head is measured. At the design stage, the value of α should be taken as 1,2.

7.3 Shape factors

7.3.1 Shape factors are introduced into discharge equations for flat-V because the geometry of flow changes when the discharge exceeds the V-full condition. Thus :

when $h_1 \leqslant h'$

$$Z_h = Z_H = 1{,}0 \quad \ldots (7)$$

when $h_1 > h'$

$$Z_h = [1 - (1 - h'/h_{le})^{5/2}] \quad \ldots (8)$$

$$\text{and } Z_H = [1 - (1 - h'/H_{le})^{5/2}] \quad \ldots (9)$$

where

$h' = b/2m$ = difference between lowest and highest crest elevations;

b is the crest width in metres.

Values of Z_h, and Z_H in terms of h_{le}/h', and H_{le}/h' are given in table 6.

7.4 Coefficient of velocity

7.4.1 The coefficient of velocity, C_v, is related to the modular coefficient of discharge, C_{De}, the ratio h'/P_1 and the ratio h_{le}/h'.

7.4.2 The coefficient of velocity, C_v, occurs in equations (2) and (4) together with the shape factor, Z_h. As indicated in 7.3.1, this shape factor is a function of h_{le}/h', one of the factors affecting C_v. Thus it is convenient to present data for the product $C_v Z_h$ in terms of h/P_1 and h_{le}/h' since C_v and Z_h are not required separately. Numerical values of this product are given in table 7.

7.5 Conditions for modular/drowned flow

7.5.1 The modular limit for flat-V weirs is not single valued as in the case of a two-dimensional weir, i.e. a weir with a horizontal crest line. In the case of the flat-V weir, the modular limit is 70 ± 5 % depending on the ratio H_{le}/h'.

7.6 Drowned flow reduction factor

7.6.1 The drowned flow reduction factor, f_v, can for practical purposes be related to the head ratio h_{pe}/H_{le}. The functional relationship is given by :

$$f_v = 1{,}078\, [0{,}909 - (h_{pe}/H_{le})^{3/2}]^{0{,}183} \quad \ldots (10)$$

where $h_{pe} = h_p - k_h$ = effective separation pocket head relative to lowest crest elevation in metres.

Numerical values obtained from the above expression are given in table 8.

7.6.2 In equation (10), the drowned flow reduction factor is related to the ratio h_{pe}/H_{le}, i.e. an expression involving total head. If equation (4) is to be used to compute discharge, f_v must be related to gauged heads. A convenient way of doing this is given in tables 9 to 13 where the product $C_v f_v$ is given in terms of h_{le}/h' and h_{pe}/h_{le}. Each table corresponds to a different range of the ratio h'/P_1, as follows :

Table 9 : $0{,}0 \leqslant h'/P_1 \leqslant 0{,}5$

Table 10 : $0{,}5 < h'/P_1 \leqslant 1{,}0$

Table 11 : $1{,}0 < h'/P_1 \leqslant 1{,}5$

Table 12 : $1{,}5 < h'/P_1 \leqslant 2{,}0$

Table 13 : $2{,}0 < h'/P_1 \leqslant 2{,}5$

7.7 Limits of application

7.7.1 The practical lower limit of upstream head is related to the magnitude of the influence of fluid properties and boundary roughness. For a well-maintained weir with a smooth crest section, the minimum head recommended is 0,03 m. If the crest is of smooth concrete or a material of similar texture, a lower limit of 0,06 m is suggested.

7.7.2 There is also a limiting value for the ratios h'/P_1 of 2,5 and there are limitations on H_1/P_2 as shown in table 5. These are governed by the scope of experimental verification and vary with cross-slope.

P_2 = elevation of the lowest crest elevation relative to the downstream bed level in metres.

8 Computation of discharge

8.1 General

8.1.1 There are two common methods of computing discharges from gauged head readings. The first obtains results by successive approximation techniques and utilises the basic "total head" equations. This method is particularly suited to solutions by computer techniques since the computer provides an efficient way of carrying out the repetitive calculations involved. The second method utilises relationships which can be derived between gauged and total heads for particular weir and flow geometries. These enable the coefficient of velocity, C_v, in the discharge equation to be assessed from tables or graphs.

