H23
2.5m Flow Channel

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SECTION 1 INTRODUCTION

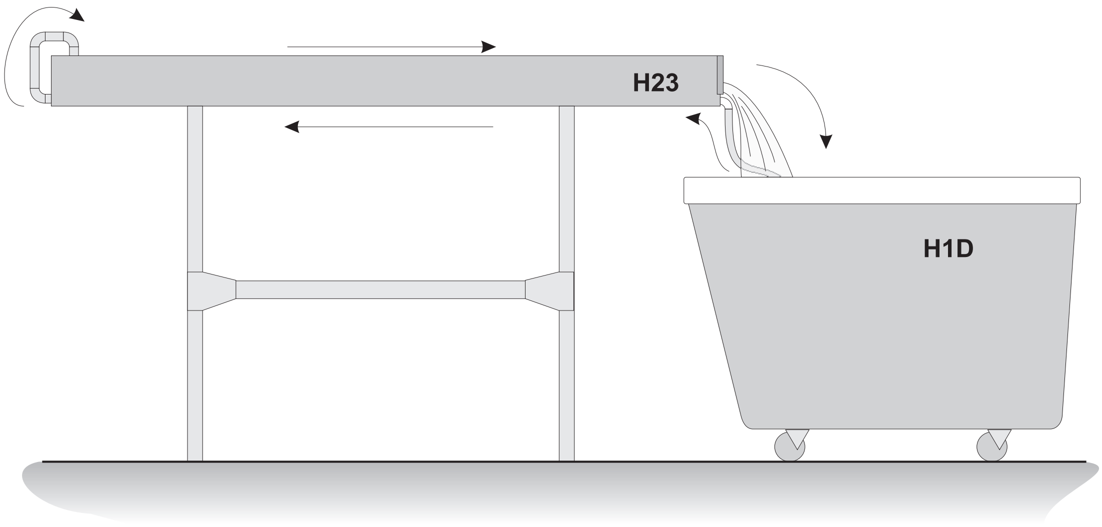
The TecQuipment H23 Flow Channel is a rectangular clear-sided channel, 2.5 m long, adjustable in height to give a change in slope, with the section nominally measuring 53.5 mm wide and 120 mm high.

A H1D Volumetric Hydraulic Bench ancillary supplies the water via the inlet hose attached, and the channel outlet is designed to be positioned over the large volumetric tank of the bench, enabling the flow rate measurements when used with the stopwatch supplied.

A number of width spacer blocks are supplied with the channel, enabling the channel to maintain its width when an experiment is being positioned. At the channel outlet there is a sluice gate, which can be adjusted or set, to control the overall water height within the channel when used in conjunction with the bench supply valve.

At the channel inlet there is a baffle to maintain steady flow during the experiments.

Installation (with optional H1D)



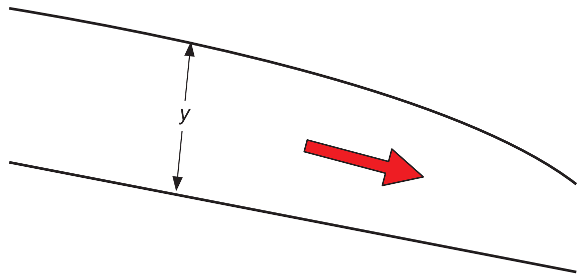
1. Place the H23 on firm, level ground.
2. Push the H1D underneath the sluice gate
3. Connect the feed from the H1D to the H23 inlet pipe, just underneath the sluice gate.
4. Ensure the H1D has enough water and start its pump. Check for leaks in the H23.
5. Make sure that water flows around the H23 as shown in the diagram.
6. Close the sluice gate to trap some water in the flow channel.
7. Use the depth gauge to measure the water depth at each end of the channel. Use the slope adjust control if necessary to adjust the channel until the water depth is equal at each end of the channel. The channel is now level.
8. To adjust the angle of the channel, one turn of the slope adjust control is equal to 0.0675 degrees.

SECTION 2 THE THEORY OF CHANNEL FLOW

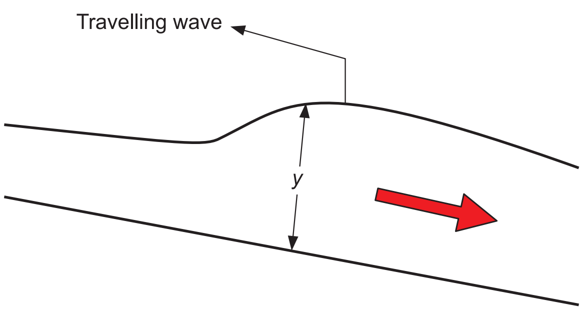
Introduction

This chapter presents a review of the theory of open-channel flow related to the experiments described in this manual. The review is brief, so for more detailed information students should refer to textbooks on fluid mechanics, hydraulics or channel flow*.

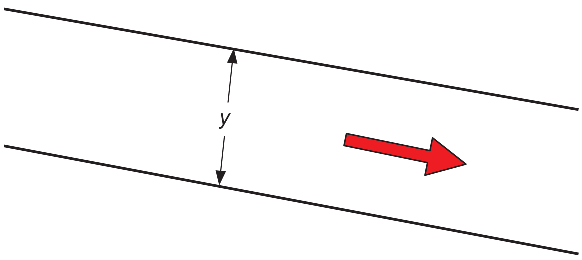
Flow of a liquid in an open channel differs from flow in a closed pipe in that it has a free surface. Although the theory applies to any liquid, the majority of practical applications are to flow of water in rivers and canals. The motion is produced essentially by gravity force, so when considering the mechanics of the flow, the property of specific weight w or ρg of the liquid is of basic importance.



(a) Steady flow-depth *y* independent of time



(b) Unsteady flow-depth *y* varies with time



(c) Uniform flow-depth *y* constant along length

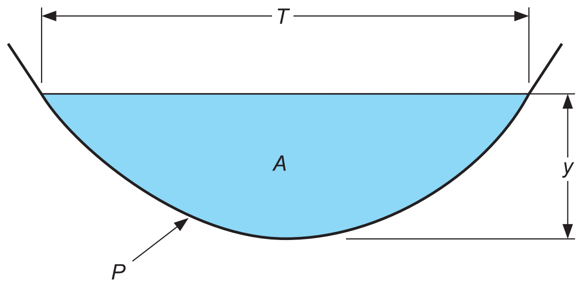
Figure 1 Illustration of steady, unsteady and uniform flow along a channel

It is convenient to distinguish between steady and unsteady flow. In Figure 1(a) the flow is steady because, at each position along the length of the channel illustrated, the depth is independent of time. Note that the flow is non-uniform, since the depth varies along the length of the channel. An example of unsteady flow is that shown in Figure 1(b), where the depth at a chosen point changes as the travelling wave passes.

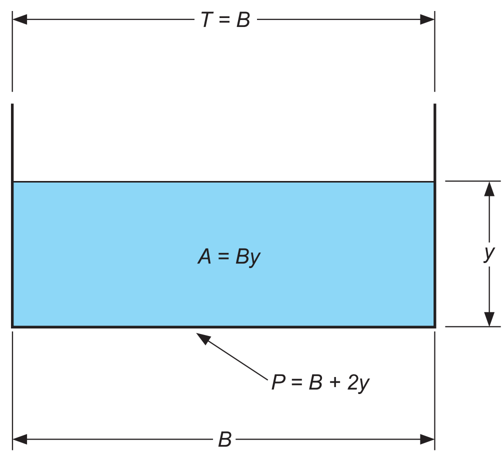
The case of uniform flow is shown in Figure 1(c). Although this condition of constant depth along the channel is of considerable theoretical importance, it rarely occurs in practice and even in the laboratory is not easy to achieve.

