Water Measurement with Flumes and Weirs

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Abstract

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This manual describes how to design, construct, and operate long-throated flumes to measure and control water flow in open channels. Broad-crested weirs with a streamlined flow contraction are members of the long-throated flume family and can be analyzed with the techniques described herein. Broad-crested weirs are particularly well adapted to irrigation canals, while long-throated flumes are well adapted to natural streams. The range of conditions over which flow can be measured with long-throated flumes and broad-crested weirs is unlimited. A large variety of examples are presented.

The manual includes a revised mathematical model for designing a flow-measuring structure in any open channel to satisfy user-specified hydraulic boundary conditions and design requirements. Upon completion of the design, the model will compute the head-versus-discharge relation and the hydraulic energy losses for the flume or weir. The computer program presented in this publication can accommodate a wide variety of structure and channel shapes as well as many different input and output units. This version greatly expands upon the capabilities of previously published programs.

Key words

Water management, broad-crested weirs, computer modeling, design, flow measurement, flumes, hydraulics, open-channel flow, discharge rating.

Preface

Each day, the continuing growth of world population places new demands on our water resources. More water is needed for all the processes of life: food production, municipal supply, industrial water use, power generation, navigation, recreation, etc. At the same time, environmental water needs are increasingly being recognized, limiting the sources of new water and further increasing the competition for available supplies. Conservation and associated water measurement are being recognized as important tools for making the best use of available water.

Improved management of our water resources is needed to ensure the equitable distribution of water to competing users. There are especially significant opportunities for conservation and more effective water use by the world's largest user: agriculture. Accurate delivery of the necessary amounts of water at the correct times can both conserve water and improve the quantity and quality of agricultural products. Thus, the water measurement and control structures described in this manual have a key role to play as we address the future water, food, and fibre needs of our world.

To improve water management, we recommend that water-measuring capability be included in all new water projects and that existing water projects be retrofitted for water measurement as soon as possible. Usually, water measurements should be planned at all points where it can be reasonably established that knowledge of the flow rate will affect management decisions. Thus, water measurements should be provided at all bifurcations or divisions of flow within a canal distribution system, at all delivery outlets, and in the stream or river from which water is diverted.

For open-channel flow measurements we recommend the modern structures described in this book, which are all members of the family of long-throated flumes. This family of devices includes broad-crested weirs with a streamlined flow contraction. Broad-crested weirs are particularly well adapted to irrigation canals, while long-throated flumes are well adapted to natural streams. The range of potential applications for the family of long-throated flumes and broad-crested weirs is unlimited, both in terms of channel types and ranges of flow rates.

The primary advantage of these flumes and weirs is the theoretical predictability of their hydraulic performance. In addition, they have the following major advantages over all other known weirs and flumes (e.g., Parshall flumes, cutthroat flumes, H-flumes, sharp-crested weirs, etc.):

- Provided that critical flow occurs in the throat, a rating table can be calculated with an error of less than 2 percent of the listed discharge. The calculation can be made for any combination of prismatic throat and arbitrarily shaped approach channel.

- The throat, perpendicular to the direction of flow, can be shaped in such a way that the complete range of discharges can be measured accurately.
- Minimal head loss over the weir or flume is required to ensure a unique relationship between the upstream sill-referenced head, h_1, and the discharge, Q.
- This head-loss requirement can be estimated with sufficient accuracy for any of these structures placed in an arbitrary channel.
- Because of their gradual converging transition, these structures have little problem with floating debris.
- Field observations and laboratory tests have shown that these structures can be designed to pass sediment transported by open channels with sub-critical flow. However, sedimentation can be a problem when sediment loads are excessive or when the flume causes a significant reduction in the approach channel flow velocity.
- Provided that the throat is horizontal in the direction of flow, a rating table can be computed using post-construction dimensions. Thus, an accurate rating table can be produced even if the flume is not constructed to the designed dimensions. The throat may also be reshaped as needed according to changing site conditions, and a new rating table can be computed using the modified dimensions.
- Under similar hydraulic and other boundary conditions, these are usually the most economical of all structures for accurately measuring open channel flows, provided that conditions are such that a weir or flume is feasible.

Because of the above advantages, these devices are useful for a variety of flow measurement applications, particularly when the structure must have a minimal impact on existing flow conditions and water surface elevations.

This publication integrates material from the 1984 book *Flow Measuring Flumes for Open Channel Systems* and the 1993 book *FLUME: Design and Calibration of Long-Throated Measuring Flumes*, which introduced the first interactive flume design software. The new Microsoft Windows version of FLUME (WinFlume) is presented in Chapter 8. This computer program can be used to develop the hydraulic design of long-throated flumes and broad-crested weirs to be constructed in user-specified channels, satisfying user-specified boundary conditions and design requirements. The program also determines the head versus discharge calibration (the rating) of newly designed structures and existing structures. WinFlume is an upgrade from the previous programs and has numerous improvements, the most notable being an improved user interface for the Windows operating systems, additional output options, and an improved design module.

Techniques for construction of these flumes and weirs have evolved significantly in recent years, and several commercially available flumes and weirs are now on the market. Chapter 3 provides information on both of these topics, including illustrations of many successful installations from around the world.

The authors wish to acknowledge the efforts of all those worldwide who have contributed to the development of long-throated flume technology. In connection with this book, special thanks are extended to Mr. Brian Wahlin of the U.S. Water Conservation Laboratory who provided a thorough review of the manuscript that led to many substantial improvements.

The range of potential applications for these flumes is unlimited. We hope that this book will contribute to the effective management of one of the earth's most widely needed, used, and wasted natural resources: water.

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1. Introduction to Water Measurement and Long-Throated Flumes

1.1 The Need for Water Measurement

World population surpassed six billion people in 1999 and continues to increase by about 2.8% per year. In 2025, the population is expected to be 8.5 billion. All of these people need water for life: food production, municipal supply, industrial water use, power generation, navigation, recreation, etc. At the same time, the environment is increasingly being recognized as a water user. With the volume of water available for various uses remaining about the same, the competition between water users will continue to increase. Already, 10% of the world population lives in countries in arid and semi-arid regions where the annually available volume of water has dropped below the critical level established by the World Bank of 1700 m^3 per capita per year (Figure 1.1). More alarming is that an additional 49% of the world population is expected to pass the water scarcity limit before 2025 (World Bank 1999).

Hence, water is an ever more important constraint. Water management improvements can promote conservation and make best use of our limited water resources, but better management depends upon the ability to accurately measure and control the flow of water at important points in a river basin or irrigation system, such as headworks, canal bifurcations, offtake structures, and drainage collection points. Good water measurement systems enable accurate accounting of water use, and permit the available water to be supplied at optimum rates to the areas where it is intended to be used. To serve this need, the structures described in this manual play a key role.

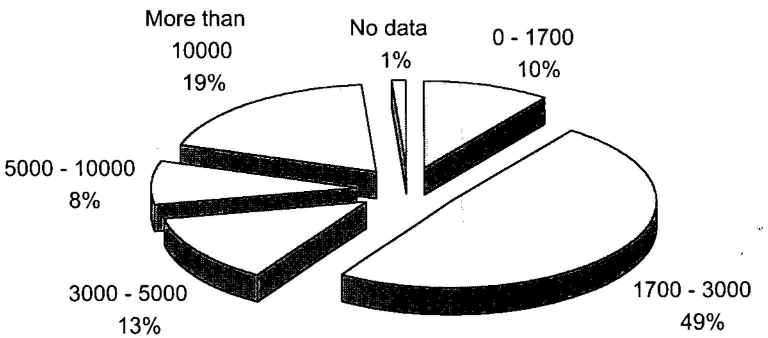


Figure 1.1 Distribution of world population among economies grouped by annual freshwater resources in cubic meters per capita (World Bank 1999).

1.2 Open-Channel Flow Measurement Using Critical-Flow Devices

The most common structures used to measure flow in open channels operate by producing *critical flow* or flow at *critical-depth* through a control section of known dimensions. Under this flow condition, the discharge through the critical section is a function of the section shape and the upstream potential energy, as indicated by the water level upstream from the structure. By definition, the presence of critical flow in the control section prevents the downstream water level and flow conditions from affecting the flow through the critical section, and the discharge can be computed as a function of the measured upstream head. Sharp-crested weirs, broad-crested weirs, and a wide variety of flumes are examples of critical-flow devices.

To apply a critical-flow device for flow measurement, one must define the particular relation between flow rate and upstream head, and the range over which it is applicable. These two issues present a significant problem for some critical-flow devices. First, the flow through the critical section of many of these devices is three-dimensional and cannot be easily analyzed with available one-dimensional hydraulic theory. These devices must be calibrated with the aid of physical models, laboratory tests, or complex three-dimensional numerical modeling; laboratory calibration tests that determine empirical discharge coefficients are the most commonly used. Second, the discharge coefficients of many critical-flow devices vary widely when operating outside of a narrow range of conditions. For example, the discharge coefficients of sharp-crested weirs change significantly if the tailwater level exceeds the crest elevation of the control section (i.e., the crest is submerged). Similarly, Parshall flumes do not maintain critical flow conditions beyond a specific submergence ratio, which varies somewhat depending on the size of the flume. Some of these devices can still be utilized for flow measurement under submerged conditions by making use of a measurement of the downstream water level and applying correction factors for submerged operation, but at the expense of additional complexity and reduced accuracy.

In this book, we focus on the application of a class of weirs and flumes whose discharge can be theoretically predicted using well-established one-dimensional hydraulic theory. These devices are also among the most tolerant of downstream submergence and are very economical and adaptable to a wide variety of situations for new construction, rehabilitation, and improvement of existing systems. As a group, we call these devices long-throated flumes (Figure 1.2), although many practitioners also use the term broad-crested weir to describe a subset of these devices. The common characteristic of these devices is a control section (or throat) whose length compared to the upstream head is sufficient to produce essentially parallel flow streamlines in the throat at the location of critical depth or critical section. To obtain this flow condition, the length of the throat in the direction of flow, L, must be greater than or equal to the total upstream sill-referenced energy head, H_1 (see section 1.4 for definitions). Specific details of this requirement and practical upper limits on the throat length are discussed in Chapter 6.

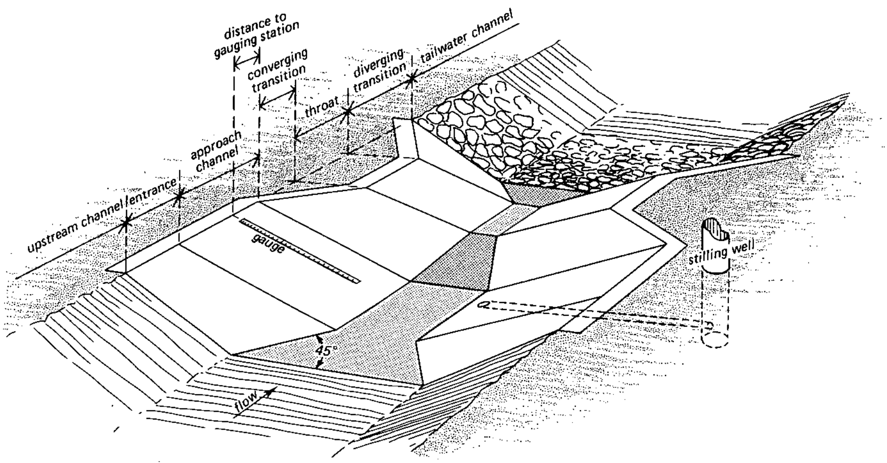


Figure 1.2 General layout of a long-throated flume.

The hydraulic performance of weirs and flumes that satisfy this requirement can be readily analyzed with available hydraulic theory, and their discharge versus head rating curves can be predicted accurately. Standard devices in convenient sizes can be selected for a specific application, or the hydraulic theory can be used to design custom structures for any application. This work is greatly facilitated by the WinFlume computer program described in Chapter 8. The theoretical predictability of these structures is also an advantage when evaluating the acceptability of construction errors and modifications; the computer model can easily be used to evaluate designs and compute rating tables using as-built dimensions.

The terms weir and flume will both be used throughout this book; the structures are similar from a hydraulic point of view and can be analyzed using a single body of hydraulic theory. In practice, a device is typically called a weir if the control section is essentially formed by raising the channel bottom; a flume is formed by narrowing the channel. If the control section is formed by both raising the channel bottom and narrowing the width, the structure is usually called a flume (Figure 1.3). However, several structures may be called either a weir or flume. When in doubt, we will use the general term long-throated flume.

1.3 History and Advantages of Long-Throated Flumes

The hydraulic theory for predicting discharge through long-throated flumes has resulted from over a century of development. The first laboratory and theoretical studies on critical-depth flumes were made by Belanger in 1849 and by Bazin in 1896. These studies were extended by Crump (see Ackers et al. 1978, and Inglis 1928), Jameson (1930), Fane (1927), Palmer and Bowlus (1936), and others in the early part of this century. The theory and dimensional requirements for these flumes were well known by the 1950's (Wells and Gotaas 1958); however, calibration still required an empirical discharge coefficient. Theoretical predictions of flow were

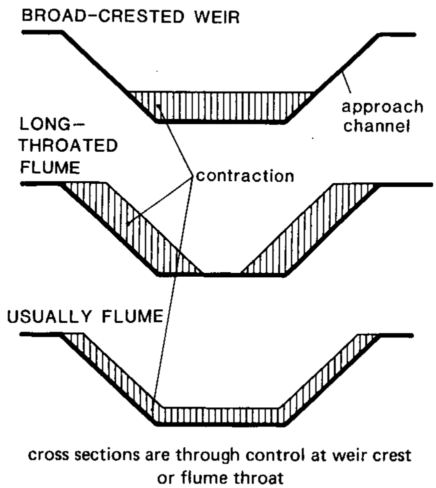


Figure 1.3 Distinctions between weirs and flumes.

investigated by Ackers and Harrison (1963) and further refined by Replogle (1975). The stage-discharge theory or calibration model of Chapter 6 is essentially that presented by Replogle (1975), with minor improvements. Bos (1978) and Bos and Reinink (1981) developed a procedure for determining the required head loss across these flumes. This general theory was incorporated into the current model, with minor modifications to make it consistent with the procedures for the stage-discharge computations. The model has been developed to assist users with the design of a flow measuring structure for an arbitrarily shaped canal. It also supplies a complete prediction of the hydraulic characteristics (head-discharge relationship and required head loss) of long-throated, critical-flow flumes and weirs.

The calculations needed to compute head-discharge relationships and evaluate design alternatives are iterative; thus, computer programs greatly facilitate analysis of these structures. Early programs written in FORTRAN used a batch mode of operation to analyze the performance of single designs (Bos et al. 1984; Clemmens et al. 1987b). In the early 1990's the International Institute for Land Reclamation and Improvement (ILRI) in The Netherlands and the Agricultural Research Service (ARS) contracted for the development of an interactive computer program for long-throated flume design (Clemmens et al. 1993). This program was written primarily in the Clipper language (with a few C subroutines) and functioned under the MS-DOS operating system. The program included on-screen graphic displays of flume geometry; a design optimization routine; features to assist with calibration of existing flumes; and printed output of rating tables, rating equations, wall gage data, and discharge-head curves. This program was known as FLUME 3.0. The WinFlume computer program included with this book is an upgrade to these previous programs with numerous improvements, the most notable being an improved user interface for the Microsoft Windows® operating systems, additional output options, and an improved design module. The hydraulic theory used by WinFlume is unchanged from the FLUME 3.0 program. WinFlume is written entirely in the Visual Basic® 4.0 programming language.

In Section 1.2 we have already mentioned some of the advantages of these modern long-throated flumes for open-channel flow measurement. Their primary advantage is, of course, the theoretical predictability of their hydraulic performance. In addition, they have the following major advantages over all other known weirs and flumes (e.g., Parshall flumes, cutthroat flumes, H-flumes, sharp-crested weirs):

- Provided that critical flow occurs in the throat, a rating table can be calculated with an error of less than 2 percent of the listed discharge. The calculation can be made for any combination of prismatic throat and arbitrarily shaped approach channel.
- The throat, perpendicular to the direction of flow, can be shaped in such a way that the complete range of discharges can be measured accurately.
- The required head loss over the weir or flume is minimal to ensure modular flow – which occurs when a unique relationship exists between the upstream sill-referenced head, h_1, and the discharge, Q (Figure 1.4).
- This head-loss requirement can be estimated with sufficient accuracy for any of these structures placed in an arbitrary channel.
- Because of their gradual converging transition, these structures have little problem with floating debris.
- Field observations and laboratory tests have shown that these structures can be designed to pass sediment transported by open channels with subcritical flow. However, sedimentation can be a problem when sediment loads are excessive or when the flume causes a significant reduction in the approach channel flow velocity.
- Provided that the throat is horizontal in the direction of flow, a rating table can be computed using post-construction dimensions. Thus, an accurate rating table can be produced even if the flume is not constructed to the designed dimensions. The throat may also be reshaped as needed according to changing site conditions, and a new rating table can be computed using the modified dimensions.
- Under similar hydraulic and other boundary conditions, these are usually the most economical of all structures for accurately measuring open channel flows, provided that conditions are such that a weir or flume is feasible.

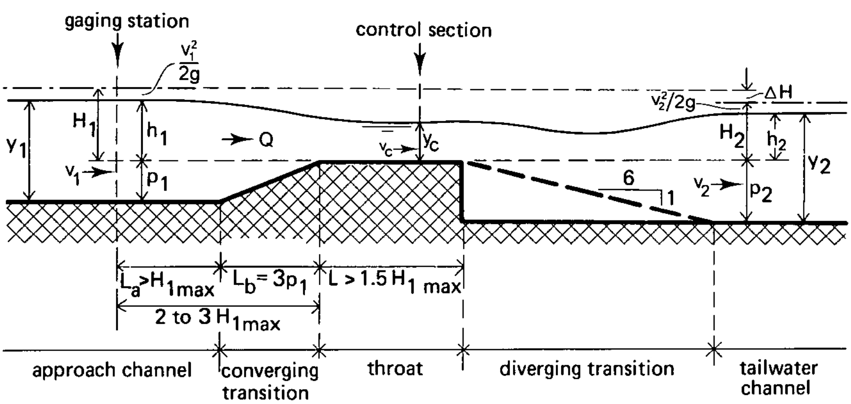


Figure 1.4 Profile of flow through a long-throated flume with stationary crest.

Because of the above advantages, these devices are useful for many flow measurement applications, particularly when the structure must have a minimal impact on existing flow conditions and water surface elevations.

1.4 Description of Long-Throated Flumes

Long-throated flumes are generally composed from five primary structural components, shown in Figure 1.2:

1. An approach channel that is necessary for the development of uniform and symmetric flow conditions and the establishment of a stable water surface whose elevation can be determined accurately. The approach channel may be lined as shown in Figure 1.2 or may be the original earthen channel.
2. A converging transition section in which the subcritical approach flow accelerates smoothly toward the throat with no discontinuities or flow separation—the transition may consist of plane surfaces or may be rounded.
3. A throat, or control section, in which the flow passes through critical depth. The throat must be horizontal in the direction of flow, but in the direction perpendicular to the flow any shape may be used.
4. A diverging transition in which the velocity of the supercritical flow exiting the throat section is reduced and energy is dissipated or partially recovered—if energy recovery is not needed, an abrupt transition can be used.
5. A tailwater channel where the water level is a function of the flow rate and the hydraulic properties of the downstream channel and structures. The range of water levels in this channel is fundamentally important to the design of the structure because it determines the elevation and size of the control section needed to maintain critical flow through the flume.

In addition to these five structural components, a gaging station in the approach channel is necessary. At the gaging station, the difference in elevation between the approach water level and the sill or crest of the throat section will be measured. This difference in elevation is known as the sill-referenced head, h_1. The flow rate through the flume will be computed as a function of this upstream sill-referenced head.

The major differences between long-throated flumes and broad-crested weirs stem from historical use of terminology rather than hydraulic properties. Both structures create flow at critical-depth by means of a contraction of the flow area in the throat of the structure (Figure 1.3). As discussed in Section 1.2, weirs are formed by raising the channel floor, and the control section is often called the sill or crest. Flumes are formed by narrowing the channel width, and the control section is usually called the throat. When the contraction is produced with a combination of raising the floor and narrowing the width, the term flume is usually used. The common characteristic of all the devices discussed in this book is a control section length sufficient to ensure the development of essentially parallel streamlines in the section in which the flow passes through critical depth. Without the long throat, one-dimensional flow theory cannot be used to obtain an accurate calibration.

Figure 1.4 shows the general profile of flow through a long-throated flume, along with some recommended dimensions for key components. The subscripts 1 and 2 refer to conditions in the approach and tailwater channels, respectively, and the subscript *c* will refer to conditions at the critical section. The reference elevation for energy levels is the bottom of the flume throat or crest of the weir sill, which will be referred to as the sill reference. The discharge is *Q*, the flow velocity is *v*, the sill height is *p*, the waterdepth is *y*, and the sill-referenced head is *h*. The total energy head, *H*, and the energy loss across the flume, ΔH, are also shown.

Figure 1.5 shows the profile of flow through a long-throated flume with a movable crest. This device functions both as a water measurement structure and as a flow regulator. In operation, the crest is moved vertically to maintain a constant upstream water level in the approach channel, thereby changing the sill-referenced head. The converging transition in these structures is modified to consist of only a radius on the upstream edge of the movable crest; there is no diverging transition.

As mentioned in the description of Figure 1.2, the five parts of the channel and structure may have a wide range of shapes and sizes. Therefore, the visual appearance of actual structures differs widely depending on function, size, and construction material of the flume or weir. Several possible configurations are illustrated in Figure 1.6, and Figures 1.7 through 1.13 show some examples of actual flumes.

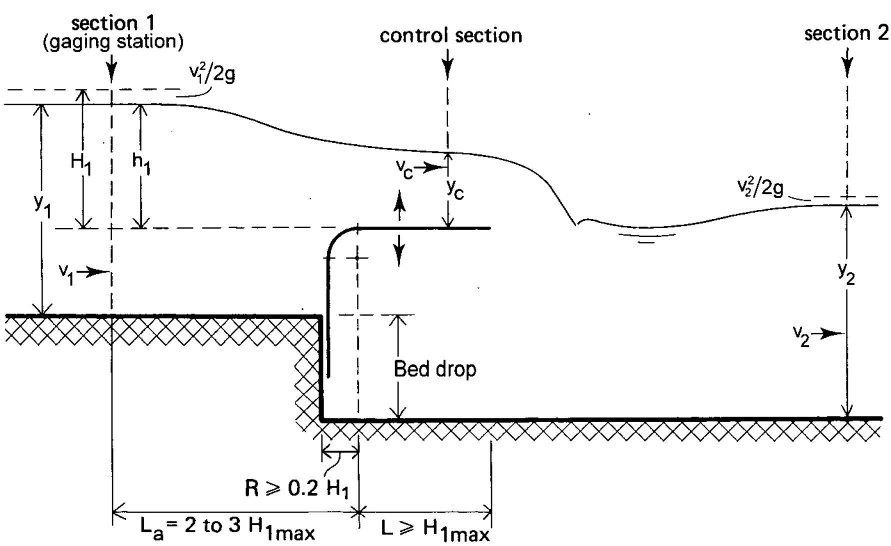


Figure 1.5 Profile of flow through a long-throated flume with vertically movable crest.

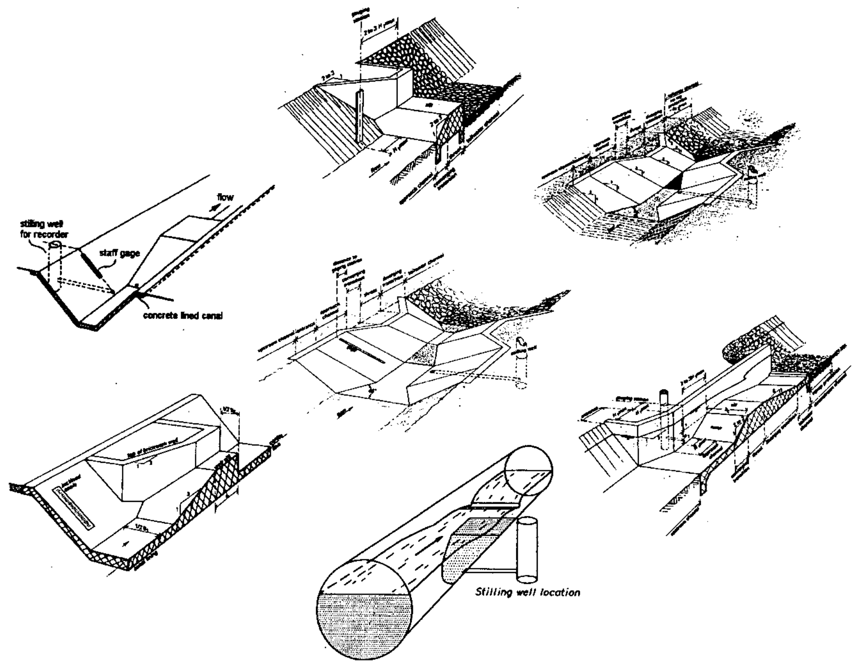


Figure 1.6 A variety of flume and weir configurations can be classified as long-throated flumes.

Figure 1.7 Portable RBC flume to measure flow in small drains, earthen canals, or irrigation furrows. This sheet-metal flume measures up to 9 liters/s. The flume can also be constructed from PVC or marine plywood. If another size and/or shape is used, the maximum capacity of the portable version of the flume can be as high as 150 liters/s (Greenland).

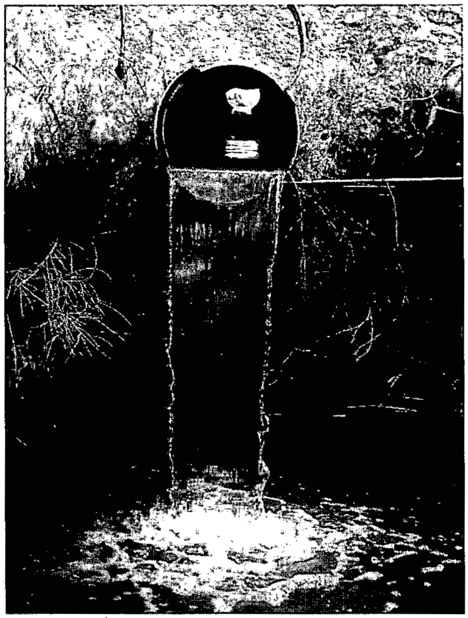


Figure 1.8 This 0.15 m high sill in a 0.60 m diameter pipe is used to measure flow from an irrigation lateral into a drainage channel. The control section can also be triangular or trapezoidal to measure low flows more accurately (Arizona).

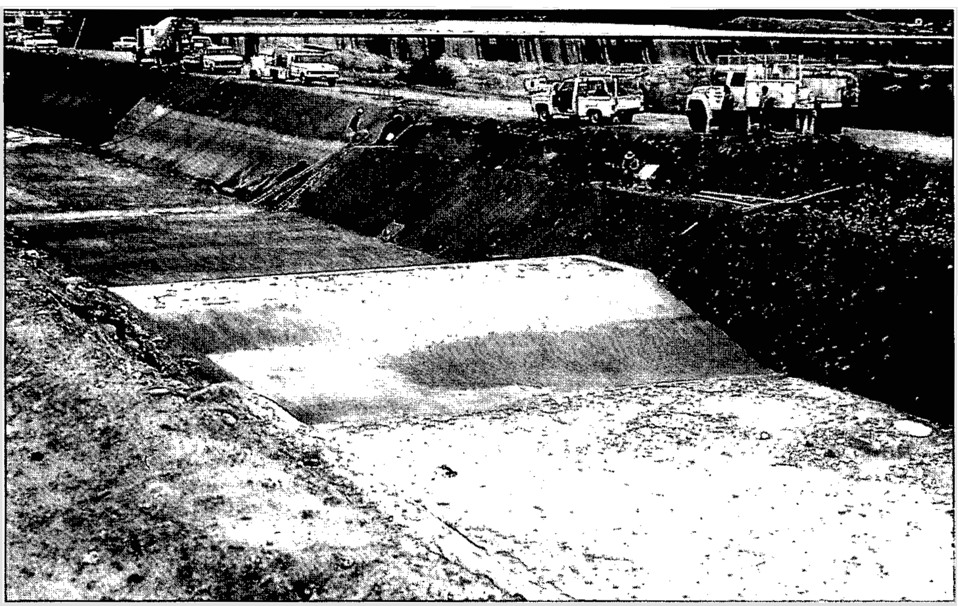


Figure 1.9 This 15.45 m wide flume has a throat length of 3.05 m. At its design capacity of 56.6 m^3/s, a head loss of only 0.12 m is required for flow to be modular. On large structures, a 6:1 downstream ramp truncated at half the total drop height reduces construction cost. Low crests (<0.5 m) often have a vertical downstream face (Arizona).

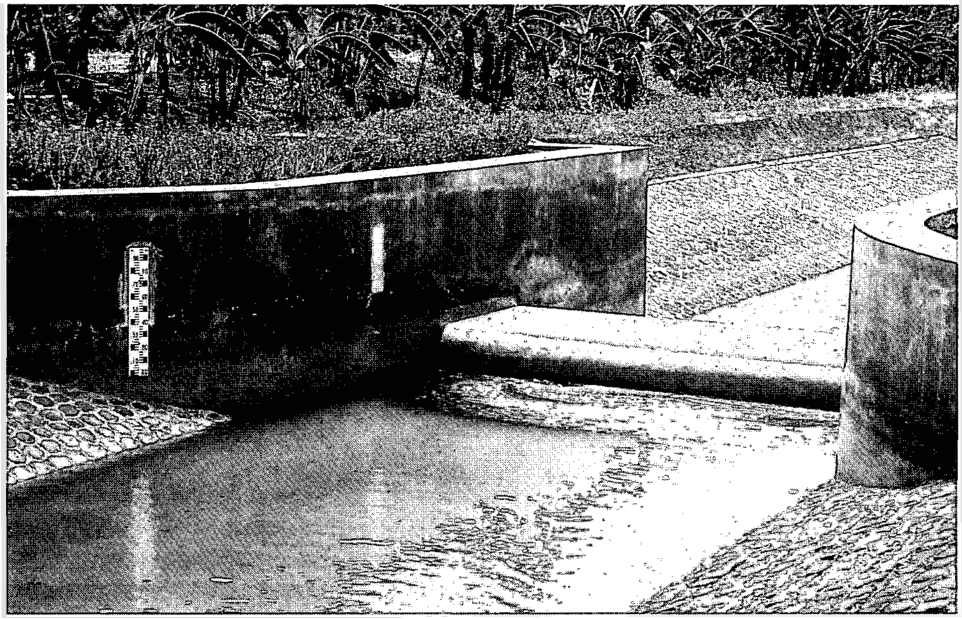


Figure 1.10 This rectangular flume in an unlined channel was constructed of masonry. All five flume parts are contained within the rectangular, lined section. The contraction was made with a bottom sill (Indonesia).

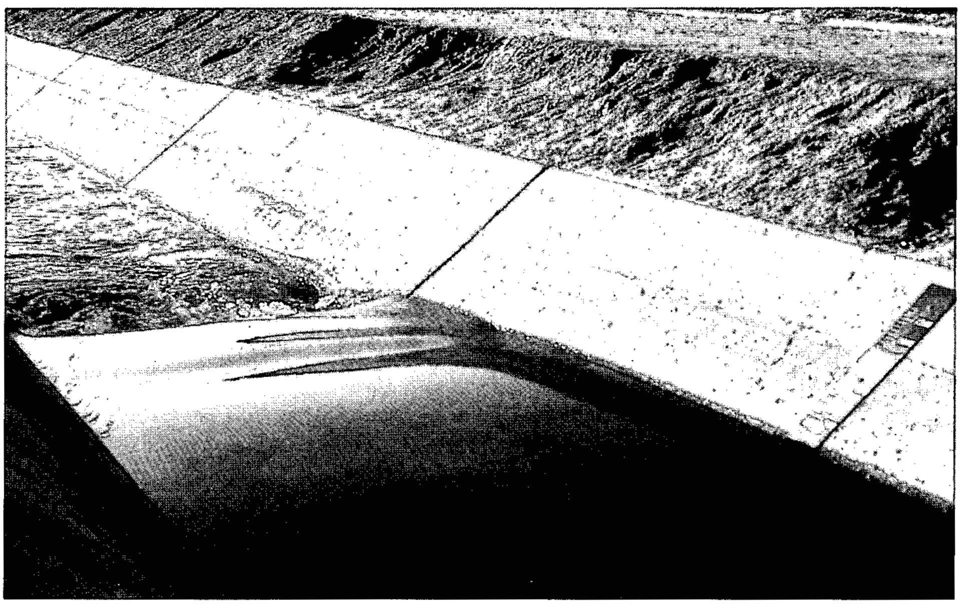


Figure 1.11 This flume was constructed by adding a bottom sill to an existing lined canal. The canal lining makes up part of the throat and the other four parts of the flume. This style is often called a Replogle flume (Arizona).

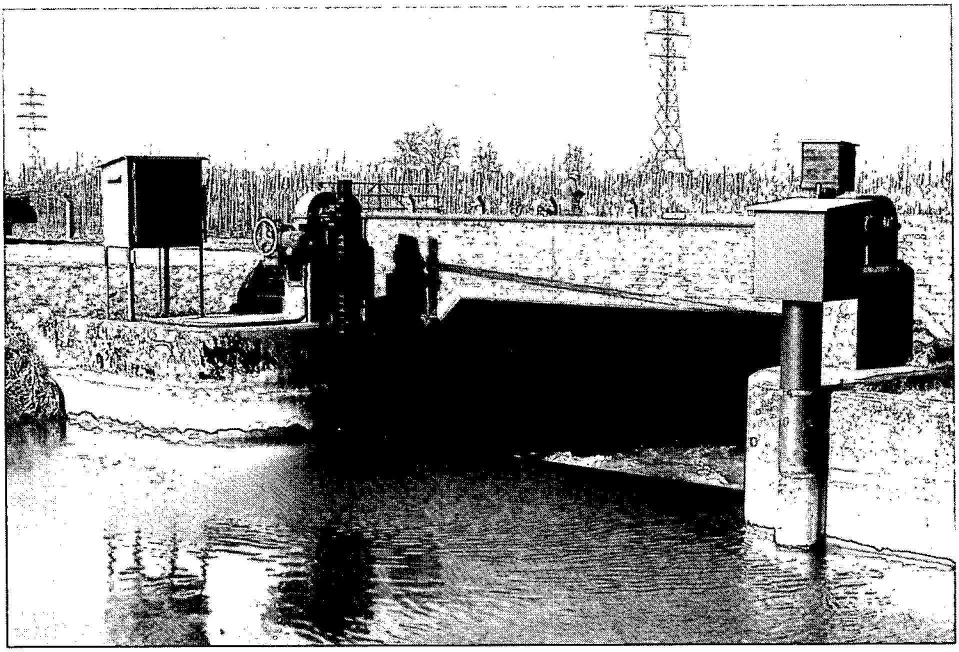


Figure 1.12 A broad-crested weir with movable crest can be used to regulate and measure the flow into a canal. This combination of functions facilitates the control of irrigation water (Netherlands).

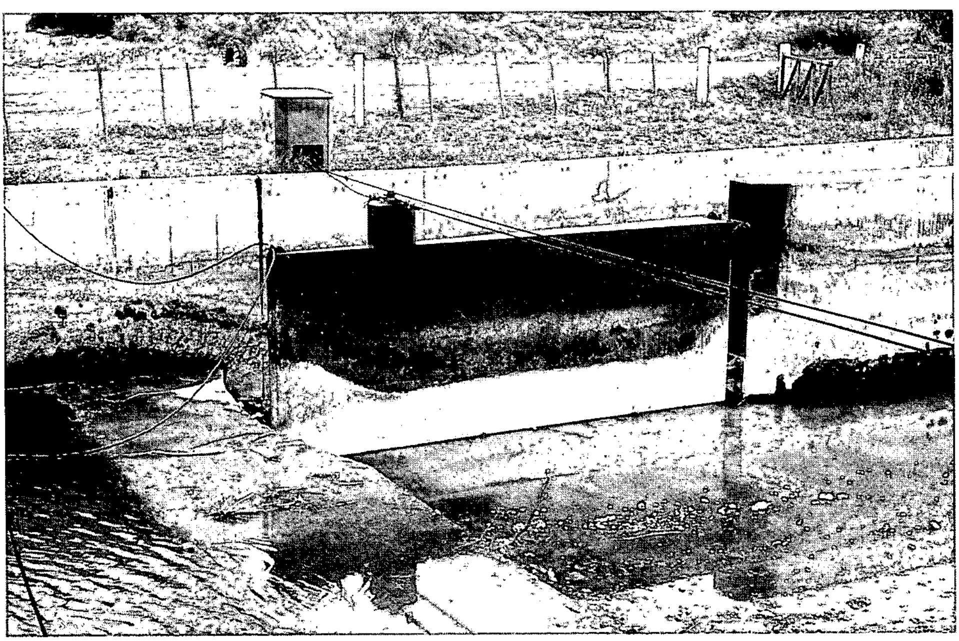


Figure 1.13 In irrigation systems, flow can be measured and divided accurately by use of a division door downstream from a broad-crested weir (Argentina).

2. Design Considerations

2.1 Introduction

Before proceeding with detailed discussions of the various types of long-throated flumes, it will be worthwhile to consider some basic issues related to the functions of flow measurement and regulation structures and the demands made upon them. This discussion will facilitate the selection of the proper type of structure for a given application. We will distinguish between two basic functions of a structure—measurement of flow rate and controlled regulation of flow rate—and will consider demands upon the structure originating from four sources:

1. The hydraulic performance,
2. The construction and/or installation cost,
3. The ease with which the structure can be operated, and
4. The cost of maintenance.

Although we will be discussing issues of relevance to all types of flow measurement and regulation structures, we will be focusing on long-throated flumes. Design procedures for long-throated flumes are given in Chapter 5. Chapter 8 describes software that can be used to aid users with the design process, including most of the design considerations given here.

A wide variety of methods and devices are available to measure flow. To determine the specific type of structure, if any, to be used, we must first consider the frequency at which measurements are needed and the duration for which they are required. Together with information on the size and type of canal, this will lead to the use of a

- Velocity-area method (a one-time measurement without the use of a structure),
- Portable and reusable structure,
- Temporary custom-built structure, or
- Permanent structure.

As Figures 1.6 through 1.11 show, structures that only measure the flow rate do not require movable parts. The upstream sill-referenced head can be measured with a variety of instruments, which will be discussed in detail in Chapter 4. If we need to measure the total volume of flow, we can use a weir or flume and a recording device that integrates the flow rate over a period of time.

Structures that both measure and regulate the flow are needed when water is taken from a reservoir, or when an irrigation canal is to be split into two or more branches. A regulating weir has a sill that is movable in a vertical direction, while a flow division structure has a divider wall that may be movable in a horizontal direction.

A regulating weir can be operated to maintain a constant upstream sill-referenced head, and thus a constant flow rate. Movable weirs are discussed in detail in Chapter 3.

2.2 Required Head Loss for Modular Flow

At the measuring site, the additional energy head needed to create critical-depth flow through a measuring flume can often be obtained from a drop in the channel bottom or from the energy head some distance upstream due to channel slope. Channel-bottom drops are often found in natural streams and can be provided in newly designed canal systems. However, if a structure has to be retrofitted to an existing channel, one must often create backwater upstream with a raised crest or narrowed throat (i.e., flow contraction) to locally increase the upstream head for use across the flume. Figure 2.1 illustrates the head loss, $H_1 - H_2$, from the upstream sill-referenced energy head, H_1, to the downstream sill-referenced energy head, H_2, which can also be expressed as $(H_1 - H_2)/H_1$, the fraction of the upstream head lost across the structure. This ratio can also be written as $(1 - H_2/H_1)$, in which H_2/H_1 is the submergence ratio. For low values of the submergence ratio, the tailwater level, y_2, (and thereby H_2) does not affect the relationship between h_1 and Q, and flow through the structure is called modular. For high H_2/H_1 ratios, flow in the throat cannot become critical, and the upstream sill-referenced head is influenced by the tailwater level, thereby affecting the relation between h_1 and Q; the flow is then non-modular. That submergence ratio separating modular from non-modular flow is called the modular limit, ML (see also Section 6.6.2).

If the downstream energy head, H_2, is less than the critical depth, y_c, at the control section, the energy available for head loss exceeds $H_1 - y_c$. In this case, there is no need to transform the kinetic energy at the control section (i.e., velocity head, $v_c^2/2g$, see Section 6.2) into potential energy downstream from the transition. The downstream water level, h_2, can be obtained even if all of the kinetic energy at the control section is lost. In other words, there is no need for a gradual transition between the throat and the downstream channel (Figure 2.2).

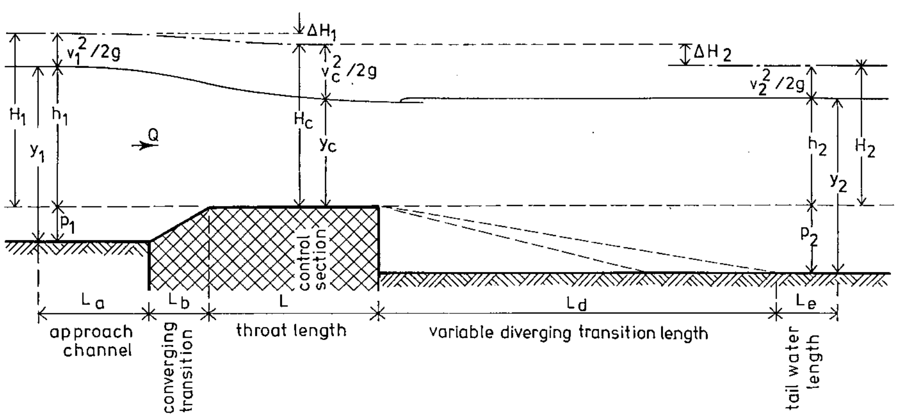


Figure 2.1 Terminology for flow through a long-throated flume.

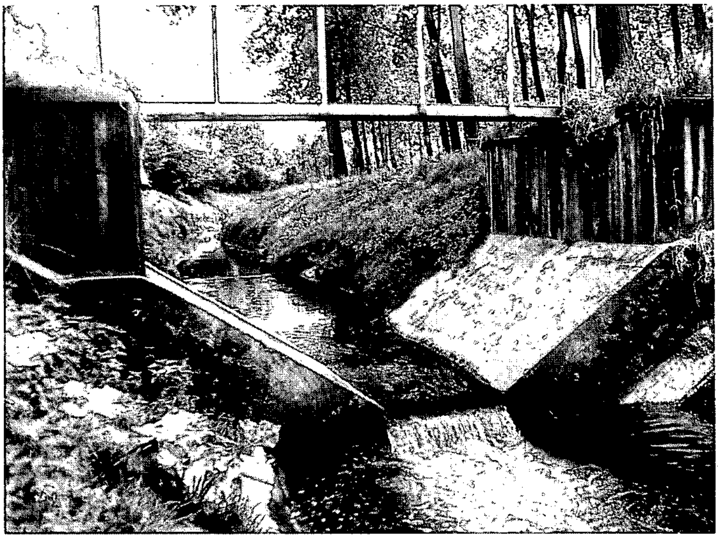


Figure 2.2 If the tailwater level is sufficiently low, there is no need for a gradual downstream transition (The Netherlands).

If the head loss over the structure is limited to such an extent that the downstream water level, h_2-level, becomes higher than the y_c-level, then some of the kinetic energy in the control section must be converted back to potential energy in the downstream channel. The amount of potential energy that can be regained depends mainly on the degree of expansion of the transition and on the ratio of wetted cross-sectional areas at the control section and at the section where h_2 is determined, A_c/A_2. Table 2.1 shows the approximate head loss, ΔH_{max}, required for typical control section shapes under the most unfavorable condition of a sudden downstream expansion into a quiescent reservoir (i.e. $v_2 = 0$). If the H_2-level is low enough to provide for a head loss of at least ΔH_{max} across the flume, then sufficient potential energy will be recovered, and a gradual transition is still not needed.

If a head loss of at least ΔH_{max} is not available, then a gradual transition can be added to recover additional potential energy. If the expansion of the downstream transition is gradual (6:1, horizontal-to-vertical), and if the flow velocity in the tailwater channel is high ($v_2 > 1$ m/s), the modular limit of long-throated flumes may exceed 0.90 (Figure 2.3). The methods presented in Chapter 6 can be used to determine the required

Table 2.1 Required head loss under most unfavorable conditions: sudden transition and $v_2 = 0$ m/s.

Shape of control section	Power u of h_1 in $Q = K_1 h_1^u$	y_c/H_1	Minimum modular limit, H_2/H_1	ΔH_{max}
Rectangle	1.5	0.67	0.60	$0.40H_1$
Typical trapezoid or parabola	2.0	0.75	0.70	$0.30H_1$
Triangle	2.5	0.80	0.76	$0.24H_1$

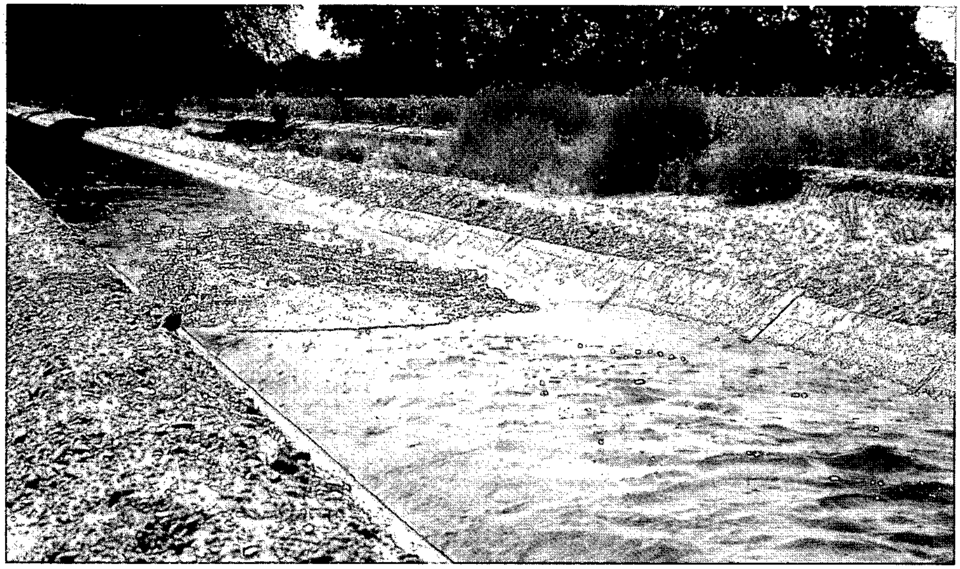


Figure 2.3 Long-throated flume with a modular limit of $ML = 0.90$ (Arizona).

energy loss over any combination of channel and flume. These methods are used in the software presented in Chapter 8. The following information is useful for any potential flume:

- Required head loss for modular flow, $H_1 - H_{2max}$,
- Maximum tailwater depth for modular flow, $y_{2max} = p_2 + h_{2max}$, and
- Modular limit $ML = H_{2max}/H_1$.

where the *max* subscript indicates the highest downstream energy or water level that will permit critical flow.

Analysis of the effect of the slope of the downstream transition will show that the modular limit increases with a more gradual expansion, primarily due to reduced turbulence from the expansion. However, very gradual transitions (slopes flatter than 10:1), lose so much energy due to friction in the long transition that the modular limit will not significantly increase (and may actually decrease). Because the construction cost of a very gradual and long transition is higher than that of a shorter one, we advise that the downstream transition be no flatter than 6:1.

Rather sudden expansion ratios like 1:1 or 2:1 are not very effective for energy conversion because the high velocity jet leaving the throat cannot suddenly change direction to follow the boundaries of the transition. In the flow separation zones that result, eddies are formed that convert kinetic energy into heat and noise. Therefore, we do not recommend the use of expansion ratios of 1:1, 2:1, or 3:1. If the length downstream from the throat is insufficient to accommodate a fully developed 6:1 transition, we recommend truncating the transition to the desired length rather than using a more sudden expansion ratio (Figure 2.4). Truncating the transition to half its

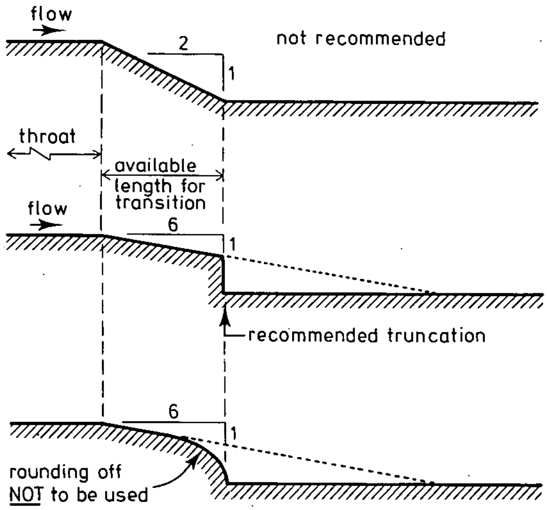


Figure 2.4 Truncation of a gradual downstream transition.

full length has a negligible effect on the modular limit. The truncation should not be rounded, since this guides the water into the channel bottom, causing additional energy losses and possible erosion.

2.2.1 Required head for desirable approach flow

To obtain an accurate flow measurement, the water level upstream from the structure must be accurately measured. This can be achieved if the approach channel Froude number is 0.5 or less. (see Section 2.11.1) The contraction, used to create backwater for meeting the head loss requirement discussed above, is often sufficient to meet this requirement. However, in some cases it may be necessary to provide additional contraction to adequately reduce the Froude number and slow the approach flow.

2.3 Required Freeboard

Freeboard is designed into channels to prevent overtopping of the embankment due to several reasons: wave action, changes in channel roughness over time, uncertainty about flow rates, etc. With reference to Figure 2.5, the ratio of flow rate at which the canal would overtop, $Q_{overtopping}$, to the maximum design discharge is

$$\frac{Q_{overtopping}}{Q_{max}} = \frac{(y_{1max} + F_1)^u}{y_{1max}^u} \qquad 2.1$$

where F_1 is the freeboard upstream from the structure. If, for example, the freeboard of an irrigation canal is 20% of the water depth at maximum flow, the constructed canal depth is $d_1 = y_{1max} + F_1 = 1.2y_{1max}$. Equation 2.1 then reads

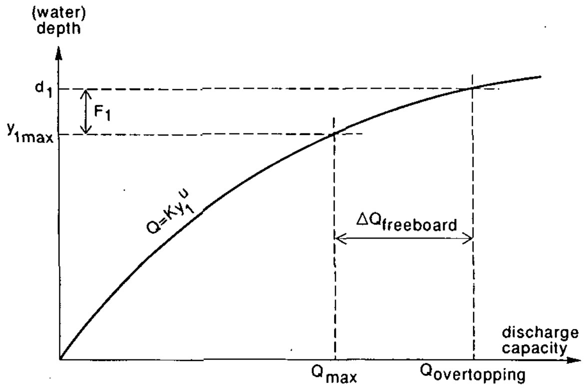


Figure 2.5 Relationship between freeboard and discharge capacity of channel.

$$\frac{Q_{overtopping}}{Q_{max}} = 1.2^u \qquad 2.2$$

The value of the power u depends on the shape of the channel (see Table 2.1). For wide and shallow channels, u is about 1.6, while for deep and narrow canals, u may be as large as 2.4. Hence, the extra canal capacity, $\Delta Q_{freeboard}$, provided by the 20% freeboard varies between 34 and 55 percent of Q_{max}.

In drainage channels, the design flow rate depends on the selected return period of the discharge from the drained area. The ratio $Q_{overtopping}/Q_{max}$ is determined by the designer based on the function of the channel. For example, the ratio might be set at 1.50 (50% safety margin), and the freeboard F_1 can be computed for the known values of y_{1max} and u using Equation 2.1.

When a flume or weir is placed in a channel, the requirements for freeboard upstream from the structure are greatly reduced because the relationship between flow rate and channel water depth is less variable because of the following effects:

- The upstream sill-referenced head is constant for a given discharge,
- An increase in channel roughness immediately upstream from the structure has a reduced effect on the water level because of the backwater effect of the structure, and
- The future collection of data on channel flow conditions will reduce the uncertainty about the flow rate.

These effects suggest that less freeboard is required immediately upstream from flow measurement flumes. We recommend a freeboard amount of 20% of the maximum sill-referenced head. In drainage canals, we recommend that a freeboard height be determined based on providing for safe conveyance of a percentage of excess flow using Equation 2.1.

Table 2.2 Values of u and Q_{max}/Q_{min} for various control shapes.

Shape of the control			Q_{max}/Q_{min} with rating error	
Basic form	Crest width, b_c with respect to h_1 at Q_{max}	u	$\leq 2\%$	$\leq 4\%$
Rectangular	All	1.5	35	100
Triangular	All	2.5	350	1970
Trapezoidal	Large	1.7	55	180
	Small	2.3	210	1080
Parabolic	All	2.0	105	440
Complex shapes	Large	Variable	> 100	> 200
	Small	Variable	> 250	> 2000

2.4 Range of Discharges to be Measured

The flow rate in an open channel tends to vary with time. The range of discharges, Q_{min} to Q_{max}, that should be measured depends strongly on the nature of the channel. Natural streams, for example, experience a considerably wider range of flows than do irrigation canals. The anticipated range of flows to be measured may be quantified with the ratio

$$\gamma = Q_{max} / Q_{min} \qquad 2.3$$

For the weirs and flumes described in this manual, the head versus discharge relationship can be expressed in the general form

$$Q = C_d K H_1^u \qquad 2.4$$

where C_d is the discharge coefficient which corrects for streamline curvature and energy loss due to friction upstream from the control section, and K is a factor that depends on the size of the structure and the units in which the discharge and head are expressed. The power u depends on the shape of the control section as follows (Clemmens and Bos 1992):

$$u = 0.5 + \frac{B_c y_c}{A_c} = \frac{B_c H_c}{A_c} \qquad 2.5$$

where B_c is the top-width of the water surface at the control section, y_c is the critical depth at the control section, H_c is the energy head at the control section, and A_c is the wetted area at the control section. For a rectangular control section u = 1.5, while for a triangular control section, u = 2.5. For all other shapes, u ranges between these values. (see Table 2.2)

The mathematical model presented in Section 6.5 produces a head-discharge rating with an error of less than 2% in the computed flow if the energy head to crest length ratio is in the range

$$0.070 \le H_1/L \le 0.70 \tag{2.6}$$

where L is the length of the flume throat. Over this range of heads, the discharge coefficient changes by about 10% (e.g., Bos (1985) shows that for $H_1/L = 0.070$, C_d averages 0.917; and for $H_1/L = 0.70$, C_d averages 1.002). Applying Equation 2.4 at Q_{min} and Q_{max} with K fixed, it follows that the corresponding range of flows that can be measured by a certain shape of the control section is

$$\frac{Q_{max}}{Q_{min}} = \left(\frac{1.002}{0.917}\right) \times \left(\frac{0.70}{0.070}\right)^u = 1.1 \times 10^u \tag{2.7}$$

Equation 2.5 shows that the value of u depends on the shape and relative width of the control section. Ranges of u are shown in Table 2.2 together with rounded-off values of Q_{max}/Q_{min}.

