Flow Through Open Channels

RAJESH SRIVASTAVA
Professor
Department of Civil Engineering
Indian Institute of Technology (IIT) Kanpur

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1 Introduction

1.1 Definition of Open Channel Flow

As the name implies, the term *open channel flow* represents flows through channels that are open to the atmosphere. One should note, however, that flow in a closed conduit (e.g., a circular pipe) may also be classified as open channel flow, if the fluid level falls below the crown of the pipe and atmospheric pressure exists on the surface. Existence of the free surface, thus, is what distinguishes the open channel flows from the closed conduit flows (or pressure flows). Therefore, free surface flows would probably be a more appropriate term for open channel flows. We will, however, use the more common nomenclature and simply use the term open channel flows. For flow through a culvert[1] draining water across and under a road (or for a storm-water drain), the flow may be an open channel flow for small discharges and may become a closed conduit flow for larger discharges. Sometimes, part of the channel may have a closed conduit flow and part may behave as an open channel (e.g., when the headwater level is above the top of the pipe and the tailwater level is below the crown). Figure 1.1 shows the flow through a culvert for different headwater and tailwater elevations, which give rise to different flow conditions.

Examples of open channel flow include flow in rivers, canals, laboratory flumes, storm-water drains, flow over weirs[2] and spillways,[3] and overland runoff (Fig. 1.2). The presence of a free surface makes the open channel flow more complicated to analyse than closed conduit flow since the cross-sectional area depends on the flow depth. Further complications arise due to the fact

[1]A culvert is a conduit or channel crossing under a road or embankment.

[2]A weir is a small dam-like obstruction in a stream or river to raise the water level or divert its flow. It is generally used for discharge estimation by measuring the height of water over the weir.

[3]A spillway is a passage for surplus water to run over or around an obstruction (generally a dam).

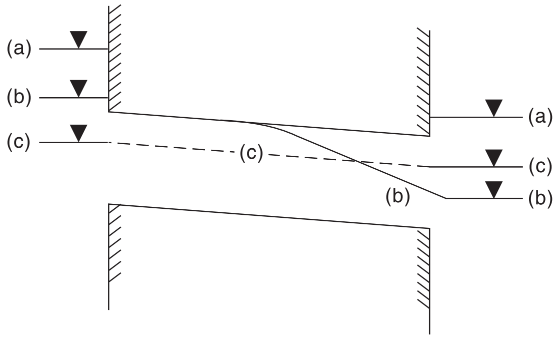


Fig. 1.1 Flow through a culvert: (a) closed conduit or pressure flow, (b) partly closed and partly open flow, and (c) open channel or free surface flow

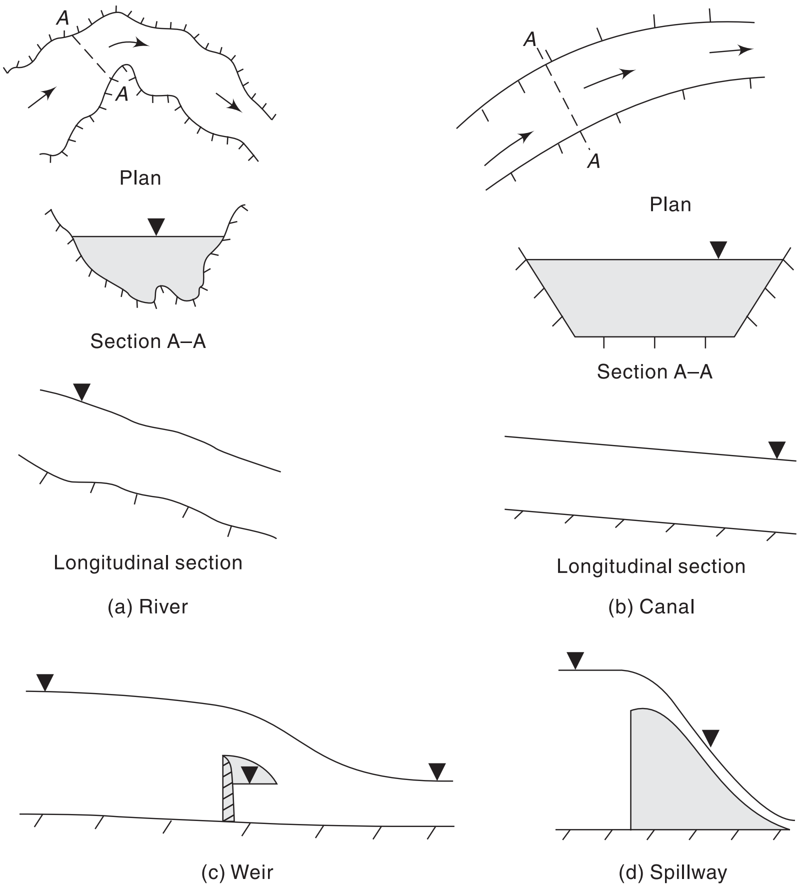


Fig. 1.2 Some examples of open channel flow

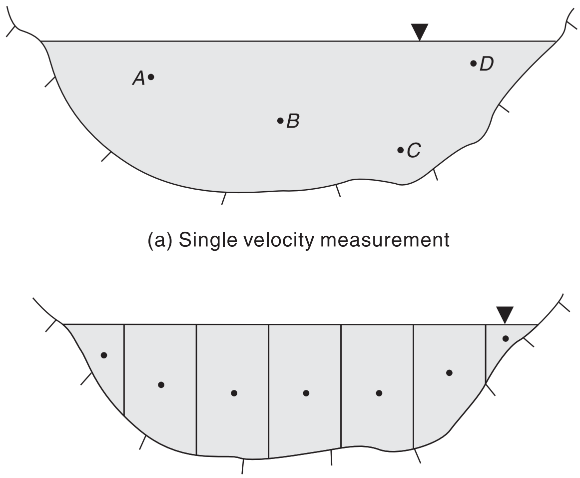
that the range of variation of boundary roughness, section shape, flow depth, and discharge is much wider for open channel flows.