Many students will be familiar with the concept of turbulent flow, in which rapid fluctuations occur in velocity. The distinction between steady and unsteady flow in open channels is not concerned with such turbulent fluctuations. Any of the flow conditions illustrated in Figure 1 could be turbulent; almost all open-channel flows are in fact turbulent.

Properties of the Channel Cross-Section



(a) Arbitrary cross-section



(b) Rectangular cross-section

Figure 2 Definition diagrams for channel cross-sections

As a preliminary to analysis of channel flow, it is necessary to define certain terms relating to the cross-section of the channel. Figure 2(a) shows a channel of

* See, for instance:
Massey, BS; Mechanics of Fluids. Van Nostrand Reinhold, London.
Chow, VT; Open-Channel Hydraulics. McGrawHill, New York.

arbitrary cross-section, having depth y measured from the channel bottom, and area A. The part of the perimeter of the cross-section where the liquid is in contact with the solid boundary is called the wetted perimeter P.

When considering flow along a length of channel, it is found that the ratio of gravity to frictional forces includes the term A/P, which has dimensions of length. This occurs so frequently in the theory that it is referred to as hydraulic radius, R:

$$R = \frac{A}{P} \tag{1}$$

Another quantity that arises in treatment of certain problems, including wave motions, is the term A/T where T is the surface width and also has dimensions of length. This is known as the hydraulic depth, D:

$$D = \frac{A}{T} \tag{2}$$

For the particular case of the rectangular cross-section shown in Figure 2(b):

$$A = By$$
$$P = B + 2y$$

and

$$T = B$$

So the definitions of hydraulic radius R and hydraulic depth D reduce to:

$$R = \frac{By}{B + 2y} \tag{3}$$

and

$$D = y \tag{4}$$

Equation (3) shows that as the channel becomes very wide in relation to its depth, i.e. when $B >> y$, then in the denominator the term $2y$ may be neglected in comparison with B. The hydraulic radius simply becomes $R = y$.

If the channel is very narrow in comparison with its depth, i.e. when $B << y$, then in the denominator the term B may be neglected in comparison with $2y$. The hydraulic radius in this case simply becomes $R = B/2$.

Volume Flow, Mass Flow, Kinetic Energy and Momentum Flow

Many students will already know that for steady flow along a pipe the velocity varies from point to point across any chosen section. For instance, in the case of laminar flow through a tube of circular section, the velocity increases parabolically from zero at the wall to a maximum at the centre. In a similar manner, the velocity over the cross-section of an open channel varies from zero along the wetted perimeter to a maximum at a point that usually lies some way below the top surface. The velocity at the surface is of course non-zero because there is no solid restraint to motion at this surface.

Despite the non-uniformity over the section, many useful results may be obtained by considering only the mean velocity V over the section. This is related to the discharge (i.e. rate of volume flow) Q and the cross-sectional area A by the equation of continuity:

$$Q = AV \tag{5}$$

For the particular case of the rectangular section,

$$Q = ByV \tag{6}$$

The rate of mass flow $\dot{m}$ is related to Q and the liquid density ρ by:

$$\dot{m} = \rho Q$$

so that

$$\dot{m} = \rho AV \tag{7}$$

which, for the rectangular channel becomes:

$$\dot{m} = \rho ByV \tag{8}$$

This mass flow carries kinetic energy and momentum with it. The kinetic energy flow rate through the cross-section is K, given by:

$$K = \tfrac{1}{2}\dot{m}V^2$$

and substituting from Equation (7):

$$K = \tfrac{1}{2}\rho AV^3 \tag{9}$$

or

$$K = \tfrac{1}{2}\rho ByV^3 \tag{10}$$

in the rectangular section. Similarly, the momentum flow rate through the section is J, given by:

$$J = \dot{m}V$$

Substituting from Equation (7):

$$J = \rho AV^2 \tag{11}$$

and for the rectangular section:

$$J = \rho ByV^3 \tag{12}$$

Froude Number and Reynolds Number

In analysis of the flow an essential dimensionless parameter known as the Froude number emerges. This is defined in terms of the velocity V, hydraulic depth D and acceleration due to gravity g, by the equation:

$$Fr = \frac{V}{\sqrt{gD}} \tag{13}$$

Now the velocity c of an infinitesimal wave, driven by gravity along an open channel, is:

$$c = \sqrt{gD} \tag{14}$$

So Fr may be regarded as the ratio of flow velocity V to the velocity c of a gravity wave. Alternatively, we can derive the Froude number using the normal methods of dimensional analysis, by associating the physical quantities ρ, V, D and g together into a dimensionless group.

A further dimensionless parameter which emerges when the effects of fluid viscosity are taken into account is the Reynolds number, Re. This is defined in terms of the velocity V, hydraulic radius R and fluid kinematic viscosity ν or absolute viscosity μ by:

$$Re = \frac{VR}{\nu} \quad \text{or} \quad \frac{\rho VR}{\mu} \tag{15}$$

Again, the Reynolds number may be derived using dimensionless analysis, this time associating ρ, V, D and μ together into a dimensionless group. The use of R to represent length in this equation should be contrasted with the use of D in Equation (12). We shall see later that the quantity that arises naturally in consideration of friction is hydraulic radius R, so it is natural to use R rather than D in the definition of Reynolds number.

Total Head and Specific Energy; Friction Slope

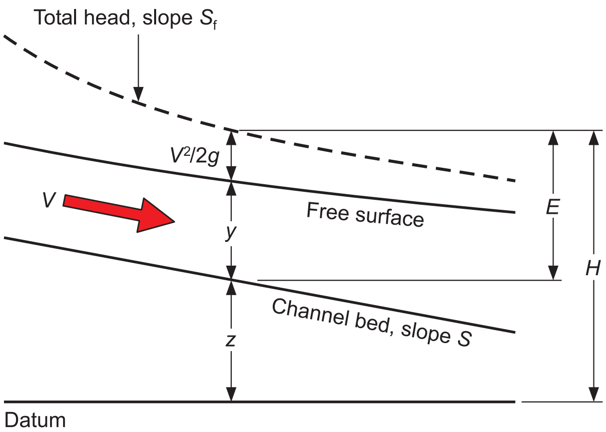


Figure 3 Non-uniform steady flow along a channel

We now consider the general characteristics of steady but non-uniform flow along a channel as shown in Figure 3. The depth y varies slowly with distance x along the channel, and the slope S of the bed is sufficiently small for the depth y, measured vertically, to be indistinguishable from the depth measured in the direction normal to the bed.

Taking atmospheric pressure as the datum of pressure, then the total head at a point in the free surface is:

$$H = y + z + \frac{V^2}{2g} \tag{16}$$

on the assumption of uniform velocity V over the whole cross-section.

For other points of the cross-section, the pressure varies hydrostatically from zero at the surface to wy at the bed (w = ρg, the specific weight), and the total head H remains constant over the whole section. It is frequently useful to relate total head to the bed of the channel, so the term specific energy E is adopted, defined by:

$$E = y + \frac{V^2}{2g} \tag{17}$$

Recalling Equation (5), this may be written in the form:

$$H = y + \frac{Q^2}{2gA^2} \tag{18}$$

Note that the slope S_f of the line of total head (known as the friction slope) is different from the slope of the water surface, which in turn is different from the slope S of the bed.

Uniform Flow; The Chezy Equation

In the special case of uniform flow shown in Figure 4, the friction slope S_f, the slope of the water surface, and the bed slope S are identical. The subscript suffix 'o' indicates the uniform condition, so the depth is now y_o, the velocity is V_o, and so on. Under this uniform flow condition, the shear stress τ_o at the wetted perimeter, as indicated on the diagram by the arrows pointing upstream, produces a resisting force which exactly balances the component of gravity force along the channel.