In irrigation canals, the ratio Q_{max}/Q_{min} rarely exceeds 35, so that all control shapes can be used. In natural drains, however, the range of flows to be measured usually will determine the shape of the control. In natural streams with a small catchment area, the range of discharges to be measured may exceed $Q_{max}/Q_{min} = 350$. However, depending on the characteristics of the catchment area, flows in the range of Q_{min} or Q_{max} may not contribute significantly to the total discharged volume of water. If a somewhat larger error (up to 4%) can be tolerated in the rating at the extreme flows, the range of flows that can be measured increases considerably to the values shown in the last column of Table 2.2 (based on $0.05 \le H_1/L \le 1.0$). These values are computed from

$$\frac{Q_{\max}}{Q_{\min}} = 1.1 \times \left(\frac{1.0}{0.05}\right)^u = 1.1 \times 20^u \tag{2.8}$$

2.5 Influence of Downstream Channel Conditions

As discussed in Section 2.2, for a structure to produce a unique relationship between the sill-referenced head in the approach channel, h_1, and the discharge, Q, the upstream water level must be sufficiently higher than the tailwater level, y_2. Hence, to design a structure, the tailwater levels must be known over the range of discharges to be measured. Usually, the tailwater levels need only be checked at Q_{min} and Q_{max}, since if flow is modular at those two flows it should also be modular at intermediate flows.

There are two situations of interest with regard to the downstream channel. First, the tailwater level downstream from the flume may be influenced only by downstream channel friction. When this occurs, the flow is said to be uniform (not varying with location along the channel) and the water depth is at normal depth. The Manning equation is often used to describe flow at normal depth:

$$Q = \frac{C_u}{n} A R^{2/3} S_f^{1/2} \tag{2.9}$$

Table 2.3 Conservative values of Manning's roughness coefficient to estimate water levels downstream from a flume.

Type of channel and description	Conservative n-value
Concrete-lined	
Float finished	0.018
Float finished, with gravel on bottom	0.020
Gunite	0.025
With algae growth	0.030
Masonry	
Cemented rubble	0.030
Dry rubble, open joints	0.035
Earthen channels	
Straight and uniform, few weeds	0.035
Winding, cobble bottom, clean sides	0.050
Non-uniform, light vegetation on banks	0.060
Not maintained, weeds and brush uncut	0.150

where Q is the discharge, n is the Manning roughness coefficient, A is the cross-sectional area of the channel, R is the hydraulic radius (area divided by wetted perimeter), S_f is the friction slope, and C_u is a constant that has a value of 1.0 when using units of meters and m^3/s, or 1.486 when using units of ft and ft^3/s. When the flow in the downstream channel is at normal depth, the friction slope and bed slope of the channel are equal. Because channel roughness changes over time, tailwater levels should be determined for the seasonal maximum downstream channel roughness. The n-values given in Table 2.3 may be used for a tentative estimate. For broad-crested weirs in channels with a uniform cross section and with tailwater at normal depth, the submergence need only be checked at maximum flow. This is because the tailwater level will generally decline faster than the upstream depth if the flow rate is reduced. Often, there are structures in the channel downstream from the flume or weir to be designed. When this occurs, the tailwater level downstream from the flume is not governed by normal depth, but by backwater (or drawdown) from the downstream structure (or obstruction, overfall, etc.). In such cases, the tailwater level depends greatly on the properties and settings of the downstream structure. From a practical standpoint, the easiest way to determine the resulting tailwater level is to measure it during the worst-case conditions. Even with a broad-crested weir, tailwater needs to be checked at both minimum and maximum flow in these situations, since backwater can cause high tailwater depths even at low flows.

2.6 Sediment Transport Capability

Besides transporting water, almost all open channels will transport sediments. To obtain reliable long-term performance of a flow measurement or regulation structure, sediment transported by the channel should be passed through the structure to the extent possible.

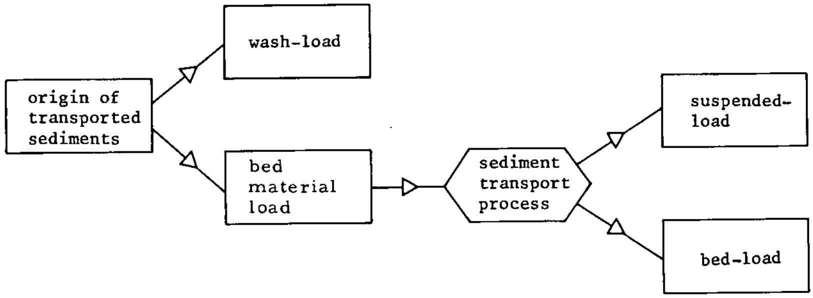


Figure 2.6 Terminology used to discuss sediment transport.

This ensures that approach flow conditions and performance characteristics will not change over time because of sediment accumulation. Sediments of varying sizes are present in most channels, and they originate from different sources and are transported by different mechanisms, as illustrated in Figure 2.6. Some basic discussion of the different terminology and possible transport mechanisms is valuable before we address the design of a sediment-discharging structure.

2.6.1 Bed load and suspended load

Bed load consists of sediment particles sliding, rolling, or bouncing (saltating) along or near the channel bed, often in the form of moving bed forms such as dunes and ripples (Figure 2.7). Suspended load refers to bed particles in transport for which the gravity force is counterbalanced by upward forces due to turbulence of the flowing water, so that the particles remain suspended above the bed for appreciable distances and are primarily transported while suspended. These definitions are necessarily imprecise, and clearly distinguishing between the two forms of transport is generally difficult in practice.

2.6.2 Bed material load and wash load

Total sediment load can be subdivided based on the size of the transported particles. Bed material load is the portion of the load consisting of particle sizes found in appreciable quantities in the shifting upper portions of the bed; transport occurs both as bed load and suspended load. Wash load is made up of particles smaller than the bulk of the bed material; these particles are always in suspension and color the water. The total sediment load is the sum of the bed material load and the wash load. However, wash load particles are usually so small (< 50mm) and have such low fall velocities that they do not contribute significantly to local scour and siltation problems, except in reservoirs, ponded canals, and fields. Passing the bed material load is the primary concern for flow measurement structures.

Bed material load can be estimated using a variety of equations, most of which compute the sediment transport capacity, T, expressed as a volume per unit width of

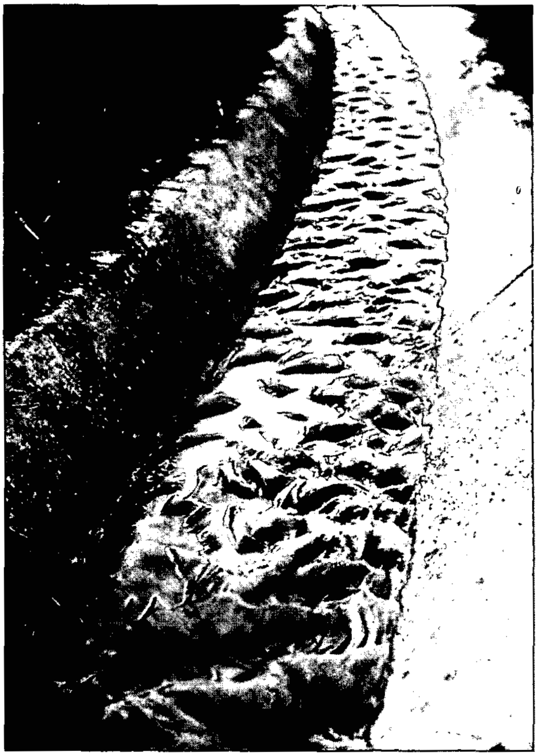


Figure 2.7 Bed-load sediment in a lined irrigation canal (Portugal).

the channel, as a function of the so-called dimensionless flow parameter, Y (Meyer-Peter and Müller 1948). Y is computed from

$$Y = \frac{\mu y S_f}{\rho_r D_a} \qquad 2.10$$

where

μ = ripple factor, which depends on the form of the channel bed; (It varies from about 0.5 for slightly rough beds to 1.0 for smoother bed forms. We conveniently use μ=1.0.)
y = water depth (in meters);
S_f = hydraulic gradient or friction slope;
ρ_r = relative density = $(\rho_s - \rho)/\rho \approx 1.65$;
ρ_s = density of sediment particles;
ρ = density of water; and
D_a = characteristic particle diameter (in meters), for example the average particle diameter.

If the flow parameter, Y, exceeds 0.047, the sediment particles on the channel bed start to slide, roll, or jump, often in the form of moving bed shapes such as dunes and ripples. This is called bed-load transport, and it can be calculated with the equation of Meyer-Peter and Müller (1948), which reads

$$X = A_1(Y - 0.047)^{1.5} \tag{2.11}$$

where A_1 is a factor with an average value of 8 and X is the transport parameter (dimensionless), which is

$$X = \frac{T}{\sqrt{\rho_r g D_a^3}} \tag{2.12}$$

with

g = acceleration of gravity (9.81 m/s^2), and
T = sediment transport in solid volume per unit width of channel (m^3/s per meter width).

For a particular channel, the values of μ, ρ_r, and D_a are fixed. The flow parameter and the sediment transport capacity per unit width change if the water depth and/or hydraulic gradient change.

2.6.3 Avoiding sediment deposition

The most appropriate method of avoiding sediment deposition in the channel reach upstream from the flume or weir is to avoid a decrease in the flow parameter, Y. This requires the product of the depth, y, and the hydraulic gradient, S_f, to remain constant. To achieve this, the structure should be designed in such a way that it creates a minimal backwater effect. With respect to the approach channel bottom, this means that the curve of Q versus $h_1 + p_1$ for the control should coincide to the extent possible with the curve of Q versus y_1 for the upstream channel (Figure 2.8). This near coincidence should occur for those flows that are expected to transport bed-load material (i.e., $Y > 0.047$ in Equation 2.10).

To obtain a reasonable match of the two curves, when the contraction is from the sides only ($p_1 = 0$), the u-value of the control section (i.e., the exponent in the approximate head-discharge rating equation $Q = K_1 h_1^u$) should equal the u-value of the upstream channel. However, if a raised sill is used, the u-value of the control should be less than that of the channel. The u-value of most trapezoidal channels varies between 2.3 for narrow-bottomed channels and 1.7 for wide channels; the shape of an appropriate control section can be selected using the information in Table 2.2.

To obtain a perfect match of the two curves, the structure should create essentially no backwater in the channel section upstream. To achieve this, a drop in the channel bottom is required that is sufficient to provide the needed flume head loss and thus guarantee modular flow. In natural channels, the structure can be located at a site where the channel bottom has a natural drop. When designing flow measurement

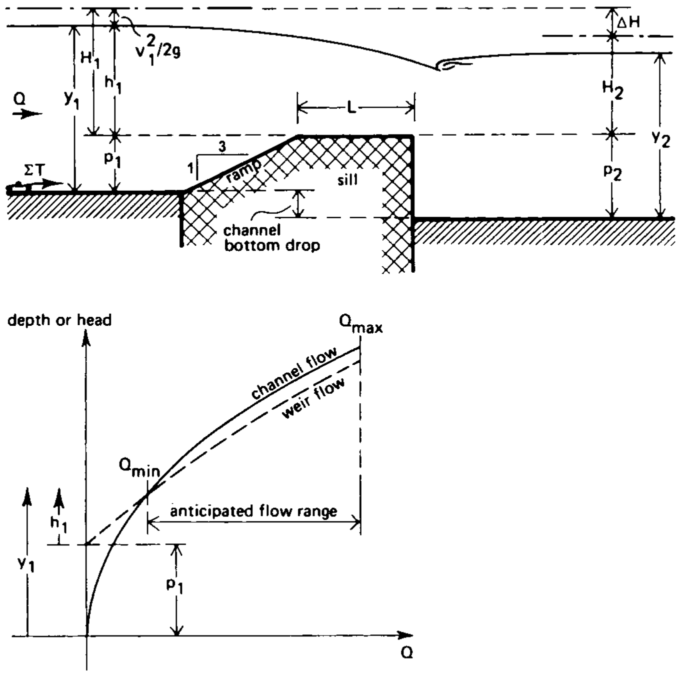


Figure 2.8 Matching of Q vs. y_1 and Q vs. h_1 curves for a sediment-discharging structure.

structures for new canal systems, we recommend including the necessary bottom drop as a part of the system design. When adding flow measurement structures to an existing canal system, it will not be possible to completely satisfy this requirement unless measurement devices can be located at existing drops in the system.

Laboratory tests have shown that both long-throated flumes and broad-crested weirs can pass all sediment that the upstream channel can transport (Bos 1985; Bos and Wijbenga 1997). Figure 2.9 shows a flat-bottomed long-throated flume that has performed satisfactorily since 1970.

If no bottom drop is available to provide for head loss over the structure, the head versus discharge curve of the structure will always be above the stage versus discharge curve of the upstream channel. Even if the structure is designed to operate with the minimum possible head loss, sedimentation will occur in the upstream channel. To avoid sedimentation at the gaging station, the approach channel should be smaller than the upstream channel. It is recommended to contract the channel so the Froude number at the gaging station equals 0.5 at maximum flow and remains as high as possible at lower flows (i.e., minimal bottom contraction). In this way sediment that reaches the structure will pass through the approach channel and the control section, and the structure thus can measure flow accurately. As mentioned before, however, sedimentation will occur in the upstream channel. This trapped sediment needs to be removed at regular intervals.

Figure 2.9 Flat-bottomed long-throated flume in sediment-transporting stream (Arizona).

2.7 Passage of Floating and Suspended Debris

Open channels, especially those that pass through forested or populated areas, transport all kinds of floating or suspended debris. If this debris is trapped by the gage or the structure, the approach channel and control section become clogged, impairing the ability of the structure to measure discharge and potentially causing overtopping of the upstream channel banks.

To avoid trapping of debris, the staff-gage or recorder housing should not interfere with the flow. All weirs and flumes described in this book are sufficiently streamlined to avoid debris trapping, provided that the debris does not exceed the size of the throat section.

If two or more weirs are installed side by side, the intermediate piers should be at least 0.30 m wide and have rounded noses. Sharp-nosed and narrow piers tend to trap debris.

2.8 Accuracy and Precision

Accuracy and precision are important characteristics of any flow measurement device. Accuracy is the ability of the device to indicate the true flow rate without systematic bias above or below the true value. Precision is the ability of the device to repeatedly produce similar measurement results. Figure 2.10 illustrates these

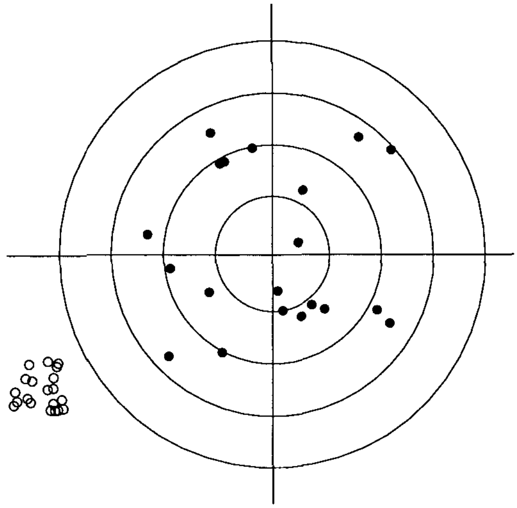


Figure 2.10 Accuracy versus precision. The set of solid points exhibits good accuracy, but low precision, while the set of open points have poor accuracy and high precision.

concepts. When analyzing these issues for a flow measurement device, we must considered the complete system made up of both the critical-flow structure and the method or instrument used to observe and record the upstream sill-referenced head. The precision and accuracy of the water-level measurement will affect the precision and accuracy of the complete measurement system.

Sources of error (the term *uncertainty* is also commonly used) in individual measurements are both systematic—having a bias that affects repeated measurements in a similar manner—and random—averaging to zero over the course of many measurements. Even if all systematic biases are eliminated, the accuracy with which a single discharge measurement can be made with a particular structure is limited by the precision with which a measurement can be reproduced—i.e., the random errors. If, independently from one another, two identical structures with perfect upstream water level sensors are constructed and the same flow is passed over each, the actual upstream sill-referenced heads will usually not be equal, and the measured flow rates will thus be different. For the flumes and weirs in this book, the flow rates indicated by the two structures will be within ±2% of the true value if the rating table is developed using the model presented in Chapter 6 or the software described in Chapter 8 and the H_1/L values are between 0.07 and 0.70. Outside this range, accuracy is reduced as described in Section 6.5.3. If the rating of a custom-designed structure is determined using the Q versus h_1 equations given in Section 6.4, the error X_C is slightly larger, roughly 5%. These errors have a 95% confidence level. In addition to the rating table error X_C, the other important source of error in the measured flow rate is the uncertainty inherent in the determination of the upstream sill-referenced head, h_1. Section 2.8.9 will discuss how we mathematically combine errors attributable to these different sources. Prior to that, we will discuss several sources of error often encountered in practice.

2.8.1 Systematic errors

Systematic errors bias each measurement, and the amount of error is predictable (if the source of the error is known). Systematic errors can arise from many different sources, such as poor flume construction, improper installation of a staff gage, improper calibration of a water level sensor, improper setting of a water level sensor relative to the sill, etc. If, for example, the gage used to measure h_1 is installed too low, all measured h_1 values are systematically higher than the true h_1 values and the measured flows are thereby larger. The error will occur until the zero setting of the gage is checked and the gage is reset. A systematic error can be eliminated or accounted for if it becomes known.

2.8.2 Random errors

If two people read an h_1 value from a gage or recorder chart, often different values will be read. A third person might read yet another value. Some of the values observed are higher and some are lower than the true h_1 value. The observed values will typically have a random distribution around the true h_1 value. Random errors affect the accuracy of individual flow rate measurements, but over the long term they do not substantially affect the accuracy of the average of multiple measurements (their effect diminishes with $N^{1/2}$, where N is the number of measurements). Some sources of random error are known, while others are unknown. The magnitude of some random errors can be reduced, for example by installing a more precise water level sensor, but random errors can never be completely eliminated.

2.8.3 Spurious errors

Spurious errors are those errors that introduce grossly false data into the measurement process. They are unpredictable in their occurrence, magnitude, and distribution, and invalidate the discharge measurement. Some typical sources of spurious errors are human mistakes, malfunctions of the automatic head recorder, or obstructions to normal flow, such as branches or other debris blocking the control section (Figure 2.11). Spurious errors can sometimes be detected by comparing flow measurements at different times or at different locations within the canal system.

2.8.4 Zero-setting errors

In addition to the potential error mentioned earlier in the actual sill-reference setting of a gage, an unstable foundation for the structure or the head-reading device can cause a drift in zero setting. If soil under the structure, stilling well, or staff gage is subject to ground frost or changes in soil moisture, the zero setting may be altered. To limit the impact of such alterations, it is recommended that the setting be checked at

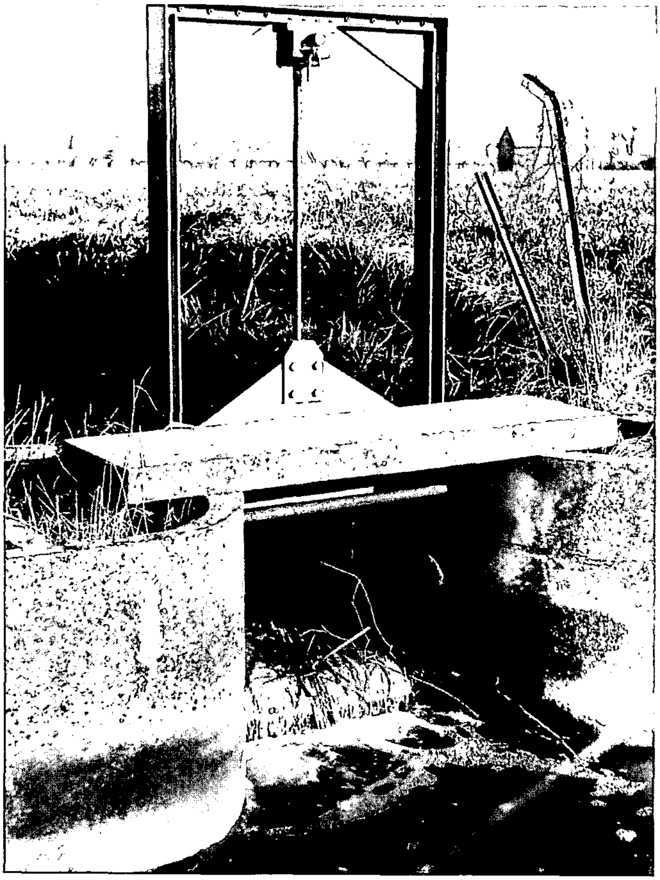


Figure 2.11 Large branches can be trapped and must be removed from the weir crest. (Courtesy of Delft Hydraulics Laboratory, The Netherlands).

least twice a year, for example, after a period with heavy frost, after a rainy season, and before the irrigation season. Also, an ice cover on the water may alter the zero setting. Detailed procedures for setting the zero of the gage are given in Section 4.9.

2.8.5 Algae growth

An important source of systematic errors in long-throated flumes is the growth of algae on the bottom and sides of the control section. The layer of algae has two effects: (1) the sill-reference level is raised by the thickness of the layer, causing an error in the head, and (2) the algae layer on the sides of the control reduces the wetted area A_c. To limit the error due to algae growth (or other dirt), the control section should be cleaned with a broom at regular intervals. Painting of the structure with a marine anti-fouling agent will reduce algae growth.

Table 2.4 Systematic and random errors for centimeter-graduated staff gages.

Staff Gage Installation	Systematic Error	Random Error
In standing water	0	0.003 m
In channel with smooth water surface	0.005 m	0.005 m
In channel with turbulent water surface	More than one graduation unit (> 0.01 m)	More than one graduation unit (> 0.01 m)

2.8.6 Head-reading error

The reading error of a staff gage is strongly influenced by the distance between the gage and the observer, the angle at which the gage must be read, the turbulence of the water, and the graduation units of the gage. A dirty gage face hinders readings and can cause serious reading errors. Staff gages should therefore be installed in locations where it is possible for the observer to clean them. The approximate magnitudes of reading errors on a staff gage with centimeter graduations are shown in Table 2.4. This shows that in turbulent water, staff-gage readings become inaccurate. The systematic error associated with turbulent flow is attributed to the general failure of most observers to accurately average the fluctuating values. Thus, to obtain accurate readings in turbulent water, a stilling well is recommended. The standing water level inside the stilling well can be measured by a

- Point gage to the nearest 0.0001 m,
- Dipstick to the nearest 0.001 m, and
- Staff gage to the nearest 0.003 m.

If the standing water level is measured by a recorder, the error in head registration depends on the float diameter, the accuracy of the zero setting, internal friction of the recorder, backlash in the mechanism, etc. The magnitude of most of these errors is inversely proportional to the square of the float diameter (see Section 4.4.5). Digital or punched-tape recorders register the head with an error of up to half of the registration unit. If a paper chart must be read, the errors depend greatly on the reduced scale at which it must be read. Depending on the care with which a recorder is installed and maintained, both the systematic and random errors in head can be 0.003 m or more. Errors of over 0.01 m are common for poorly maintained recorders. Pressure transducers can experience changes over time in both the offset (which affects the zero setting error) and gain (slope of response). Transducers should be re-calibrated periodically (e.g., annually).

Table 2.5 Percentage errors in discharge due to changes in as-built dimensions.

1% Change in Dimension of:	Discharge Error, %	Remarks
Upstream ramp length	0.01	Slope of ramp may vary from 2.5:1 to 4.5:1
Sill height, p_1	0.03	Influences approach velocity
Sill or throat length, L	0.1	Depends on value of H_1/L
Bottom width of control, b_c	up to 1	Depends on percentage change in wetted flow area at control
Wetted flow area at control	1	Linear relation

1° Change of:	Discharge Error, %	Remarks
Cross slope of sill	0.1	Has minor effect on area of flow
Sill slope in direction of flow	up to 3	Is most difficult factor to correct for
Side slope of control	0.5	Depends on change of wetted flow area

2.8.7 Stilling-well lag error

A stilling well dampens the fluctuations of the main channel water surface by the use of a small-diameter connecting pipe that limits the transfer of turbulent energy into the stilling well. A side effect of this damping is that the water level inside the well will lag behind the outside water level during a rising or falling flow. Such a systematic lag error also may occur if a leaking stilling well is used to measure heads in a lined canal constructed through pervious soils. A continuous flow of water is required through the connecting pipe into the stilling well to supply the leak. This flow through the connecting pipe requires some head loss, so the head in the well will always be lower than the outside head. Stilling well lag errors can be reduced by eliminating leakage out of the well and by increasing the size of the connecting pipe.

2.8.8 Construction-related errors

To justify the use of the rating tables given in this book or developed using the mathematical model or software, the constructed dimensions of the structure must be sufficiently close to the design dimensions for which the rating table was developed. A change in these dimensions will create an error between the true flow and that indicated by the rating tables. The relative order of magnitude of these added systematic errors is illustrated in Table 2.5. The most important errors are caused by the changes in the wetted flow area at the control section. The given rating tables may be corrected by the percentages shown in Table 2.5, provided that the sum of all errors does not exceed 5 percent. For larger deviations, we recommend generating a new rating table by use of the model of Section 6.5 or the software of Chapter 8. If the sill or throat is not level, but slopes in the direction of flow, this influences both the flow rating and the modular limit of the structure. It is more effective to level the sill than to attempt to correct for slopes larger than 2°.

2.8.9 Combination of errors

As discussed earlier, the measured flow rate is subject to two sources of error:

X_C = error in the rating table given in Appendix 4 or generated by the software in Chapter 8, and
X_{h1} = error in the measurement of the upstream sill-referenced head.

The value of X_{h1} is a combination of all errors affecting h_1 calculated by the equation

$$X_{h1} = 100\% \frac{\delta_{h1}}{h_1} = 100\% \frac{\sqrt{\delta_{ha}^2 + \delta_{hb}^2 + \mathrm{K} + \delta_{hn}^2}}{h_1} \qquad 2.13$$

where δ_{ha}, δ_{hb}, etc., are the various errors affecting the head measurement. The total error X_Q in the measured flow rate can then be calculated using the equation

$$X_Q = \sqrt{X_C^2 + (uX_{h1})^2} \qquad 2.14$$

where u is the head-discharge exponent. When using the procedures given in Chapters 5 or 8 to design a structure, the user supplies information on

- The range of flows to be measured, Q_{min} to Q_{max},
- The percentage errors that are allowed in one single flow rate measurement at minimum and maximum flow (X_{Qmax} and X_{Qmin}), and
- The random reading error δ_{h1} for the selected head measurement device. Table 2.6 shows some common values.

Table 2.6 Common magnitude of errors in determining sill-referenced head.

Device	Reading error, δ_{h1}, in h_1 if head detection device is located in: Open channel	Stilling well	Remarks
Point gage	Not applicable	0.1 mm	Commonly used for research
Dipstick	Not applicable	1 mm	Good for research and field use
Staff gage	4 mm	4 mm	$Fr_1 < 0.1$
	7 mm	5 mm	$Fr_1 = 0.2$
	> 15 mm	7 mm	$Fr_1 = 0.5$
Pressure bulb and recorder	up to 20 mm	Not required	Very suitable for temporary installations (error is 2% of h_{1max})
Bubble gage and recorder	10 mm	Not required	Stilling well not required but can be used
Float-operated recorder	Not applicable	5 mm	Stilling well is required
Flow totalizer	—	—	Some additional random and systematic errors are possible

Based on the user-selected shape of the control section and associated value of u, one can compute the heads, h_{1min} and h_{1max}, needed to maintain the user-desired accuracy at Q_{min} and Q_{max}. It is important to realize that most errors contributing to X_C and to X_{h1} have a random distribution. Hence, if many ($N > 15$) discharge measurements are made to calculate the volume of water that passes a structure during a period (day, week, etc.), these random errors tend to offset (their effect being proportional to $1/N^{0.5}$) and can be neglected. As a result, the error in the measured volume of flow is due to systematic errors only. Of these, the errors in zero-setting are the most common (see Section 4.9).

2.9 Sensitivity of the Metering Structure

We have seen that one source of error in the measured flow rate is the accuracy with which the upstream sill-referenced head, h_1, can be determined. The flow rate error that results from an incorrect determination of h_1 is a function of the sensitivity of the structure, S. For modular flow, the sensitivity can be computed from

$$S = \frac{100\Delta Q}{Q}\% = 100\,u\,\frac{\Delta h_1}{h_1}\% \qquad 2.15$$

where Δh_1 = the difference between the observed and true value of h_1.

The source of the error Δh_1 could be an unnoticed change in water level, a head reading error on scale or paper, the mislocation of the gaging station, an error in zero-setting of the scale or recorder, the internal resistance of the recorder, or a host of other possibilities. Figure 2.12 shows values of S as a function of the ratio $\Delta h_1/h_1$ and the u-value, the latter being indicative of the shape of the control section. Narrow-bottomed control shapes, such as the trapezoid or triangle have the greatest sensitivity, and are thus most susceptible to errors originating from the determination of h_1. Most structures have the highest sensitivity when h_1 is small, since the magnitude of Δh_1 is usually relatively constant.

There are both advantages and disadvantages to structures with high sensitivity. A structure with a high sensitivity will have the ability to pass large percentage changes in flow rate with little change in sill-referenced head. However, it will also be prone to large percentage errors in measured flow rate if h_1 cannot be determined accurately. Conversely, a structure with a low sensitivity (i.e., Q changes relatively little for a given change in h_1), is better able to resolve small differences in flow, since they cause relatively large changes in h_1 that can be easily measured.

2.10 Flexibility of Two Structures

Flow measurement and regulation structures are often constructed at canal bifurcations. When flow changes pass through a bifurcation, the manner in which the flow changes are distributed among the branches of the canal system is a function of

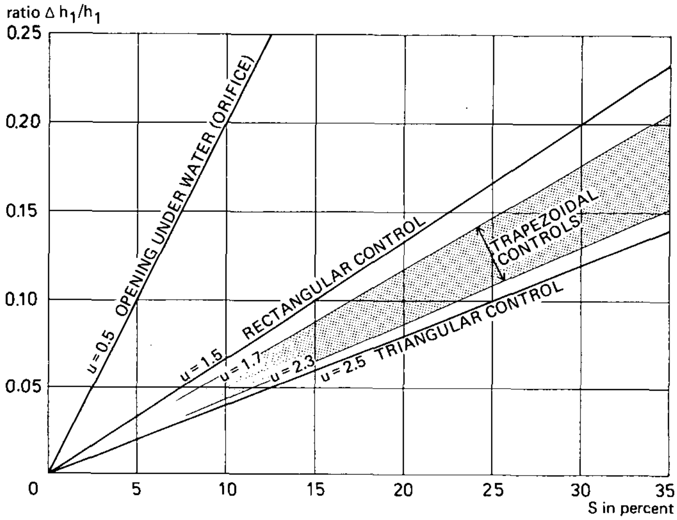


Figure 2.12 Sensitivity of critical-flow devices as a function of relative change in head and shape of control section.

the sensitivities of the structures on each branch. If we imagine a bifurcation at which a flow change of ΔQ arrives, the discharge at the bifurcation increases to $Q+\Delta Q$ and the water level at station "Up" in Figure 2.13 also increases. This will cause increases of the flow to each branch, ΔQ_s and ΔQ_o, where the subscripts s and o refer to the structure in the continuing supply canal and the offtake canal, respectively. To describe the relative division of ΔQ into ΔQ_s and ΔQ_o, the flexibility, F, is used. F is defined as the ratio of the relative flow change in the offtake canal divided by the relative flow change in the continuing supply canal, expressed mathematically as

$$F = \left(\frac{\Delta Q_o}{Q_o}\right) \Big/ \left(\frac{\Delta Q_s}{Q_s}\right) \tag{2.16}$$

where Q_o and Q_s are the original flow rates. Referring back to Equation 2.15, the flexibility can be expressed as the ratio of the sensitivities

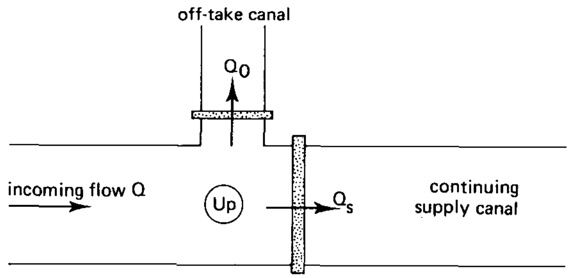


Figure 2.13 Definition sketch for an irrigation canal bifurcation.

$$F = \frac{S_o}{S_s} = \frac{u_o h_{1,s}}{u_s h_{1,o}} \qquad 2.17$$

Some general observations can be made about the division of water at a canal bifurcation for structures having flexibilities less than 1.0, equal to 1.0, or greater than 1.0.

2.10.1 Flexibility = 1.0

When $F = 1.0$, the flow change will be divided in proportion to the original flow rates, Q_o and Q_s. Although this seems a straightforward issue to deal with, there can be some unanticipated consequences when dealing with multiple bifurcations on a single supply canal. Figure 2.14a illustrates what happens to the flow rates if a ditchrider, wishing to transfer a flow of 100 liters/s to a downstream bifurcation, closes offtake 1 and opens offtake 3 without adjusting the intermediate offtake 2. Because the incoming flow rate at bifurcation 2 increases, the flows $Q_{o,2}$ and $Q_{s,2}$ both increase in proportion to their original values. An additional 11 liters/s is diverted into offtake 2, the desired additional 100 liters/s is delivered to offtake 3, and the remaining flow in the supply canal is unintentionally reduced by 11 liters/s. The reverse happens if offtake 3 is closed and offtake 1 opened to accept the flow originally being delivered to offtake 3 (Figure 2.14b). The incoming flow at bifurcation 2 changes, causing a reduction of flow at offtake 2 and an increase in the flow remaining in the supply canal.

If a bifurcation is to have a flexibility of 1.0 for all incoming flow rates and heads, the structures on the continuing supply canal and the offtake canal must have control sections of the same shape and their sills must be at the same elevation.

2.10.2 Flexibility < 1.0

When $F < 1.0$, the flow change is divided so that ΔQ_o is proportionally small with respect to Q_o, and ΔQ_s is proportionally large with respect to Q_s. Most of ΔQ remains

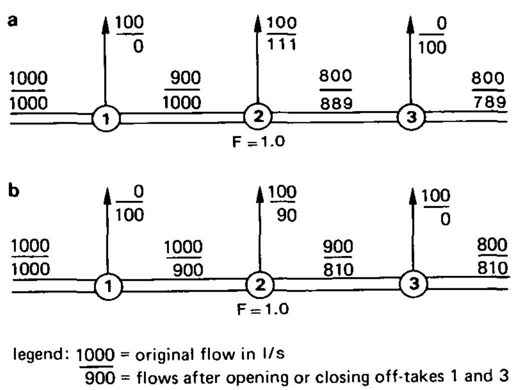


Figure 2.14 Change of water division over canal bifurcations if F = 1.0 at structure 2.

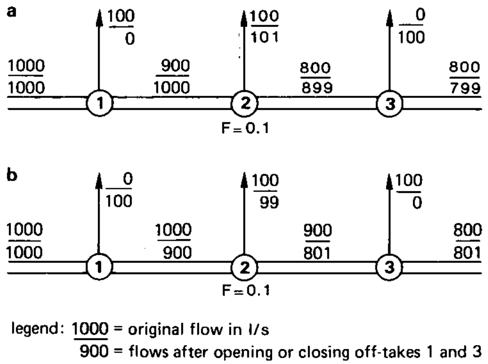


Figure 2.15 Change of water division over canal bifurcations if F = 0.1 at structure 2

in the supply canal as illustrated in Figure 2.15 for a flexibility of $F = 0.1$. Such low flexibilities occur if the upstream sill-referenced head of the offtake is large with respect to that of the weir in the continuing supply canal, or more effectively, if the offtake consists of an underwater opening (orifice) for which the u value is 0.5 (see Figure 2.16). (For details on orifices, refer to Bos 1989). From the example, we see that despite not making any adjustments at offtake 2, the unintended flow changes are slight, and within the accuracy with which flow rates can be measured. Selecting structures at a bifurcation so that the flexibility is low has great advantages for the operation of an irrigation canal system. A downside to low flexibility offtakes is the fact that if the increased flow in Figure 2.15a is not taken out at offtake 3 due to

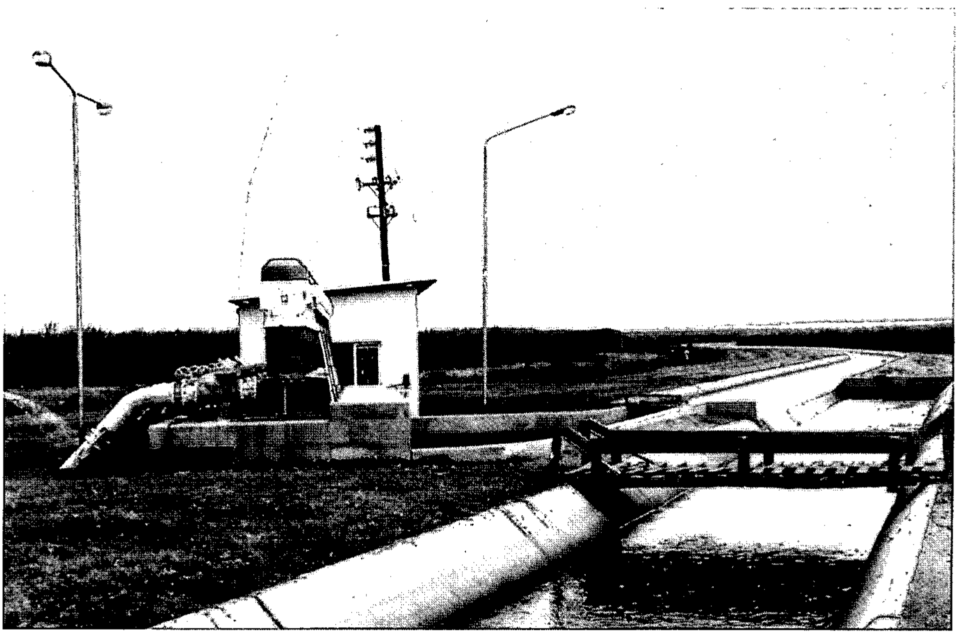


Figure 2.16 Canal bifurcation with low flexibility. Continuing supply spills over wide weir, while offtake Flow, Q_O is pumped into a piped lateral (South Africa).

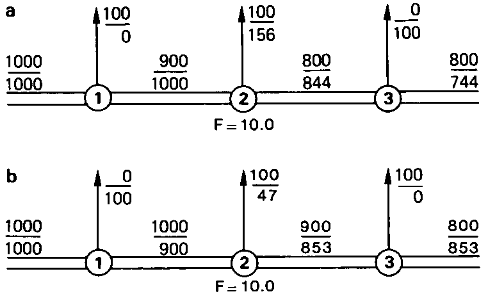


Figure 2.17 Change of water division over canal bifurcations if F = 10 at structure 2.

equipment malfunction, improper operation, etc., the increased flow in the supply canal downstream from offtake 3 can become a large percentage of the original flow rate as the flow moves further downstream. To avoid overtopping of the canal, this excess flow will have to be spilled at a suitable site (see Figure 2.16). If bifurcation 2 had a larger flexibility, some of the excess flow would be diverted into offtake 2, thereby reducing the size of the flow to be spilled from the supply canal.

2.10.3 Flexibility> 1.0

An example of the performance of a system having offtakes with a large flexibility is shown in Figure 2.17. Large unintended flow changes occur in the offtakes, creating shortages or excess flows in the main supply canal. Obviously, irrigation canal bifurcations with large flexibility do not facilitate the uniform supply of water to the various users and thus are not recommended for this purpose. Yet bifurcations with a flexibility of $F > 10$ are very useful if the continuing supply canal must be protected against overtopping. The offtake receives most of the excess flow and discharges it into a surface drain. Likewise, when flow shortages occur, they are felt most heavily by the users receiving water from the high-flexibility offtakes.

Flexibilities of $F > 10$ can occur if the offtake is a weir operating under a low head and if the continuing canal is equipped with an undershot gate (orifice) or a full-flowing culvert. The use of these "side weirs" is recommended upstream from reaches where overtopping of the canal may cause severe damage.

2.11 Selecting a Location

All structures for measuring or regulating the rate of flow should be located in a channel reach where the flow approaching the structure is uniform, an accurate value of h_1 can be measured, and sufficient head loss can be created to obtain modular flow conditions needed for a unique Q versus h_1 relation. The head loss requirement was considered in Section 2.2. Here we will summarize the requirements for the upstream channel that ensure good approach flow and a stable upstream water surface, and then several other important considerations related to the structure site and the channel reach upstream and downstream from it.

2.11.1 Approach length requirements for the upstream channel

To ensure suitably uniform approach flow conditions and a stable upstream water level that can be accurately measured, the channel upstream from the flow measurement structure should satisfy the following requirements, evaluated at the maximum design flow:

- The Froude number should not exceed 0.5 at the gaging station or for a distance of 30 times H_{1max} upstream from the gaging station. If feasible, better measurements can be obtained if the Froude number is limited to a value no greater than 0.2. For channels with high sediment loads, the Froude number should be kept relatively high. The Froude number is calculated by use of the equation

$$Fr_1 = \frac{v_1}{\sqrt{\frac{gA_1}{B_1}}} \qquad 2.18$$

  where v_1 is the average flow velocity, g is the acceleration of gravity, A_1 is the cross-sectional area perpendicular to the flow, and B_1 is the top-width of the water surface.
- The upstream channel should be straight and uniform for a distance of at least 30 times H_{1max} upstream from the gaging station.
- There should be no flow of highly turbulent water (e.g., undershot gates, drop structures, hydraulic jumps) into the upstream channel for a distance of 30 times H_{1max} upstream from the gaging station.
- If there is a bend close to the structure (closer than 30 times H_{1max}), the water surface elevations at the two sides of the structure will be different. Reasonably accurate measurements can be made (added error about 3%) if the upstream straight channel has a length equal to at least six times H_{1max}. In this case, the water level should be measured at the inner bend of the channel.
- To ensure accurate head measurement, there should be no offsets or sudden changes in sidewall alignment within a distance of H_{1max} upstream from the gaging station (see Figures 3.28 and 3.29). Such offsets could cause local flow separation that would affect the measurement of h_1.

It is not always possible to fully satisfy the requirements listed above. In situations in which the upstream water level proves to be unsteady or approach flows are found to be significantly non-uniform, baffles or wave suppressors may be used to improve the situation, as shown in Figures 2.18 and 2.19. If baffles are used, the distance from the baffles to the gaging station should be at least 10 times H_{1max}.

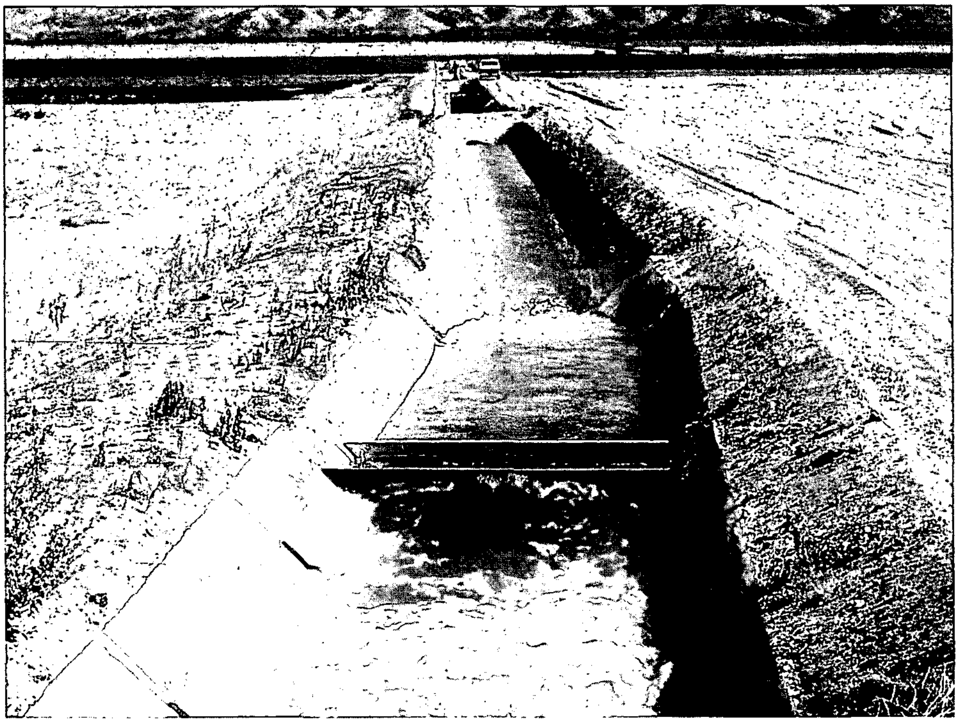


Figure 2.18 A wave suppressor stills the upstream water level to improve the head-measurement accuracy upstream from a broad-crested weir (Arizona).

The requirements related to the length of the upstream channel that are summarized above differ somewhat from the recommendations provided in previous design guides. For channels with so-called efficient sections (e.g., a rectangular channel with depth equal to half its width), the requirement of 30 times $H_{1_{max}}$ produces a similar approach length requirement as the previous recommendation of ten times the average channel width. For channels that deviate significantly from the efficient-section assumption, an approach length based upon $H_{1_{max}}$ should produce a more appropriate recommendation than one based on channel width.

2.11.2 Additional site considerations

The survey of a channel to find a suitable location for a structure should also provide information on the following relevant factors that affect the performance of the structure and determine its effect on the channel and nearby structures.

- The channel reach should have a stable bottom elevation. In some channel reaches, sedimentation occurs in dry seasons or low-flow periods. These sediments may be eroded again during the wet season. Such sedimentation changes the approach velocity toward the structure or may even bury the structure, while the erosion may undercut the foundation of the structure.

Figure 2.19 The Froude number in the approach channel to the weir was reduced to 0.5 by flattening the hydraulic gradient. A fixed division board is used to divert flow for irrigation (Argentina).

- All factors affecting the water level on the downstream side of the future structure must be identified. The water level may be controlled only by friction in the downstream channel, or it may also be affected by downstream confluences with other channels, operation of gates, reservoir operation, etc. The channel water levels greatly influence the contraction (i.e., sill height and width) needed to obtain modular flow.
- Based on the channel water levels and the required sill height in combination with the Q versus h_1 relation of the structure, the possible inundation of upstream surroundings should be studied. Such inundation can cause sedimentation or affect the operation of upstream structures.
- Leakage around and beneath the structure due to the head differential created across it must be cut off at reasonable costs. Also, a stable foundation resistant to significant settling must be secured.
- To avoid sedimentation upstream from the structure, the h_1 versus Q curve upstream from the structure should be matched to the y_1 versus Q curve of the existing channel, as discussed in Section 2.6.3.

2.12 Selecting a Measuring Structure

Although it is possible to select a suitable discharge measurement structure or a flow-rate regulator without this section, an engineer may need some assistance in selecting the most appropriate one. There are several different stages in the process of selection:

- Describe the measurement site and list all demands. The designer needs to list the hydraulic characteristics of the site and describe the environment in which the structure should function. The form given in Figure 2.20 can be used in this process.
- Read Chapters 2 and 3 to evaluate the various types of structures that can measure the range of flows. At this stage, the designer should decide on the type of structure: portable, temporary, permanent, with movable crest, etc.
- Read Chapter 5 on the design of structure. For common canal sizes, you may select a standard flume. Otherwise, WinFlume (Chapter 8) may be used to design a structure.
- Answer the question: "Is the structure acceptable?" Here the designer compares the hydraulic and structural properties of the selected structure to the actual demands. The preceding sections of this chapter should be referred to when making this comparison. See Chapter 5 for additional details on design and selection of long-throated measuring flumes.

NAME OF SITE:______________________________ DATE:__________

HYDRAULIC DEMANDS:

Range of flow to be measured, Q	Present water depth in channel, y_2	Minimum permitted error in measurement, X_Q
Q_{min} = m^3/s Q_{max} = m^3/s	y_{2min} = m y_{2max} = m	X_{Qmin} = % X_{Qmax} = %

HYDRAULIC DESCRIPTION:

Channel bottom with	b_1 =	m
Channel side slope	z =	m
Channel depth	d =	m
Maximum allowable water depth	y_{1max} =	m
Manning's n	n =	
Hydraulic gradient	S_f =	
Available drop in water surface at site	Δh =	m
Drop in channel bottom at site	Δp =	m

Sketch of channel cross section:

FUNCTION OF STRUCTURE

Measurement only	☐
Regulation and measurement of flow rate	☐

Concrete lined: ☐

Earthen channel: ☐

Profile along bottom of channel over length of $100b_1$

PERIOD OF STRUCTURE SERVICE

Day	☐	Season	☐
Month	☐	Permanent	☐

DESCRIPTION OF ENVIRONMENT

Irrigation System		Drainage System	
Main	☐	From irrigated area	☐
Lateral	☐	Artificial drain	☐
Farm ditch	☐	Natural drain	☐
In field	☐		

Plan of site:

FURTER DESCRIPTION
(attach photo)

Figure 2.20 Form for recording information about a potential flow measurement site.

3. Flume and Weir Types and Construction Methods

There are a wide variety of flow measurement methods and devices available for any given channel where water measurement is needed. The intent here is to assist the reader with the selection of a device that is relatively easy to construct, inexpensive, and suitably accurate. In this chapter, we give the reader some ideas on types of structures that can be built in a wide variety of situations. This chapter deals primarily with construction and installation methods. Later chapters deal with design and calibration, although they are often related. For example, a structure's ability to accurately measure flow (i.e., design) will often influence which structure can be selected, and thus how simple the construction can be.

3.1 Introduction

As discussed in Section 1.4, long-throated flumes and broad-crested weirs include five primary structural components:

1. An approach channel of sufficient length (from the gaging station to the converging section) to keep the gaging station upstream from the drawdown associated with critical flow. The approach channel can be either earthen or lined and its cross section can have any shape. Both lining material and cross-section shape may differ from the tailwater channel.
2. A converging transition section in which the sub-critical approach flow accelerates smoothly toward the throat. Depending on the construction material of the structure, the transition may consist of plane surfaces or may be rounded. The sole purpose of this transition is to avoid flow separation at the upstream edge of the throat of the structure.
3. A throat, or control section, in which the flow passes through critical depth. The throat must be horizontal in the direction of flow over a length, L, so that

   $$0.07 \leq H_1/L \leq 0.7 \qquad 3.1$$

   where H_1 is the energy head in the approach channel, relative to the throat invert. In the direction perpendicular to the flow, any shape may be used, as long as it can be mathematically described. The software of Chapter 8 allows for fourteen possible shapes.
4. A diverging transition in which the velocity of the supercritical flow exiting the throat section is reduced and energy is either dissipated (Chapter 7) or partially recovered. If energy recovery is not needed, an abrupt transition can be used in most cases. Otherwise a 6-to-1 sloping transition is added. The ramp can be truncated to limit the length of the structure (see Section 2.2, Figure 2.4).
5. A tailwater channel, which either is earthen or lined. The channel may have any shape and size, and may be different from the approach channel. The range of

water levels in this channel is fundamentally important to the design of the structure. It determines the elevation and size of the control section needed to maintain critical flow conditions in the flume throat throughout the range of flows to be measured (see Sections 2.5 and 5.3.2).

Using the design considerations of Chapter 2, the designer has considerable freedom to decide on the shape and construction material of each of the above 5 parts of the structure. Starting with the same channel conditions, two different designers are likely to select two different flumes. If both flumes meet all design criteria, however, both are effective. This chapter gives examples of possible structures. For the general design process and hydraulic theory of long-throated flumes, refer to Chapters 5 and 6.

Although the measurement of water seems to be logical in regions where water is scarce, there is regular opposition to the improved control and management of water. Part of this resistance is due to the anticipated reduction in water supply to users who improve water management. Other opposition has a psychological background. This particularly plays a role for structures in earthen canals with low flow velocities. The bottom contraction needed to accelerate the flow to critical velocity is considerable. The resulting "wall" across the bottom of the channel tends to frighten water users who have not had hydraulic training, as they often believe it will reduce their flow of water. A contraction from the side (a flume) tends to create less opposition than a contraction from the bottom only (a weir).

For water resource management in environmentally sensitive areas, the flume can be camouflaged such that the structure becomes nearly invisible. Figure 3.1 shows a brown and green painted flume made of marine quality plywood. A weir sill hidden in a road culvert also is suitable for water management in scenic areas.