8.2 Successive approximations method

8.2.1 Computation using individual head measurements

The method of successive approximations is a well known approach to the conversion of gauged heads to total heads. It is applicable to any weir geometry, and can be used in both modular and drowned flow conditions.

The method is as follows :

a) Use equation (1) if the flow is modular and equation (3) if the weir is drowned.

b) Determine the coefficient of discharge, C_{De}, and the head correction factor, k_h, using table 5.

c) Compute the effective gauged head, $h_{le} = h_1 - k_h$.

d) Determine the value of $K_1 = 0{,}8\, C_{De}\sqrt{g}\, m$.

Hence $Q = K_1 Z_H H_{le}^{5/2}$ for modular flow

and $Q = K_1 f_v Z_H H_{le}^{5/2}$ for drowned flow.

e) Determine the cross-sectional area of flow, $A = B(h_1 + P_1)$, and hence the velocity head in terms of discharge

$$\frac{\alpha v^2}{2g} = \frac{\alpha Q^2}{2gA^2} = K_2 Q^2 \qquad \ldots (11)$$

where B is the width at upstream gauging section in metres.

f) Assume, as a first approximation, that $h_{le} = H_{le}$ and compute the discharge. In this step, the value of Z_H is obtained from table 6 and, for drowned flow, the value of f_v is obtained from table 8.

g) Use this approximate discharge to determine the velocity head and then use these data to calculate an improved value of the total head at the gauging section.

h) Compute a more refined discharge value using this total head value.

j) Repeat steps g) and h) until the difference between successive discharge values is an order of magnitude less than the required uncertainty.

The above method enables discharges to be calculated from individual gauged head readings.

8.2.2 Computation of modular stage-discharge function

The previous method does not provide the quickest way of computing a modular stage-discharge graph for a particular weir installation where such a plot is required. A more concise method of obtaining the theoretical calibration curve is to calculate, first of all, the relationship between total head and discharge and then to convert total head to gauged head. This conversion will normally require less loops of the successive approximation cycle than in the first method described in 8.2.1.

The principle of the method is as follows :

a) Using the total head equation (1), calculate a series of values of Q for a series of assumed values of H_{le}; Z_H is obtained from

$$Z_H = [1 - (1 - h'/H_{le})^{5/2}] \qquad \ldots (12)$$

C_{De} and k_h are obtained from table 5, and Z_H can be read from table 6.

b) The next step is to convert the series of total head values H_{le} to the corresponding gauged head values.

c) Making as a first approximation the assumption that the upstream water level is at the elevation given by the total head, deduce the cross-sectional area of the approach channel, and hence work out the velocity of approach. An approximate value of the gauged head is deduced from

$$h_1 = H_{le} - \alpha v^2/2g + k_h \qquad \ldots (13)$$

d) This provides an improved estimate of water level, which is used to revise the original value for the approach velocity, and hence to obtain a further value of the gauged head h_1. This procedure is repeated until the difference between successive estimates of the gauged head is an order of magnitude less than the required uncertainty.

e) Steps c) and d) are repeated for each pair of values of H_{le} and Q, thus providing a complete modular stage-discharge curve for the structure.

8.3 Coefficient of velocity method

8.3.1 Modular flow conditions

Equation (2) is used in this method of computing discharge from the known quantities, P_1, m, h' and h_1. The method is as follows :

a) Calculate $h_{le} = h_1 - k_h$ using the appropriate value of k_h from table 5.

b) Note the appropriate value of C_{De} from table 5.

c) Calculate the ratios h'/P_1 and h_{le}/h'. Look up the appropriate value of $C_v Z_h$ from table 7.

d) Calculate $h_{le}^{5/2}$.

e) Substitution of known values in equation (2) then gives the discharge directly.