Consider an element of length δx; the wetted area is $P_o\delta x$ so the frictional resistance is $\tau_o P_o \delta x$. The volume of fluid contained in the element is $A_o\delta x$ so the gravity force on this fluid is $\rho g A_o \delta x$ acting vertically downwards. The component of this in the direction of the channel is $\rho g A_o S \delta x$ as indicated on Figure 4. Equating this to the frictional resistance:

$$\tau_o P_o \delta x = \rho g A_o S \delta x$$

or

$$\tau_o = \rho g \frac{A_o}{P_o} S$$

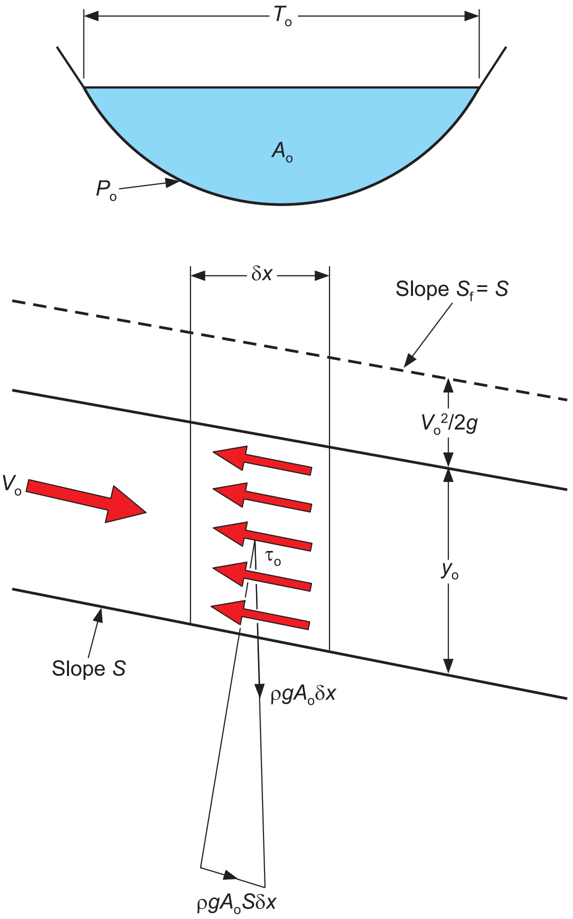


Figure 4 Uniform steady flow along a channel

Following the practice used in treatment of pipe flow the shear stress, τ_o is now written as:

$$\tau_o = \tfrac{1}{2}\rho V_o^2 f$$

where f = Dimensionless friction factor.

Eliminating τ_o from the two equations:

$$\tfrac{1}{2}\rho V_o^2 f = \rho g \frac{A_o}{P_o} S$$

Now, following Equation (1):

$$\frac{A_o}{P_o} = R_o$$

where R_o = Hydraulic radius for this uniform flow.

Simplifying the result, we obtain:

$$V_o = \left[\frac{2g}{f}\right]^{1/2} (R_o S)^{1/2}$$

It is common practice to replace the factor $(2g/f)^{1/2}$ by a simple factor C, called the Chezy factor, so we have:

$$V_o = C(R_o S)^{1/2}$$

(19)

This is the Chezy equation for uniform steady flow along a channel. As indicated earlier, V_o is the velocity and R_o the hydraulic radius under uniform conditions. It is permissible to omit the suffixes and write:

$$V = C(RS)^{1/2}$$

(20)

provided we recall that this equation is restricted to conditions of uniform flow only.

In the case of pipe flow, many students will know that the friction factor f varies with Reynolds number and with pipe surface roughness. Similarly, the Chezy factor, C, varies with the size and roughness of the channel.

Note that C is not dimensionless, having dimensions of $\left[L^{1/2}T^{-1}\right]$, so the numerical value that applies in a given case will vary with the units adopted. For example, the typical value $C = 50\ \mathrm{m}^{1/2}\mathrm{s}^{-1}$ becomes $C = 90.6\ \mathrm{ft}^{1/2}\mathrm{s}^{-1}$ in Imperial units•.

Non-Uniform Flow; Normal Depth, Critical Depth

Returning to the case of non-uniform flow in Figure 3, we know that the friction slope S_f differs for S because E is not constant along the channel. We have, from the geometry of Figure 3:

$$S_f = S - \frac{dE}{dx}$$

(21)

By use of Equation (18) and the expression $\frac{dA}{dy} = T$, from Figure 2, we obtain (after some manipulation) the following result for rate of change of depth along the channel:

$$\frac{dy}{dx} = \frac{S - S_f}{1 - Fr^2}$$

(22)

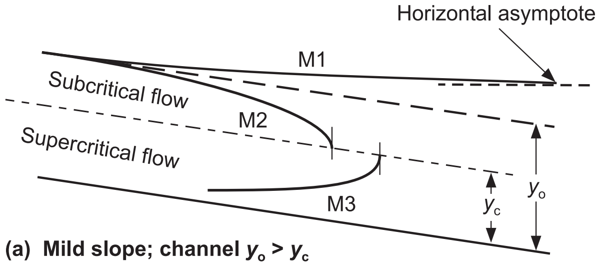
in which Fr is the Froude number defined by Equation (13).

It is instructive to consider the general behaviour of Equation (22), without going into great detail. We first define two further terms; the normal depth y_o at which uniform flow appears when $S = S_f$ and $dy/dx = 0$, and the critical depth at which the Froude number is unity,

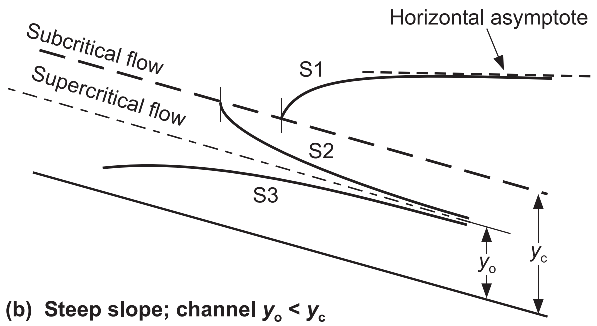
• 1 ft = 0.3048 m exactly.

so that $Fr^2 = 1$ and $dy/dx \to \infty$. It is necessary to draw a distinction between the cases when $y_o > y_c$ and $y_o < y_c$.

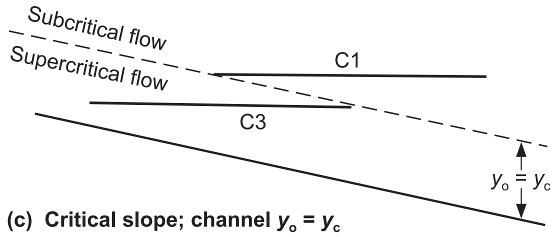
When $y_o > y_c$, the normal flow is subcritical, so $Fr < 1$, and the channel is said to be of mild slope.



(a) Mild slope; channel $y_o > y_c$



(b) Steep slope; channel $y_o < y_c$



(c) Critical slope; channel $y_o = y_c$

Figure 5 Surface profiles in channels of various slopes

Consider the following cases as illustrated in Figure 5(a):

1. $y > y_o > y_c$

Since $y > y_o$, the friction slope will be less than the channel slope, viz. $S_f < S$. Also, since $y > y_c$, the flow is subcritical so $Fr^2 < 1$. Both numerator and denominator in Equation (22) are positive, so dy/dx is also positive.

Moreover, as $y \to y_o$, $S_f \to S$, then $dy/dx \to 0$ so $y = y_o$ is an asymptote to the surface profile.

As $y \to \infty$, $V \to 0$ so $S_f \to 0$ because friction becomes vanishingly small at zero velocity; also $Fr \to 0$. So, in Equation (22), $dy/dx \to S$ as y tends to infinity, which means that the surface tends to a horizontal asymptote. This is the curve M1 on Figure 5(a).

2. $y_o > y > y_c$

The friction slope is now greater than the channel slope, viz. $S_f > S$, but $Fr^2 < 1$ still. The numerator is therefore negative and the denominator is positive in Equation (22) so that dy/dx is now negative.