1.2 Importance of the Study of Open Channel Flows

The study of open channel flows finds numerous applications in civil engineering (and also in some other branches of engineering, e.g., chemical and mechanical). Some of the applications of the principles of open channel flow are discussed in this section.

1.2.1 Measuring the Discharge in a River or Canal

The amount of water carried by a channel per unit time is a very important parameter in various projects. For example, a flood control project would need an accurate estimate of the maximum flow expected in a river. Similarly, an estimate of the discharge in a canal would help to plan the irrigation water allocation for various outlets. One way of measuring the discharge is to measure the flow velocity and multiply it by the flow area. However, as we would see later, since the velocity at a cross section varies from one point to the other, we have to decide the location of measurement (Fig. 1.3). For example, in Fig. 1.3(a), point B appears to be the most logical choice for measuring the velocity due to its central location. Obviously a single velocity measurement may result in large error and it would be preferable to subdivide the channel



(a) Single velocity measurement

(b) Division into subsections

Fig. 1.3 Discharge measurement in open channels area into different subareas and perform velocity measurements in each [Fig. 1.3(b)].

1.2.2 Developing a Relationship between the Depth of Flow and the Discharge in a Channel

Since the depth of flow is much easier to measure than the velocity, it would be much more efficient to develop a relationship between the water depth[4] and the discharge. We can then measure the flow depth at any time and obtain a quick estimate of the discharge at that time. When a large reach of channel has essentially *uniform* flow, the flow depth can be related to the discharge by considering a balance of the driving and resisting forces (Fig. 1.4). This would, however, require an estimate of the channel resistance, which may not be known with acceptable accuracy. Other techniques of estimating discharge with the help of measurement of depth, without a need to know the channel resistance, include the flow under a sluice gate, over a weir, through a flume, at a drop, etc. (Fig. 1.5). Estimation of the discharge from the measured depths in these cases requires a proper application of the conservation of mass and the energy and momentum principles.

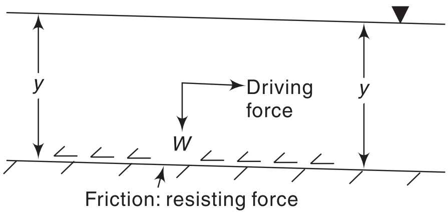


Fig. 1.4 Uniform flow in open channels

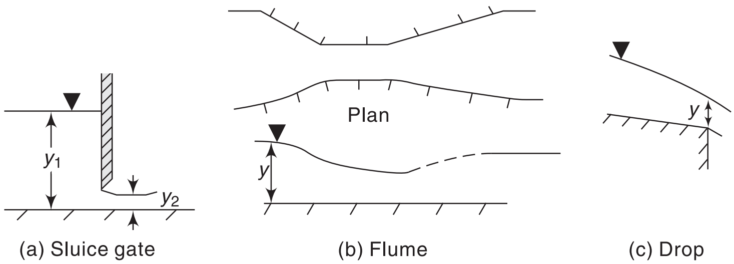
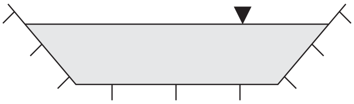


Fig. 1.5 Discharge measurement using flow depths

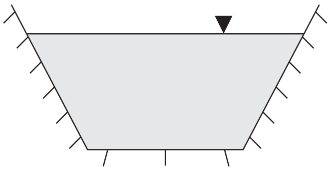
[4]The water depth also varies across a section. Typically the water depth represents the depth above the deepest point of the section.

1.2.3 Designing a Canal to Carry Given Amount of Water

Design of a canal is largely dependent on the discharge to be carried by it. For example, an irrigation canal has to be designed keeping in mind the water requirements at different locations along its length, which, in turn, depends on the crop pattern and their water requirements. There are aspects of design such as the shape and width of cross section, the bed level and the alignment that have to be decided by the engineer. While the bed level and longitudinal alignment are generally governed by the topography, there could be an infinite number of combinations of channel shape and dimensions which would be able to carry the *design discharge*. One should be able to choose the cross section which would be the optimum[5] under given conditions (Fig. 1.6).



(a) Wide and shallow



(b) Narrow and deep

Fig. 1.6 Various designs for the same discharge

1.2.4 Estimating the Area of Submergence due to Construction of a Dam on a River

Any obstruction in a channel affects the flow conditions and results in a change in the flow depth and velocity. Some of these effects may not be critical (for example, a bridge on a river changes the overall flow depth and velocity only marginally)[6] but some other may cause a considerable change. A large dam on a river (or even a small weir across a canal) would cause the water to back-up behind it (Fig. 1.7). Analysis of this phenomenon, known as the backwater effect, has an important bearing on estimating how much area would be submerged under the reservoir created by the dam. Since the backwater effect continues for a long distance (sometimes a few kilometres) upstream of the dam, one should have the tools to compute the variation of flow depth along the river upstream of the dam, in order to be able to estimate the submergence.

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[5]The optimum, of course, is generally based on economic considerations, e.g., the cost of excavation, filling, lining, etc. However, sometimes social and/or political factors may have to be considered in designing an optimum channel. We assume that optimum refers to the minimum cost.

[6]Near the bridge piers, however, there may be significant change which may adversely affect the foundation. We will not discuss these in this book.

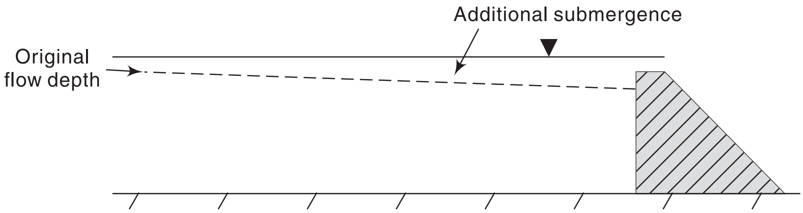


Fig. 1.7 Backwater effect of a dam

1.2.5 Preventing Very High Velocity Flows from Damaging the Channel

Water falling from a high elevation (or flowing through a very steep channel) may have a very large velocity. If left unchecked, this may cause damage to the channel bed. Energy dissipating structures could be used to ensure that the channel bed is not subjected to this high velocity flow. However, there is a less expensive way of dissipating the energy which creates a slow moving pool of water downstream of the high velocity jet (Fig. 1.8). For adequate energy dissipation, one must be able to analyse this flow situation and decide about the water depth in the slow moving portion as well as the length of floor which would still be subjected to the high velocity jet.