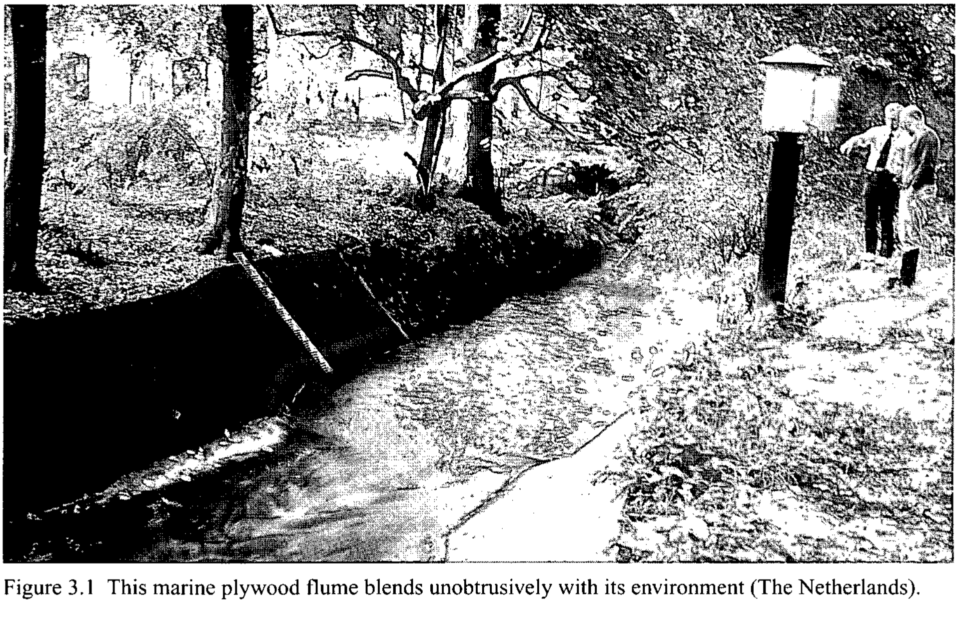


Figure 3.1 This marine plywood flume blends unobtrusively with its environment (The Netherlands).

Some water managers question the accuracy with which broad-crested weirs and long-throated flumes can measure flow. Following construction, a current-meter discharge measurement is often used to "calibrate" the flume. However, the error (random and systematic components) in the measurement with a well-maintained current-meter is about ±5%, provided that the channel is stable and standard practices are followed. The error often exceeds ±8%. The current-meter discharge-measurement errors are significantly greater than the ±2% systematic error in the flume rating. Thus, only a large number of current meter measurements (more than 15) would provide, on average, flow estimates that are as accurate as the basic flume rating. The software described in Chapter 8 offers a means for comparing measured h_1 vs. Q data against the calculated flume rating. If the difference is significant, both the current-meter discharge measurement and the flume should be checked for sources of error (see Section 2.8 and Wahlin et al. 1997).

When introducing flow measurement in an environment where water users are not familiar with weirs and flumes, the use of a portable flume or temporary flume is recommended to demonstrate the effect of the structure on the water surface elevation upstream from the structure (backwater) and on the downstream discharge. A number of alternatives are available:

- Portable RBC Flumes for flows up to 50 liters/s (1.75 ft^3/s) (Section 3.3.3),
- Portable Adjust-A-Flumes for flows up to 180 liters/s (6.4 ft^3/s) (Section 3.3.3),
- Large Adjust-A-Flumes for flows up to 2.5 m^3/s (88 ft^3/s) (Sections 3.3.3 and 3.5.1), and
- Flumes of sheet material in concrete lined canals for flows up to about 3.0 m^3/s (106 ft^3/s) (Section 3.2.3).

In irrigated areas the flow in canals is often divided over two or more branches of the canal system. Two groups of broad-crested weirs are available for the measured division of flow:

- **Weirs with a vertically movable crest** — On such a structure, the weir crest can be lowered or raised with respect to the water level of the main or lateral irrigation canal so that the diverted flow through the offtake can be measured and regulated. Movable weirs also can be placed in the continuing supply (main or lateral) canal. Besides the two above-mentioned functions, a movable weir can be used to also check the upstream water level. Flow over movable-crest weirs is limited by mechanical considerations, since the movable crest must structurally support the weight of the water flowing over it. For example, the flow over a relatively large 4.0 m wide weir with 1.0 m crest length is limited to 6.0 m^3/s. To obtain higher discharges, multiple side-by-side weirs may be used.

  The accuracy of water delivery through key structures can be further improved by adjusting the structure with a hydraulic cylinder or an electric motor drive. If these drives are automatically activated based on feedback from a head measurement device (see Section 4.4), the upstream sill-referenced head, and thus the flow rate over the weir, can be held constant.

- **Weirs with a partition board** — The flow divisor consists essentially of a broad-crested weir with rectangular control section and a partition board downstream from the crest. The partition board may be fixed or movable depending on the need to change the division of flow over the canal branches.

The advantages of combining the measuring and regulation functions into one structure are: (1) less hydraulic head is needed; (2) a multipurpose structure usually is cheaper to construct than the two separate structures (one for regulation and one for measurement) it replaces; and (3) the operation of one structure is less time-consuming. Owing to the latter, gatemen or ditchriders tend to distribute irrigation water more accurately, and a higher efficiency of irrigation water use can be attained.

3.2 Structures in Small Lined Canals

In the past, flow measurements with flumes and weirs were costly, of variable accuracy, and difficult to apply in field situations. Major problems included the requirement of reshaping the canals to accommodate a limited assortment of calibrated devices, as is the case for Parshall flumes, and the requirement for relatively large water surface drops associated with sharp-crested weirs. Not the least of these problems was the general inability to control installation errors and readout errors for an accurate, convenient, and reliable head reading. Many of these problems are significantly reduced by the use of long-throated flumes and the hydraulically related broad-crested weirs provided in this book (Figure 3.2).

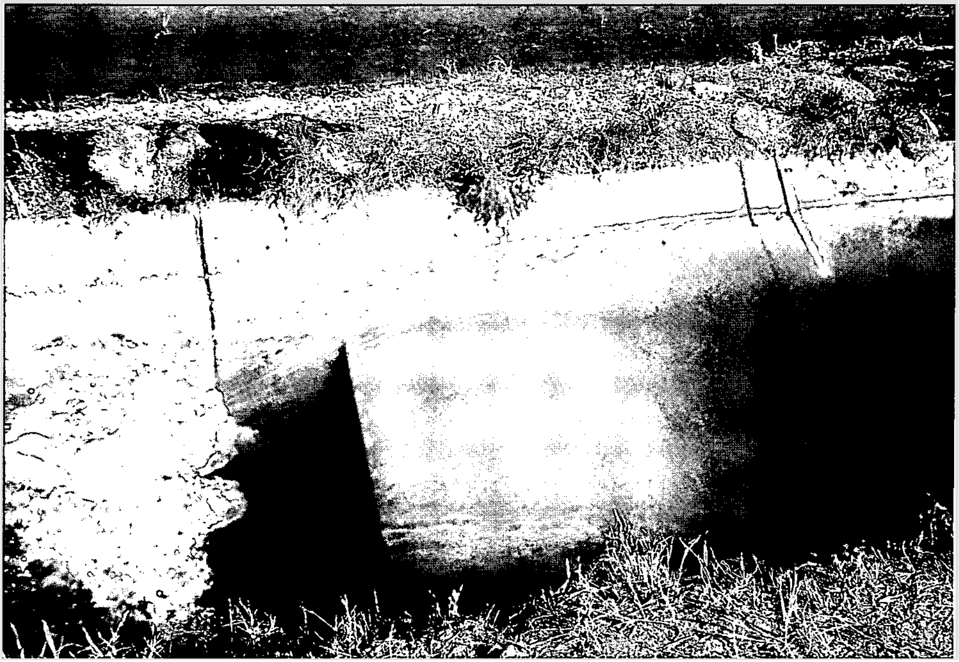


Figure 3.2 Broad-crested weir operating at design discharge (Arizona).

The construction of a small broad-crested weir in a concrete-lined (irrigation) canal is relatively easy and straightforward. The existing canal lining can serve as: 1) approach channel; 2) part of the converging transition, throat, and the diverging transition; and 3) tailwater channel. If no energy head needs to be recovered, an abrupt diverging transition is sufficient, and the weir has only two sections that need to be constructed: a throat section with a length L, and a converging transition with a length equal to three times the sill height, or $3p_1$ (Figure 3.3).

As was shown in Figure 1.3, the contraction of the canal can be from the sides, from the bottom, or both. Most lined canals are irrigation canals with a ratio of maximum to minimum flows, Q_{max}/Q_{min}, below 5. Since broad-crested weirs (bottom contractions) can satisfy this range of flows and are generally much easier and less costly to construct, they are recommended for this application. Design procedures, standard sizes (defined in terms of crest width, b_c), and rating tables are discussed in Section 5.5.1.

The constructed crest width, b_c, should be as close to the values in the rating tables as the design accuracy of the measurement, because a 1% error in b_c will produce about a 1% error in discharge (see Table 2.5). Although the sill height p_1 is important in selecting the weir because it controls the water surface elevation and thus the modular limit and freeboard relationships, its precise vertical dimension is not at all critical to the weir head-discharge relationship. For example, p_1 can vary by ±10% before causing noticeable calibration changes. Also, the crest length L can be adjusted ±10% without significant effect. The ramp length of $3p_1$ is also approximate and its purpose is to convey the water smoothly to the weir crest. Thus the ramp may be straight or

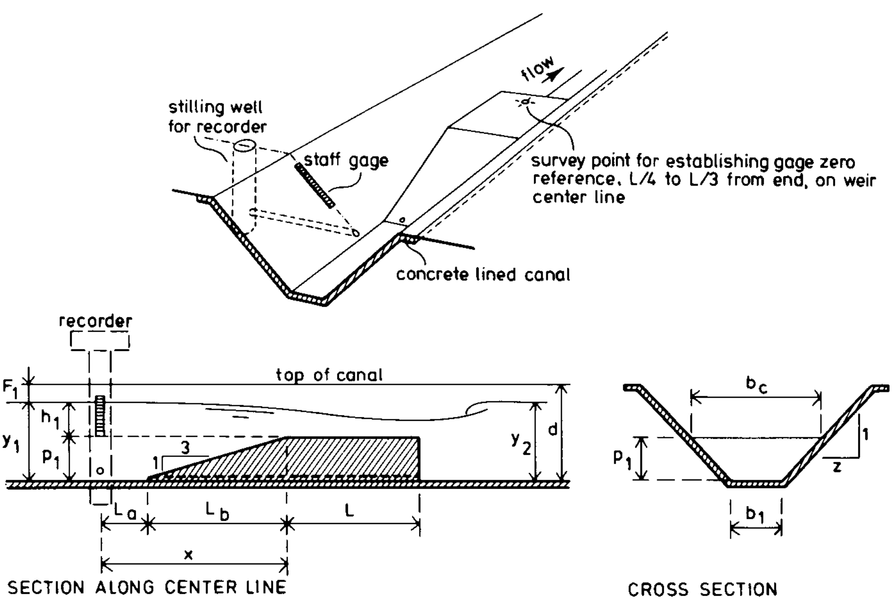


Figure 3.3 Broad-crested weir in a concrete lined canal.

gently curved, but straight is usually more readily constructed. These liberal tolerances should not be an excuse for sloppy or poor construction, but they can make construction quick and easy. More accurate rating tables can be developed by measuring the exact structure dimensions and using the computer program of Chapter 8.

3.2.1 Cast-in-Place Structures

Trapezoidal channels

Following site selection and the design of a structure, cast-in-place construction is straightforward. If the canal has construction joints, the structure should be placed so that the gage location is at least 0.5 m downstream from the joint. If that is difficult, locate the gage slightly upstream from the joint and position the sill at distance $(L_a + L_b)$ downstream as listed in the rating tables and shown in Figure 3.4. If possible, construction joints in the area of the sill should be avoided. Construction joints occurring in the ramp section are acceptable. If a construction joint is located between the weir sill and gage, care should be exercised to ensure that no vertical movement occurs. Otherwise, the zero-reference (sill-reference) of the gage may not remain reliable.

The suggested construction sequence is

- In a workshop, make two forms from galvanized angle iron to the cross sectional shape of the control section, with the edge on one angle (not the flat side) making the shape. These edges will be used as the forms for screeding and troweling the concrete. For a flume with bottom plus side contraction, these forms shape the bottom and sides of the control. For weirs with bottom contraction only, the top of the angle irons have a length of b_c since the sides of the canal function as sides of the control.

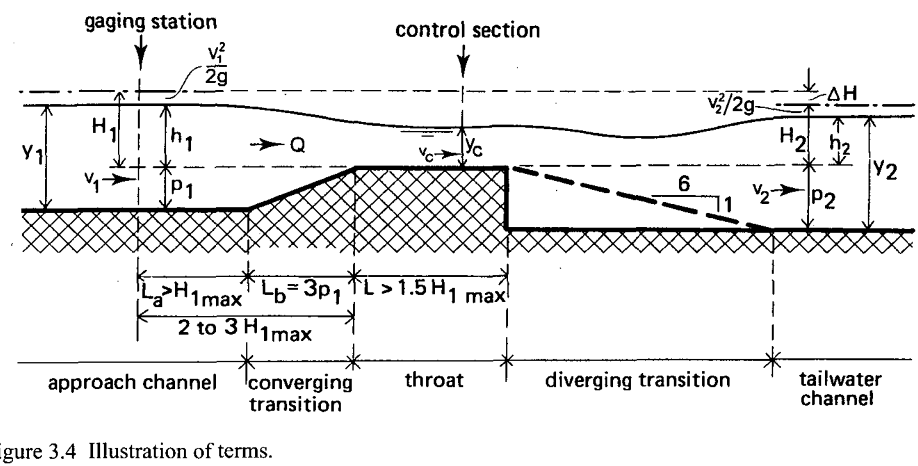


Figure 3.4 Illustration of terms.

- Attach the two forms together by bolting or welding spacers such that the outside of the screed edges are a distance *L* (the crest length) apart. Other methods can be used to hold the forms, such as the steel-rod spacers commonly used for retaining-wall construction.
- Cut sections of 4-mm (3/16 in.) welded wire fabric with 100-mm (4 in.) center-to-center spacing in the shape and size of the converging transition. Please note that this reinforcement must have a minimum concrete cover of 25 mm (1 in.).
- Cut a length of drain pipe long enough to allow water to pass by the structure, once poured. (see also Section 3.7).
- At the selected site, place the drain tube anywhere along the bottom, not necessarily at the centerline.
- The maximum required thickness of sill and ramp is 0.10 m (4 in.). The remaining volume can be formed of compacted soil. If the structure has no downstream ramp, the compacted soil should be longer than the crest.
- Install the angle iron forms and wire fabric (Figure 3.5). Both should be fixed in position so that they cannot move during placement of the concrete (Figure 3.6). The forms should be checked with a carpenter's level to ensure that the top edges are level in the cross-canal direction and with respect to each other.
- Next, fill the forms with a concrete mix that has been shown to be suitable for local waterborne chemicals and weathering conditions. The crest and ramp(s) can be poured at the same time. Use the tops of the metal forms as guides for screeding and troweling the sill to a level surface. The metal frame is left permanently in the concrete. Thus for small weirs, both the sill and the ramp can be constructed at the same time without waiting or returning to the site. Both, the ramp and the sill may be broom-finished. The ramp need not taper to zero thickness, but can be ended abruptly when it becomes about 0.05 m (2 in.) thick. (Figure 3.7)

Figure 3.5 The edges of the control section of the flume are formed by steel strips that stay in the concrete. Wire fabric is used as reinforcement of the ramp.

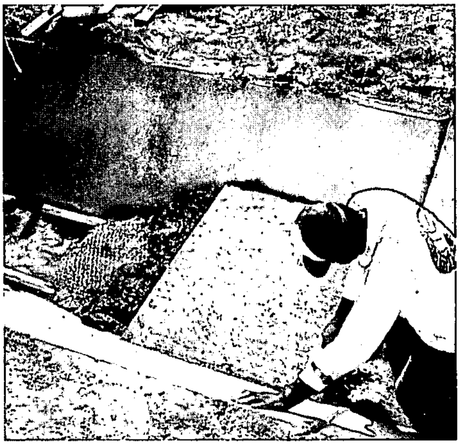


Figure 3.6 The first concrete for the crest has been poured.

Figure 3.7 The ramp can be hand-troweled into place.

- With a side-contracted flume, the sides of the control section and transitions cannot be poured on the same day as the bottom sill and ramps (unless special, weighted forms are constructed). The concrete in the ramp bottoms and sill has to set sufficiently (about 24 hours, depending on local conditions), before the sides can be hand-troweled into place.
- Finally, the wall gage should be placed as described in Section 4.9.

Although these structures can be placed as solid concrete, placing a shell, as described above, puts less stress on the lining material and foundation and allows the structure to be removed without great difficulty, if necessary.

When the flow velocity in a lined canal is very low, the simple bottom sills are not appropriate since the flow depth over the weir would be too shallow to measure accurately. For this situation, a rectangular weir/flume can be constructed within the trapezoidal canal, as shown in Figure 3.8 (see also Figure 1.10). This flume can be constructed by first pouring a sill and ramp as described above. Then, the vertical walls of the throat are constructed of brickwork or any suitable material. The vertical wall of the converging transition need not be truly vertical but may gradually slope back toward the canal wall.

Rectangular channels and box culverts

Construction of a broad-crested weir in a rectangular channel is similar to construction in a trapezoidal channel. Because the crest does not have the sloping walls to rest on, the crest must be supported by anchoring to the walls or supported from the floor with structural members. Also, since the ramp is not forced down onto the sidewall by the flowing water as in a trapezoidal channel, there is less to stop the

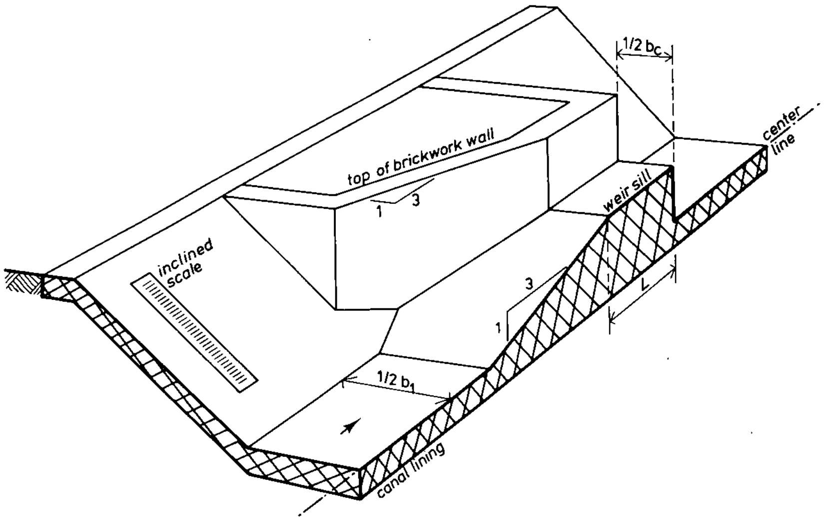


Figure 3.8 Structure with rectangular control section in lined trapezoidal canal.

ramp and crest from being forced downstream. Thus we recommend that the structure be firmly anchored to the floor or the walls. Otherwise, the construction sequence is the same as for the trapezoidal channels.

In some cases, it may be desirable to narrow the structure. For trapezoidal channels, we recommend constructing the sill and then narrowing the crest. For rectangular structures, it is easier to narrow the structure with block or brickwork, and then add the sill, if needed.

Culverts are an attractive site to construct a measuring flume in otherwise unlined channels, since the foundation for the structure is already in place. If the flume or weir is placed near the outlet of the culvert, additional scour protection may be needed (see Chapter 7).

Other channel shapes

There are a variety of other lined channel cross-section shapes, including semi-circular, U-shaped, circular (e.g., culverts), parabolic, triangular, etc. Where sloping sidewalls exist, construction procedures are essentially the same as for trapezoidal channels. If the sidewalls are vertical, then the construction considerations for rectangular channels are appropriate. The main objective is to build a sill that is level in both directions and has a cross-section that is smooth and uniform throughout the throat.

3.2.2 Prefabricated concrete structures

In principle, all of the cast-in-place concrete weirs of Section 3.2.1 can be precast as reinforced concrete beams and slabs. The locally available equipment for transporting

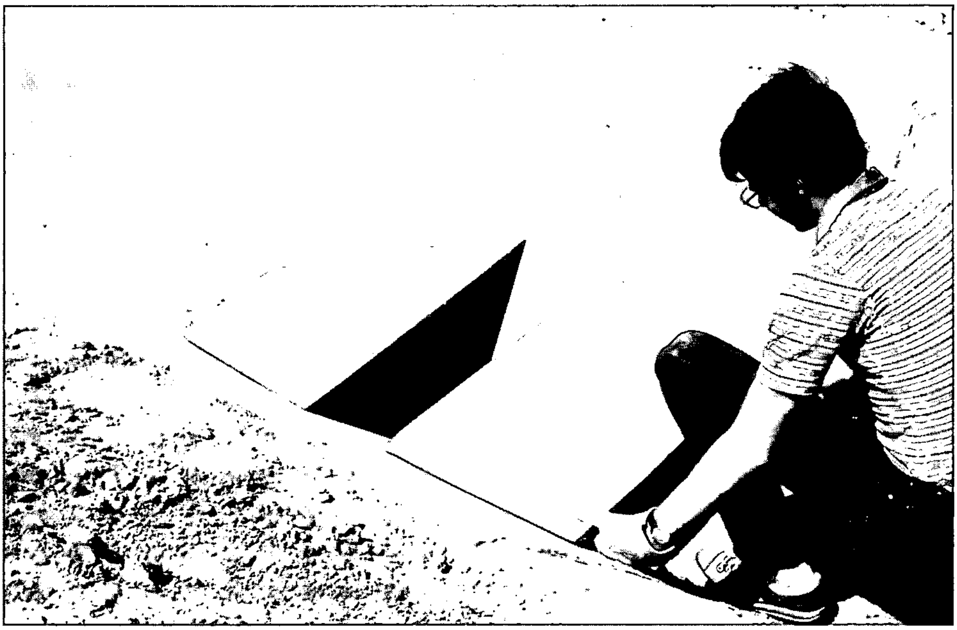


Figure 3.9 Small precast weir being placed in lined canal.

and placing the parts would generally control the sizes that could be precast and the particular number of pieces into which the weir would need to be divided. For example, the weir shown in Figure 3.9 was successfully precast in two parts: the sill and the ramp. The parts each weighed about 45 kg (100 lb) and could be handled by one or two persons without any special hoists or equipment. The two pieces were made in special forms constructed of galvanized sheet steel (Figure 3.10). The slopes and dimensions were calculated carefully and laid out so that the finished parts would fit well in the canal of interest. The effort necessary to construct these forms is probably not justified except for large-scale production. To conserve concrete and maintain manageable weight, inserts in the forms were made so that while the edges were all basically 100-mm (4 in.) thick, the center portions of the sill top and ramp were only about 35 mm (1.5 in.) thick. Reinforcing wire mesh and reinforcing bars were incorporated into the design.

Another larger size was precast for use in an irrigation district in La Paz County, Arizona. In this case, the size and weight of the pieces required mechanical equipment for handling since the sills were 1.22 m (4 ft) wide and 0.76 m (2.5 ft) long and about 60 mm (2.5 inches) thick with an extra 150-mm (6-in.) thick beam along the long dimension to maintain rigidity across the channel when installed in the canal. An indentation was cast into the top of the upstream beam in order to accept the ramp section and to help support it. The ramp was cast as a 60-mm (2.5-in.) thick flat slab with the edges shaped to match the sill edge and canal sidewalls but with no structural beam along any edge. The narrow bottom end of the ramp was not tapered to match the canal floor since the 3:1 slope would have resulted in a thin and fragile portion. Rather, it was simply finished at a right angle to the slope. The resulting discontinuity

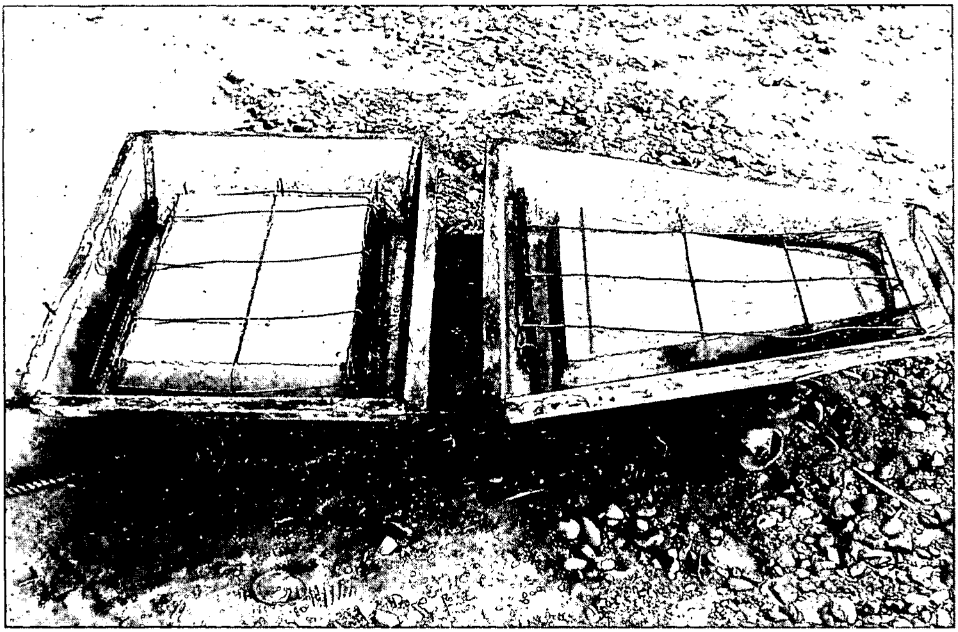


Figure 3.10 Galvanized metal forms for pouring precast weir sill and upstream ramp.

is not significant to the function of the measuring weir. Cement and sand mortar were used to caulk irregularities and aid in securing the sill and ramp into position (Figure 3.11).

A drain hole, replacing the drain pipe described in Section 3.2.1 and later in Section 3.7, was cast into the bottom edge of the ramp section. For handling purposes, eye bolts were cast into the sections and were cut off after placement in the canal. The pieces weighed about 145 kg (300 lb) each. No special precautions were necessary to secure the pieces against water pressure. The sections have, on occasion, been removed from the canal and relocated, but they are not easily portable. Figure 3.12 shows the stock of parts for these weirs.

3.2.3 Temporary structures

Temporary structures are those that are constructed off-site, so that they can be assembled at the desired site, used for a short period of time, disassembled, and moved to another site or simply discarded. These are typically lighter weight than the precast concrete structures, but not so easily moved that they would be considered portable. These have been constructed in sizes ranging from 100 liters/s to 6 m^3/s (3.5 to 212 ft^3/s). The flume and weir surfaces are typically made from marine plywood or sheet metal. The structural members supporting these surfaces can be made from angle iron or lumber. These structures are usually in place for at least one irrigation season and include wall-mounted gages. Occasionally they are used for single irrigation events and flow-survey work, with head-detection from portable devices

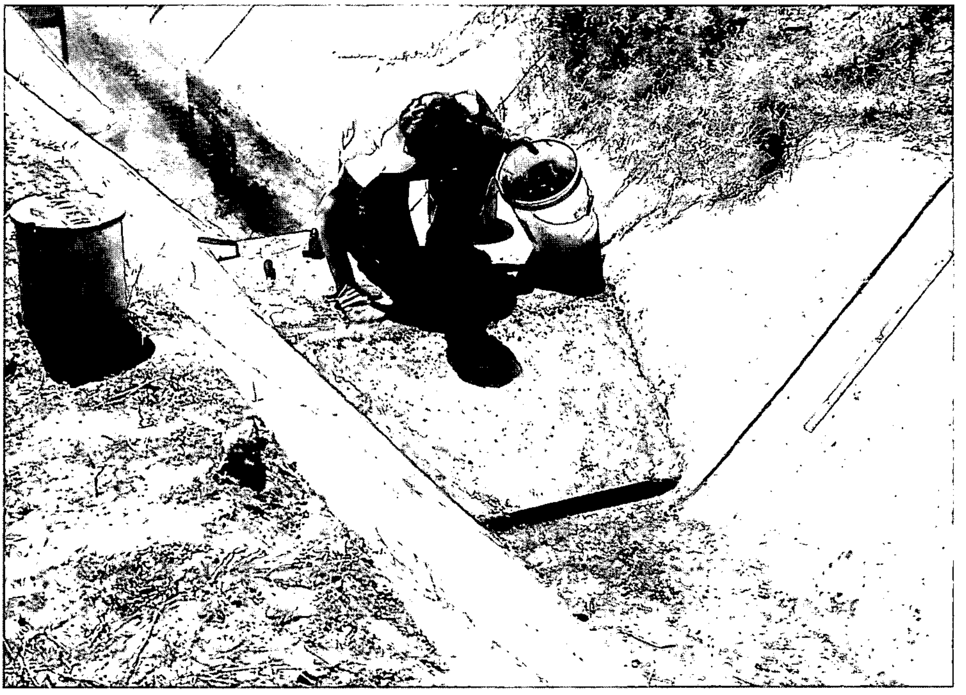


Figure 3.11 Edges of the precast weir sections are grouted with cement and sand mortar (Arizona).

Figure 3.12 Stock of precast weir sills with 1.22 m (4 ft) wide crest. Note the indentation in the top of the upstream beam that helps to support the ramp section.

(see Section 3.2.4). Temporary and portable structures are often useful for determining if a particular flume will function adequately for a given canal. When insufficient field data are available, the normal design process can be avoided by testing several temporary structures until one is found that works adequately. If the temporary structure works, a permanent structure of that size can then be constructed.

The simplest temporary structures for lined canals can be constructed from marine plywood and construction lumber. Marine plywood is recommended since it can withstand submergence for long periods of time without swelling. It is recommended to coat the plywood with a sealer and marine anti-fouling paint for longer durability. For flow-survey work, less expensive grades of plywood have been used successfully (Figure 3.13). The sheet for the weir crest should be beveled on the sides to approximately match the canal side slopes. The sheet or sheets for the ramp also need to be beveled for the canal sidewalls. This angle is slightly less than the sidewall angle since the ramp is sloping. The end of the ramp also needs to be angled where it meets the crest so that there is not a gap between the ramp and the crest and so that the ramp does not protrude above the crest. The ramp can be shortened so that it does not touch the canal bottom. A 10 or 20-mm gap can easily be plugged with soil.

When the ramp is made of heavy plywood (e.g., 15 to 20-mm thick) and is narrower than 1.5 m, a support structure under the ramp is usually not needed, particularly since deflection of the ramp is not critical. The crest, however, should always be supported with a structural framework since deflection affects the weir calibration. Lumber, for example 40 mm by 140 mm (2-by-6), can be used to support the crest, where the lumber spans the width of the crest from sidewall to sidewall. The crest

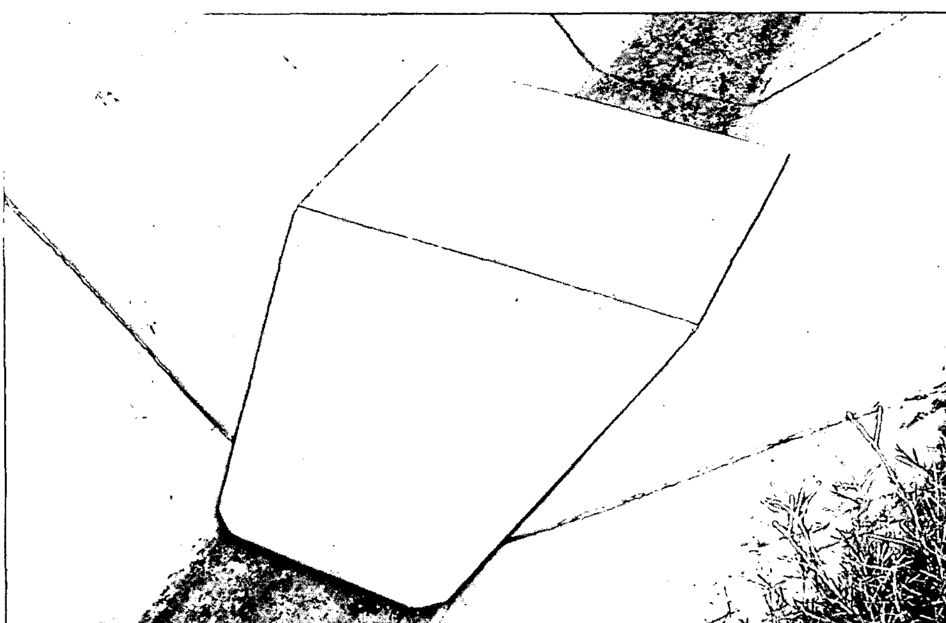


Figure 3.13 A temporary broad-crested weir constructed from commercial-grade plywood.

should be supported at both the upstream end where the ramp is attached and at the downstream end (with or without a ramp). The support on the upstream edge should be placed so that it supports both the crest and the ramp. This requires a slight bevel of the upstream edge to support the ramp. For monitoring over a short time period, the ramp need not be firmly attached to the crest and its support structure. For longer use, the ramp should be attached to the supporting lumber with screws or by other means.

Angle iron is an alternative to lumber for the support structure. Light-weight angle iron (that looks like webbing rather than being solid) is convenient and often sufficiently strong. Marine plywood and light-weight angle iron have been used for structures up to 3 m (10 ft) wide and for flows up to 6 m^3/s (200 ft^3/s).

A number of structures have been made from sheet steel and angle iron. Figure 3.14 shows the basic fabrication of a 1.53 m (5 ft) wide weir. The sill is made of two structural angles attached to the underside of a sheet forming the crest. On sills that are over 0.5 m long (length *L*), three structural angles should be used. Structural angles made from heavy gage sheet (16 gage, or about 1.6-mm thick steel sheet) are

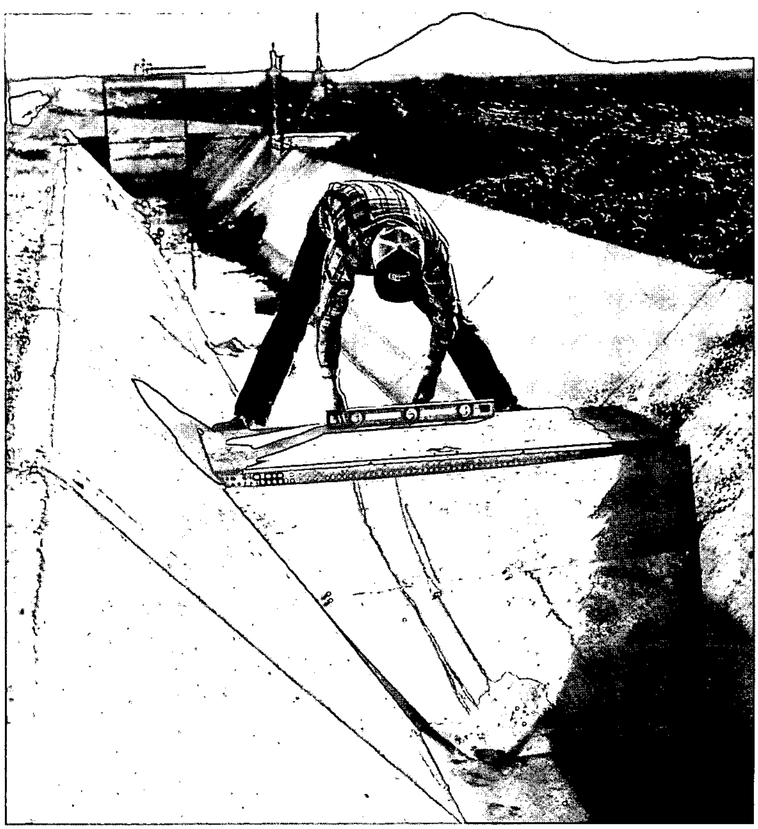


Figure 3.14 The sill of a temporary sheet metal weir should be level. The bends at the corner of the structural angles can be used to secure the sill to the canal wall. After the sill is leveled and secured, the upstream ramp is attached. (Arizona).

suitable for the sizes mentioned. The angles should be about 80 mm (6 in.) on each leg, and have a length equal to the weir width. The downward pointing leg of each angle end is then bent so that it will match the canal side slope *z* and serve as a tab for securing the sill to the canal walls. The ramp is then cut to match the sill width, and a tab is included to secure it to the sill edge. The tab is bent on a 3:1 angle so that the sill and ramp smoothly intersect without overlapping. Like prefabricated concrete weirs, local materials, available tools, labor skills, and type of use will somewhat control whether the weirs are bolted together or spot welded, and painted or otherwise coated for rust protection. The weir shown in Figure 3.14 uses three galvanized, 50 x 76 x 3 mm (2 x 3 x $^{1}/_{8}$ in.) steel angles available commercially. The sill top was 16 gage galvanized steel sheet (about $^{1}/_{6}$ in. or 1.6 mm thick). Also, the two ramp sections are shown with some stiffening members made from the galvanized sheet. Because the edge of the ramp rests on the canal wall and is fixed to the sill beam, very little additional stiffness is needed. In practice, water pressure holds all parts in place and the canal fastenings are needed only to prevent wind damage when the canal is empty and to prevent accidental movement by people servicing the canal or by animals. Simple structural analysis should be made to determine the expected deflection of the sill during use. Figure 3.15 shows the ramp section for a large temporary metal weir being hoisted into place.

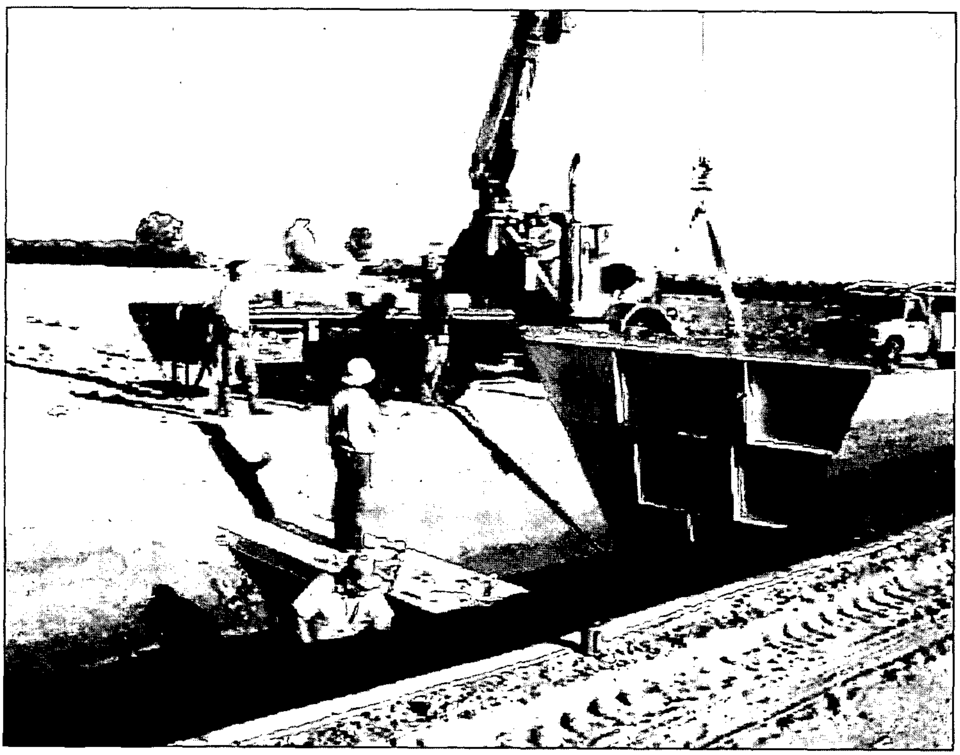


Figure 3.15 The upstream ramp of a 3 m wide (10 ft) weir is hoisted into place after the sill has been installed (Arizona).

These temporary weirs are especially useful when measurements are needed during an irrigation season and the canal cannot be taken out of service for any extended period. They can be installed in a few minutes, and in many cases have been installed with the canal at partial flow. This is somewhat dangerous because of the large hydraulic forces involved. When the pieces of the structure begin to block the flow, they can be sucked down very hard, trapping fingers and hands under them. With flowing water, once the pieces are put down, it may be impossible to move them by hand, except for very small structures. Thus when at all possible, we recommend temporary interruption of the flow for installation. However, if the flow cannot be totally stopped, we recommend the following sequence. Place the sill in the flow at the desired location, level and secure to the canal walls, with lead anchors at all four corners, if possible. This step is often not dangerous since the sill does not significantly block the flow, and may even be above the water surface. Installing the ramp with flowing water is the tricky and dangerous part. If the sill is above the water, the ramp can be attached to the sill above the water, for example by sheet metal screws inserted into predrilled holes. The ramp can then be lowered into the water, the force of which will suck the ramp down into place. Placing the ramp in flowing water when the crest is under water is difficult, and dangerous for all but very small structures. To accomplish this, the ramp must be "floated" into the upstream edge of the crest. The ramp must be kept level to avoid blocking the flow and causing large forces against it. Once the ramp is in contact with the edge of the crest, lower the upstream end slightly and the force of the water will suck it down into position.

Warning
If the ramp is lowered into flowing water, it will be sucked down into position rapidly and with great force. Serious injury is possible to trapped fingers and feet!

Temporary structures made of steel and wood have also been constructed for shapes other than trapezoidal. Figure 3.16 shows a system that can be assembled to make temporary measurements in circular channels. The sill is a rectangular sheet metal section, while the ramp is cut as half of an ellipse to fit the profile of the circular channel bottom. A point gage is used to sense the upstream head. Figure 3.17 illustrates the details of the system and demonstrates how to develop the layout of the ellipse-shaped ramp section.

3.2.4 Portable structures

Trapezoidal channels

Similar to the weirs described in Section 3.2.2 and 3.2.3, the portable weir for use in concrete (slip form) lined canals requires only a weir sill and a converging transition or ramp. The canal provides all other necessary surfaces to a reasonable accuracy. In designing a portable weir, two major demands are to be met:

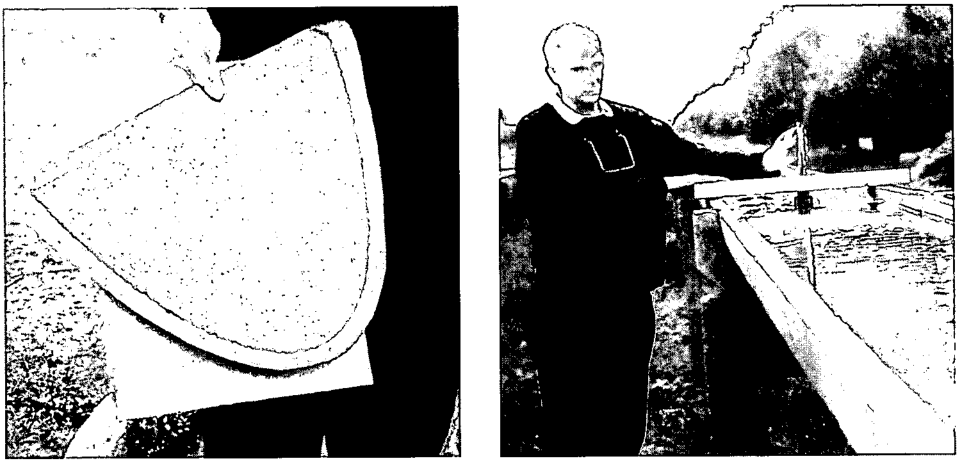


Figure 3.16 This portable system is used for temporary measurements in circular sections. The ramp (left) is half of an ellipse and fits the profile of the channel bottom. A point gage (right) is used to measure the upstream head (Morocco).

- a head-reading device must be attached to the structure that enables the determination of the upstream sill referenced head h_1 without the need for precision surveying after each installation of the weir, and
- the structure should be installable and removable by one person.

A weir for accomplishing the two goals of convenient head determination and portability is shown in Figure 3.18 ("Turtle Flume"). The weir shown was designed for use in a small canal with a 0.30 m (1 ft) bottom width, 1:1 side slopes, and a construction depth of 0.61 m (2.0 ft). Typical discharges for such small canals range from about 0.03 to 0.35 m^3/s (0.1 to 1 ft^3/s). To accommodate this wide range of flows, this portable weir was designed with the largest sill height that would pass 0.35 m^3/s (1 ft^3/s) without overtopping a 0.61 m (2 ft) deep canal and still allow a few centimeters of freeboard. A 0.30 m (1 ft) high weir sill adequately meets these criteria. Once the sill height has been chosen, the criteria for other weir dimensions come from the limitations on the range of H_1/L that can be used (see Sections 2.4 and 6.5.3) and from the method of construction of the weir. To reduce the size of the device, this particular weir uses a steeper converging ramp (2:1) than is typically recommended for permanent structures; the accuracy of the device is still suitable for portable application. If the size or shape of the canal differs from the above, or if the range of flows that commonly have to be measured is lower, the weir shape and size can be changed to fit those conditions. A rating table for this weir is given in Appendix 4, Table R.2, weir C_e. If a rating table is not available that matches the selected sill height p_1, weir crest length L, and cross-sectional shape, a custom-fitted rating table must be produced using the procedures of either Section 6.5 or Chapter 8.

Figure 3.19 shows the features of this portable weir constructed from aluminum angles, pipe, and sheet. Figure 3.20 provides detailed construction drawings. All angles and pipes are connected by welding, and the sheet is fastened with pop rivets.

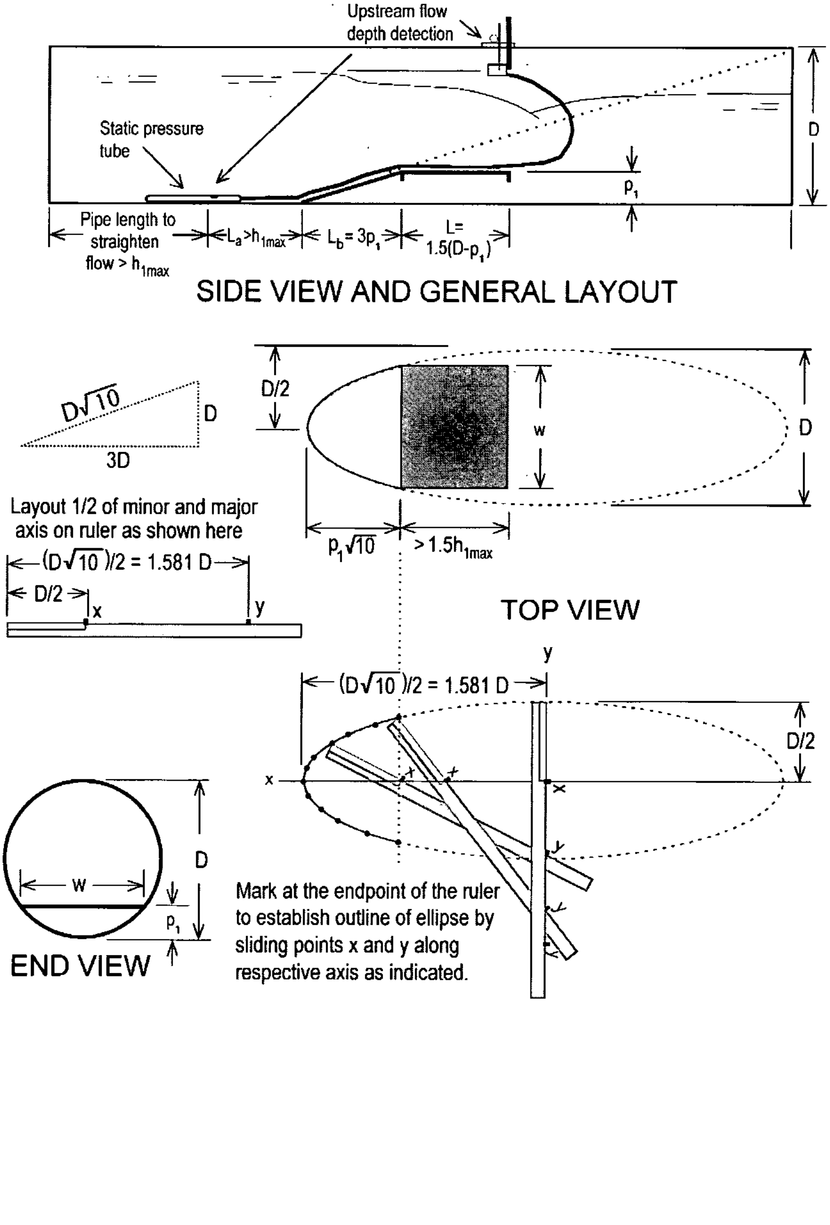


Figure 3.17 Layout scheme for portable long-throated measurement structures in partially full circular conduits.

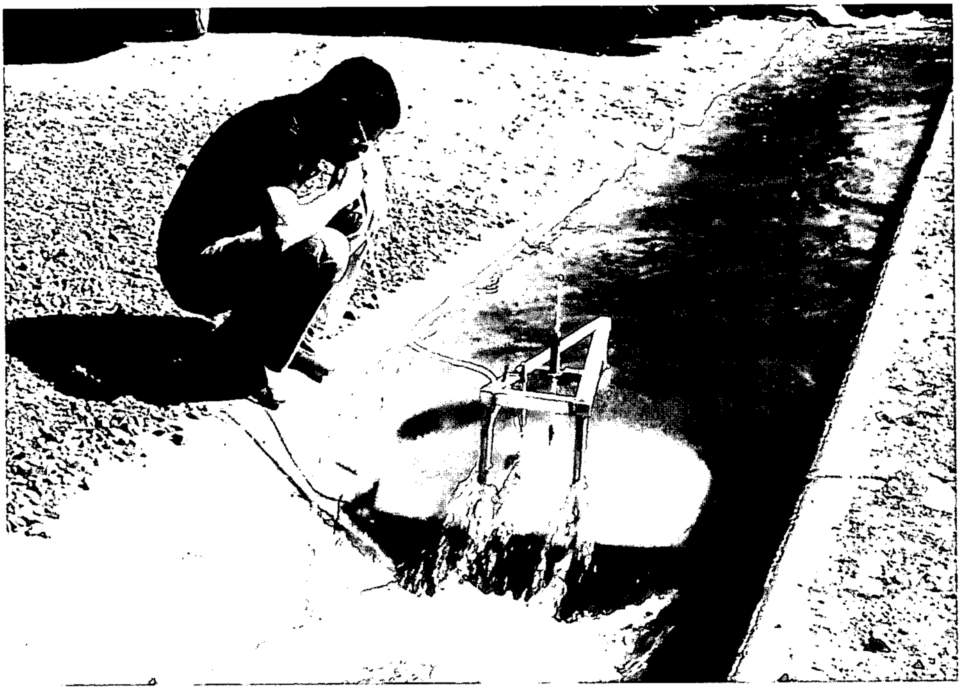


Figure 3.18 Portable weir for use in a concrete lined canal. Air is being purged from the sensing pipe and tubing. (Arizona).

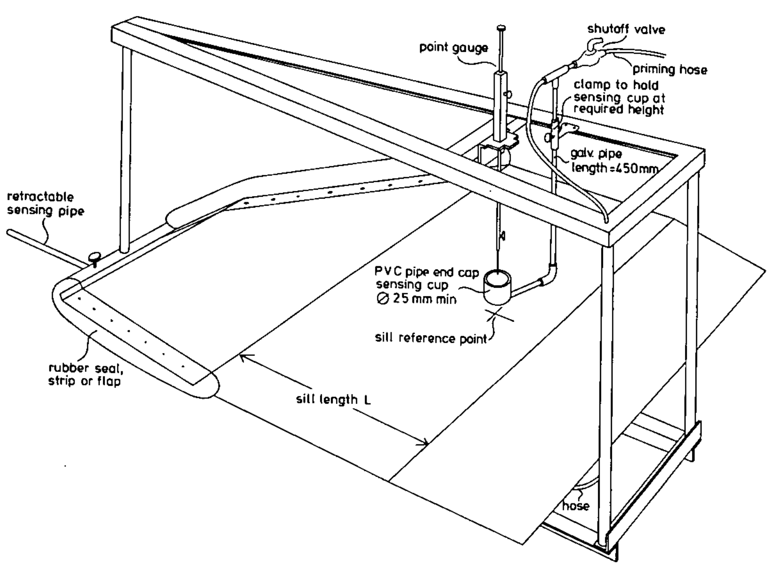
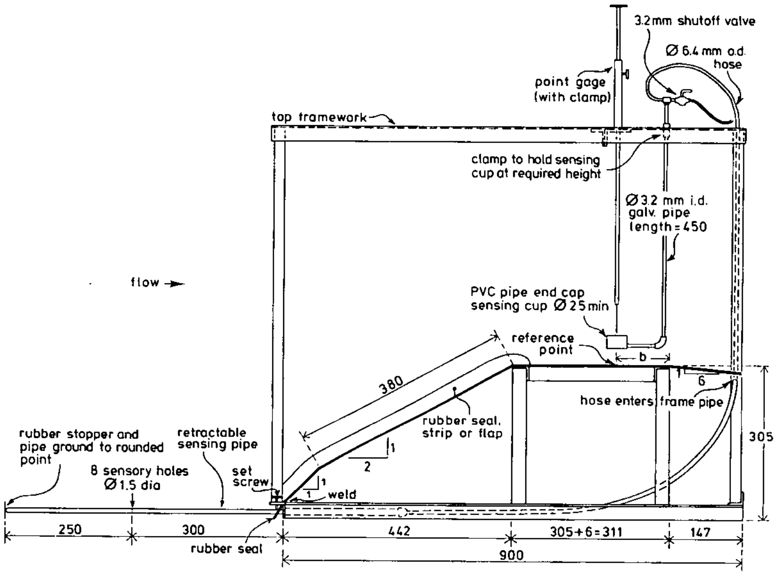
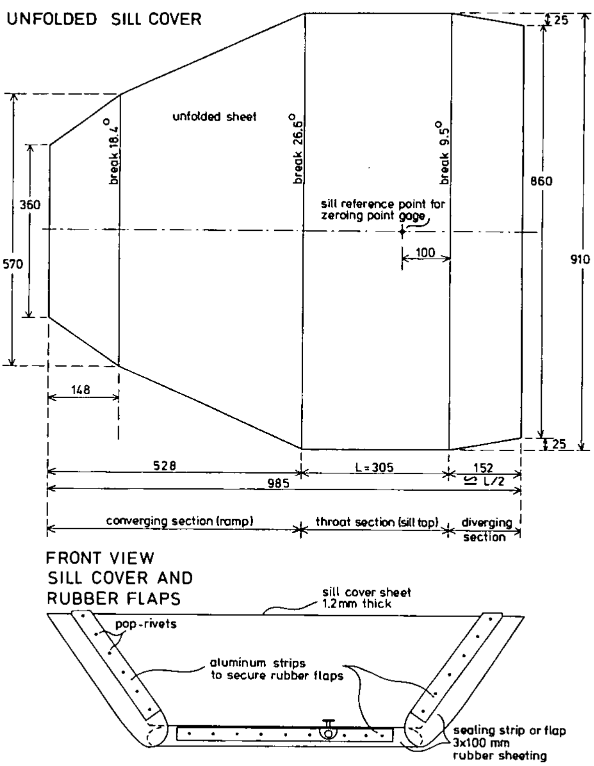


Figure 3.19 Isometric view of portable weir for use in a concrete-lined canal.



