WORLD METEOROLOGICAL ORGANIZATION

TECHNICAL NOTE No. 117

USE OF WEIRS AND FLUMES IN STREAM GAUGING

Report of the Commission for Hydrology

WMO-No. 280

Secretariat of the World Meteorological Organization – Geneva – Switzerland

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NOTE

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FOREWORD

At its third session (Geneva, 1968) the WMO Commission for Hydrometeorology (now the Commission for Hydrology) established a working group to prepare, *inter alia,* a Technical Note on weirs, flumes and other stream-gauging structures. The members of the working group were Mr. R. W. Carter (U.S.A.) (Chairman), Mr. R. W. Herschy (U.K.), Mr. H. Jansen (Federal Republic of Germany), Mr. V. P. Saban (U.S.S.R.), Mr. J. Stachy (Poland) and Mr. P. W. Strilaeff (Canada).

The present Technical Note has accordingly been prepared by the working group. It is hoped that the publication will provide very useful and practical guidance to all services and individuals taking regular hydrometric measurements with weirs and flumes, and especially for all those who intend to establish such structures within their hydrological networks.

It is with great pleasure that I express the gratitude of WMO to Mr. R. W. Carter and to the other members of the working group for the time and effort they have devoted to the preparation of this Technical Note.

D. A. Davies
Secretary-General

SUMMARY

The aim of this Technical Note is to supply the reader with technical information on flow-measuring methods utilizing artificial control sections of such a shape that head-discharge relationships can be determined from measured water levels without the necessity of calibration – i.e., by application of a discharge formula.

Chapter 1 provides general guidelines on the use and installation of such artificial controls. Detailed description, basic equations, design and relevant figures and tables are given for different types of thin-plate weirs, for broad-crested and other long-base weirs, and for standing-wave flumes in Chapters 2, 3, and 4 respectively.

RESUME

Cette Note technique a pour but de présenter au lecteur des renseignements techniques sur les méthodes de mesure des débits dans lesquelles on utilise des sections de contrôle artificielles ayant une forme qui permet de déterminer les relations charge-débit à partir de mesures du niveau de l'eau sans devoir procéder au tarage, c'est-à-dire en appliquant une formule de calcul des débits.

Le chapitre 1 donne des directives générales sur l'utilisation et l'installation de ces sections de contrôle artificielles. Puis la Note décrit en détail, respectivement dans les chapitres 2, 3 et 4, les différents types de déversoirs en mince paroi, les déversoirs à large seuil et à longue base et les canaux à ondes stationnaires, et indique les équations fondamentales et les principes de construction à appliquer, tous ces renseignements étant accompagnés de figures et de tableaux.

РЕЗЮМЕ

Целью данной технической записки является ознакомление читателя с технической информацией относительно методов измерения потока, использующих искусственные контрольные гидрометрические створы такой формы, которая позволяет определить зависимость между напором и расходом воды из измеренных уровней воды без необходимости проведения калибровки, т. е. применяя формулу расхода воды.

В главе 1 даются общие руководящие указания по использованию и установке подобных искусственных контрольных гидрометрических створов. Подробное описание, основные уравнения, расчет и соответствующие рисунки и таблицы для различных типов водосливов с тонкой стенкой, для водосливов с широким порогом и других водосливов с широким основанием и для лотков стоячей волны даются в главах 2, 3 и 4 соответственно.

RESUMEN

Esta Nota Técnica tiene por objeto facilitar al lector información técnica sobre los métodos de medida del caudal que utilizan secciones de control artificiales, cuya forma permite determinar la relación existente entre la carga y el caudal, a partir de medidas del nivel del agua, sin tener que recurrir a una calibración, es decir aplicando une fórmula de cálculo del caudal.

En el capítulo 1 se indican las directrices generales que deben observarse para la utilización e instalación de dichas secciones de control. En los capítulos 2, 3 y 4 figuran, respectivamente, descripciones detalladas de los diversos tipos de vertederos de pared fina, los de pared gruesa y otros de base amplia, los conductos abiertos de onda estacionaria, así como las ecuaciones fundamentales y los principios de construcción que deben aplicarse, quedando ilustrado todo ello mediante las correspondientes figuras y cuadros.

1. GENERAL

1.1. INTRODUCTION

1.1.1. These notes deal with flow measuring methods utilizing artificial control sections of such shape that head-discharge relationships can be determined from measured water levels without the necessity for calibration, ie by application of a discharge formula.

1.1.2. There exists a limited range of different weirs and flumes that have well-established relationships between head and discharge, and the most commonly used of these are described in the following sections. Only under reasonably favourable field conditions can the established formulae accurately predict the discharge, however. It is important therefore that, if a measuring structure is required to measure flow directly from water level readings, every care must be taken in the construction and operation of the device, and the most suitable formula be used. There is no substitute, however, for in-place calibration for all devices discussed here.

1.1.3. Where conditions other than those specified here are encountered, in-place calibration is necessary to establish the extent of departure from the standard formula or to develop the head discharge function if the gauging structure is not one of those covered in this note.

1.2. SCOPE

1.2.1. The measuring devices to be discussed here may be catalogued into three groups:

a. The various types of thin-plate weirs are generally employed in the hydraulics laboratory and on small, clear-flowing streams; particularly where above average accuracy is desired and adequate maintenance can be provided, such as for small research watersheds.

b. On small streams and canals conveying sediment and debris and in other situations where the head loss associated with a thin-plate weir is unacceptable, flumes are preferable. Certain types may also be used drowned, permitting operation with very small head loss but at some loss of accuracy.

c. On larger streams, various types of broad-crested, triangular profile, and round-shaped weirs are frequently employed. On larger streams it is usually impractical as well as unwise to place such structures excessive in height above the normal streambed. When submerged flows may exist over some part of the discharge range, the discharge relationships presented may not apply as the discharge would cease to be a function solely of upstream level. In case of doubt, such structure should be field rated. The triangular profile weir may be operated successfully when drowned, with double gauging of water levels.

1.3. SELECTION OF STRUCTURE

1.3.1. The criteria which may be considered in choosing a structure are:

a. Range of discharge
b. Accuracy
c. Head loss
d. Cost

1.3.2. The range of discharge to be gauged when related to the range of head will decide whether the structure should have a rectangular flow cross-section at the throat or control section. Such a type is suitable if:

$$\frac{Q_{max}}{Q_{min}} < \left(\frac{H_{max}}{H_{min}}\right)^{3/2}$$

where Q_{max} and Q_{min} are the maximum and minimum discharges to be measured, and H_{max} and H_{min} are the maximum and minimum allowable total heads on the structure. Otherwise a structure of Vee-shaped flow section, such as a Vee-notch or trapezoidal-throated flume, or alternatively a compound weir with crests at different levels, will be necessary.

1.3.3. A structure must measure flow sufficiently accurately over the whole range of flow it is desired to gauge. Since the basic calibrations of the structures discussed are generally better than about ±2 per cent, the overall accuracy of measurement will depend principally on the accuracies of construction and measurement of head, and on the sensitivity of the structure to flow conditions in the approach. The accuracy of construction can of course be carefully controlled and the effects of errors in head measurement can be minimized by designing the structure so as to avoid measurement of small heads, say less than about 0.08 m. But the effect of flow conditions in the approach has to be considered more fully.

1.3.4. Special problems are posed in the design of suitable structures for channels with steep gradients (38, 39); errors in calibration can arise due to fluctuations in bed level and the presence of standing waves in the approach channel, and also due to the failure to assess accurately the energy coefficient, α, at the level gauging section. From this point of view, rivers and other open channels have been classified (40) into three categories: those of gentle slope with Froude numbers less than 0.25, ie slopes less than about 1:1000, in which the calibrations of gauging structures are unlikely to be affected by the factors mentioned above; those of moderate slope, with Froude numbers between 0.25 and 0.5 (slopes between about 1:1000 and 1:250) in which small, though significant errors can arise; and those of steep slope with Froude numbers greater than 0.5 (slopes greater than about 1:250) in which large gauging errors occur. For channels with Froude numbers greater than about 0.6 accurate gauging can only be achieved by constructing an entirely different form of structure from the conventional weir or flume, such as that described in reference (39). This special problem, however, is outside the scope of this guide.

1.3.5. By careful attention to design, the errors occurring in channels of moderate slope can be reduced (39). In particular for a channel carrying an appreciable sediment load it may be preferable to construct a flume rather than a weir for flow measurement: a flume with side contractions only can maintain a nearly constant Froude number in the approach over a wide range of flow. By designing the flume so that this Froude number is equal to that previously existing at the site the fluctuation in bed level will be minimized.

1.3.6. In many cases it is necessary to reduce to a minimum the head loss across a structure at maximum discharge. In an irrigation system water levels may have to be maintained as high as possible so that the system can command the maximum area of land. At a new structure in a river the level of the banks may require that the water be backed up as little as possible above downstream levels. Alternatively the possibility of scour downstream of a structure may demand that the energy to be dissipated be minimized; in this connection it is worth pointing out that flow over a weir is two-dimensional with a uniform discharge intensity across the width, making energy dissipation downstream easier than in the case of a flume.

1.3.7. A basis for comparing the head loss at various structures was described in a recent paper (41). This showed that for the weirs and rectangular throated flumes considered, the lowest head losses were achieved at submerged Crump weirs and at round-nose horizontal crest weirs with sloping back faces operating at the submergence limit. The head loss across flumes, including submerged Parshall flumes, is greater than the weirs mentioned above under comparable flow conditions.

1.3.8. Cost will often be the principal criterion in making a choice, particularly in the cases where several types of structure would be suitable on the other grounds. The cost consists principally of the capital cost of the construction and of the ancillary equipment such as water level recorders. The construction cost is closely related to the size of the structure; the paper cited (41) compares the relative volumes of different types of structure for given hydraulic conditions while bearing in mind the other considerations of accuracy and head loss mentioned above. At the same time the operating cost of a structure, which is made up of the costs of maintenance and of processing the data, could influence the choice to be made. For example, the size of a submerged structure may be much smaller than a free-flowing structure; however it will require two water level recorders and two stilling wells instead of one of each, and the computation of discharge will be more complicated and hence more costly. The additional costs of computation and of maintenance of the second recorder and well, plus their cost of installation, may in some circumstances more than offset the savings achieved in the size of the structure.

1.3.9. If choice is restricted to devices having a rectangular flow section, other than a thin-plate weir, the structures having the smallest volume of construction are as follows, for various ranges of F_d, the Froude number of the flow in the channel downstream:

$F_d < 0.2$. The rectangular profile weir (in the variable coefficient range) followed by the round-nose horizontal crest and truncated Crump weirs.

$0.2 < F_d < 0.4$. The drowned Crump weir, followed by the round-nose horizontal crest and truncated Crump weirs.

$0.4 < F_d < 0.6$. The round-nose horizontal crest weir followed by the truncated Crump weir and the rectangular-throated standing wave flume.

1.4. INSTALLATION CONDITIONS

1.4.1. The complete measuring installation consists of an approach channel, gauging 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, cross-sectional shape of channel, channel roughness and the influence of control devices upstream or downstream of the gauging structure.

1.4.2. Maintenance of the measuring structure and the approach channel is important to secure accurate continuous measurements. The approach channels to weirs should be kept clean and free from accumulations of silt and vegetation. The structure must be kept clean and free from clinging debris and ice and care shall be taken in the process of cleaning to avoid damage to thin-plate and other weirs with vulnerable corners. The stilling well and the connecting pipe or slot from the approach channel must also be kept clean and free from deposits and ice.

1.4.3. The weir should be located in a straight reach, which should be well maintained to avoid local obstructions, roughness or unevenness of the bed. A preliminary survey should be made of the physical and hydraulic features of the proposed site, to check that it conforms (or may be made to conform) to the requirements necessary for measurement by the structure. 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.
b. The regularity of the existing velocity distribution.
c. The avoidance of a steep channel (but see 1.4.8.).
d. The effects of any 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 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 sealing-in of river installations.
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 in natural channels.
i. The clearance of rocks or boulders from the bed of the approach channel.
j. Effect of wind. Wind can have a considerable effect on the flow over a weir, especially when it is wide and the head is small and when the prevailing wind is in a transverse direction.

1.4.4. On all installations the flow in the approach channel shall be smooth, free from disturbance and have an even velocity distribution. Conditions in the approach channel can usually be verified by inspection or measurement for which several methods are available, such as velocity rods,floats 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.

1.4.5. An even velocity distribution can be obtained by having a long straight approach channel free from projections either at the side or on the bottom. The cross-section should be reasonably uniform and the channel straight for a length equal to at least ten times the width of the weir. In an artificial channel where no debris or matter is carried in suspension, suitable flow conditions can often be provided by suitably placed baffles formed by vertical laths, but there should be no baffle nearer to the point of measurement than ten times the maximum head to be measured.

1.4.6. If the entry to the approach channel is through a bend or if the flow is discharged into the channel through a conduit of smaller cross-section, or at an angle, then a longer length of straight approach channel may be required to achieve an even velocity distribution.

1.4.7. In a natural channel, it would be uneconomical to line the bed and banks of the stream with concrete for the distance given in 1.4.5. A contraction in plan will be required if the width of the gauging structure is less than the width of the stream. Wing walls to effect this contraction should be symmetrically disposed with respect to the centre-line of the stream and should be gently curved at their upstream ends, for example as shown in Figures 3.1, 3.3 and 3.7. They should be plane, straight and parallel through to at least a distance H_{max} upstream of the head measurement section, the curved portion upstream of that point being to a radius of not less than $2H_{max}$.

1.4.8. Under certain conditions, a standing wave 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 30 times the maximum head, flow measurement will be feasible, subject to confirmation that a regular velocity distribution exists at the gauging station. If a standing wave occurs within this distance the approach conditions must be modified eg by altering the weir characteristics, if measurement errors are to be avoided.

1.4.9. When the velocity distribution in the approach channel differs considerably from the normal, the discharge characteristics are altered. Consequently, flow measurements made with non-standard weir installations are subject to error. An indication of the magnitude of error associated with a variety of nonuniform velocity distributions can be obtained from ref 1. An approximate method of compensating for the error has been suggested (ref 2) but the procedure requires preliminary measurement of velocities in the approach channel in order to establish certain velocity-distribution characteristics of the installation.

1.4.10. Although any deviation from the ideal conditions of either very uniform velocity or a normal velocity distribution may lead to errors in flow measurement, quantitive information on the influence of velocity distribution is inadequate to define the acceptable limits of departure from the ideal distributions. However, Figure 1.1 provides some guidance to the type of velocity distribution and evenness thereof that are acceptable in practice (ref 3). The isovels plotted in Figures 1.1 (e) and (f) provide examples of observed normal velocity distributions, which are clearly acceptable. Figure 1.1 (a) shows some skewness, but nevertheless approximates to a normal distribution. Figures 1.1 (b) and 1.1 (c) show appreciable departure from uniformity, and are considered representative of the maximum acceptable departure from ideal approach conditions for the tolerances given.

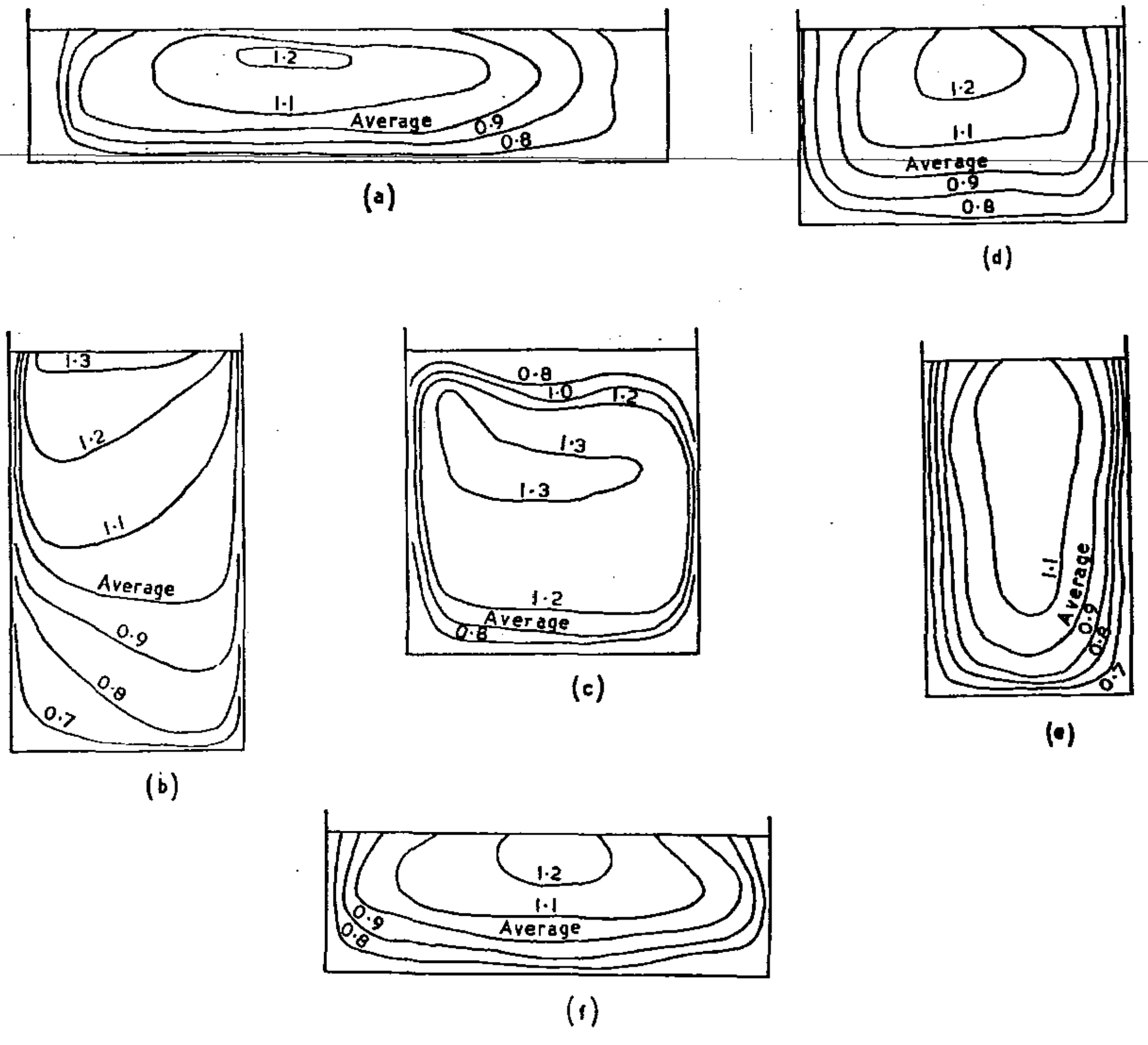


FIGURE 1.1. EXAMPLES OF VELOCITY PROFILES IN THE APPROACH CHANNEL

1.4.11. The flow conditions downstream of the structure are important only in that they control the tail water level which may influence operation. The altered flow conditions due to the construction of the weir or flume might cause shoaling downstream of the structure, which in time might raise the water level sufficiently to cause drowning. Any accumulation of material downstream of the structure should therefore be removed periodically.

1.5. GAUGING STRUCTURE

1.5.1. The structure shall be rigid and watertight and capable of withstanding flood flow conditions without distortion or fracture. The weir crest should be at right angles to the direction of flow: the flume axis should be aligned to the direction of flow.

1.5.2. The surfaces of the structure, and of the vertical abutments flanking it, shall be smooth: they may be constructed in concrete with a smooth cement finish, or surfaced with a smooth noncorrodible 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 on the horizontal crest of the weir and in the throat of the flume, but may be relaxed a distance along the profile ½H_{max} upstream and downstream of the crest or throat. The structure should be measured on completion and actual dimensions should be used in the computation of discharge.

1.6. COMPOUND WEIRS

1.6.1. It is permissible to incorporate weirs at different crest elevations in a single gauging structure, but unless the sections with different crest elevations are physically separated by means of intermediate piers so that two-dimensional flow is preserved over each section of the crest, errors may be introduced. Divide piers should extend throughout the full depth of flow. The parallel section should extend from H_{max} upstream of the weir block to the end of the weir block. To avoid sharp curvatures at the cutwaters, the thickness of the piers should be at least $0.2H_{max}$ (lower crest) and also not less than half the difference in the elevations of the adjacent crests. They should have streamlined noses. Flow conditions at and near the pier cutwaters will be improved if the bed level upstream of the low crest section is set below that of the high crest sections, to yield similar values of H/P at the highest discharge to be measured.

1.6.2. In making allowance for the velocity of approach when computing discharge from a total head equation, the actual cross-sectional area of flow at the gauging section should be used in the relationship $V_a = Q/A$ where Q is the total flow, obtained by summing the discharges through each section of crest. Successive approximation may be required in reducing the discharge from gauged head. The C_v relationships given for simple weirs do not apply (see 3.1.5).

1.7. MEASUREMENT OF HEAD

1.7.1. The head upstream of the weir crest may be measured by a vertical or inclined gauge, a hook, point, wire or tape gauge where spot measurements are required, or by a recording gauge where a continuous record is required. The location of the head measurement station is dealt with in later sections. It is usual to measure the head in a separate stilling well to reduce the effects of time-dependent surface irregularities. When this is done, it is also desirable to measure the head in the approach channel as a check.

1.7.2. The stilling well must be vertical and of sufficient height and/or depth to cover the full range of water levels. At the recommended position for the measurement of head the well should be connected to the approach channel by means of a pipe or slot.

Both the well and the connecting pipe or slot must 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 the float at all stages. The float should not be nearer than 0.075 m (0.25 ft) to the wall of the well.

The pipe or slot should have its invert not less than 0.06 m (0.2 ft) below the lowest level to be gauged, and it should terminate flush with the boundary of the approach channel and at right angles thereto. The approach channel boundary should be plane and smooth (equivalent to carefully finished concrete) within a distance of ten times the diameter of the pipe or width of slot from the centre-line 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 should not be rounded or burred.

1.7.3. Adequate additional depth should be provided in the well to avoid the danger of the float grounding on the bottom or any accumulation of silt or debris. The stilling 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.

1.7.4. 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, consistant with ease of maintenance, to damp out oscillations due to short periodic waves. In some installations a valve may be provided for these purposes.

No firm rule can be laid down for determining the size of the connecting pipe or slot, because this is dependent on the circumstances of the particular installation, eg 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 10 cm (4 in) diameter pipe is usually suitable for flow measurement in the field. 3 mm ($\frac{1}{8}$ in) may be appropriate for precision head measurement with steady flows in the laboratory.

1.7.5. Initial setting of the zero of the head measuring device accurately with reference to the crest level of the weir, and regular checking of this setting thereafter, is essential if overall accuracy is to be attained. This is especially important if measurements of small heads are necessary.

An accurate means of checking the zero must be provided. The instrument zero should be obtained by a direct reference to the weir crest, and a record of the setting made in the approach channel and in the stilling well. A zero check based on the water level (either when the flow ceases or just begins) is liable to serious error due to surface tension effects and should not be used.

2. THIN-PLATE WEIRS

A full description of proposed standards for the use of weirs is given in the recommendations of the International Standards Organization (ref 4). Most of the salient points of this section are taken from this document and from BS 3680 Parts 4A and 4B (ref 3).

2.1. TRIANGULAR-NOTCH WEIRS

2.1.1. Within the range of conditions for which the available experimental data are competent, the triangular-notch, thin-plate weir (or V-notch) is one of the most precise measuring devices for clean water not carrying sediment, provided it is not submerged. It is inexpensive, simple to construct and install, and relatively insensitive to the installation environment, but is dependent on careful maintenance. A standard triangular-notch weir is shown in Figure 2.1, and consists of a symmetrical, V-shaped notch in a vertical, thin-plate. The line which bisects the angle of the notch shall be vertical and equidistant from the sides of the channel. The weir plate shall be smooth and plane, especially on the upstream side, and it shall be perpendicular to the sides as well as the bottom of the channel. The crest surfaces of the notch shall be plane surfaces, which shall form sharp, right-angle corners at their intersection with the upstream face of the weir plate. The width of the crest surfaces (measured perpendicular to the face of the plate) shall be between 0.03 and 0.08 in (1 to 2 mm). It is particularly important that the upstream edges of the notch be sharp; that is, that they be machined or filed perpendicular to the upstream face of the weir plate, free of burrs and scratches, and untouched by abrasive cloth or paper. The down-stream edges of the notch shall be chamfered if the weir plate is thicker than the allowable crest width. The surface of the chamfer shall make an angle of not less than 60 deg with the surface of the crest.

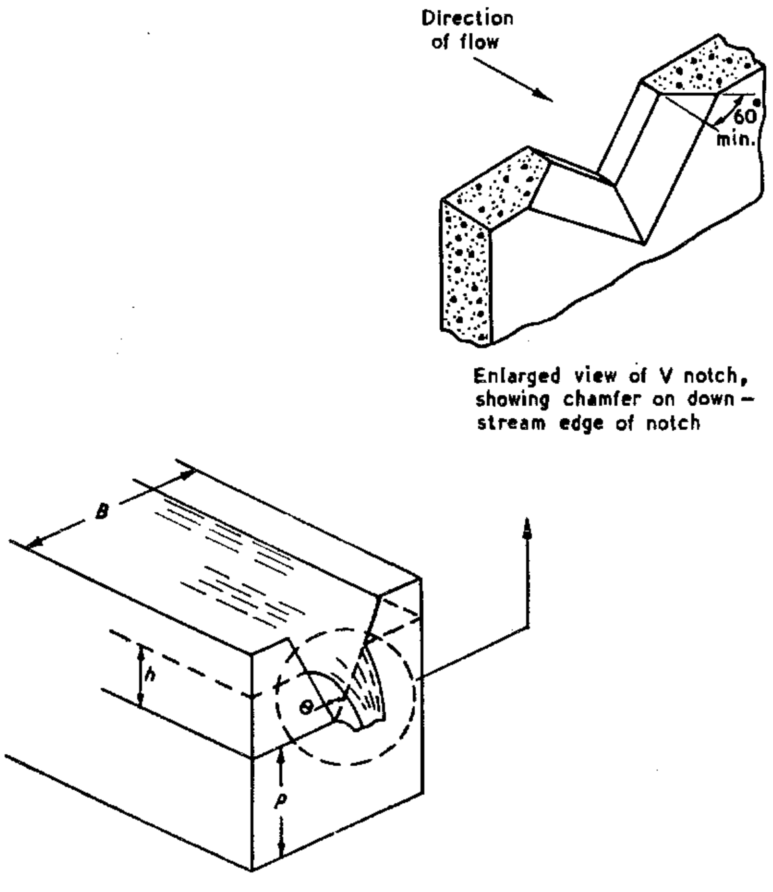


FIGURE 2.1. THE TRIANGULAR THIN-PLATE (SQUARE-EDGED) WEIR (V-NOTCH)

2.1.2. Piezometers or a point-gauge station for the measurement of the head on the weir shall be located a sufficient distance upstream from the weir to avoid the region of surface draw-down. On the other hand, they shall be close enough to the weir that the energy loss between the section of measurement and the weir shall be negligible. For these standards it is recommended that the head measurement section be located a distance equal to from three to four times the maximum head (3 to 4 h_{max}) upstream from the weir.