As $y \to y_o$, $S_f \to S$ as before and $y = y_o$ is an asymptote to the surface profile.

As $y \to y_c$, $Fr^2 \to 1$, so $dy/dx \to \infty$ and the surface profile becomes vertical. This is the curve M2 on Figure 5(a).

3. $y_o > y_c < y$

As for (2) above, $S_f > S$, but now $Fr^2 > 1$. Both numerator and denominator are negative and dy/dx is positive, and dy/dx tends to infinity as $y \to y_c$. This is the curve M3 on Figure 5(a).

In a similar manner, the S1, S2 and S3 profiles shown in Figure 5(b) may be inferred for the case of a steep slope channel defined by the condition $y_o < y_c$, so that the normal flow is supercritical. The special case when $y_o = y_c$ is the critical slope channel, for which the profiles degenerate into horizontal surfaces is indicated in Figure 5(c).

Figure 6 illustrates how these profiles may typically be produced by the presence of various structures in the length of a channel. These can readily be set up in the laboratory and are well worth qualitative study.

Rapidly-Varied Flow

The preceding discussion of gradually varied flow provides a basis for understanding changes produced by frictional effects over long lengths of channel. However, depth changes frequently occur over relatively short lengths at structures such as weirs. Different treatments, which usually neglect frictional effects, are then required. We use the concepts of continuity, energy and momentum.

Variation of Specific Energy with Depth of Flow

In many of the experiments we find that there are considerable changes of depth of flow under conditions of constant specific energy. It is instructive to consider an associated question, namely, how the specific energy varies with depth of flow. Consider a given discharge Q passing along a channel with a given cross-section, the notation being defined in Figure 2(a). The specific energy E is given by Equation (18) as:

$$E = y + \frac{Q^2}{2gA^2} \tag{18}$$

We now imagine y to vary, keeping Q constant. As $y \to \infty$ then $A \to \infty$ also. The first term on the right-hand side of Equation (18) tends to infinity and the second tends to zero so $E \to \infty$ as $y \to \infty$.

Now consider the case when $y \to 0$ and $A \to 0$. The first term on the right-hand side of Equation (18) now tends to zero, but the second term tends to infinity, so again $E \to \infty$. E therefore tends to infinity both when $y \to 0$ and $y \to \infty$, so we expect E to have some minimum value in this range of y. The stationary value of E is found by differentiation of Equation (18):

$$\frac{dE}{dy} = 1 + \frac{Q^2}{2g}\frac{d}{dy}\left[\frac{1}{A^2}\right]$$

or

$$\frac{dE}{dy} = 1 + \frac{Q^2}{2g}\frac{d}{dA}\left[\frac{1}{A^2}\right]\frac{dA}{dy}$$

Now

$$\frac{dA}{dy} = T$$

As may be seen in Figure 2(a), imagining a small increase δy in the depth produces a small increase $A = Ty$ in the cross-sectional area.

Therefore,

$$\frac{dE}{dy} = 1 - \frac{Q^2 T}{g\,A^3}$$

Recalling Equation (2):

$$D = \frac{A}{T}$$

and from Equation (5):

$$Q = AV$$

This result may be rewritten as:

$$\frac{dE}{dy} = 1 - Fr^2 \qquad \textbf{(23)}$$

The minimum value of E occurs when $\frac{dE}{dy}$ = 0, i.e. when Fr = 1. This has already been identified as the critical condition, for which the subscript 'c' is now used. Putting the critical condition into Equation (17) for E:

$$E = y + \frac{V^2}{2g}$$

We obtain the minimum value E_c as:

$$E_c = y_c + \frac{V_c^2}{2g}$$

at the critical condition,

$$\frac{V_c^2}{g\,D_c} = 1$$

and substituting in the previous equation:

$$E_c = y_c + \frac{D_c}{2} \qquad \textbf{(24)}$$

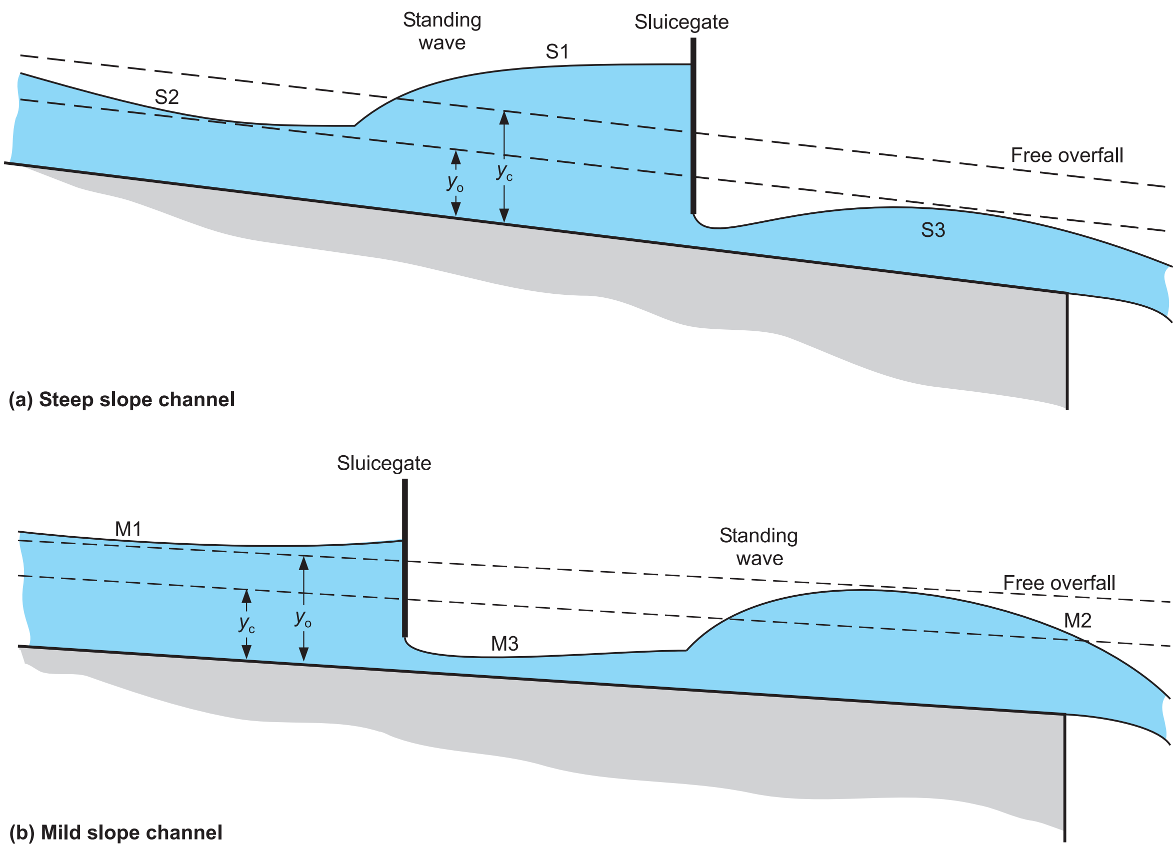


Figure 6 Typical occurrence of surface profiles in mild and steep sloped channels

For the particular case of a rectangular channel in which $D = y$ as shown in Equation (4):

$$E_c = \frac{3}{2} y_c \tag{25}$$

The foregoing analysis is illustrated in Figure 7, where specific energy E is plotted horizontally against depth y plotted vertically. The graph of y is simply the dashed straight line through the origin, and the graph of $\left(V^2/2g\right)$ is the dashed curve falling from left to right. Adding the curves horizontally gives the specific energy E as the sum of y and $\left(V^2/2g\right)$.