8.3.2 Drowned flow conditions

Equation (4) is used in this method of computing discharge from known values of P_1, m, h', h_1 and h_p. The calculation proceeds as follows :

a) Calculate $h_{le} = h_1 - k_h$ and $h_{pe} = h_p - k_h$ using table 5 for the value of k_h.

b) Note the appropriate value of C_{De} from table 5.

c) Evaluate h_{pe}/h_{le}, h_{le}/h' and h'/P_1. Read off the appropriate value of $C_v f_v$ from whichever of tables 9 to 13 is applicable to the calculated value of h'/P_1.

d) Determine the value of Z_h using table 6;

e) Calculate $h_{le}^{5/2}$.

f) Substitution of known values in equation (4) then determines the discharge.

Examples of these computational methods are given in clause 10.

8.4 Accuracy

8.4.1 The overall accuracy of measurement will depend on :

a) the accuracy of construction and finish of the weir;

b) the accuracy of the head measurements;

c) the accuracy of other measured dimensions;

d) the accuracy of the coefficient values;

e) the accuracy of the form of the discharge equations.

The method by which estimates of these constituent uncertainties may be combined to give the overall uncertainty in computed discharges is given in clause 9.

8.4.2 The uncertainties (95 % confidence limits) on modular discharge coefficients are given in table 5. These reflect the random and systematic errors which occur in calibration experiments and also the real but marginal changes in coefficient values which occur with changing discharge.

9 Errors in flow measurement

9.1 General

9.1.1 The uncertainty of any flow measurement can be estimated if the uncertainties from various sources are combined. These contributions to the total uncertainty may be assessed and will indicate whether the rate of flow can be measured with sufficient accuracy for the purpose in hand. This clause provides sufficient information for estimating the uncertainties of measurements of discharge.

9.1.2 The error in a result is the difference between the true rate of flow and that calculated using the discharge equations quoted in this International Standard. Thus the error is, by definition, unknown but the uncertainty of the measurement may be estimated. The term uncertainty denotes the deviation from the true rate of flow within which the measured rate of flow is expected to lie some nineteen times out of twenty (the 95 % confidence limits).

9.2 Sources of error

9.2.1 The sources of error in the discharge measurement can be identified by considering the form of equation (4) i.e.

$$Q = J\, C_{De}\, C_v f_v \sqrt{g}\, m\, Z_h\, h_{le}^{5/2} \qquad \ldots (14)$$

where J is a numerical constant not subject to error.

Errors in g, the acceleration due to gravity, may be ignored. Hence the only sources of error which need to be considered are :

a) The modular discharge coefficient C_{De} for which numerical values and estimates of uncertainties are given in table 5.

b) The coefficient of velocity, C_v. The following approximate expression may be used to determine the uncertainty in C_v :

$$X_{C_v} = 0{,}5\, h_1/P_1\ (\%) \qquad \dots (15)$$

c) The crest cross-slope, m. Numerical values will depend on the accuracy of construction and subsequent measurement of the structure.

d) The shape factor, Z_h. When flow is confined within the V, the value of Z_h is unity and there is no uncertainty in this value. When flow is above the V-full condition, the value of Z_h depends on h', the height of the V, and h_{le} the upstream gauged head; see equation (8). Both these quantities will be of reasonable magnitude and errors in heads will not normally be significant at this stage. Thus there is again a negligible degree of uncertainty in Z_h i.e. $X_{Z_h} = 0$.

e) The effective head, h_{le}. The uncertainty in h_{le} will depend on uncertainties in head measurement, zeroing of the gauge, head correction factors and uncertainties associated with the number of readings. Thus

$$X_{h_{le}} = \pm \frac{100\sqrt{eh_1^2 + eh_{le}^2 + ek_h^2 + (2S_{\bar{h}_1})^2}}{h_1} \qquad \dots (16)$$

where

eh_1 is the uncertainty in the measurement of upstream head in metres. Any uncertainty which does not change randomly during a series of measurements should be included here for example backlash and friction;

eh_{le} is the uncertainty in the determination of the gauge zero;

ek_h is the uncertainty in k_h;

$2S_{\bar{h}_1}$ is the uncertainty in the mean of n readings of upstream head, see 9.3. It is associated with the random fluctuations in a series of measurements.