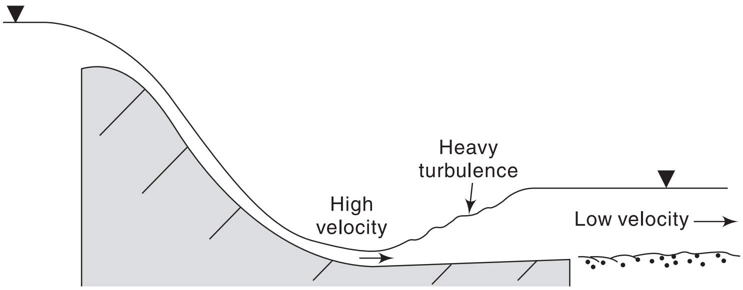


Fig. 1.8 Energy dissipation using a slow moving pool

1.2.6 Estimating the Change in the Flow Conditions due to a Change in the Bed Width or Bed Elevation

A variety of flow situations involve a change in the width of the channel or its bed level. For example, carrying a canal over a river through an aqueduct[7] may be made less expensive if the canal width is reduced (Fig. 1.9). When a

[7]An aqueduct is a bridge-like structure supporting a conduit or canal passing over a river or low ground.

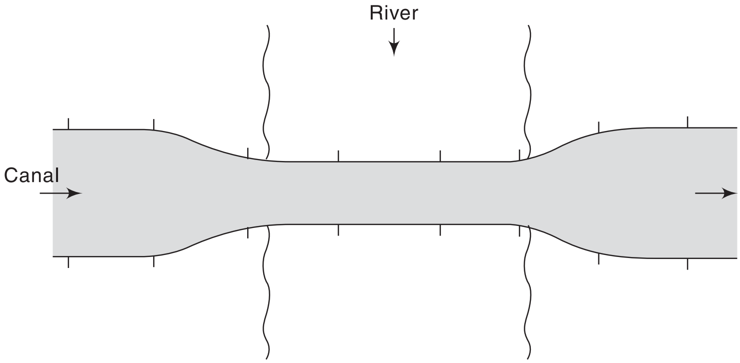


Fig. 1.9 Reduction in width of a canal at an aqueduct

channel having a flow with a uniform velocity and depth encounters such transitions, the flow characteristics undergo a change and we would require to estimate this change in order to design such structures.

1.2.7 Estimating the Change in the Flow Depth due to the Overland Runoff

During periods of intense rain, and for some time after the rain has stopped, the rainwater would flow over the land surface towards the river (Fig 1.10). This would result in an increase in discharge of the river and make the analysis of the flow more complicated due to this spatial variation. Similarly, at low discharges, the evaporation loss may be a significant fraction of the discharge and has to be accounted for. Sometimes, a weir is constructed in the sidewall of a channel to divert water when the water level rises above a fixed level, i.e., the level of the weir crest. Here also we will have the discharge in the channel decreasing in the downstream direction. In order to find the flow depth and velocity (and the diverted discharge), we should have the ability to analyse the flows with varying discharge.

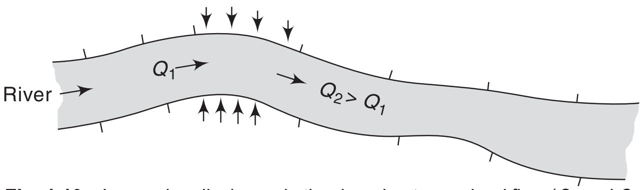


Fig. 1.10 Increasing discharge in the river due to overland flow (Q_1 and Q_2 are discharges at the locations shown.)

1.2.8 Estimating the Time Taken by a Flood Wave to Pass through a Given Length of a River

During heavy floods, there is a significant variation in the discharge with space as well as time. A flood wave entering a given reach of the river gets modified as it travels down the channel due to the increase in water level and the consequent storage of water in the channel. As a result, generally the peak reduces (unless there are tributaries joining the channel or significant overland flow takes place) and the time base increases (because the stored water is released over a long period of time). Figure 1.11 illustrates a flood wave passing through a river. It is important for the design of structures built on the river to know the approximate flood level and discharge at different locations and it requires the capability of incorporating both spatial and temporal variations in flow characteristics.

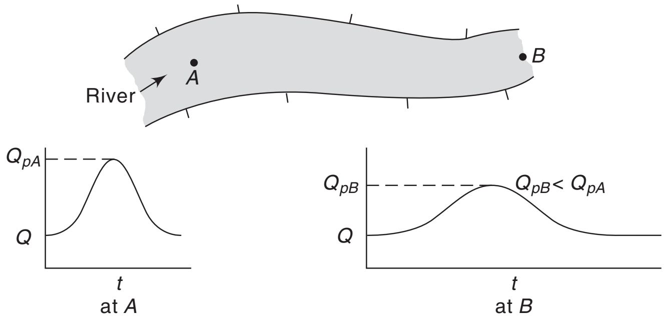


Fig. 1.11 A flood wave passing down a river (Q_p denotes the peak discharge.)

1.2.9 Estimating the Amount of Sediment Carried by a Channel

Most human-made channels have erodible boundaries and the force of the flowing water may sometimes be sufficient to loosen the boundary particles and carry these with it. Even for channels with non-erodible boundaries, sediments may be carried by the overland flow to the channel. The presence of sediments causes additional resistance to flow and may lead to deposition in some reaches (Fig. 1.12) if the sediment-carrying capacity of the flow becomes less than the amount of sediment being carried (for example, if the channel bed becomes flat and the velocity reduces). In order to analyse this behaviour, one should be conversant with the mechanics of sediment transport.[8]

[8]However, as we would stress again a little later, the 'mechanics of sediment transport' is itself a separate subject. We will not cover it extensively in this book but will provide enough background for the reader to be able to answer the basic questions related to the sediment-carrying capacity of a channel.