For any value of E greater than E_c there are two possible depths of flow - one greater than y_c in the range of subcritical flow and one less than y_c in the range of supercritical flow. No solution is possible if E is less than E_c. It is again emphasised that the analysis and the diagram of Figure 7 apply only to a specified flow rate Q along a channel of specified cross-sectional dimensions.

Figure 8 shows how the flow may pass through a constriction in a channel from the subcritical state, the specific energy being constant throughout. The channel is of rectangular cross-section for simplicity and has a width B upstream and downstream of the contraction, which has throat width b. For any specified discharge Q we may construct two curves of specific energy, one for width B, and the other for width b. Since, at any chosen value of y, the velocity in the throat is higher than in the upstream and downstream sections the curve for the throat width b lies to the right of the other curve, as seen on Figure 8.

There is only one possible solution to the question of determining the depths at the points a_1, a_2 and c which lie upstream, downstream and at the throat, and this solution is obvious from the specific energy diagram. It is given by the vertical line a_1 c a_2 on the diagram, through the point of minimum specific energy of the curve for width b. The line must be vertical since the specific energy is constant. It cannot be moved to the left, as there then would be no intersection with the curve for width b, and if it were moved to the right there would be two different depths at the single point c, which is absurd.

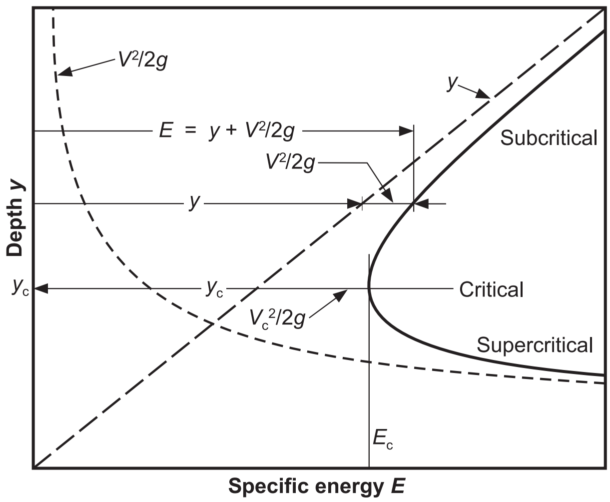


Figure 7 Variation of specific energy with depth of flow

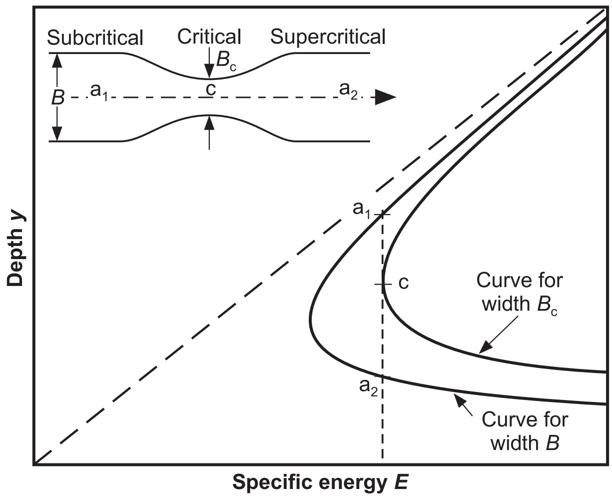


Figure 8 Variation of specific energy with depth of flow from a subcritical state

SECTION 3 FLOW OVER A SHARP-CRESTED WEIR

Introduction

The sharp-crested weir is frequently used as a device for measuring discharge in a channel. It is simple to install, and provided that it conforms to prescribed requirements, it may be used with confidence in conjunction with standard calibration data. In this experiment we establish the relationship between head over the weir and discharge.

Equipment

Sharp-crested weir with air vent, depth gauge and vernier calliper.

Procedure

The channel is first set horizontal. The horizontal position may be checked easily using the depth gauge at each end of the channel. Measure the height of the weir using the vernier calliper. It is then placed vertically in the channel, upstream of the outlet. With the point of the depth gauge resting on the weir crest, the scale is read for the datum, so that subsequent measurement of water level is referred to zero at the weir crest.

Water is then admitted to the channel until a convenient maximum flow is obtained. It will not be possible to achieve any flow if the water is too low, e.g. below the level of the plenum box above the impeller. The maximum may be set either by the pump capacity or the available depth of flow in the channel. The discharge is then measured. The head over the weir is measured using the depth gauge upstream of the weir. To obtain a good accuracy, it is desirable to measure the head several times over the interval and to record the mean value. The flow is then reduced in stages, and at each stage both the discharge and the head are measured.

It is important that all times during the measurements, the underside of the jet issuing from the weir should spring clear of the downstream face of the weir plate, and vent pipes are provided at assist the separation of the jet by admitting air into this region. From time to time, and particularly at low heads, it is necessary to blow a little air along this pipe to maintain the separation of the jet from the weir plate. Measurements should cease when it is no longer possible to ensure separation of the jet from the weir plate.

Theory

Figure 9 illustrates the flow. The height of the crest above the channel bed is a and the height of the water surface above the crest is h. Considering a typical streamline from a point in the upstream flow to a point in the plane of the weir, we note that on the assumption of uniform velocity V in the upstream flow, the specific energy E is given by:

$$E = a + h + \frac{V^2}{2g} \tag{26}$$

and this specific energy is constant over the cross-section.

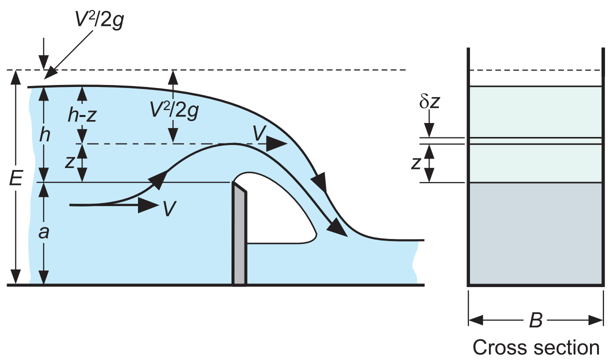


Figure 9 Flow over a sharp-crested weir

Suppose that the velocity along a typical streamline in the plane of the weir is v and the height of the streamline above the weir crest is z. Then, if there is no loss of head along the streamline, and the pressure in the plane of the weir is atmospheric, Bernoulli's equation is:

$$E = a + h + \frac{V^2}{2g} = a + z + \frac{v^2}{2g} \tag{27}$$

Ignoring the velocity head $\frac{V^2}{2g}$ in the approach channel, Equation (27) gives the velocity over the weir as:

$$v = \sqrt{2g\left(h - z\right)} \tag{28}$$

The element of discharge through an element of height δz and width B is then:

$$\delta Q = v B \delta z$$

or

$$\delta Q = \sqrt{2g\left(h - z\right)} B \delta z \tag{29}$$

provided that v is horizontal.

The total discharge Q may then be obtained, ignoring the contraction of the jet in the plane of the weir, as:

$$Q = \int_{o}^{h} \sqrt{2g\left(h - z\right)}\, B dz \tag{30}$$

Performing the integration,

$$Q = \frac{2}{3} B \sqrt{2g}\, h^{3/2} \tag{31}$$

It is now necessary to introduce a dimensionless discharge coefficient C into the equation to allow for the many assumptions made in the derivation, giving:

$$Q = C \frac{2}{3} B \sqrt{2g}\, h^{3/2} \tag{32}$$

as the weir equation.

In the experiment we aim to verify the power-law dependence of Q and h, and to establish the value of C. Although the theory is specifically for a sharp-crested weir, a similar treatment clearly applies to spillways with rounded crests, although the value of C will vary from case to case.