The uncertainty in the head correction factor, k_h, should be taken as ek_h = 0,2 mm. The uncertainties eh_1 and eh_{le} must depend on an assessment, of probable uncertainties by the user.

f) The separation pocked head, h_{pe}. The uncertainties in h_{pe} will depend on uncertainties in head measurement, zeroing of the gauge, head correction factors and uncertainties associated with the number of readings. Thus

$$X_{h_{pe}} = \pm \frac{100\sqrt{eh_p^2 + eh_{pe}^2 + ek_h^2 + (2S_{\bar{h}_p})^2}}{h_p} \qquad \dots (17)$$

where

eh_p is the uncertainty in the measurement of the separation pocket head. The sources of errors are systematic, for example backlash and friction;

eh_{pe} is the uncertainty in the determination of the gauge zero;

$2S_{\bar{h}_p}$ is the uncertainty in the mean of n readings of the separation pocket head (see 9.3). It is associated with the random fluctuations in a series of measurements.

The uncertainties eh_p and eh_{pe} must depend on an estimate, by the user of probable errors. If only one reading of head is made, then the random uncertainties, $2S_{\bar{h}_l}$ or $2S_{\bar{h}_p}$, must be estimated (see 9.5.4).

g) The drowned flow reduction factor, f_v. There are three factors which influence the uncertainty in f_v :

a) the uncertainties in the laboratory determination of the f_v versus h_{pe}/H_{le} relationship;

b) the uncertainties in the measurement of the upstream effective head, h_{le};

c) the uncertainties in the measurement of the separation pocket head, h_{pe}.

A suitable expression for the combined uncertainty is :

$$X_{f_v} = \pm\, 5\,(1 - f_v)\sqrt{1 + X_{h_{le}}^2 + X_{h_{pe}}^2} \qquad \dots (18)$$

9.3 Kinds of error

9.3.1 Errors can be classified as random or systematic, the former affecting the reproducibility of measurement and the latter affecting its true accuracy. The standard deviation of a set of n measurements of a variable Y may be estimated from the equation

$$S_Y = \left[\frac{\sum_{i=1}^{n} (\bar{Y} - Y)^2}{n - 1}\right]^{1/2} \qquad \dots (19)$$

where $\bar{Y}$ is the observed mean. The standard deviation of the mean is then given by

$$S_{\bar{Y}} = \frac{S_Y}{\sqrt{n}} \qquad \dots (20)$$

and the uncertainty of the mean is twice $S_{\bar{Y}}$ (for 95 % probability) if the number of measurements, n, is large.

9.3.2 A measurement can also be subject to systematic error : the mean of a large number of measured values would thus still differ from the true value of the quantity being measured. An error in a gauge zero, for example, will produce a systematic error. As repetition of the measurement does not eliminate systematic errors, the actual value could only be determined by an independent measurement known to be more accurate.

9.4 Errors in quantities given in this International Standard

9.4.1 All the errors in this category are systematic. The values of the discharge coefficients, etc., quoted in this International Standard are based on an appraisal of experiments, carefully carried out with sufficient repetition of readings.

9.4.2 However, when measurements are made on other similar installations, systematic discrepancies between coefficients of discharge may occur, due to variations in the surface finish of the device, its installation, the approach flow conditions, etc.

9.4.3 The probable uncertainties in the coefficients and the corrective term k_h quoted in previous clauses of this International Standard are based on a consideration of the deviation of experimental data from the given working equations and a comparison of the equations themselves.

9.5 Errors in quantities measured by the user

9.5.1 Both random and systematic errors will occur in measurements in this category.

9.5.2 Since neither the methods of measurement nor the way in which they are to be made are specified, no numerical values for uncertainties in this category can be given.

9.5.3 The uncertainty in the gauged head should be determined from an assessment of the separate sources of uncertainty, for example the gauge sensitivity, the zero setting uncertainty, temperature effects, the backlash in the indicating mechanism, the residual random uncertainty in the mean of a series of measurements, etc.