(a)



(b)

Figure 3.20 Construction drawings for portable weir.

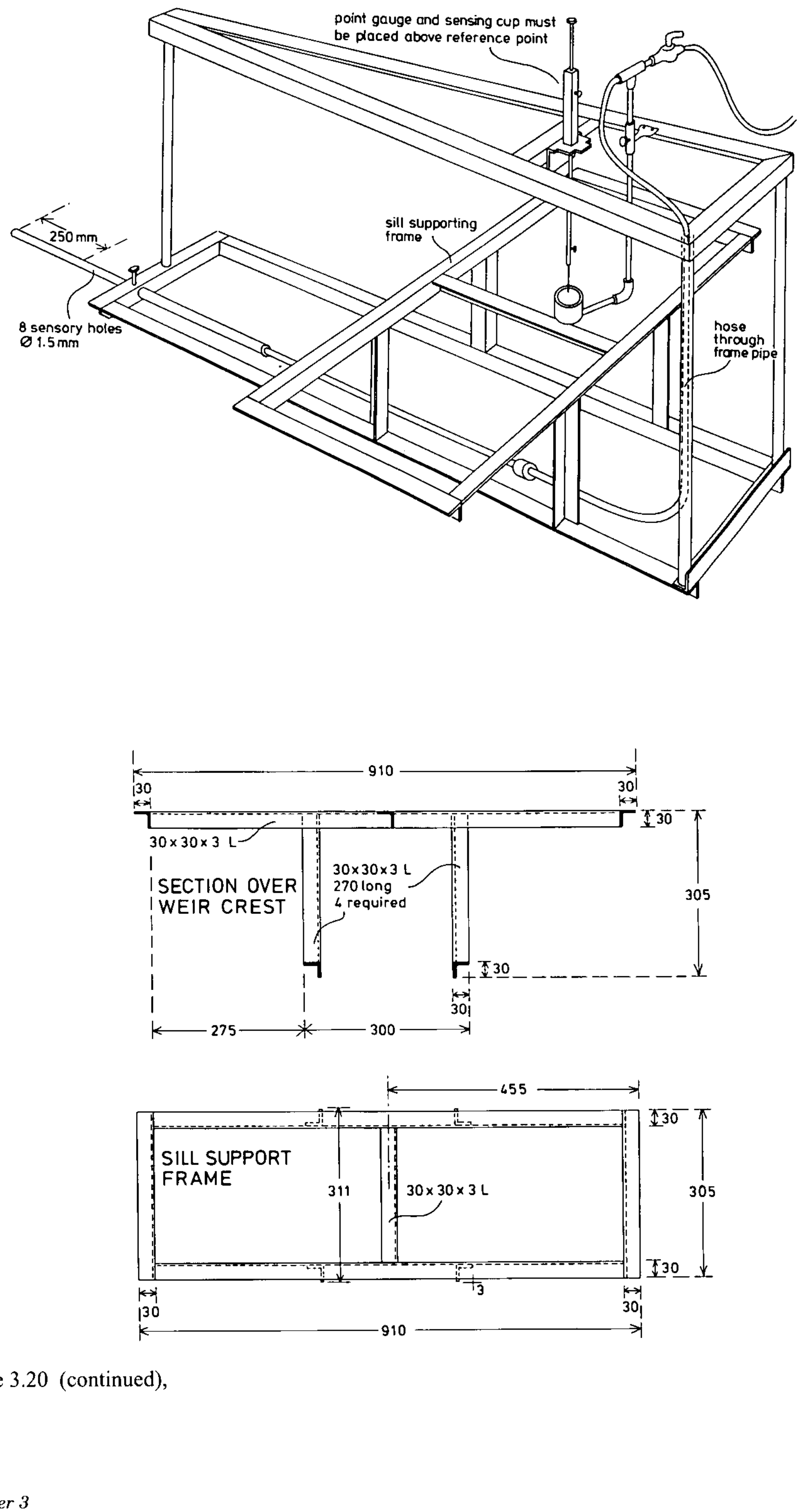


Figure 3.20 (continued),

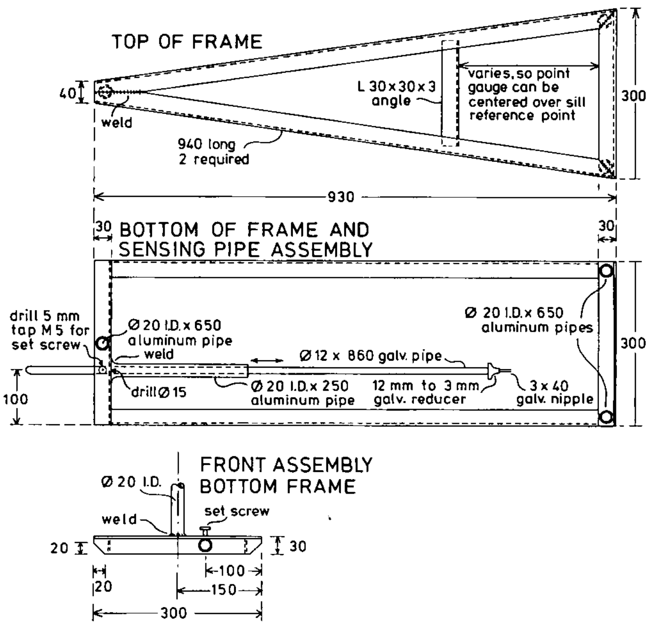


Figure 3.20 (continued).

The weight of the welded version shown is about 10 kg (25 lb). Leakage past the weir is controlled by rubber sealing strips made of 3.2-mm (1/2-in.) thick sheeting cut about 0.10 m (4 in.) wide. The seal is fastened to the sides of the ramp and to the lower front angle of the framework by aluminum strips and pop rivets. It is helpful if, during construction, the rubber sheeting is stretched slightly so that it warps strongly upward, especially across the bottom of the weir. Water pressure then causes a good seal against the canal walls and bottom. Insufficient stretching may allow the rubber to turn under the weir frame during placement, negating its effectiveness.

Translocation of the point gage and stilling-well cup to a position above the point of zero registration makes portable weirs insensitive to minor problems of leveling in all directions and eliminates the need for lateral transfers of elevations by surveying techniques (Figure 3.21). Thus careful leveling is not necessary, just gross adjustments readily observed by the eye (see also Section 4.10). The upstream water level can be sensed by attaching flexible tubing from the cup to a sensing pipe. The sensing pipe can either be attached to the portable flume (and retractable as shown here), or be separate (see also Sections 4.9 and 4.10). For structures with side walls, as in Figure 3.22, a side-wall tap replaces the sensing pipe.

The temporary structures described in Section 3.2.3 can often be used as portable structures. To accomplish this requires a portable head detection method, which is described in Section 4.9.1.

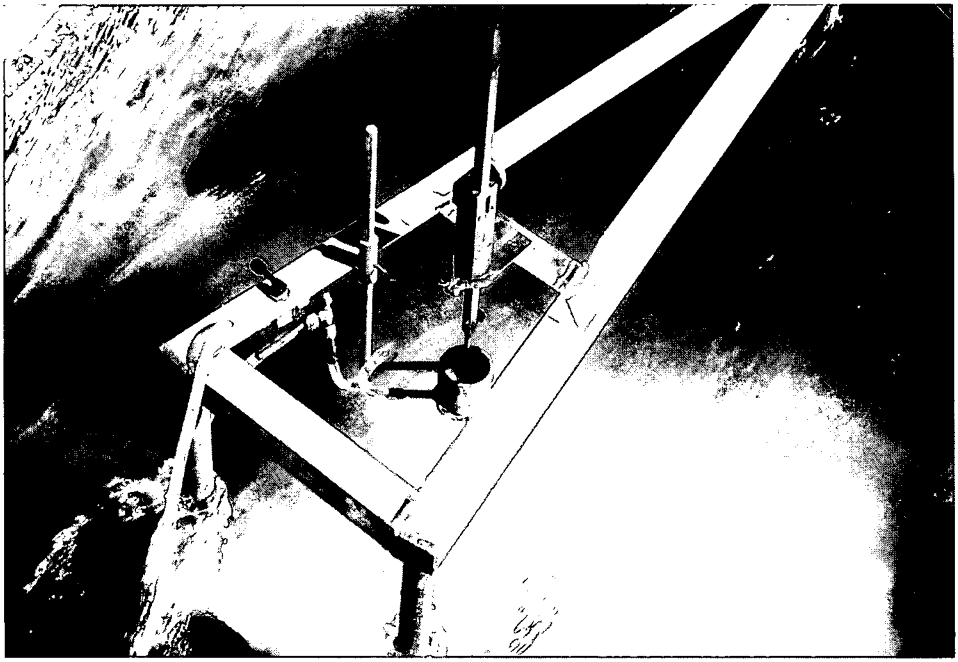


Figure 3.21 Sensing cup and point gage on portable weir.

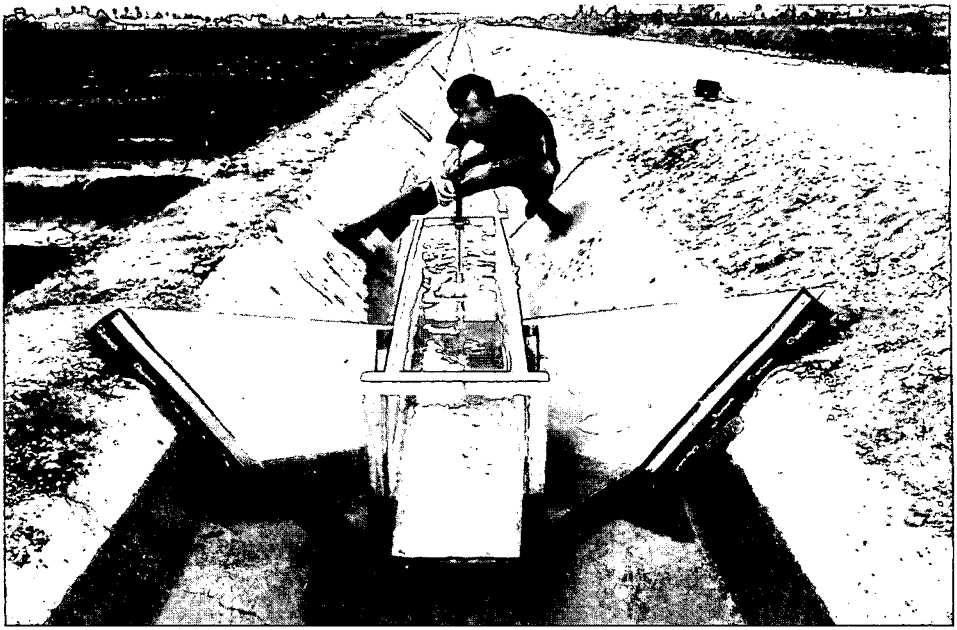


Figure 3.22 Loose triangular check boards and rubber seals stop leakage around the flume. A point gage and translocated stilling well can be used to measure the head. A different set of check boards may be used for unlined channels (Arizona).

Other channel shapes

Figure 3.22 shows a temporary weir with rectangular control section constructed of plywood. The two loose wing walls can block flow along the "box weir" in a concrete-lined canal or in an earthen channel. Rounding or beveling the upstream edge of the sidewalls is recommended to reduce flow separation and increase the range over which the structure can be used. The dimensional requirements of this weir are the same as for the rectangular structures of Section 3.3.1. Thus the sizes and shapes for these weirs are unlimited. The rating for a wide range of flow conditions can be obtained from Appendix 4, Tables R.3 and R.4. The method of measurement of upstream sill-referenced head on this weir is discussed in Sections 4.9 and 4.10.

The temporary devices of Section 3.2.3 for other canal shapes can also be used as portable devices, as shown in Figure 3.16. These are particularly convenient for small channels.

3.3 Structures in Small Earthen Channels

The cross-sectional shapes of natural streams, earthen irrigation canals, and drainage canals vary widely. Generally, they are much shallower and wider than the concrete-lined canals of Section 3.2. For a given size of irrigation canal, the discharge is usually less for the earthen canal because the permissible flow velocities are less. Because of this wide variety of possible shapes for unlined channels and the related range of flows to be measured, a wide selection of measuring structures must be available to the designer in order to obtain accurate flow measurement.

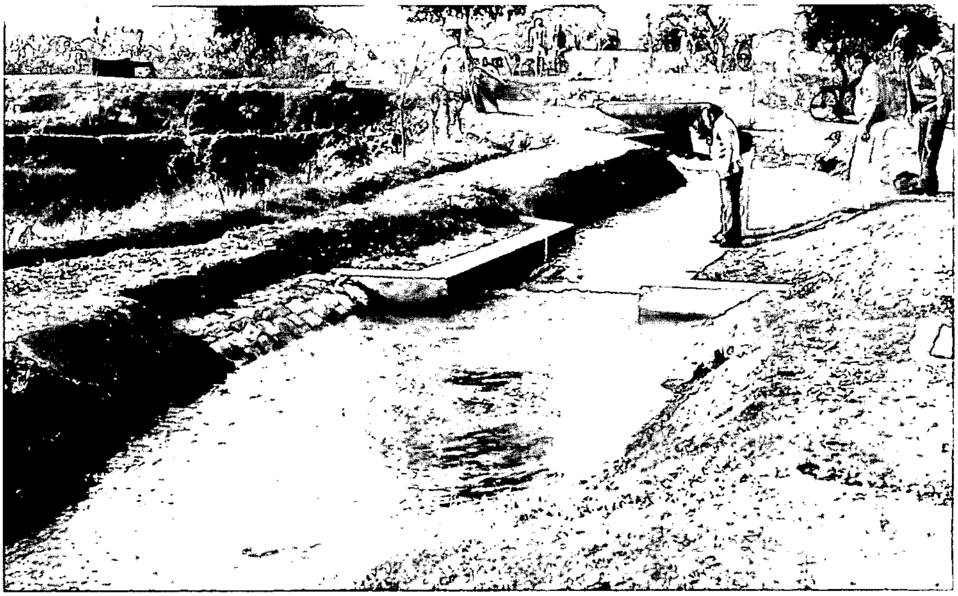


Figure 3.23 Flow measuring structure with a rectangular control section in an earthen canal (India).

In this section, a number of alternative shapes of broad-crested weirs and long-throated flumes are described. The weirs have a rectangular control section, the width of which can be varied (see Figure 3.23). The flumes have a triangular control section, which makes them particularly suitable for drains and natural streams. Design procedures, standard sizes, and rating tables are discussed in Section 5.5. Also, the trapezoidal weir shapes of Section 3.2.1 can be used in earthen channels. Other cast-in-place shapes (e.g., a complex trapezoid) are discussed in Section 3.4, which deals with large canals.

Weirs and flumes for earthen (unlined) channels require a structure that contains the following basic parts: entrance to approach channel, approach channel, converging transition, throat, diverging transition, stilling basin, and riprap protection. As illustrated in Figure 3.24 the discharge measurement structure for an earthen channel is longer, and thus more expensive, than a structure in a concrete-lined canal because, in the latter, the approach channel and sides of the control section are already available and the riprap is not needed. The rating tables and head-loss requirements for the standard-size flumes and weirs for earthen channels assume that the structure contains all of the sections shown in Figure 3.24.

The purpose of the approach canal shown in Figure 3.24 is to provide a known flow area and velocity of approach at the head-measurement station. If the upstream sill-referenced head is not measured in the specified approach canal, but in the wider earthen upstream channel, the standard rating tables must be adjusted (see Section 6.4.5). Further shortening of the full-length structure of Figure 3.24 can be obtained by deleting the diverging transition or the rectangular tailwater channel. The diverging transition may be deleted if there is adequate drop across the structure such

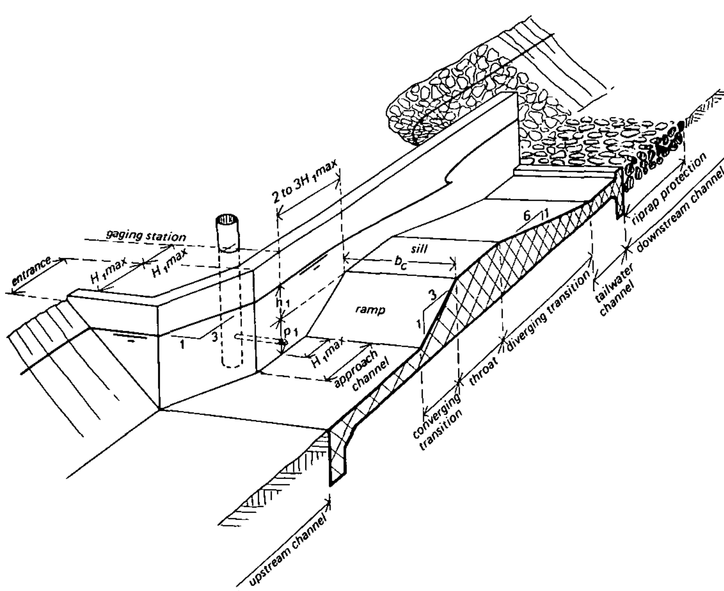


Figure 3.24 Components of flow measuring structure with rectangular control for earthen canals.

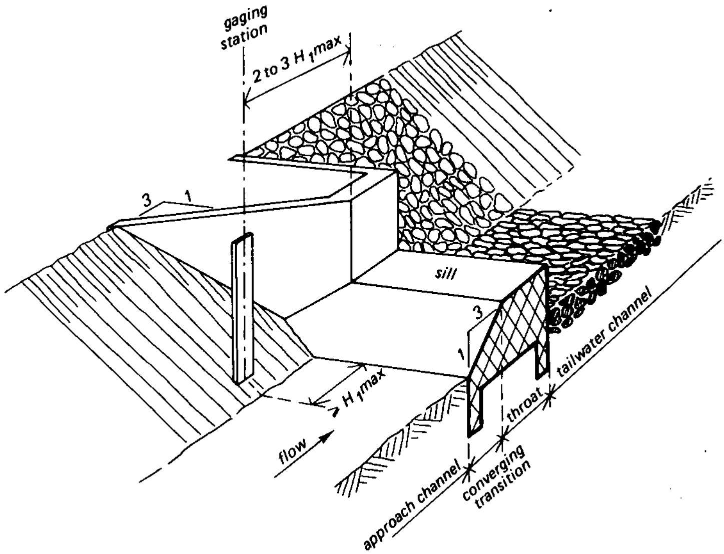


Figure 3.25 Truncated flume for earthen channels.

that submergence of the structure from downstream backwater is not a concern. (see Section 5.5.2 for further details). If the converging transition connects the earthen channel directly to the throat and no diverging transition and stilling basin are required, a structure such as that illustrated in Figure 3.25 can be used. A structure of this type is commonly used in drains or irrigation canals.

The downstream transition is optional, depending on the head drop available at the site. However, in all cases the channel downstream from the throat must be protected from erosion and undercutting of the structure's foundation (see Figure 7.10). The downstream part of the structure can consist of an extension of the cross section constructed upstream, a diverging transition, and/or riprap protection. See Chapter 7 to determine the requirements for minimizing erosion downstream.

3.3.1 Cast-in-place structures

Compacted fill is necessary to support cast-in-place structures when they are constructed in earthen channels. For flow rate capacities less than 3 m^3/s (100 ft^3/s), the fill volume and weight can be expected to cause no foundation problems. The fill material should be readily compacted and non-swelling. Usually granular material is most suitable, provided the covering concrete, head wall, and toe wall are competent for limiting seepage.

Rectangular-throated structures

In constructing the weirs or flumes of Figures 3.24 and 3.25 the designer may select any locally available construction material. For example: the wing and sidewalls can

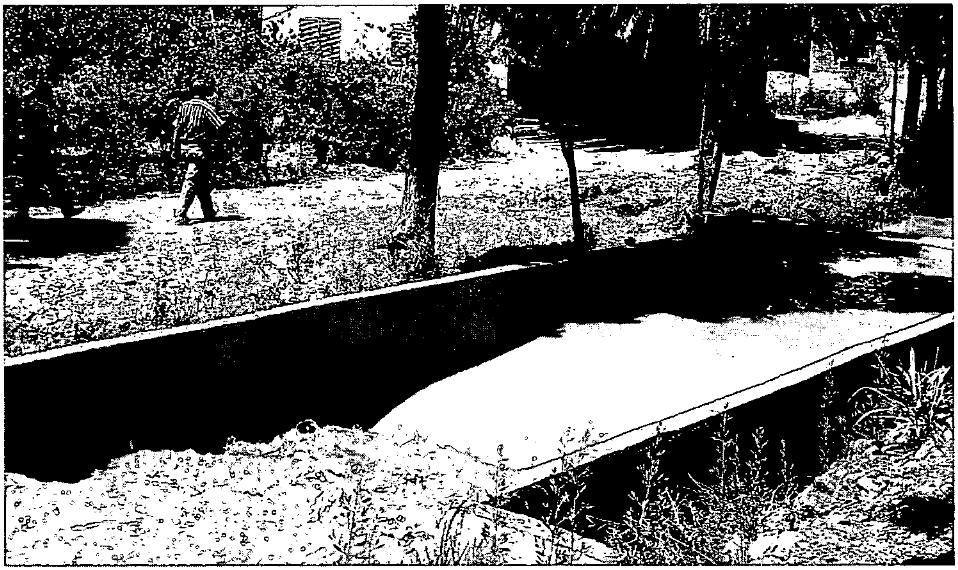


Figure 3.26 Rectangular-throated flume constructed entirely from reinforced concrete (Argentina).

be brickwork containing a mortar-plastered sill (see Figure 3.23); the entire structure can be made of reinforced concrete (see Figure 3.26); or a (wooden) sheet piling can be driven across the channel in which a steel or aluminum control section is bolted onto seals (see Figure 3.27). The latter method of construction is very suitable for soils with a low bearing capacity.

When constructed with brick (block or stone) and mortar (Figures 3.28 and 3.29), care must be taken to assure that flow does not bypass the structure or flow under the structure and cause it to settle. Cutoff walls are required to keep this from happening. These cutoff walls should extend into the soil below the structure and into the canal banks on the sides of the structure. The extent of the cutoff walls depends upon the size of the structures, the soil, and the head loss across the structure. For small irrigation structures in many soils, a cutoff wall which extends at least 0.3 m (1 ft) into the canal invert and banks is often sufficient.

If concrete is available, the lower cutoff walls and the floor of the approach and tailwater sections can be poured together. If concrete blocks are used for the vertical walls, it is recommended to use reinforcing steel to tie the floor and walls together. If brick or stone are used for the floor, it is recommended to start the first layer of brick or stone by pressing them into the partially wet concrete to assure that a seam between the floor and walls does not develop. The vertical walls and cutoff walls can then be laid. At the same time, the weir crest and ramp(s) can be roughly formed with block, brick or stone (see Figure 3.28). After these have all been constructed, mortar can be used to plaster the entire surface of the block, brick or stone on the side where water flows to fill in the gaps and to provide a relatively smooth surface, particularly in the area of the throat. Care should be taken to assure that there are no protrusions of stone or block into the flow and that the cross section is relatively uniform through the

Figure 3.27 Metal broad-crested weir on wooden sheet piling (The Netherlands).

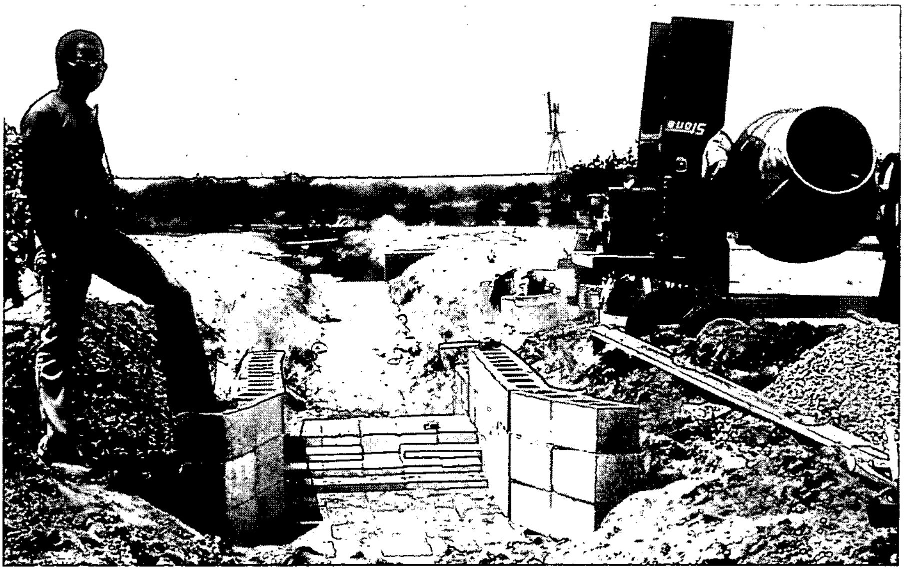


Figure 3.28 Upstream view of a flume being constructed from concrete block. The sill and the bulk of the ramp are constructed from block, and the ramp surface is finished with hand-troweled concrete. Mortaring the entire inside surface is recommended.

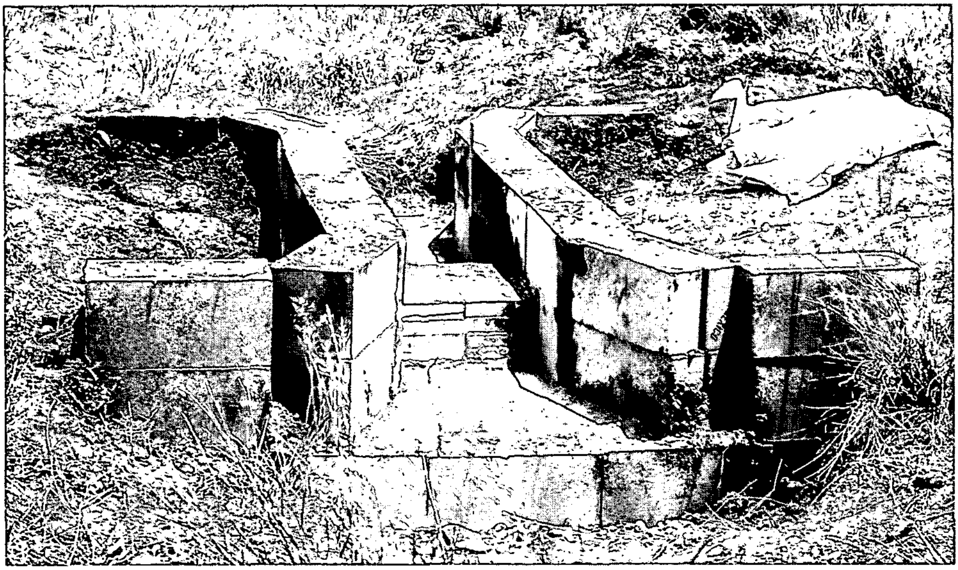


Figure 3.29 Downstream view of a rectangular-throated flume constructed from concrete block. The riprap protection downstream from the structure has not yet been placed.

structure. A level should be used to make sure that the crest is level in all directions, with no major depressions or protrusions. Downstream from the structure, rock or stone should be placed to avoid erosion of soil and subsequent undercutting of the structure (see Figure 3.23).

Once the structure has been completed a wall gage can be placed in the approach channel. Anchors for the gage can be placed in the wet mortar, and the gage set to the proper elevation at a later time with surveying techniques as describe in Section 4.9. The throat width should be measured and an as-built calibration prepared.

Trapezoidal-throated structures

A structure can be built that resembles a short section of lined canal with a sill placed in the bottom as discussed in Section 3.2.1. The structural requirements are the same as for the rectangular structures that are given in Section 3.3.1. Short-cutting these design requirements will ultimately lead to structural failure. The medium-sized flume for Kirwin Canal (Figure 3.30) is an example of this construction technique. The definition for this flume is also provided with the software described in Chapter 8. Its maximum capacity is 5.5 m^3/s (193 ft^3/s) and the flume is constructed with reinforced concrete. The construction sequence is as follows:

1. Following the topographical work on the layout and relative elevation of the flume, the first parts of the flume that should be constructed are the stilling well plus its connecting pipe(s) and the cutoff walls at the ends of the canal section. Both cutoff walls should be at least 0.6 m (2 ft) deep (Figure 3.31). Backfill must be well compacted.

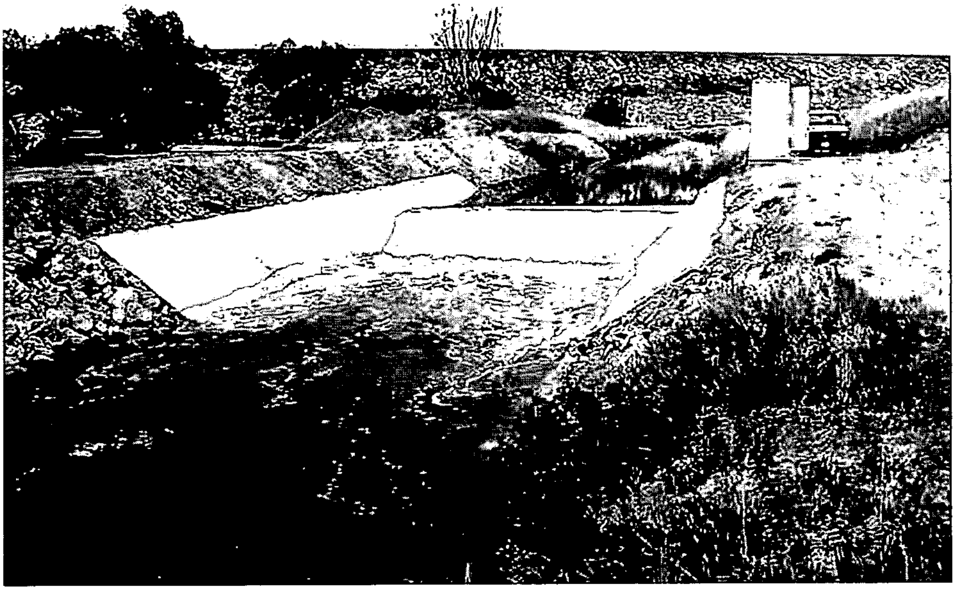


Figure 3.30 Flow over a flume near the head of Kirwin Canal, constructed as part of a short section of "lined" canal added to an earthen channel. Design capacity is 5.46 m^3/s (193 ft^3/s) (Kansas).

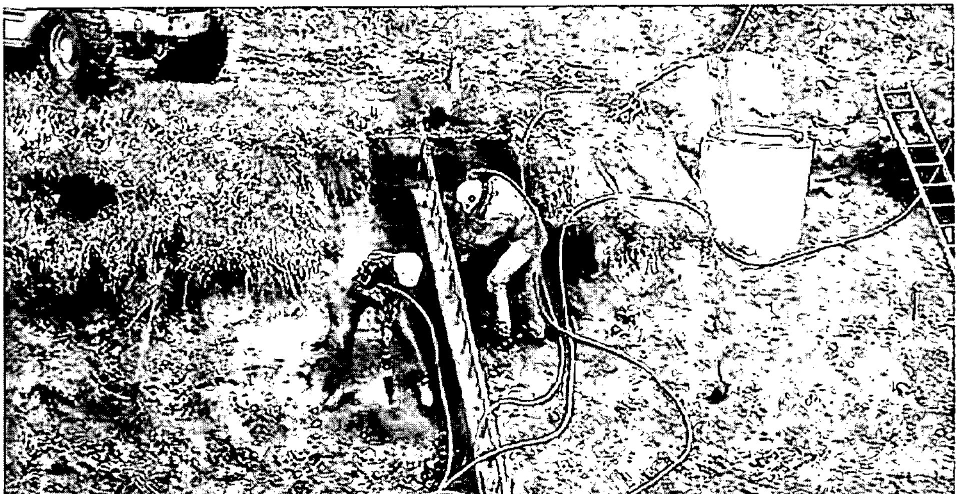


Figure 3.31 Soil around the stilling well and cutoff walls is being compacted to avoid settlement of the structure.

2. Place the drain pipe, and compact the fill to support the crest and ramps of the flume. The compacted fill must consist of a non-shrinking and non-swelling material. A sand-gravel mix is effective.
3. Install the frame that forms the edges of the throat. For larger structures, it is convenient also to have a metal frame at the intersection of the ramps and the sides of the flume and the bottom of the canal section (Figure 3.32).
4. Install reinforcement consisting of a welded wire fabric with 4-mm (3/16-in.) diameter wires at 300-mm center-to-center spacing. The reinforcement should be about 50 mm (2 in.) from the surface of the concrete. The thickness of the cutoff wall, ramps, and crest is 150 mm (6 in.); for the sides and the bottom of the canal section, 80 mm (3 in.) is sufficient.
5. Check if the crest is level in the direction of flow and at the correct elevation.
6. Concrete now can be placed. To avoid cracks in large concrete slabs due to shrinking, construction joints are needed in larger flumes. A common sequence of placement is: crest, upstream and downstream canal bottoms, ramps, and sides.
7. Install a riprap protection downstream from the concrete in accordance with the design rules given in Section 7.3 (Figure 3.33).
8. Install the wall mounted staff gage in accordance with the zero-setting procedure of Section 4.9 (Figure 3.34).

Rating Tables R.1 and R.2 in Appendix 4 were computed for a trapezoidal approach (and tailwater) channel of a given size. If an earthen approach channel of significantly different size is used, then the rating tables must be corrected for approach velocity by the method of Section 6.4.5. Similarly, if the tailwater channel is of significantly different size, then the head-loss values will also need to be recalculated. The software in Chapter 8 can be used to compute as-built rating tables and head-loss requirements.

Figure 3.32 The edges of the sill and the ramps consist of an angle iron that remains in the concrete.

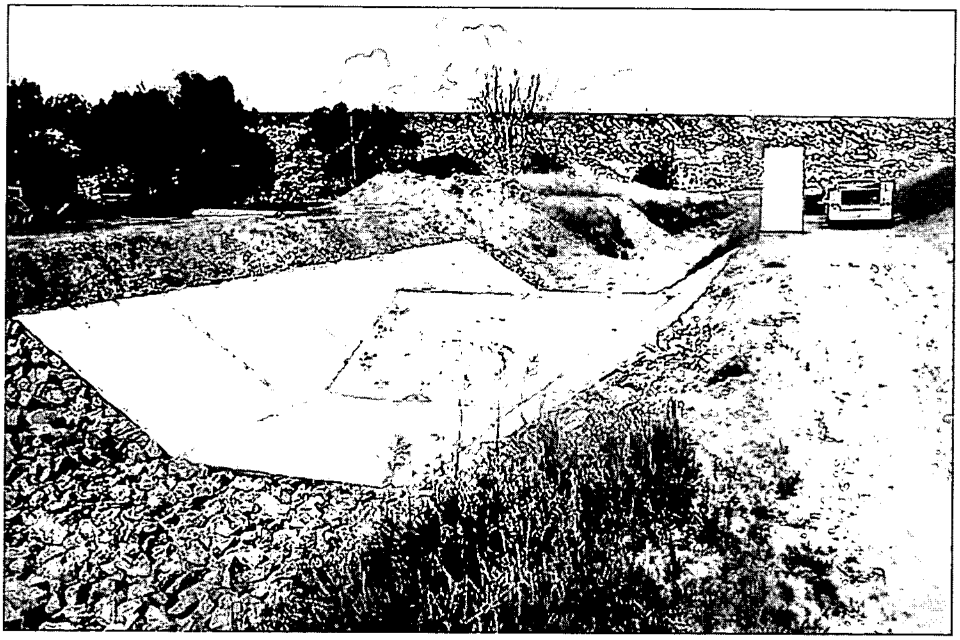


Figure 3.33 Riprap protection is provided downstream from the flume.

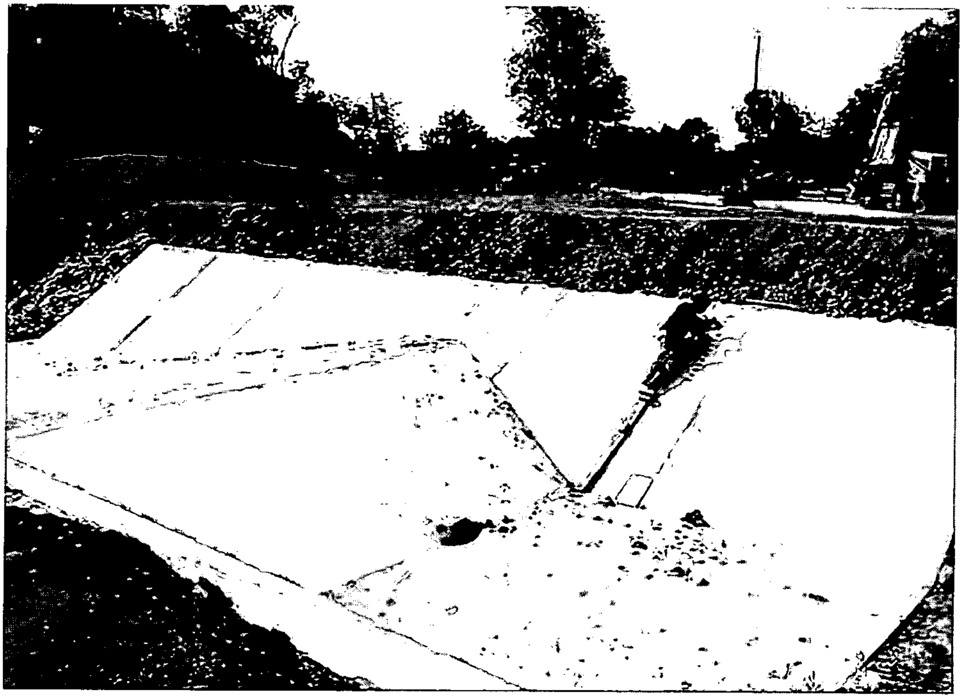


Figure 3.34 Installing the wall gage.

Triangular-throated structures

For monitoring return flows and operational spillage from irrigation systems or for measuring flows in natural streams, a structure is needed that can measure a wide range of flows. As shown in Section 2.4, a structure with triangular control section is very suitable for this purpose because the range of flows that can be accurately measured with one device is very wide (350:1). As an example of this group of structures, Figure 3.35 shows a flume that was retrofitted to an irrigation lateral canal. The figure also illustrates the very low drop in water surface needed for modular flow, only $0.1H_1$.

The flume illustrated in Figure 3.35 has 3:1 side slopes in the 1.20 m (4-ft) long throat section. The approach section is 1.8 m (6 ft) long and also has 3:1 side slopes and a bottom width of 0.6 m (2 ft). The flow range for this flume is 0.006 m^3/s (0.2 ft^3/s) to 2.34 m^3/s (80 ft^3/s). A stilling well and recorder system was installed on this flume. A typical structure is shown in Figure 3.36. The method of construction is similar to the trapezoidal-throated flume, described above. The example flume shown in Figure 3.36 has 3:1 sloping sidewalls in the throat. The designer, however, is free to choose different side slopes. (see Section 5.5.3).

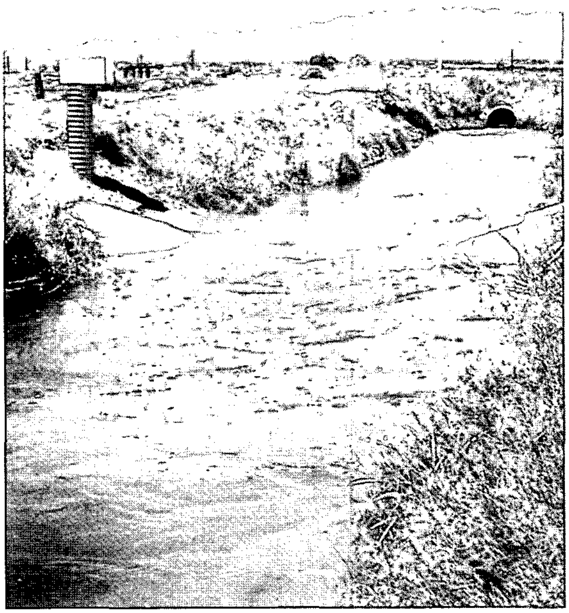


Figure 3.35 Triangular-throated flume in irrigation lateral canal (Arizona).

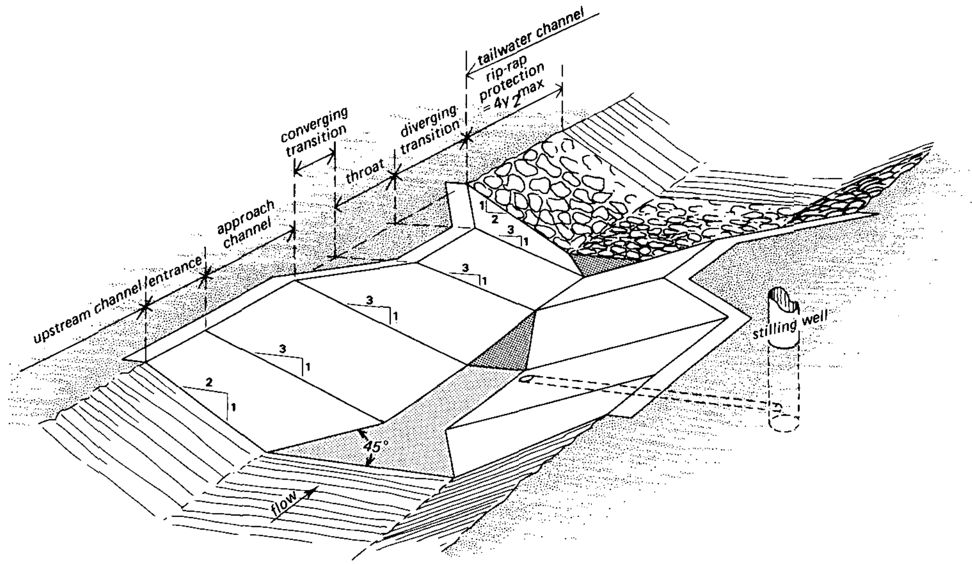


Figure 3.36 Schematic drawing and layout of triangular-throated flume.

3.3.2. Prefabricated structures

Prefabricated structures for earthen channels need to be constructed in such a way that flow does not pass under or around the structure. Several methods can be used to provide appropriate cutoff walls. For weirs constructed from sheet metal, the throat and converging transition are attached together as one unit, including floors and walls. The approach section, diverging transition, and tailwater channel are optional. Wooden sheet piling can be driven into the ground to provide an appropriate cutoff wall, as was shown in Figure 3.27. The opening in the cutoff wall should approximately fit the shape of the prefabricated weir. The weir is then attached to the cutoff wall in the area of the throat. A seal must be provided between the weir and cutoff walls to avoid leakage. Downstream from the structure, rock or stone should be placed to avoid erosion of soil and subsequent undercutting of the structure. If the wall gage is placed in the unlined approach channel, special care must be taken to assure that the gage does not settle after it has been set to the proper elevation. This may require a stiff member (wood, steel, etc.) to be well anchored into the ground. This staff should be placed so that it does not catch floating debris and so that it can be reached from the canal bank for cleaning (see Figure 4.10).

Another style of prefabricated weir for an unlined channel is shown in Figure 3.37. Here, a section of concrete pipe is used as a section of channel in which the weir is constructed. The pipe essentially provides the surfaces for the approach and tailwater channels and the sides for the throat and transition sections. Trenches are dug for the cutoff walls and are cast in place, as shown in Figure 3.38. The pipe section is then lifted into place while the concrete for the cutoff walls is still wet. Here, care needs to be taken to assure that the throat is level in the direction of flow, and that the pipe section is rotated to the correct position so that the weir is level transverse to the flow.

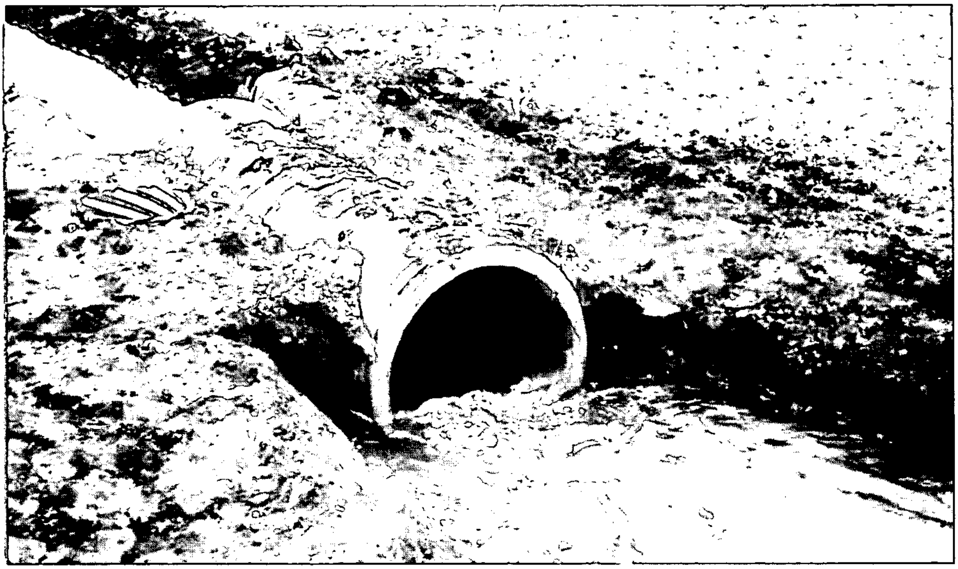


Figure 3.37 Prefabricated weir constructed in a section of concrete pipe (California).

A wall gage can be mounted on the side of the pipe wall; however, this would make it difficult to read unless a hole was cut into the pipe. Often a stilling well is placed on the outside of the pipe section (Figure 3.38). A hole is cut in both the pipe and the stilling well so that a pipe can be used to connect them. Care should be taken to assure that the stilling well does not settle. Soil should be backfilled around the cutoff walls, the stilling well, and pipe section to assure that everything remains in place. Downstream from the structure, rock or stone should be placed to avoid erosion of soil and subsequent undercutting of the structure.

Prefabricated weirs installed in smaller pipe sections can be used as portable measurement devices. (e.g., see Figure 4.26)

3.3.3 Portable and temporary structures

Trapezoidal-shaped control

Five portable RBC flumes (Clemmens et al. 1984) were designed for use in furrows and small earthen channels (Figure 3.39). These flumes are scale models of each other in which the width of the sill crest b_c varies from 50 to 200 mm. Because all other flume dimensions are proportional to b_c, each structure has a different overlapping flow range. For sizes of b_c and the related throat length and flow ranges, see Table 5.5.

To facilitate construction, the shape of the flume is kept relatively simple. As a result, it can be constructed from most sheet materials. Commonly, 1-mm thick galvanized sheet metal is used. The construction drawings are given in Figure 3.40. All dimensions are related to the throat width, b_c.

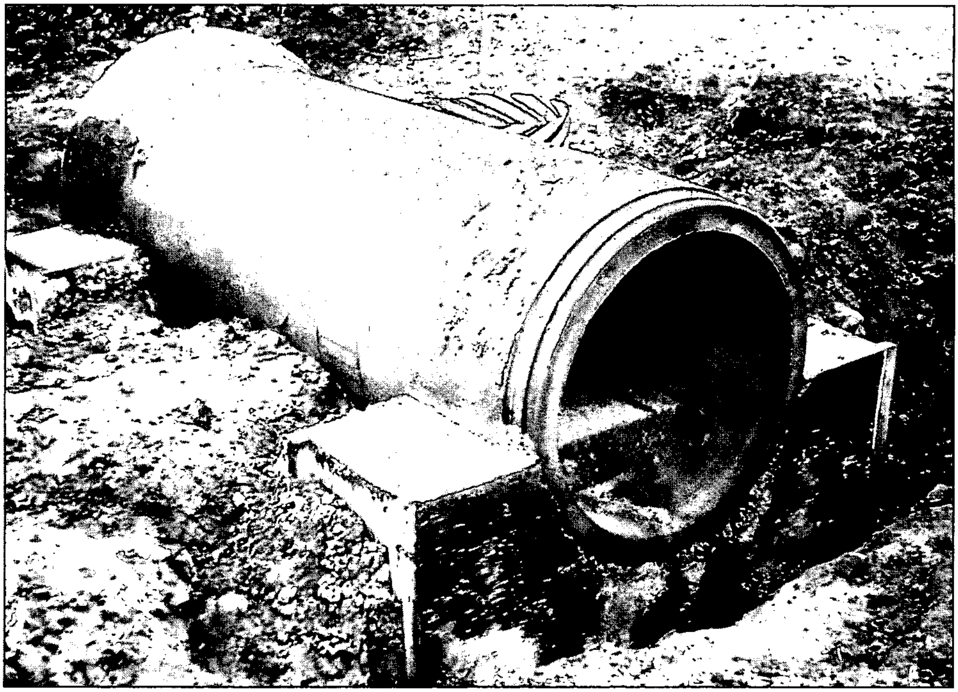


Figure 3.38 The pipe section is set into place while the concrete for the cutoff walls is still wet.

Figure 3.39 Portable RBC flumes are very suitable for flow survey work (Arizona).

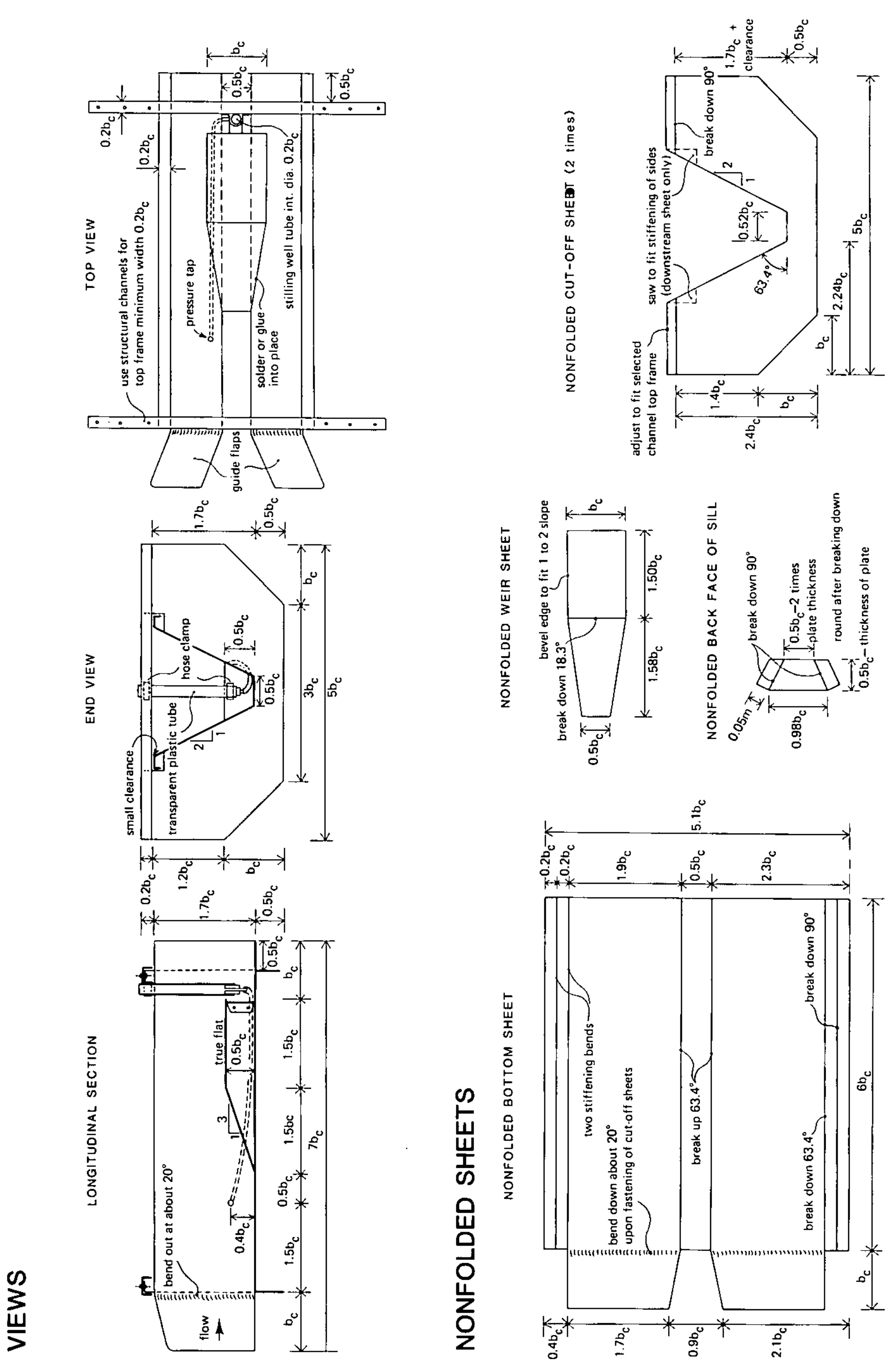


Figure 3.40 Construction drawings for portable RBC flumes. Dimensions are given in terms of b_c, so that the drawing applies to all five flume sizes.

The head h_1 is measured in a translocated stilling well. The stilling well is mounted near the control section to minimize changes in the sill reference of the well caused by a slightly non-level installation. Cross-slope leveling of the flume is facilitated by keeping the upstream edge of the cutoff parallel to the water surface. Leveling of the longitudinal slope may be done by use of a carpenter's level. Experienced users can soon judge adequate leveling and will not require a carpenter's level. If the portable RBC flume is installed for seasonal or semi-permanent flow measurement, we advise using the alternative stilling-well location on the side of the flume (Figure 3.41). Otherwise, the unattended tube would collect floating debris. This location is also recommended on the two smallest (b_c = 50 mm or 75 mm) flumes because it allows the use of a larger-diameter stilling-well tube. If the alternate (side mounted) stilling-well location is used, the stilling well should be located at a distance of $1.5b_c$ upstream from the downstream end of the flume.