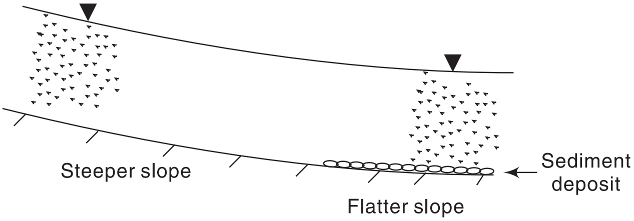


Fig. 1.12 Deposition of sediments in a channel

1.2.10 Studying the Spread of Pollutants in a River

Increasing pollution of rivers by municipal and industrial waste (especially in India, and not much in most of the developed countries) has led to increased awareness of the problem and significant efforts to reverse the damage already done. A number of industries situated near rivers and discharging their waste (with little or no treatment) into them have been closed or shifted to other places to reduce the amount of pollutants. However, the rivers are still heavily polluted. Any effort on improving the riverwater quality would need a thorough understanding of the process of advection and dispersion[9] of pollutants (Fig. 1.13).[10]

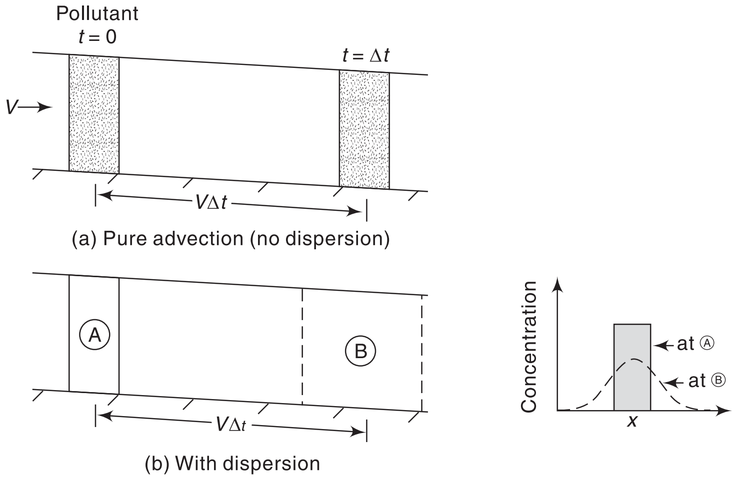


Fig. 1.13 Transport of pollutants through an open channel (V is the velocity of flow, t is the time, and x the distance.)

[9] *Advection* refers to the transport of pollutants due to the bulk movement of water and *dispersion* refers to the spreading of the pollutants about their 'mean location' due to concentration gradient or velocity variations. Detailed description is given in Chapter 9.

[10] Again, this topic is too vast to be covered here and is more related to environmental engineering than open channel flow. However, the brief treatment given in this book will enable one to understand some elementary concepts of the process.

This book would expose the reader to all the applications of the open channel flow discussed in this section and many more similar applications. Even if the reader does not find the solution to a problem, the basic principles provided throughout the text would help him or her in finding the solution.

1.3 Overview

A description of any subject is based on some assumptions about the background of the reader. For this book, we assume that the reader is familiar with the basic concepts of fluid mechanics, including fluid properties, forces in stationary and moving fluids, control volume analysis (Reynold's transport theorem), continuity, momentum, and energy equations for closed conduit flows, laminar and turbulent flows in circular pipes, and boundary layer theory. In addition, basic knowledge of mathematical and numerical methods, including differential calculus and solution of differential equations, is assumed. To aid the reader in recalling some of these prerequisites, Appendices A, B, and C list the important formulae and concepts (without detailed description) needed during the study of this book. A reference to these Appendices has been made at appropriate places in the text. The following example illustrates how the reader can take help of the Appendices.

Example 1.1 A culvert in the form of a 5 m long circular pipe (0.3 m internal diameter) carries water from one side of a road embankment to the other. The invert level and crown elevation as shown in Fig. 1.14 are 100.00 m and 100.35 m respectively. The pipe is made of concrete (mean surface roughness height 0.6 mm). What would be the discharge when the headwater and tailwater elevations are respectively (a) 100.60 m and 100.30 m, (b) 100.35 m and 100.30 m, and (c) 100.20 m and 100.15 m? Ignore all losses other than the friction loss.

Solution

Since only the friction loss is to be considered, the difference between the headwater and tailwater levels should be equal to the head loss due to the friction.

For case (a), the culvert behaves as a closed conduit and Darcy–Weisbach equation (*Appendix A*) is used to write

$$h_f = f\frac{L}{D}\frac{V^2}{2g}$$

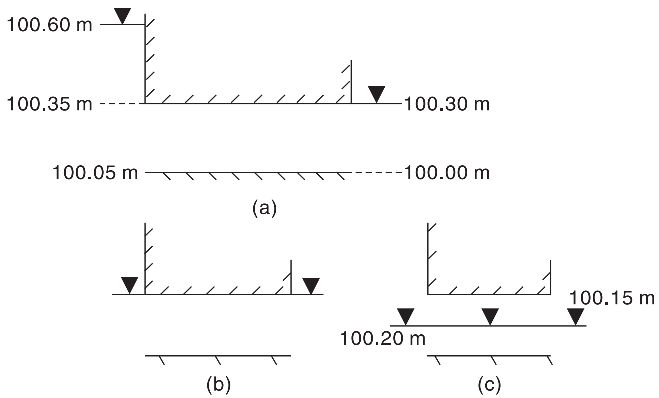


Fig. 1.14 Culvert working as a closed conduit and an open channel

$$\Rightarrow \quad 100.60 - 100.30 = f\frac{5}{0.3}\frac{V^2}{2\times 9.81}$$

$$\Rightarrow \quad fV^2 = 0.353 \tag{1.1}$$

The velocity of the flow may be obtained through iterations using the Moody chart (Fig. A.1) or Churchill equation (A.21) or the discharge may be obtained directly using Eq. (A.22). We demonstrate the iterative method using the Moody chart here.