2.1.3. The discharge nappe shall be fully ventilated and unsubmerged.

2.1.4. Basic equation:

The basic equation of discharge for the triangular-notch weir is

$$Q = C \frac{8}{15} \sqrt{2g} \tan \frac{\theta}{2} h^{5/2} \qquad (2.1)$$

in which Q is the volume rate of flow or discharge (cu ft per sec or cu m per sec), C is the coefficient of discharge (nondimensional), g is the acceleration due to gravity (ft per sec per sec or m per sec per sec), θ is the angle included between the sides of the notch (radians or degrees), and h is the piezometric head or height of the upstream liquid surface referred to the vertex of the notch (ft or m).

2.1.5. In Eq 2.1, C may be described as a function of three geometric ratios and two fluid-property ratios involving viscosity and surface tension. For a given liquid over a limited temperature range, the combined effects of viscosity and surface tension are related to the absolute magnitude of h alone.

2.1.6. Notches with fully-developed contractions, standard angles.

The three sizes of V-notches most commonly used are:

a. The 90 degree notch in which the dimension across the top is twice the vertical depth $\left(\tan \frac{\theta}{2} = 1\right)$

$$Q = \frac{8}{15} \sqrt{2g}\, C_D\, h^{5/2} \qquad (2.2)$$

b. The ½90 degree notch ($\theta = 53°8'$) in which the dimension across the top is equal to the vertical depth $\left(\tan \frac{\theta}{2} = 0.5\right)$

$$Q = \frac{4}{15} \sqrt{2g}\, C_D\, h^{5/2} \qquad (2.3)$$

c. The ¼90 degree notch ($\theta = 28°4'$) in which the dimension across the top is half the vertical depth $\left(\tan \frac{\theta}{2} = 0.25\right)$

$$Q = \frac{2}{15} \sqrt{2g}\, C_D\, h^{5/2} \qquad (2.4)$$

The coefficient values with the corresponding discharge for 90 degree, ½90 degree and ¼90 degree notches are given in Table 2.1 in metric units and in Tables 2.2, 2.3 and 2.4 in foot second units (ref 3).

The general installation conditions must comply with Section 1.4 and the following limitations on h, P, h/P, b and h/B must be observed in the application of these tables.

a. h shall not be less than 2 in (0.05 m) nor more than 15 in (0.38 m).
b. The vertex height, P, shall exceed 1.5 ft (0.45 m).
c. h/P shall not exceed 0.4.
d. The width of the approach channel, B, shall exceed 3.0 ft (0.9 m).
e. h/B shall not exceed 0.20.

TABLE 2.1. DISCHARGE OF WATER OVER V-NOTCHES (METRIC)

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·050	0·608 0	0·008 03	0·615 3	0·004 06	0·650 8	0·002 15
0·051	0·607 5	0·008 43	0·614 9	0·004 27	0·649 8	0·002 25
0·052	0·606 9	0·008 84	0·614 5	0·004 48	0·648 8	0·002 36
0·053	0·606 4	0·009 26	0·614 1	0·004 69	0·647 8	0·002 47
0·054	0·605 9	0·009 70	0·613 7	0·004 91	0·646 8	0·002 59
0·055	0·605 4	0·010 15	0·613 3	0·005 14	0·645 9	0·002 71
0·056	0·604 9	0·010 61	0·613 0	0·005 37	0·644 9	0·002 83
0·057	0·604 5	0·011 08	0·612 6	0·005 61	0·644 0	0·002 95
0·058	0·604 1	0·011 56	0·612 2	0·005 86	0·643 2	0·003 08
0·059	0·603 6	0·012 06	0·611 8	0·006 11	0·642 4	0·003 21
0·060	0·603 2	0·012 57	0·611 4	0·006 37	0·641 7	0·003 34
0·061	0·602 8	0·013 09	0·611 1	0·006 63	0·641 0	0·003 48
0·062	0·602 3	0·013 62	0·610 8	0·006 91	0·640 3	0·003 62
0·063	0·601 9	0·014 17	0·610 5	0·007 18	0·639 6	0·003 76
0·064	0·601 5	0·014 73	0·610 1	0·007 47	0·639 0	0·003 91
0·065	0·601 2	0·015 30	0·609 8	0·007 76	0·638 3	0·004 06
0·066	0·600 8	0·015 88	0·609 5	0·008 06	0·637 6	0·004 21
0·067	0·600 5	0·016 48	0·609 2	0·008 36	0·637 0	0·004 37
0·068	0·600 1	0·017 10	0·609 0	0·008 67	0·636 4	0·004 53
0·069	0·599 8	0·017 72	0·608 7	0·008 99	0·635 8	0·004 70
0·070	0·599 4	0·018 36	0·608 4	0·009 32	0·635 2	0·004 86
0·071	0·599 0	0·019 01	0·608 1	0·009 65	0·634 6	0·005 03
0·072	0·598 7	0·019 67	0·607 9	0·009 99	0·634 0	0·005 21
0·073	0·598 3	0·020 35	0·607 6	0·010 33	0·633 5	0·005 39
0·074	0·598 0	0·021 05	0·607 3	0·010 69	0·632 9	0·005 57

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·075	0·597 8	0·021 76	0·607 1	0·011 05	0·632 4	0·005 75
0·076	0·597 5	0·022 48	0·606 8	0·011 41	0·631 8	0·005 94
0·077	0·597 3	0·023 22	0·606 6	0·011 79	0·631 3	0·006 13
0·078	0·597 0	0·023 97	0·606 4	0·012 17	0·630 8	0·006 33
0·079	0·596 7	0·024 73	0·606 1	0·0125 6	0·630 3	0·006 53
0·080	0·596 4	0·025 51	0·606 0	0·012 96	0·629 8	0·006 73
0·081	0·596 1	0·026 30	0·605 8	0·013 36	0·629 3	0·006 94
0·082	0·595 8	0·027 10	0·605 6	0·013 77	0·628 9	0·007 15
0·083	0·595 5	0·027 92	0·605 4	0·014 19	0·628 5	0·007 37
0·084	0·595 3	0·028 76	0·605 2	0·014 62	0·628 0	0·007 59
0·085	0·595 0	0·029 61	0·605 0	0·015 05	0·627 6	0·007 81
0·086	0·594 8	0·030 48	0·604 8	0·015 49	0·627 2	0·008 03
0·087	0·594 5	0·031 36	0·604 6	0·015 94	0·626 7	0·008 26
0·088	0·594 2	0·032 25	0·604 4	0·016 40	0·626 4	0·008 50
0·089	0·594 0	0·033 16	0·604 2	0·016 86	0·626 0	0·008 74
0·090	0·593 7	0·034 09	0·604 0	0·017 34	0·625 6	0·008 98
0·091	0·593 5	0·035 03	0·603 8	0·017 82	0·625 2	0·009 22
0·092	0·593 3	0·035 98	0·603 6	0·018 30	0·624 8	0·009 47
0·093	0·593 1	0·036 96	0·603 4	0·018 80	0·624 4	0·009 73
0·094	0·592 9	0·037 95	0·603 2	0·019 30	0·624 0	0·009 98
0·095	0·592 7	0·038 95	0·603 0	0·019 81	0·623 6	0·010 25
0·096	0·592 5	0·039 97	0·602 8	0·020 33	0·623 3	0·010 51
0·097	0·592 3	0·041 01	0·602 6	0·020 86	0·622 9	0·010 78
0·098	0·592 1	0·042 06	0·602 4	0·021 39	0·622 6	0·011 06
0·099	0·591 9	0·043 12	0·602 2	0·021 94	0·622 2	0·011 33

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·100	0·591 7	0·044 20	0·602 1	0·022 49	0·621 9	0·011 61
0·101	0·591 4	0·045 30	0·601 9	0·023 05	0·621 5	0·011 90
0·102	0·591 2	0·046 41	0·601 7	0·023 62	0·621 2	0·012 19
0·103	0·591 0	0·047 54	0·601 6	0·024 20	0·620 9	0·012 49
0·104	0·590 8	0·048 69	0·601 4	0·024 78	0·620 5	0·012 78
0·105	0·590 6	0·049 85	0·601 3	0·025 37	0·620 2	0·013 09
0·106	0·590 4	0·051 03	0·601 1	0·025 98	0·619 9	0·013 39
0·107	0·590 2	0·052 22	0·600 9	0·026 59	0·619 6	0·013 71
0·108	0·590 1	0·053 44	0·600 8	0·027 20	0·619 3	0·014 02
0·109	0·589 9	0·054 67	0·600 6	0·027 83	0·619 0	0·014 34
0·110	0·589 8	0·055 92	0·600 5	0·028 47	0·618 7	0·014 66
0·111	0·589 7	0·057 19	0·600 3	0·029 11	0·618 4	0·014 99
0·112	0·589 6	0·058 47	0·600 2	0·029 76	0·618 1	0·015 33
0·113	0·589 4	0·059 77	0·600 0	0·030 42	0·617 9	0·015 66
0·114	0·589 2	0·061 08	0·599 8	0·031 09	0·617 6	0·016 01
0·115	0·589 1	0·062 42	0·599 7	0·031 77	0·617 3	0·016 35
0·116	0·589 0	0·063 77	0·599 5	0·032 46	0·617 1	0·016 70
0·117	0·588 9	0·065 14	0·599 4	0·033 15	0·616 9	0·017 06
0·118	0·588 8	0·066 53	0·599 2	0·033 86	0·616 6	0·017 42
0·119	0·588 6	0·067 93	0·599 1	0·034 57	0·616 4	0·017 78
0·120	0·588 5	0·069 35	0·598 9	0·035 29	0·616 2	0·018 15
0·121	0·588 3	0·070 79	0·598 8	0·036 02	0·616 0	0·018 53
0·122	0·588 2	0·072 24	0·598 7	0·036 77	0·615 8	0·018 91
0·123	0·588 1	0·073 72	0·598 5	0·037 51	0·615 5	0·019 29
0·124	0·588 0	0·075 22	0·598 4	0·038 27	0·615 3	0·019 68

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·125	0·588 0	0·076 73	0·598 2	0·039 04	0·615 1	0·020 07
0·126	0·587 9	0·078 27	0·598 1	0·039 82	0·614 8	0·020 46
0·127	0·587 8	0·079 82	0·598 0	0·040 60	0·614 6	0·020 86
0·128	0·587 7	0·081 39	0·597 9	0·041 40	0·614 4	0·021 27
0·129	0·587 6	0·082 98	0·597 8	0·042 20	0·614 1	0·021 68
0·130	0·587 6	0·084 58	0·597 6	0·043 02	0·613 9	0·022 09
0·131	0·587 5	0·086 21	0·597 5	0·043 84	0·613 7	0·022 51
0·132	0·587 4	0·087 85	0·597 3	0·044 67	0·613 5	0·022 94
0·133	0·587 3	0·089 51	0·597 2	0·045 51	0·613 3	0·023 37
0·134	0·587 2	0·091 19	0·597 1	0·046 36	0·613 1	0·023 80
0·135	0·587 2	0·092 89	0·597 0	0·047 22	0·612 9	0·024 24
0·136	0·587 1	0·094 61	0·596 8	0·048 09	0·612 7	0·024 68
0·137	0·587 0	0·096 34	0·596 7	0·048 97	0·612 5	0·025 13
0·138	0·586 9	0·098 10	0·596 6	0·049 86	0·612 3	0·025 59
0·139	0·586 9	0·099 87	0·596 5	0·050 75	0·612 1	0·026 04
0·140	0·586 8	0·101 67	0·596 4	0·051 66	0·611 9	0·026 51
0·141	0·586 7	0·103 48	0·596 2	0·052 58	0·611 7	0·026 97
0·142	0·586 7	0·105 32	0·596 1	0·053 51	0·611 5	0·027 44
0·143	0·586 6	0·107 17	0·596 0	0·054 44	0·611 3	0·027 92
0·144	0·586 6	0·109 04	0·596 0	0·055 39	0·611 2	0·028 40
0·145	0·586 5	0·110 93	0·595 9	0·056 35	0·611 0	0·028 89
0·146	0·586 4	0·112 84	0·595 8	0·057 32	0·610 8	0·029 38
0·147	0·586 3	0·114 76	0·595 7	0·058 30	0·610 6	0·029 88
0·148	0·586 2	0·116 71	0·595 6	0·059 29	0·610 5	0·030 38
0·149	0·586 2	0·118 67	0·595 6	0·060 29	0·610 3	0·030 89

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·150	0·586 1	0·120 66	0·595 5	0·061 30	0·610 2	0·031 40
0·151	0·586 1	0·122 67	0·595 4	0·062 31	0·610 0	0·031 92
0·152	0·586 0	0·124 71	0·595 2	0·063 34	0·609 9	0·032 45
0·153	0·586 0	0·126 76	0·595 2	0·064 37	0·609 7	0·032 97
0·154	0·585 9	0·128 83	0·595 1	0·065 42	0·609 5	0·033 50
0·155	0·585 9	0·130 93	0·595 0	0·066 48	0·609 3	0·034 04
0·156	0·585 9	0·133 04	0·594 9	0·067 55	0·609 1	0·034 58
0·157	0·585 8	0·135 17	0·594 8	0·068 63	0·609 0	0·035 13
0·158	0·585 8	0·137 32	0·594 8	0·069 71	0·608 8	0·035 68
0·159	0·585 7	0·139 50	0·594 7	0·070 81	0·608 7	0·036 24
0·160	0·585 7	0·141 69	0·594 6	0·071 92	0·608 5	0·036 80
0·161	0·585 7	0·143 91	0·594 5	0·073 04	0·608 3	0·037 37
0·162	0·585 6	0·146 14	0·594 4	0·074 17	0·608 2	0·037 94
0·163	0·585 6	0·148 40	0·594 4	0·075 31	0·608 0	0·038 52
0·164	0·585 5	0·150 67	0·594 3	0·076 46	0·607 9	0·039 11
0·165	0·585 5	0·152 97	0·594 2	0·077 62	0·607 7	0·039 69
0·166	0·585 5	0·155 29	0·594 1	0·078 79	0·607 6	0·040 29
0·167	0·585 4	0·157 63	0·594 1	0·079 98	0·607 4	0·040 89
0·168	0·585 4	0·159 99	0·594 0	0·081 17	0·607 3	0·041 49
0·169	0·585 3	0·162 37	0·593 9	0·082 37	0·607 1	0·042 10
0·170	0·585 3	0·164 77	0·593 8	0·083 58	0·607 0	0·042 72
0·171	0·585 3	0·167 19	0·593 7	0·084 81	0·606 9	0·043 34
0·172	0·585 2	0·169 64	0·593 7	0·086 04	0·606 8	0·043 97
0·173	0·585 2	0·172 10	0·593 6	0·087 28	0·606 7	0·044 60
0·174	0·585 1	0·174 59	0·593 5	0·088 54	0·606 5	0·045 24

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·175	0·585 1	0·177 09	0·593 4	0·089 80	0·606 3	0·045 88
0·176	0·585 1	0·179 63	0·593 3	0·091 08	0·606 2	0·046 53
0·177	0·585 1	0·182 19	0·593 3	0·092 37	0·606 1	0·047 18
0·178	0·585 1	0·184 78	0·593 2	0·093 67	0·606 0	0·047 84
0·179	0·585 1	0·187 38	0·593 1	0·094 97	0·605 9	0·048 51
0·180	0·585 1	0·190 01	0·593 0	0·096 29	0·605 7	0·049 18
0·181	0·585 1	0·192 65	0·592 9	0·097 62	0·605 6	0·049 86
0·182	0·585 0	0·195 31	0·592 9	0·098 96	0·605 5	0·050 54
0·183	0·585 0	0·198 00	0·592 8	0·100 32	0·605 4	0·051 22
0·184	0·585 0	0·200 71	0·592 7	0·101 68	0·605 3	0·051 92
0·185	0·585 0	0·203 45	0·592 6	0·103 05	0·605 1	0·052 61
0·186	0·585 0	0·206 21	0·592 6	0·104 44	0·605 1	0·053 32
0·187	0·585 0	0·208 99	0·592 5	0·105 84	0·605 0	0·054 03
0·188	0·585 0	0·211 80	0·592 5	0·107 26	0·604 9	0·054 75
0·189	0·585 0	0·214 63	0·592 4	0·108 67	0·604 8	0·055 47
0·190	0·585 0	0·217 48	0·592 3	0·110 10	0·604 7	0·056 20
0·191	0·585 0	0·220 34	0·592 3	0·111 55	0·604 5	0·056 93
0·192	0·584 9	0·223 22	0·592 2	0·113 00	0·604 4	0·057 66
0·193	0·584 9	0·226 12	0·592 2	0·114 47	0·604 3	0·058 41
0·194	0·584 9	0·229 06	0·592 1	0·115 95	0·604 2	0·059 16
0·195	0·584 9	0·232 03	0·592 0	0·117 43	0·604 1	0·059 92
0·196	0·584 9	0·235 01	0·592 0	0·118 93	0·604 1	0·060 68
0·197	0·584 9	0·238 02	0·591 9	0·120 44	0·604 0	0·061 45
0·198	0·584 9	0·241 06	0·591 9	0·121 97	0·603 9	0·062 22
0·199	0·584 9	0·244 11	0·591 9	0·123 51	0·603 8	0·063 00

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·200	0·584 9	0·247 19	0·591 8	0·125 06	0·603 8	0·063 79
0·201	0·584 9	0·250 28	0·591 8	0·126 62	0·603 7	0·064 58
0·202	0·584 8	0·253 39	0·591 7	0·128 19	0·603 5	0·065 37
0·203	0·584 8	0·256 52	0·591 7	0·129 77	0·603 4	0·066 17
0·204	0·584 8	0·259 69	0·591 6	0·131 36	0·603 3	0·066 98
0·205	0·584 8	0·262 88	0·591 6	0·132 96	0·603 3	0·067 80
0·206	0·584 8	0·266 10	0·591 5	0·134 57	0·603 2	0·068 62
0·207	0·584 8	0·269 34	0·591 5	0·136 20	0·603 1	0·069 44
0·208	0·584 8	0·272 61	0·591 4	0·137 84	0·603 0	0·070 28
0·209	0·584 8	0·275 90	0·591 3	0·139 49	0·602 9	0·071 11
0·210	0·584 8	0·279 21	0·591 3	0·141 15	0·602 9	0·071 96
0·211	0·584 8	0·282 54	0·591 2	0·142 82	0·602 8	0·072 81
0·212	0·584 8	0·285 88	0·591 2	0·144 50	0·602 7	0·073 66
0·213	0·584 7	0·289 24	0·591 1	0·146 20	0·602 6	0·074 53
0·214	0·584 7	0·292 64	0·591 1	0·147 92	0·602 5	0·075 39
0·215	0·584 7	0·296 07	0·591 0	0·149 64	0·602 5	0·076 27
0·216	0·584 7	0·299 53	0·591 0	0·151 38	0·602 4	0·077 15
0·217	0·584 7	0·303 01	0·591 0	0·153 13	0·602 3	0·078 03
0·218	0·584 7	0·306 51	0·590 9	0·154 89	0·602 2	0·078 93
0·219	0·584 7	0·310 04	0·590 9	0·156 66	0·602 2	0·079 82
0·220	0·584 7	0·313 59	0·590 8	0·158 44	0·602 1	0·080 73
0·221	0·584 7	0·317 17	0·590 8	0·160 24	0·602 0	0·081 64
0·222	0·584 7	0·320 77	0·590 8	0·162 04	0·601 9	0·082 55
0·223	0·584 7	0·324 39	0·590 7	0·163 86	0·601 8	0·083 47
0·224	0·584 7	0·328 03	0·590 7	0·165 70	0·601 8	0·084 41

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·225	0·584 6	0·331 68	0·590 6	0·167 54	0·601 7	0·085 35
0·226	0·584 6	0·335 35	0·590 6	0·169 40	0·601 7	0·086 29
0·227	0·584 6	0·339 07	0·590 6	0·171 27	0·601 6	0·087 24
0·228	0·584 6	0·342 82	0·590 5	0·173 15	0·601 5	0·088 19
0·229	0·584 6	0·346 59	0·590 5	0·175 04	0·601 5	0·089 15
0·230	0·584 6	0·350 39	0·590 4	0·176 95	0·601 4	0·090 11
0·231	0·584 6	0·354 21	0·590 4	0·178 86	0·601 3	0·091 08
0·232	0·584 6	0·358 06	0·590 4	0·180 79	0·601 3	0·092 07
0·233	0·584 6	0·361 93	0·590 3	0·182 74	0·601 2	0·093 06
0·234	0·584 6	0·365 82	0·590 3	0·184 69	0·601 2	0·094 05
0·235	0·584 6	0·369 74	0·590 2	0·186 66	0·601 1	0·095 04
0·236	0·584 6	0·373 69	0·590 2	0·188 64	0·601 0	0·096 05
0·237	0·584 6	0·377 66	0·590 2	0·190 69	0·601 0	0·097 06
0·238	0·584 6	0·381 66	0·590 1	0·192 63	0·600 9	0·098 08
0·239	0·584 6	0·385 68	0·590 1	0·194 65	0·600 8	0·099 10
0·240	0·584 6	0·389 73	0·590 1	0·196 68	0·600 8	0·100 13
0·241	0·584 6	0·393 80	0·550 0	0·198 72	0·600 7	0·101 16
0·242	0·584 6	0·397 90	0·590 0	0·200 79	0·600 6	0·102 20
0·243	0·584 6	0·402 02	0·590 0	0·202 87	0·600 6	0·103 25
0·244	0·584 6	0·406 17	0·590 0	0·204 96	0·600 5	0·104 30
0·245	0·584 6	0·410 34	0·590 0	0·207 05	0·600 4	0·105 36
0·246	0·584 6	0·414 54	0·589 9	0·209 16	0·600 3	0·106 42
0·247	0·584 6	0·418 77	0·589 9	0·211 27	0·600 3	0·107 50
0·248	0·584 6	0·423 02	0·589 8	0·213 40	0·600 2	0·108 58
0·249	0·584 6	0·427 30	0·589 8	0·215 55	0·600 2	0·109 67

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·250	0·584 6	0·431 60	0·589 8	0·217 72	0·600 2	0·110 77
0·251	0·584 6	0·435 93	0·589 8	0·219 90	0·600 1	0·111 87
0·252	0·584 6	0·440 28	0·589 8	0·222 09	0·600 1	0·112 99
0·253	0·584 6	0·444 66	0·589 7	0·224 29	0·600 0	0·114 10
0·254	0·584 6	0·449 07	0·589 7	0·226 49	0·600 0	0·115 23
0·255	0·584 6	0·453 50	0·589 7	0·228 73	0·600 0	0·116 35
0·256	0·584 6	0·457 96	0·589 7	0·230 98	0·599 9	0·117 49
0·257	0·584 6	0·462 45	0·589 7	0·233 23	0·599 9	0·118 63
0·258	0·584 6	0·466 96	0·589 6	0·235 49	0·599 8	0·119 78
0·259	0·584 6	0·471 50	0·589 6	0·237 77	0·599 8	0·120 94
0·260	0·584 6	0·476 06	0·589 6	0·240 05	0·599 7	0·122 10
0·261	0·584 6	0·480 65	0·589 5	0·242 35	0·599 6	0·123 26
0·262	0·584 6	0·485 27	0·589 5	0·244 66	0·599 6	0·124 43
0·263	0·584 6	0·489 91	0·589 4	0·246 99	0·599 5	0·125 61
0·264	0·584 6	0·494 53	0·589 4	0·249 33	0·599 5	0·126 80
0·265	0·584 6	0·499 28	0·589 4	0·251 68	0·599 5	0·127 99
0·266	0·584 6	0·504 00	0·589 3	0·254 04	0·599 4	0·129 20
0·267	0·584 6	0·508 76	0·589 3	0·256 42	0·599 4	0·130 41
0·268	0·584 6	0·513 53	0·589 2	0·258 81	0·599 3	0·131 62
0·269	0·584 6	0·518 34	0·589 2	0·261 21	0·599 3	0·132 84
0·270	0·584 6	0·523 17	0·589 2	0·263 63	0·599 2	0·134 07
0·271	0·584 6	0·528 02	0·589 1	0·266 06	0·599 2	0·135 29
0·272	0·584 6	0·532 91	0·589 1	0·268 51	0·599 1	0·136 53
0·273	0·584 6	0·537 82	0·589 1	0·270 98	0·599 1	0·137 78
0·274	0·584 6	0·542 76	0·589 1	0·273 47	0·599 0	0·139 03

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		$m^3/s \times 10$		$m^3/s \times 10$		$m^3/s \times 10$
0·275	0·584 6	0·547 72	0·589 1	0·275 96	0·599 0	0·140 30
0·276	0·584 6	0·552 72	0·589 0	0·278 45	0·598 9	0·141 57
0·277	0·584 6	0·557 74	0·589 0	0·280 97	0·598 9	0·142 84
0·278	0·584 6	0·562 83	0·589 0	0·283 51	0·598 9	0·144 13
0·279	0·584 7	0·567 94	0·589 0	0·286 07	0·598 8	0·145 42
0·280	0·584 7	0·573 06	0·589 0	0·288 63	0·598 8	0·146 71
0·281	0·584 7	0·578 19	0·588 9	0·291 19	0·598 7	0·148 02
0·282	0·584 7	0·583 35	0·588 9	0·293 77	0·598 7	0·149 33
0·283	0·584 7	0·588 53	0·588 9	0·296 38	0·598 7	0·150 65
0·284	0·584 7	0·593 75	0·588 9	0·299 01	0·598 6	0·151 97
0·285	0·584 7	0·598 99	0·588 9	0·301 63	0·598 6	0·153 30
0·286	0·584 7	0·604 25	0·588 8	0·304 27	0·598 5	0·154 64
0·287	0·584 7	0·609 55	0·588 8	0·306 91	0·598 5	0·155 98
0·288	0·584 7	0·614 87	0·588 8	0·309 59	0·598 5	0·157 34
0·289	0·584 7	0·620 23	0·588 8	0·312 29	0·598 4	0·158 70
0·290	0·584 7	0·625 60	0·588 8	0·314 99	0·598 4	0·160 06
0·291	0·584 7	0·631 01	0·588 7	0·317 69	0·598 3	0·161 43
0·292	0·584 7	0·636 45	0·588 7	0·320 40	0·598 3	0·162 81
0·293	0·584 7	0·641 95	0·588 7	0·323 15	0·598 3	0·164 20
0·294	0·584 8	0·647 48	0·588 7	0·325 91	0·598 2	0·165 59
0·295	0·584 8	0·653 03	0·588 7	0·328 69	0·598 2	0·166 99
0·296	0·584 8	0·658 58	0·588 6	0·331 46	0·598 1	0·168 40
0·297	0·584 8	0·664 16	0·588 6	0·334 24	0·598 1	0·169 82
0·298	0·584 8	0·669 76	0·588 6	0·337 04	0·598 1	0·171 24
0·299	0·584 8	0·675 39	0·588 5	0·339 85	0·598 0	0·172 67