A suitable sequence to assemble the RBC flume is as follows:

- Fasten the cutoff sheets to the bottom sheet. The easiest method is to rivet them in place by using four metal strips bent 90°, and then make watertight joints with silicone sealer.
- Adjust back-face of sill to fit accurately and pop rivet into place. Be sure to drill holes to fasten pipe clamp before riveting.
- Glue weir sill into place with silicone gel. Note that side edges of sill plate must be beveled so that sill width equals the desired b_c value.
- Pop rivet or bolt channel top frames to folded edge of cutoff sheets.

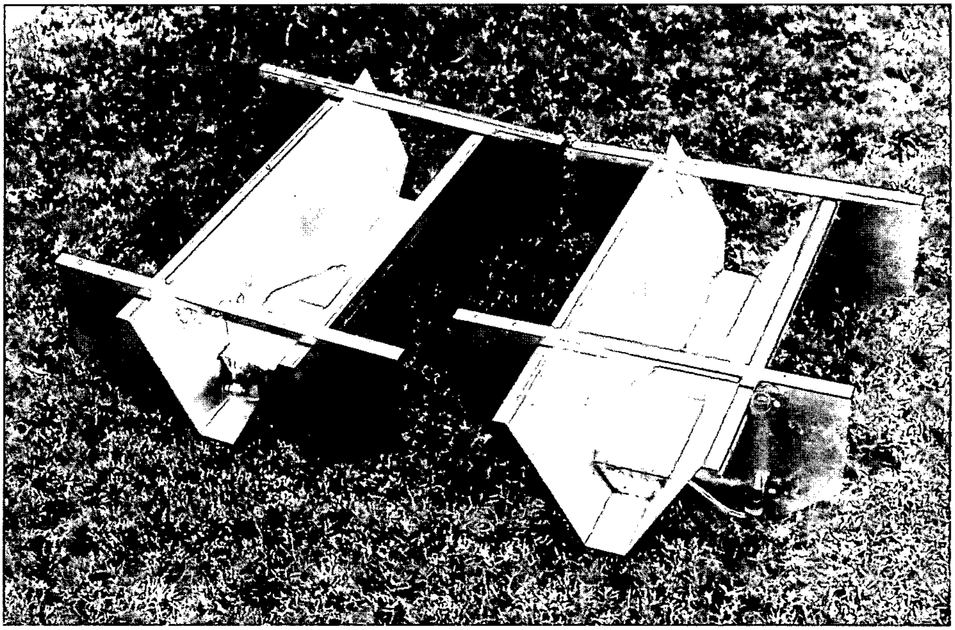


Figure 3.41 Portable RBC flumes with standard and alternative stilling-well locations. These flumes have a 100-mm throat bottom width.

- Insert length of copper tubing through perforated rubber plug, and glue this plug into stilling-well tube.
- Mount the stilling-well tube. Adjust the tube so that the top of the rubber plug coincides with the crest of the sill, and fasten pipe clamps.
- Now, solder or glue the length of copper pipe at the hole where it passes through the side-wall of the flume.
- Solder a short 90° bent length of copper tubing into the pressure-tap hole. The end of the tubing should be flush with the surface of the side-wall sheet. This pressure tap hole must be perpendicular to the flume side wall to avoid a systematic error in the head measurement (see Figure 4.13).
- Connect the above two lengths of stilling-well tubing with transparent plastic tubing. The tubing must slope down along the bottom sheet and up to the bottom of the stilling well. No upward intermediate bends may exist in this tubing because they will trap air and thus cause errors in the head reading.

Commercial versions of the RBC flumes are available in both stainless steel and fiberglass. (Figure 3.42). The use of an RBC flume with the dipstick method of measurement (See Sections 4.3, 4.6 and 4.10) is shown in Figure 3.43.

Figure 3.42 Commercially available RBC flume constructed from fiberglass.

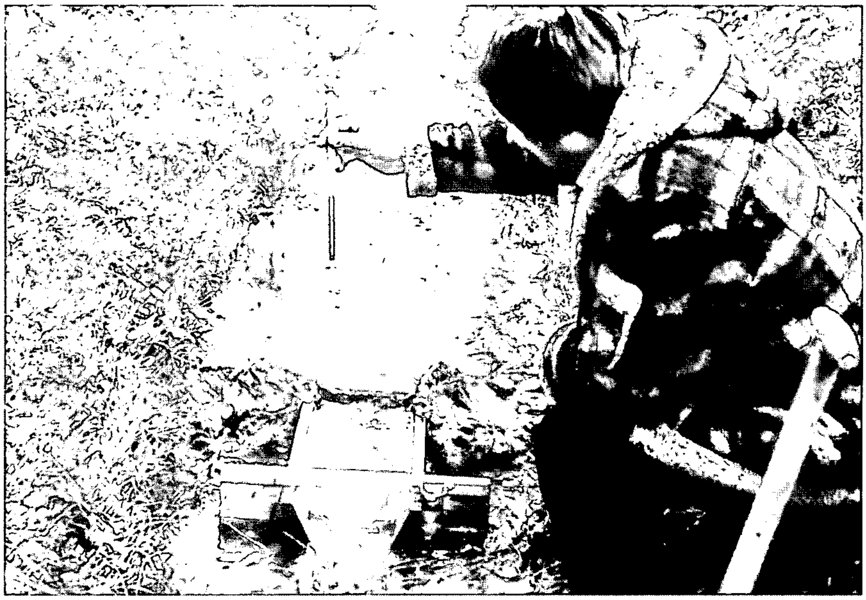


Figure 3.43 The discharge through this RBC flume is read directly from a hardwood dipstick, which has been specially marked. Lines are also etched on the stilling well so that the user can easily see when the flow has stabilized. The flume has a 100 mm throat bottom width (The Netherlands).

Rectangular-shaped control

Because the bottom width of a rectangle is wider than a trapezoid having the same area, a portable weir with rectangular control section can have a relatively large capacity. Rectangular flumes like those discussed in Section 3.3.1 can be constructed from fiberglass, wood, or sheet metal and used as a portable structure (Figure 3.44-wooden flume, Figure 3.45-sheet metal flume). At least one size is available commercially in fiberglass (Figure 3.46). A sheet of canvas, plastic or other suitable material is attached near the upstream end of the flume. This sheet is attached to the flume so that no water can leak between the flume and the sheet. A shovel is used to excavate soil and the cutoff sheet is then buried to assure that no water flows around or under the structure. The depth to bury this cutoff sheet depends on the soil, but 0.1 m (4 in.) is often sufficient. These structures can be placed on the soil and leveled, or rods can be driven into the ground to stabilize the structure. These rods slide through collars that are rigidly attached to the flume (Figure 3.47). A locking screw that holds the collar to the rod is used to hold the flume in place vertically.

These flumes can easily be placed in a dry channel, but it is difficult to judge the proper elevation at which to set them. Placement of these flumes in flowing water is possible, but adjustment of flume elevation can be difficult due to the weight of the water. The Adjust-A-Flume was developed to avoid the difficulty in adjusting the elevation of the flume while in flowing water (Figure 3.48). The device consists of two rectangular metal boxes that are adjustable with respect to each other (Figure 3.49). The outer section remains fixed once set in the channel. The throat section can be easily raised to the proper elevation so that the flow is not overly submerged and so that the drop through the flume is not excessive. These flumes avoid most of the difficulties associated with the use of portable flumes in earthen channels.

Figure 3.44 Portable rectangular-throated flume constructed of wood (Egypt)

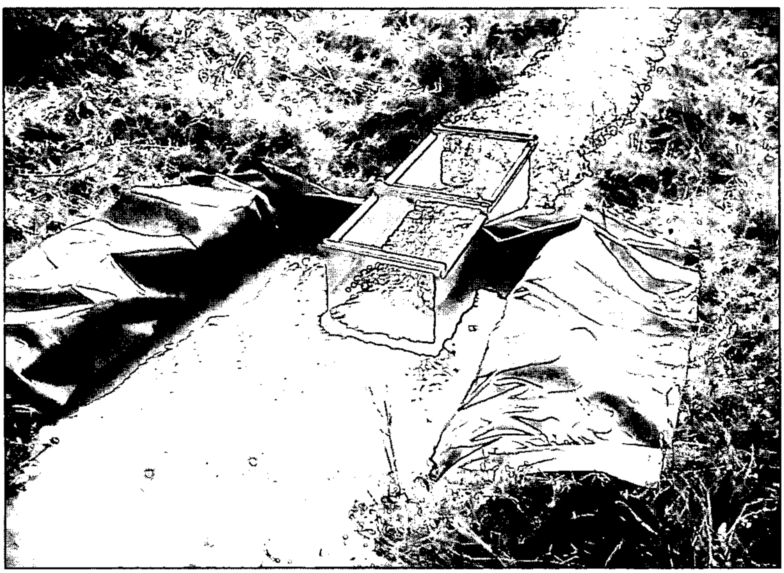


Figure 3.45 Portable rectangular-throated metal flume.

Figure 3.46 Commercially available portable rectangular-throated flume constructed from fiberglass.

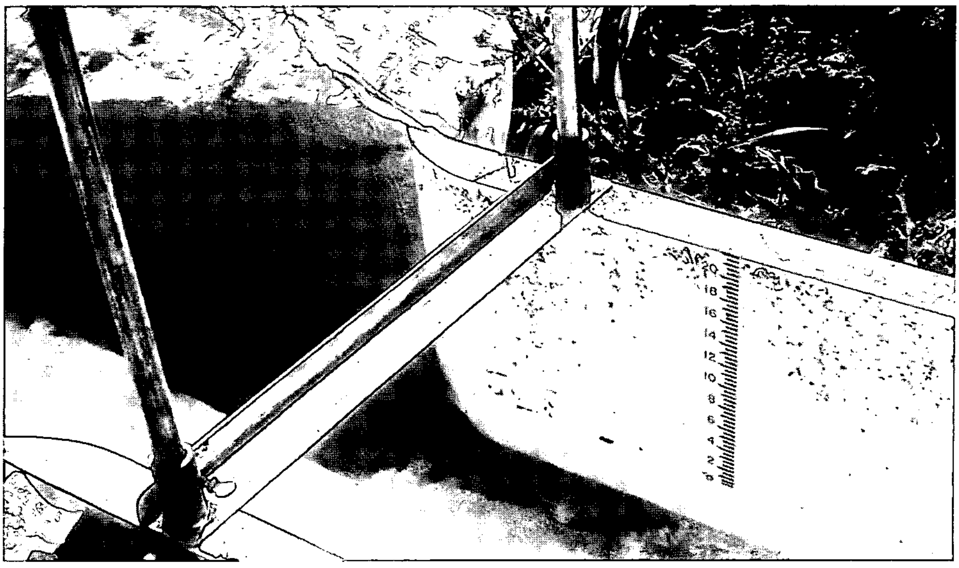


Figure 3.47 The fiberglass rectangular flume is leveled by sliding the flume vertically along the rods and then locking in place.

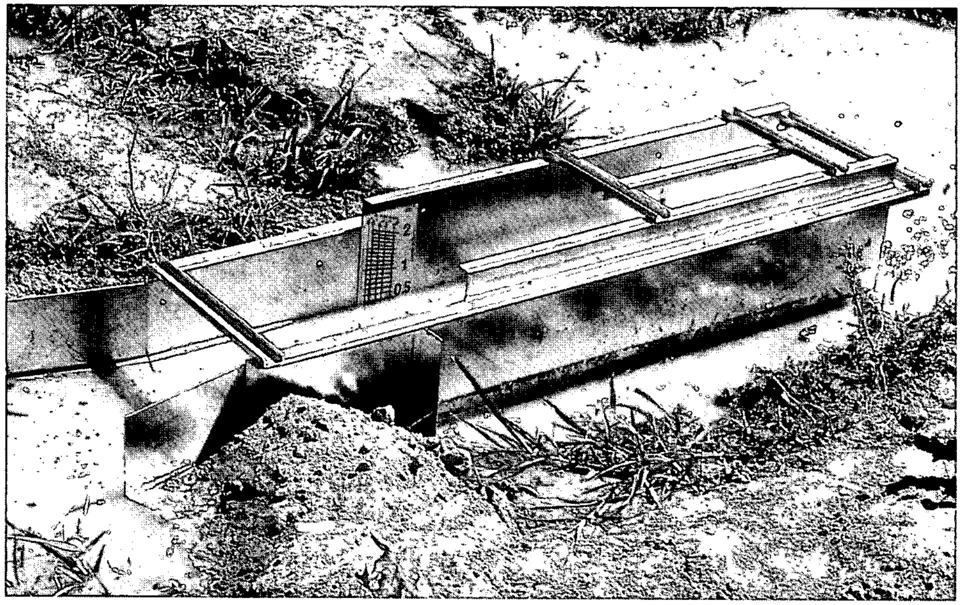


Figure 3.48 Commercial version of adjustable-sill flume (Adjust-A-Flume, Nu-way Flume and Equipment Co., P.O. Box 814, Delta CO 81416 USA). (Trade and company names are shown for the benefit of the reader and do not imply endorsement or preferential treatment of the company or product listed).

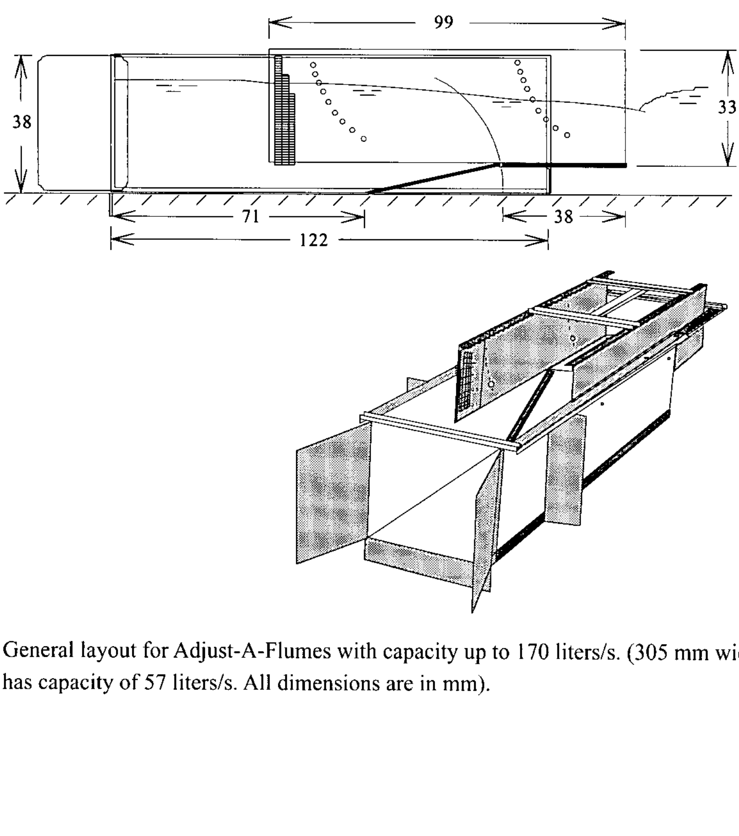


Figure 3.49 General layout for Adjust-A-Flumes with capacity up to 170 liters/s. (305 mm width shown has capacity of 57 liters/s. All dimensions are in mm).

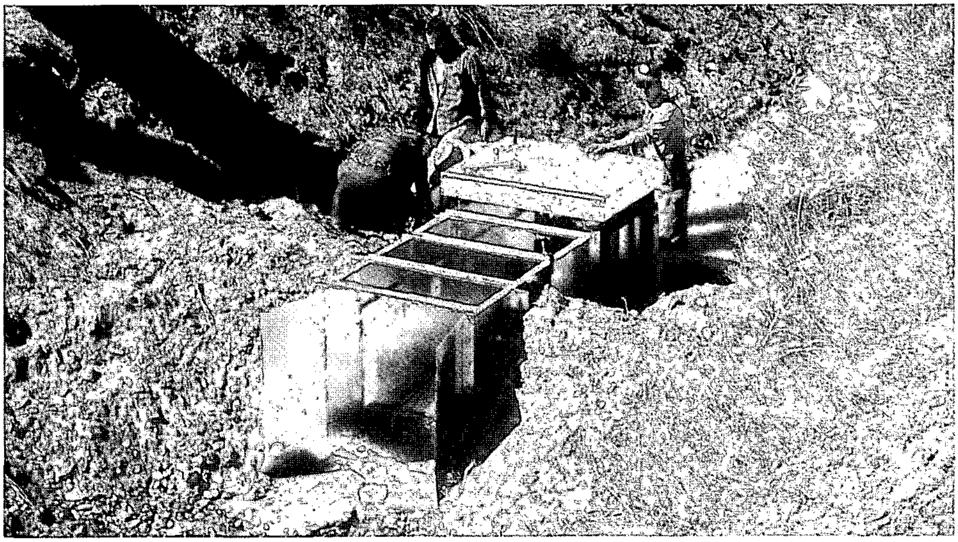


Figure 3.50 A 0.76-m (30-in.) wide adjustable-sill flume. The lifting mechanism is contained in the metal box on top of the flume.

The capacities of the smaller, portable Adjust-A-Flumes are 60 liters/s (2 ft^3/s), 120 liters/s (4 ft^3/s), and 180 liters/s (6 ft^3/s) in widths b_c of 0.305 m (1 ft), 0.610 m (2 ft), and 0.915 m (3 ft), respectively. The weirs can be fabricated from standard sized sheet steel. The sides of the inner box, which hold the gage, are punched with sets of bolt holes on each side of the frame that line up with vertically spaced bolt holes in the canal section walls. The downstream holes in the frame are sufficiently slotted forward so that the upstream holes can match the next set of higher or lower holes before moving the downstream bolts. This facilitates maintaining a level throat after movement and bears the weight of water over the throat. This method of adjustment slows the ability to change sill heights, but improves the stability and ruggedness of the device. The ramp is hinged and sealed at the leading edge of the sill. Additional seals are needed between the ramp and the sides of the rectangular canal section. All other contacts are metal-to-metal and seal sufficiently against leaking. As mentioned, the weir requires only one hinge. The ramp section can be raised to wash sediments through the flume for quick cleaning.

For larger, non-portable versions of the Adjust-A-Flumes (Figure 3.50), a gear and cranking mechanism is attached to the flume. The ramp is hinged at both ends so that the ramp and sill are part of a hinged parallelogram in which the sill top remains horizontal at all sill heights and the ramp slope varies. These structures are discussed in more detail in Section 3.5.1.

3.4 Structures in Large Canals

The hydraulic theory of Chapter 6 is valid for any size channel. Hence, a weir or flume can be designed to measure the flow in natural streams and irrigation or

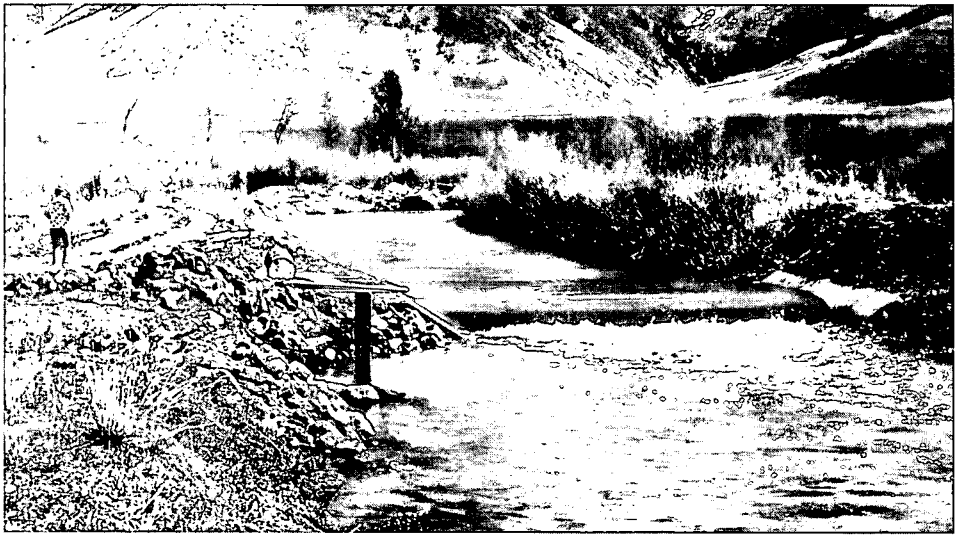


Figure 3.51 Trapezoidal weir for flow measurement in a natural stream (Lost River, Idaho).

drainage canals of all sizes (Figure 3.51) The construction of large weirs in main or lateral irrigation canals may, however, require special attention because of both structural and foundation problems. For example, adding excessive concentrated loads to previously lined sections of canal, which are likely to be on nearly saturated soils, invites settling and concrete cracking. In unlined canals, the addition of fill material to make a large sill, frequently in excess of 1 m high, also causes large concentrated loading. On previously constructed and operated unlined canals, the bottom soils are likely to be unstable. A solution that appears feasible is to remove much of the unstable material and replace it with easily drainable coarse material. The sill with ramps at both ends is then constructed as a compacted fill much like a low earth dam. The requirement for impervious backfill is less important than the requirement for minimum differential settlement. Therefore, coarse-textured, easily drained fill is desirable. The fill is compacted and shaped to the rough profile of the finished sill, and, where time permits, allowed to settle for a time period consistent with local soil conditions. Some sites may require an extensive foundation design. The compacted sill material is then covered with 100 to 150 mm (4 to 6 in.) of reinforced concrete. All slopes are on flat enough grades that flat-slab techniques can be used for economical concrete placement. Minor settling, including differential settling, can be periodically compensated for, by recomputing the calibration using either the computer model in Chapter 8 or the methods of Chapter 6.

Trapezoidal-shaped control

A large broad-crested weir was constructed on a main irrigation canal operated by the Salt River Project near Phoenix, Arizona (see Figures 3.52 and 3.53). The maximum canal capacity is 57 m^3/s (2000 ft^3/s) but normally the canal operates near 25 to 50 m^3/s (900 to 1800 ft^3/s). The dimensions of the sill and ramps were determined using the computer model in Chapter 8. The sill is 16.45 m (54 ft) wide, 1.20 m (3.9 ft)

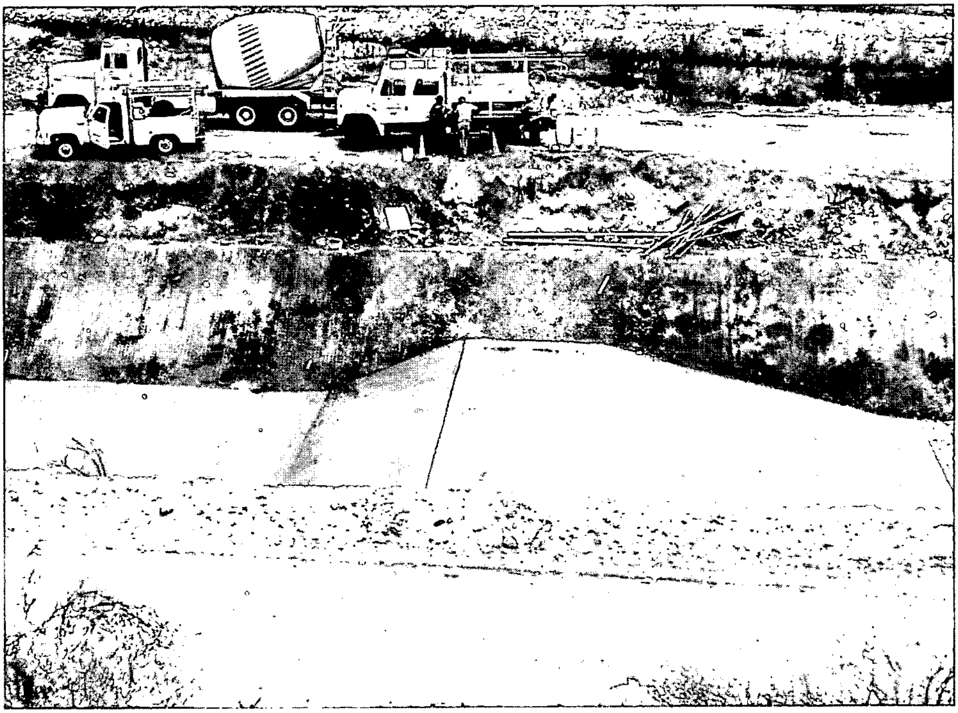


Figure 3.52 Large weir on Arizona Canal during construction.

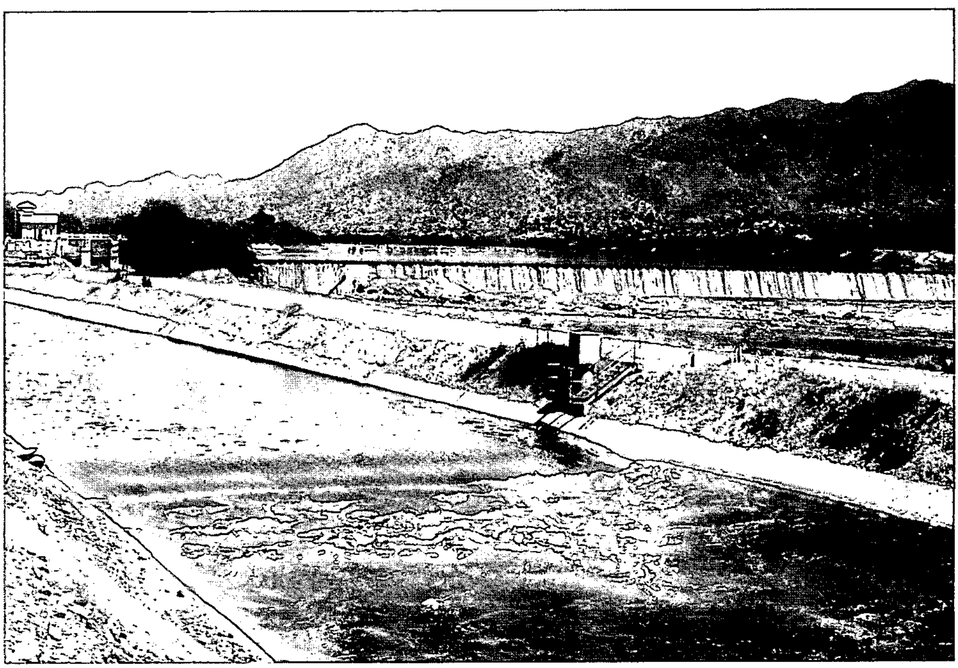


Figure 3.53 Large weir on Arizona Canal at approximately 34 m³/s (1200 ft³/s).

high, and 3.65 m (12 ft) long. The sidewall slope of the concrete canal lining and the throat section is 1.2:1. The bottom width of the lined canal is 13.10 m (43 ft). The approach ramp is 3:1, and the exit ramp is 6:1. A construction drawing of this weir is shown in Figure 3.54.

Triangular-shaped control

Triangular-shaped control sections allow accurate measurement over a large range of discharges. Thus they are well suited to large canals and natural streams. Several examples are shown in Figures 3.55 through 3.57.

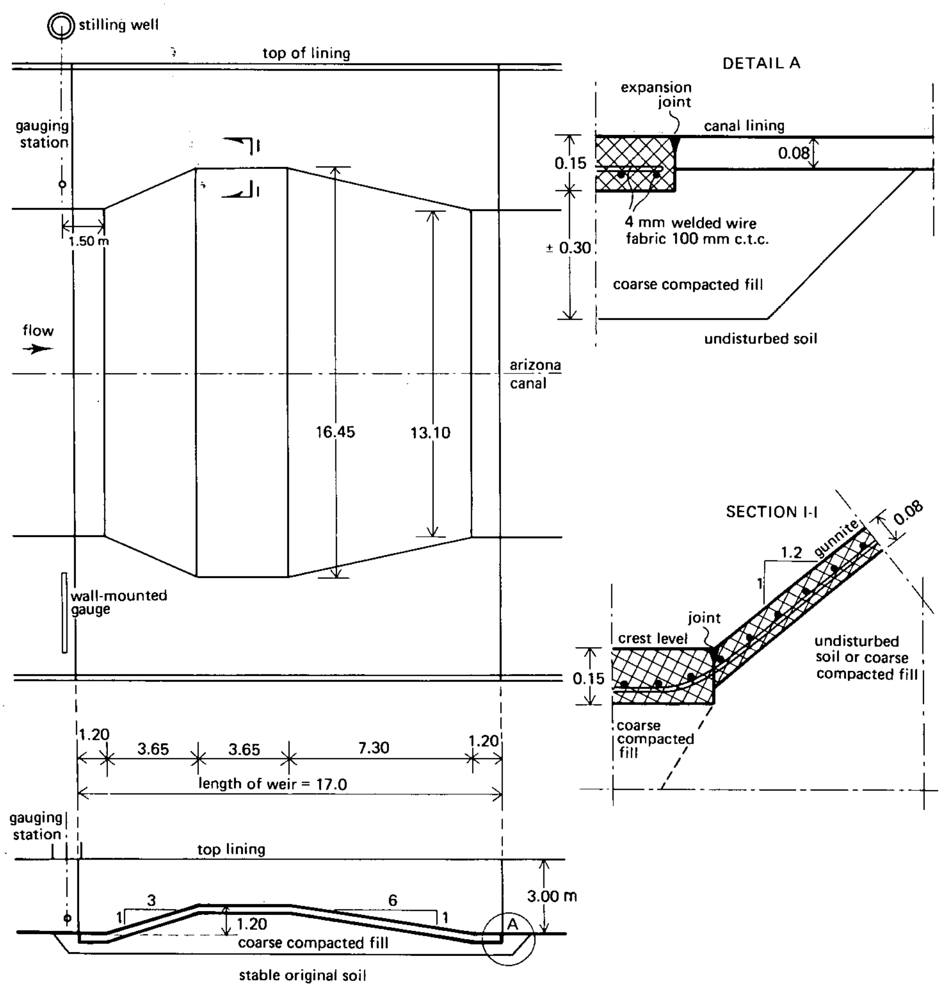


Figure 3.54 Construction drawing of large weir on the Arizona Canal. All dimensions are in meters.

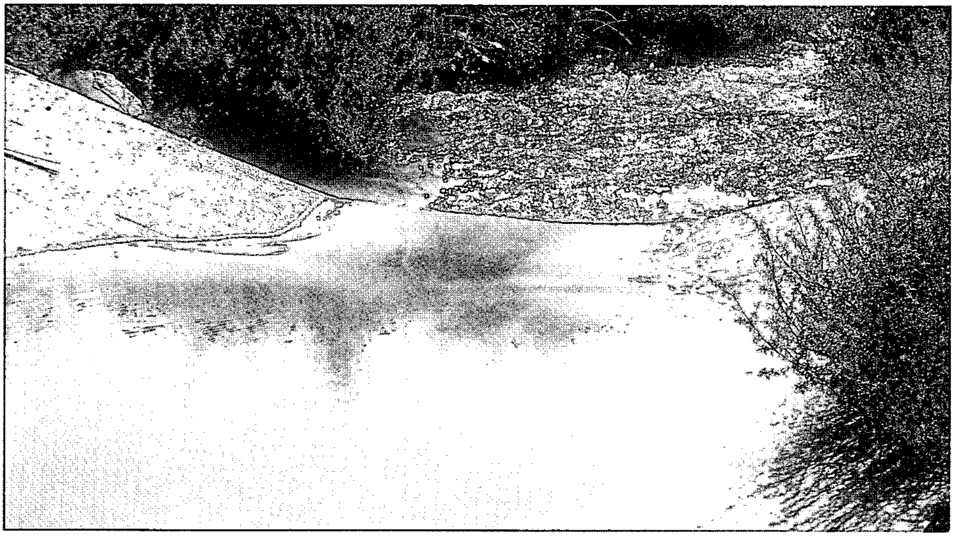


Figure 3.55 Triangular flume in natural drainage channel (Arizona).

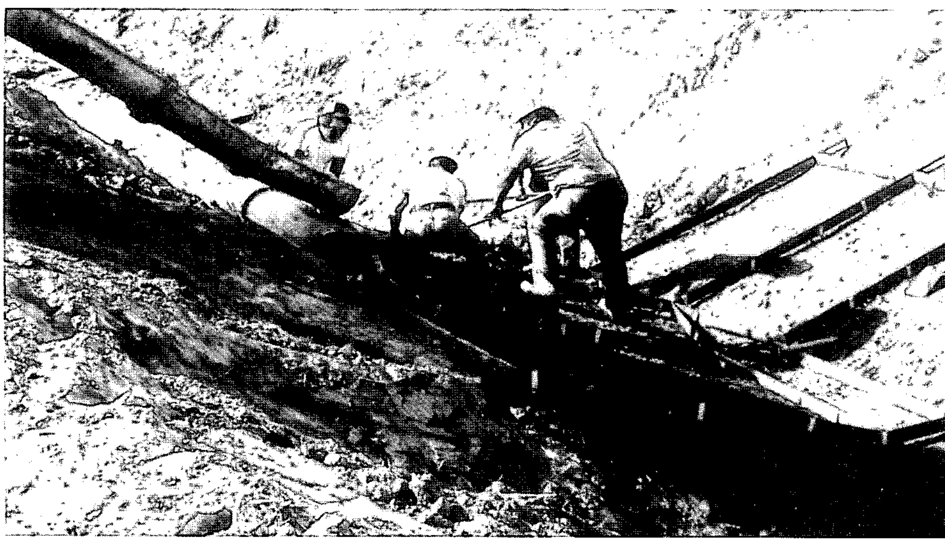


Figure 3.56 Concrete is poured for the downstream toe wall and the diverging transition of a flume with a triangular control section (Arizona).

The simplest method for constructing this type of flume is to (1) excavate unstable soil; (2) place compacted fill; (3) install stilling well and pipe and compact fill around pipes; (4) excavate two cut-off trenches to at least 0.60 m (2 ft) deep; (5) install 4 mm-diameter welded wire fabric of 100- to 150-mm (4- to 6-in.) spacing; (6) place forms to mark edges of concrete slabs; (7) pour concrete for both toe walls plus related entrance and diverging transitions and for the converging transition (Figure 3.56); (8) upon hardening of this concrete, remove forms and pour

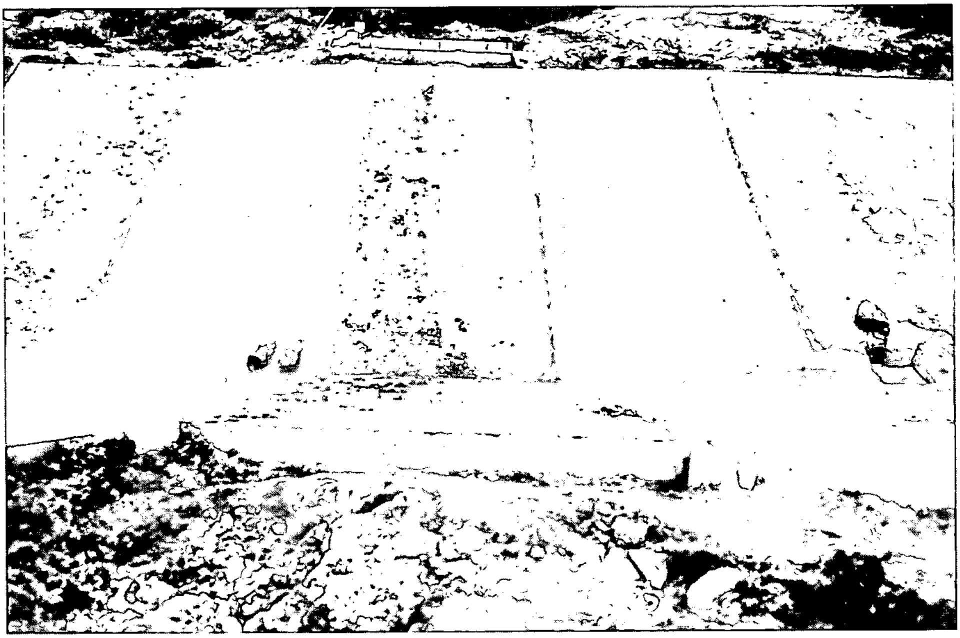


Figure 3.57 For this flume with a triangular control, concrete was placed in an alternating pattern. The upstream and downstream ramps have just been placed. The capacity of this flume is 23 m^3/s (800 ft^3/s) (Ohio).

concrete for approach channel and throat (Figure 3.57); (9) place riprap protection; and (10) finish stilling-well inlet pipe flush with concrete and install recorder. The triangular-shaped flume in Figure 3.36 has 3:1 sloping sidewalls in the throat. The designer, however, is free to choose different side slopes.

Complex-shaped control

When a large range of discharges are to be measured, triangular flumes are often used. However at low flows, the accuracy of these flumes is often low, due partly to the low head and imperfections in the triangular shape as it comes to a point, and also because of the high sensitivity to head-measurement errors (owing to the large exponent, u, in the head-discharge equation. See Table 2.2 and Section 2.9). The accuracy of these structures can be improved by creating a complex shape that essentially widens the lower part of the triangle to make a trapezoidal shape, as shown in Figure 3.58). Such a structure effectively becomes two flumes – a trapezoidal flume for small flows and essentially a triangular flume at large flows. Such structures have been useful for canals that carry irrigation water during one season and small municipal flows during the remainder of the year. (Figure 3.59).

Construction of these weirs is similar to the large triangular flumes described above. Some additional care must be taken to properly form the throat and to provide a transition from the trapezoidal throat to the larger converging transition for the triangular section.

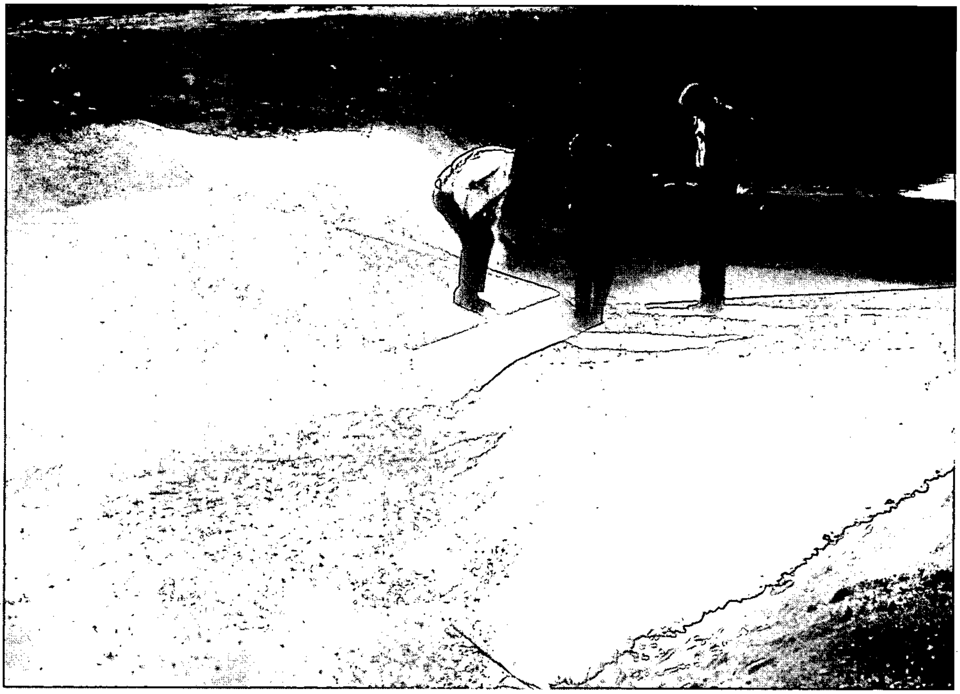


Figure 3.58 Complex trapezoidal flume on the Roosevelt Water Conservation District (Arizona).

Figure 3.59 Complex trapezoidal flume on the Croke Canal (Colorado).

3.5 Movable Weirs

Movable weirs have proven their value for over 80 years in irrigated areas where flow over an off-take structure is variable because of changing crop rotations and water requirements during the growing season (Butcher 1921 and 1922; Romijn, 1932). The layout for a typical offtake is shown in Figure 3.60, with dimensions shown in proportion to the maximum energy head, H_{1max}.

3.5.1. Movable weir types

Depending on the water depth in the approach canal to the weir and on the maximum head required over the weir crest, three basic types of movable weirs can be distinguished:

1. Bottom-gate type–the weir crest is sealed behind a movable bottom gate or a fixed wall.
2. Bottom-drop type–the weir crest is sealed behind the vertical back-wall of a drop in the canal bottom.
3. Pivoting type–the weir crest and ramp are part of a parallelogram moving between parallel sidewalls.

Bottom-gate type

This movable weir consists of two interconnected gates and a weir crest that are mounted in a steel guide frame (see Figure 3.61). The movable weir is connected to a

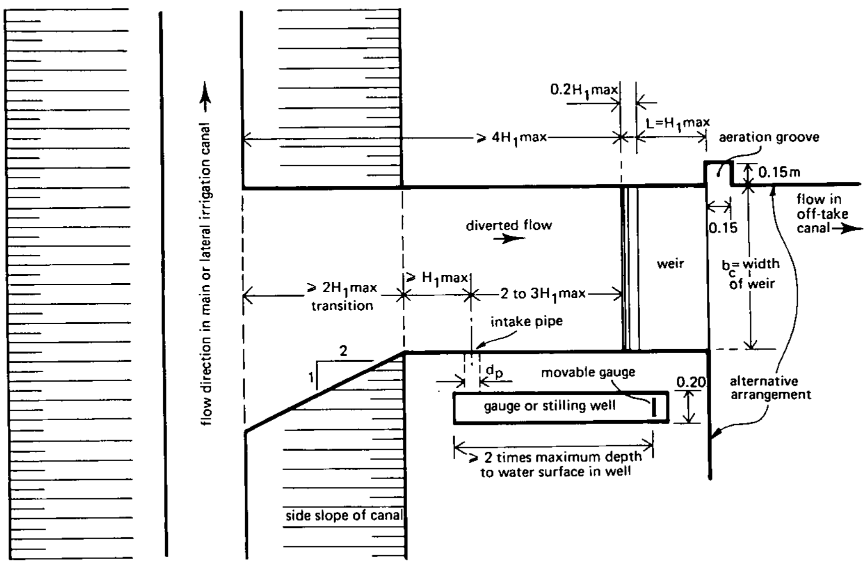


Figure 3.60 General layout of an offtake weir (Bos 1989).

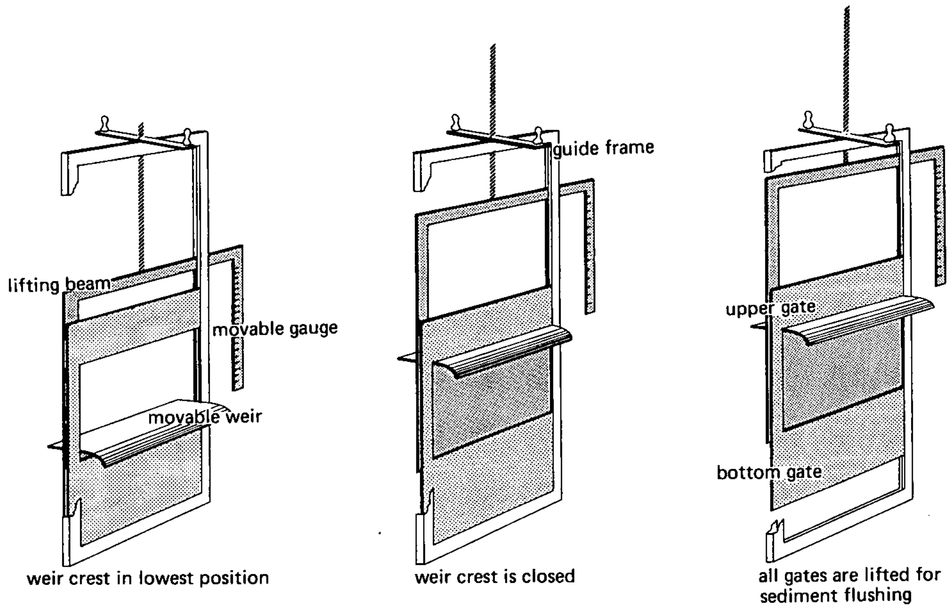


Figure 3.61 Movable weir with bottom gate (Bos 1974).

horizontal lifting beam by means of two steel strips. It has a vertical travel of H_{1max} and can be padlocked in place at all desired levels. The bottom gate is blocked in place under operational conditions and acts as a bottom terminal for the movable weir. The upper gate is connected to the bottom gate by means of two steel strips that are enclosed by the frame grooves. This gate acts as a stop for the movable weir.

As mentioned, the upper gate (and thus the bottom gate) is locked in place during normal flow conditions. However, to flush sediments that have collected upstream of the structure, both gates can be unlocked and raised by moving the weir crest upwards. After the flushing operation, the gates are pushed in place again by lowering the weir crest. To discourage misuse of the structure, the maximum flow capacity beneath the lifted bottom gate must be less than the flow over the weir in its lowest position. For this to occur, the travel of the upper gate is restricted so that the bottom gate cannot be lifted higher than $0.5H_{1max}$ above the approach canal bottom.

The weir is placed between the vertical walls of a short approach canal, the walls being flush with the groove frame. The upstream head over the weir crest h_1 is measured in this approach canal at a distance of between 2 and 3 times H_{1max} upstream from the weir face. The dimensions of the approach canal should comply with the requirements indicated in Figure 3.60. The 2:1 flare of the right-hand abutment must also be used on the left-hand side if the centerline of the weir structure is parallel to or coincides with the centerline of the undivided supply canal (in-line structure) or if the water is drawn directly from a reservoir or storage basin.

If several movable weirs are combined in a single structure, intermediate piers must be provided so that one-dimensional flow is preserved over each weir unit, allowing

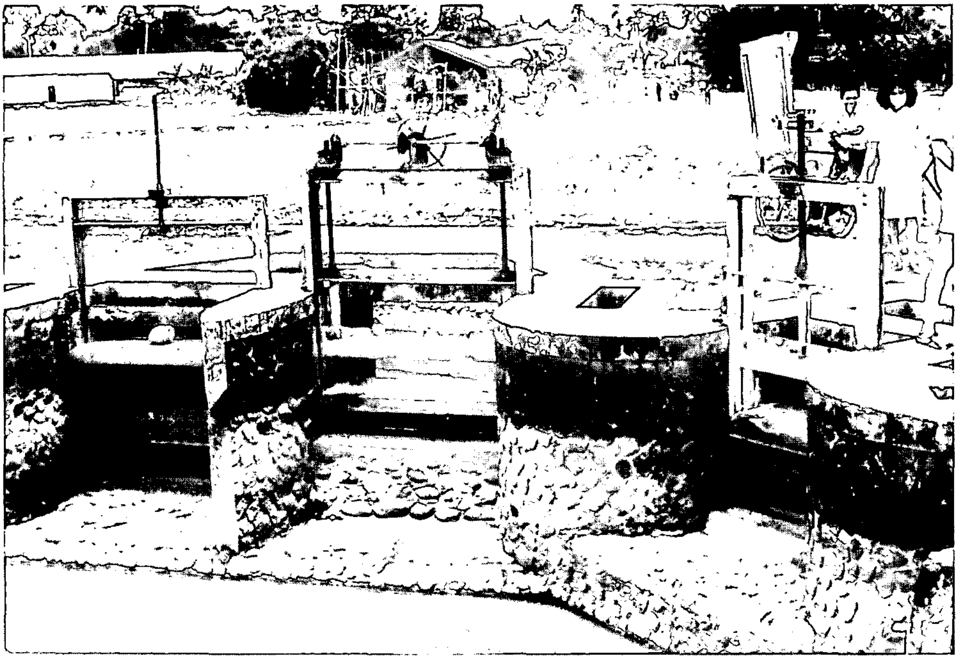


Figure 3.62 Several different movable-crest weirs (Indonesia).

the head h_1 to be measured independently for each unit (see Figure 3.62). The parallel section of the pier should therefore commence at a distance of H_{1max} upstream from the head measurement station and extend to the downstream edge of the weir crest. Piers must have streamlined noses, for example, semi-circular. To avoid large velocity differences over short distances, the thickness of the pier should be equal to or more than 0.65 H_{1max}, with a minimum of 0.30 m (1 ft).

Bottom-drop type

With this movable weir, the crest is sealed behind the vertical backwall of a drop in the canal bottom. The weir, illustrated in Figure 3.63, can be raised and lowered by a self-sustaining hand gear. It can be raised high enough to cut off the flow at full supply level in the supply canal. When the weir is raised, leakage is negligible. Under normal operating conditions, the travel of the weir must be limited so that its crest cannot be lower than $p_1 = {}^1/_3 H_{1max}$ above the approach canal bottom. In its highest position, the bottom seal must not be disconnected (see Figure 3.63).

In the shallow approach canal, flow velocities will be rather high, usually preventing the accumulation of sediments in this section. If, however, the alternative canal bottom shown in Figure 3.63 is used, sediments may accumulate. If so, these should be removed periodically. The paragraphs discussing the approach canal and intermediate piers for the bottom-gate type weir also apply to the bottom-drop-type weir.

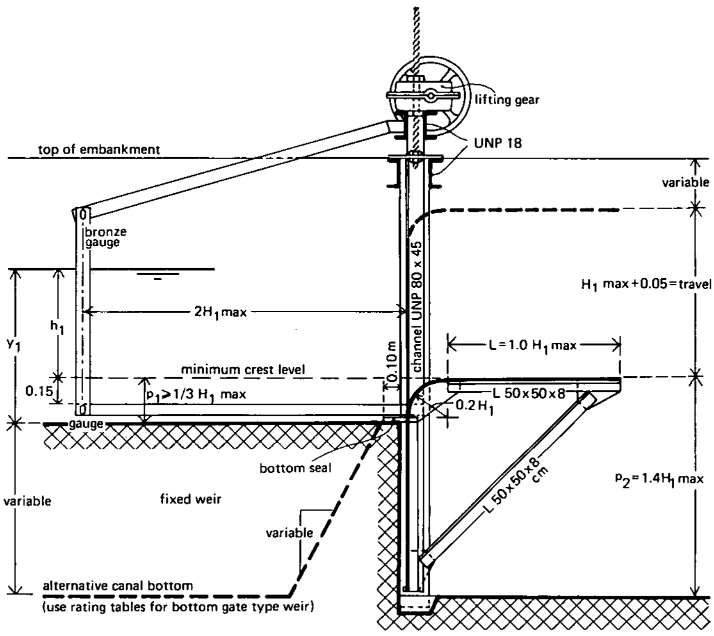


Figure 3.63 Longitudinal section through a bottom-drop type movable weir.

Pivoting box-frame type

The pivoting weir consists of a rectangular box-frame containing the weir sill and the sides of the control section. The sidewalls of the control section extend a distance $2H_{1max}$ upstream from the leading edge of the sill to allow the mounting of a staff gage. The box-frame moves up and down within a rectangular canal section. The method by which the box-frame is moved depends on the size of the structure. A portable version is discussed in Section 3.3.3 Large size permanent versions (see Figure 3.50) of the movable weir have been designed and installed in the Northwest United States, based on modification of an overshot gate. These have the bottom as well as the top of the ramp hinged. The adjustable box-frame is part of a hinged parallelogram in which the sill top remains horizontal at all sill heights and the ramp slope varies from nearly zero to about 1:1 (horizontal to vertical) as the parallelogram is adjusted to accommodate an existing flow depth in the canal. The ramp is sealed at the bottom, along the sides, and at the hinge with the leading edge of the sill. The latter seal rounds the ramp-to-sill transition so that the steep (1:1) ramp slope does not cause flow separation. The box-frame moves within a rectangular canal section. The hoist cables run in between the box-frame and this rectangular (approach) canal section.

3.5.2 Groove arrangements

The groove arrangement for weirs with a width between 0.30 and 1.50 m (1 and 5 ft) can be rather simple; the gates and related hoist strips and profiles move in narrow grooves with metal-to-metal water sealing. Water leakage through the horizontal terminals is prevented by using rubber seals.

The groove arrangement and terminal seals for the movable weir with bottom gate shown in Figure 3.61 are given in Figure 3.64. As shown, the groove profiles are flush with the sidewalls of the approach canal. Also, the 8 x 50 mm hoist strips fit entirely in the 10-mm-wide grooves. As a result, the width of the weir equals the width of the approach canal. The crest moves in between these concrete or brickwork sidewalls with a clearance of about 5 mm (0.2 inch). These clearances have no detectable influence on the accuracy of the flow measurement.

As stated in Section 3.5.1, movement of the upper and bottom gates is restricted to minimize the accidental loss or unauthorized passage of water. For this purpose, an 8 x 60 mm strip is welded to the top corner of the upper gate. This strip fits in a related groove and terminates 0.20 m (8 in.) below the top corner construction of the frame (bottom gate is closed). A padlocked blocking wedge fits into a hole (10 x 40 mm) through the frame directly above this strip. If the wedge is removed, the bottom gate can be opened 0.20 m (8 in.), which allows less flow through this opening than over the weir, to discourage misuse.

For weirs with a bottom gate, the vertical weir gate is usually placed so that about half the weir crest is upstream from the gate plane to minimize the torque in this gate near the grooves. Weirs that move behind a bottom drop, however, often have the vertical weir gate at the upstream end of the rounded weir nose (see Figure 3.63) so that somewhat more stiffness is needed along the groove edge of the gate. This stiffness can be obtained by welding the gate to an angle iron, whose angle then moves in a groove, as illustrated in Figure 3.65. The angle iron also serves to raise and lower the weir. Figure 3.65 also shows an example of a bottom terminal and seal for a weir with a bottom drop. The rounded lower edge of the bottom gate is important; it allows the gate to be lowered again after it is raised above the approach canal bottom (for maintenance).

If movable weirs need to be wider than 1.50 m (5 ft), the profiles used in the grooves must become heavier to carry the greater hydraulic forces and the forces needed to move the weir. An example of a suitable groove arrangement is shown in Figure 3.66. The bottom seal for weirs between 1.5 and 4.0 m can be as illustrated in Figure 3.64 and 3.65.

Although Section 3.5.3 gives construction details for a 1.50-m (5-ft) wide weir, we recommend consulting a mechanical engineer if any alterations must be made to the given drawings.