Since $k_s/D = 0.002$, we start with the rough pipe value of $f = 0.023$ from the Moody chart and, using Eq. (1.1), obtain the velocity as 3.92 m/s. The corresponding Reynolds number, Re, is computed as 1.18×10^6 and the friction factor is read from the chart as 0.0235. The next iteration results in a velocity of 3.88 m/s, which is very close to the value at the previous iteration and no further iterations are required. The discharge is therefore, **0.274 m^3/s**.

For case (b), the culvert may be considered as either closed conduit or open channel since both the headwater and the tailwater levels are *just touching* the respective crown levels. However, since we do not yet know how to find the discharge for an open channel, we again use the pipe-flow equation with the head loss equal to 0.05 m, and obtain $fV^2 = 0.0589$. Starting with $f = 0.023$, we get $V = 1.60$ m/s and Re $= 4.8 \times 10^5$. From the Moody chart, $f = 0.025$, which results in $V = 1.53$ m/s. Further iterations are not needed and the discharge is **0.108 m^3/s**. Note that Eq. (A.22) would directly (without any need of iterations) provide the discharge as 0.274 m^3/s in case (a) and 0.111 m^3/s in case (b).

Case (c) involves flow with a free surface condition since both the headwater and tailwater levels are below the corresponding crown elevations. At this stage, we are not able to estimate the discharge. By the time the reader finishes the next chapter, however, he or she would be able to solve problems like this.

Now that we have seen our limitations in solving open channel flow problems, we would discuss how to study these flows. We first classify open channel flows on the basis of channel characteristics and flow properties, and then take up the study of these different types of flow one by one with increasing degree of complexity, starting with Chapter 2 with the simplest case where the flow depth, velocity, channel cross section, etc. are not changing with time and location (i.e., uniform flow). Unlike most other books on the subject, we will not describe the basic equations of continuity, momentum, and energy in the initial chapters. We assume that the reader is already familiar with the concepts of these equations in other fields. However, adaptation of these concepts to open channel flow have been introduced as and when needed. For example, the study of uniform flow does not require the direct application of the general momentum or energy equation. Therefore, these equations have not been presented in Chapter 2. The energy equation and the concept of specific energy have been introduced only when they are needed in the chapter on Nonuniform Flow. Also, since a very good book dealing with the historical development of the field of hydraulic engineering is available (Rouse and Ince 1980), we have limited the discussion of historical facts to a minimum.[11] References have not been provided for very old or hard-to-find articles. In such cases, the author's name and the year have been mentioned to put things in the proper historical perspective.

1.4 Classification of Open Channel Flows

Classifying the open channel flow in different categories helps us in determining which parameters are significant and which can be safely ignored while analysing the flow. For example, in flow through closed conduits, we know that if the flow is laminar, the boundary roughness does not influence the frictional losses. Similarly, for very high Reynolds number, the friction factor is not affected by the Reynolds number and is only dependent on the relative roughness of the boundary. Therefore, for free surface flows also, we

[11]In fact, at some places we have taken the liberty of ignoring the chronological order of events if it suits our main aim of explaining things in a logical sequence.

would expect some parameters to be important in some flow situations and insignificant in some other situations. We classify the open channel flows based on channel characteristics and flow properties; look at some examples and see how this classification affects the analysis of flow.

1.4.1 Classification Based on Channel Characteristics

Channels can be classified as prismatic and non-prismatic channels, natural and artificial channels, and rigid boundary and mobile boundary channels. Accordingly, the flows through channels can also be classified.

Prismatic and nonprismatic channels

A channel is said to be *prismatic* if it is in the form of a prism, i.e., the cross section and the bed slope do not change along the channel length. If there is a change in cross section and/or slope, the channel is *nonprismatic*. A laboratory flume laid at a constant bed slope and with a uniform cross section is a prismatic channel while a river with varying cross-section and bed slope is a nonprismatic channel (Fig. 1.15). Obviously it is much simpler to analyse the flow in a prismatic channel since the invariance of cross section and bed slope implies that the flow depth, velocity, etc. will be same at every section for a given discharge under normal flow conditions. However, for a nonprismatic channel, flow characteristics may change along the length making the analysis more difficult. Also, the variation in cross section leads to additional losses due to expansion and contraction, which should be accounted for in the analysis. In this book, we will assume that the channel is prismatic, unless specified otherwise (e.g., in Section 3.6.3).

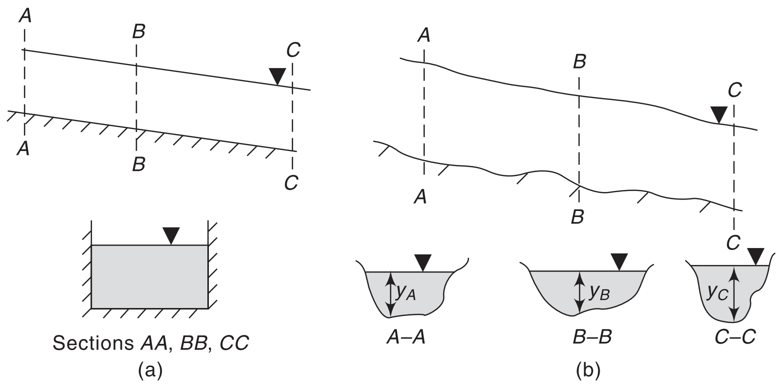


Fig. 1.15 Prismatic and nonprismatic channels

Natural and artificial channels

A river, an estuary,[12] and a land surface during overland runoff, are all examples of natural channels while a laboratory flume, a canal, and a parking lot during overland runoff, represent artificial (or human-made) channels. Generally, natural channels would be nonprismatic while the artificial channels are likely to be prismatic. Also, natural channels typically have an irregular cross section [Fig. 1.15(b)] while artificial channels have regular (e.g., circular, trapezoidal, rectangular) cross sections [Fig. 1.15(a)]. This may lead to ambiguity in defining the flow depth for natural channels. Conventionally, the flow depth is measured as the height of water surface above the deepest point in the cross section.