SECTION 4 THE VENTURI FLUME

Introduction

The venturi flume is formed by a smooth contraction in the cross-section of a channel, along which the water accelerates to a throat, followed by a smooth expansion back to the original cross-section. In the expanding section the water may continue to accelerate in a supercritical flow, or it may decelerate in a critical flow. If there is supercritical flow in the expanding section, the conditions at the throat must be critical. It is this feature which permits the flume to be used as a measuring device, needing only measurement of the upstream head to obtain the discharge. Compared with the weir it is usually more expensive to build, but it has the advantages of utilising a lower head than that required by a weir, and of being effectively self-cleaning.

Equipment

Venturi flume, depth gauge, vernier calliper, Pitot tube; a sluice gate is also desirable.

Procedure

The venturi flume, comprising two trapezoidal liners which fit to the side of the channel, is set carefully in position at a station upstream of the outlet from the channel. The leading edge of the contraction is best placed at a convenient point of the scale which runs along the channel. Since the surfaces of the venturi flume are carefully machined, when the liners are gently pressed against the moistened channel walls, they are retained in position by surface tension. The width of the channel and of the throat of the flume should be measured using a vernier calliper.

The depth gauge is now ready for a datum when the point then touches the channel bed.

Before starting the quantitative experiment, it is worthwhile observing the general characteristics of the flows produced in the flume. By steadily increasing the flow, water is admitted to the channel, and the flow through the flume may then be observed to accelerate along the contracting and the diverging portions, becoming super-critical in the diverging section, and remaining supercritical right up to the channel outlet. If the sluice gate is now placed near the outlet, and lowered until the gate interferes with the flow, a standing wave is produced between the venturi flume outlet and the sluice gate. By reducing the sluice gate opening, the standing wave may be forced upstream into the venturi flume as indicated in Figure 10.

Changes in the downstream conditions have no effect on the flow upstream of the venturi flume, as can easily be checked by observing the point gauge in upstream surface. When the standing wave advances to the throat, however, the flow there ceases to be critical, and the upstream level will then rise. These phenomena are worth observing, and should be interpreted in the light of the discussion on page 6.

To calibrate the venturi flume, measurements are taken of flow and of depth upstream with conditions of supercritical flow downstream of the throat. Starting with maximum flow, the depth at a point upstream is measured during the interval. The flow is then reduced in steps, the flow and depth being recorded at each stage.

It is also instructive to establish the profile of the water surface through the flume. The flow is set to a chosen value and the depth gauge is traversed along the length. Readings are taken at convenient points. The profile of the specific energy is found by traversing the Pitot tube, but in this case much wider intervals may be used, as the specific energy is virtually constant.

Theory

Figure 11 indicates flow through the venturi flume, with subcritical conditions upstream. In the contracting section the flow accelerates to the critical condition which occurs in the throat, and downstream of the throat, the acceleration continues in supercritical flow. In the simple one-dimensional theory presented here, discontinuities occur in the surface slope, as indicated by the dotted line in Figure 11. In practice, however, the water surface falls smoothly through the flume.

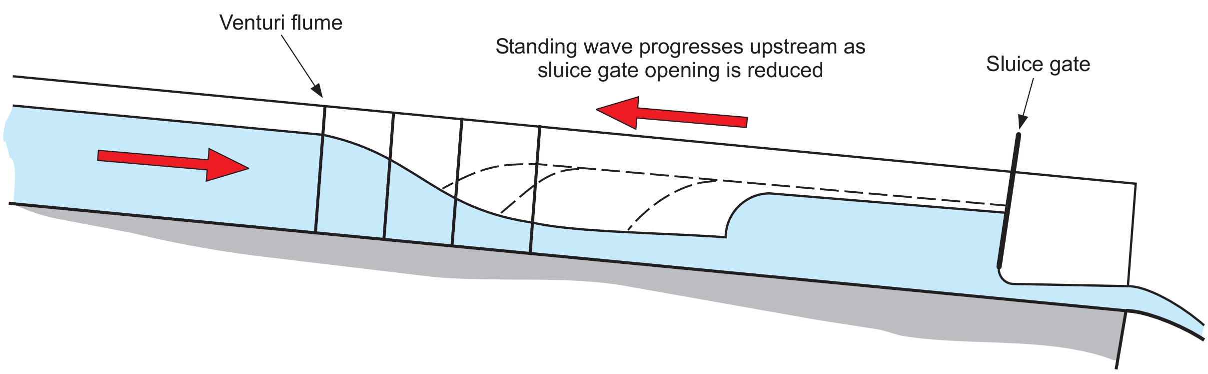


Figure 10 Control of position of standing wave by adjustments of sluice gate opening

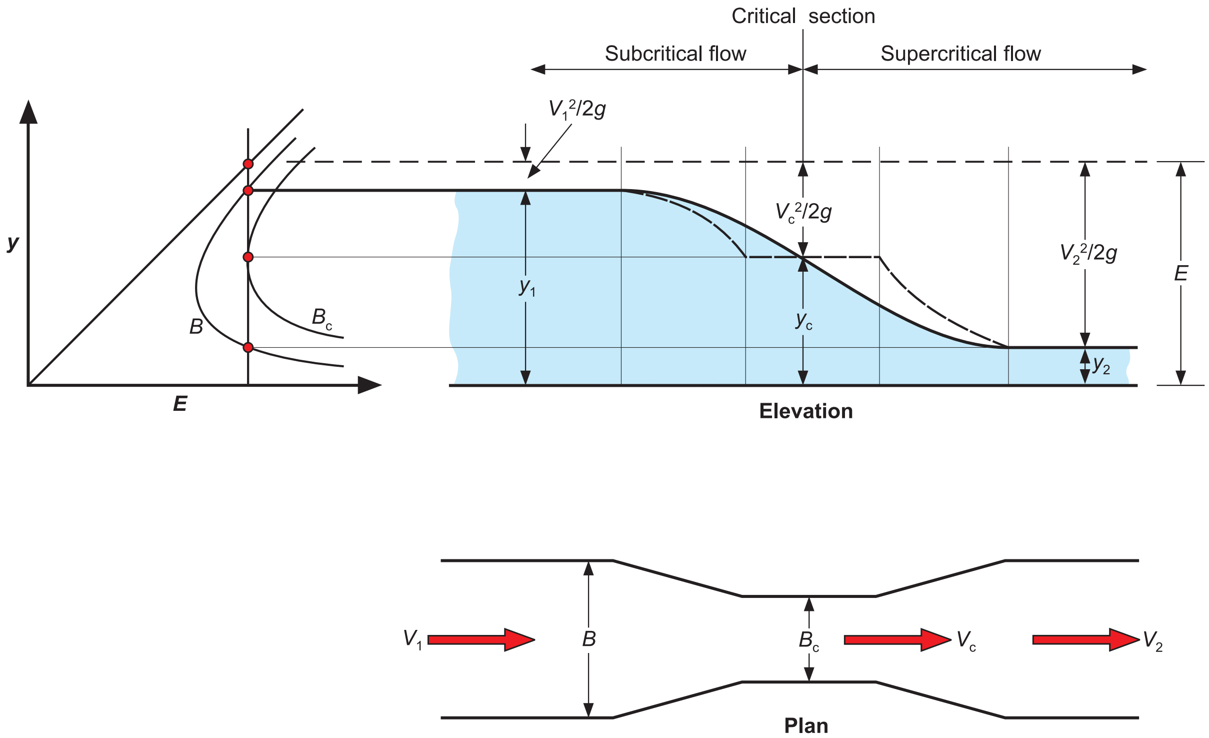


Figure 11 Flow through the venturi flume

To obtain an expression for discharge Q in terms of specific energy E, we note that the critical condition at the throat gives, according to Equation (25):

$$E = \frac{3}{2} y_c$$

(33)

Since the value of the Froude number at the throat is unity:

$$\frac{V_c^2}{g\,y_c} = 1$$

(34)

The discharge through the flume is given in terms of the velocity V_c and area $B_c y_c$ at the throat by:

$$Q = B_c y_c v_c$$

(35)

From these three equations, we find after a little reduction:

$$Q = B_c \sqrt{g} \left[\frac{2E}{3} \right]^{3/2}$$

(36)

Now inserting a discharge coefficient C to take account of the reduction in Q due to frictional losses:

$$Q = C B_c \sqrt{g} \left[\frac{2E}{3} \right]^{3/2}$$

(37)

SECTION 5 FLOW UNDER A SLUICE GATE

Introduction

The sluice gate provides a convenient means of flow regulation, especially in irrigation and drainage schemes where flow has to be distributed in networks of interconnected channels. In this experiment we measure the discharge under the gate and establish the effective coefficient of discharge.