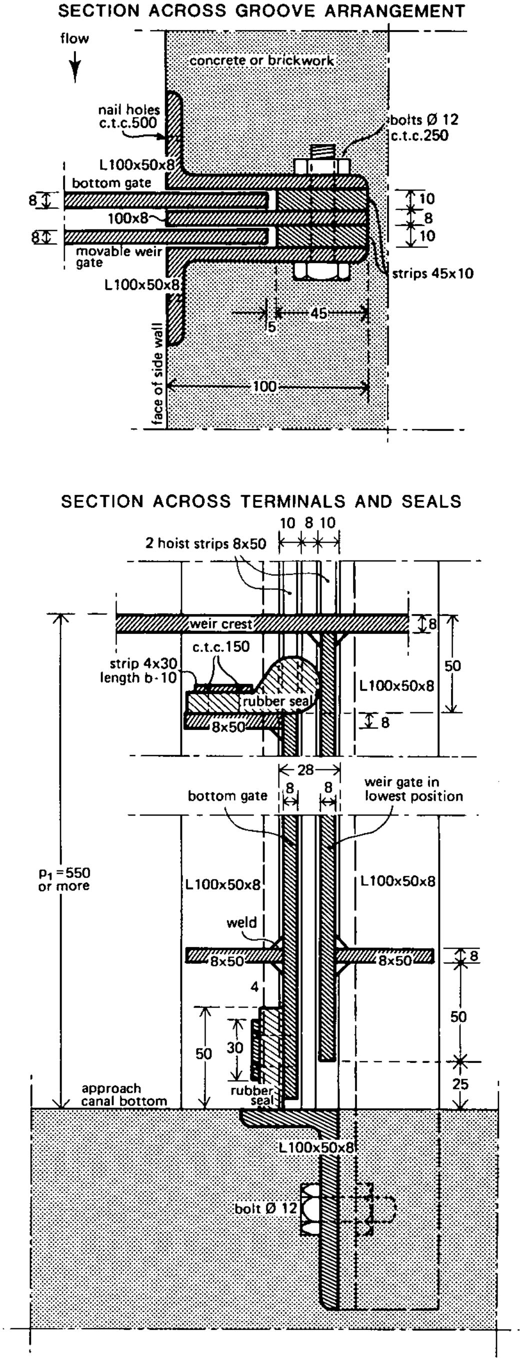


Figure 3.64 Sections showing groove arrangements, terminals and seals for movable weir with bottom gate (dimensions in mm).

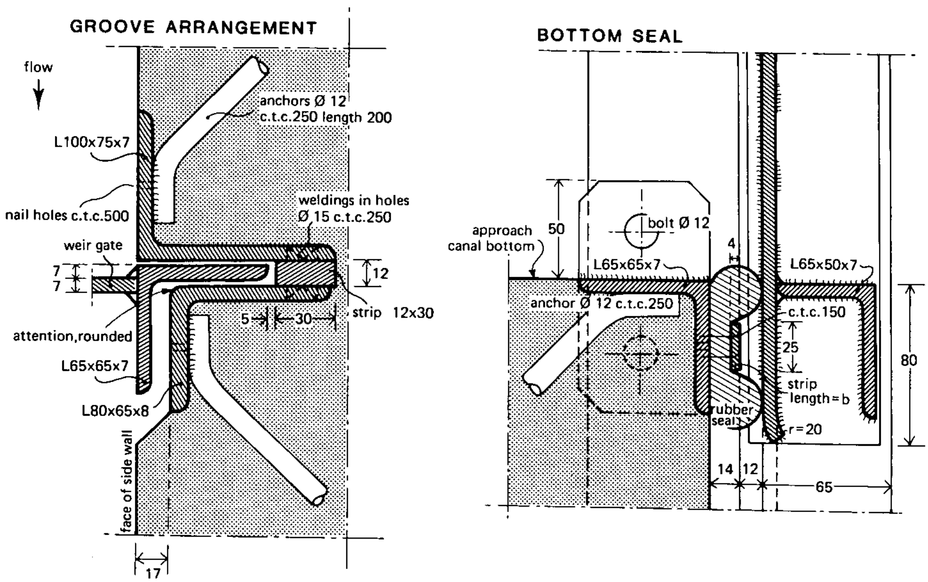


Figure 3.65 Sections showing groove arrangement and bottom seal for weir with bottom drop (dimensions in mm).

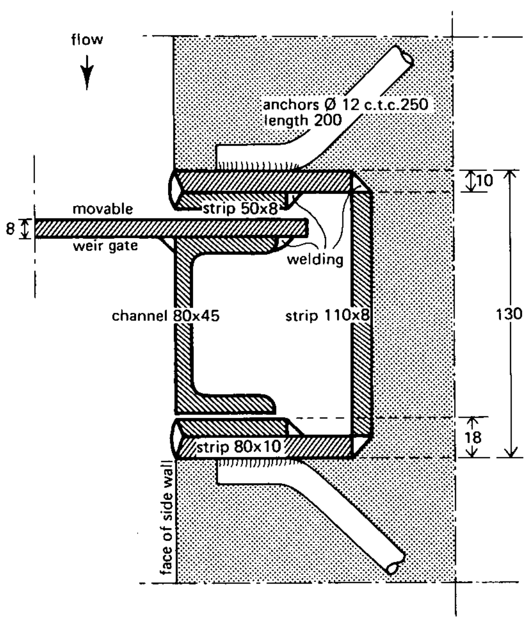


Figure 3.66 Sections showing groove arrangement for weirs between 1.5 m and 4.0 m wide (dimensions in mm).

3.5.3 Lifting devices

Force needed to lift movable weir

Lifting devices for movable weirs vary from the simple gear and chain lift to an elaborate electrically operated unit with remote level control. The type selected depends on the gate size, the maximum hydraulic head under which the gate will operate, the type of canal in which it is installed (farm or project), the speed of gate travel, and the method of operation.

A variety of devices are available commercially to meet the needs of many applications. Some devices that can be used on weirs narrower than 1.50 m (5 ft) are described in Section 3.5.3.

In order to operate any movable weir, the lifting device must overcome several forces. These include the weight of the weir and gate(s), weight of the hoist beam and stem, the frictional resistance caused by the hydraulic pressure against the gate(s), and the weight of the water above the weir crest. To determine the lifting force required to move a weir, the following equation is used:

$$F = fTb_c + W + \rho g h_1 b_c L \tag{3.2}$$

where

F = lifting force required (N),
f = friction coefficient (dimensionless),
b_c = weir width (m),
W = weight of movable weir (and gates) plus hoist strips, beam and stem (N),
g = acceleration due to gravity (9.81 m/s^2),
ρ = mass density of water (kg/m^3),
h_1 = head over weir crest (m), and
T = the area of the shaded triangle or trapezoid in Figure 3.67 (kg/s^2).

Figure 3.67 shows the hydraulic pressure on the gates and weirs in four extreme weir positions. Figure 3.67A illustrates the position for which F becomes maximum with a bottom-gate-type weir. This is because $T = 0.5\rho g y_1^2$ and because all gates and the weir must be lifted for sediment flushing. In Figure 3.67D, the force F becomes maximum for the bottom-drop-type weir because (with $p_1 = 0.33H_{1max}$) the value of $T = 0.4\rho g h_1^2$ and because both other terms of Equation 3.2 have a maximum value. Two different friction coefficients are used in Equation 3.2. The first is for unseating the gates from their locked position. An approximate value of $f = 0.6$ has been determined to be conservative. After the gate has been moved upward a small distance, this high initial friction is no longer present and the coefficient drops to approximately $f = 0.3$. These values are approximate and will vary depending on how long it has been since the gate has been moved, whether the gate is partially covered with silt or sand, and whether the contact faces are lubricated or dry.

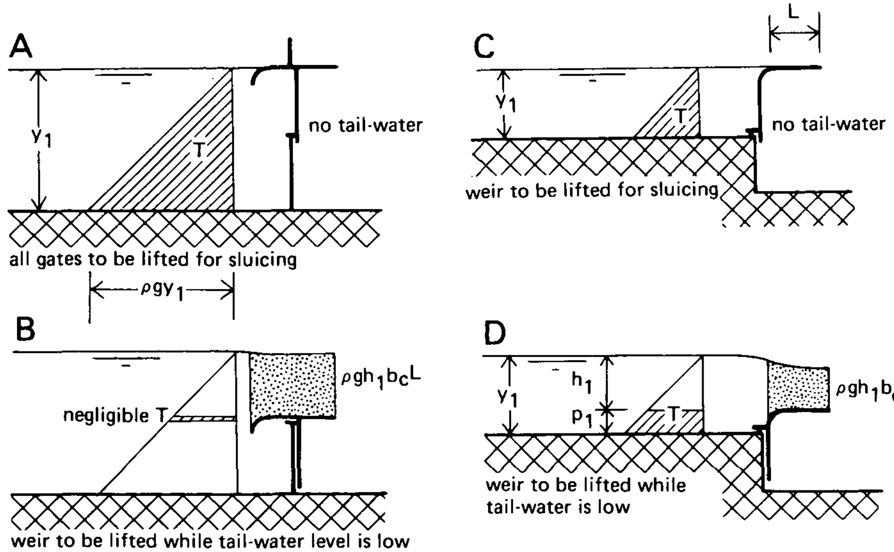


Figure 3.67 Examples of hydraulic pressure on movable weirs.

Types of lifting devices

After determining the force F, the next step is to select a combination of lifting device and related stem. For the smaller lifting forces, the gear lift and jack lift are very suitable. The advantage of these low-cost devices is that they can be welded together in most machine workshops.

Gear lift

The gear lift was developed by Fullerform[1] (1977) to provide a low-cost device for the precise adjustment of small gates. Constructed of heavy-gage steel to the dimensions shown in Figure 3.68, it will provide trouble-free service on the 0.30 m (1-ft) wide weir. If the handle has a length of 6 times the radius of the available gear wheel, and a one-hand pull of 120 N (about 25 lb) is exerted, this device can produce a lifting force, F, of about 700 N (150 lb).

Jack lift

The jack lift (Figure 3.69), if constructed using the dimensions shown in Figure 3.70, will also provide a reliable method for the accurate adjustment of small weirs. Usually, the jack handle is 6 to 7 times the distance between its hinge point and the lift stem. Because of this leverage, the lifting force is about 750 N (160 lb) if a pull of 120 N (25 lb) is applied to the handle. With this device, however, it is entirely possible to use two hands and increase the pull by using body weight. As a result, lifting forces of up to $F = 3000$ N (650 lb) are feasible if a weir has to be moved only occasionally.

[1] Trade and company names are shown for the benefit of the reader and do not imply endorsement or preferential treatment of the company or product listed.

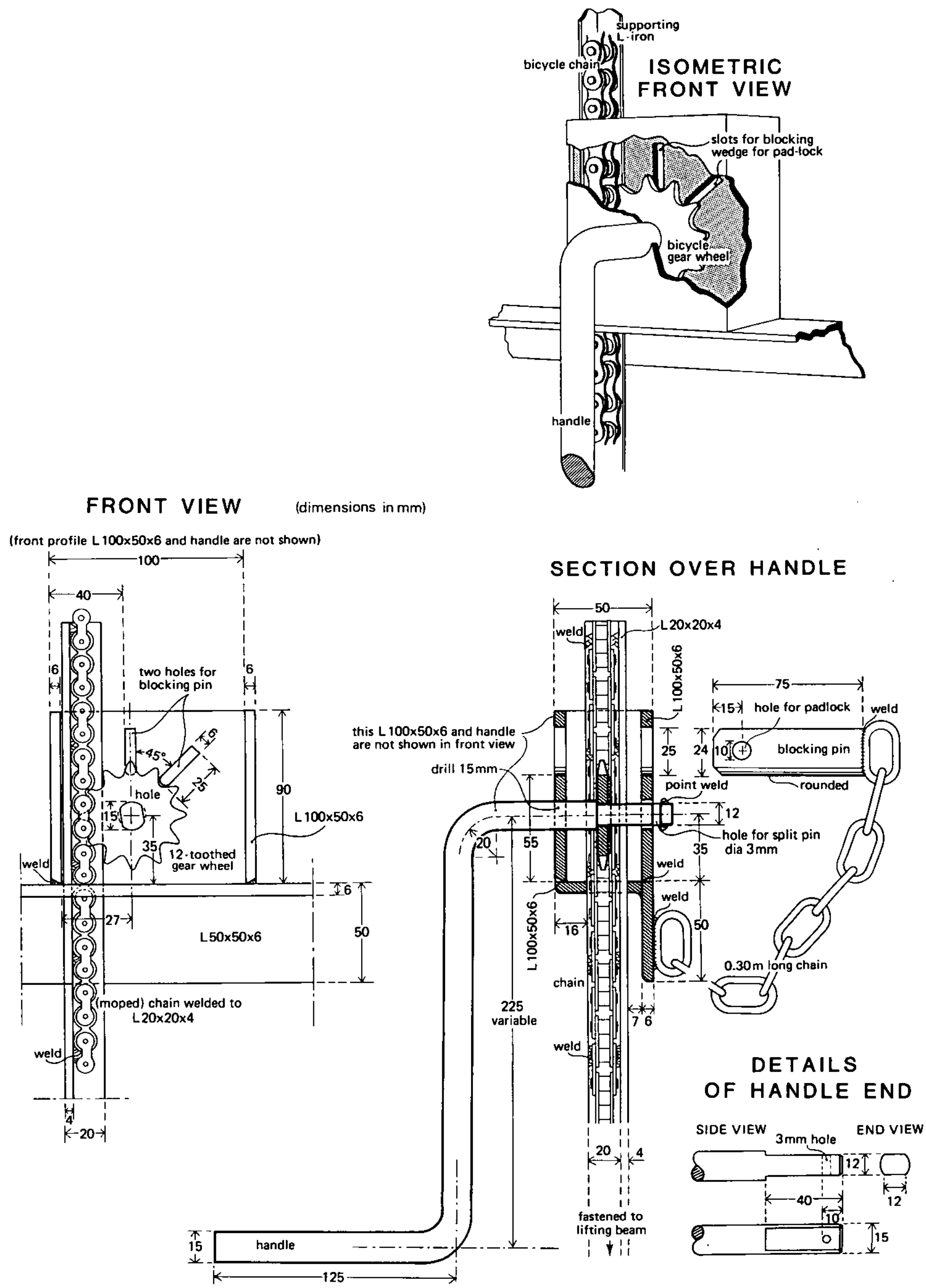


Figure 3.68 Construction drawing for gear lift (dimensions in mm).

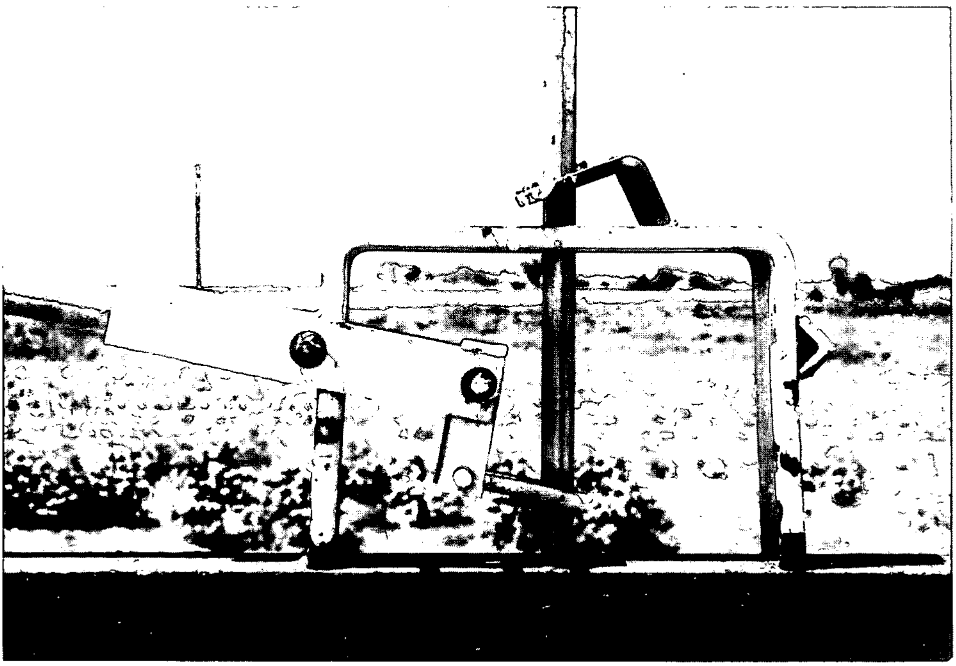


Figure 3.69 Jack lift.

Handwheel lift

For weirs that are raised or lowered regularly and require a lifting force greater than 750 N (160 lb), a handwheel lift is recommended. In practice, this means that weirs with the commonly used crest length of L = 0.50 m and a width greater than the minimum width of b_c = 0.30 m should be equipped with a handwheel.

Many kinds of handwheels with either cast iron or bronze lift nuts are commercially available. Because of their higher corrosion resistance, greater durability, and higher efficiency, we strongly recommend the use of bronze lift nuts rather than cast iron lift nuts. To select a suitable handwheel and lift stem combination, consult the manufacturers for detailed information. The data provided in Figure 3.71 may be used for preliminary design.

To avoid unauthorized changes in the weir crest or division board position, a length of chain should be welded to the wheel support for use in padlocking the wheel. If the handwheel is removed upon operating the weir or board, this chain must make one turn around the wheel seat of the lift nut so that no other tool can be used to adjust the structure.

Weirs having a width of b_c = 1.50 m (5 ft) or less can be moved with a single lifting device placed above the center of the weir on the top beam(s) of the frame (Figure 3.72). An example of the top beam arrangement is given in Figure 3.73. The top corner is bolted to enable removal of the entire weir for maintenance.

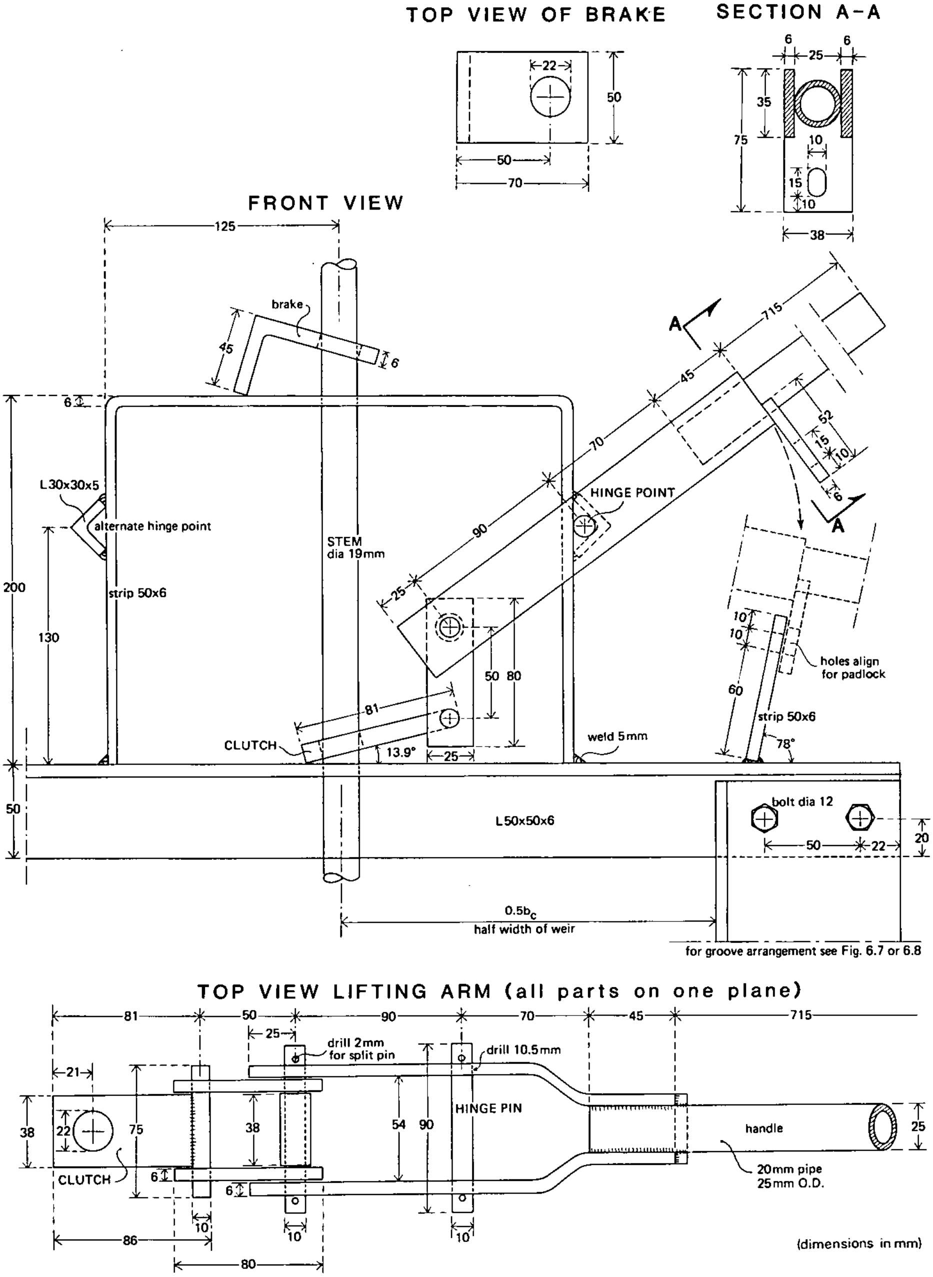
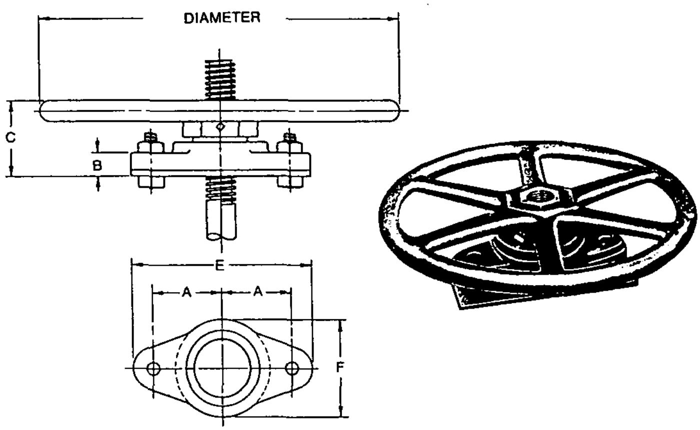


Figure 3.70 Construction drawing for jack lift (dimensions in mm).



Handwheel diameter mm	Stem diameter mm	Turns of handwheel to move gate 25 mm	Lifting * force in N for 120 N pull on wheel	Dimensions in mm					
				A	B	C	E	F	bolt diameter
250	22	2.5	4300	70	16	80	180	95	12
	29	2.5	3400						
360	22	2.5	5800	70	16	80	180	95	12
	29	2.5	4800						
450	29	2.5	5800	90	29	100	230	120	16
	38	2.0	4300						
610	29	2.5	7250	90	29	100	230	120	16
	38	2.0	5800						
760	29	2.5	8700	90	29	100	230	120	16
	38	2.0	7250						

Figure 3.71 Handwheel dimensions (adapted from ARMCO Steel Corporation 1977).

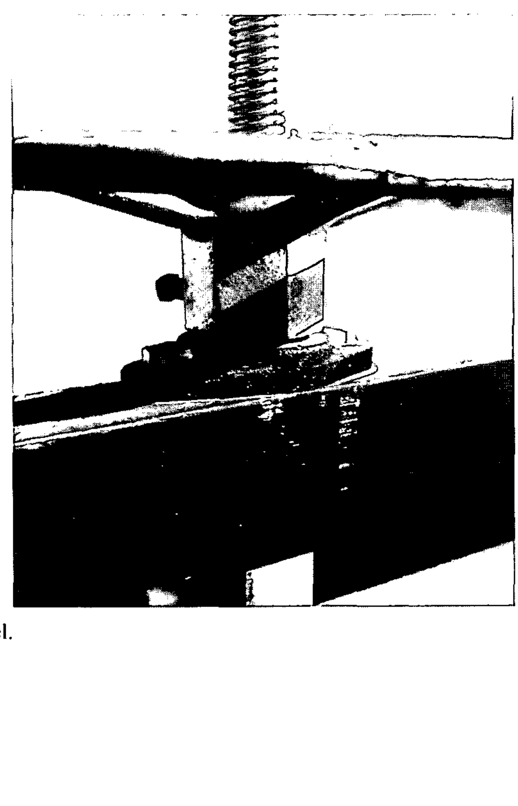


Figure 3.72 Handwheel.

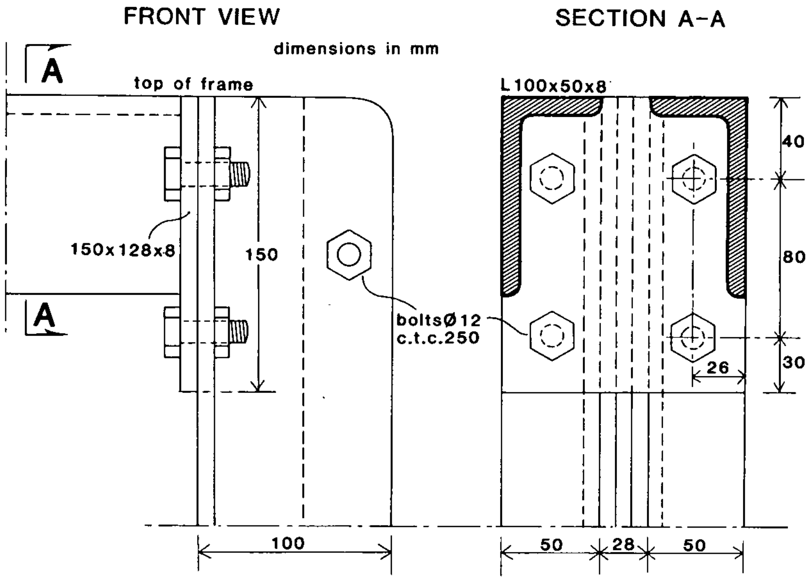


Figure 3.73 Detail of top corner of frame for weir with bottom gate (see Figure 3.64 and 3.65).

3.5.4 Sample construction drawing

As discussed in Chapter 1, the bottom and sidewalls of the structure may be constructed from brickwork, reinforced concrete, or a combination of these materials. As illustrated in Figure 3.62, masonry with natural stone gives good results. The choice of the construction material should be based on factors such as the availability of materials and the skill of laborers, both of which influence the construction cost of the structure. From a hydraulic viewpoint, there is no preference for a particular construction material.

The movable weir is well suited to regulating and measuring the flow that is taken from a main or lateral irrigation canal. In the example shown in Figure 3.74, we use reinforced concrete since this is the most common construction material. The structure accommodates a standard size weir with bottom drop (L = 0.50 m). If the approach canal bottom is lowered, however, a weir with a bottom gate can be installed in the concrete structure.

3.6 Flow Divisors

Many of the world's older irrigation systems are cooperative stock companies in which the individual water users have rights to proportional parts of the water supply furnished by the canal system, the division being in proportion to the stock owned in the canal company. Under this system it was often considered unnecessary to measure the flow very accurately; it was only important for each user to get their proportionate share. This led to the use of a variety of divisors and division boxes (Cone 1917; Neyrpic 1955; Cipolletti 1886). Our attention will be confined to divisors that can be used for accurate flow measurement and for making the intended division of the water.

A flow divisor consists of a broad-crested weir with rectangular control section and a partition board. The partition board has a sharp (less than 10 mm thick) upstream edge. If the intended division of flow remains near constant, the partition board can be a fixed steel plate, otherwise the board is movable.

- In the upstream reaches of an irrigation system, where water needs to be divided over canals serving areas with about the same cropping pattern, divisors with a fixed board are common. Depending on the number of canals, one or more boards can be used (Figure 3.75). The board starts at the downstream edge of the weir crest and continues downstream for a distance of H_{1max}. The board divides the overfalling nappe without interfering with the shape of the nappe.
- In the middle and downstream reaches of an irrigation system, the division of water into lateral canals needs to be adjusted to match changing downstream requirements. The flow divisor with movable portition board is the only structure that can accomplish this task by adjusting only one "gate". For proportional flow the board is adjusted so that

$$\frac{b_{c,o}}{b_c} = \text{intended part of } Q \qquad 3.3$$

where $b_{c,o}$ is the distance (width) from the center of the sharp edged board to the side of the control section, and b_c is the width of the weir (Figure 3.76). The board can be kept in this position until the intended division of water changes. If flow into the lateral off-take needs to equal Q_o, while the incoming flow, Q, changes, the width $b_{c,o}$ should be adjusted so that

$$b_{c,o} = b_c \frac{Q_o}{Q} \qquad 3.4$$

For a given upstream head, the incoming flow, Q, can be read from a rating table or calculated with the head-discharge equation (see Chapter 8). If board movement is automated, a local control device can activate an electric motor to adjust the board to the intended value of $b_{c,o}$ and thus maintain constant flow into the off-take.

The division board is constructed as a V-shaped box, being closed at all sides and hinged to the division wall. The transition between the board and the wall is streamlined in such a way that no shock waves are created. The board moves with a narrow (about 1-cm) spacing behind the weir crest, with the position adjusted by a cable and pulley system (Figure 3.77 and 3.78). No seals are required under the board and near the hinges. Because there is usually little head difference across the board,

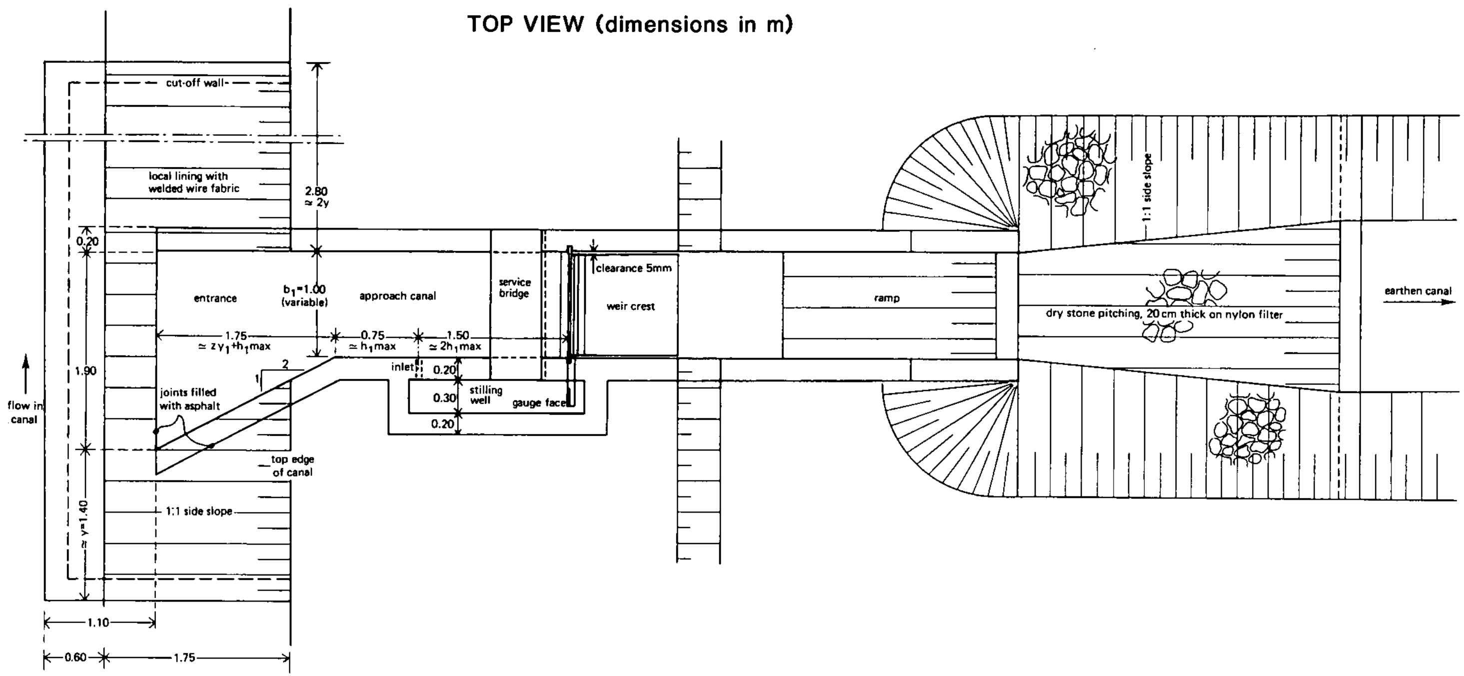


Figure 3.74 Sample offtake structure.

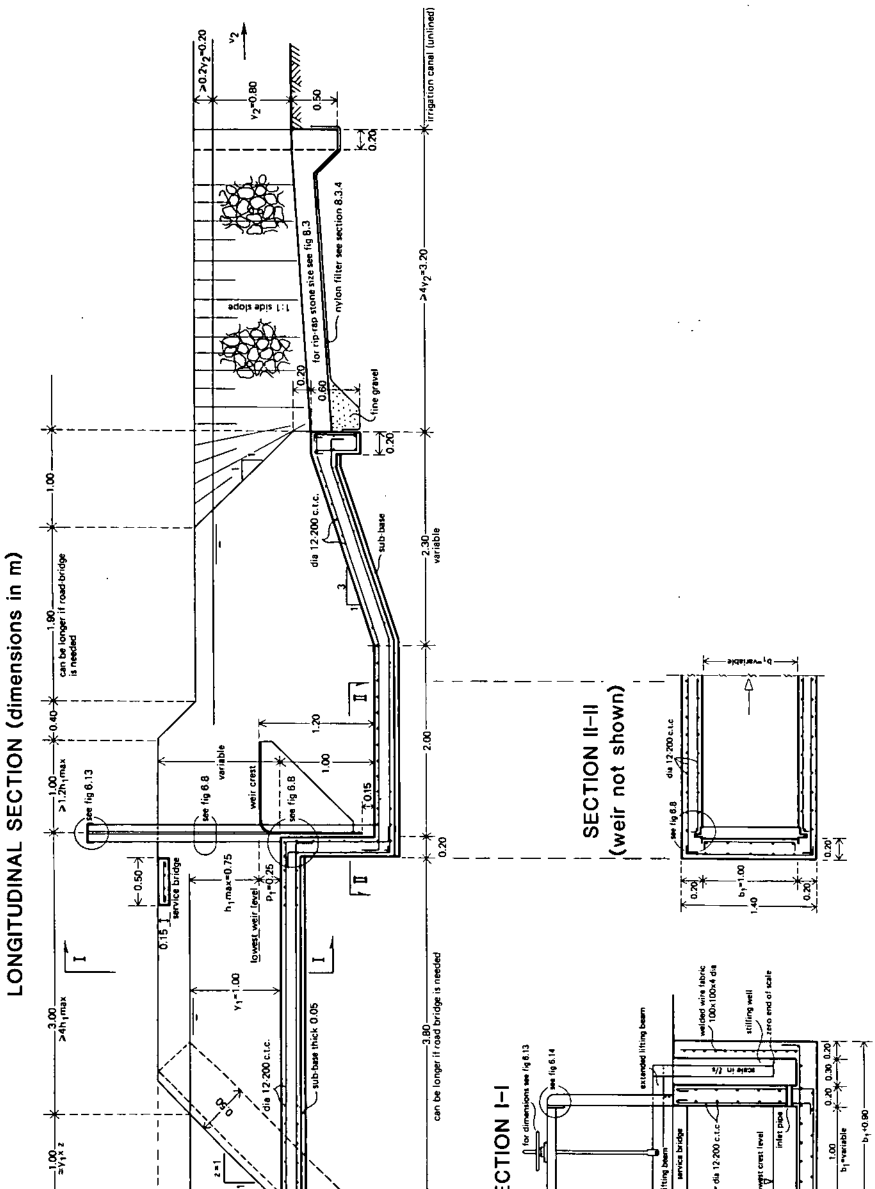


Figure 3.74 (continued)

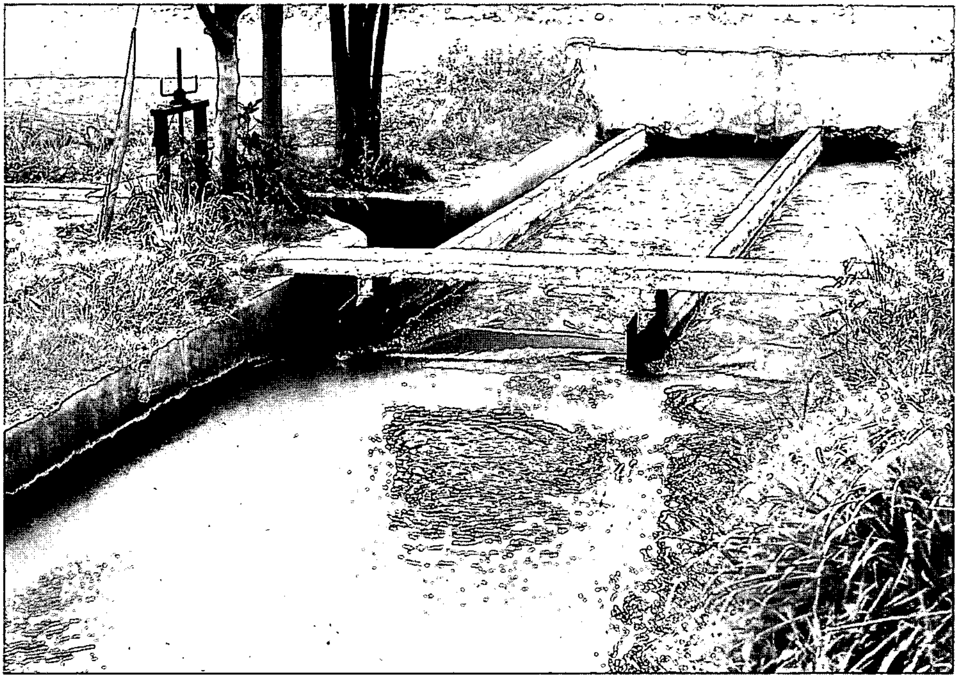


Figure 3.75 Flow divisor with fixed boards in an irrigation canal (Argentina).

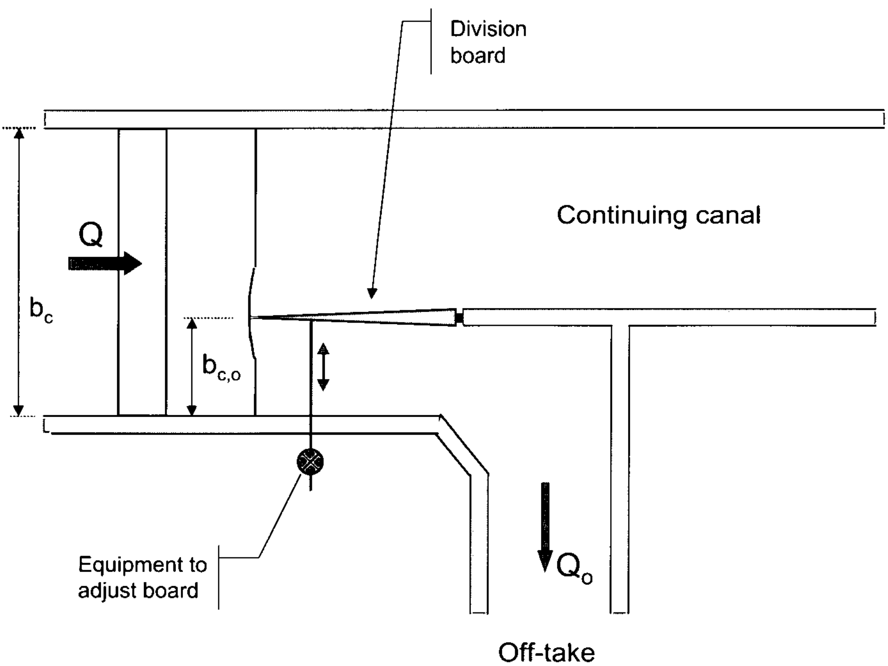


Figure 3.76 General layout of a flow divisor.

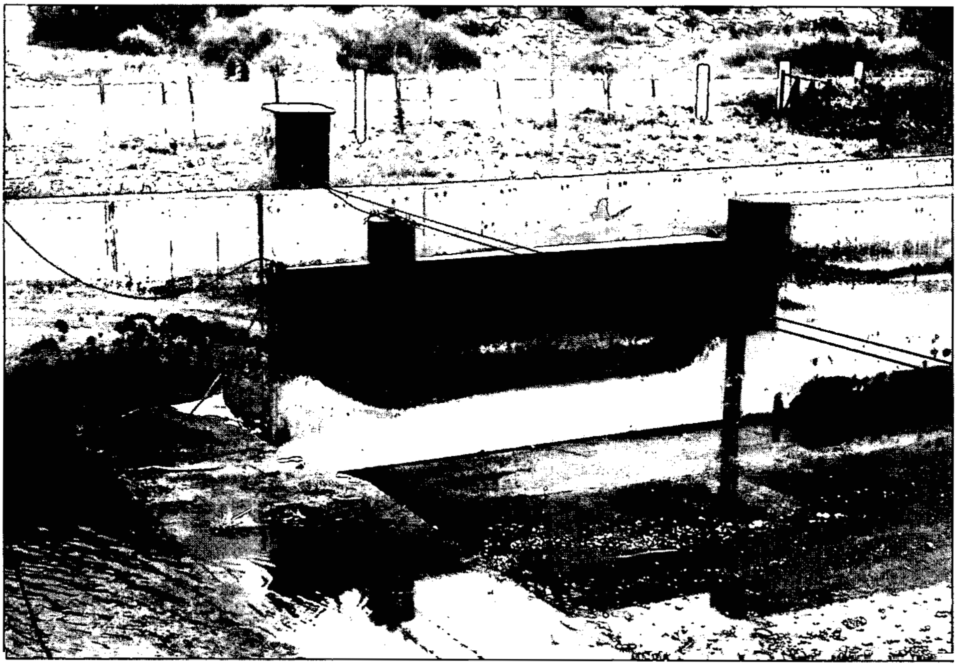


Figure 3.77 Movable flow divisor behind broad-crested weir showing division door. The door position is adjustable with a cable. (Argentina)

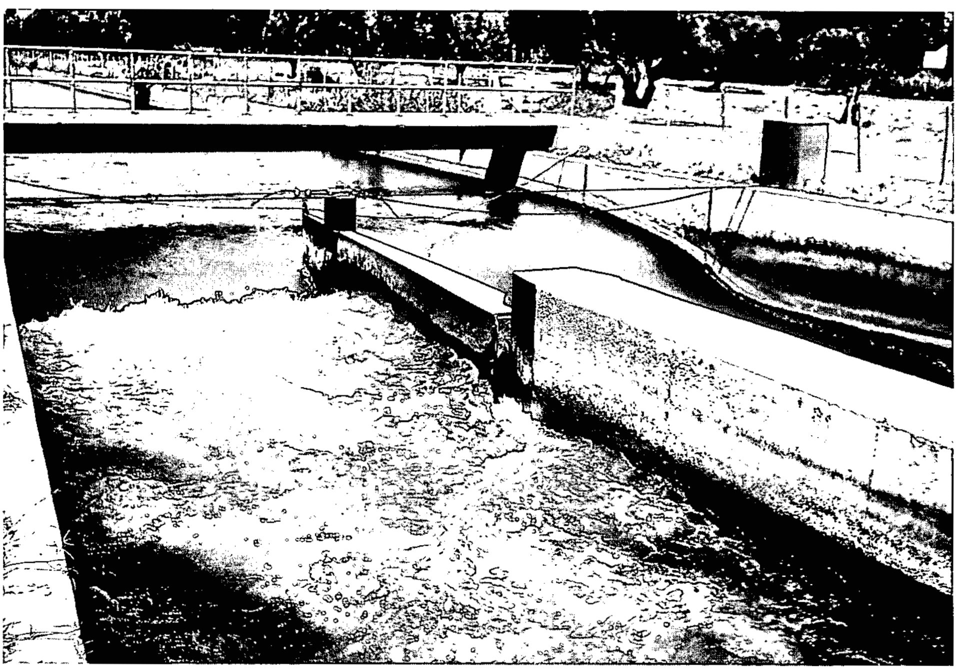


Figure 3.78 Flow divisor for 65 m^3/s (same structure as Figure 3.77) (Argentina).

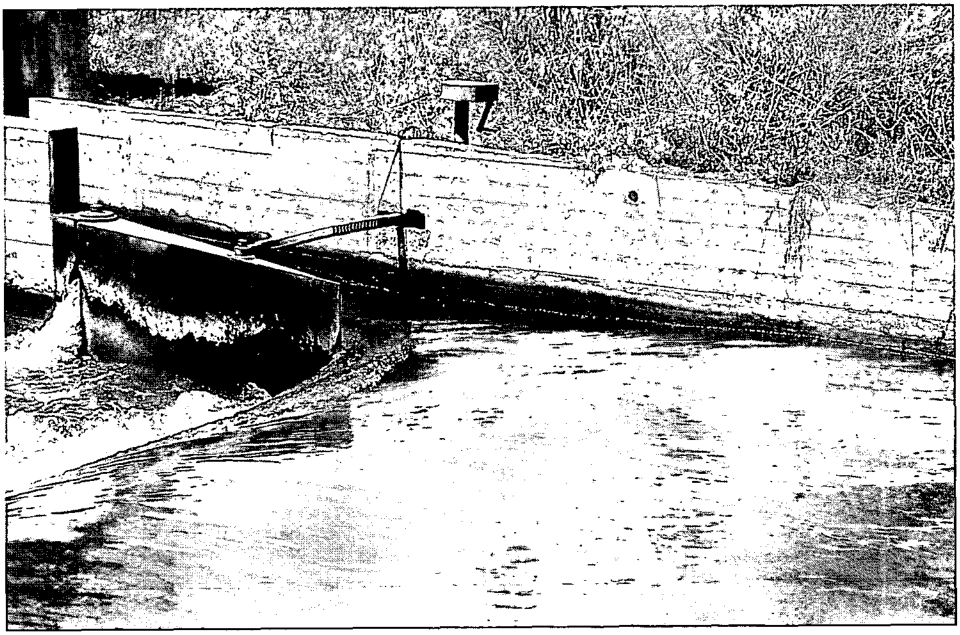


Figure 3.79 Movable flow divisor door can be adjusted with a hand-crank that moves an adjusting rod. (Argentina).

flow through the spacing is negligible. For smaller structures, an adjusting rod and hand crank can replace the cable and pulley system (Figure 3.79). The length of the board depends on the required travel distance of the board. The angle of the board with the centerline of the weir should always be less than 1:6.

The flow divisor cannot be used to cut off flow into the lateral canal. For emergency canal closures a gate should be placed downstream from the divisor. Under normal operational conditions the gate must be lifted entirely out of the water. The gate should not be used for flow control because this would take flow control away from the weir, submerge a portion of the crest, and invalidate the flow measurement.

If the divisor board is parallel to the direction of flow over the broad-crested weir and the water level in both downstream canals is equal, the only force to overcome to move the board is the friction on the hinges. Maximum force is needed to move the board away from its maximum deflection (being $^1/_6\, L_{board}$). This maximum force is approximately:

$$F = {}^1/_6\, L_{board}\, \Delta H\, H_c\, \rho g \qquad 3.5$$

This force is considerably less than the force needed for vertical weir movement.

3.7 Drain Pipe through Weir

Since weirs contract the channel from the bottom, a pool will be maintained upstream, even with no flow. A drain pipe should be installed through the base of the weir for convenient winter drainage and control of mosquitoes in summer, particularly if the canal is for intermittent use, as is usually the case for canals in a tertiary unit or a large farm system. As a rule of thumb, a pipe diameter of approximately $D_p = L_p/50$ is recommended, where L_p is the length of the drain pipe. This yields pipe diameters of 25 to 75 mm (1 to 3 in.) for typical small to medium size weirs. The discharge through the drain pipe for a head loss across the weir of Δh is (Bos 1989)

$$Q = \frac{\pi}{4} D_p^2 \sqrt{\frac{2g\Delta h}{\xi}} \qquad 3.6$$

For such "small diameter pipes" friction losses in the pipe are more significant than the entrance and exit losses, and the coefficient ξ is approximately

$$\xi = 1.9 + f\,{}^{L_p}\!/_{D_p} \qquad 3.7$$

where f is the Darcy-Weisbach friction coefficient. For smooth, small diameter pipes a value of $f = 0.020$ is appropriate (Bos 1978). Hence, for the ratio $L_p/D_p = 50$ the coefficient $\xi = 2.9$. Equation 3.6 yields Figure 3.80 from which the pipe discharge can be read as a function of D_p and Δh.

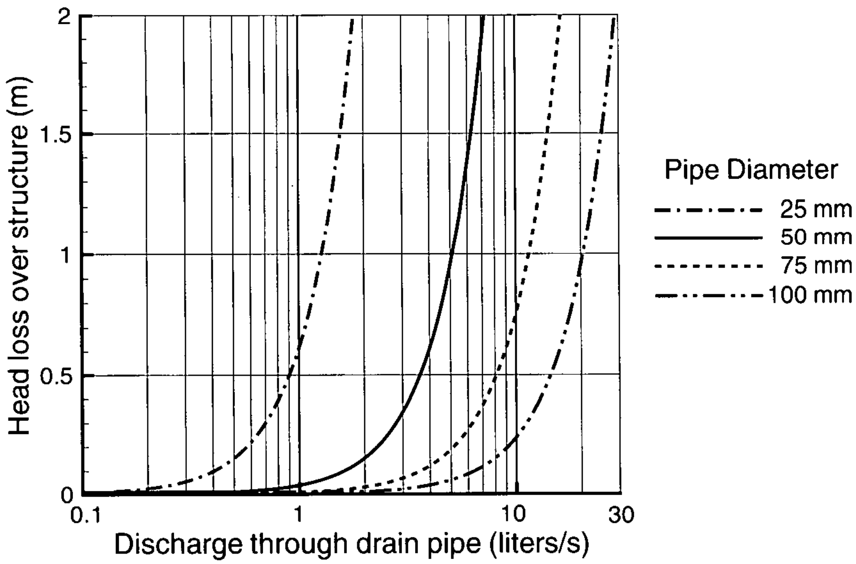


Figure 3.80 Discharge through drain pipe with $L_p/D_p = 50$, as a function of diameter and head loss over the structure.

For example, Figure 3.80 shows that flow through a 25 mm (1 in.) drain pipe (appropriate for typical small irrigation weirs) with Δh = 0.6 m (2 ft) is 1.0 liters/s (0.035 ft^3/s). Such a flow is negligible with respect to Q_{max} of the weirs for which such a drain would be appropriate. If the drain diameter is larger than $L_p/50$, or if the weir operates near its minimum capacity during long periods, the drain pipe should be at least partially blocked. This can be done with any available materials such as valves, brick covers, styrofoam cups, or rags. The latter can be quickly pushed out of the tube with a stick or rod when drain-down of the canal is desired. Complete blockage is not required. Sediments, which tend to accumulate at the base of the ramp, may plug the drain pipe and should be cleaned out periodically.

4. Measurement of Head

4.1 Introduction

Factors influencing the accuracy of a single flow-rate measurement were considered in Section 2.8, and the importance of accurate measurement of upstream sill-referenced head was discussed. In fact, the measurement of head is so important that the success or failure of the measuring structure often depends almost entirely upon the effectiveness of the gage, sensor, or recorder used.

When we use the term *sill-referenced head*, we mean that the head is measured with respect to the invert of the control section of the structure—i.e., the section at which the flow passes through critical depth. This control section is located in the flume throat at a distance of about $L/3$ from the downstream edge of the sill (Figure 4.1). In the direction of flow, the top of the sill (weir crest or invert of flume throat) must be truly level. If minor undulations in the elevation of the sill occur along its length, we recommend that the level at the control section be used as the sill-reference level rather than taking the average along the length of the sill. If the sill is intended to be horizontal in the direction perpendicular to the flow, then the average level across the width of the sill at the control section should be used as the sill-reference level.

The gaging or head-measurement station should be located sufficiently far upstream from the structure to avoid the area of water surface drawdown, yet it should be close enough for the energy loss between the gaging station and the structure to be

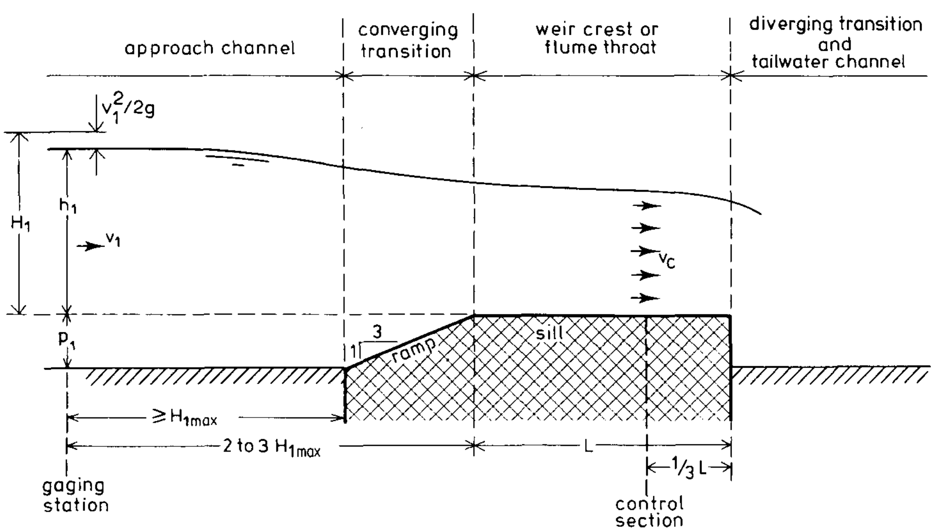


Figure 4.1 Schematic diagram showing general terminology and location of gaging station and control.

negligible. This means it will be located at a distance between two and three times H_{1max} from the leading edge of the sill or at H_{1max} from the beginning of the converging transition, whichever is greater (Figure 4.1)

If only occasional flow measurements are required, the water level at the gaging station can be measured by a vertical or an inclined gage installed in the approach channel. If continuous flow records are needed, or if the flow rate is to be transmitted electronically to a distant location, a water level transducer and/or an automatic recorder will be needed. Regardless of the type of head-measurement device used, it should be located to one side of the approach channel to minimize its interference with the flow approaching the structure.