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·300	0·584 8	0·681 06	0·588 5	0·342 68	0·598 0	0·174 10
0·301	0·584 8	0·686 75	0·588 4	0·345 52	0·597 9	0·175 55
0·302	0·584 8	0·692 46	0·588 4	0·348 37	0·597 9	0·177 00
0·303	0·584 8	0·698 21	0·588 4	0·351 24	0·597 9	0·178 45
0·304	0·584 8	0·703 98	0·588 3	0·354 12	0·597 8	0·179 92
0·305	0·584 8	0·709 80	0·588 3	0·357 02	0·597 8	0·181 39
0·306	0·584 8	0·715 68	0·588 3	0·359 95	0·597 8	0·182 87
0·307	0·584 9	0·721 59	0·588 3	0·362 90	0·597 7	0·184 35
0·308	0·584 9	0·727 50	0·588 3	0·365 85	0·597 7	0·185 85
0·309	0·584 9	0·733 41	0·588 2	0·368 80	0·597 6	0·137 35
0·310	0·584 9	0·739 36	0·588 2	0·371 77	0·597 6	0·188 85
0·311	0·584 9	0·745 34	0·588 2	0·374 77	0·597 6	0·190 37
0·312	0·584 9	0·751 35	0·588 2	0·377 79	0·597 5	0·191 89
0·313	0·584 9	0·757 38	0·588 2	0·380 81	0·597 5	0·193 42
0·314	0·584 9	0·763 44	0·588 1	0·383 84	0·597 4	0·194 95
0·315	0·584 9	0·769 54	0·588 1	0·386 87	0·597 4	0·196 50
0·316	0·584 9	0·775 66	0·588 1	0·389 95	0·597 4	0·198 05
0·317	0·584 9	0·781 81	0·588 1	0·393 04	0·597 3	0·199 60
0·318	0·584 9	0·788 02	0·588 1	0·396 15	0·597 3	0·201 17
0·319	0·585 0	0·794 28	0·588 1	0·399 27	0·597 2	0·202 74
0·320	0·585 0	0·800 57	0·588 1	0·402 41	0·597 2	0·204 32
0·321	0·585 0	0·806 85	0·588 1	0·405 53	0·597 2	0·205 90
0·322	0·585 0	0·813 14	0·588 0	0·408 67	0·597 1	0·207 50
0·323	0·585 0	0·819 47	0·588 0	0·411 84	0·597 1	0·209 10
0·324	0·585 0	0·825 83	0·588 0	0·415 03	0·597 0	0·210 71

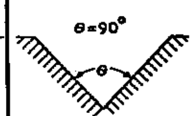
Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·325	0·585 0	0·832 22	0·588 0	0·418 24	0·597 0	0·212 32
0·326	0·585 0	0·838 63	0·588 0	0·421 47	0·597 0	0·213 95
0·327	0·585 0	0·845 88	0·588 0	0·424 71	0·596 9	0·215 58
0·328	0·585 0	0·851 55	0·588 0	0·427 96	0·596 9	0·217 21
0·329	0·585 0	0·858 06	0·588 0	0·431 23	0·596 8	0·218 86
0·330	0·585 0	0·864 59	0·588 0	0·434 51	0·596 8	0·220 51
0·331	0·585 0	0·871 16	0·588 0	0·437 79	0·596 8	0·222 17
0·332	0·585 0	0·877 75	0·587 9	0·441 07	0·596 7	0·223 84
0·333	0·585 0	0·884 38	0·587 9	0·444 38	0·596 7	0·225 51
0·334	0·585 0	0·881 03	0·587 9	0·447 73	0·596 7	0·227 19
0·335	0·585 0	0·897 72	0·587 9	0·451 08	0·596 6	0·228 88
0·336	0·585 0	0·904 48	0·587 9	0·454 46	0·596 6	0·230 58
0·337	0·585 1	0·911 28	0·587 9	0·457 85	0·596 5	0·232 28
0·338	0·585 1	0·918 11	0·587 9	0·461 25	0·596 5	0·234 00
0·339	0·585 1	0·924 91	0·587 9	0·464 67	0·596 5	0·235 72
0·340	0·585 1	0·931 75	0·587 9	0·468 10	0·596 4	0·237 44
0·341	0·585 1	0·938 62	0·587 9	0·471 53	0·596 4	0·239 18
0·342	0·585 1	0·945 51	0·587 8	0·474 97	0·596 3	0·240 92
0·343	0·585 1	0·952 44	0·587 8	0·478 42	0·596 3	0·242 67
0·344	0·585 1	0·959 40	0·587 8	0·481 91	0·596 3	0·244 42
0·345	0·585 1	0·966 38	0·587 8	0·485 42	0·596 2	0·246 19
0·346	0·585 1	0·973 40	0·587 8	0·488 95	0·596 2	0·247 96
0·347	0·585 1	0·980 45	0·587 8	0·492 49	0·596 1	0·249 74
0·348	0·585 1	0·987 53	0·587 8	0·496 04	0·596 1	0·251 52
0·349	0·585 1	0·994 71	0·587 8	0·499 58	0·596 1	0·253 32

Head	90 degree V-notch		½90 degree V-notch		¼90 degree V-notch	
	Coefficient C_D	Discharge	Coefficient C_D	Discharge	Coefficient C_D	Discharge
m		m³/s × 10		m³/s × 10		m³/s × 10
0·350	0·585 2	1·001 92	0·587 7	0·503 13	0·596 0	0·255 12
0·351	0·585 2	1·009 12	0·587 7	0·506 72	0·596 0	0·256 93
0·352	0·585 2	1·016 33	0·587 7	0·510 33	0·595 9	0·258 75
0·353	0·585 2	1·023 56	0·587 7	0·513 97	0·595 9	0·260 57
0·354	0·585 2	1·030 82	0·587 7	0·517 58	0·595 9	0·262 40
0·355	0·585 2	1·038 12	0·587 6	0·521 21	0·595 8	0·264 24
0·356	0·585 2	1·045 45	0·587 6	0·524 87	0·595 8	0·266 09
0·357	0·585 2	1·052 80	0·587 6	0·528 56	0·595 7	0·267 94
0·358	0·585 2	1·060 19	0·587 6	0·532 27	0·595 7	0·269 81
0·359	0·585 2	1·067 67	0·587 6	0·535 96	0·595 7	0·271 68
0·360	0·585 3	1·075 19	0·587 5	0·539 67	0·595 6	0·273 55
0·361	0·585 3	1·082 73	0·587 5	0·543 40	0·595 6	0·275 44
0·362	0·585 3	1·090 24	0·587 5	0·547 17	0·595 5	0·277 33
0·363	0·585 3	1·097 78	0·587 5	0·550 96	0·595 5	0·279 23
0·364	0·585 3	1·105 36	0·587 5	0·554 73	0·595 5	0·281 14
0·365	0·585 3	1·112 97	0·587 4	0·558 51	0·595 4	0·283 06
0·366	0·585 3	1·120 63	0·587 4	0·562 31	0·595 4	0·284 98
0·367	0·585 3	1·128 37	0·587 4	0·566 16	0·595 4	0·286 91
0·368	0·585 4	1·136 15	0·587 4	0·570 03	0·595 3	0·288 85
0·369	0·585 4	1·143 91	0·587 4	0·573 91	0·595 3	0·290 80
0·370	0·585 4	1·151 67	0·587 4	0·577 80	0·595 2	0·292 75
0·371	0·585 4	1·159 47	0·587 4	0·581 71	0·595 2	0·294 72
0·372	0·585 4	1·167 30	0·587 4	0·585 60	0·595 2	0·296 69
0·373	0·585 4	1·175 16	0·587 3	0·589 50	0·595 1	0·298 67
0·374	0·585 4	1·183 10	0·587 3	0·593 45	0·595 1	0·300 65
0·375	0·585 5	1·191 11	0·587 3	0·597 42	0·595 0	0·302 64
0·376	0·585 5	1·199 14	0·587 3	0·601 41	0·595 0	0·304 65
0·377	0·585 5	1·207 12	0·587 3	0·605 42	0·595 0	0·306 66
0·378	0·585 5	1·215 15	0·587 3	0·609 44	0·594 9	0·308 67
0·379	0·585 5	1·223 20	0·587 3	0·613 46	0·594 9	0·310 70
0·380	0·585 5	1·231 28	0·587 2	0·617 47	0·594 8	0·312 73
0·381	0·585 5	1·239 40	0·587 2	0·621 50	0·594 8	0·314 77

NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

TABLE 2.2. DISCHARGE OF WATER OVER A 90 DEGREE V-NOTCH

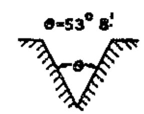
Head in inches	Co-efficient C_D	Dis-charge ft³/s	Head in inches	Co-efficient C_D	Dis-charge ft³/s	Head in inches	Co-efficient C_D	Dis-charge ft³/s	Head in inches	Co-efficient C_D	Dis-charge ft³/s
2·0	0·6076	0·02949	5·5	0·5868	0·35718	9·0	0·5846	1·2189	12·5	0·5849	2·7723
2·1	0·6062	0·03324	5·6	0·5867	0·37357	9·1	0·5846	1·2530	12·6	0·5850	2·8286
2·2	0·6050	0·03726	5·7	0·5865	0·39034	9·2	0·5846	1·2877	12·7	0·5850	2·8850
2·3	0·6039	0·04157	5·8	0·5863	0·40755	9·3	0·5846	1·3230	12·8	0·5850	2·9422
2·4	0·6028	0·04615	5·9	0·5861	0·42520	9·4	0·5846	1·3588	12·9	0·5850	3·0000
2·5	0·6017	0·05102	6·0	0·5860	0·44337	9·5	0·5846	1·3953	13·0	0·5850	3·0585
2·6	0·6008	0·05619	6·1	0·5859	0·46199	9·6	0·5846	1·4323	13·1	0·5850	3·1176
2·7	0·5999	0·06166	6·2	0·5858	0·48108	9·7	0·5846	1·4699	13·2	0·5850	3·1774
2·8	0·5990	0·06742	6·3	0·5857	0·50063	9·8	0·5846	1·5080	13·3	0·5851	3·2385
2·9	0·5981	0·07349	6·4	0·5856	0·52064	9·9	0·5846	1·5468	13·4	0·5851	3·2997
3·0	0·5975	0·07991	6·5	0·5855	0·54113	10·0	0·5846	1·5862	13·5	0·5851	3·3616
3·1	0·5968	0·08664	6·6	0·5854	0·56208	10·1	0·5846	1·6261	13·6	0·5851	3·4242
3·2	0·5960	0·09367	6·7	0·5853	0·58352	10·2	0·5846	1·6667	13·7	0·5851	3·4875
3·3	0·5853	0·10104	6·8	0·5852	0·60543	10·3	0·5846	1·7078	13·8	0·5852	3·5521
3·4	0·5947	0·10876	6·9	0·5851	0·62783	10·4	0·5846	1·7496	13·9	0·5852	3·6168
3·5	0·5940	0·11680	7·0	0·5851	0·65082	10·5	0·5846	1·7919	14·0	0·5852	3·6822
3·6	0·5934	0·12520	7·1	0·5851	0·67432	10·6	0·5846	1·8349	14·1	0·5852	3·7483
3·7	0·5929	0·13396	7·2	0·5850	0·69819	10·7	0·5846	1·8785	14·2	0·5853	3·8158
3·8	0·5924	0·14307	7·3	0·5850	0·72269	10·8	0·5846	1·9227	14·3	0·5853	3·8833
3·9	0·5919	0·15254	7·4	0·5850	0·74769	10·9	0·5846	1·9675	14·4	0·5853	3·9516
4·0	0·5913	0·16235	7·5	0·5850	0·77321	11·0	0·5847	2·0133	14·5	0·5854	4·0212
4·1	0·5908	0·17254	7·6	0·5849	0·79910	11·1	0·5847	2·0593	14·6	0·5854	4·0909
4·2	0·5903	0·18310	7·7	0·5849	0·82565	11·2	0·5847	2·1060	14·7	0·5854	4·1613
4·3	0·5899	0·19406	7·8	0·5849	0·85272	11·3	0·5847	2·1534	14·8	0·5855	4·2332
4·4	0·5896	0·20544	7·9	0·5849	0·88031	11·4	0·5847	2·2013	14·9	0·5855	4·3051
4·5	0·5892	0·21716	8·0	0·5848	0·90828	11·5	0·5847	2·2499	15·0	0·5855	4·3777
4·6	0·5889	0·22931	8·1	0·5848	0·93693	11·6	0·5848	2·2995			
4·7	0·5886	0·24185	8·2	0·5848	0·96611	11·7	0·5848	2·3494			
4·8	0·5882	0·25475	8·3	0·5848	0·99584	11·8	0·5848	2·3999			
4·9	0·5880	0·26814	8·4	0·5847	1·0259	11·9	0·5848	2·4511			
5·0	0·5878	0·28193	8·5	0·5847	1·0567	12·0	0·5848	2·5029			
5·1	0·5876	0·29614	8·6	0·5847	1·0881	12·1	0·5849	2·5558			
5·2	0·5874	0·31076	8·7	0·5847	1·1200	12·2	0·5849	2·6090			
5·3	0·5872	0·32581	8·8	0·5847	1·1525	12·3	0·5849	2·6628			
5·4	0·5870	0·34128	8·9	0·5846	1·1853	12·4	0·5849	2·7172			



NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

TABLE 2.3. DISCHARGE OF WATER OVER A ½/90 DEGREE V-NOTCH

Head in inches	Co-efficient C_D	Dis-charge ft³/s	Head in inches	Co-efficient C_D	Dis-charge ft³/s	Head in inches	Co-efficient C_D	Dis-charge ft³/s	Head in inches	Co-efficient C_D	Dis-charge ft³/s
2·0	0·6150	0·01492	5·5	0·5964	0·18151	9·0	0·5905	0·61558	12·5	0·5881	1·3937
2·1	0·6140	0·01683	5·6	0·5961	0·18978	9·1	0·5904	0·63271	12·6	0·5881	1·4218
2·2	0·6130	0·01888	5·7	0·5959	0·19830	9·2	0·5903	0·65013	12·7	0·5880	1·4499
2·3	0·6120	0·02106	5·8	0·5957	0·20704	9·3	0·5902	0·66783	12·8	0·5880	1·4786
2·4	0·6111	0·02339	5·9	0·5955	0·21601	9·4	0·5901	0·68581	12·9	0·5880	1·5077
2·5	0·6103	0·02587	6·0	0·5952	0·22516	9·5	0·5900	0·70407	13·0	0·5880	1·5371
2·6	0·6095	0·02850	6·1	0·5950	0·23458	9·6	0·5900	0·72275	13·1	0·5879	1·5665
2·7	0·6088	0·01329	6·2	0·5948	0·24423	9·7	0·5899	0·74159	13·2	0·5879	1·5966
2·8	0·6081	0·03422	6·3	0·5946	0·25412	9·8	0·5898	0·76073	13·3	0·5879	1·6270
2·9	0·6074	0·03732	6·4	0·5944	0·26423	9·9	0·5898	0·78028	13·4	0·5879	1·6578
3·0	0·6068	0·04058	6·5	0·5942	0·27458	10·0	0·5897	0·80000	13·5	0·5878	1·6886
3·1	0·6062	0·04400	6·6	0·5940	0·28517	10·1	0·5897	0·82015	13·6	0·5878	1·7200
3·2	0·6057	0·04760	6·7	0·5938	0·29600	10·2	0·5896	0·84046	13·7	0·5878	1·7518
3·3	0·6052	0·05136	6·8	0·5936	0·30706	10·3	0·5895	0·86106	13·8	0·5877	1·7837
3·4	0·6047	0·05530	6·9	0·5934	0·31837	10·4	0·5894	0·88196	13·9	0·5877	1·8161
3·5	0·6042	0·05940	7·0	0·5932	0·32992	10·5	0·5893	0·90316	14·0	0·5876	1·8487
3·6	0·6037	0·06368	7·1	0·5930	0·34171	10·6	0·5892	0·92467	14·1	0·5876	1·8819
3·7	0·6032	0·06814	7·2	0·5928	0·35375	10·7	0·5891	0·94647	14·2	0·5875	1·9151
3·8	0·6027	0·07278	7·3	0·5926	0·36604	10·8	0·5891	0·96874	14·3	0·5875	1·9490
3·9	0·6022	0·07760	7·4	0·5925	0·37864	10·9	0·5890	0·99115	14·4	0·5874	1·9829
4·0	0·6018	0·08262	7·5	0·5923	0·39143	11·0	0·5890	1·0140	14·5	0·5874	2·0175
4·1	0·6014	0·08782	7·6	0·5922	0·40454	11·1	0·5889	1·0371	14·6	0·5874	2·0525
4·2	0·6010	0·09321	7·7	0·5920	0·41784	11·2	0·5889	1·0606	14·7	0·5873	2·0874
4·3	0·6006	0·09879	7·8	0·5919	0·43146	11·3	0·5888	1·0842	14·8	0·5873	2·1231
4·4	0·6002	0·10456	7·9	0·5918	0·44535	11·4	0·5888	1·1084	14·9	0·5873	2·1592
4·5	0·5998	0·11053	8·0	0·5917	0·45950	11·5	0·5887	1·1327	15·0	0·5872	2·1952
4·6	0·5994	0·11670	8·1	0·5915	0·47383	11·6	0·5887	1·1574			
4·7	0·5990	0·12306	8·2	0·5914	0·48851	11·7	0·5886	1·1823			
4·8	0·5987	0·12965	8·3	0·5912	0·50337	11·8	0·5885	1·2076			
4·9	0·5983	0·13642	8·4	0·5911	0·51858	11·9	0·5884	1·2331			
5·0	0·5980	0·14341	8·5	0·5910	0·53406	12·0	0·5883	1·2590			
5·1	0·5977	0·15061	8·6	0·5909	0·54982	12·1	0·5883	1·2853			
5·2	0·5973	0·15800	8·7	0·5908	0·56584	12·2	0·5882	1·3121			
5·3	0·5970	0·16562	8·8	0·5907	0·58214	12·3	0·5882	1·3389			
5·4	0·5967	0·17346	8·9	0·5906	0·59872	12·4	0·5881	1·3650			



NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

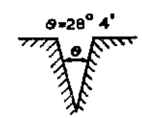
TABLE 2.4. DISCHARGE OF WATER OVER A ¼/90 DEGREE V-NOTCH

Head in inches	Co-efficient C_D	Dis-charge ft³/s
2·0	0·6500	0·00789
2·1	0·6475	0·00888
2·2	0·6450	0·00993
2·3	0·6428	0·01106
2·4	0·6410	0·01227
2·5	0·6393	0·01355
2·6	0·6376	0·01491
2·7	0·6360	0·01634
2·8	0·6345	0·01785
2·9	0·6331	0·01945
3·0	0·6317	0·02112
3·1	0·6304	0·02288
3·2	0·6292	0·02472
3·3	0·6281	0·02665
3·4	0·6270	0·02867
3·5	0·6260	0·03077
3·6	0·6250	0·03296
3·7	0·6240	0·03525
3·8	0·6231	0·03762
3·9	0·6222	0·04008
4·0	0·6213	0·04264
4·1	0·6205	0·04531
4·2	0·6197	0·04806
4·3	0·6189	0·05090
4·4	0·6182	0·05385
4·5	0·6175	0·05689
4·6	0·6169	0·06005
4·7	0·6163	0·06330
4·8	0·6158	0·06666
4·9	0·6152	0·07013
5·0	0·6146	0·07369
5·1	0·6140	0·07736
5·2	0·6135	0·08114
5·3	0·6130	0·08503
5·4	0·6125	0·08902

Head in inches	Co-efficient C_D	Dis-charge ft³/s
5·5	0·6120	0·09313
5·6	0·6115	0·09734
5·7	0·6110	0·10166
5·8	0·6106	0·10611
5·9	0·6102	0·11067
6·0	0·6098	0·11534
6·1	0·6093	0·12011
6·2	0·6089	0·12501
6·3	0·6085	0·13003
6·4	0·6081	0·13516
6·5	0·6077	0·14041
6·6	0·6073	0·14578
6·7	0·6070	0·15129
6·8	0·6067	0·15591
6·9	0·6063	0·16264
7·0	0·6060	0·16851
7·1	0·6057	0·17452
7·2	0·6054	0·18064
7·3	0·6051	0·18688
7·4	0·6049	0·19326
7·5	0·6046	0·19978
7·6	0·6043	0·20641
7·7	0·6041	0·21318
7·8	0·6039	0·22011
7·9	0·6037	0·22715
8·0	0·6034	0·23430
8·1	0·6032	0·24160
8·2	0·6030	0·24903
8·3	0·6028	0·25659
8·4	0·6026	0·26428
8·5	0·6024	0·27215
8·6	0·6022	0·28014
8·7	0·6020	0·28827
8·8	0·6018	0·29654
8·9	0·6017	0·30495

Head in inches	Co-efficient C_D	Dis-charge ft³/s
9·0	0·6015	0·31352
9·1	0·6013	0·32223
9·2	0·6012	0·33108
9·3	0·6010	0·34007
9·4	0·6009	0·34920
9·5	0·6007	0·35848
9·6	0·6005	0·36790
9·7	0·6003	0·37746
9·8	0·6002	0·38717
9·9	0·6001	0·39702
10·0	0·6000	0·40701
10·1	0·5999	0·41716
10·2	0·5998	0·42746
10·3	0·5996	0·43791
10·4	0·5995	0·44851
10·5	0·5994	0·45926
10·6	0·5993	0·47017
10·7	0·5991	0·48124
10·8	0·5990	0·49248
10·9	0·5989	0·50388
11·0	0·5988	0·51544
11·1	0·5987	0·52713
11·2	0·5986	0·53898
11·3	0·5985	0·55100
11·4	0·5984	0·56319
11·5	0·5983	0·57555
11·6	0·5982	0·58806
11·7	0·5981	0·60072
11·8	0·5980	0·61353
11·9	0·5979	0·62650
12·0	0·5978	0·63962
12·1	0·5977	0·65291
12·2	0·5976	0·66637
12·3	0·5975	0·68000
12·4	0·5974	0·69380

Head in inches	Co-efficient C_D	Dis-charge ft³/s
12·5	0·5973	0·70777
12·6	0·5972	0·72190
12·7	0·5971	0·73619
12·8	0·5970	0·75065
12·9	0·5969	0·76527
13·0	0·5968	0·78005
13·1	0·5967	0·79500
13·2	0·5966	0·81012
13·3	0·5965	0·82541
13·4	0·5964	0·84087
13·5	0·5963	0·85650
13·6	0·5962	0·87830
13·7	0·5961	0·88827
13·8	0·5960	0·90441
13·9	0·5959	0·92073
14·0	0·5958	0·93722
14·1	0·5957	0·95388
14·2	0·5956	0·97072
14·3	0·5955	0·98774
14·4	0·5954	1·0049
14·5	0·5953	1·0223
14·6	0·5952	1·0399
14·7	0·5951	1·0576
14·8	0·5950	1·0755
14·9	0·5949	1·0936
15·0	0·5948	1·1118



NOTE. The number of significant figures given in the columns for coefficient and discharge should not be taken to imply a corresponding accuracy in the knowledge of the values given, but only to assist in interpolation and analysis.

2.1.7. Notches with angles between 20° and 100°

A comprehensive study and re-analysis of the available experimental data on the flow of water through triangular-notch weirs indicates that the fluid-property influences can be allowed for by adjusting the measured value of h; the result is a modified equation of discharge (ref 5).

$$Q = C_e \frac{8}{15} \sqrt{2g} \tan\frac{\theta}{2} h_e^{5/2} \text{ (Kindsvater-Shen)} \tag{2.5}$$

in which C_e is a function of the geometric ratios alone,

$$C_e = f\left(\frac{h}{P}, \frac{P}{B}, \theta\right). \tag{2.6}$$

and h_e, the effective head, is

$$h_e = h + k_h. \tag{2.7}$$

In Eq 2.7, k_h is an absolute measure (ft or m) of the combined effects of viscosity and surface tension. Figure 2.2.