The sheet of water emerging from the aperture is supercritical so that a standing wave can be formed downstream of the gate, and the experiment may be extended to study this standing wave and the variation of specific energy through it.

Equipment

Sluice gate, depth gauge, Pitot tube; a second sluice gate or other restriction for controlling the downstream flow is also desirable.

Procedure

The sluice gate is fixed in the channel at a convenient station upstream of the channel outlet. The aperture is set accurately to the desired value. The depth gauge is now ready for a datum when the point touches the channel bed.

To obtain the discharge characteristic of the gate, a set of values of upstream head and discharge is obtained with conditions of supercritical flow downstream. The flow to the channel is gradually increased by opening the pump control until the water level upstream of the gate settles to the highest value which may conveniently be read on the depth gauge. The flow is then measured along with the head upstream of the gate. The flow is then reduced in stages and at each stage both the discharge and the head are measured.

A standing wave may be produced in the flow by a suitable restriction near the channel outlet; for instance a further sluice gate may be used. By traversing the depth gauge and then the Pitot tube along the length of the channel, the surface profile and the specific energy profile may be recorded. The discharge should be measured from time to time during these traverses.

Theory

The essential features of the flow are shown on Figure 12. The depth upstream of the sluice gate is h, and the jet emerging from the aperture of height a has a thickness C_c, where C_c is the contraction coefficient. The static head across the aperture varies from $(h-a)$ at the upper edge to h at the bottom, so the average static head is $(h-a/2)$. Writing:

$$E = h + \frac{V^2}{2g}$$

where V is the velocity of approach, the average total head on the aperture is $(E-a/2)$.

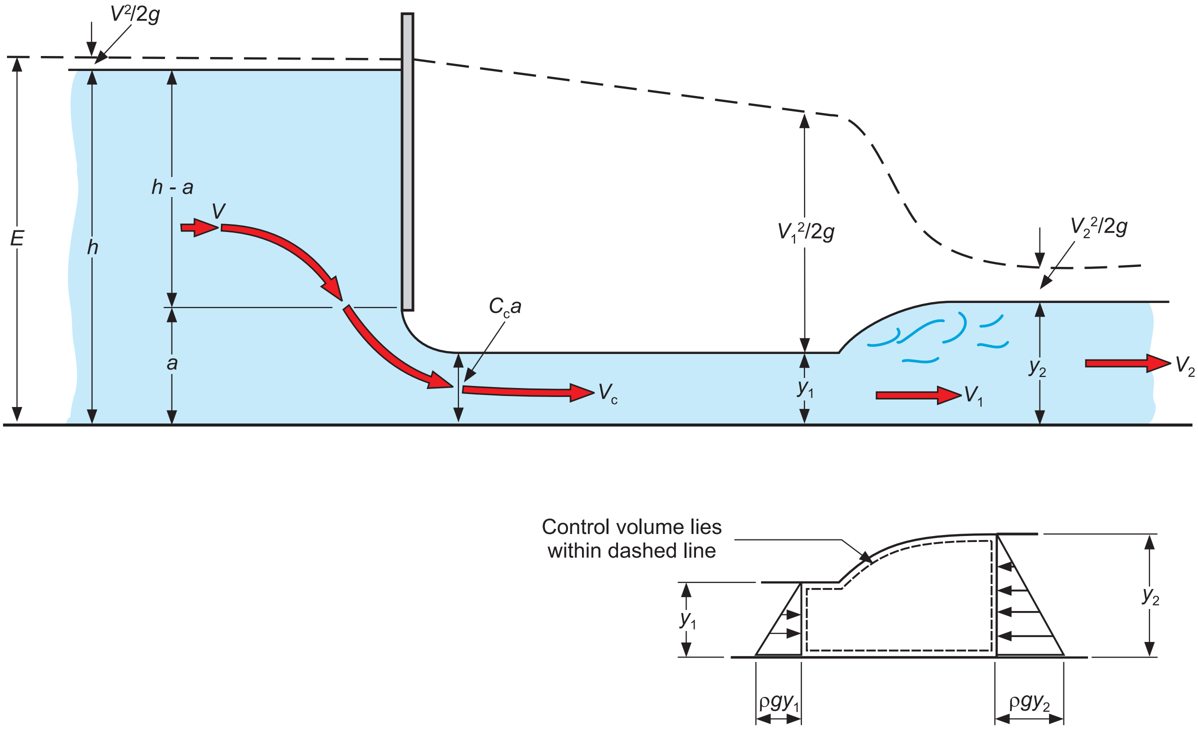


Figure 12 Flow under a sluice gate with a standing wave downstream

The average velocity in the contracted section, across which the pressure is hydrostatic, is therefore:

$$V_c = C_v \sqrt{2g\left(E - \frac{a}{2}\right)}$$

(38)

where C_v is the velocity coefficient which allows for the slight loss in total head up to the section.

The discharge Q may now be written as the product of flow area and velocity:

$$Q = C_c\, a\, B C_v \sqrt{2g\left(E - \frac{a}{2}\right)}$$

(39)

Replacing the product C_cC_v by the discharge coefficient C, we obtain:

$$Q = C\, a\, B \sqrt{2g\left(E - \frac{a}{2}\right)}$$

(40)

Using the measured values of Q and E we can deduce the value of C from this equation

Coming now to the standing wave, where the depths of flow upstream and downstream are y_1 and y_2, there is an irreversible loss of specific energy through the wave, the energy being dissipated by turbulence in the flow. It is therefore not permissible to use Bernoulli's equation through the wave. However, the continuity and momentum equations are still applicable. The equation of continuity may be written, following Equation (8):

$$\dot{m} = \rho B y_1 V_1 = \rho B y_2 V_2$$

(41)

To derive the momentum equation, we first obtain the force on the fluid within the control volume shown on Figure 12 due to hydrostatic pressure. The pressure on the left-hand section varies linearly from zero at the surface to gy at the bed, so the mean pressure is $(\rho g y_1)/2$. This acts upon area By_1. Applying the same reasoning to the right-hand section, the net force from left to right due to hydrostatic pressure is seen to be:

$$\frac{\rho g B y_1^2}{2} - \frac{\rho g B y_2^2}{2}$$

or

$$\frac{\rho g B\left(y_1^2 - y_2^2\right)}{2}$$

The incoming momentum flux at the left-hand section is $\rho B y_1 V_1^2$, following Equation (12), and the outgoing momentum flux at the right-hand section is $\rho B y_2 V_2^2$. So the net momentum outflow rate from left to right is:

$$\rho B\left(y_2 V_2^2 - y_1 V_1^2\right)$$

Equating the net force to the net momentum outflow rate gives the momentum equation as:

$$\frac{\rho g B\left(y_1^2 - y_2^2\right)}{2} = \rho B\left(y_2 V_2^2 - y_1 V_1^2\right)$$

which reduces to:

$$y_1^2 - y_2^2 = \frac{2}{g}\left(y_2 V_2^2 - y_1 V_1^2\right)$$

(42)

Eliminating V_2 between Equation (41) and Equation (42) and simplifying gives:

$$y_1 + y_2 = \frac{2 y_1}{g y_2} V_1^2$$

(43)

Writing:

$$Fr_1^2 = \frac{V_1^2}{g y_1}$$

(44)

leads to the quadratic equation:

$$\frac{y_2^2}{y_1} + \frac{y_2}{y_1} - 2Fr_1^2 = 0$$

(45)

for which the positive root is:

$$\frac{y_2}{y_1} = \frac{1}{2}\left[-1 + \left(1 + 8Fr_1^2\right)^{1/2}\right]$$

(46)

This equation may be used to predict the depth of flow downstream of a standing wave in terms of the conditions upstream of the wave.