4.2 Selection of Head-Measurement Device

The selection of a suitable head-measurement device contributes greatly to the success or failure of the structure and to the value of the collected data. The three most important factors that influence the choice of a device are

- Frequency of discharge measurements,
- Type of structure over which the head must be measured, and
- Allowable error in head detection.

With respect to the first two items, detailed information is given in the remainder of this chapter. For the third item, we rewrite Equation 2.14 to determine the allowable relative random error (uncertainty) in the h_1 reading

$$X_{h1} = \sqrt{\frac{X_Q^2 - X_C^2}{u^2}} \qquad 4.1$$

Within the range $0.07 < H_1/L < 0.7$, $X_C = 1.9\%$ if the structure is calibrated with the mathematical model of Section 6.5 which is used in the software described in Chapter 8. The exponent u is determined from the shape of the control section (see Table 2.2 for u-values). The allowable error, X_Q in one single flow measurement is a user-specified value for both Q_{min} and Q_{max}. The calculated X_{h1} value is used to determine the allowable head-measurement error δ_{h1}

$$\delta_{h1} = \frac{h_1 \times X_{h1}}{100\%} \qquad 4.2$$

Common values of errors, δ_{h1} , in the measurement of sill-referenced head with various devices are listed in Table 4.1. Because of the small value of h_{1min} at the minimum flow to be measured, the user-specified allowable error, X_{Qmin}, may be too small to yield a sufficiently accurate head-measuring device from Table 4.1. In this case, the designer has two choices:

Table 4.1 Common errors in reading sill-referenced head.

Device	Reading error δ_{h1} if head detection device is in: Open channel	Stilling well	Remarks
Point gage	Not applicable	0.1 mm	Commonly used for research
Dipstick	Not applicable	1 mm	Good for research and field use
Staff gage	4 mm	4 mm	$Fr_1 < 0.1$
	7 mm	5 mm	$Fr_1 = 0.2$
	> 15 mm	7 mm	$Fr_1 = 0.5$
Pressure bulb and recorder	Up to 20 mm	Not required	Very suitable for temporary installations (Error is 2% of h_{1max})
Bubble gage and recorder	10 mm	Not required	Stilling well not required, but can be used
Float-operated recorder	Not applicable	5 mm	Stilling well is required
Flow totalizer used with any of above devices	-	-	Some additional random and systematic errors are possible

1. Allow a greater error in the measured discharge at minimum flow, or
2. Redesign the structure with a narrower bottom width to produce a larger value of h_{1min}.

4.3 Gages

When continuous measurements of flow rate are not needed, or in channels where the fluctuation of flow is gradual, periodic readings on a calibrated physical gage may be satisfactory. Depending on the type of flume and the required accuracy of the head reading (see previous section), a point gage, dipstick, or staff gage may be used.

4.3.1 Point gage

A point gage is the most accurate head-measurement instrument (error of ±0.1 mm). Its use is normally restricted to research facilities. The point gage is always used in combination with a stilling well. The point gage consists of a pointed, graduated rod suspended above the water surface and raised or lowered in relation to a fixed measurement scale, often including a vernier scale to increase measurement accuracy. The rod is lowered until the point just touches the water surface, and the vertical position of the point is then read from the vernier scale.

4.3.2 Dipstick

A dipstick is inserted into the water in the stilling well until the end of the stick rests on a base corresponding to the exact sill-reference level of the structure. The stilling well used in combination with a dipstick should have a sufficiently large diameter so that the stick does not raise the water level upon insertion. Even then, the stick should be inserted slowly until it rests on its reference point. A dipstick can supply very accurate information on head (error of ±0.001 m). Most portable RBC flumes use a hardwood dipstick that is directly marked in flow rate units (see Figure 3.43).

4.3.3 Staff gage

A staff gage should be placed in such a manner that the water level can be read from the canal bank and so that its surface can be cleaned by the observer. For earthen channels, the gage can be mounted vertically on a support structure placed in the flowing stream. The support structure should not interfere with the flow of water through the flume throat or over the weir crest, and it should not catch floating debris.

For concrete-lined canals, the gage can be mounted directly on the canal wall. For sloping canal walls, the length indicated on the gage will be greater than the corresponding vertical water depth. The relative slope lengths versus vertical lengths for the most commonly used side slopes are shown in Figure 4.2 (see also Section 8.9.7).

Within an irrigation project, it is convenient to mark the gages of structures in liters/s, m^3/s, ft^3/s, or other units of discharge rather than in head units. Once the gage has been mounted and checked, this eliminates the possibility of using the wrong rating tables. Direct read-out gages can also be used on movable weirs (see Section 4.8).

With the software of Chapter 8, one can calculate a basic rating table showing discharge versus head (one discharge value in each line of the table). The software will also provide the vertical gage marking distances for a direct reading gage. In contrast, the ditchrider's rating table can be printed with either the vertical gage marking distances or the distances along a sloped canal bank, allowing the use of a standard linear gage installed on a slope. The wall gage module of the software also can provide the gage dimensions and produce wall gages for mounting directly against the slope of the approach channel. An example of an inclined direct-reading gage is shown in Figure 4.3. For this gage, the marks need not be more than about 3 or 4 cm apart, since interpolation between marks will give reasonable accuracy. For example, on the gage shown, there is a 2.5 cm difference in elevation (4.5 cm along the sloped wall gage) between 2.20 and 2.40 m^3/s. Interpolation between these marks by eye is relatively easy. With experience, an observer can easily read the discharge to within ±4% of the true discharge.

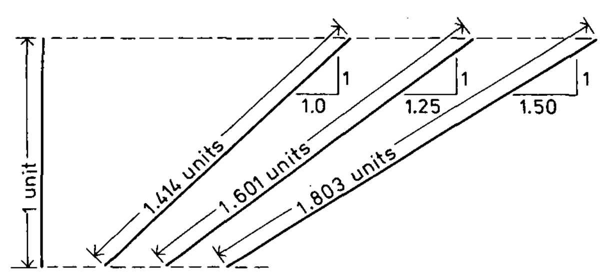


Figure 4.2 Multipliers for layout of an inclined gage.

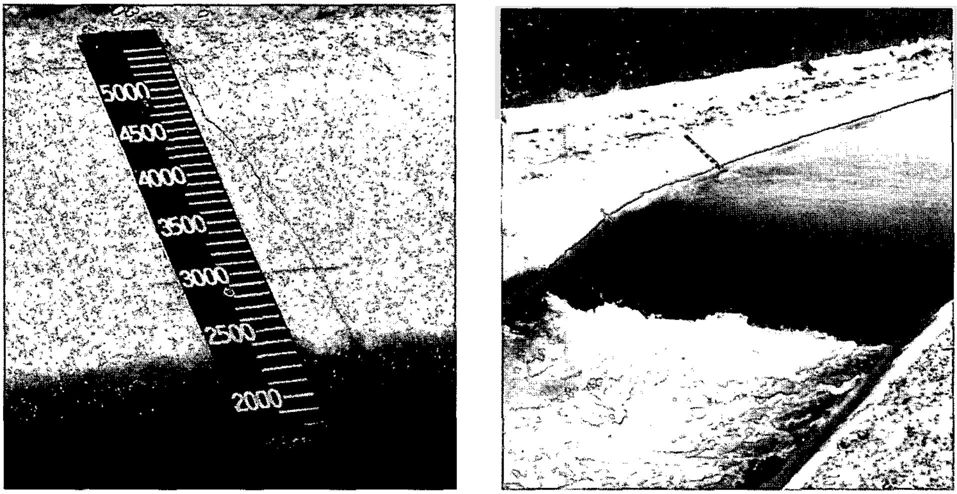


Figure 4.3 The inclined gage is mounted against the right canal bank. The gage is labeled with discharge units.

Most permanent gages are constructed from plates of enameled steel, cast aluminum, or polyester. Baked enamel steel gages with linear scales are available from commercial sources. These gages will last for a very long time. Gages marked in discharge units can be custom-ordered in large quantities, but are considerably more expensive. Spray enamel paints with UV protection can also be used to make gages on steel. These are not as durable as the baked enamel, but are considerably less expensive. Gages in discharge units can also be made by stamping aluminum bar stock with a hammer, chisel, or metal-stamping dies. These gages require periodic cleaning, so they must be accessible.

4.4 Automatic Recorders and Water Level Sensors

Automatic water-level recorders create a permanent record of the variation of water surface elevations as a function of time. A sensor converts the water level into physical motion and/or an electrical signal that can be recorded on paper, magnetic tape, or other electronic form by the recorder (data logger). Automatic recorder systems have several advantages over ordinary gages:

- In channels with daily fluctuations of flow, continuous records provide the most accurate means of determining the daily average and total flow.
- The entire hydrograph is recorded with the maximum and minimum water levels as a function of time. This provides data on the reaction time of the channel system to upstream changes in flow.
- Observations can be made at remote places where observers are not available or in locations that are not accessible under all weather conditions.

A number of meteorological instrument manufacturers produce a variety of commercially available sensors and recorders. In some cases, the sensor and recorder systems are integrated together into a single system, while in other cases the sensor and recorder are separate devices. The type of sensor chosen for the site can have important ramifications for the design of the structure and appurtenances. Some of the most common types of sensors and their important characteristics are described below.

4.4.1 Submerged pressure transducers

Pressure transducers convert the hydrostatic pressure of water at a given depth into an electrical signal that can be recorded. Transducers are available in many different configurations that exploit a variety of properties of different materials or devices to accomplish this conversion. Submerged pressure transducers can be suspended in a stilling well or installed in a protective pipe which is perforated to admit water. The transducer is fastened in place, submerged below the minimum expected water level. To produce an output of gage pressure (i.e., referenced to ambient atmospheric pressure), the transducer is vented to the atmosphere via a vent tube integrated into the cable carrying the electronic output signal of the transducer. The free end of the vent tube should terminate in the instrument enclosure, and a desiccant should be used to prevent water vapor entry into the vent tube, as this can lead to corrosion of the transducer and errors in its output. A flexible bladder can be used as a desiccant replacement, provided that expansion and contraction of the bladder does not change the pressure (i.e., the pressure inside and outside the bladder must be the same).

Advantages of pressure transducers are the relative simplicity of installation, since a stilling well is not required, and their accuracy, which can range from ±1.0 to ±0.1 percent of the maximum range that can be measured by the transducer. Accuracy and cost are generally proportional. Disadvantages include the need to maintain the desiccant pack associated with the vent tube and the requirement to protect the transducer from freezing or remove it from service during the winter. The most significant calibration issue for pressure transducers is avoiding drift of their output at zero pressure, since this can lead to relatively large percentage errors in flow rate at minimum discharge conditions. Fouling of the opening to the transducer can also be a problem.

4.4.2 Pressure bulb

This instrument consists of a flexible bulb that is placed in a perforated metal container for protection and connected by an air tube to a mechanical pressure gage and recorder or to a pressure transducer with an electronic output. The container and flexible bulb are fixed in place below the minimum water level to be recorded. Any change in water level changes the pressure inside the system and thus is recorded. Advantages of this recorder are that the container and bulb do not require a stilling well and the distance between the bulb and the recorder may be up to 50 m (175 ft). Hence, the installation of the system is simple and relatively cheap while the recorder can be placed at a suitable location.

The major disadvantage of the pressure bulb is that the error in the recorded water level is generally ±2% of the maximum range that can be measured by the recorder. If this range, for example, is 1.0 m, the error in recorded head is ±0.02 m for all heads. As a result, at the minimum flow condition the measured flow rate can be rather inaccurate. Also, system leaks can cause operational failure.

Despite these disadvantages, the pressure bulb is very suitable for relatively temporary installations and sites at which the greatest accuracy is not necessary. A regular calibration between the staff-gage reading and the recorded water level is required for this type of instrument to maintain sufficient accuracy.

4.4.3 Bubblers

This instrument consists of a tube that is usually fastened with its open end at least 0.05 m below the lowest water level to be recorded. The tube is connected to a supply of air from a cylinder of compressed air or a small compressor and to a pressure gage or a pressure transducer plus a recorder. Air flows very slowly from the open end of the tube, and the pressure required to overcome the head of water above the end of the pipe is measured and recorded. The method by which the pressure is measured and recorded may be similar to that of the pressure bulb or may involve recent electronic devices. The advantages and disadvantages are somewhat similar to the pressure transducer system already described, except that the transducer is not submerged, so it need not be removed in freezing weather and there is less scaling and fouling of the transducer. Not submerging the transducer has proven to dramatically improve the reliability of bubbler systems compared to submerged pressure sensors.

Relatively long transmitting distances can be achieved with the bubbler system. Installations of 300 m have been used. On these long lines, it is best to use two small 3-mm inside-diameter tubes for economy and accuracy. One tube carries the bubble air supply from the source to the bubble outlet at 3 to 5 bubbles per second. The second tube is attached as a branch line as near as practical to the bubble outlet, preferably within 2 to 5 m. This second tube then senses the bubble pressure at the desired distance. Because there is essentially no flow in the sensing line after stabilization, there are no appreciable friction losses. On the source line, even 3 to 5 bubbles per second cause a significant pressure drop in several hundred meters, and thus the source line cannot also serve as the sensing line at long distances. Thermal gradients and pressure change in the sensing line may become significant only if large vertical distances are encountered. Figure 4.4 illustrates the schematic arrangement of a remote bubble gage.

Transmission distance is limited primarily by the allowable response time needed for a change in flow to be detected. The gage becomes more sluggish with increasing sensing line length because larger volumes of air must be moved to achieve a new stable pressure reading. For 300 m of 3-mm line (inside diameter), stability is usually reached in several seconds, depending on the volume sensing requirements

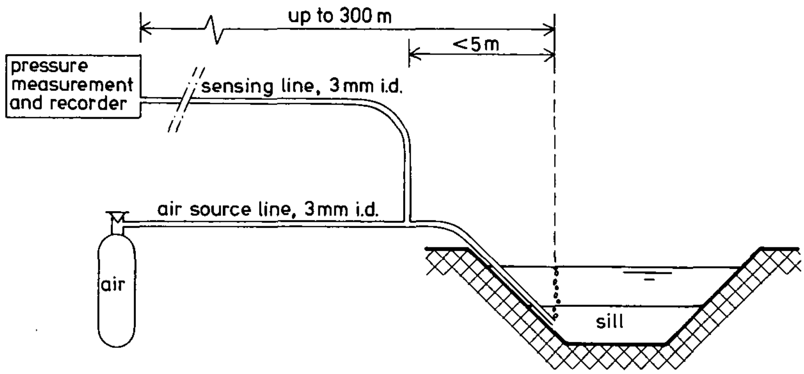


Figure 4.4 Schematic for a remote recording bubbler system that is not sensitive to transmission distances up to 300 m.

of the pressure sensing gage. For example, a large-bore manometer requires more volume shift than a small pressure gage, but the manometer may be more sensitive.

Self-contained bubbler systems have been developed in recent years that integrate a small pressure compressor and optional pressure tank, the transducer, and associated electronics into a low-power unit that can easily function continuously on solar power (Figure 4.5). Another variation on the bubbler concept is the double-bubbler, in which bubbles are delivered alternately through two different tubes that terminate a fixed vertical distance apart in the water column. If the same transducer is used to sense the atmospheric pressure and the pressure in each tube, one can compensate for changes in the transducer calibration, producing a more accurate measurement. Accuracy of commercially available bubbler systems has improved in recent years and can now be on the order of ±0.003 m (±0.001 ft).

4.4.4 Ultrasonic level sensors

Ultrasonic level sensors are mounted above the water surface and determine the position of the free surface by measuring the transit time of an acoustic pulse that travels from the sensor down to the water surface and is reflected back up to the sensor. To achieve useful accuracy, the sensor must be temperature compensated, since the speed of sound in air varies with temperature. Ultrasonic level sensors can be installed with or without a stilling well; a stilling well is preferred because it reduces waves on the water surface that can reduce the measurement accuracy. Details of the particular sensor should be considered when designing the stilling well, as the acoustic signal transmitted by the sensor radiates out in a cone pattern. The signal thus may be reflected back up to the sensor off the walls of the stilling well (especially if the walls are rough), causing the sensor to measure this distance rather than the distance to the water surface. Conversely, installing the sensor directly above a relatively small-diameter smooth-walled pipe that extends down into the water works well with some sensors. This is because the acoustic signal is not reflected back up the pipe due to the flat angle of incidence of the acoustic signal with the pipe wall.

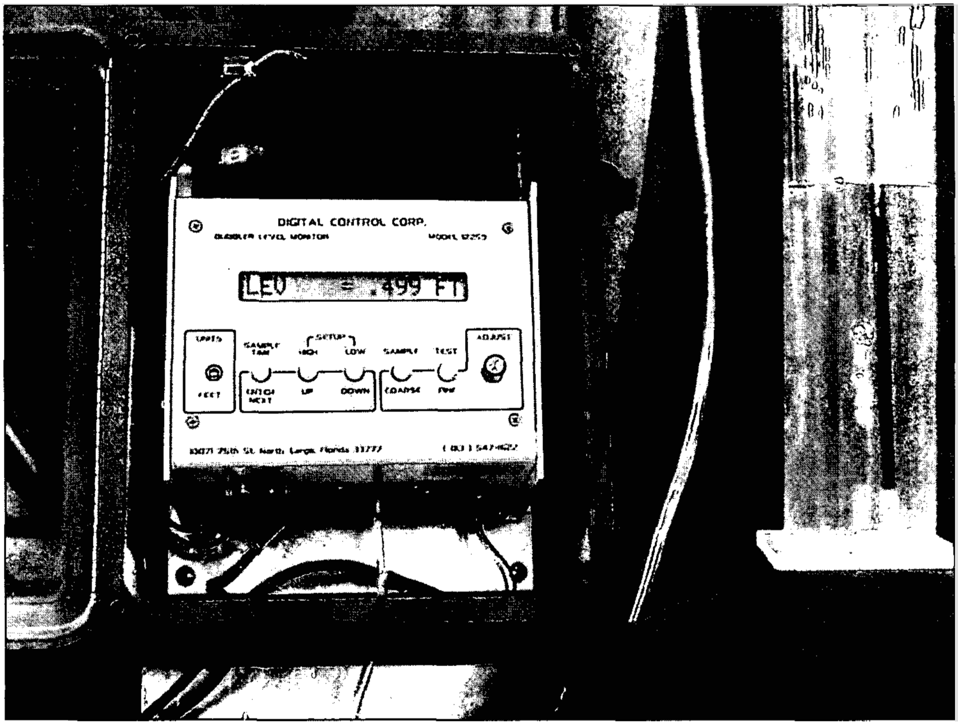


Figure 4.5 A self-contained bubbler water level sensor. (Courtesy Digital Control Corporation, Largo, Florida, USA).

Advantages of ultrasonic level sensors are the relative ease of installation and the fact that they do not physically contact the water surface, making them a good choice for sites with polluted or corrosive waters. Disadvantages are the fact that they have only moderate accuracy, and, even when temperature-compensated, are affected by temperature gradients that may exist in the air space between the sensor and the water surface. Temperature gradients can be extreme in many irrigation applications, especially in stilling wells located in the daytime sun where the top of the stilling well or instrument enclosure can reach temperatures of 60°C or higher. Ultrasonic level sensors also require periodic maintenance to ensure a clear path between the sensor and the water surface; spider webs beneath the sensor have been known to cause erroneous measurements.

4.4.5 Float-operated recorder

Float-operated recorders have been one of the most commonly used instruments for measuring water level because of their relatively low cost, good accuracy ($\delta h_1 = \pm 0.005$ m), and wide availability. The instrument consists of a float of sufficiently large diameter, which is attached to a tape or cable that passes around the float wheel of a recorder and then to a counterweight. The float rises and falls with the water level, and its movement rotates the float wheel and thus is recorded. To function properly, the float must be located in standing water. Thus, a stilling well is required on all field installations (see Section 4.6).

Care should be taken to ensure that when the float is rising its counterweight does not lodge on top of the float but keeps well above it or passes the float. If a high degree of accuracy is required, the counterweight should not be permitted to become submerged over part of the operating range since this will change the submergence of the float and thus affect the recorded water level. This systematic error may be prevented by

- Locating the counterweight inside a separate watertight and water-free pipe.
- Mounting two different-sized drums on the axle of the recorder. The larger diameter drum serves to coil up the float wire and the small diameter drum coils up the counterweight wire, yielding reduced movement of the counterweight relative to the float. The drums require a spiral groove for coiling up several turns of wire, otherwise there is an error due to coiling of cable on top of itself. Tapes cannot be used with this method.
- Extending the stilling-well pipe to such a height that the counterweight does not touch the float wheel at low stage nor the water surface at the maximum expected stage.

Most of the earlier recorders relied on the friction drive of a cable on the float wheel of the recorder. To improve the accuracy of the head measurements, we recommend that a recorder be equipped with a calibrated float tape that passes over the float wheel. The float and counterweight should be attached to the ends of the tape by ring connectors. If the recorder is not equipped with a tape index pointer, one should be attached either to the shelter-house floor or to the instrument case. The purpose of the calibrated tape and the index pointer is to enable the observer to easily check the registered water level against the actual water level in the float well and also against the water level shown on the independently placed staff gage. As such, the tape and index pointer provide an immediate check on whether the recorder mechanism, the float system, and the inlet pipe or slots are functioning properly.

Sensors for detecting float-pulley rotation

Two principal types of sensors are available to measure the rotation of the float-pulley. Precision potentiometers provide an analog output proportional to the rotation of the pulley and movement of the water surface. These potentiometers are typically limited to ten rotations. Incremental shaft encoders provide a pulse-type output each time the pulley rotates through a specific interval. The size of the pulley and the rotation increment between pulses determine the precision of the measurement. The random error in water level measurement will be ± one-half this amount. A disadvantage of incremental shaft encoders is that their output describes only the relative movement of the water surface. If the data logger loses power, the motion of the pulley will not be recorded while the power is off, and the exact position of the water level when power is restored will not be known. A recent

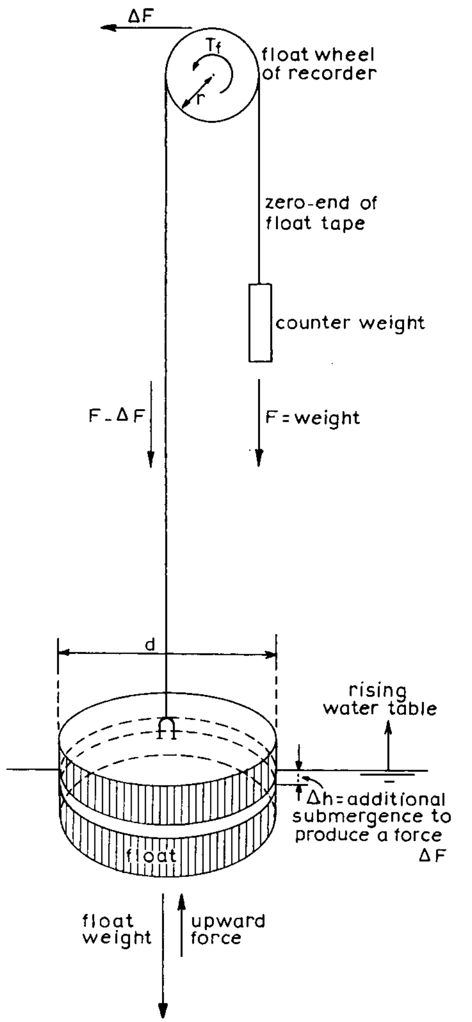


Figure 4.6 Forces acting on a float tape (Kraijenhoff van de Leur, 1972).

improvement is the absolute-position shaft encoder, which provides a digital output indicating the absolute position of the shaft, relative to the mechanical limits of rotation of the encoder.

Sizing the float

If the water level in a stilling well does not move, the float-wheel of a recorder does not move either, and the tensile force F in the tape between the float and the wheel is equal to that in the tape between the wheel and counterweight (see Figure 4.6). A changing water level can only be recorded if the float-wheel is being turned. This wheel, however, turns only if a certain initial resistance is overcome. This resistance, which is due to friction in the recorder and on the axle, can be expressed as a resisting torque, T_f, on the shaft of the float wheel. Since the counterweight exerts a constant tensile force, F, on the float tape, the resisting torque can be overcome only if the tensile force in the tape between the float and the wheel changes by a small amount ΔF in such a way that

$$\Delta F r > T_f \qquad 4.3$$

where ΔF is the change in tensile force in the float tape between float and wheel, r is the radius of the float wheel, and T_f is the resisting torque due to friction on the float-wheel axle. When we have, for example, a continuously rising water level in the well, a decrease in the tensile force ΔF is required, which is possible only if the upward force acting on the submerged part of the float increases. Consequently, the float lags behind the rising water level by a distance δh_1, so that the volume of the submerged float section will increase by

$$\Delta V = \frac{\pi}{4} d^2 \delta h_1 \qquad 4.4$$

where d is the diameter of the float. According to Archimedes' law, the upward force will increase an amount equal to the weight of the additional displaced volume of water; hence

$$\Delta F = \frac{\pi}{4} d^2 \delta h_1 \rho g \qquad 4.5$$

Substitution of Equation 4.5 into Equation 4.3 shows that the friction in the recorder and on the axle causes a registration error of the water level

$$\delta h_1 > \frac{4 T_f}{\rho g \pi d^2 r} \qquad 4.6$$

This lagging motion of the float produces a systematic error; a rising water level is always registered too low, and a falling water level is registered too high.

Accepting the internal friction moment T_f as a fixed property of the recorder, this systematic error can only be reduced by enlarging either the float diameter d or the radius of the float wheel, r. If, for example, for an infrequently maintained recorder, $T_f = 0.002$ N·m (or kg·m²/s²), while $\rho = 1000$ kg/m³, $g = 9.81$ m/s², and $r = 0.05$ m, then a float diameter of 0.03 m would cause a float lag of

$$\delta h_1 > \frac{4 \times 0.002}{1000 \times 9.81 \times \pi \times 0.03^2 \times 0.05} = 0.0058 \text{ m (or 5.8 mm)}$$

With a larger float diameter, $d = 0.30$ m, the systematic float lag decreases to $\delta h_1 = 0.06$ mm. This shows clearly that small-diameter floats should not be used with flume and weir recorders; they are meant to measure slowly varying ground-water table elevations in small-diameter observation wells.

The internal friction varies considerably with the type of recorder, its age, state of maintenance, and so on. The minimum float diameter is usually recommended for each recorder type by the manufacturer. It should be taken into account, however, that the relative error due to float lag becomes large for small heads. Float diameters of less than 0.15 m (6 in.) are not recommended for use with flumes and weirs.

Submergence of the counterweight and an increase in the quantity and weight of float tape or cable on one side of the float wheel (and consequently a decreasing weight on the other side) also cause a changing tape force at the float as the water level changes. This change in force ΔF produces a systematic registration error δh_1 that varies with the change in water level, ΔL, from some reference level. If the float tape has a weight of w per unit length and we assume that the counterweight does not become submerged, the registration error due to the changing weight of tape on each side of the pulley is computed by balancing the weight change, $2w\Delta L$, with an offsetting change in buoyancy of the float (Equation 4.5) produced by a change in float submergence, yielding

$$\delta h_1 = \frac{8w\Delta L}{\rho g \pi d^2} \qquad 4.7$$

The equation shows that this systematic error can also be reduced by increasing the float diameter, or by using a lighter float tape.

The reader should note that all of the phenomena just described produce systematic errors that are in addition to the stilling well lag error described in Section 2.8.7.

4.4.6 Calibrating water level sensors

Periodic calibration of water level sensors is one of the tedious tasks necessary for successful flow measurement using flumes and weirs. Sensors can be calibrated in an office or laboratory environment prior to first installation, but once installed, quick field calibration is much preferred over removal and recalibration of the sensor in the laboratory. The following calibration procedure for submerged water level sensors such as pressure transducers and bubblers has proven to be accurate and straightforward, thereby promoting frequent recalibration of sensors in the field. The procedure calibrates the sensor only, and does not ensure that the sensor is properly set with respect to the sill-reference level of the structure. Procedures for zeroing sensors to the sill-reference level are given in Section 4.9.

The procedure for calibrating submerged water level sensors is as follows:

- Install the sensor by suspending from a fixed point, for example by fastening the cable of a submerged pressure transducer to a cap that fits over the top of the transducer's protective pipe.
- Read the output from the sensor when it is located in its permanent measurement position.
- Raise the sensor by a known vertical height by inserting a precision spacer between the top of the sensor pipe and the cap attached to the sensor cable. Read the output from the sensor in this position and use with the previous reading to compute the slope of the water level versus sensor output relation, called sensor or transducer gain (see Figure 4.7).

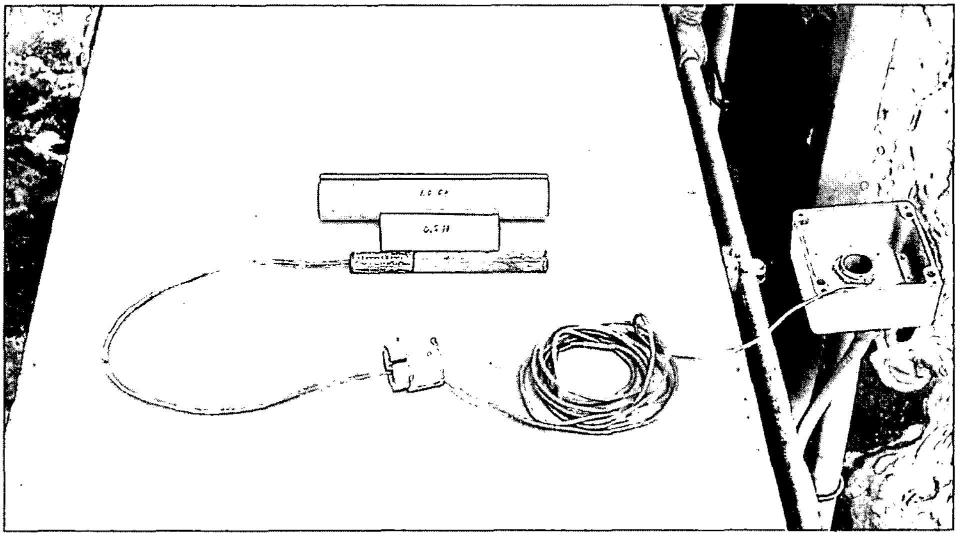


Figure 4.7 A set of precision spacers used to calibrate a submerged pressure transducer.

- Raise the sensor out of the water and read the output to determine the sensor output at zero pressure.

An alternative is to have a rack on which the pressure transducer is mounted. The rack is removable and has two positions that are a fixed, known distance apart. One position is at the operating level and the second position is used to determine transducer gain. To calibrate the sensor, a reading is taken with the rack in each position and with the sensor out of the water.

4.5 Flow Totalizing and Logging

Often one of the objectives of a flow-rate measurement is to obtain information on the volume of water that passes through a channel during a particular period. Calculating this total flow from the recorded hydrograph can be a tedious job, which is often delayed. To simplify this task, commercially available flow totalizers can be used. All modern flow totalizers are electronic. They consist of the following three components: (1) one of the recorders described earlier; (2) a microprocessor that corrects the registered water level to obtain upstream sill-referenced head and calculates the flow rate from the Q versus h_1 equation for the particular structure; and (3) a totalizer that instructs the microprocessor to calculate the discharge at a preset time interval, multiplies the calculated flow rate by the elapsed time since the previous measurement, and adds this incremental volume to the running total.

Storing measured heads with a data logger is an alternative to totalizing. One advantage of storing heads over totalizing is that the entire record can be examined and data errors eliminated. The data stored in the data logger are usually downloaded to a personal computer for analysis. Transmission of data in real time to a central site

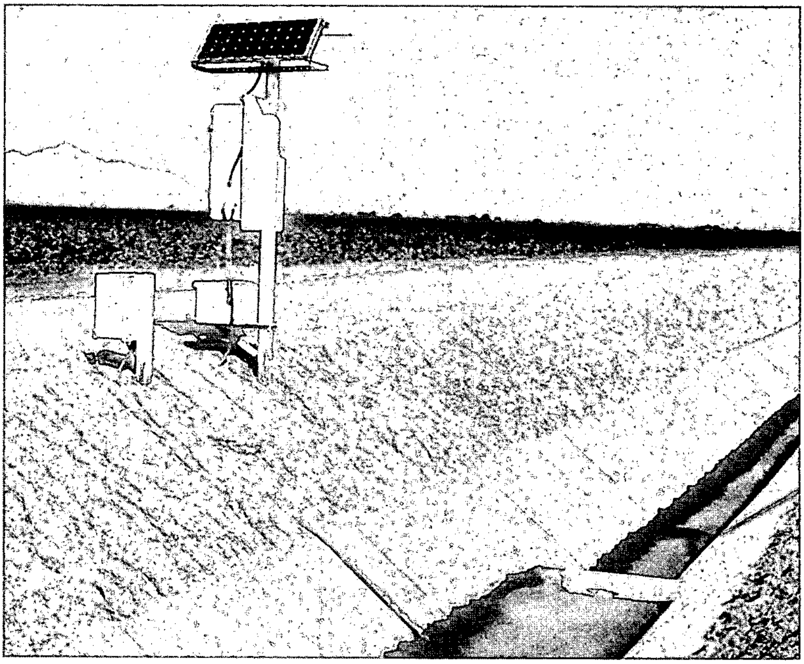


Figure 4.8 Telemetry equipment used to transmit real-time flow measurement data to project headquarters (Arizona).

is gradually taking the place of data logging because it eliminates the need for large amounts of storage within the logger, it does not require the operator to periodically visit the site, and it provides operators with immediate information regarding problems at the site.

The greatest need for information about total flow volumes is related to the accounting and billing aspects of an irrigation project, although such information is also useful to project operators when attempting to deliver a specific volume of water to a portion of a project. Today's computerized telemetry systems can provide project operators with near real-time information on flow rate and accumulated flow volumes (Figure 4.8), while integrated database technology can maintain historical records needed for billing and water accounting purposes.

4.6 Stilling Wells

A stilling well is used for two purposes: (1) to facilitate the accurate registration of a piezometric or water level at a gaging station where the water surface in the channel is disturbed by surges or wave action, and/or (2) to house the float of a float-operated automatic recorder. Even when a stilling well is used, a secondary staff gage should also be installed in the canal, because the staff gage will enable comparison of the outside water level with the head in the stilling well (Figure 4.9). This will help indicate clogging of the pipe(s) connecting the approach channel with the stilling well.

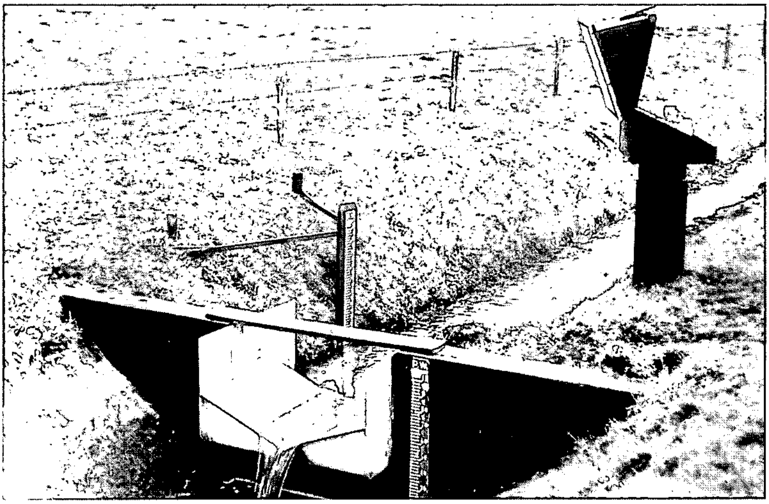


Figure 4.9 Recorder and staff gages for a broad-crested weir. (Courtesy of Agricultural University, Wageningen, The Netherlands.)

The cross-sectional dimensions of the well depend mainly on the method by which the head in the well is measured. We will distinguish between three basic methods of measuring the head: (a) a dipstick, (b) a staff gage, or (c) a float-operated recorder.

Dipstick

If the well is used in combination with a dipstick, a minimum inside diameter of 0.15 m (0.5 ft) is advisable to give access to a hand. A reference point on which the stick will rest and whose elevation coincides with the exact sill-reference location is provided inside the well. A dipstick can supply very accurate information on head (error of ±0.001 m). An example of a well is shown in Figure 4.10. The stick can be a section of plastic or aluminum pipe which slides over the rod that is illustrated. When using a dipstick, be careful not to displace water in the stilling well abruptly, because this will cause a temporary rise in water level and hence an incorrect reading. To avoid such an error, the diameter of the dipstick should be less than one-third of the internal diameter of the stilling well, and the dipstick should be inserted slowly.

Stilling well for staff gage

If a well is used in combination with a staff gage (as in Figure 4.11), the length of the well, as measured from the face of the gage, should not be less than twice the depth to the minimum water level in the well. This provides a satisfactory angle from which the water level can be observed against the gage. The well width should not be less than 0.20 m to allow sufficient room for the gage to be attached by screws to the side of the well.

Stilling well for float

If the well must accommodate the float of an automatic water level recorder, it should be of adequate size and depth to give clearance around the float at all stages. If the

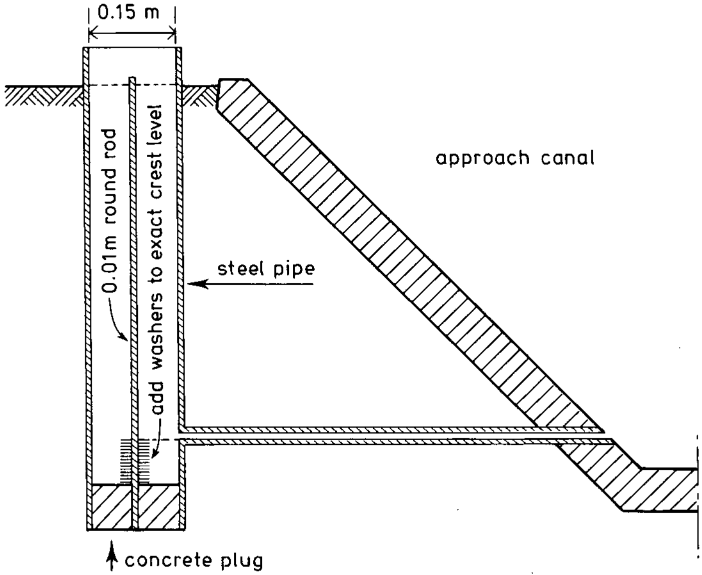


Figure 4.10 Example of a stilling well designed for use with a dipstick. (Dipstick is not shown.)

well is a metal, polyvinyl chloride (PVC), or concrete pipe, its diameter should be at least 0.06 m (0.2 ft) larger than the diameter of the float to avoid capillary effects; if the well is rectangular and constructed of brickwork, concrete, wood, or similar materials, the float should not be nearer than 0.08 m (0.25 ft) to the wall of the well. The bottom of the well should be some distance, say 0.15 m (0.5 ft), below the lowest intake, to avoid the danger of the float touching the bottom or touching any silt that might have accumulated, because the latter may stick to the float and shift the zero reading. Accumulated silt should be removed from the well at regular intervals.

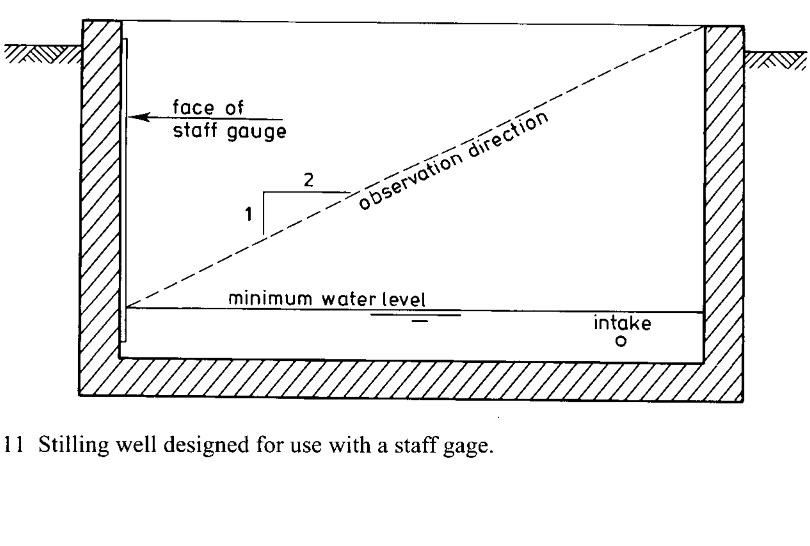


Figure 4.11 Stilling well designed for use with a staff gage.

4.6.1 Construction

In general, an access door should be provided to allow the recorder setting to be checked and to permit the removal of silt without the well having to be entered from the top, which would usually require removing the recorder. If the well is set back into the channel embankment, the access door should be placed just above the embankment; if the well is installed in the channel, the door should be placed just slightly above low water. A second access door will allow the length of the float tape to be adjusted and gears to be changed without requiring removal of the recorder. To avoid corrosion problems, the hinges of the access doors should be of a rust-resistant metal such as stainless steel, brass, or bronze. A simpler solution is to support the

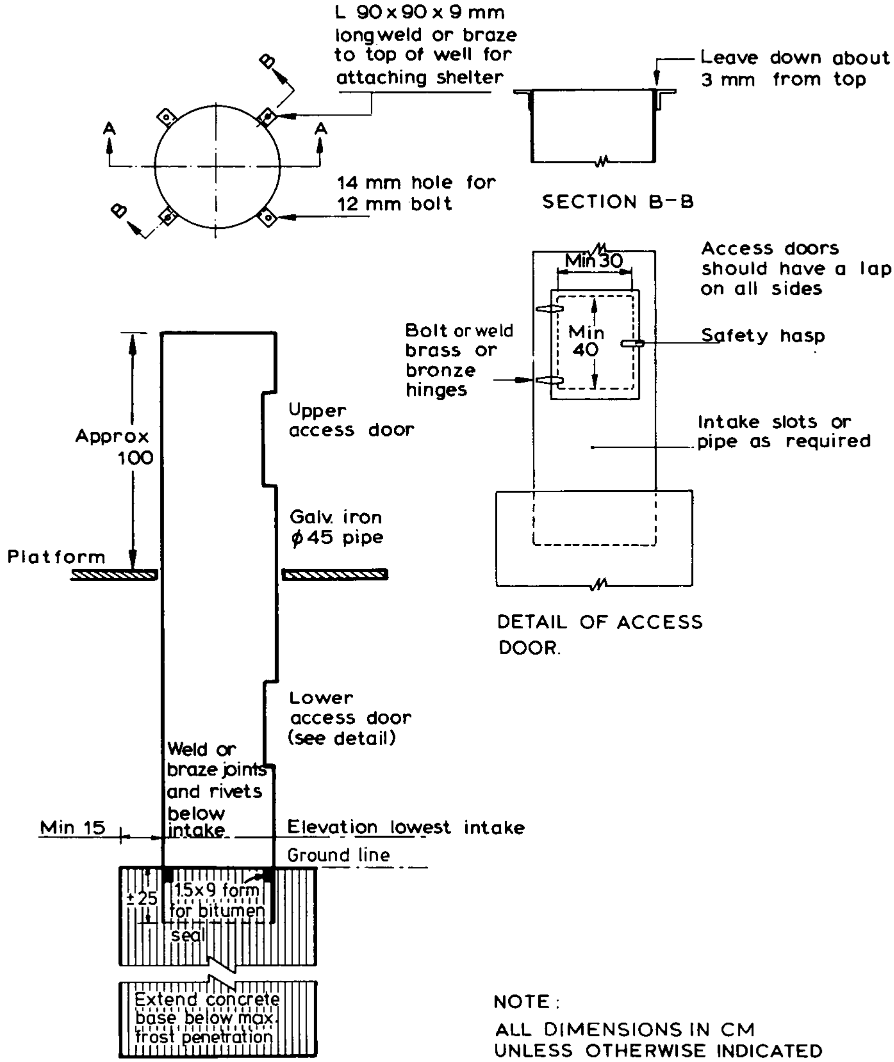


Figure 4.12 Example of a steel stilling well (after Brakensick, Osborn, and Rawls, 1979).

door by wing nuts on short bolts welded to the well. An example of a stilling well with upper and lower access doors is shown in Figure 4.12.

The foundation level of both the structure and the stilling well should be well below the maximum expected frost penetration and sufficiently below the minimum bed level of the canal or stream to provide stability and eliminate undercutting. To prevent the stilling well and intake from functioning as a shortcut for groundwater flow and to facilitate zero-setting of a recorder, the well should be watertight. The inner base of a steel well should be sealed with tar where it meets the concrete foundation. Since the primary purpose of the stilling well is to eliminate or reduce the effects of surging water and wave action in the open channel, the cross-sectional area of the intake should be small. On the other hand, the loss of head in the intake during the estimated maximum rate of change in stage should be limited to 0.005 m (0.2 inches). This head loss causes a systematic error, with a rising water level always recorded too low and a falling water level recorded too high (see also Section 2.8.7). As a general guide to the size and number of intakes, their total cross-sectional area should be approximately 1% of the inside horizontal cross-sectional area of the well.

The intake pipe or slot should have its opening at least 0.05 m (2 inches) below the lowest level to be gaged, and it should terminate flush with and perpendicular to the boundary of the approach channel. If the pipe does not terminate perpendicular to the channel flow direction, a systematic error in the head measurement is possible. The magnitude of this error can approach $v^2/2g$, where v is the flow velocity along the boundary of the channel. The sign of this error (+ or -) is shown in Figure 4.13. The area surrounding the intake pipe or slot should be carefully finished with concrete or equivalent material over a distance of 10 times the diameter of the pipe or width of the slot. Although the minimum requirement is one slot or pipe, on field installations it is advisable to install at least two intake pipes at different levels to avoid the loss of valuable data if one intake should become clogged.

In most stilling wells, the intake pipes will require cleaning periodically, especially those in channels carrying sediments. Permanent installations can be equipped with a flushing tank as shown in Figure 4.14. The tank is filled either by hand pump or with a

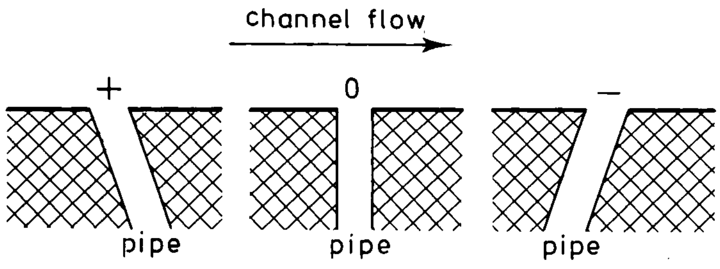


Figure 4.13 Sign of the systematic error in head measurement caused by nonperpendicular pressure tap.

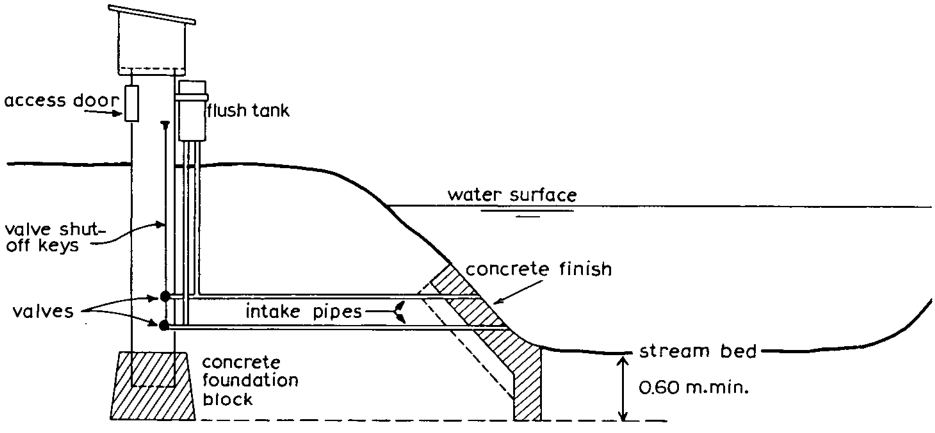


Figure 4.14 Example of a stilling well intake pipe system with flush tank.

bucket, and a sudden release valve will flush water through the intake pipe, thereby removing the sediment. For tightly clogged pipes and on temporary or semi-permanent structures, satisfactory cleaning can usually be provided by a sewer rod or "snake."

Plugging of the intake pipe can be delayed by the construction of a large cavity in the floor of the approach channel at the head-measurement station (Figure 4.15). Its size may be on the order of 0.1 m^3 (3.5 ft^3). The stilling-well pipe enters this cavity and is fitted with a pipe elbow that is turned down so that sediment cannot fall directly into the pipe. The cavity must fill with sediment before the stilling-well pipe can be clogged. The cavity should be covered with a steel plate flush with the bottom of the approach channel. Taking into consideration the probable increased bed-load trapping of transverse slots in this plate and the low quality pressure detection likely with parallel slots, the use of an array of 3-mm holes drilled into the 6-mm grating plate is advisable. Laboratory tests showed no pressure detection anomalies, and field use showed no sedimentation plugging problems, although periodic cleaning of the grating and cavity are required.

4.6.2 Protection against freezing

During winter, it may be necessary to protect the stagnant water in the stilling well against freezing. This can be done by employing one or more of the following methods, depending on location and climate. For a stilling well that is set into the bank, an insulating sub-floor can be placed inside the well just below ground level. Care should be taken, however, to ensure that both the float and counterweight can still move freely over the range of water levels expected during winter. If the well is heated with an electric heater or cluster of lights, or if a lantern or oil heater is suspended just above the water level, the sub-floor will reduce the loss of heat. A reflector to concentrate the light or heat energy onto the water surface will increase the heating efficiency.

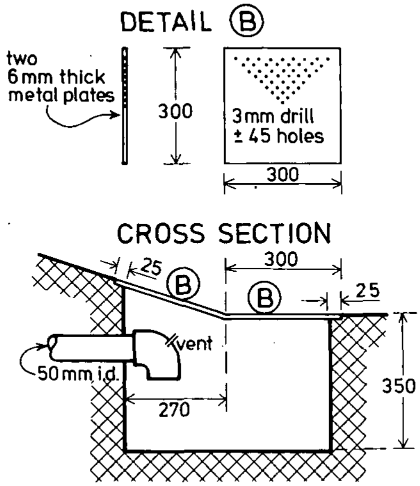


Figure 4.15 Details of pipe cavity (dimensions in millimeters) (after Replogle, Reikerk, and Swindel, 1978).

A layer of low-freezing-point oil, such as fuel oil, around the float can be used as protection. The thickness of the oil layer required equals the greatest thickness of ice expected, plus some allowance for water-stage fluctuations. To prevent leakage of oil and erroneous records, a watertight float well will be necessary. Since the mass density of oil is less than that of water, the oil will stand higher in the float well than the water surface in the open channel. Consequently, the recorder must be adjusted to give the true water stage. If the stilling well is large compared with the float, it is advisable to install the float in an inner pipe and to place the oil in this pipe to avoid the danger of oil being spilled into the open channel. The inner pipe should be open at the bottom so that water may pass freely in and out of it.

4.7 Instrument Shelters

Recorder housings can range from those used for permanent stations on large streams, which are large enough for the observer to enter (See Figure 4.16), to very simple ones, just large enough to cover the recorder and hinged to lift in the same direction as the instrument cover. A major disadvantage of the latter type is that it does not facilitate servicing the recorder during bad weather, and furthermore, the shelter provides insufficient room for the storage of charts and other supplies. For typical flume and weir applications, the instrument shelter should meet the following criteria:

- The shelter door should be hinged at the top so that when it is opened it will provide cover for the observer.
- An iron strip with a small notch near one end should be attached to either side of the door and should run through a staple on each side of the door opening, thus holding the opened door in position, even against gusting winds.
- To prevent vandalism, all hinges and safety hasps should be placed so that they cannot be removed while the door is locked.

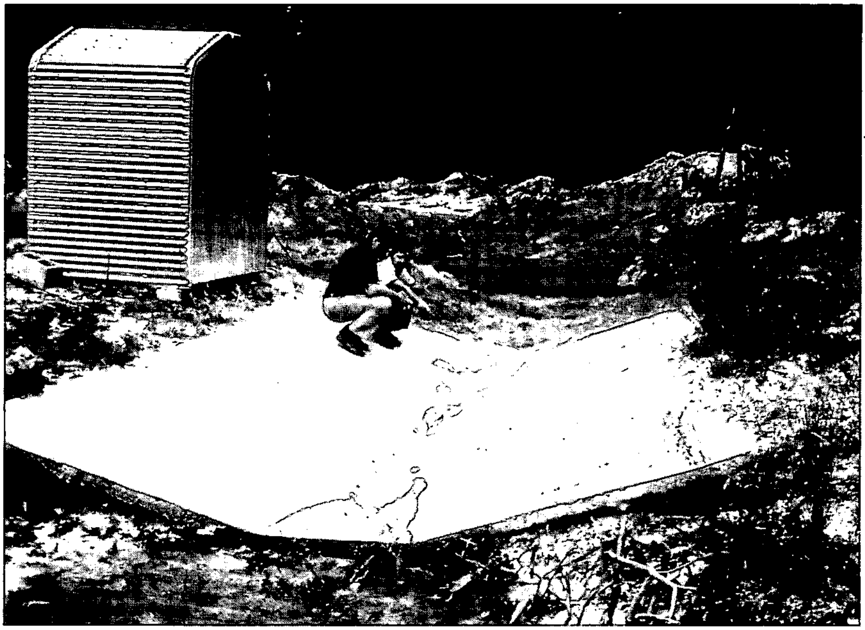


Figure 4.16 This triangular-throated flume is equipped with a large instrument shelter (Florida).

- The flooring should be solid and of a suitable hardwood that will not warp.
- The shelter floor should be anchored to the well, for example, by bolting it at the four corners to small angle irons welded to the top of the float well.

An instrument shelter meeting these criteria is shown in Figure 4.17. An alternate stilling well and instrument shelter design is shown in Figure 4.18.