9.5.4 The above component uncertainties should be calculated as percentage standard deviations at the 95 % confidence limits but when the value of the component uncertainty is determined from only a single measurement, the uncertainty is said to be rectangularly distributed and may be taken, for the purposes of this International Standard, to be the (plus or minus) limits within which the true value is known to lie with certainty (i.e. half the estimated maximum deviation).

9.6 Combination of uncertainties to give the overall uncertainty in discharge

9.6.1 The uncertainty in discharge is given by the expression

$$X_Q = \pm \sqrt{X_{C_{\mathrm{De}}}^2 + X_{C_v}^2 + X_{f_v}^2 + X_m^2 + 6{,}25\,X_{h_{\mathrm{le}}}^2} \qquad \ldots (21)$$

where X_Q is the uncertainty in computed discharge (per cent).

9.6.2 It should be noted that the uncertainty in discharge is not single valued for a given device, but will vary with flow. It may therefore be necessary to consider the uncertainty at several discharges covering the required range of measurement.

10 Examples

10.1 Modular flow at low discharge ($h_{\mathrm{le}} < h'$)

10.1.1 A flat-V weir has a crest cross-slope of 1/20,30. The crest width and approach channel width are both 36,00 m and the mean upstream bed level is 0,82 m below the lowest crest elevation.

Calculate the discharge when the observed upstream gauged head is 0,621 m. Ten successive readings of this head produce a standard deviation of the mean of 0,5 mm and the estimated uncertainty in the gauge zero is 1,0 mm. The basic measurements with their estimated uncertainties are given below :

$m = 20{,}30 \quad (\pm\ 0{,}2\ \%)$

$b = 36{,}00\ \mathrm{m}\ (\pm\ 0{,}005\ \mathrm{m})$

$P_1 = 0{,}82\ \mathrm{m}\ (\pm\ 0{,}001\ \mathrm{m})$

$h_1 = 0{,}621\ \mathrm{m}\ (\pm\ 0{,}003\ \mathrm{m})$

$h' = 0{,}887\ \mathrm{m}\ (\pm\ 0{,}001\ \mathrm{m})$

The appropriate coefficient and head correction values are obtained from table 5, as follows :

$C_{\mathrm{De}} = 0{,}620 \quad (\pm\ 3{,}2\ \%)$

$k_h = 0{,}000\,5\ \mathrm{m}\ (\pm\ 0{,}000\,2\ \mathrm{m})$

10.1.2 Solution by successive approximation method (see 8.2)

The appropriate discharge equation is :

$$Q = 0{,}8\, C_{\mathrm{De}} \sqrt{g}\, m Z_H H_{\mathrm{le}}^{5/2} \qquad \ldots (22)$$

a) $h_{\mathrm{le}} = h_{\mathrm{l}} - k_h = 0{,}620\,5$ m.

b) $h_{\mathrm{le}} < h'$, hence flow is confined within the V and $Z_H = 1{,}000$ from table 6.

c) $0{,}8\, C_{\mathrm{De}} \sqrt{g}\, m = 31{,}54\ \mathrm{m}^{1/2}/\mathrm{s}$.

Hence $Q = 31{,}54\, H_{\mathrm{le}}^{5/2}$ (m^3/s). ... (23)

d) The area of cross-section, $B(h_1 + P_1)$, is 51,88 m^2.

Hence, assuming $\alpha = 1{,}2$, the velocity head is given by :

$$\frac{\alpha v^2}{2g} = \frac{\alpha Q^2}{2gB^2(h_1 + P_1)^2} = \frac{Q^2}{44\,000} \qquad \ldots (24)$$

From these basic values, the calculation of discharge proceeds as shown in table 3.

Table 3

	$\frac{\alpha v^2}{2g}$ [From equation (24)] m	H_{le} $H_{\mathrm{le}} = h_{\mathrm{le}} + \frac{\alpha v^2}{2g}$ m	Q [From equation (23)] m^3/s
First approximation	0,000 0	0,620 5	9,57
Second approximation	0,002 1	0,622 7	9,65
Third approximation	0,002 1	0,622 7	9,65

Hence the discharge is 9,65 m^3/s.