SECTION 6 SUGGESTIONS FOR FURTHER EXPERIMENTS

The range of possible experiments in the flow channel is very wide. Using additional equipment, the techniques described in the previous sections may readily be adapted to provide many further useful results, and some of the possibilities are outline below.

An important extension of the venturi flume concept is the broad-crested weir illustrated in Figure 13. In this case, the flow over the crest of the weir is critical, so that the discharge equation becomes, following Equation (37):

$$Q = CB\sqrt{g}\left[\frac{2}{3}H\right]^{3/2} \qquad \textbf{(47)}$$

The weir may be constructed from components with various heights *a* and weir lengths *L*, and may have either a square or rounded leading edge. The surface and specific energy profiles may be plotted and compared with calculated values. The equipment for this experiment is included as standard.

The sharp-crested weir is used mainly in laboratory work for accurate gauging, but in the field other forms of weirs and spillways are frequently used, and these may be calibrated using the methods described in previous chapters.

Spillways are frequently used in practice as overflow structures from dams. Provision for overflow is essential to prevent the water from overtopping the dam with possibly disastrous consequences. A spillway usually has a rounded crest which leads to an inclined chute that often is formed on the downstream face of the dam. This chute may terminate in a simple horizontal apron, but if the dam is high, the velocity of the water leaving the apron will lead to excessive scour of the bed of the channel downstream of the apron. In such circumstances, it is necessary to dissipate the kinetic energy of the discharge from the spillway.

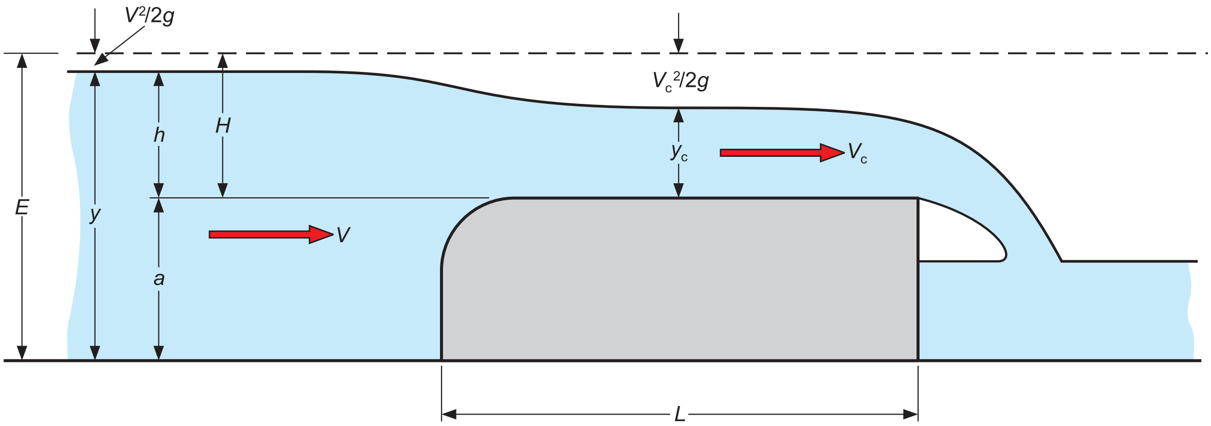
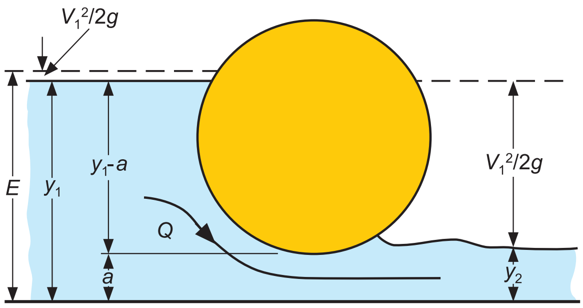


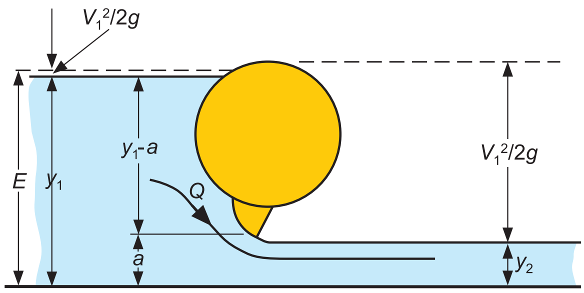
Figure 13 Flow over a broad-crested weir

SECTION 7 THE DRUM GATE

The sluice gate described in Section 5 provides a simple means of flow regulation. The gate is located in vertical guides which take the hydrostatic thrust, and in large constructions a considerable lifting force is required to shift the gate against frictional resistance in the guides.



a) Drum gate



b) Drum gate with sector

Figure 14 Typical drum gates

Various improved mechanical arrangements have been used to avoid this difficulty. The drum gate illustrated in Figure 14(a) is essentially a cylindrical roller, which may be moved either vertically or along an inclined track, usually by rolling on a rack-and-pinion mechanism. It is frequently designed to work with water passing both under and over the drum. In some constructions, vibrations have been noticed, and these are usually attributed to periodic movement of the separation point of the flow from the surface of the drum. Consequently, drum gates are frequently fitted with a sector or shield as illustrated in Figure 14(b). This serves the purpose of fixing the separation point at the tip of the sector. The effort required to roll the cylinder up and down the rack is clearly much less than that required to slide a simple gate of the same dimensions.

In the experimental arrangements provided, the drum gate may be set to a desired opening, and the discharge coefficient established.

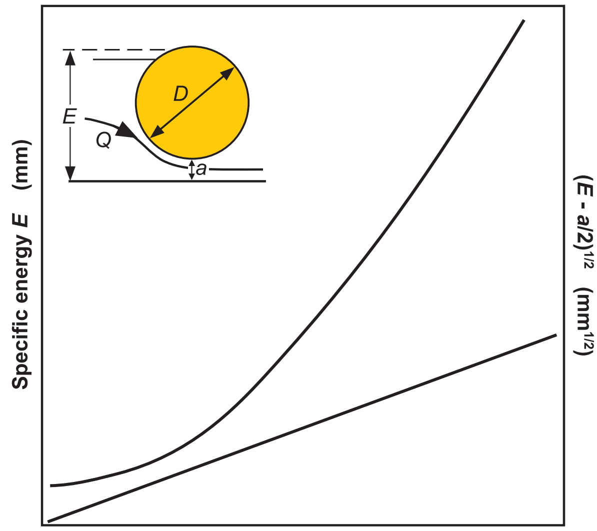


Figure 15 Discharge characteristics of a drum gate

A typical set of results is presented in Figure 15. Following the treatment of the sluice gate in Section 5, the discharge Q may be written in terms of gate opening a, channel width B, and total head of the upstream flow measured above the centre height of the gate opening, $(E - a/2)$ as:

$$Q = C\,a\,B\sqrt{2g\left(E - \frac{a}{2}\right)}$$

Questions for Discussion

1. How would you expect C to vary with change of the opening a between the drum and the channel floor? Check your expectations by experiment.
2. Sketch the distribution of hydrostatic pressure on the upstream surface of the drum when the gate is shut. Show, that in addition to the horizontal component of hydrostatic force experienced by the sluice gate, there is a vertical component acting in the upwards direction.