Rigid boundary and mobile boundary channels

If the material on the bed and sides of a channel is loose and easily movable due to the flow of water,[13] the channel is called a *mobile boundary channel*. Conversely, if the material is not easily movable (e.g., a metal flume, concrete lined canal), the channel is a *rigid boundary channel*. Clearly, analysis of flow through a mobile boundary channel is more complicated than that of flow through a rigid boundary channel, due to the process of sediment erosion and deposition and the resulting additional resistance to flow. These processes may also occur in rigid boundary channels due to sediment inflow from elsewhere (e.g., from the river into a lined canal or from upstream areas to a parking lot). Sediment transport in channels is a subject in its own right and there are various textbooks available on this topic (e.g., Garde and Ranga Raju 2000). In most of this book, therefore, we will consider the channels to be rigid boundary channels carrying no sediments. A brief description of flow through mobile boundary channels is provided in Chapter 8.

1.4.2 Classification Based on Flow Properties

Various flow properties and the classification of flows based on these properties are discussed in this subsection.

Temporal variation—steady and unsteady flows

A flow is said to be *steady* if the flow characteristics (e.g., discharge, depth, velocity) do not change with time [Fig. 1.16(a)]. If a change is observed with

[12] An *estuary* is an arm of the sea at the lower end of a river, i.e., where the river joins the sea. The tide meets the river current in the estuary.

[13] Since water is the most common fluid flowing in an open channel, we will assume that the flowing fluid is water, unless otherwise mentioned.

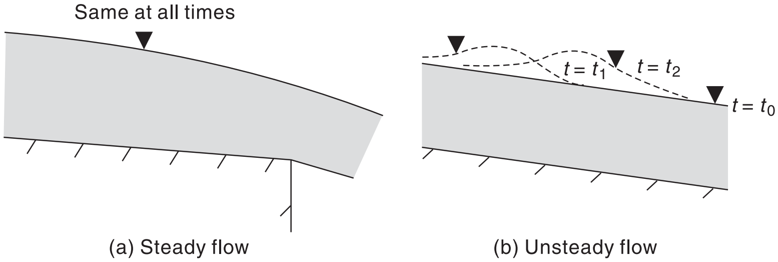


Fig. 1.16 Steady and unsteady flows

time, the flow is *unsteady* [Fig. 1.16(b)]. In general, it is almost impossible to have a strictly steady flow. No matter how good a control exists on the flow conditions, there is bound to be some change with time. Since the analysis of the steady flow is simpler than that of the unsteady flow (due to the absence of temporal derivatives), one may decide to treat a given flow as steady under certain conditions (e.g., discharge/depth variation within, say, 1% of a mean value). Sometimes, it is possible to change the frame of reference and convert an unsteady flow situation into an equivalent steady flow (e.g., in a tidal wave, by moving with the wave).

Spatial variation—one-dimensional, two-dimensional, and three-dimensional flows

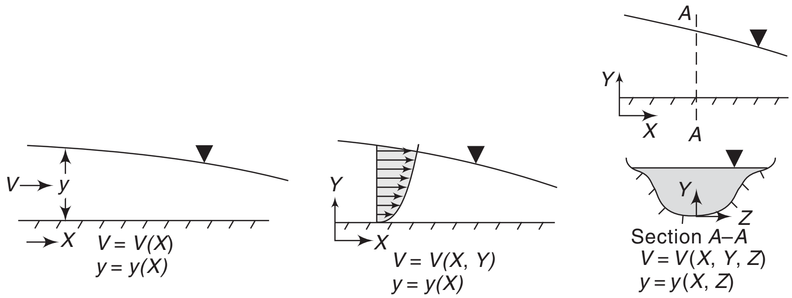
Any point in space may be conveniently described through a three-dimensional orthogonal coordinate system. The particular system to be used for a problem depends on the relevant geometry. For example, flow through a circular pipe is best studied in a cylindrical coordinate system. In open channel flows, we would commonly use the Cartesian coordinates (X, Y, Z),[14] with X-axis along the channel bed in the direction of flow (longitudinal distance), Y orthogonal to X upwards (channel depth),[15] and Z orthogonal to both X and Y (channel width) following the right hand rule[16] (see Fig. 2.1). While analysing a

[14]Generally, lowercase letters are used to represent the coordinate axes. We use uppercase letters in this book to distinguish the axes from the flow depth (y) and the bed elevation (z).

[15]Note that the depth is not measured vertically but perpendicular to the bed. Generally, the bed slope is very small and the vertical depth and perpendicular depth are almost identical. But for steep channels, such as flow over a spillway, these two depths will differ significantly from each other.

[16]If the thumb, index finger, and middle finger of the right hand are held in a mutually orthogonal position, such that the thumb indicates the X-axis and the index finger the Y-axis, then the middle finger indicates the Z-axis. Thus, if X points in the downstream direction, and Y upwards, Z will point towards right when looking at a cross-section from the upstream side.

particular reach of the channel, the origin of the coordinate system is preferably placed at the deepest point of the most upstream section. In general, the flow characteristics would be functions of all three ordinates, e.g., the velocity will vary along the channel length and also across a cross section. However, to simplify the analysis, we may approximate the actual (three-dimensional) velocity distribution by a simpler profile. For example, if the channel is very wide, we may consider the flow to be independent of Z-location (two-dimensional in the X-Y plane). This assumption would be valid for most of the cross section except very near the sides, where velocity would depend on Z. Similarly, the Y-dependence of velocity may be removed by averaging the velocity across the depth (two-dimensional, X-Z plane). The most commonly used technique for reducing the dimensionality of a problem is the averaging of velocity over the entire cross section. This average velocity is then only a function of X (one-dimensional or 1-D) and the governing equations become simpler[17] without loosing too much of the detail. (In most cases our primary interest is in knowing the variation of flow characteristics along the channel length and not across a section.) In this book, we will adopt the 1-D simplification by using the cross section average velocity [Fig. 1.17(a)]. A brief discussion of the velocity variation in a cross section is provided, however, in Chapter 3 to make the reader aware of the degree of approximation introduced by the 1-D assumption.