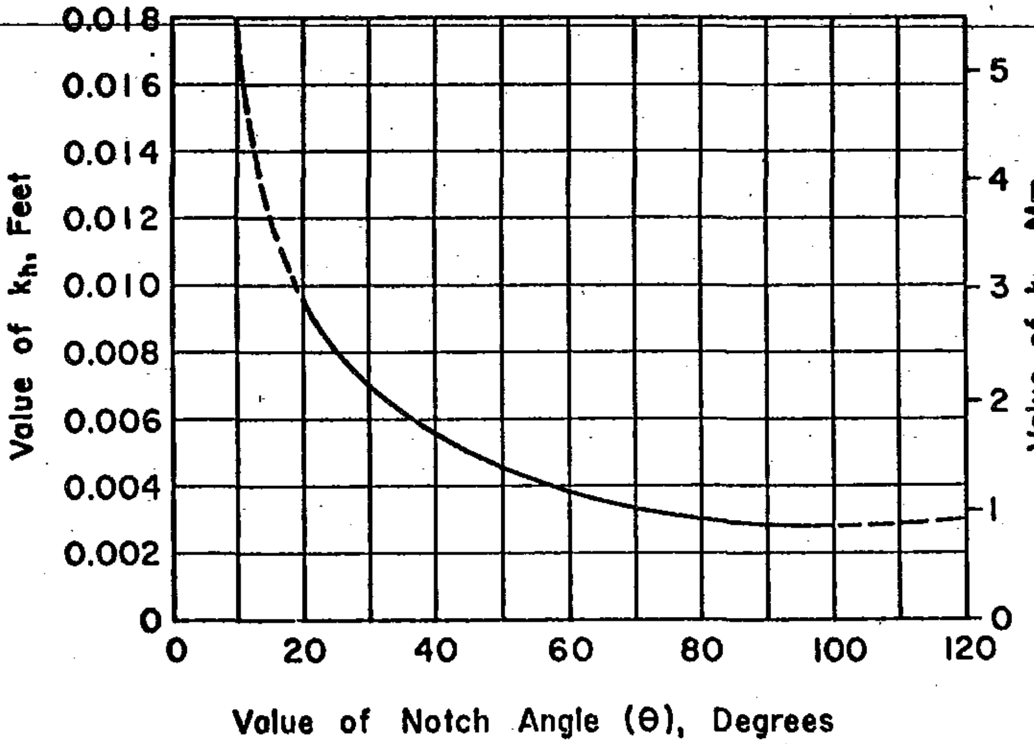


FIGURE 2.2. VALUE OF k_h RELATED TO NOTCH ANGLE

relates k_h to various notch angles and is based on experimental data from many sources, (refs 6, 7, 8, 9, 10)

The coefficient of discharge is shown in Figure 2.3 for a 90-degree notch weir for a wide range of values of

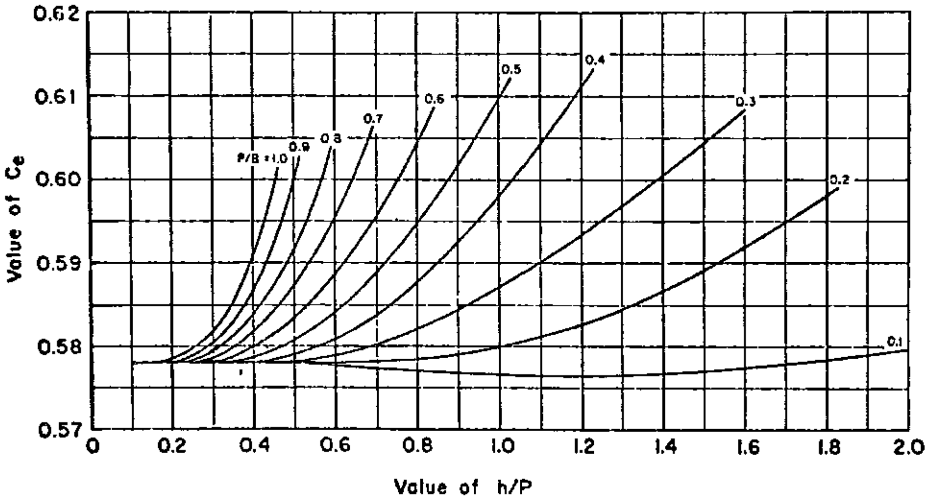


FIGURE 2.3. COEFFICIENT OF DISCHARGE C_e (θ = 90 DEGREES)

h/P and P/B using data in ref 6.

Triangular-notch weirs covering a range of values of θ from 10 to 127 deg have been studied by a large number of competent investigators. However, the range of values of P/B and h/P covered by the available data is quite limited. For such weirs, which can be described as "fully contracted," the available experimental data give the values of C_e shown in Figure 2.4 as a function of notch angle.

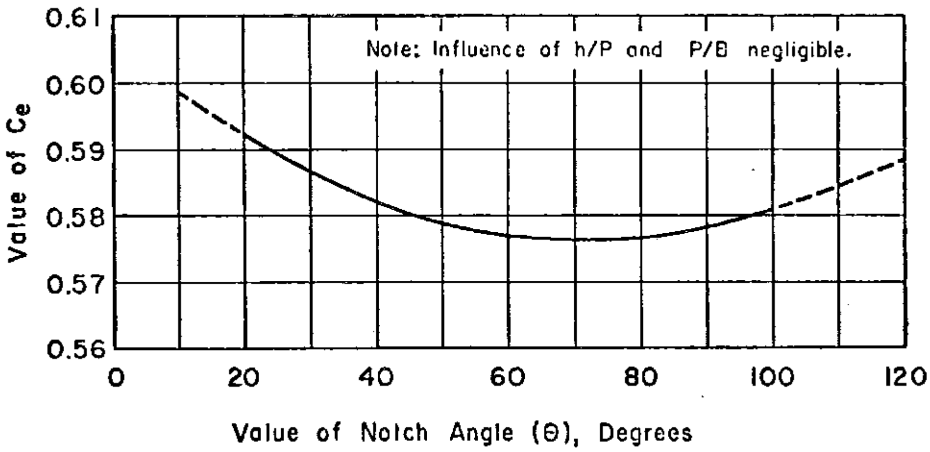


FIGURE 2.4. COEFFICIENT OF DISCHARGE C_e RELATED TO NOTCH ANGLE

Figure 2.3 might be used to indicate approximately the relative influence of h/P and P/B on the quantities shown in Figures 2.2 and 2.4, but in the absence of experimental confirmation for angles other than 90 degrees an unknown error may be introduced.

Practical limitations on the magnitude of h are related to the "clinging nappe" phenomenon which characterizes low heads. To ensure a freely discharging, stable nappe, a minimum value of h = 0.2 ft (0.06 m) is recommended for notch angles between 20 and 100 degrees.

For notch angles other than 90 degrees the following limitations apply to the application of equation 2.5 above:

a. h shall not be less than 2 in (0.05 m) nor more than 15 in (0.38 m).
b. The vertex height, P, shall exceed 1.5 ft (0.45 m).
c. h/P shall not exceed 0.4.
d. The width of the approach channel, B, shall exceed 3.0 ft (0.9 m).
e. h/B shall not exceed 0.20.

Because of the wider experimental confirmation a 90 degrees notch may continue to be used when the sides and/or floor of the approach channel have some influence on the contraction of the nappe, the limitations being less restrictive as follows:-

a. h shall be not less than 2 in (0.05 m) nor more than 24 in (0.60 m).
b. The vertex height P, shall exceed 0.5 ft (0.15 m).
c. h/P shall not exceed 1.2.
d. The width of the approach channel B shall exceed 2.0 ft (0.6 m).
e. h/B shall not exceed 0.40.

2.2. RECTANGULAR THIN-PLATE WEIRS

2.2.1. The rectangular, thin-plate weir as defined for these standards is a general classification of which the rectangular-notch weir is the basic form and the full-width or "suppressed" weir is a limiting example. Figure 2.5 shows a standard rectangular-notch weir.

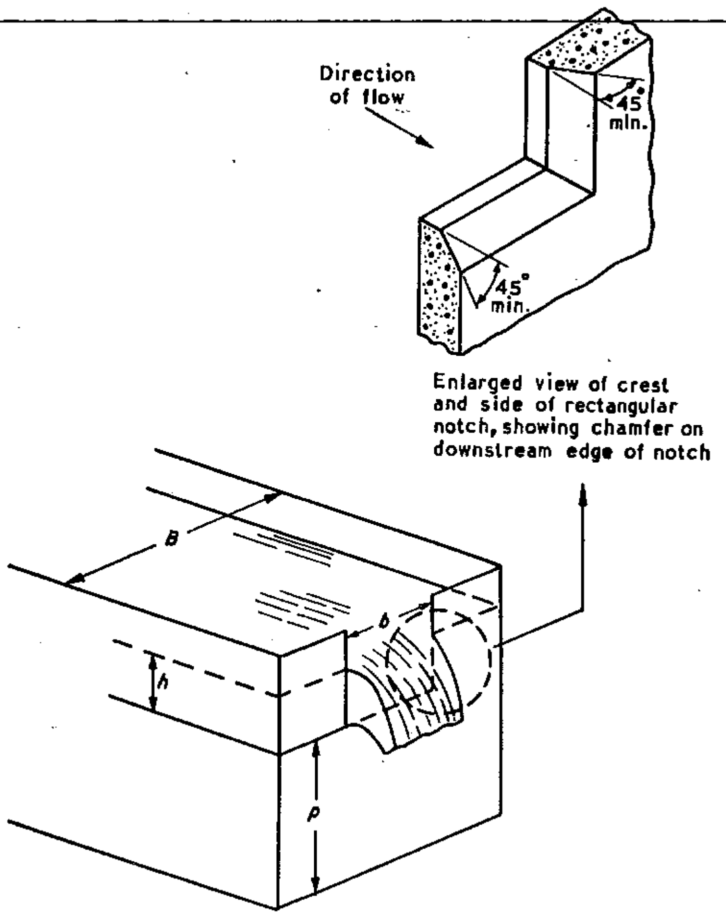


FIGURE 2.5. THE RECTANGULAR-NOTCH, THIN-PLATE WEIR

2.2.2. The standard weir shall consist of a rectangular notch symmetrically located in a vertical, thin-plate. The plate shall be smooth and plane, especially on the upstream side, and it shall be perpendicular to the sides as well as the bottom of the channel. The crest of the weir notch shall be a horizontal plane surface, which shall form a sharp right-angle corner at its intersection with the upstream face of the weir plate. The sides of the notch shall be vertical plane surfaces which shall make sharp 90-degree intersections with the plane of the crest, and which, in shape and width, shall be identical with the crest surface.

2.2.3. The upstream edges of the sides and crest of the weir must be sharp; that is, they shall be machined or filed perpendicular to the upstream face of the weir plate, free of burrs and scratches. Abrasive cloth or paper must not be used because of the danger of rounding the edge. The downstream edges of the notch shall be chamfered if the weir plate is thicker than the allowable crest width. The surfaces of the chamfers shall make an angle of not less than 45 degrees with the surfaces of the notch.

2.2.4. If the length of the crest be equal to the width of the channel (ie a full-width weir), it is especially important that the sides of the channel be vertical, plane, parallel and smooth in the near vicinity of the weir. The sides of the channel above the level of the crest of a full-width weir shall extend at least 0.3 ft (0.09 m) beyond the plane of the weir.

2.2.5. The discharge nappe shall be fully ventilated and unsubmerged. Provisions for ventilation of the nappe should ensure that the pressure over the sides and nappe surfaces is atmospheric. The tailwater level should be low enough to ensure that it shall not interfere with the ventilation or free discharge of the jet. The nappe shall not be permitted to cling to the downstream face of the weir.

2.2.6. The head on the weir shall be measured a sufficient distance upstream from the weir to avoid the regions of surface draw-down. On the other hand, it shall be close enough to the weir for the energy loss between the section of measurement and the weir to be negligible. The head measurement section shall be located a distance equal to between three and four times the maximum head (3 to $4h_{max}$) upstream from the weir.

2.2.7. Basic Discharge Equation:

The basic equation of discharge for rectangular, thin-plate weirs is the equation attributed to Poleni and Dubuat,

$$Q = C \frac{2}{3} \sqrt{2g}\, bh^{3/2} \tag{2.8}$$

in which Q is the volume rate of flow or discharge (cu ft per sec or cu m per sec), C is the coefficient of discharge (nondimensional), g is the acceleration due to gravity (ft per sec per sec or m per sec per sec), b is the width of the notch (ft or m), and h is the piezometric head or height of the upstream liquid surface referred to the level of the crest (ft or m).

2.2.8. In Eq 2.8, C is essentially a function of two geometric ratios and two fluid-property ratios. Both fluid-property ratios can be replaced with the quantities h and b for a given fluid. It follows that, for a single liquid and a limited range of temperatures,

$$C = f_2\left(\frac{b}{B}, \frac{h}{P}, h, b\right) \tag{2.9}$$

It has been shown (ref 11) that the relatively minor effects represented by h and b in Eq (2.9) can be evaluated by adjustment of their measured values. The result is a modified equation of discharge

$$Q = C_e \frac{2}{3} \sqrt{2g}\, b_e\, h_e^{3/2} \text{ (Kindsvater-Carter)} \tag{2.10}$$

in which C_e is a function of the two remaining geometric ratios alone, $C_e = F\left(\frac{b}{B}, \frac{h}{P}\right)$ (2.11)

whereas, b_e, the effective width, is $b_e = b + k_b$, (2.12)

and h_e, the effective head, is $h_e = h + k_h$ (2.13)

In Eqs 2.12 and 2.13, k_b and k_h are absolute measures (ft or m) of the combined effects of viscosity and surface tension.

2.2.9. Evaluation of C_e , k_b and k_h for Water:

Values of the coefficients required in the above equations are available from many series of experiments (refs 12, 13, 14, 15). The analysis and experiments in ref 10 show the relationship between C_e and h/P to be linear for all values of b/B. Thus, for the full-width or "suppressed" weir (b/B = 1.0),

$$C_e = 0.602 + 0.075 \frac{h}{P} \qquad (2.14)$$

Similar equations for other values of b/B are:

$$(b/B = 0.9):\ C_e = 0.598 + 0.064 \frac{h}{P} \qquad (2.15)$$

$$(b/B = 0.8):\ C_e = 0.596 + 0.045 \frac{h}{P} \qquad (2.16)$$

$$(b/B = 0.7):\ C_e = 0.594 + 0.030 \frac{h}{P} \qquad (2.17)$$

$$(b/B = 0.6):\ C_e = 0.593 + 0.018 \frac{h}{P} \qquad (2.18)$$

$$(b/B = 0.4):\ C_e = 0.591 + 0.0058 \frac{h}{P} \qquad (2.19)$$

$$(b/B = 0.2):\ C_e = 0.589 - 0.0018 \frac{h}{P} \qquad (2.20)$$

$$(b/B \longrightarrow 0):\ C_e = 0.587 - 0.0023 \frac{h}{P} \qquad (2.21)$$

Figure 2.6 shows values of k_b derived from the experiments

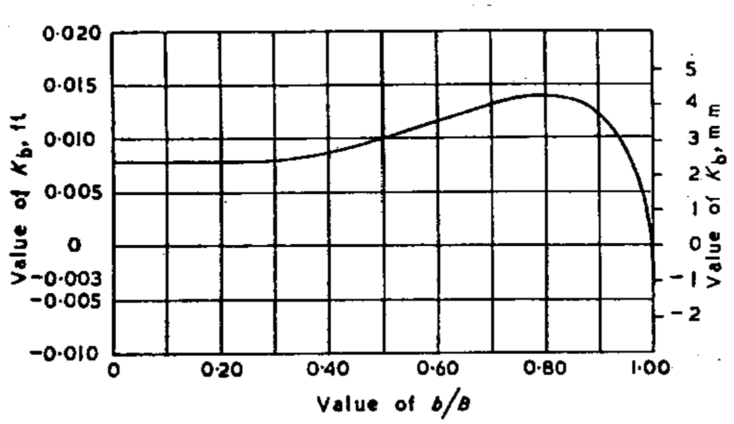


FIGURE 2.6. VALUE OF k_b RELATED TO b/B

reported in ref 10. The figure indicates that k_b is a function of b/B, and that it reaches a maximum positive value of 0.014 ft (4.3 mm) at b/B = 0.8 and a maximum negative value of −0.003 ft (0.9 mm) at b/B = 1.0.

A constant, positive value of k_b = 0.003 ft (0.9 mm) was found to be applicable to the full variety of conditions investigated and is recommended for use with the standard, rectangular, thin-plate weir.

2.2.10. Limitations on h/P, h, b and P for the Standard Weir:

Practical limitations on h/P are related to the observation that head-measurement difficulties and errors result from surges and waves which occur in the approach channel at larger values of h/P (in combination with larger values of b/B). For full-width weirs (b/B = 1.0), the recommended maximum value of h/P is 2.0. Although limitations on h/P corresponding to smaller values of b/B have not been established it may be assumed that the same limitation applies.

Practical limitations on the magnitude of h are related to the nappe "clinging" phenomenon which characterize very low heads. To insure a freely discharging, stable nappe, a minimum value of h=0.1 ft (0.03 m) is recommended.

Limitations on the magnitude of b are related to uncertainties regarding the surface tension and viscosity effects represented by the quantity k_b. A tentative minimum value of b = 0.5 ft (0.15 m) is recommended.

There is disagreement in the literature regarding an independent correlation between the weir height (P) and the coefficient of discharge. It is acknowledged, however, that inaccuracies of measurement are associated with low values of P, especially in combination with high values of h/P. It is recommended that P be limited to values greater than 0.3 ft (0.09 m).

The specifications for a standard installation include the requirement that the velocity in the channel upstream from the weir be such as to simulate the normal velocity distribution in a smooth, horizontal, rectangular channel. The recommended values of C_e, k_b and k_h were derived from laboratory tests which satisfied this requirement.

2.2.11. Alternative equations for rectangular thin-plate weirs:

A number of discharge formulae have been derived for particular categories of rectangular thin-plate weirs, based on experimental work. Several of these are acknowledged to provide a reliable figure for the discharge, provided the limitations for the particular equations are observed. The Rehbock equation for full-width weirs for example, has an established reputation, and equations for other categories of weir have been standardized in Europe.

2.2.12. The full-width weir:

A full-width weir is one which extends across the full-width of the approach channel (B/b = 1.0). This weir is also referred to as a suppressed weir.

The discharge equation is:

$$Q = \frac{2}{3}\sqrt{2g}\, C_D\, b\, h_c^{3/2} \qquad (2.23)$$

in which $C_D = 0.602 + 0.083$ h/P (Rehbock, ref 16) (2.24)

where h is the gauged head,
P is the weir height.

$$h_c = h + 0.004 \text{ ft } (0.0012 \text{ m}) \qquad (2.25)$$

The general installation conditions shall apply, and the following practical limitations on h/P, h, b and P shall be strictly observed in the application of this equation.

a. h/P shall not exceed 1.0.
b. The head h shall be between 0.1 ft and 2.5 ft (0.03 m to 0.75 m).
c. The weir width b shall be at least 1.0 ft (0.03 m).
d. The weir height P shall be not less than 0.3 ft (0.10 m).

2.2.13. Weirs with fully developed contractions:

Fully developed contractions occur when the bed and walls of the channel upstream of the weir are sufficiently remote from its crest and sides for the channel boundaries to have no significant influence on the contractions of the nappe. The alternative discharge equation for rectangular thin-plate (square-edged) weirs with fully developed contractions is:

$$Q = \frac{2}{3}\sqrt{2g}\ C_D\ b\ h^{3/2} \qquad (2.26)$$

in which $C_D = 0.616\ (1 - 0.1h/b)$(Hamilton-Smith, ref 17) (2.27)

where h is the head of water above the crest,
b is the width of the notch.

The general installation conditions shall apply and the following limitations on h/b, h, b, B-b and P shall be observed in the application of this equation.

a. The walls of the approach channel shall not be less than twice the maximum head from the vertical lips (ie $\frac{B-b}{2}$ shall be greater than 2h).
b. The height of the weir shall not be less than twice the maximum head.
c. h/b shall not exceed 0.5.
d. The head h shall be between 0.25 ft and 2.0 ft (0.075 to 0.60 m).
e. The weir width b shall be at least 1.0 ft (0.30 m).
f. ~~The weir height P shall be not less than 1.0 ft (0.30 m).~~
g. When B(h + P) is less than 10 bh, the approach velocity is not negligible, and this may be allowed for by replacing h in the equation quoted, by h^1:

thus $h^1 = h + 1.4\left(\frac{V_a^2}{2g}\right)$ where V_a is the mean velocity in the approach channel.

Provided the approach channel is sufficiently large to render the velocity of approach negligible, and the weir complies also with conditions a to g above, the shape of the approach channel is unimportant. The fully-contracted form of weir may be used with non-rectangular approach channels under these circumstances.

2.2.14. Partially contracted weirs:

If, due to the proximity of the walls and/or the bed of the approach channel, the contractions are not fully developed, the weir is defined as a partially contracted weir. The alternative discharge equation for partially contracted thin-plate weirs is:

$$Q = \frac{2}{3}\sqrt{2g}\ C_D b\ h^{3/2} \qquad \text{in which } C_D \text{ is the coefficient of discharge.} \qquad (2.27)$$

Where h is in feet

$$C_D = \left\{0.578 + 0.037\left(\frac{b}{B}\right)^2 + \frac{0.01187 - 0.00985\left(\frac{b}{B}\right)^2}{h + 0.005}\right\}\left\{1 + 0.5\left(\frac{b}{B}\right)^4\left(\frac{h}{h+P}\right)^2\right\} \qquad (2.28)$$

OR where h is in metres

$$C_D = \left\{0.578 + 0.037\left(\frac{b}{B}\right)^2 + \frac{0.00365 - 0.0030\left(\frac{b}{B}\right)^2}{h + 0.0016}\right\}\left\{1 + 0.5\left(\frac{b}{B}\right)^4\left(\frac{h}{h+P}\right)^2\right\} \qquad (2.29)$$

(Association Suisse de Normalisation ref 18)

The general installation conditions shall apply. The following limitations on h/P, b/B, h, b and P are to be strictly observed in the application of this equation:

a. h/P shall not exceed 1.0.
b. The head h shall exceed 0.08 x b/B ft (0.025 x b/B m) but be less than 2.6 ft (0.8 m).
c. b/B shall not be less than 0.3.
d. The weir height P shall be at least 1.0 ft (0.30 m).

3. BROAD-CRESTED AND OTHER LONG-BASE WEIRS

3.1. GENERAL EQUATIONS

3.1.1. Critical depth theory, coupled with experiment, has shown that the discharge over a round-nose broad-crested weir may be represented by the following equation and although the theory is not strictly applicable to triangular profile and rectangular profile weirs, this formula is used as the basic equation for all long-base weirs.

$$Q = C_D\left(\frac{2}{3}\right)^{3/2} \sqrt{(g)}\, b\, H^{3/2} \tag{3.1}$$

where

Q = discharge

C_D=coefficient of discharge (non dimensional)

g = gravitational acceleration

b = width of weir crest

H = total head

3.1.2. Since the total head, H, cannot be measured in practice, the discharge equation in terms of gauged head, h, may be written as follows:

$$Q = C_v\ C_D\left(\frac{2}{3}\right)^{3/2} \sqrt{(g)}\, b\, h^{3/2} \tag{3.2}$$

where C_v is a further dimensionless coefficient allowing for the effect of approach velocity on the measured water level upstream of the weir. By definition

$$C_v = \left(\frac{H}{h}\right)^{3/2} \tag{3.3}$$

3.1.3. The total head is related to the gauged head through:

$$H = h + \alpha\, V_a^2/2g \tag{3.4}$$

where V_a is the mean velocity in the approach channel at the gauging section, and α is a coefficient (the kinetic energy or Coriolis coefficient) which takes account of the fact that the kinetic energy head exceeds $V_a^2/2g$ if the velocity distribution across the section is not uniform. In applying the equations in this Standard, α may be taken as unity.

3.1.4. From equations (3.2) (3.3) and (3.4), it may be deduced (ref 19) that

$$(C_v^{2/3} - 1)^{1/2} = 2\, C_v\, C_D\, bh/3\sqrt{(3)}\, A \tag{3.5}$$

where A is the cross-sectional area of the approach channel below the observed water level, at the gauging section. Thus C_v may be deduced in terms at $C_D bh/A$. To avoid the complicated solution of equation (3.5) in deducing C_v, Figure 3.2 has to be prepared giving the relation between C_v and $C_D bh/A$. The value of the coefficient C_D depends on the profile of the weir crest, and is dealt with later under the headings of the three types of weir. Equation 3.5, and hence Figure 3.2 are not applicable to compound weirs.

3.1.5. For a compound weir,

$$Q = C_{D1}\left(\frac{2}{3}\right)^{3/2} \sqrt{(g)}\, b_1 H_1^{3/2} + C_{D2}\left(\frac{2}{3}\right)^{3/2} \sqrt{(g)}\, b_2 H_2^{3/2} + \text{etc.} \tag{3.6}$$

where suffixes 1, 2 etc refer to the sections of crest at different elevations.

3.1.6. Equation (3.4) may alternatively be expressed as

$$H = h + Q^2/(2g\ A^2) \tag{3.7}$$

which is applicable generally to all gauging structures. Equation (3.6) may then be written:

$$\begin{aligned} Q = {} & C_{D1}\left(\frac{2}{3}\right)^{3/2} \sqrt{g}\ b_1\ [h_1 + Q^2/(2g\ A^2)]^{3/2} \\ & + C_{D2}\left(\frac{2}{3}\right)^{3/2} \sqrt{g}\ b_2\ [h_2 + Q^2/(2g\ A^2)]^{3/2} \\ & + \text{etc.} \end{aligned} \tag{3.8}$$

This can be solved by successive approximations by first putting $Q = O$ in the terms in square brackets [], and thus deducing a first approximation to the total discharge Q_1. A second approximation Q_2 is obtained by using Q_1 in the term in square brackets, and so on.

In general:

$$\begin{aligned} Q_n = {} & C_{D1}\left(\frac{2}{3}\right)^{3/2} \sqrt{g}\ b_1\ [h_1 + Q^2_{n-1}/(2g\ A^2)]^{3/2} \\ & + C_{D2}\left(\frac{2}{3}\right)^{3/2} \sqrt{g}\ b_2\ [h_2 + Q^2_{n-1}/(2g\ A^2)]^{3/2} \\ & + \text{etc.} \end{aligned} \tag{3.9}$$

The successive approximation procedure must be continued until the difference between Q_n and Q_{n-1} is well within the required precision.

3.2. ROUND-NOSE HORIZONTAL-CREST WEIRS

3.2.1. The standard weir comprises a truly level and horizontal crest, between vertical abutments. The upstream corner should be rounded in such a manner that flow separation does not occur, and downstream of the horizontal crest there should be either i. a rounded corner, ii. a downward slope or iii. a vertical face. The weir should be set at right angles to the direction of flow in the approach channel. The dimensions of the weir and its abutments shall conform with the requirements indicated in Figure 3.1. The radius of the upstream nose must not be less than $0.2\ H_{max}$. The length of the horizontal portion of the weir crest should not be less than $1.75\ H_{max}$ nor should the sum of the crest length and nose radius be less than $2.25\ H_{max}$.

3.2.2. The head on the weir should be measured at a point far enough upstream of the crest to be clear of the effects of draw-down, but close enough to the weir that the energy loss between the section of measurement and the upstream edge of the weir crest shall be negligible. It is recommended that the head-measurement section be located a distance of between two and three times H_{max} upstream of the weir block.

3.2.3. Flow is modular when it is independent of variations in tailwater level. For this to occur, assuming subcritical conditions in the tailwater channel, the tailwater total head level must not rise beyond a certain percentage of H above crest height. With a vertical downstream face, this percentage is dependent on H/P: 66 per cent for low values of H/P^1, rising to 75 per cent at H/P^1 of 0.5 and 80 per cent at H/P^1 of 1.0 and over. For a gently curved or sloping downstream face, the modular limit may be taken as 5 per cent higher throughout. In the above, P^1 is the height of the crest above downstream bed level.

3.2.4. The basic discharge equation is given in 3.1 in terms of both total and gauged head. Eq 3.2 may be used to evaluate discharge, with the appropriate value of C_v read from Figure 3.2.

3.2.5. For water at ordinary temperatures, C_D is a function of head, h, the crest length in the direction of flow, the roughness of the crest, and the ratio h/b. It may be expressed by the following equation which takes account of the development of a boundary layer on the crest (after Ippen see refs 20, 21, 22, 23)

$$C_D = \left(1 - \frac{2xL}{b}\right)\left(1 - \frac{xL}{h}\right)^{3/2} \tag{3.10}$$

where L is the length of the horizontal section of the crest in the direction of flow, and x is a factor which allows for the influence of the boundary layer on the crest and abutments.

For well-finished installations such as might be used in a laboratory for gauging clean water, $x = 0.003$. For field installations of well-finished concrete or similar construction, $x = 0.005$.

3.2.6. The practical lower limit of h is related to the magnitude of the influence of fluid properties and boundary roughness. The recommended lower limit is 0.06 m (0.2 ft) or 0.03 L, whichever is the greater.

The limitations on H/P arise from difficulties experienced when the Froude number in the approach channel exceeds 0.5, coupled with inadequate experimental confirmation at high values of H/P. The recommended upper limit is H/P = 2.5.

The limitation on H/L arises from the necessity to ensure sensibly hydrostatic pressures at the critical section on the crest: H/L should not exceed 0.6.

The height of the weir, P, should not be less than 0.15 m (0.5 ft). The crest width b must not be less than 0.30 m (1 ft), nor less than H_{max} nor less than L/5.

3.3. TRIANGULAR PROFILE CRUMP WEIRS

3.3.1. A weir with a triangular profile in the direction of flow provides an economic structure for gauging river discharge with little afflux, especially if it is provided with facilities for double gauging and can thus operate drowned. For this purpose, the downstream measurement point may be located either just beyond the crest, tapping into the separation pocket, or beyond the weir in the more conventional way. (See refs 24, 25, 26).

3.3.2. The standard Crump weir comprises an upstream slope of 1 (vertical) to 2 (horizontal) and a downstream slope of 1 (vertical) to 5 (horizontal). The intersection of these two surfaces forms a straight line crest, horizontal and at right angles to the direction of flow in the approach channel. Particular attention should be given to the crest itself, which should possess a well-defined corner of durable construction. The crest may be made of precast concrete sections, carefully aligned and jointed or may have a non-corrodible metal insert, as an alternative to in-situ construction throughout. The dimensions of the weir and its abutments shall conform with the requirements indicated in Figure 3.3. Weir blocks may be truncated but not so as to reduce their dimensions in plan to less than 1.0 H (maximum) for the 1:2 slope, and 2.0 H for the 1:5 slope.

3.3.3. The head on the weir should be measured at a point far enough upstream of the crest to be clear of the effects of draw-down, but close enough to the weir that the energy loss between the section of measurement and the upstream edge of the weir shall be negligible. This condition is satisfied if the head measurement section is at a distance $\frac{H_{max}}{P} \times H_{max}$ upstream of the toe of the structure. In some practical situations very high accuracy is not needed at the maximum discharge to be gauged, and in order to save expense the head measurement section may then be brought closer to the toe of the weir than $\frac{H^2_{max}}{P}$, provided it is not closer than $\frac{1}{2}H_{max}$.

(For a truncated weir block the reference point, ie the upstream toe of the 1:2 slope, should be where the 1:2 slope if produced downwards would meet the bed; and for a compound weir, the reference point should be the upstream toe of the weir block which extends furthest upstream).

3.3.4. Flow is modular when it is independent of variations in tailwater level. For all flow conditions the tailwater total head level must not rise beyond 75 per cent of the upstream total head H above crest height, if the flow is not to be affected by more than one per cent, for subcritical conditions in tailwater channel.

3.3.5. The basic discharge equation is given in 3.1 in terms of both total and gauged head, Eq 3.2 may be used to evaluate discharge with the appropriate value of C_v read from Figure 3.2. For water at ordinary temperatures C_D is independent of h except at very low heads when fluid properties influence the coefficient. Under modular conditions for h > 0.06 m (0.2 ft), C_D is constant and equals 1.150. For h < 0.06 m (0.2 ft) C_D is given by the following equations.

$$C_D = 1.150\ (1 - 0.0003/h)^{3/2} \text{ where h is in metres} \tag{3.11}$$

or

$$C_D = 1.150\ (1 - 0.001/h)^{3/2} \text{ where h is in feet} \tag{3.12}$$

(ref 25)

3.3.6. When the weir is drowned, the coefficient of discharge is affected by the ratio H_d/H where H_d is the total energy level, relative to crest height, beyond the weir. It has also been established (ref 24) that there is a unique relationship between the coefficient of discharge and the ratio h_p/H, where h_p is the piezometric pressure measured by a tapping just downstream of the crest into the separation pocket (see Fig 3.4). The coefficient of discharge is reduced by a factor f, which can be read from Figure 3.5 in terms of h_p/H or H_d/H, depending on the method of measurement used downstream. To avoid successive approximation in deducing total head, Figure 3.6 may be used to evaluate f from h_p/h. In applying equation 3.2 it should be noted that the reduced coefficient value of fC_D, must be used when evaluating C_v from Figure 3.2.

3.3.7. The practical lower limit of 'h' is related to the magnitude of the influence of fluid properties and boundary roughness. For a well-maintained weir with a crest section of smooth metal or its equivalent, the recommended lower limit of 'h' is 0.03 m (0.1 ft); for a weir with a crest section of fine concrete or materials having similar texture, the recommended lower limit of 'h' is 0.06 m (0.2 ft).

The limitations of H/P arise from difficulties experienced when the Froude number in the approach channel exceeds 0.5 coupled with inadequate experimental confirmation at high values of the ratio H/P. The recommended upper limit of H/P is 3.0.

The height of the weir, P, should not be less than 0.06 m (0.20 ft). The limit of b and b/H relates to boundary layer effects at the sides of the weir, for which no allowance is made in the formulae given. To reduce these to an acceptable value, b should not be less than 0.3 m (1 ft) and b/H should not be less than 2.0.

3.4. RECTANGULAR PROFILE WEIRS

3.4.1. This type of weir has been intensively studied over the years, (refs 27, 28, 29, 30, 31). The crest of the weir shall be a horizontal, rectangular plane surface and may be constructed in concrete with a smooth cement finish or surfaced with a smooth non-corrodible material (see Fig 3.7). The upstream and downstream end faces of the weir shall be smooth, vertical plane surfaces, and they shall be perpendicular to the sides and bottom of the channel in which the weir is located. The upstream face, in particular, shall form a sharp, right-angle corner at its intersection with the plane of the crest.

There is no limitation on the ratio of $\frac{L}{P}$ defining the geometric proportions of the weir. It is essential, however, that the geometry should comply with the limitations on $\frac{h}{h + P}$ and h/L given later. For flow measurement in channels, $\frac{L}{P} = 2$ may be a convenient value to adopt.

3.4.2. The head on the weir should be measured at a point far enough upstream of the crest to be clear of the effects of draw-down, but close enough to the weir that the energy loss between the section of measurement and the upstream edge of the weir shall be negligible. This condition is satisfied if the gauge point is at a distance of between 2 H_{max} and 3 H_{max} upstream of the upstream face of the weir.

3.4.3. Flow is modular when it is independent of variations in tailwater level. For this to occur, assuming sub-critical conditions in the tailwater channel, the tailwater total head level must not rise beyond 66 per cent of the upstream total head H above crest height.

3.4.4. The basic discharge equation is given in 3.1 in terms of both total head and gauged head. Eq 3.2 may be used to evaluate discharge, with the appropriate value of C_v read from Figure 3.2.

The coefficient of discharge for the standard weir is a function of five independent, nondimensional ratios, three being geometric ratios and two fluid property ratios involving viscosity and surface tension.

By specifying limits on h, b, and P such that viscosity and surface tension are not significant factors and because it has been found, reference 27, that h/b has a negligible influence on the discharge coefficient for the standard weir, C_D may be reduced to a function of two variables

$$C_D = f\left[\frac{h}{L}, \left(\frac{h}{h + P}\right)\right] \tag{3.13}$$

within the limits indicated in 3.4.6.

It is convenient to sub-divide the coefficient data into two distinct ranges:

a. the constant coefficient range

and

b. the variable coefficient range.

3.4.5. The value of the discharge coefficient C_D remains constant in only a limited range of h/L and h/(h + P) values, these limits being

$$0.08 \leq h/L \leq 0.33$$
$$0.18 \leq h/(h + P) \leq 0.36$$

The mean coefficient value within the constancy limits is 0.848 and is known as the basic coefficient. To obtain the actual coefficient in the variable range, it is necessary to multiply the basic coefficient by a correction factor, F. The values for the correction factor, to allow for the influences of either h/L or h(h + P) or both, are shown in Figure 3.8 (Singer, ref 31).

3.4.6. The following general limitations are to be observed:

$h \geq 0.06$ m (or 0.2 ft)
$b \geq 0.3$ m (or 1 ft)
$P \geq 0.15$ m (or 0.5 ft)
$0.08 \leq h/L \leq 0.85$
$0.18 \leq h/(h + P) \leq 0.6$