(a) One-dimensional (b) Two-dimensional (c) Three-dimensional

Fig. 1.17 One-, two-, and three-dimensional flows

Spatial variation—uniform and nonuniform flows

Uniform flow indicates that the flow depth, velocity, and discharge do not change in the longitudinal direction. As mentioned for steady flow, it would

[17]The averaging process introduces some complications during computation of momentum and energy of flow. These are accounted for by using the so-called *correction factors*, which will be discussed at appropriate places.

be rare to achieve a strictly uniform flow but a number of flow situations may be idealized as uniform flow. For example, flow in a long prismatic laboratory flume may show minor variations in depth, velocity, and discharge with space and time. However, for all practical purposes, it may be analysed as steady uniform flow. Clearly, assumption of uniformity leads to a drastic simplification of analysis since we need to obtain the flow characteristics at a single section only as all characteristics remain same at all sections.

If the flow is not uniform, i.e., the flow depth, velocity, or discharge is varying spatially, it is known as *nonuniform* (*or varied*) *flow*. Flow over a weir (depth decreases as we approach the weir) and flow upstream of a dam (depth increases as we approach the dam) are some examples of nonuniform flows (Fig. 1.18). Looking at the temporal and spatial variability together, it should be obvious that unsteady uniform flow is practically non-existent since it would require the flow conditions to be same throughout the channel at some instant and then change uniformly over the entire channel length at the next instant. Therefore, to simplify the classification, the term *unsteady* flow is used to indicate *unsteady nonuniform* flow. Similarly, *uniform* flow implies *steady uniform* flow (since unsteady uniform flow is non-existent) and *nonuniform* flow implies *steady nonuniform* flow (since the unsteady nonuniform flow has already been classified as unsteady flow).

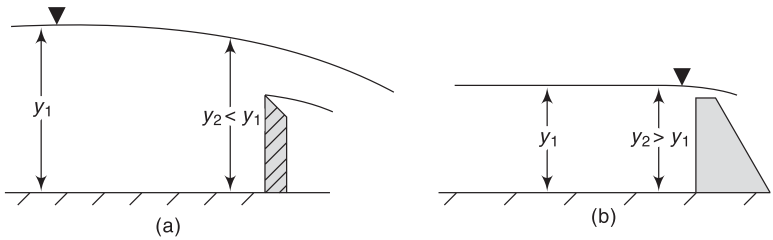


Fig. 1.18 Uniform and nonuniform flows

Nonuniform flow can be further classified into gradually varied flow (GVF)[18] when the depth changes gradually, rapidly varied flow (RVF) when the depth changes significantly over a short distance, and spatially varied flow (SVF)[19] when the discharge changes due to lateral inflow or outflow.

[18]The flow is classified as gradually varied if the streamline curvature is so small that pressure distribution at a section may be assumed to be hydrostatic.

[19]For most flow situations, we assume that the discharge is constant along the channel length. Therefore, spatially varied flow could be a classification by itself. Here we include it in nonuniform flow but devote a separate chapter to it (Chapter 6) in keeping with the generally followed practice.

The flow upstream of a dam may be treated as gradually varied [Fig. 1.18(b)], flow over a weir is rapidly varied [Fig. 1.18(a)], and flow in a side channel spillway[20] and a side weir is spatially varied (Fig. 1.19). The computation of water depth (and velocity) at different locations in a GVF is simpler than that in a RVF since the pressure distribution in GVF is assumed to be hydrostatic and is, therefore, a function of the flow depth. For a RVF, the pressure distribution is not hydrostatic and generally experimental observations are utilized to help in the computations. The SVF is further complicated by the variation of discharge along the length.

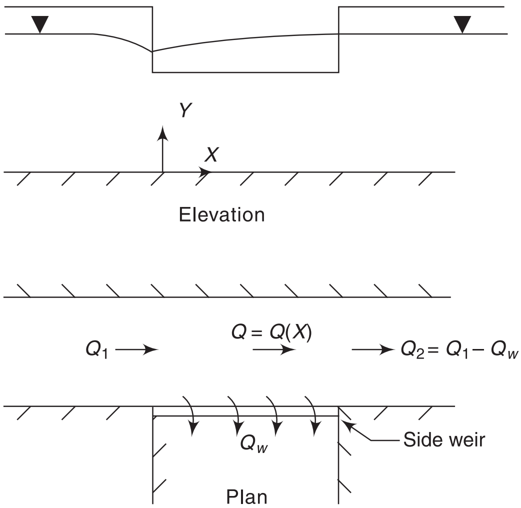


Fig. 1.19 Spatially varied flow at a side weir

Effect of viscosity—laminar and turbulent flows

Similar to the flow through a closed conduit, the flow through an open channel may be classified according to the relative magnitude of inertial and viscous forces. For a circular pipe, the state of flow (laminar or turbulent) depends on the *Reynolds number*

$$\mathrm{Re} = \frac{\rho V L}{\mu} \tag{1.2}$$

where ρ is the mass density, V is the cross section average velocity, L is a characteristic length, and μ is the dynamic viscosity. Generally, the pipe

[20]A side channel spillway has its crest parallel to the channel downstream of the spillway. The discharge in the channel increases along its length as additional water spills over the crest.

diameter is used as the characteristic length, and the critical Reynolds number (below which the flow is laminar) is about 2,300.[21] Since open channel flows occur in channels with various cross-sectional shapes, we need to define another characteristic length. For non-circular conduits, the *hydraulic radius*, R,[22] defined as the area of cross section divided by the perimeter, is used as the characteristic length. Considering that the hydraulic radius of a circular pipe is equal to one-fourth of the diameter, the critical Reynolds number, with hydraulic radius as the characteristic length, is about 2,300/4, i.e., 575. For open channels, the hydraulic radius is defined as the area of flow section divided by the wetted perimeter. For a very wide channel, the hydraulic radius may be replaced by the flow depth to obtain a rough idea about the Reynolds number. Since water is the fluid in most open channel flows,[23] we use ρ =1,000 kg/m^3 and μ = 0.001 N s/m^2. For a typical velocity value of 1 m/s and flow depth of 1 m, we get Re = 1,000,000. Measurements suggest that the critical value of Re for open channel flows is also about 500. Therefore, the flow through open channels for all practical cases would be turbulent.[24] If we look at very slow flows with velocity of the order of a few centimetres per second, the flow depth would have to be of the order of a few centimetres to sustain laminar flow. Such conditions may exist only during sheet flow of overland runoff. Hence, we will not consider laminar flow in this book.