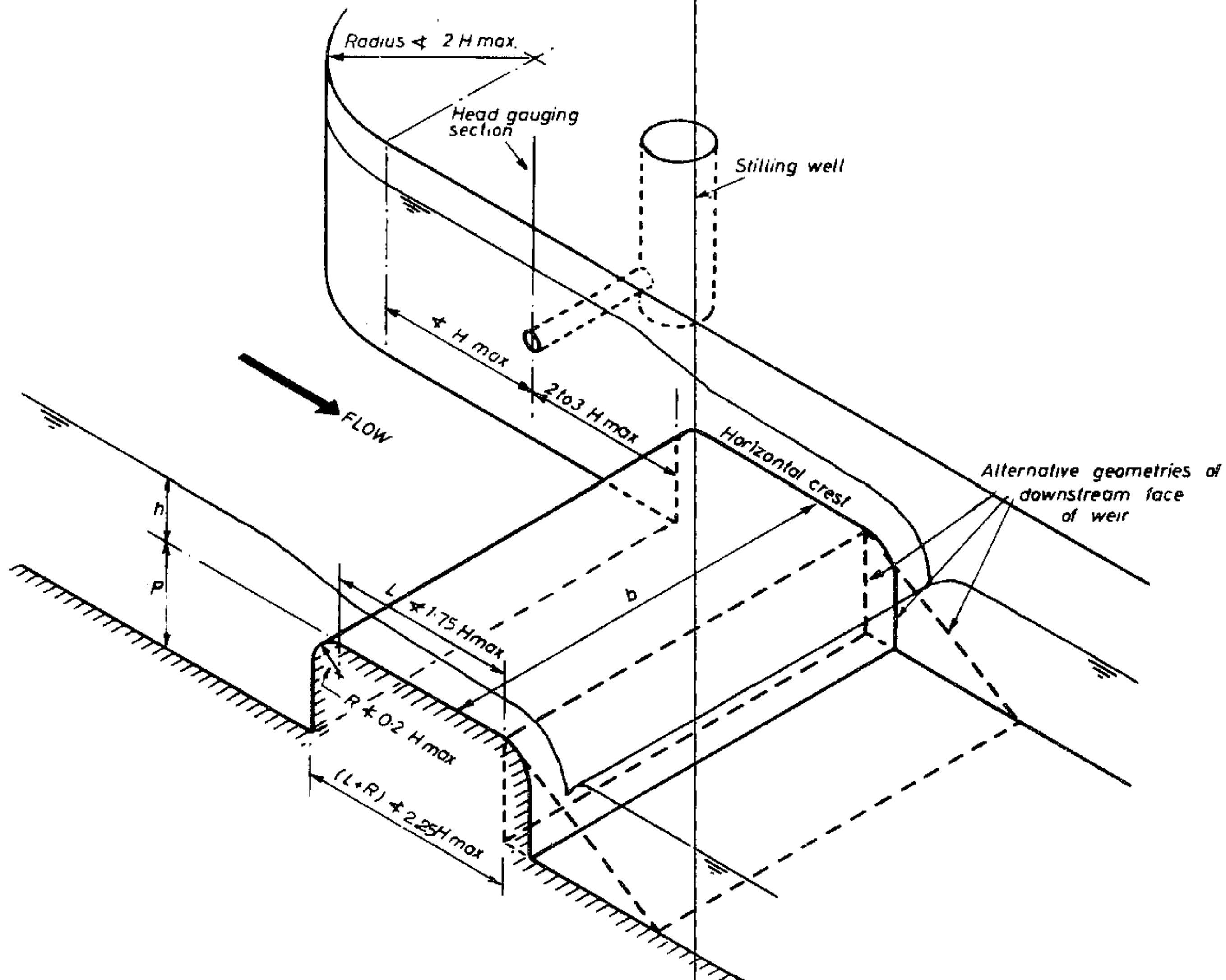


FIGURE 3.1. ROUND-NOSE HORIZONTAL-CREST WEIR

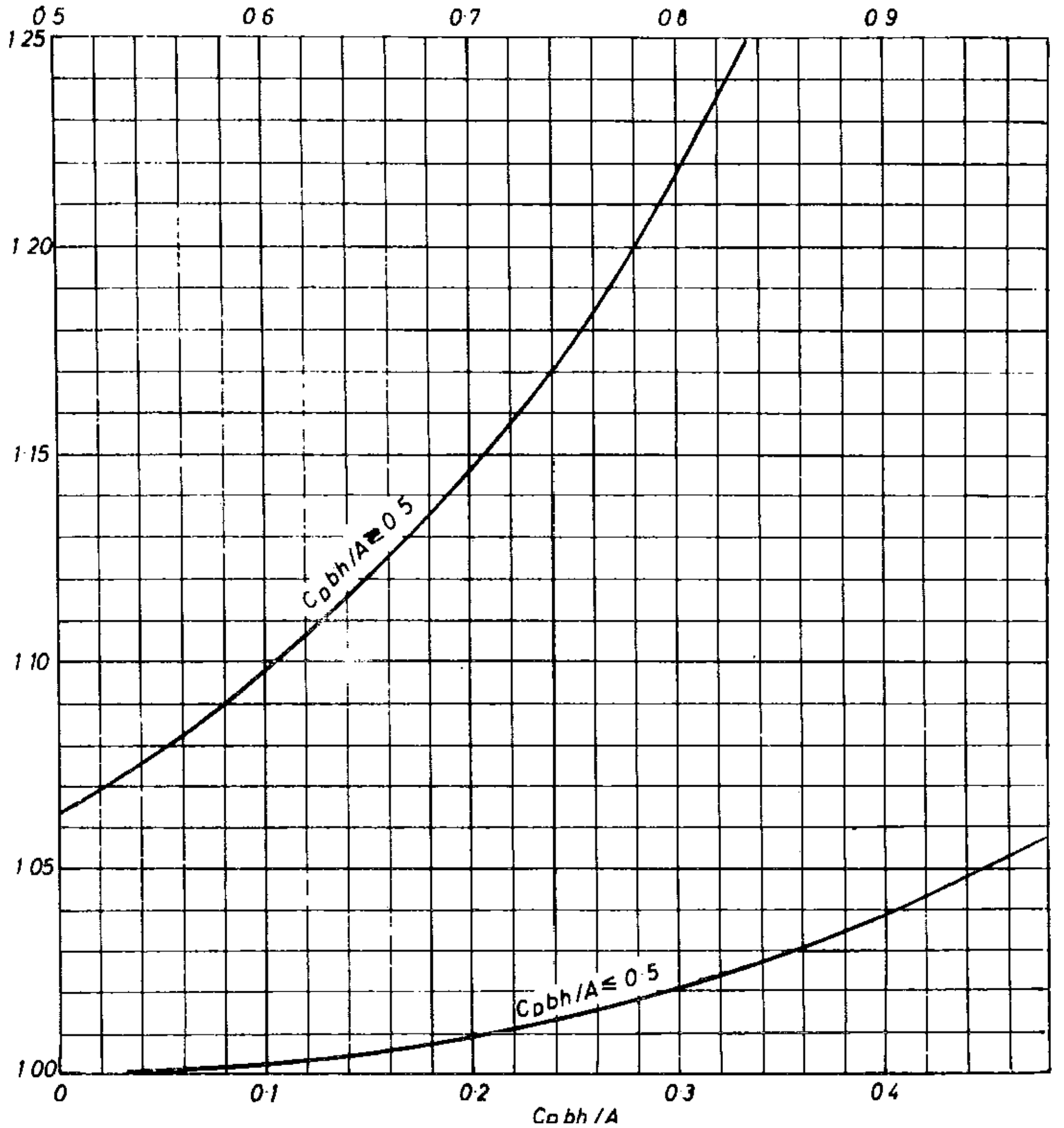


FIGURE 3.2. COEFFICIENT OF APPROACH VELOCITY, C_V

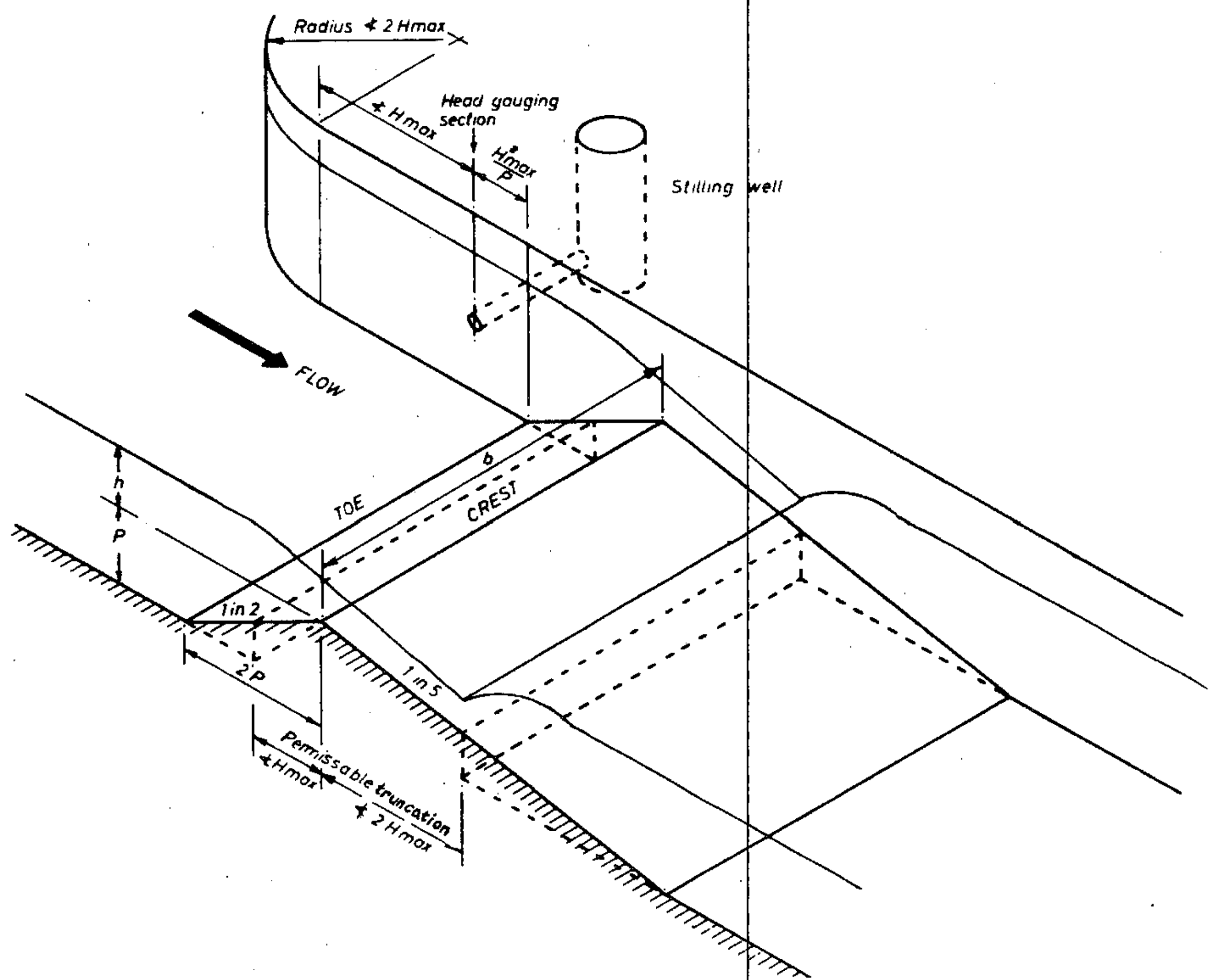


FIGURE 3.3. TRIANGULAR PROFILE WEIR

CRUMP WEIR

SUGGESTED DESIGN CAPACITY 2000 CUSECS

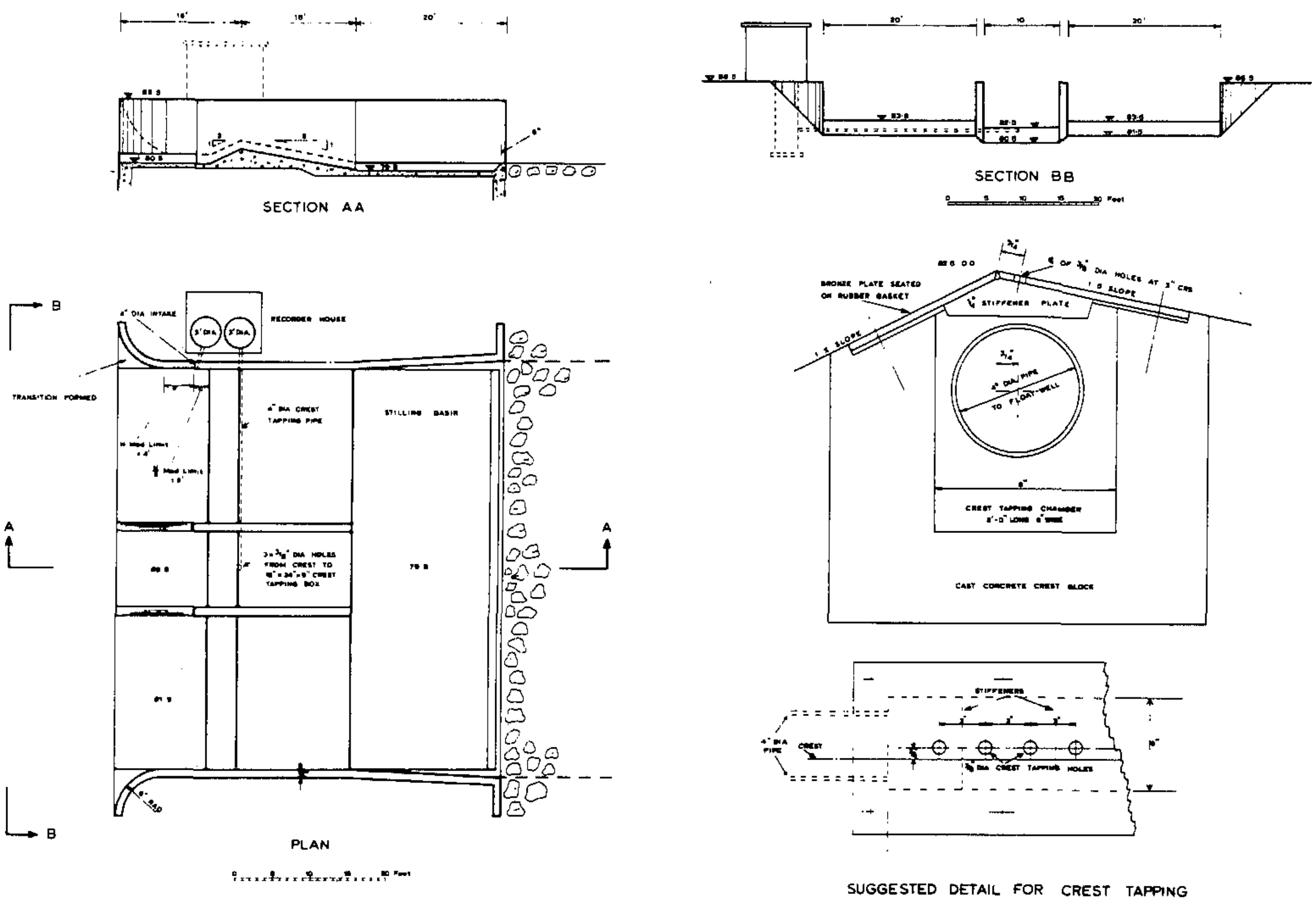


FIGURE 3.4. COMPOUND CRUMP WEIR, WITH DETAIL OF CREST TAPPING

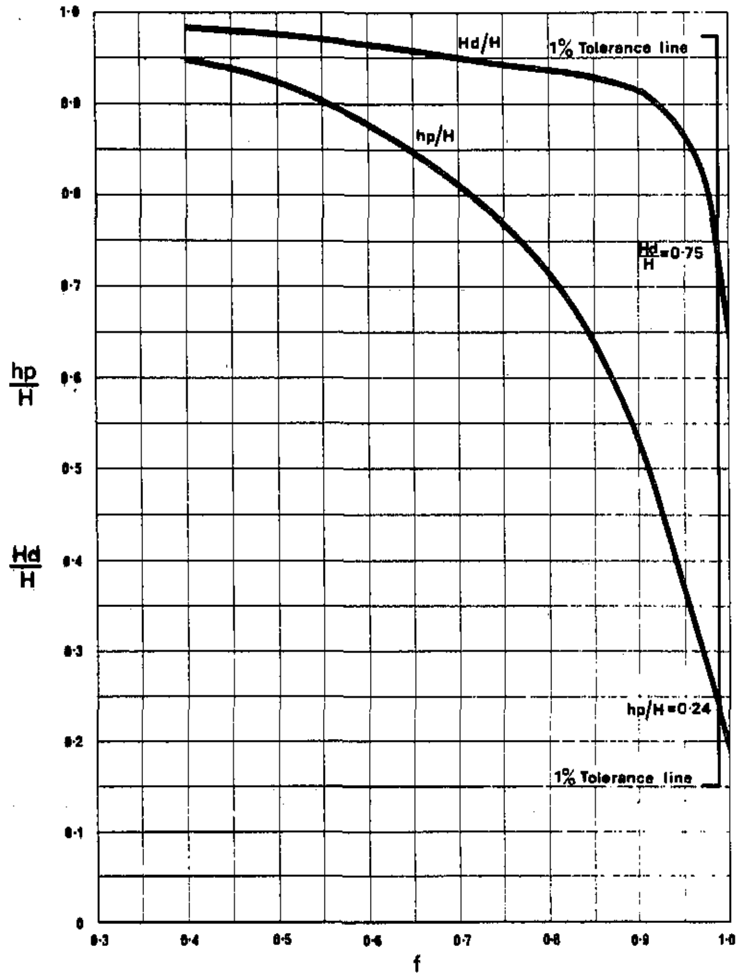


FIGURE 3.5. DROWNED FLOW REDUCTION FACTOR FOR CRUMP WEIR IN TERMS OF h_p/H and H_d/H

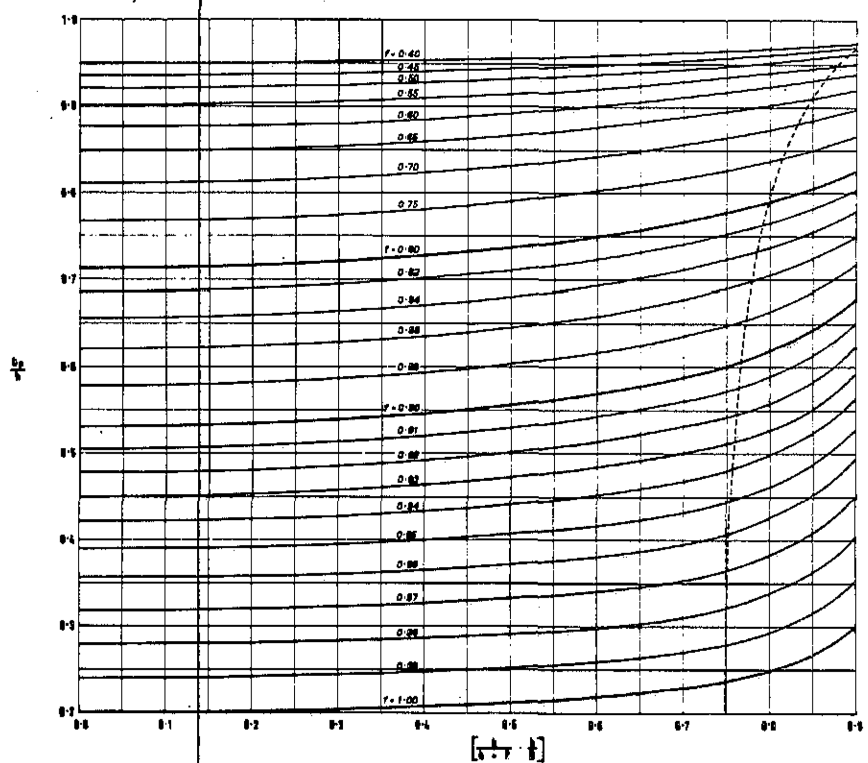


FIGURE 3.6. DROWNED FLOW REDUCTION FACTOR FOR CRUMP WEIR IN TERMS OF h_p/h and $h/(h+p)$

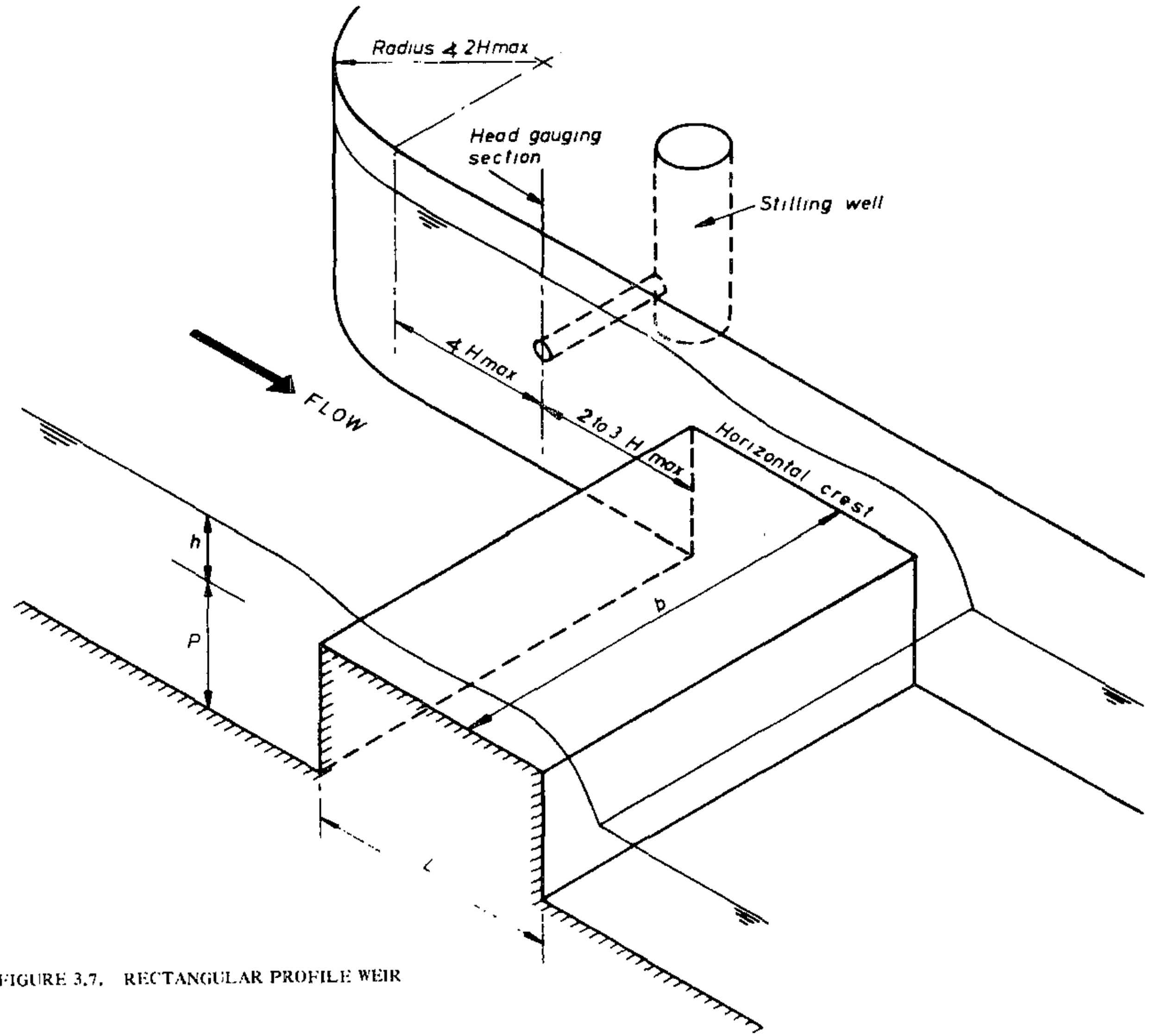


FIGURE 3.7. RECTANGULAR PROFILE WEIR

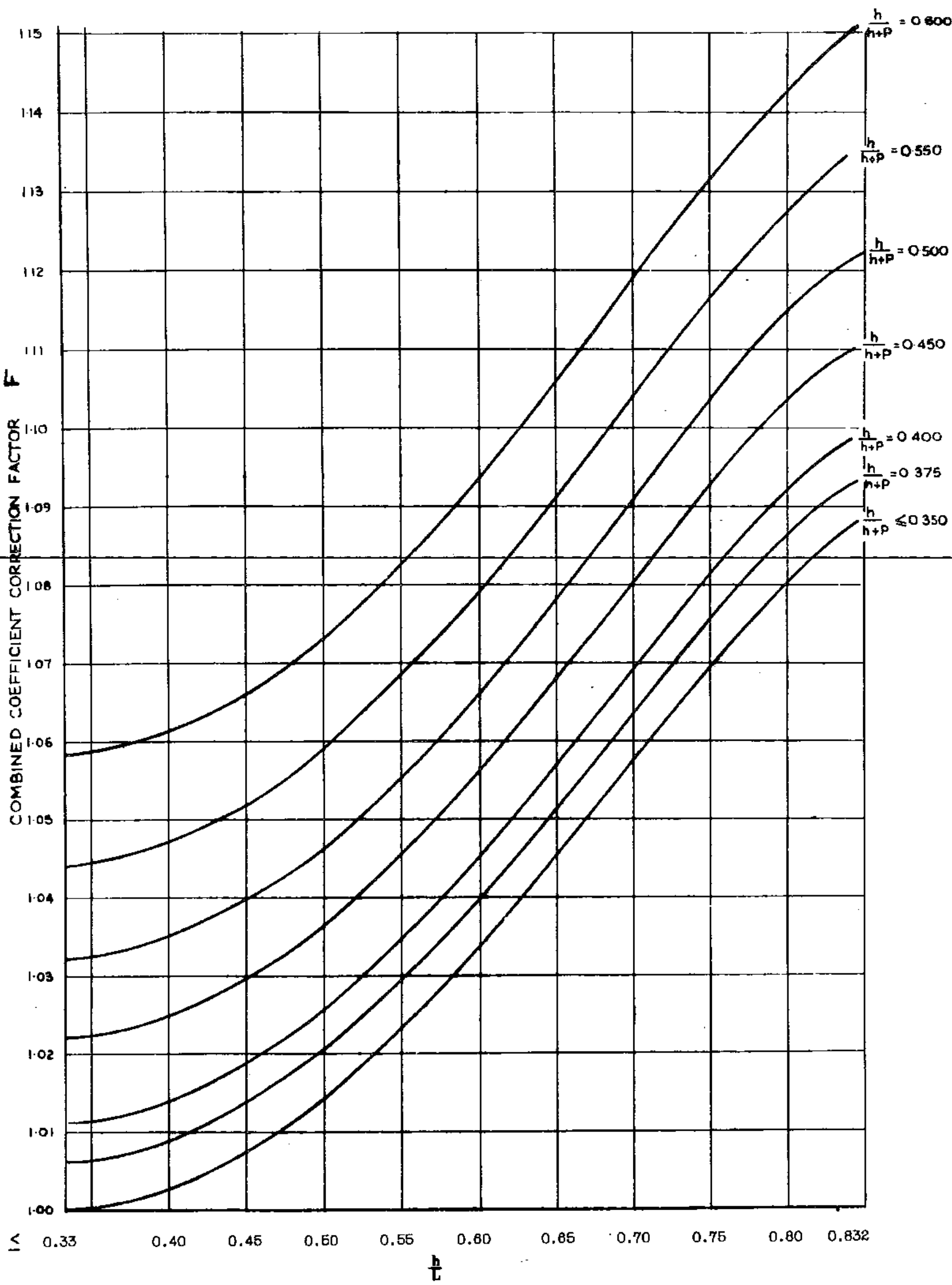


FIGURE 3.8. RECTANGULAR PROFILE WEIRS, COMBINED COEFFICIENT CORRECTION FACTOR FOR THE $\frac{h}{L}$ and $\frac{h}{h+P}$ PARAMETERS

4. STANDING WAVE FLUMES

4.1. GENERAL

4.1.1. A standing-wave flume is essentially a streamlined structure built into an open channel to form a contraction through which the velocity of the flowing water is increased with a consequent fall in water level. A wide variety of flumes have been designed but only the three which will cover a wide range of use and which have received acceptance and field testing will be presented. Selection of the flume to use should be dictated by the type of flow anticipated.

4.1.2. The rectangular throated standing-wave flume is constructed at zero slope and features the measurement of head upstream of the throat in subcritical flow. These factors demand that free flow exists at all times and that special consideration be given to maintaining adequate velocity in the approach channel if sediment and debris exist in the flow to be measured.

4.1.3. R L Parshall (32) proposed changes in the design of the standing-wave flume, the most essential of which was a drop in the floor. This drop stabilized the hydraulic jump that exists beyond the supercritical flow zone. The throat width of the earlier Parshall flumes ranged in size from 3 inches (76 mm) to 8 feet (2.44 m). Flumes with throat widths of 10 feet (3.05 m) to 50 feet (15.24 m) were later constructed and field calibrated. (33) More recently Parshall flumes of 1-inch and 2-inch (25 mm and 50 mm) sizes were calibrated (34). Head-discharge ratings are thus available for a large range in throat width. While supercritical flow exists in the throat of the Parshall Flume, head is measured upstream in subcritical flow and may be affected by deposition of sediment and debris. Parshall flume will operate submerged. These first two flumes are recommended for measuring flows in irrigation systems having relatively sediment free water and where the range of flow is limited.

4.1.4. Both the Parshall and rectangular throated, standing-wave flumes have narrow ranges of discharge because of their vertical walls and are unsuited to the accurate measurement of low discharges. The obvious solution to this problem was the sloping of the flume walls to produce a flume of trapezoidal shape. The following notes cover these three types of flume only, namely the rectangular throated standing-wave flume, the Parshall flume, and the trapezoidal flume.

4.2. RECTANGULAR THROATED STANDING WAVE FLUMES

4.2.1. The rectangular throated flume consists of a constriction of rectangular cross-section symmetrically disposed with respect to the approach channel.

This is the most common type of flume and the easiest to construct. It can be adapted to suit most installations except when loss of head is important in non-rectangular channels.

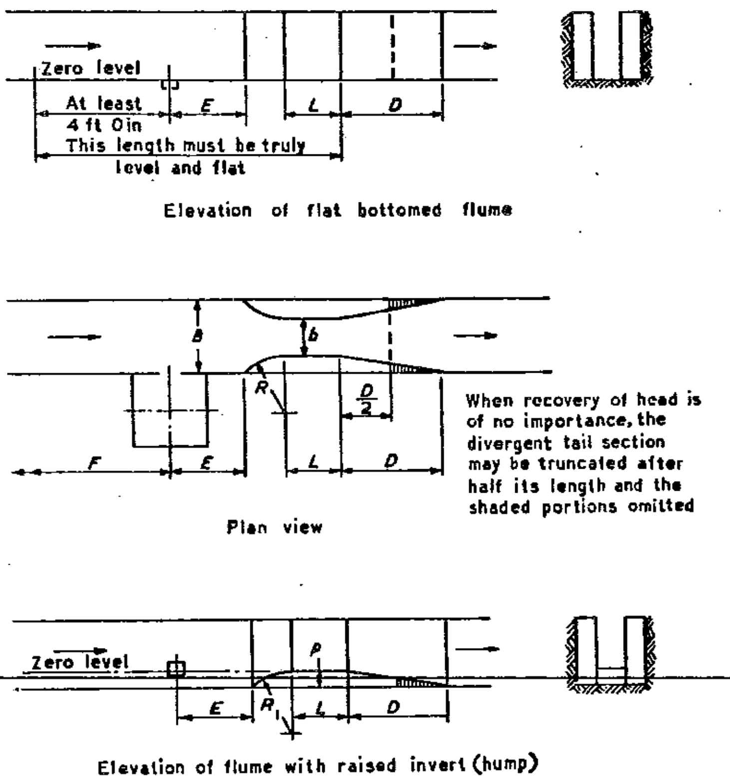
There are three types of rectangular flume:

a. with side contractions only,
b. with bottom contraction or hump only,
c. with both side and bottom contractions.

The type to be used depends on downstream conditions at various rates of flow, the maximum rate of flow, the permissible head loss and the limitations of the h/b ratio, and whether or not the stream carries sediment.

4.2.2. The flume will have the geometry and dimensions indicated on Figure 4.1. The invert of the throat shall be level throughout its width and length. The sides of the flume throat shall be vertical and parallel and square with the invert, so that the width of the throat is correct from top to bottom and end to end. The surfaces of the throat and curved approach shall be smooth; they may be constructed in concrete with a smooth finish, or lined with a smooth non-corrodible material. The centre line of the throat shall be in line with the centre line of the approach channel.

4.2.3. The velocity in the throat must pass through the critical velocity. To ensure this, the invert level shall at all times be at such elevation as to produce free flow for all design discharges. Normally the dimensions of the venturi flume should be such that the depth of water upstream of the contraction is at least 1.25 times that downstream at all rates of flow. Nevertheless, it may be possible to reduce this difference provided that the occurrence of free discharge is confirmed.



b = Width of flume throat
B = Width of approach channel
h_m = Max. head on flume
E = $3h_m$ to $4h_m$
F = At least $10B$
P = Height of hump
L = Length of throat = $1{\cdot}5h_m$

D = $3(B-b)$
R = $2(B-b)$
R_1 = $4P$ (for bottom contraction)
On flumes with both side and bottom contractions, the contractions must commence at the same point and R_1 should be adjusted to suit.

FIGURE 4.1. RECTANGULAR THROATED FLUME IN RECTANGULAR CHANNEL

The maximum design stage upstream of the flume shall be determined from the discharge - such that no flooding of the upstream surroundings shall be caused by the installation.

4.2.4. The following installation conditions shall be observed:

a. The invert level of the throat shall not be lower than the dead water level in the channel, ie the water level downstream at zero flow.
b. The length of the flume throat shall be not less than 1½ times the maximum total head to be measured.
c. The surfaces of the throat and curved approach shall be smooth; they may be constructed in concrete with a smooth cement finish, or lined with a smooth non-corrodible material.
d. The invert of the approach channel shall be level from the flume to a point at least 4 feet (1.22 m) upstream of the point of measurement, and at no point in this length shall rise higher than the invert level of the flume throat.
e. At the downstream end of the flume throat each side and/or the invert of the tail section should have a divergence of not more than 1 in 6.
f. The flow condition in the approach channel for at least 20 times the channel width upstream of the flume shall be subcritical, ie,

$$\bar{V} < \sqrt{\frac{gA}{B_s}} \qquad (4.1)$$

where A = the cross-sectional area
B_s = the surface width.

4.2.5. The head on the flume should be measured at a distance between 3 and 4 times the maximum head upstream of the flume, preferably in a separate gauge chamber connected to the approach channel by a pipe whose entry is normal to the direction of flow, and flush with the wall.

4.2.6. The discharge equation for rectangular throated flumes is:

$$Q = \frac{2}{3}\sqrt{\left(\frac{2}{3}g\right)}\;C_v\;C_D\;b\;h^{3/2} \tag{4.2}$$

in which C_D is coefficient of discharge depending on the development of a boundary layer in the throat
C_v is coefficient of velocity depending on velocity in the approach channel.

4.2.7. Coefficient of discharge C_D. The coefficient of discharge for a flume with rectangular throat shall be determined from the following formula (see refs 3, 35)

$$C_D = \left(\frac{b}{b + 0.004L}\right)^{3/2}\left(\frac{h - 0.003L}{h}\right)^{3/2} \tag{4.3}$$

where L = length of throat.

Values of C_D for various heads in relation to the width and length of the throat are shown in Table 4.1.

4.2.8. Coefficient of velocity C_v. The velocity coefficient depends on the cross-sectional area of the approach channel in relation to the throat of the flume. On the assumption that the cross-sectional area of the approach channel is rectangular, the velocity coefficient shall be determined from the following formulae:

Flumes with side contractions only,

$$\left(\frac{2b}{3\sqrt{3}B}\right)^2 .\;C_v^2 - C_v^{2/3} + 1 = 0 \tag{4.4}$$

Values of C_v for various ratios $\frac{b}{B}$ are shown in Table 4.2.

Flumes with both side and bottom contractions, and flumes with bottom contractions (hump) only,

$$\left(\frac{2b}{3\sqrt{3}B}\right)^2 .\left(\frac{h}{h + P}\right)^2 .\;C_v^2 - C_v^{2/3} + 1 = 0 \tag{4.5}$$

where P = height of invert level of throat above invert level of approach channel, ie height of hump.

(The mean value of P may be taken if the bed of the approach channel is uneven).

In cases where the approach channel is not truly rectangular in section where h is measured, B should be determined from the equation:

$$B = \frac{\text{cross-sectional area at } h}{h + P}$$

Values of C_v for various heads in relation to P and to suit various width ratios are shown in Table 4.3.

The error involved in the implicit assumption that C_v and C_D are mutually independent factors is negligible.

4.2.9. The general installation conditions given earlier shall apply. The following limitations shall be observed:

a. b must not be less than 0.33 ft (0.1 m).
b. $\frac{b}{B} \cdot \frac{h}{h + P}$ must not be more than 0.7.
c. $\frac{h}{b}$ must not be more than 3.
d. h must not be less than 0.16 ft (49 mm) or more than 6.0 ft (1.8 m).
e. When $\frac{h_{max}}{P}$ is less than 0.3 and P greater than 3 ft (0.9 m) in a flume formed by a hump only, the structure should be treated as a broad crested weir.

TABLE 4.1. DISCHARGE COEFFICIENTS FOR RECTANGULAR THROATED FLUMES

$$C_e = \left(\frac{b}{b + 0.004L}\right)^{3/2} \left(\frac{h - 0.003L}{h}\right)^{3/2}$$

$\frac{L}{b}$	$\frac{h}{L}$: 0·70	0·65	0·60	0·55	0·50	0·45	0·40	0·35	0·30	0·25	0·20	0·15	0·10	0·05
0·2	0·9924	0·9919	0·9913	0·9906	0·9898	0·9888	0·9876	0·9860	0·9839	0·9809	0·9764	0·9690	0·9542	0·9103
0·4	0·9912	0·9907	0·9901	0·9894	0·9886	0·9876	0·9864	0·9848	0·9827	0·9797	0·9752	0·9678	0·9530	0·9092
0·6	0·9900	0·9895	0·9889	0·9883	0·9875	0·9865	0·9852	0·9836	0·9815	0·9785	0·9741	0·9667	0·9519	0·9081
0·8	0·9888	0·9883	0·9878	0·9871	0·9863	0·9853	0·9840	0·9825	0·9803	0·9774	0·9729	0·9655	0·9502	0·9070
1·0	0·9876	0·9872	0·9866	0·9859	0·9851	0·9841	0·9829	0·9813	0·9792	0·9762	0·9717	0·9644	0·9496	0·9059
1·2	0·9865	0·9860	0·9854	0·9847	0·9839	0·9829	0·9817	0·9801	0·9780	0·9750	0·9706	0·9632	0·9485	0·9048
1·4	0·9853	0·9848	0·9842	0·9835	0·9827	0·9818	0·9805	0·9789	0·9768	0·9739	0·9694	0·9620	0·9474	0·9038
1·6	0·9841	0·9836	0·9831	0·9824	0·9816	0·9806	0·9793	0·9778	0·9757	0·9727	0·9683	0·9609	0·9462	0·9027
1·8	0·9829	0·9824	0·9819	0·9812	0·9804	0·9794	0·9782	0·9766	0·9745	0·9715	0·9671	0·9598	0·9451	0·9016
2·0	0·9818	0·9813	0·9807	0·9800	0·9792	0·9782	0·9770	0·9754	0·9733	0·9704	0·9660	0·9586	0·9440	0·9005
2·2	0·9806	0·9801	0·9795	0·9789	0·9781	0·9771	0·9758	0·9743	0·9722	0·9692	0·9648	0·9575	0·9429	0·8995
2·4	0·9794	0·9787	0·9784	0·9777	0·9769	0·9759	0·9747	0·9731	0·9710	0·9681	0·9637	0·9563	0·9417	0·8984
2·6	0·9783	0·9778	0·9772	0·9765	0·9757	0·9748	0·9735	0·9720	0·9699	0·9669	0·9625	0·9552	0·9406	0·8973
2·8	0·9771	0·9766	0·9761	0·9754	0·9746	0·9736	0·9724	0·9708	0·9687	0·9658	0·9614	0·9541	0·9395	0·8963
3·0	0·9759	0·9755	0·9749	0·9742	0·9734	0·9724	0·9712	0·9696	0·9676	0·9646	0·9602	0·9529	0·9384	0·8952
3·2	0·9748	0·9743	0·9733	0·9731	0·9723	0·9713	0·9701	0·9685	0·9664	0·9635	0·9591	0·9518	0·9373	0·8941
3·4	0·9736	0·9731	0·9726	0·9719	0·9711	0·9701	0·9689	0·9673	0·9653	0·9623	0·9580	0·9507	0·9362	0·8931
3·6	0·9725	0·9720	0·9714	0·9708	0·9700	0·9690	0·9678	0·9662	0·9641	0·9612	0·9568	0·9495	0·9350	0·8920
3·8	0·9713	0·9708	0·9703	0·9696	0·9688	0·9678	0·9666	0·9651	0·9630	0·9601	0·9557	0·9484	0·9339	0·8909
4·0	0·9702	0·9697	0·9691	0·9685	0·9677	0·9667	0·9655	0·9639	0·9618	0·9589	0·9546	0·9473	0·9328	0·8899
4·2	0·9690	0·9685	0·9680	0·9673	0·9665	0·9656	0·9643	0·9628	0·9607	0·9578	0·9534	0·9462	0·9317	0·8888
4·4	0·9679	0·9674	0·9668	0·9662	0·9654	0·9644	0·9632	0·9616	0·9596	0·9566	0·9523	0·9451	0·9306	0·8878
4·6	0·9667	0·9663	0·9657	0·9650	0·9642	0·9633	0·9521	0·9605	0·9584	0·9555	0·9512	0·9439	0·9295	0·8867
4·8	0·9656	0·9651	0·9646	0·9639	0·9631	0·9621	0·9609	0·9594	0·9573	0·9544	0·9500	0·9428	0·9284	0·8857
5·0	0·9645	0·9640	0·9634	0·9628	0·9620	0·9610	0·9593	0·9583	0·9562	0·9533	0·9490	0·9418	0·9274	0·8847

NOTE. The number of significant figures given in the columns for coefficient of discharge should not be taken to imply a corresponding accuracy but only to assist in interpolation and analysis.

TABLE 4.2. VELOCITY COEFFICIENTS FOR RECTANGULAR THROATED FLUMES WITH LEVEL INVERTS

$$\left(\frac{2b}{3\sqrt{3}B}\right)^2 C_v^2 - C_v^{2/3} + 1 = 0$$

$\frac{b}{B}$	C_v	$\frac{b}{B}$	C_v
0·10	1·0022	0·44	1·0476
0·15	1·0051	0·46	1·0526
0·20	1·0091	0·48	1·0579
0·22	1·0110	0·50	1·0635
0·24	1·0132	0·52	1·0695
0·26	1·0155	0·54	1·0760
0·28	1·0181	0·56	1·0829
0·30	1·0209	0·58	1·0901
0·32	1·0240	0·60	1·0980
0·34	1·0272	0·62	1·1064
0·36	1·0308	0·64	1·1153
0·38	1·0346	0·66	1·1250
0·40	1·0386	0·68	1·1353
0·42	1·0430	0·70	1·1465

NOTE. The number of significant figures given in the columns for coefficient of discharge should not be taken to imply a corresponding accuracy but only to assist in interpolation and analysis.

TABLE 4.3. VELOCITY COEFFICIENTS FOR RECTANGULAR THROATED FLUMES WITH BOTH SIDE AND BOTTOM CONTRACTIONS

$$\left(\frac{2b}{3\sqrt{3}B}\right)^2\left(\frac{h}{h+P}\right)^2 C_v^2 - C_v^{2/3} + 1 = 0$$

$\frac{b}{B}$	$\frac{h}{h+P}$								
	1·0	0·9	0·8	0·7	0·6	0·5	0·4	0·3	0·2
0·10	1·0018	1·0018	1·0014	1·0011	1·0008	1·0006	1·0004	1·0002	1·0001
0·15	1·0051	1·0041	1·0032	1·0025	1·0018	1·0013	1·0008	1·0005	1·0002
0·20	1·0091	1·0073	1·0058	1·0044	1·0032	1·0018	1·0014	1·0008	1·0004
0·25	1·0143	1·0115	1·0091	1·0069	1·0051	1·0035	1·0018	1·0013	1·0006
0·30	1·0209	1·0168	1·0132	1·0100	1·0073	1·0051	1·0032	1·0018	1·0008
0·35	1·0290	1·0232	1·0181	1·0137	1·0100	1·0069	1·0044	1·0025	1·0011
0·40	1·0386	1·0308	1·0240	1·0181	1·0132	1·0091	1·0058	1·0032	1·0014
0·45	1·0500	1·0397	1·0308	1·0232	1·0168	1·0115	1·0073	1·0041	1·0018
0·50	1·0635	1·0500	1·0388	1·0290	1·0209	1·0143	1·0091	1·0051	1·0022
0·55	1·0793	1·0620	1·0476	1·0357	1·0255	1·0175	1·0110	1·0061	1·0027
0·60	1·0980	1·0760	1·0579	1·0429	1·0308	1·0209	1·0132	1·0073	1·0032
0·65	1·1203	1·0921	1·0695	1·0503	1·0367	1·0248	1·0156	1·0086	1·0038
0·70	1·1469	1·1108	1·0829	1·0606	1·0429	1·0290	1·0181	1·0100	1·0044
0·75	—	1·1330	1·0980	1·0711	1·0500	1·0336	1·0209	1·0115	1·0051
0·80	—	—	1·1155	1·0829	1·0579	1·0386	1·0240	1·0132	1·0058
0·85	—	—	1·1358	1·0960	1·0664	1·0441	1·0272	1·0149	1·0065
0·90	—	—	—	1·1108	1·0760	1·0500	1·0308	1·0168	1·0073
0·95	—	—	—	1·1279	1·0864	1·0564	1·0346	1·0188	1·0082
1·00	—	—	—	1·1469	1·0980	1·0635	1·0386	1·0209	1·0091

NOTE. The number of significant figures given in the columns for coefficient of velocity should not be taken to imply a corresponding accuracy but only to assist in interpolation and analysis.

4.3. PARSHALL FLUME

4.3.1. The flumes will have the geometry and dimensions indicated in Figure 4.2 and Table 4.4 respectively. The invert of the coverging approach section shall be level throughout its width and length and it shall be the zero datum for both upstream and downstream head measurements. All wall sections shall be perpendicular and plane. The accuracy of construction of the very small flumes is extremely important if the tabular values are to be used. The surfaces of the flume shall be of smooth concrete, galvanized steel or other smooth non-corrodible material.

4.3.2. The flumes will be installed with the throat centreline in line with the centre of the approach channel. The flow should enter the converging section reasonably well distributed across the entrance width. The entering flow should also be free of surges and waves and visible surface boils. Subcritical flow shall exist in the flume approach. For best results, flumes should be installed at such elevations as to operate without submergences which require corrections to free-flow (to be described later). Under no circumstances should submergences be greater than 95 per cent.

4.3.3. Discharge Ratings:

Figure 4.3 contains the discharge rating curves for the 2-inch (50 mm) through 9-inch (0.23 m) size Parshall flumes both for free flow and for different degrees of submergence.

Tables 4.5 and 4.6 give the free-flow discharge ratings for flumes 1 to 8 feet (0.3 to 2.4 m) and 10 to 50 feet (3.04 to 15.24 m) in size respectively. The general discharge equations which apply to the 1 to 8 (0.3 to 2.4 m) and 10 to 50 (3.04 to 15.24 m) foot sizes respectively for free-flows are:

$$Q = 4bh_a^{1.522b^{0.026}} \qquad (4.6)$$

and

$$Q = (3.6875b + 2.5)\, h_a^{1.6} \qquad (4.7)$$

where b is the throat width and h is the piezometric head measured 2/3 A distance along the approach wall upstream.

4.3.4. The larger the flume the greater the submergence that can be tolerated before the free-flow discharge is reduced. For the 2 inch (50 mm) to 9 inch (0.23 m) flumes, the discharge for different submergences may be determined from Figure 4.3. Correction factors for computing the effect of submergences for the 1 to 50 foot (0.3 to 15.2 m) sizes are given in Figure 4.4. The discharge under submerged conditions is equal to the free-fall discharge minus the product of two correction factors, Q_c and k_s thus,

$$Q_s = Q_f - k_s \ Q_c$$

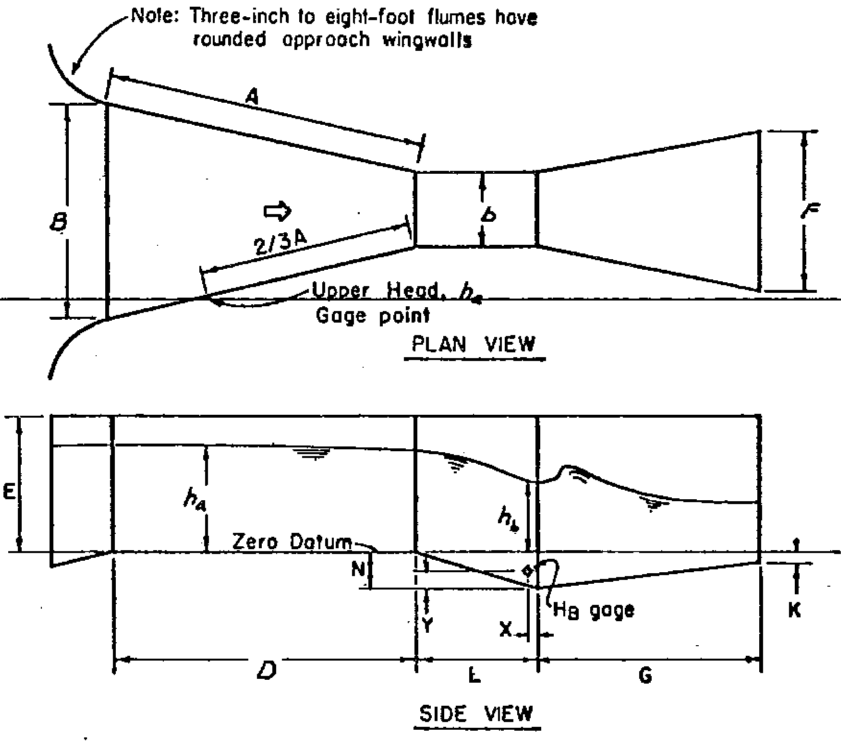


FIGURE 4.2. CONFIGURATION AND DESCRIPTIVE NOMENCLATURE FOR PARSHALL FLUMES

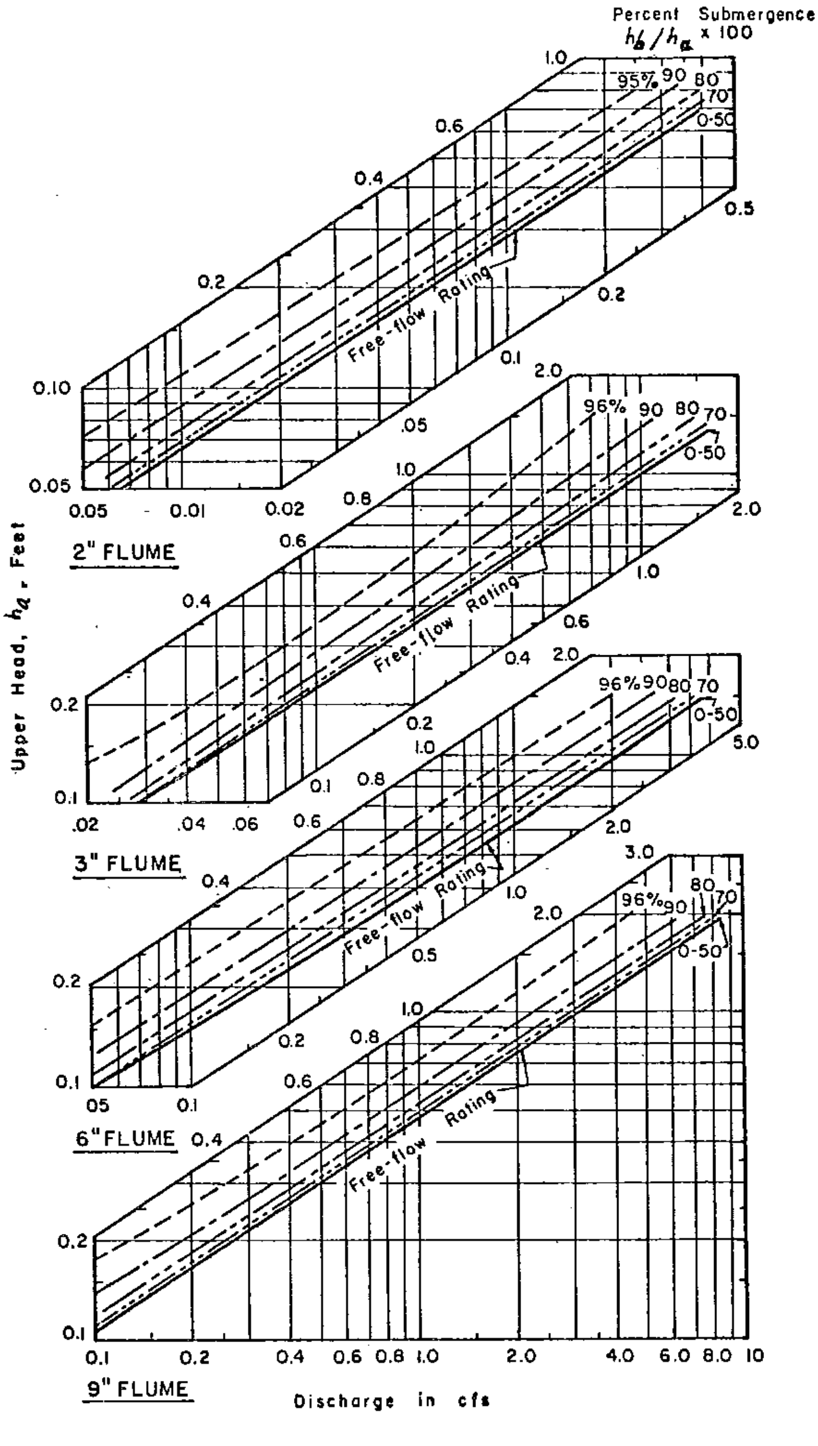
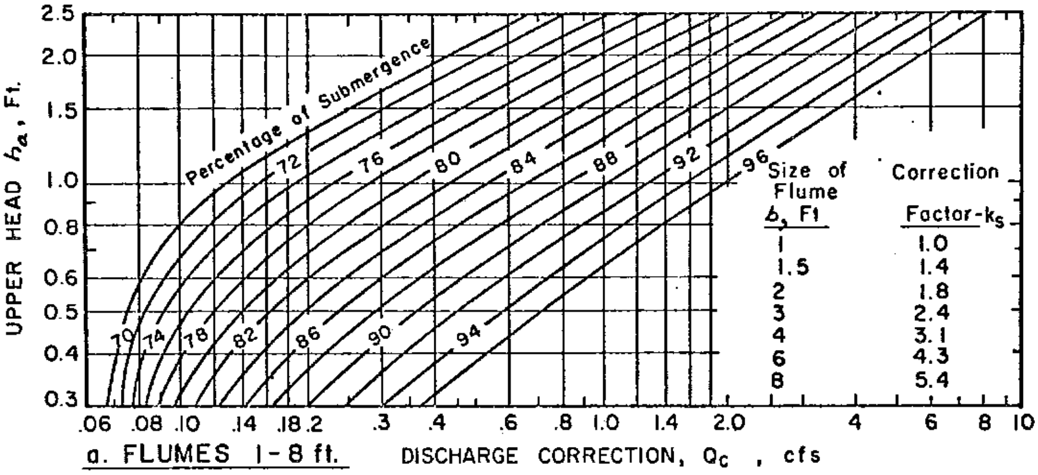


FIGURE 4.3. DISCHARGE RATING FOR "INCH" PARSHALL FLUMES FOR BOTH FREE-FLOW AND SUBMERGENCE CONDITIONS



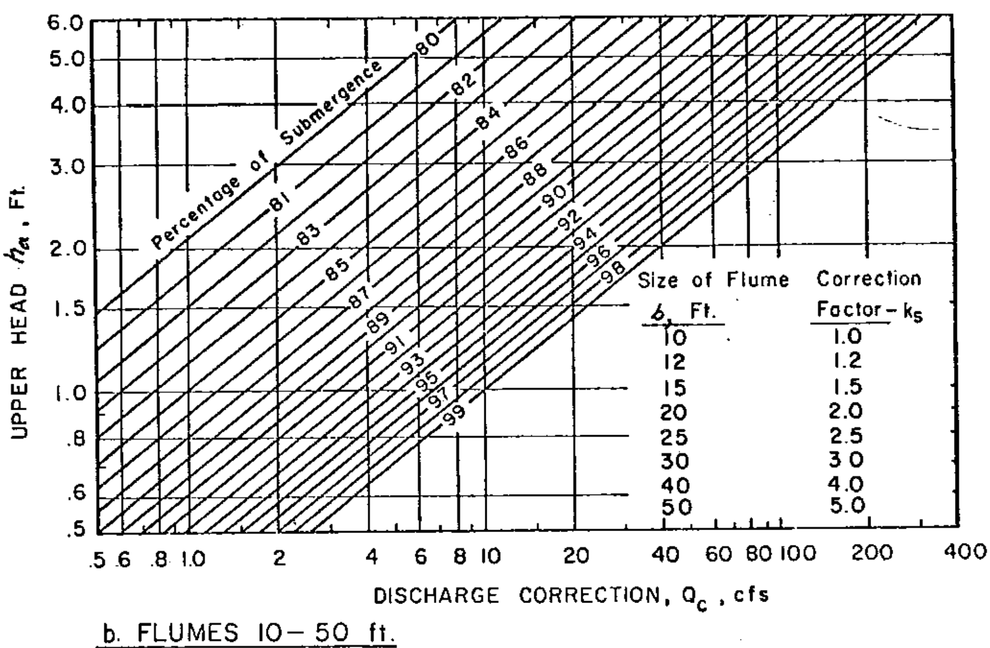


FIGURE 4.4. CORRECTION FACTORS FOR SUBMERGED FLOW THROUGH 1 - TO 50 - FT (0.30 TO 15.24 m) PARSHALL FLUMES

TABLE 4.4. DIMENSIONS AND CAPACITIES OF ALL SIZES OF STANDARD PARSHALL MEASURING FLUMES

	Widths		Axial Lengths			Wall	Vertical distance below crest		Con-	Gage Points			Free Flow Capacities	
Size; Throat width b	Upstream end B	Down-stream end F	Con-verging Section D	Throat Section L	Diverging Section G	Depth in Con-verging Section E	Dip at Throat N	Lower end of flume K	verging wall length A*	h_a, dist. upstream of crest**	h_b x	h_b y	Min.	Max.
inches	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	feet	cfs	cfs
1	0.549	0.305	1.17	0.250	0.67	0.5-0.75	0.094	0.062	1.19	0.79	0.026	0.042	0.005	0.15
2	.700	.443	1.33	.375	.83	0.50-0.83	.141	.073	1.36	.91	.052	.083	.01	.30
3	.849	.583	1.50	.500	1.00	1.00-2.00	.188	.083	1.53	1.02	.083	.125	.03	1.90
6	1.30	1.29	2.00	1.00	2.00	2.0	.375	.25	2.36	1.36	.167	.25	.05	3.90
9	1.88	1.25	2.83	1.00	1.50	2.5	.375	.25	2.88	1.93	.167	.25	.09	8.90
Feet														
1.0	2.77	2.00	4.41	2.0	3.0	3.0	.75	.25	4.50	3.00	.167	.25	.11	16.1
1.5	3.36	2.50	4.66	2.0	3.0	3.0	.75	.25	4.75	3.17	.167	.25	.15	24.6
2.0	3.96	3.00	4.91	2.0	3.0	3.0	.75	.25	5.00	3.33	.167	.25	.42	33.1
3.0	5.16	4.00	5.40	2.0	3.0	3.0	.75	.25	5.50	3.67	.167	.25	.61	50.4
4.0	6.35	5.00	5.88	2.0	3.0	3.0	.75	.25	6.00	4.00	.167	.25	1.30	67.9
5.0	7.55	6.00	6.38	2.0	3.0	3.0	.75	.25	6.50	4.33	.167	.25	1.60	85.6
6.0	8.75	7.00	6.86	2.0	3.0	3.0	.75	.25	7.0	4.67	.167	.25	2.60	103.5
7.0	9.95	8.00	7.35	2.0	3.0	3.0	.75	.25	7.5	5.0	.167	.25	3.00	121.4
8.0	11.15	9.00	7.84	2.0	3.0	3.0	.75	.25	8.0	5.33	.167	.25	3.50	139.5
10	15.60	12.00	14.0	3.0	6.0	4.0	1.12	.50	9.0	6.00	.75	1.00	6	300
12	18.40	14.67	16.0	3.0	8.0	5.0	1.12	.50	10.0	6.67	.75	1.00	8	520
15	25.0	18.33	25.0	4.0	10.0	6.0	1.50	.75	11.5	7.67	.75	1.00	8	900
20	30.0	24.00	25.0	6.0	12.0	7.0	2.25	1.00	14.0	9.33	.75	1.00	10	1340
25	35.0	29.33	25.0	6.0	13.0	7.0	2.25	1.00	16.5	11.00	.75	1.00	15	1660
30	40.4	34.67	26.0.	6.0	14.0	7.0	2.25	1.00	19.0	12.67	.75	1.00	15	1990
40	50.8	45.33	27.0	6.0	16.0	7.0	2.25	1.00	24.0	16.00	.75	1.00	20	2640
50	60.8	56.67	27.0	6.0	20.0	7.0	2.25	1.00	29.0	19.33	.75	1.00	25	3280

* For sizes 1' to 8', $A = b/2 + 4$

** H_A located 2/3 A distance from crest for all sizes; distance is wall length, not axial

Note: Flume sizes 3 inches through 8 feet have approach aprons rising at a 1:4 slope and the following entrance roundings: 3 through 9 inches, radius = 1.33 feet; 1 through 3 feet, radius = 1.67 feet; 4 through 8 feet, radius = 2.00 feet.

TABLE 4.5. DISCHARGE TABLE FOR PARSHALL MEASURING FLUMES, SIZES 1 FOOT TO 8 FEET FOR FREE-FLOW CONDITIONS

h_a	1 foot	1.5 feet	2 feet	3 feet	4 feet	5 feet	6 feet	7 feet	8 feet
feet	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs
0.10	0.11	0.15							
.15	.20	.30	0.42	0.61					
.20	.35	.51	.66	.97	1.26	1.55			
.25	.49	.71	.93	1.37	1.80	2.22	2.63	3.02	3.46
.30	.64	.94	1.24	1.82	2.39	2.96	3.52	4.08	4.62
.4	.99	1.47	1.93	2.86	3.77	4.68	5.57	6.46	7.34
.5	1.39	2.06	2.73	4.05	5.36	6.66	7.94	9.23	10.5
.6	1.84	2.73	3.62	5.39	7.15	8.89	10.6	12.4	14.1
.7	2.33	3.46	4.60	6.86	9.11	11.4	13.6	15.8	18.0
.8	2.85	4.26	5.66	8.46	11.3	14.0	16.8	19.6	22.4
.9	3.41	5.10	6.80	10.2	13.6	16.9	20.3	23.7	27.0
1.0	4.00	6.00	8.00	12.0	16.0	20.0	24.0	28.0	32.0
1.2	5.28	7.94	10.6	16.0	21.3	26.7	32.1	37.5	42.9
1.4	6.68	10.1	13.5	20.3	27.2	34.1	41.1	48.0	55.0
1.6	8.18	12.4	16.6	25.1	33.6	42.2	50.8	59.4	68.1
1.8	9.79	14.8	19.9	30.1	40.5	50.8	61.3	71.8	82.3
2.0	11.5	17.4	23.4	35.5	47.8	60.1	72.5	84.9	97.5
2.2	13.3	20.2	27.2	41.3	55.5	69.9	84.4	98.9	113.6
2.4	15.2	23.0	31.1	47.3	63.7	80.3	97.0	113.7	130.7

TABLE 4.6. DISCHARGE TABLE FOR PARSHALL MEASURING FLUMES, SIZES 10 FEET TO 50 FEET FOR FREE-FLOW CONDITIONS

h_a	10 feet	12 feet	15 feet	20 feet	25 feet	30 feet	40 feet	50 feet
feet	cfs	cfs	cfs	cfs	cfs	cfs	cfs	cfs
0.30	5.75	6.75	8.4	11.1	13.8	16.5	21.8	27.3
0.4	9.05	10.85	13.3	17.7	21.8	26.1	34.6	43.2
0.5	13.0	15.4	19.1	25.1	31.2	37.2	49.5	61.8
0.6	17.4	20.6	25.5	33.7	41.8	50.0	66.2	82.6
0.7	22.2	26.2	32.7	43.1	53.4	64.0	84.8	105.5
0.8	27.5	32.7	40.4	53.4	66.3	79.2	105	131
0.9	33.3	39.4	48.9	64.3	80.1	95.5	127	158
1.0	39.4	46.8	57.9	76.3	94.8	113.2	150	187
1.2	52.7	62.6	77.3	102.0	127.0	152	201	250
1.4	67.4	80.1	99.0	130.5	162	194	257	320
1.6	83.5	99.1	122.8	162	201	240	318	396
1.8	100.9	119.8	148.0	195	243	290	384	479
2.0	119.4	141.8	175.3	232	287	343	454	567
2.2	139.0	165.0	204	269	334	400	530	660
2.4	159.9	189.8	235	310	384	459	609	758
2.6	181.7	215.7	267	352	437	522	692	864
3.0	228.4	271.2	335	442	549	656	870	1084
3.5	294	347	429	566	703	840	1113	1387
4.0	363	430	531	700	870	1040	1379	1717
4.5	437	518	641	846	1051	1255	1664	2073
5.0	517	614	759	1002	1244	1486	1970	2453
5.5			885	1166	1448	1730	2295	2860
6.0			1016	1340	1664	1988	2638	3285

Note: Available data indicates that extension of the above ratings to greater heads is reliable.

4.4. TRAPEZOIDAL THROATED FLUMES

4.4.1. Trapezoidal throated flumes may be designed (refs 35, 36) to cope with many different flow conditions, and the optimum throat geometry (ie bed width and side slopes) will depend on the range of flow to be measured and on the characteristics of the stream in which it is to be installed. Design methods by which the geometry might be selected to approximate to a predetermined stage discharge relation are comprehensively described in ref 36 and outlined later.

4.4.2. Trapezoidal flumes should have a geometry generally as indicated in Figure 4.5. In some circumstances however it will be appropriate to make the invert of the throat level with the invert of the approach channel, ie p = o: this will be the case if sediment has to be conveyed through the flume. The entrance and exit transitions may be plane or curved surfaces if so desired.

4.4.3. The flume should be installed with the throat centre-line in line with the centre of the approach channel. Sub-critical flow must exist in the flume approach, and the flume should be installed at such an elevation as to operate with free discharge throughout the range. The surfaces of the flume shall be of smooth concrete, galvanised steel or other smooth non-corrodible material. The throat section is of particular importance and must have a level invert and be truly prismatic, the sloping walls being plane surfaces, symmetrically disposed and making a sharp inter-section with the invert of the throat.

4.4.4. Several methods of evaluating the discharge through trapezoidal flumes are available, ref 36 provides methods of preparing detailed rating curves for any geometry by computation or semi-graphical techniques, making full allowance for the influence of the boundary layer in the throat, and taking into account the surface texture of the throat. The starting point for this method is the flow condition in the throat ie it is based on the occurrence of critical flow therein. From this the relationship of total head to discharge is determined, and a rating curve in terms of an upstream gauging of water level is developed.

4.4.5. It is also possible to derive a general discharge equation for trapezoidal flumes in the form of a direct algebraic relationship such as has been given for most of the structures previously described.

$$Q = C_d \; \frac{2}{3} \sqrt{\left(\frac{2}{3} g\right)} \; C_s \; b \; H^{3/2} \tag{4.8}$$

in which

Q = discharge

C_d = coefficient allowing for fluid property effects, ie head loss between the point at which levels are measured and the control section within the throat.

C_s = shape factor, taking account of the non-rectangular throat geometry.

b = bed width of throat.

H = total head upstream of the flume.

In the above

$$C_s = f\left(m \; H_c/b\right) \tag{4.9}$$

where

m = slope of throat walls (m horizontal in 1 vertical)

and

H_c = total head at the critical section.

C_d in equation 4.8 will vary with the size of flume and the head, as well as with the roughness of the throat, lying approximately in the range 0.95 in small installations (of order of 0.5 cusec) to 0.98 in large installations (of order of 200 cusecs). These values of C_d are suitable for design purposes although not considered accurate enough for computing the flume calibration.

4.4.6. An approximate calibration in terms of total head can be derived from equations 4.8 and 4.9, with the aid of Figure 4.5, noting that

$$H_c = H\ C_d^{2/3} \tag{4.10}$$

Because of the influence of head losses on C_d and the need to allow for velocity of approach, direct application of a discharge equation is not very convenient, however.