Effect of gravity—subcritical and supercritical flows

Gravity is the driving force behind flows through open channels. Hence, it stands to reason that the ratio of inertial to gravitational forces will play a major role in open channel flow analysis. Using dimensional analysis with inertial force $\rho V^2 L^2$ and gravitational force $\rho L^3 g$, where V is the average velocity, L is a characteristic length, and g is the gravitational acceleration, we obtain the ratio of these forces as V^2/gL. Following the convention of

[21]Under carefully controlled conditions, laminar flow may be obtained for a Reynolds number as high as 40,000! The *lower* critical Reynolds number has been reported variously in the range of 2000 to 2300.

[22]Some books use the notation R_h for the hydraulic radius to distinguish it from the pipe radius. However, in this book, we would use pipe diameter, D_0, and not its radius. Therefore, we use R for hydraulic radius.

[23]Sometimes, we will have seawater as the flowing fluid (estuaries). In laboratory experiments, we may use some other fluid to study, say, the effects of surface tension and viscosity. Throughout this book, we assume that the fluid is water unless mentioned otherwise.

[24]In fact, the Reynolds number, Re, is generally so high that the *friction factor* becomes independent of Re (fully rough zone of flow).

using the first power of velocity, we define a dimensionless number (the *Froude number*) as

$$F_r = \frac{V}{\sqrt{gL}} \tag{1.3}$$

to represent the relative importance of inertial and gravitational forces. It would be shown in Chapter 3 that the characteristic length used in the Froude number is the *hydraulic depth*, D, defined as the ratio of area of flow to the top width of the flow section. Further, it would be shown that

(i) the energy of flow at any section for a given discharge is minimum when $F_r = 1$ (Chapter 3),
(ii) the sum of pressure force and momentum of flow at any section for a given discharge is minimum when $F_r = 1$ (Chapter 4), and
(iii) a small amplitude disturbance created at the free surface in stationary water moves with a velocity of $\sqrt{gD}$ (Chapter 7) implying that the flow velocity is equal to wave velocity when $F_r = 1$.

Thus the condition $F_r = 1$ is quite critical in applying the energy and momentum equations (and also in analysing the spreading of a disturbance) and is known as *critical flow*. Clearly, $F_r > 1$ denotes a velocity larger than the critical velocity (and depth smaller than the critical depth) and is termed *supercritical*, rapid, or shooting flow, and $F_r < 1$ denotes a velocity smaller than the critical velocity and is termed *subcritical*, streaming, or tranquil flow. As F_r becomes larger than (or smaller than) 1, the energy keeps on increasing from its minimum value at $F_r = 1$. For a supercritical flow, therefore,

(i) the energy will increase with a decrease in depth,
(ii) the sum of pressure force and momentum will increase with a decrease in depth, and
(iii) a small disturbance on the surface will not be able to travel upstream[25] since $V > \sqrt{gD}$.

Conversely, for a subcritical flow,

(i) the energy will increase with an increase in depth,
(ii) the sum of pressure force and momentum will increase with an increase in depth, and
(iii) a small disturbance on the surface will be able to travel upstream.

[25]This provides us a rough, but quick, method of estimating whether a flow is supercritical or subcritical. If we drop a stone in a channel and the resulting wave moves both upstream and downstream (of course, the upstream movement will be much slower than the downstream one), the flow would be subcritical.

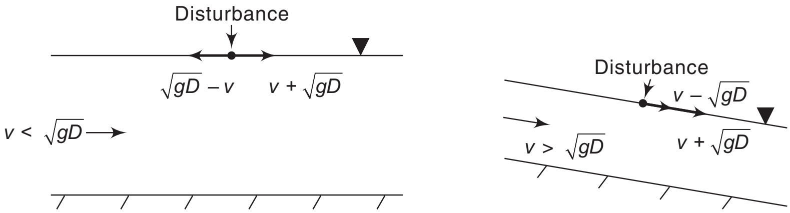


Fig. 1.20 Subcritical and supercritical flows

Most flows in nature are subcritical but the flow over a spillway, a steep laboratory flume, or downstream of a sluice gate, would be supercritical (Fig. 1.20). As we will see in later chapters, the contrasting nature of variation of energy and momentum for subcritical and supercritical flows would be important while analysing gradually varied flow profiles, hydraulic jump,[26] and channel transitions involving change of width and/or bed level.

REFERENCES

Garde, R.J. and Ranga Raju, K.G. (2000). *Mechanics of Sediment Transportation and Alluvial Stream Problems*, 3rd edn, New Age International, New Delhi.

Rouse, H. and Ince, S. (1980). *History of Hydraulics*, reprint. Iowa Institute of Hydraulic Research, University of Iowa, Iowa City.

EXERCISES

1.1 Classify the following flows as uniform, gradually varied, rapidly varied, spatially varied, or unsteady flow:

(a) Flow in a long laboratory flume
(b) Flow near the ungated end of a laboratory flume
(c) Flow upstream of a dam
(d) Flow over a weir
(e) An open drain by the side of a road during a rainfall event
(f) Progress of a tidal wave in an estuary
(g) Flow in an unlined canal considering evaporation and seepage losses

1.2 For a short laboratory flume, the effect of end conditions causes the flow to be nonuniform. What would you do to achieve a nearly uniform flow?

[26]A hydraulic jump is formed when a supercritical flow meets a subcritical flow.

1.3 An estuary carries water at a velocity of 1.5 m/s and flow depth of 2 m. A tidal wave moves upstream at a constant velocity of 2 m/s. How would you convert this into a steady state flow situation? What would be the apparent flow velocity upstream of the wave after the conversion?

[3.5 m/s]

1.4 In a sugar factory, molasses (dynamic viscosity = 7 Ns/m^2, mass density = 1500 kg/m^3) is carried away in a 2 m wide rectangular open channel at a flow depth of 1 m and volumetric flow rate of 0.5 m^3/s. Is the flow laminar or turbulent? What would be the speed of a small disturbance created on the surface? What is the Froude number? [3.13 m/s, 0.08]