4.4.7. Usually a theoretical calibration is derived for a gauging structure for the whole range of discharge in one computation, and the most logical method of preparing such a calibration for a critical depth device is to start with flow conditions at the throat. A tabulated calculation by desk computer proceeds as follows.*

a. Select range of values of d_c, the critical depth in the throat (a roughly logarithmic series is more convenient than an arithmetic series)

b. For each depth evaluate the water surface width and cross-sectional area (W_c and A_c respectively).

$$W_c = b + 2md_c \tag{4.11}$$

$$A_c = (b + md_c)\ d_c \tag{4.12}$$

c. The critical velocity is given by

$$V_c = \sqrt{g\ A_c/W_c} \tag{4.13}$$

d. The corresponding discharge is

$$Q = A_c\ V_c \tag{4.14}$$

e. The total head at the critical section is

$$H_c = d_c + A_c/2W_c \tag{4.15}$$

f. In order to evaluate the head loss✓ between the upstream gauging section and the critical section, it is assumed that most occurs within the prismatic throat section, of length L_c. The throat Reynolds number is given by

$$Re = V_c\ L_c/\nu \tag{4.16}$$

where ν for water at 15°C = 1.23 x 10^{-5} ft^2/s (1.14 x 10^{-6} m^2/s)

g. The relative roughness of the throat is

$$L_c/k_s \tag{4.17}$$

where k_s is the equivalent sand roughness (after Nikuradse) of the material used to construct the flume. Suitable values range from 0.0001 ft (0.03 mm) for sheet metal, through 0.001 ft (0.3 mm) for pre-cast or in-situ concrete to 0.002 ft (0.6 mm) for planed wood.

h. The coefficient of total skin drag, C_f, is obtained from Figure 4.7.

i. The head loss is computed from

$$h_f = \frac{C_f\ L_c\ P_c}{2W_c} \tag{4.18}$$

where

$$P_c = b + 2d_c\sqrt{1 + m^2} \tag{4.19}$$

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* A program for a digital computer would follow similar lines, of course

✓ A parallel computation on the basis of the boundary layer displacement in the throat is academically preferable, but insufficient evidence about the growth of a boundary layer in these circumstances is available. It is known however (see ref 36) that the use of the drag coefficient on a flat plate gives overall coefficient values very close to those observed experimentally even though the throat condition is not strictly analogous to a flat plate in an infinite fluid.

j. The total head upstream of the throat is obtained from

$$H = H_c + h_f \tag{4.20}$$

where H is measured relative to the throat invert as datum.

k. To convert from total head H to gauged head h, the expression

$$h \doteq H - \frac{v_a^2}{2g} \tag{4.21}$$

is used, where v_a is the mean velocity in the approach channel at the gauging section.

l. Before applying eqn 4.21, the geometry of the approach channel is used to compute its cross-sectional area, A_a. Assuming the approach channel is of trapezoidal section

$$A_a = (h + P)\,[B + m_a (h + P)] \tag{4.22}$$

In the above, P is the height of the hump at the throat, B is the bed width of the approach channel, and m_a is its side slope (m_a horizontal to 1 vertical)

m. Because h occurs implicitly in the right hand side of eqn (4.21), a method of successive approximation is needed to work out h.

<u>1st approximation:</u>

Set h = H and hence calculate A_{a1} from 4.22. Insert v_{a1} in eqn 4.21, where $v_{a1} = Q/A_{a1}$, and thus obtain h_1.

<u>2nd approximation:</u>

Obtain A_{a2} from 4.22 inserting $h = h_1$ Hence $v_{a2} = Q/A_{a2}$, and h_2 from eqn 4.21.

<u>3rd approximation etc:</u>

Repeat procedure until $h_n - h_{n-1}$ is too small to significantly affect the accuracy. (It is suggested that $h_n - h_{n-1}$ should not exceed 0.1 per cent of h_{n-1}).

n. Having thus worked out pairs of values of Q and h for a series of values of d_c, the calibration curve for the flume may be plotted to large logarithmic scales. Discharge values for intermediate values of h may be read off as necessary. Note that the calibration curve is not a straight line on a log-log plot.

4.4.8. The following observations should be noted:-

a. H/L should not exceed 0.50 if high accuracy is required. If high accuracy is not needed in the upper part of the discharge range, H_{max}/L may be allowed to rise to 0.67.
b. At all elevations the width between the throat walls must be less than the width between the approach channel walls at the same elevation, ie: there must be a contraction wherever the water surface lies.
c. The sloping walls of the throat and the approach section must continue upwards without change of slope far enough to contain the maximum discharge to be measured. (For the design of flumes with trapezoidal throats that are contained within vertical walls, see ref 36).
d. The flume must not be submerged. The limiting value for H_d/H where H_d is the total energy level downstream relative to the invert of the throat is dependent on the gradualness of the exit transition (see ref 37)

1 in 20 expansion, H_d/H	94 per cent
1 in 15 expansion, H_d/H	88 per cent
1 in 10 expansion, H_d/H	85 per cent
1 in 5 expansion, H_d/H	80 per cent
1 in 3 expansion, H_d/H	75 per cent

e. The Froude number in the approach channel normally should not exceed 0.5, ie:

$$Fr_a = \frac{v_a}{\sqrt{(gA_a/W_a)}} \leq 0.5 \tag{4.23}$$

where W_a is the water surface width, b + 2m (h + P) in the approach channel.
Where sediment is being carried, it may be desirable to allow Fr_a to rise to 0.6 to avoid deposition in the approach channel.

f. The exit and entrance transitions may be formed of either plane or cylindrical surfaces as shown in Figure 4.5. The convergence at the entrance and expansion at the exit should not exceed 1 in 3 at the walls and floor.

g. b should be at least 0.1 m (0.3 ft) and h should be at least 0.03 m (0.1 ft).

4.4.9. The head on the flume should be measured at a distance of between 1 and 4 times the maximum head upstream of the entrance transition, preferably in a separate gauge chamber connected to the approach channel by a pipe whose entry is normal to the direction of flow and flush with the wall.

4.4.10. There is considerable flexibility in the design of a trapezoidal throated flume, and this permits the designer to select values of m and b which will provide an exact match to a predetermined relationship of head to discharge at two flows. A convenient graphical method was derived in ref 36, utilising equation 4.8 in the form

$$\frac{Q}{\frac{2}{3}\sqrt{\left(\frac{2}{3}g\right)}\, bH^{3/2}} = f\left(\frac{mH}{b}\right) \tag{4.24}$$

on the assumption that $H \simeq H_c$ for design purposes.

4.4.11. Required values of Q and H are estimated from a knowledge or estimate of the existing stage-discharge relationship at the site, paying due regard to the head loss needed for free flow, practical limitations on the height of the throat above the stream bed (it must not be below "no flow" level), and the limitations given earlier. The two values of

$$\frac{Q}{\frac{2}{3}\sqrt{\left(\frac{2}{3}g\right)}\, bH^{3/2}}$$

corresponding to these boundary conditions are worked out and plotted on transparent material against H as abscissa on the same logarithmic scales as in Figure 4.6. Vertical and horizontal guide-lines are added to the transparent overlay, which is then moved up or down, to the right or to the left without rotation until the two plotted points lie on the curve. The intercept of y = 1 on the overlay with the y axis of Figure 4.6 gives 1/b and the intercept of x = 1 on the overlay with the x axis of Figure 4.6 gives m/b and hence m. An example is shown by dotted lines for which $Q/\frac{2}{3}\sqrt{\left(\frac{2}{3}g\right)}\, H^{3/2}$ and H values are 3.07, 2.82 m 1.34, 0.21 m corresponding to discharges of 25 and .01 m^3/s respectively. These requirements are met by a flume in which b = 1.22 m and m = 0.90.

4.4.12. Example of alternative designs of flumes for the following conditions:-

Range of flow	$0.01\ m^3/s$ - $17\ m^3/s$
Froude number in approach channel at maximum discharge	0.37
Longitudinal slope of channel upstream	1:236
Longitudinal slope of channel downstream	1:50
Kutters 'n' for channel	0.03
Channel side slopes	1:2
Channel width	2.2 m

DESIGN COMPUTATIONS

Design details		Flume 1	Flume 2
Throat width	m	0.61	0.15
Maximum head	m	2.06	2.21
Minimum head	m	0.07	0.07
Maximum discharge	m^3/s	17	17.7
Minimum discharge	m^3/s	0.03	0.01
Throat length	m	3.5	3.7
Entrance transition	m	2.5	3.1
Exit transition	m	2.5	3.1
Throat side slopes		1:2	1:2

4.4.13. The methods employed in assessing the magnitude of the error in discharge as measured by the trapezoidal throated flume, as well as by other structures, may be found in references 3 and 42.

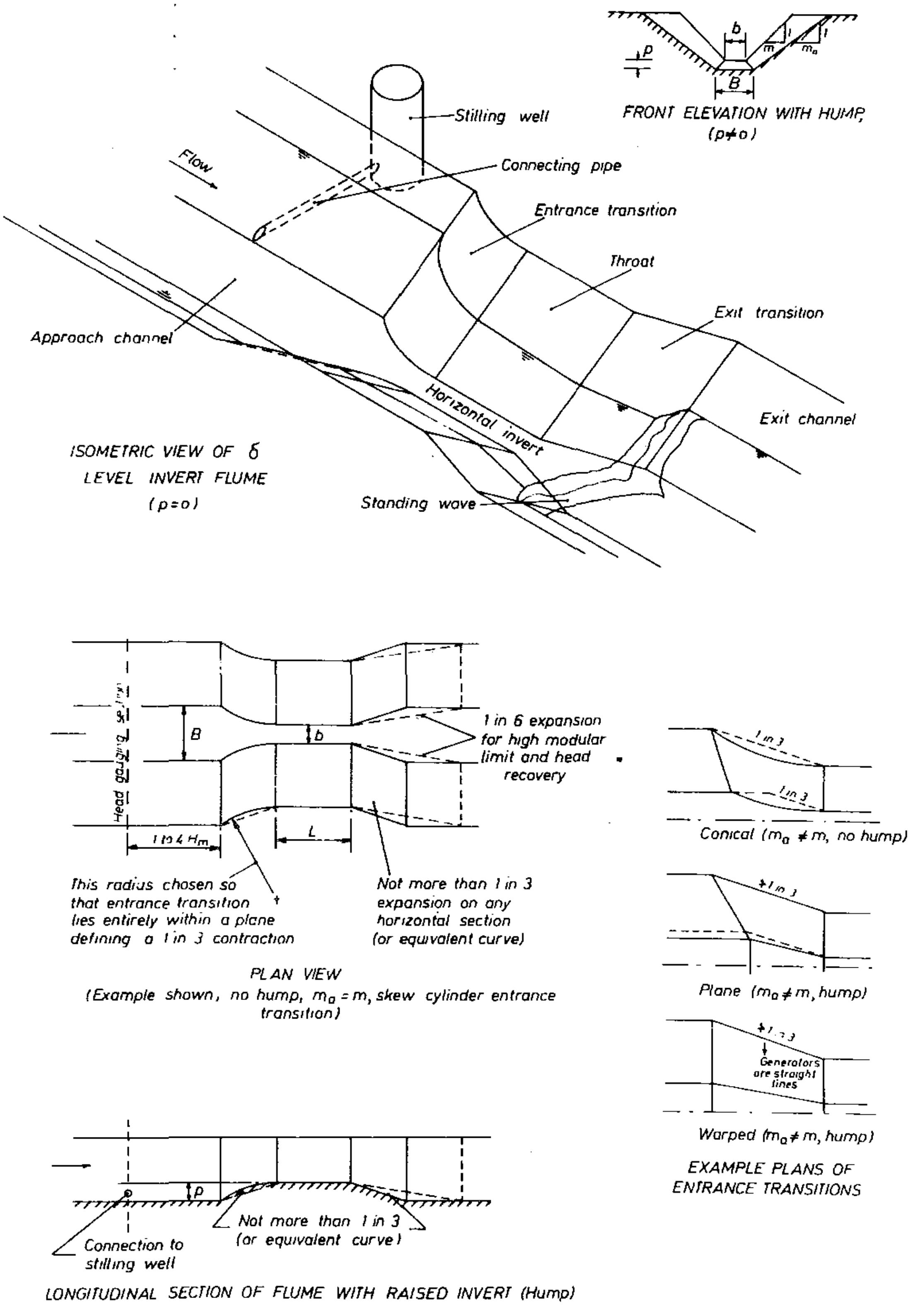

FIGURE 4.5. LAYOUT OF TRAPEZOIDAL THROATED FLUME

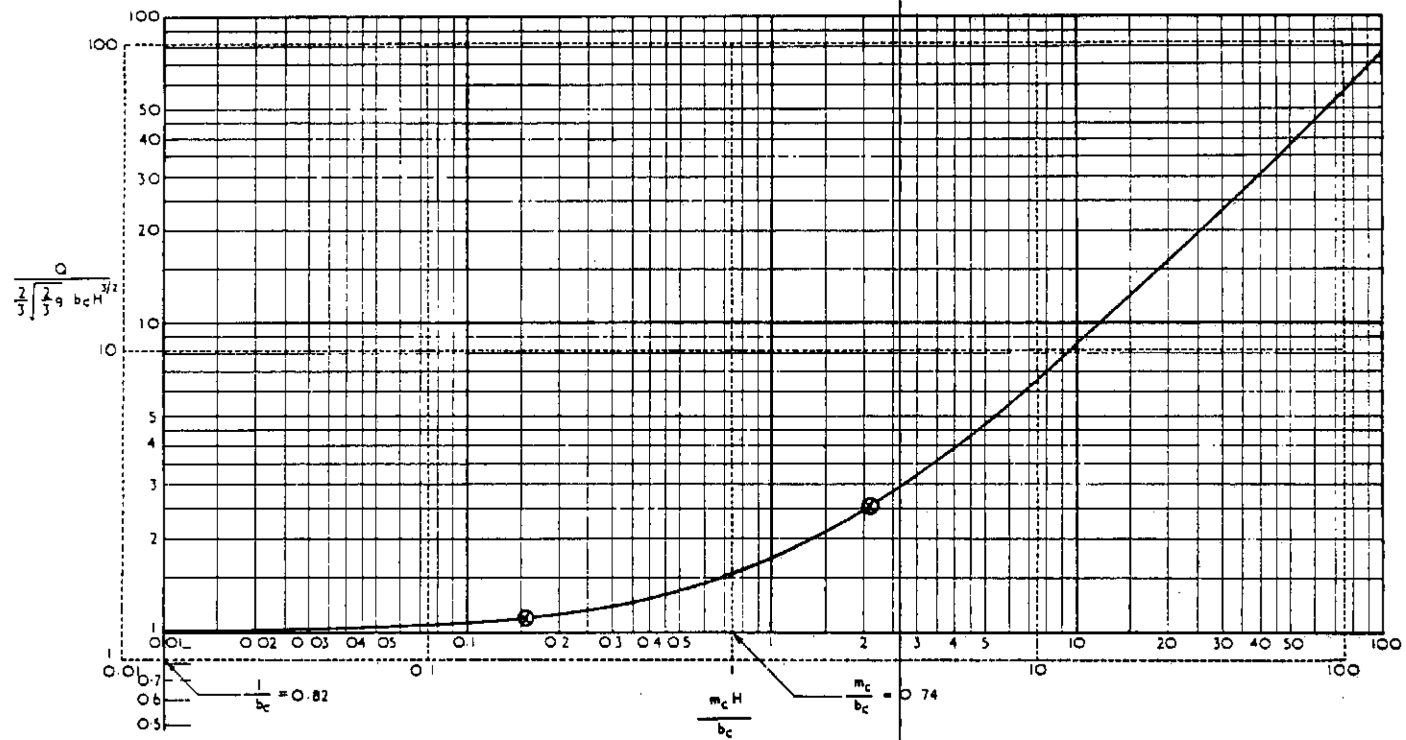

FIGURE 4.6. SHAPE COEFFICIENT FOR TRAPEZOIDAL THROATED FLUME

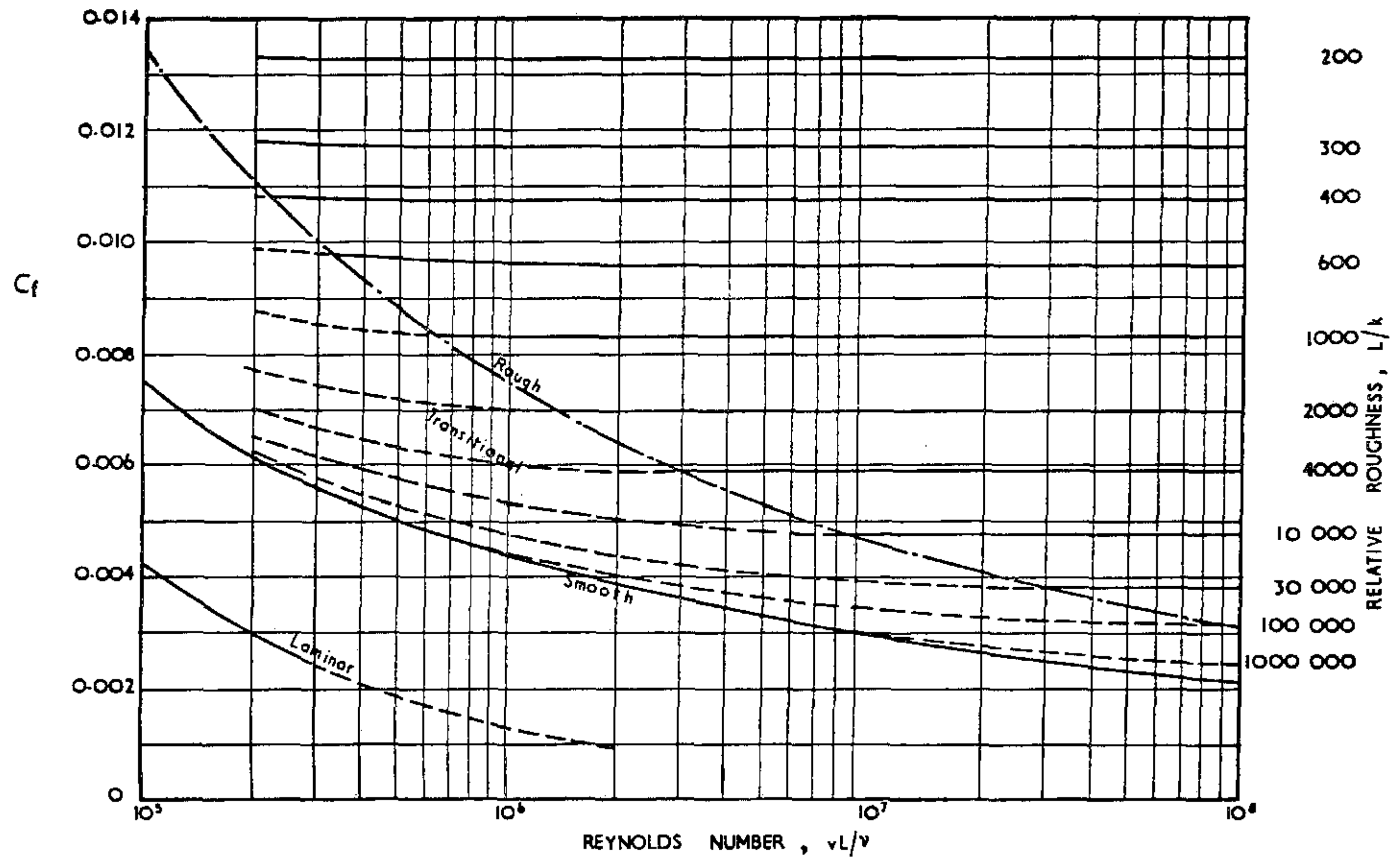

FIGURE 4.7. COEFFICIENT OF TOTAL SKIN DRAG

REFERENCES

1. Schoder, Ernest W and Kenneth B Turner, "Precise Weir Measurements," Transactions of the American Society of Civil Engineers, Vol 93, 1929, pp 999-1110.

2. Discussion by Ralph W Powell of "Discharge Characteristics of Rectangular, Thin-Plate Weirs," by Carl E Kindsvater and Rolland W Carter, Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, June 1958, Paper 1690, pp 23-26.

3. i. British Standard Methods of Measurement of Liquid flow in open channels, BS 3680, Part 4A, Thin-plate weirs and venturi flumes, 1965.

   ii. British Standard Methods of Measurement of Liquid flow in open channels, BS 3680, Part 4B, Long Base Weirs 1969.

4. International Standards Organisation, Draft recommendation No 1438, Liquid flow measurement in open channels using thin-plate weirs and venturi flumes.

5. Shen, John, "A Preliminary Report on the Discharge Characteristics of Triangular Thin-Plate Weirs," unpublished report prepared for the Research Section, Surface Water Branch, Water Resources Division, Geological Survey, US Department of the Interior, Washington, DC 1959.

6. Numachi, F, T Kurokawa, and S Hutizawa, "Uber den Uberfallbeiwert eines rectwinkling-dreieckigen Messwehrs, "Transactions of the Society of Mechanical Engineers (Japan), Vol 6, No 22, February 1940, p 110.

7. Barr, James, "Experiments on the Flow of Water over Triangular Notches," Engineering (London), Vol 89, April 8 and 15, 1910, p 435.

8. Yarnall, D R "Accuracy of the V-Notch Weir Method of Measurement," Transactions of the American Society of Mechanical Engineers, Vol 48, 1926, pp 939-964.

9. Greve, F W, "Flow of Water through Circular, Parabolic, and Triangular Vertical Notch-Weirs," Research Bulletin No 40, Engineering Experiment Station, Purdue University, Lafayette, Indiana, March 1932.

10. Lenz, A T, "Viscosity and Surface Tension Effects on V-Notch Weir Coefficients," Transactions of the American Society of Civil Engineers, Vol 108, 1943, pp 759-782.

11. Kindsvater, Carl E, and Rolland W Carter, "Discharge Characteristics of Rectangular, Thin-Plate Weirs," Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, December 1957, Paper 1453; and March 1959, Paper 1989, p 45.

12. Cone, V M, "Flow through Weir Notches with Thin Edges and Full Contractions," Journal of Agricultural Research, US Department of Agriculture, Vol 23, March 6, 1916, p 1051.

13. Villemonte, James R, "Submerged-weir Discharge Studies," Engineering News Record, December 25, 1947, p 866-869.

14. Bazin, M H, "Experiences nouvelles sur l 'ecoullement on deversoir," Annales des Ponts et Chaussees, Vol 7, Ser 7 1896.

15. Horton, R E, "Weir Experiments, Coefficients and Formulas," Water-Supply Paper 200, Geological Survey, US Department of the Interior, Washington, DC 1907.

16. Rehbock, T, "Wassermessung mit scharfkantigen Uberfallwehren, Z Ver Dtsch Ing, Vol 73 (1929), p 81.7.

17. Hamilton-Smith "Hydraulics", 1886, also reported by Borrell, J G "Fully contracted weirs", J Instn of Engrs, Australia, Vol x, 1929, October p 361.

18. Schweizerisher Ingenieur and Architekten - Verein, S I A Techn Normen No 109 (1924). Normen fur Wassermessungen bei Dorchfuhrung von Abnabmerersuchen and Wasserkraftmaschinen.

19. Jameson, A H, "The development of the Venturi flume", Water and Water Engg Vol 32, (1930), p 105.

20. Ippen, A T, "Channel transistors and controls,"Chap VIII, Engineering Hydraulics (ed H Rouse), John Wiley & Sons, London, 1950.

21. Hall, G W, "Analytical determination of the discharge characteristics of broad-crested weirs using boundary layer theory". Proc Instn Civ Engrs 1962, Vol 22, June, pp 172-190.

22. Harrison, A J M, "The streamlined broad-crested weir", Proc Instn Civ Engrs, 1967, Vol 38 December pp 657-678.

23. Harrison, A J M, "Boundary layer displacement thickness on flat plates,"Proc Am Soc Civ Engrs, 1967, Vol 93, HYd July, pp 79-91.

24. Crump, E S, "A new method of gauging stream flow with little afflux by means of a submerged weir of triangular profile", Proc Instn Civ Engrs, part 1, Vol 1, March 1952, pp 223-242.

25. Burgess, J S, and White, W R, "The Triangular profile (Crump) weir: two dimensional study of discharge characteristics," UK Hydraulics Research Station, report INT 52, May 1966.

26. Crump Weir Design. Water Resources Board Technical Note No 8 1965.

27. Tracy, H J, "Discharge Characteristics of Broad-Crested Weirs," Circular 397, Geological Survey, US Department of the Interior, Washington, DC 1957.

28. Woodburn, J G, "Tests of Broad-Crested Weirs," Transactions of the American Society of Civil Engineers, Vol 96, 1932, p 387.

29. Keutner, Chr, "Stromungsvorgange an breitkronigen Wehrkorpern und Einlaufbauwerken," Der Bauingenieur, Vol 15, 1934, p 366.

30. Prentice, T H, "Hydraulics of the Broad-Crested Weir," Master's thesis, Columbia University, Department of Civil Engineering, New York, NY 1935.

31. Singer, J, "Square-edged broad-crested weir as a flow measurement device," Wat & Wat Engg 1964, Vol 68, No 820, June, pp 229-235.

32. Parshall, R L, 1926, The improved venturi flume: Am Soc Civil Engineers Trans, Vol 89, p 841-851.

33. Parshall, R L, 1932 and 1953, Parshall flumes of large size: Colorado Agr Expt Sta Bull 386; revised as Bull 426A.

34. Robinson, A R, 1957, Parshall measuring flumes of small sizes: Colorado Agr and Mech Coll Agr Expt Sta, Bull 61.

35. Ackers, P, "Comprehensive formulae for critical-depth flumes," Water and Wat Engg 1961, Vol 61, July, pp 296-306.

36. Ackers, P and Harrison, A J M, "Critical-depth flumes for flow measurement in open channels," Hydraulics Research Paper No 5, London, HMSO, 1963.

37. Inglis, C C, "Notes on standing wave flumes and flume meter falls," Gov of Bombay, P W Dept Tech paper No 15, Bombay, 1928.

38. Harrison, A J M, "Some problems concerning flow measurement in steep rivers", J Instn Wat Engrs London, August 1965, Vol 19, No 6, pp 469-477. Discussion February 1966, Vol 20, No 1, pp 10-14.

39. Harrison, A J M and Owen, M W, "A new type of structure for flow measurement in steep streams", Proc Instn Civ Engrs, London, February 1967, Vol 36, pp 273-296.

40. Harrison, A J M and Owen, M W, Discussion of ref 39, Proc Instn Civ Enggrs, London, August 1967, Vol 37, pp 811-818.

41. Harrison, A J M, "Factors governing the choice of a hydraulic structure for flow measurement". Trans VIIth Congress, International Commission on Irrigation and Drainage, Mexico City, 1969.

42. Herschy, R W, "The evaluation of errors at flow measurement stations," Water Resources Board, England, January 1969.