Condensation can be reduced by gluing or spraying a 3-mm (1/8-inch) layer of cork to the inside of both the metal shelter and the recorder cover and by applying a sealer or vapor barrier to the cork. Silica gel can be used as a desiccant, but the moisture should be removed from the gel at regular intervals by heating it in an oven to about 150°C (300°F).

4.8 Head Measurement Over a Movable Crest

To control flow over a weir, a crest that moves up and down can be used. Hence, a wall-mounted staff gage at the head-measurement station does not provide a value for the upstream sill-referenced head, h_1, unless the weir crest elevation is registered separately so that it can be subtracted from the gaged water level. This subtraction can be accomplished automatically with commercially available differential head recorders. Very often, however, a recorder is not needed at each structure. Direct readings of the sill-referenced head, h_1, or of the flow rate can be made with the following scale arrangements, the use of which is strongly recommended in addition to the differential head recorder. Direct reading gages facilitate weir operation.

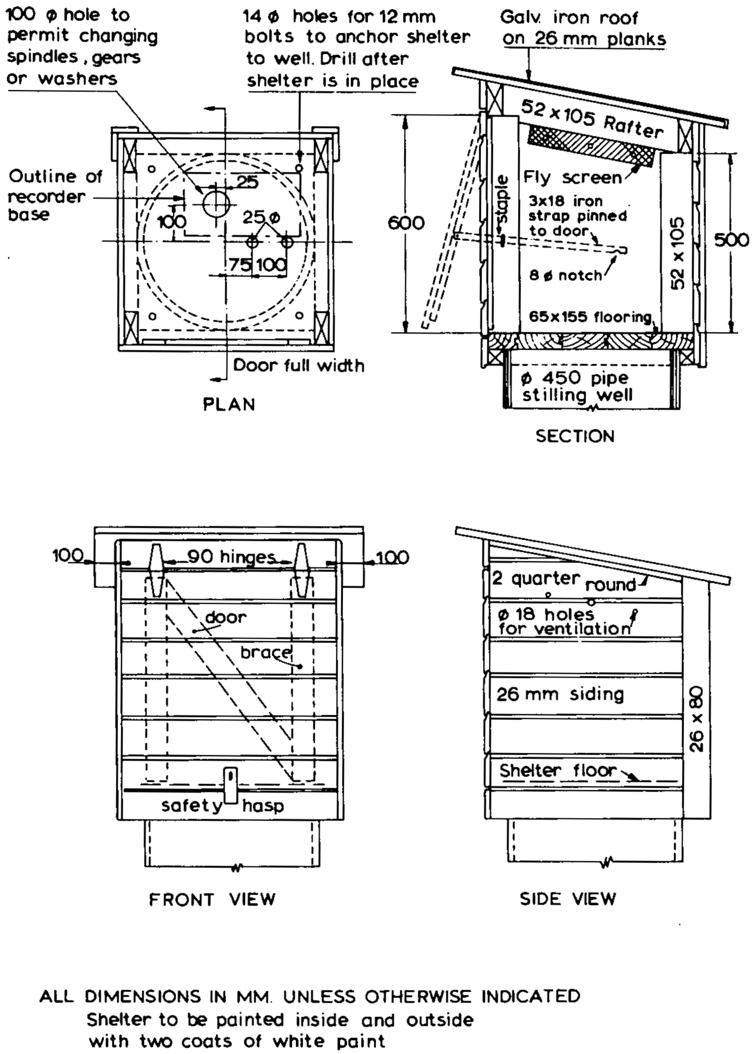


Figure 4.17 Example of an instrument shelter (after Brakensick, Osborn, and Rawls, 1979).

4.8.1 Gage and scales

Figure 4.19 shows a flow-registration system that is very sturdy and is difficult to alter by unauthorized use. The system consists of two scales and a staff gage. The first scale, which is graduated in length units, is mounted to the stationary guide frame of the weir. The second scale, marked in discharge units, is attached by means of a steel support to the hoist strip and lifting beam of the crest. Hence, the first scale has a fixed position while the second scale moves up and down with the weir crest. The numbering on the length scale is the same as that of the staff gage installed in the approach channel to the weir.

Figure 4.18 Automatic recorder mounted in an alternative instrument shelter (Belgium).

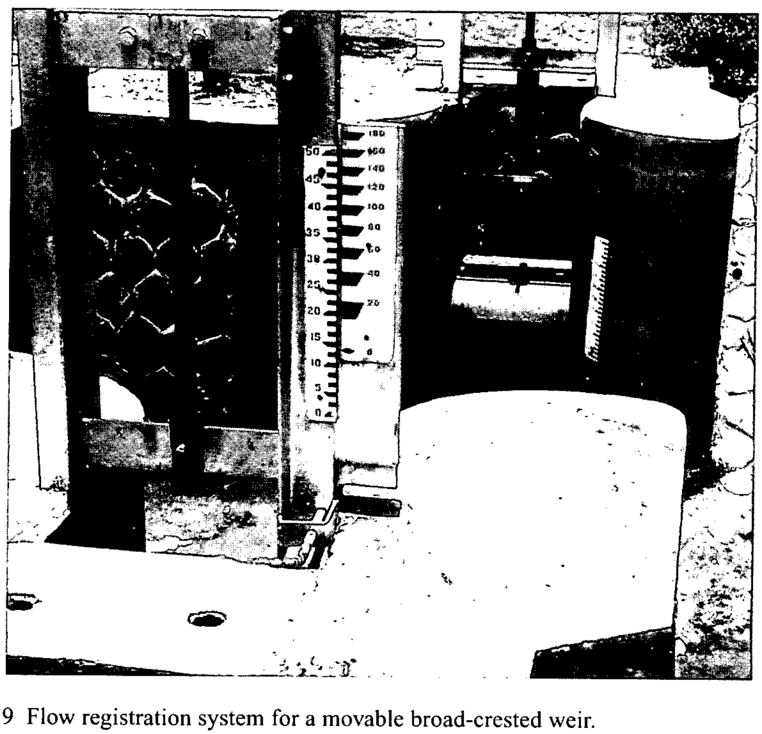


Figure 4.19 Flow registration system for a movable broad-crested weir.

The discharge scale and the length scale are fastened with respect to each other in such a way that if the crest is placed exactly at the approach-channel water level, the reading on the length scale across from the zero on the discharge scale is the same as the reading on the upstream staff gage. If the weir crest is lowered, the weir flow is read across from that point which corresponds with the staff-gage reading. The procedure to determine the weir flow is as follows:

- Read the gage in the approach channel and remember this reading.
- Find the corresponding point on the length scale.
- Read the flow rate on the discharge scale across from this point on the length scale.

4.8.2 Automatic recorder

Figure 4.20 shows a schematic of a differential head meter that can drive an automatic recorder. The system is designed in such a way that with either a lowering of the float or a raising of the extended lifting beam (attached to the weir crest), the free-hanging disc wheel C will be pulled down half this distance. Thus a point on the circumference of disc wheel A will move this same "half distance" against the recorder wheel. Upon adjustment for this "half" movement of the disc wheel (e.g., with a pulley of different diameter), the difference between the stilling-well water level and the weir crest, h_1, can be recorded directly. Zero setting of the recorder can be accomplished by adjusting the swivel or set screw with which the wire is fastened to the extended lifting beam.

4.9 Gage Placement and Zero-Setting

The accurate determination of the sill-referenced head, h_1, is the most important factor in obtaining accurate discharge measurements. This head can be measured by a water level gage or recorder only if the observed water level is known with respect to the weir-sill or flume-crest level at the control section (see Figure 4.1). The best method for accurately referencing the water level sensor to the sill level depends on several factors, including the size of the canal in which the structure is located, the flow rate in the channel during the setting procedure, and the available equipment.

4.9.1 Setting the zero of the recorder

There are several methods for zeroing a water-level recorder, three of which are particularly suitable. The recorder can be set when the canal is dry, when water is ponded over the flume, or when water is flowing through the canal. For all methods, the reference point for determining the upstream depth should be located along the flume centerline at a point roughly one-third of the throat length upstream from the downstream end of the throat (see Figure 4.1). This will help to correct for any errors in leveling the flume crest. If the flume is truly level, any point on the flume crest will work adequately. The "levelness" of the flume crest should be checked during the zero-setting procedure.

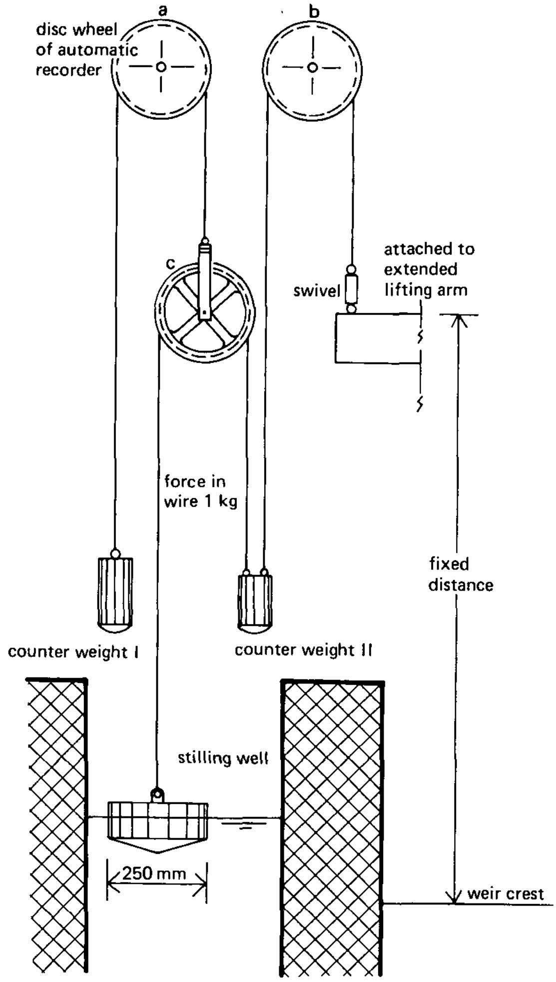


Figure 4.20 Schematic of a differential head meter that can drive an automatic recorder.

The following zero-setting methods assume that the sill-reference elevation can be measured during the procedure. Especially on wide structures, this is not always practical. A stable benchmark (bronze cap poured in concrete) should be added to such structures, and its elevation established with respect to the sill-reference point. The second setting procedure (empty canal) can then be used with the point gage above the benchmark, provided that the stilling-well pipe can be plugged temporarily, as required by the empty canal method.

Setting of recorder using a pond

In small channels that do not discharge water during the setting procedure, a small pond can be used to set the recorder. Instructions for setting the water level recorder in an existing stilling well are as follows:

- Form a temporary earthen dam, or place a watertight check gate immediately upstream from the stilling-well pipe and a second dam or gate downstream from the control section.
- Raise the water level in the thus-formed pond until it is at least 0.05 m above the sill crest, but preferably to the most common (or design) water level in the channel.
- Place the water-level recorder on the floor of the shelter or on the shelf, and install all recorder-related equipment in position to record.
- Observe the record for about 5 minutes to see if the setup is watertight. If the water level drops during this period, find the leak and repair it.
- Place a dipstick or ruler into the pond at the sill-reference location and read the head over the structure crest/sill to ±1 mm (0.003 ft). Repeat this step for a check.
- Adjust the float tape and the tape index pointer in such a way that the above sill-referenced head is read opposite the pointer. (Note: some recorders do not use this setup.)
- Adjust the recorder to show the last reading of the sill-referenced head.
- Repeat the procedure with the water at a different level.

Setting of recorder for an empty canal

If the construction of two temporary dams is not practical, the equipment illustrated in Figure 4.21 can be used. The procedure is as follows:

- Place the water-level recorder on the floor of the shelter or on the shelf; install all additional parts in position to record.
- Install a point gage in the control section at the centerline of the weir/flume (above the sill-reference location). Use a temporary, stiff support. Close the stilling-well pipe with a rubber stopper that has a small piece of tubing running through it (e.g., copper tubing). Connect a transparent hose from the tubing to a small funnel or cup.
- With the point gage, take a reading with the point resting on the weir sill or flume throat bottom in the control section. Read to ±1 mm (0.003 foot) or more precisely.
- Raise the point gage sufficiently high so that the funnel can be placed below the point. The funnel support may be either placed on the structure or attached to the point-gage support (see Figure 4.21).
- Add water to the stilling well to raise the water level in the stilling well until it is about 0.01 m (0.03 ft) below the edge of the funnel. Check to see that no air bubbles remain in the transparent hose and that there are no leaks at the stopper or in the tubing.
- Lower the point gage and read the level of the water surface in the funnel. Read the level on the recorder immediately. Repeat this step as a check.
- Calculate the difference in point-gage readings to find the sill-referenced head over the structure crest.
- Set the recorder to read the sill-referenced head obtained from the difference between the two point-gage readings.
- Check the preceding four steps at a different water level to reduce the chance of calculation or procedural mistakes.

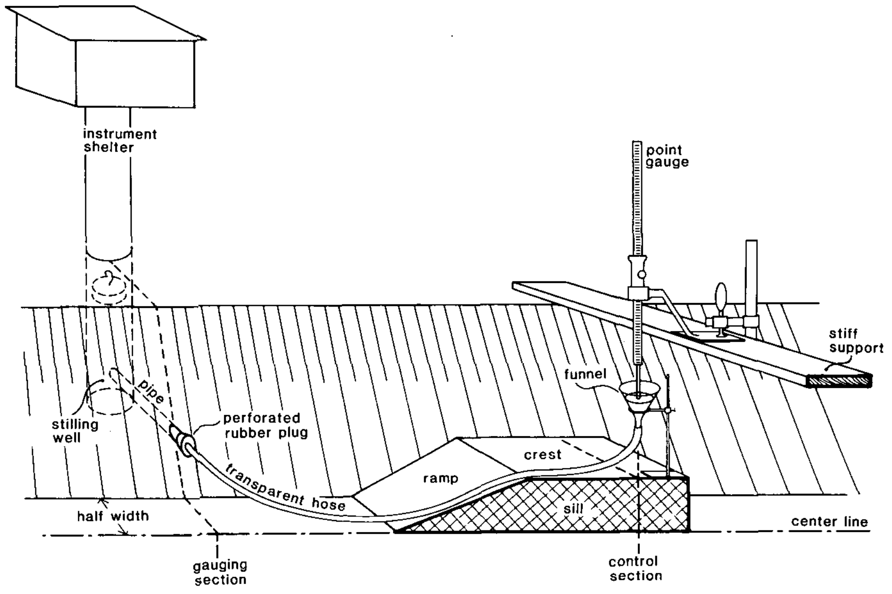


Figure 4.21 Half-view of weir showing zero-setting equipment for an empty canal.

Setting of recorder in flowing water

This method differs only slightly from the point-gage method, and can be as quick and reliable as any of the other methods. The apparatus required is shown in Figure 4.22. It includes 1) a depth sensing pipe, with a handle to place it in the water and remove it, 2) a cup that acts as a stilling well, 3) a tube connecting the sensing pipe and cup, 3) a point gage, and 4) a support beam to span the canal, to which the cup and point gage are mounted. This method is appropriate for many portable and temporary structures, particularly those structures without an approach section where a side-wall tap can be easily mounted.

The sensing pipe and hose may be any practical size. Standard 12-mm (1/2-in.) and 19-mm (3/4-in.) pipe and related parts have been used. The upstream end can be plugged with a rubber stopper and the combined pipe and stopper rough-ground to a rounded point, or the point can be welded shut and the weld ground to a rounded point. The location of the sensing holes is also not critical (typically 15-20 diameters from the rounded end of the pipe), because the pipe can lie on the canal bottom or be at any location below the water surface, parallel to the main flow, and located so that the sensing holes are near the selected distance upstream from the structure. The sensing cup (see Figure 3.21) can be fashioned from a section of pipe or from an end cap for larger plastic pipe. The point gage is commercially available through laboratory supply houses. Less sophisticated point gages can also be made in a well-equipped machine shop.

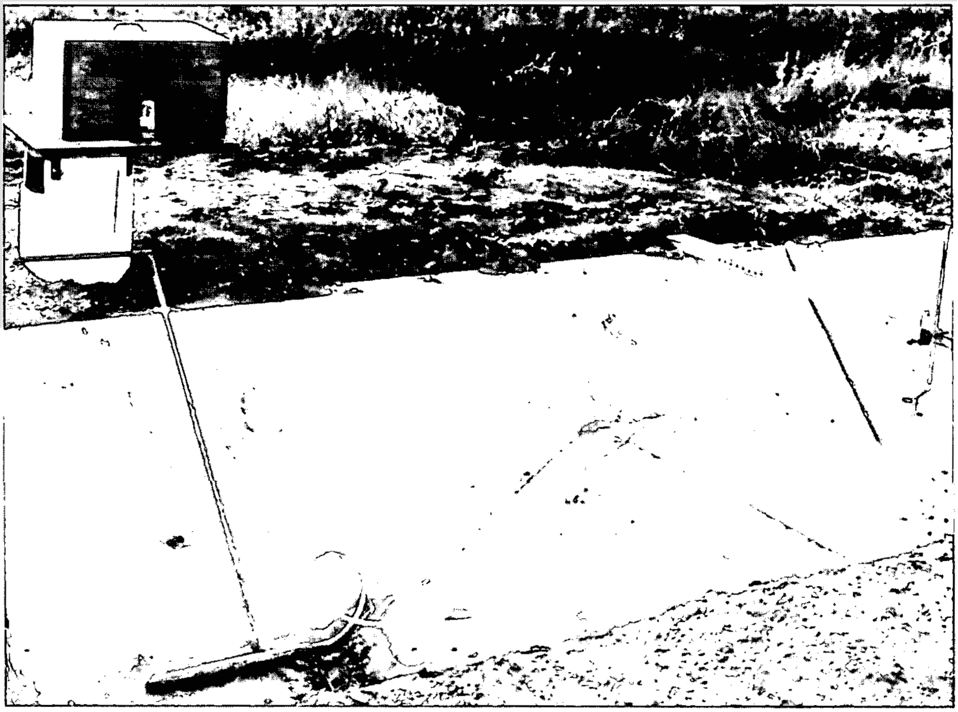


Figure 4.22 Equipment for setting a recorder in a canal with flowing water; this equipment is also used with temporary weirs.

The operating procedure is as follows:

- Place the water-level recorder on the floor of the shelter or on the shelf, and install all related equipment in position to record.
- Attach the point gage and the funnel or flat-bottomed cup to a rigid support that can span the flow of water. Attach a transparent hose to the perforated sensing pipe. The perforations are about 0.3 m from the rounded and closed nose of the sensing pipe. (For details of this type of sensing pipe, see Figure 3.20)
- Place the support with the point gage across the canal. Place the sensing pipe in the flowing stream, pointing the rounded nose directly into the direction of the flow and locating the pipe sidewall sensing holes at the gaging station.
- With the point gage, take a reading with the point resting on the weir sill or flume throat bottom in the control section (sill-reference point). Read to ±1 mm (0.003 foot) or more precisely. Do not lean on the support for the point gage. Deflection will change the point gage readings.
- Raise the point gage sufficiently high so that the funnel or cup can be placed below the point gage. (Note: Do not move the point-gage setup between these readings.)
- Lower the cup to below the water level. Purge all air from the transparent hose and attach it to the cup. Raise the cup so that the water level is several centimeters deep in the bottom of the cup, and so that the cup is above the flowing water level.

- Lower the point gage and read the water level in the cup. Repeat this step as a check. It may take a minute or so for the water level in the cup to stabilize. Compute the difference between the point-gage readings (water level minus crest level) to determine the upstream sill-referenced head to be set on the recorder.
- Set the recorder to read the head obtained above. If practical, verify the setting by repeating after the water flow has changed somewhat.

4.9.2 Placement of staff gages

For unlined canals, a vertical support for the staff gage is most suitable. A satisfactory permanent support can be constructed with a section of 180 mm (7 in.) channel iron embedded about 0.50 m (1.65 ft) in a concrete block and extended above the block to the maximum height required. The concrete block should extend well below the maximum expected frost penetration and at least 0.60 m (2 ft) below the minimum bed level of a natural stream. The top of the block should be 0.10 m (4 in.) below the lowest head to be measured. A staff of durable wood, roughly 0.02 by 0.15 m (1 by 6 in.), is bolted to the channel iron above the concrete block, and the enameled gage section is fastened to this staff with stainless steel screws.

For lined canals, the staff gage can be mounted on the inclined canal side walls. Mounting the sidewall gage in an irrigation canal is slightly different from placing a vertical gage or a recorder. Often, the canal side slopes are not exactly as intended, either because the entire canal was somewhat tilted during construction or the wall has moved. If this occurs, and a pre-marked gage that reads directly in discharge units is to be installed, a systematic error will occur due to the difference between the actual wall slope and that assumed during gage construction. To eliminate or reduce this kind of field error, the gage should be mounted relative to the weir crest so that the most common discharge range is the most accurately located. The greatest errors will then occur at gage readings that are seldom used.

The gage can be mounted to the wall with lead or plastic anchors or other suitable concrete anchor-bolting systems. Wooden plugs in drilled concrete holes are not usually durable and should be avoided. Slotted holes in the gage can be used to adjust the gage to the proper final elevation, or the holes can be carefully measured and field drilled to match the anchor locations, which usually drift slightly from their intended locations. Always check the gage after it has been fastened to ensure that it has not slipped. A second gage point (e.g., zero) should also be checked. This will likely disclose any gross errors in arithmetic in placing the gage, and will indicate whether the side-slope error is small or large. For flumes in an irrigation canal, an error in the zero reading of more than a centimeter would be cause for concern and should be carefully checked. If the side slope is very far off, it should be measured separately and a new gage constructed.

The procedure to locate and mount the staff gage correctly is illustrated in Figure 4.23:

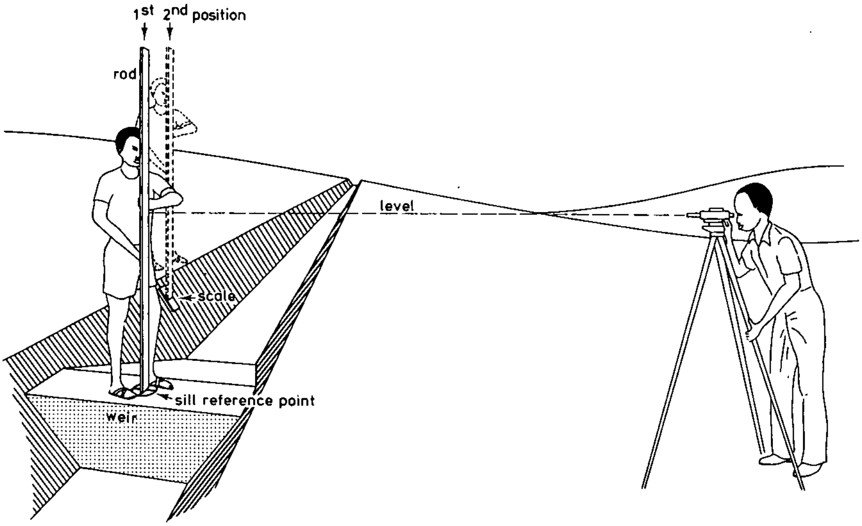


Figure 4.23 Stages of setting a gage on a lined canal side slope.

- Determine the location of the gaging section by using Figure 4.1 and mark this section on the canal side slope.
- With a surveyor's level, take a back sight on the sill crest at the sill-reference point (first position in Figure 4.23) to get the sill-reference level. All rod readings are to ±1 mm (0.003 foot) or more precise.
- Find the most common discharge, Q, to be measured and read the related h_1-value from the proper rating table.
- Subtract this h_1-value from the back sight obtained at the first position to find the value that must be read on the rod if it were to be placed on the mark for the above h_1- or Q-value on the scale.
- Place the gage on the side slope in approximately the correct location (second position in Figure 4.23). Set the rod on the gage at the mark for the most common discharge. Move the gage and rod up or down to the proper rod reading computed above (Figure 4.24).
- Mark the gage holes or slots and the gage top and bottom on the canal wall. Drill the holes, secure the anchors, and tentatively attach the gage to the canal wall.
- Check the rod reading on the gage at the most common discharge and adjust the gage to the correct location (i.e., repeat above steps), and fasten securely.

The same procedure can be used for vertical staff gages. If surveying equipment is not available, other methods similar to those used for zeroing recorders can be used.

4.10 Operation of Portable Structures

Upon installation of a portable or temporary structure into flowing water, the required head loss will cause a noticeable backwater effect upstream from the device. The upstream extent of the backwater effect is mainly determined by the

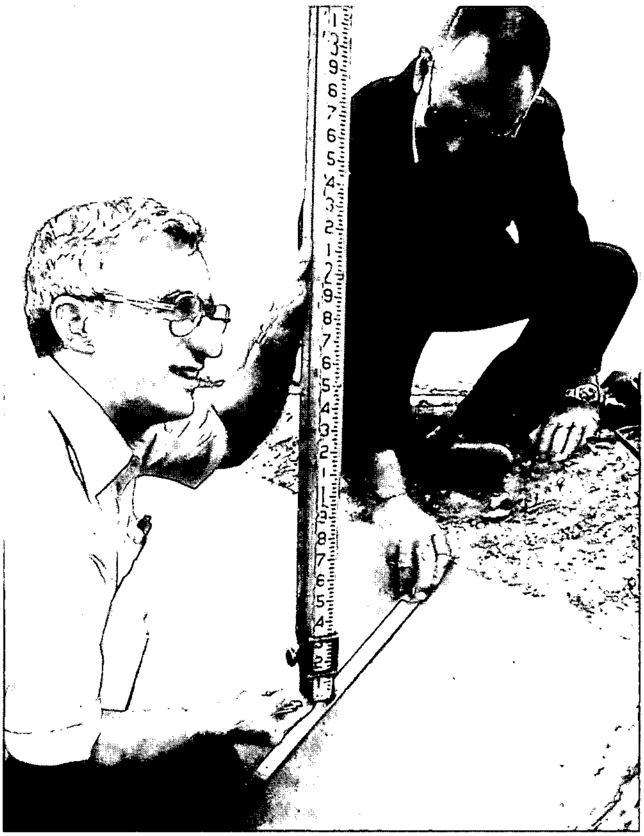


Figure 4.24 Setting of the rod on the inclined gage.

head loss over the structure and the hydraulic gradient of the channel. As a rule of thumb, this distance is about equal to the head loss across the flume divided by the canal bed slope, $\Delta h/S_b$. In flat-gradient canals, it is important to use a structure that needs little head loss for modular flow for the following reasons:

- An upstream water level that is too high may affect the flow through upstream control structures.
- Seepage through the channel banks may increase and thus reduce the flow at the measuring site because of the increased upstream water depth. This is especially the case in earthen canals.
- The backwater effect causes a delay between the installation of the structure and the resumption of steady flow. Minimizing the backwater effect minimizes the delay.

As indicated in Section 1.3, the head loss required for modular flow through a long-throated flume is minimal compared to all other structures. The rating tables of Appendix 4 show this minimal head loss requirement for several types of structures.

Figure 4.25 shows that following installation of the portable weir into the flowing water, the discharge downstream from the site will decrease while the wedge-shaped

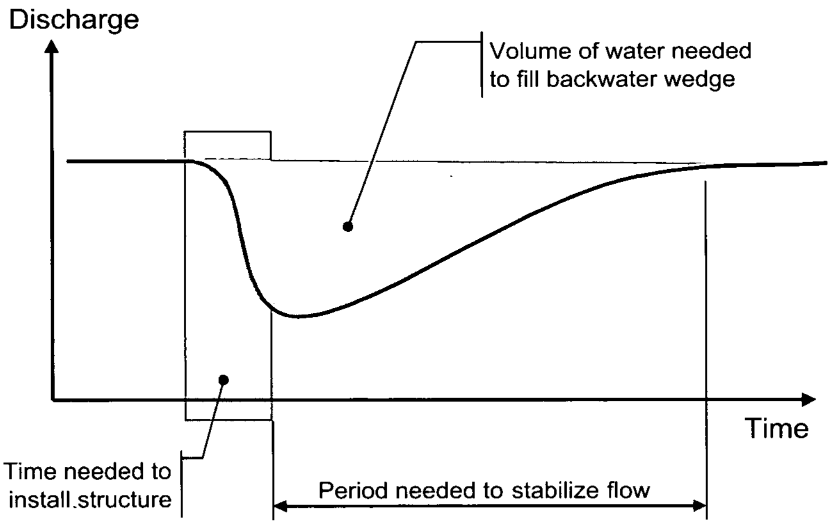


Figure 4.25 Change of flow downstream of the measuring site due to the installation of a portable structure.

volume of water between the original water surface and the backwater curve is filled. Especially in flat gradient canals and with low discharges, several head readings should be taken to determine whether flow is stable. During the time required for the flow rate to stabilize, check to be sure that flow is not passing under or around the structure. If there is leakage in earthen channels, it should be minimized by packing additional soil around the structure or reinstalling cutoff sheets. For lined canals, reset the structure (first remove if necessary) or provide additional seals between the structure and the canal lining.

Two methods of head measurement are available for portable and temporary structures: a dipstick or gage with a translocated stilling well that is attached to the structure (e.g., Figures 3.22 and 3.43) or a loose point gage and head sensing pipe (e.g., Figures 3.16 and 4.22).

4.10.1 Attached stilling well

With small capacity portable flumes, such as the RBC flumes described in Section 3.3.3, it is practical to measure the head h_1 in a translocated stilling well. The stilling well is mounted near the control section to minimize the influence of a slightly non-level installation on the measured head, h_1, in this well (Figure 4.26). Cross-slope leveling of the flume is accomplished by keeping the top upstream edge of the device parallel to the upstream water surface. For the circular portable flume shown in Figure 4.26, cross-leveling is more difficult because there is not a straight top edge to compare to the upstream water level, so the center placement of the translocated stilling well is even more important. Leveling of the longitudinal slope can be performed using a carpenter's level. Experienced users can soon judge adequate

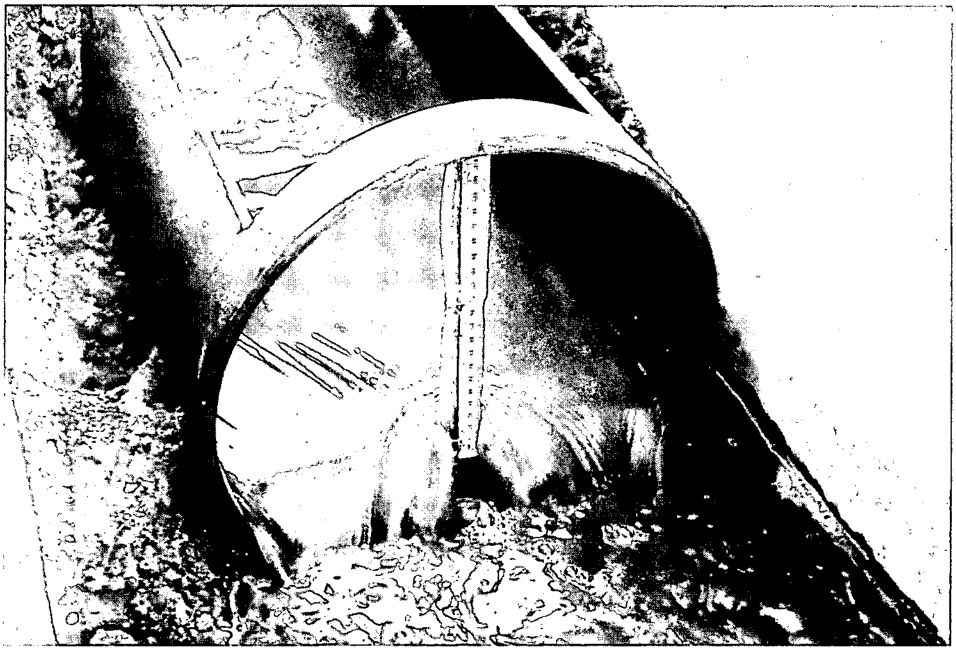


Figure 4.26 A small, portable weir in a circular pipe section with a translocated stilling well on the centerline of the device (Morocco).

leveling and will not require the carpenter's level. If the portable flume is installed for seasonal or semi-permanent flow measurement, we advise using the alternative stilling-well location on the side of the flume (see Figure 3.41) to prevent the unattended tube from collecting floating debris. Use of a dipstick or point gage for these stilling wells is discussed in Sections 4.3 and 4.6.

4.10.2 Loose head-sensing pipe

For larger-capacity flumes, combining a fixed gaging station, approach channel, converging section, and throat into one device would yield a long and heavy structure. To make the structure more easily portable, the head-sensing equipment is separated from the structure, or made retractable as in the weirs of Figures 3.18 through 3.21. The temporary structure may remain in the channel during the period of flow measurement while the head-sensing equipment remains portable. The equipment consists of a head-sensing pipe, stilling well cup and point gage as described in Section 4.9.1 for zero setting of a recorder in flowing water. The stilling well cup is translocated to a position above the control section to make the measurement insensitive to minor problems of leveling of the temporary crest. This also eliminates the need for lateral transfers of elevations by surveying techniques. Thus, careful leveling is not necessary, just gross adjustments readily observed by the eye.

By following this procedure, an accurate discharge measurement (error ±3%) can typically be made in less than 10 minutes:

- Determine the location of the gaging station by using Figure 4.1 and mark this section on the canal side slope. Place the sensing pipe in the canal at this location.
- Attach the point gage and flat-bottomed cup to a rigid support that can span the flow of water. Attach a transparent hose to the perforated sensing pipe. The perforations are about 0.3 m from the rounded and closed nose of the sensing pipe. (For details of this type of sensing pipe, see Section 4.9.1 and Figure 3.20).
- Place a stiff support with attached point gage across the canal. Place the sensing pipe in the flowing stream, pointing the rounded nose directly into the direction of the flow and locating the pipe sidewall sensing holes at the gaging station.
- With the point gage, take a reading with the point resting on the weir sill or flume throat bottom in the control section (sill-reference point). Read to ±1 mm (0.003 ft) or more precisely.
- Raise the point gage sufficiently high so that the cup can be placed below the point gage. (Note: Do not move the point-gage setup between these readings.)
- Lower the cup to below the water level. Purge all air from the transparent hose (see Figure 3.18) and attach it to the cup. Raise the cup so that the water level is several centimeters deep in the bottom of the cup, and so that the cup is above the flowing water level.
- Lower the point gage and read the water level in the cup. Repeat this step as a check. It may take a minute or so for the water level in the cup to stabilize. Compute the difference between the point-gage readings to determine the upstream sill-referenced head and read the flow rate from a table.

As already mentioned, this head sensing equipment can be used to make accurate (X_Q = ±3%) discharge measurements. To a great extent this is due to the accuracy with which the upstream sill-referenced head is measured with a point gage (see Section 2.8).

5. Design Process

5.1 Introduction

This chapter presents the issues relevant to the design of flow-measuring flumes: satisfying flume design criteria, meeting the conditions of the channel in which the measuring structure is placed, and satisfying the objectives of the designer. Guidelines are provided for designing structures for some of the common conditions encountered, and the design process is presented and illustrated with examples.

The methods presented in this book can be used to design a flow-measurement or flow-control structure in a channel of arbitrary shape and size, provided that an acceptable structure is feasible. The shapes of the approach and tailwater channels may be different. Most natural and artificial channels can be modeled using one of the following shapes:

- Simple trapezoid,
- Rectangle,
- V-shape,
- Circle,
- U-shape,
- Parabola, or
- Complex trapezoid.

The size of the channel cross-section should be obtained from design drawings or measured in the field. Additional shapes are available for use in the flume throat to create flow contraction and provide for convenient construction. These shapes are shown in Figure 5.1.

5.2 Design Criteria

Design of long-throated flumes and broad-crested weirs should be based on the following criteria:

- Critical flow should occur so that the relationship between upstream head and discharge is not influenced by the water level downstream from the structure. When this occurs, flow is considered modular (i.e., the structure is performing as a module). Obtaining critical flow conditions that can be reliably rated requires proper design of the throat section of the structure, i.e., sufficient sill height and appropriate width and length of the throat section (see Section 6.3.3).
- The structure should not raise the upstream water level so much that it overtops or damages the upstream channel, or interferes with operation of the channel (e.g., causes flow to be improperly divided upstream).

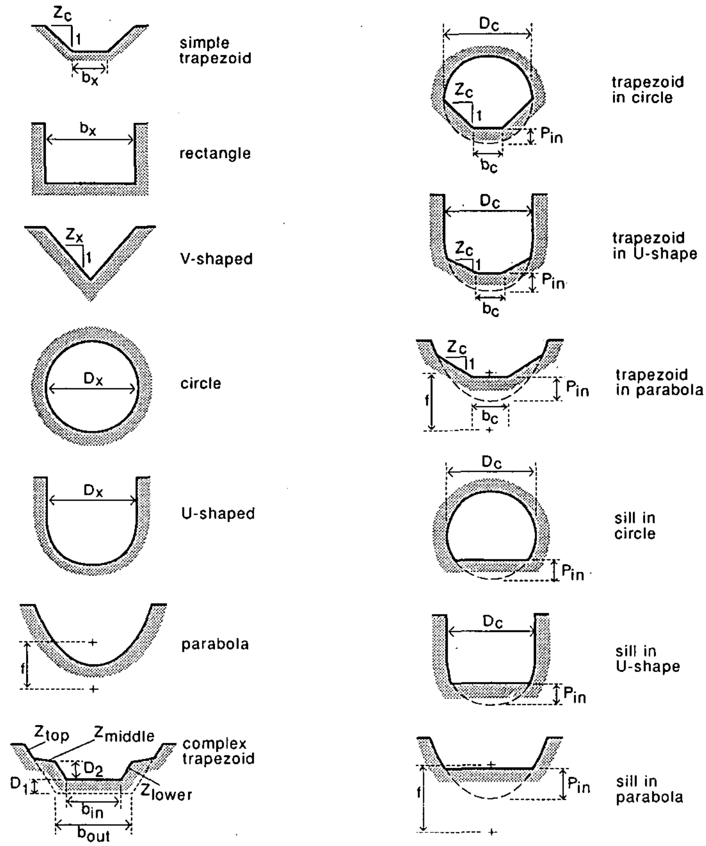


Figure 5.1 Alternative shapes for the approach and tailwater channels and the control section.

- The structure should be designed so that the upstream water level can be reliably and accurately read and so that the relationship between head and discharge is maintained. This requires, among other things, that the Froude number in the channel is sufficiently low that standing waves or a choppy water surface do not occur in the approach section of the flume.
- The structure should also be designed so that, with appropriate head-measurement techniques, the measured flow rates are sufficiently accurate for the intended purpose over the range of flows to be measured.

The design process presented in this chapter is intended to assure that these design criteria can be satisfied.

The first step in flume design is to determine the conditions under which the structure must operate. This starts with determining the conditions in the existing channel. It is assumed that the designer has chosen a preliminary site for the structure based on the criteria discussed in Chapter 2. However, it is recognized that the chosen site may not be suitable hydraulically, requiring that another site be chosen. Thus, design can be an iterative process.

5.3 Defining Existing-Channel Conditions

While we encourage flumes to be designed for new channel systems, it is recognized that most structures will be retrofitted into existing channels. The design procedures discussed below apply equally to both. However, our use of the term "existing channel" is also meant to apply to new channels that are proposed. Once the channel cross-section, elevations, location, etc. are proposed, from the standpoint of flume design, it becomes an existing channel.

5.3.1 Range of flows to be measured

As discussed in Section 2.4, one must consider the range of flows to be measured. This is the primary range over which an accurate flow measurement is required. There may be low flows that occur but are not of concern for accurate measurement, for example during the winter season when no water deliveries are being made from the channel. Some channels experience flood flows that also do not require accurate measurement, and thus do not influence the design of an accurate structure, except that one must ensure that the flume can pass these flood flows within the existing channel. Thus, a flume can be submerged by flood flows that are beyond the measurement range of interest, as long as it will pass the flow. (It is important to note that when a flume becomes submerged, the rating developed using the theory of Chapter 6 and the computer program in Chapter 8 is not accurate because critical depth does not occur. When submerged, the upstream head for a given flow rate will be greater than that predicted.) The range of flows can have a significant influence on the design of the structure, since all conditions must be satisfied over the full range of discharges—i.e., downstream submergence, accuracy, Froude number, freeboard, and flume dimensions.

5.3.2 Determining head-discharge relationship of existing channel

The water level at any particular point in a channel is influenced by the channel roughness and cross section properties, the flow rate, and backwater caused by flow obstructions downstream. In the absence of backwater (or drawdown) effects, the flow depth at a given discharge is uniform over the length of the channel. The flow depth under uniform flow is known as normal depth. Under uniform flow, the relationship between flow rate and water depth can be described by the well-known Manning equation

$$Q = \frac{C_u}{n} A R^{2/3} S_f^{\;1/2} \qquad 5.1$$

where C_u is a coefficient that depends upon units (1.0 for discharge in m^3/s and distances in meters and 1.486 for discharge in ft^3/s and distances in feet), A is the cross-sectional flow area, R is the hydraulic radius (area divided by wetted

Table 5.1 Conservative values of Manning's roughness coefficient to estimate water levels downstream from a flume.

Concrete-lined	
Float finished	0.018
Float finished, with gravel on bottom	0.020
Gunite	0.025
With algae growth	0.030
Masonry	
Cemented rubble	0.030
Dry rubble, open joints	0.035
Earthen channels	
Straight and uniform, few weeds	0.035
Winding, cobble bottom, clean sides	0.050
Non-uniform, light vegetation on banks	0.060
Not maintained, weeds and brush uncut	0.150

perimeter), S_f is the friction slope (equal to the bottom slope under uniform flow), and n is the Manning roughness coefficient. Values of Manning's n for common channel materials are given in Table 5.1. If the depth-discharge relationship in the channel is governed by Equation 5.1, then we can use this equation to estimate conditions downstream from a proposed flume and to assist in flume design.
When the Manning equation is valid, the relationship between depth and discharge is relatively well behaved and passes through zero depth and zero discharge. However, when the channel cross section is poorly defined, Equation 5.1 may be difficult to apply, and if the site is influenced by backwater, Equation 5.1 cannot be used at all because flow is no longer at normal depth. In either of these cases it may be easier to express the depth-discharge relationship empirically with a power equation such as

$$Q = K_1 y_2^u \qquad 5.2$$

where y_2 is the water depth in the channel downstream from the proposed flume and K_1 is an empirical constant. The value of K_1 depends on the size of the channel, while the value of the power u depends on the shape of the channel. For wide and shallow channels under uniform flow (i.e., when Equation 5.1 applies), u is about 1.6, while for deep and narrow lined canals, u may be as high as 2.4. The values of K_1 and u can be determined by curve-fitting through two points of known discharge and depth. The necessary data can be obtained from field measurements or by analytical means, such as a detailed backwater analysis (for details, consult a hydraulics text or one of many available computer programs).

When the site is strongly influenced by backwater, the relationship given by Equation 5.2 is not adequate. For example, if the backwater is caused by a downstream weir, the depth-discharge relation will no longer pass through zero depth and zero discharge. In such cases, a modified form of Equation 5.2 has proven satisfactory for flume design work, namely

$$Q = K_1 (y_2 - K_2)^u \qquad 5.3$$

where K_2 is an empirical constant that accounts for the fact that the depth is greater than zero at zero flow.

For flume design, we are interested in the water levels downstream from the proposed flume site (tailwater levels) at both maximum and minimum flow. In almost all cases, if the flow in the flume is modular at these extremes, it will be modular in between. However, it is often not practical to measure the tailwater levels at these two exact flow rates, and water levels are instead measured at some intermediate discharges. The above relationships can be used in various forms to estimate the water levels downstream from a proposed flume at maximum and minimum flow based on measurements or estimates made at other flow rates (see Figure 5.2). There are several methods available to do this depending on what information is available. The following methods are discussed:

- Calculation of Q versus y_2 using Manning's equation and a user specified roughness coefficient (Manning's n) and friction slope (usually equal to the bottom slope);
- Extrapolation through one measured Q versus y_2 data point using Manning's equation, and assuming constant but unknown roughness and slope;
- Extrapolation using a power curve of the form $Q = K_1 y_2^u$, where K_1 and u are constants determined by fitting to two Q versus y_2 data points;
- Extrapolation using a power curve of the form $Q = K_1 (y_2 - K_2)^u$, where K_2, u, and K_2 are constants determined using three Q versus y_2 data points; and
- Interpolation/extrapolation using a Q versus y_2 lookup table.

The last method allows determination of tailwater conditions that are not well modeled by any of the other methods.

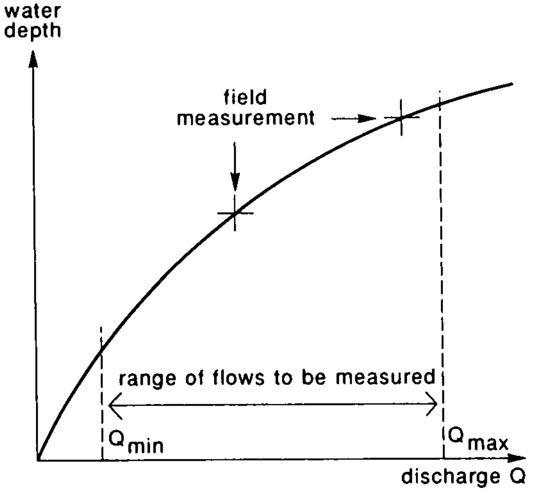


Figure 5.2 Terminology related to water level versus discharge curve of a channel.

Manning's equation

If the flow in the downstream channel is at normal depth, tailwater depths at minimum and maximum flow can be determined using a combination of the continuity equation and Manning's equation (5.1). When the flow in the downstream channel is at normal depth, the friction slope and bed slope of the channel are equal. For the flow in the downstream channel to be at normal depth, the downstream channel must be of uniform cross-section, slope, and roughness for a sufficient distance such that the water level at the flume site is controlled solely by the frictional resistance of the downstream channel (i.e., no backwater effects).

For a given discharge, Q, Equation 5.1 can be solved iteratively for y_2, which is embedded within the $AR^{2/3}$ term. If a flume is to be designed in a new canal, the shape, dimensions, and bottom slope of the canal should be taken from the design drawings. However, a value of the Manning roughness coefficient should be selected that reflects the worst expected seasonal and maintenance conditions. This n-value should thus be higher than the n-value used in the canal design. The n-values given in Table 5.1 may be used for a preliminary estimate.

Manning's equation using one Q versus y_2 value

If values of the Manning roughness coefficient and bed slope are not available, a single field-measured value of y_2 and the corresponding discharge can still be used to apply Manning's equation, assuming that the slope and n-value are constant. Equation 5.1 can be rearranged to obtain

$$\frac{C_u S_f^{1/2}}{n} = \frac{Q}{AR^{2/3}} \qquad 5.4$$

While $C_u S_f^{1/2}/n$ remains constant, Equation 5.4 can be applied to other discharges in the form

$$\frac{Q_1}{(AR^{2/3})_1} = \frac{Q_2}{(AR^{2/3})_2} \qquad 5.5$$

With one depth-discharge pair known (e.g., the left side of Equation 5.5), Equation 5.5 can be solved for discharge at any other water depth. Specific values of S_f and n do not need to be determined individually.

Power curve using two Q versus y_2 values

The two-point power curve method can be used regardless of whether or not the flow in the downstream channel is at normal depth. It should be noted that this method implies that at a discharge of zero, the tailwater level is also zero. The general procedure is to estimate the water depth in the channel at two discharges and then to solve Equation 5.2 based on these two data pairs. Applying Equation 5.2 to each data pair gives two equations and two unknowns, K_1 and u. The two data pairs can be obtained by measuring the discharge and tailwater levels at the site where the structure is to be built, by computing tailwater levels using a backwater analysis such as the standard-step or direct-step methods, or for new canals the data points may be taken from the design report. Then Equation 5.2 is used to compute water levels at Q_{min} and Q_{max}. To obtain the most accurate estimate of the tailwater depth at Q_{min} and Q_{max}, the two measured data points should be as near as practical to the minimum and maximum discharges to be measured.

Power curve with offset using three Q versus y_2 values

This method is similar to the two-point power curve method, except that it does not assume the tailwater level to be zero when the discharge is zero. This method might be appropriate when the downstream water level is controlled by a weir or overflow gate, so that the tailwater level is non-zero at zero discharge. Three sets of Q and y_2 values are required, and one of the data points must be the tailwater level at zero flow. These data are used to determine the coefficients K_1, u, and K_2 in Equation 5.3. Again, to obtain good estimates of the tailwater depth at Q_{min} and Q_{max}, two of the measured data points should be as near as practical to the minimum and maximum discharges to be measured. If field data are used, the data should be obtained during the season when the downstream channel roughness is a maximum.

Q versus y_2 lookup table

The final method for determining tailwater levels is the use of a lookup table relating Q and y_2. WinFlume can accept up to 20 pairs of Q-y_2 values. Tailwater levels at discharges between the minimum and maximum flow provided in the lookup table are determined by linear interpolation; outside of the bounds of the lookup table, linear extrapolation is used. This method should only be used if none of the other methods properly describes the tailwater conditions at the site.

Example

Given: a canal with bottom width b = 0.30 m, side slopes of 1:1, depth d = 0.55 m, bottom slope S_b = 0.00050 m/m, and a Manning roughness coefficient, n = 0.015, and a range of discharges from Q_{min} = 0.05 m^3/s to Q_{max} = 0.15 m^3/s;

Find: the tailwater levels assuming water flows at normal depth (i.e., no backwater effects from downstream structures and $S_f = S_b$).

Solution: Equation 5.1 can be used to determine the tailwater levels, which are y_2 = 0.240 m at Q_{min} and y_2 = 0.412 m at Q_{max}. Note that this requires trial and error or an iterative solution since depth is not expressed as a function of discharge. The WinFlume software described in Chapter 8 makes these calculations automatically. Solution of the Manning equation gives the following values:

Discharge, Q (m^3/s)	Depth, y_2 (m)	Area, A (m^2)	Hydraulic Radius, R (m)
0.05	0.240	0.130	0.132
0.07	0.284	0.166	0.150
0.10	0.339	0.217	0.172
0.12	0.371	0.249	0.184
0.15	0.412	0.293	0.200

Manning equation using one Q versus y_2 value

Now suppose that you only make one observation of depth and discharge, say at 0.10 m³/s and 0.339 m depth. If we calculate the value on the right hand side of Equation 5.4 from the above values, we obtain

$$\frac{Q}{AR^{2/3}} = 1.490 \qquad \text{and} \qquad \frac{C_u S_f^{1/2}}{n} = 1.491$$

the difference being due to roundoff errors. Solving for water depth at maximum and minimum discharge, we get tailwater depths of 0.239 m and 0.412 m, respectively.

Power curve using two Q versus y_2 values

Now suppose that we have two head discharge measurements; 0.284 m at 0.07 m³/s and 0.371 m at 0.12 m³/s. Applying Equation 5.2 for both of these conditions, we get

$$0.07 = K_1(0.284)^u \qquad \text{and} \qquad 0.12 = K_1(0.371)^u$$

Solving these two equations for the two unknowns gives u = 2.017 and K_l = 0.887. These can then be applied in Equation 5.2 at Q_{min} and Q_{max} to give tailwater depths of 0.240 m and 0.414 m, respectively. (Note: Differences are due to roundoff error). If backwater from downstream structures was influencing the water depth at the structure, extrapolation with these two methods from measured values to water depths at Q_{min} and Q_{max} would introduce some error, because the first method assumed flow was at normal depth, and the second method is only an empirical approximation of the depth-discharge relationship.

5.3.3 Froude number of existing channel

The Froude number in the approach channel, Fr_1, which is the square root of the ratio of inertial to gravity forces, plays an important role in flume and weir design (Equation 2.18). At the location of critical flow in the flume throat, the Froude number is unity. Upstream from the flume throat where velocity is lower, the Froude number is less than unity. Although flumes with any approach channel Froude number are theoretically possible, flumes with high approach channel Froude numbers are problematic. Even in standard channel designs, Froude numbers in the range 0.85 to 1.15 are avoided due to unstable behavior. Above a Froude number of about 0.5, standing waves caused by channel transitions make determination of upstream water level and head subject to very large errors. To be conservative, we recommend that the Froude number in the approach channel be limited to 0.45 (although the computer design routine allows the Froude number to be as high as 0.5).

A low Froude number in the approach channel helps to reduce potential flow measurement errors in two ways. First, the measurement of upstream head is more accurate, due to the elimination of standing waves and excessive turbulence. When estimating errors in water level measurement, a Froude number of 0.2 or lower is recognized as providing a significant improvement in head-measurement uncertainty. Second, flume calibrations are less sensitive to construction anomalies at low Froude numbers. For this purpose, Froude numbers below 0.3 are preferred. However, where sedimentation is a potential problem, it may be necessary to have a large approach channel Froude number to keep sediment moving through the flume. Under these conditions, the idea is for the flume to cause a minimum reduction in approach flow velocity, regardless of the Froude number.

For a given discharge, an increase in upstream water depth will decrease the Froude number. The limit on the approach channel Froude number effectively establishes the minimum upstream water depth for that discharge, regardless of flume shape or configuration. If such a depth is feasible (i.e., does not overtop the canal) and provides adequate freeboard, then a flume can be designed that does not violate the Froude number criteria. Otherwise, a flume cannot be designed for that channel for that discharge, regardless of flume cross section or submergence considerations. The only recourse would be to enlarge the channel by increasing its width or depth.

5.3.4 Freeboard of existing channel

There is an inherent tradeoff in flume design between maximizing the amount of contraction to avoid submergence from downstream tailwater and minimizing the amount of contraction to provide adequate freeboard, to avoid overtopping the upstream channel. This is fundamental to the design process. Several methods can be used to define the required freeboard (percentage of depth, percentage of head on flume, numeric value), and even then, the designer may select a design that violates the freeboard requirements at maximum flow. In general, freeboard is provided to account for the uncertainty in the head-discharge relationship of the channel and allow for the possibility of wave action. Sources of uncertainty in the head-discharge relationship include changes in flow resistance over time, sediment deposition, or the presence of temporary obstructions. Wind and disturbances to the flow at the surface can cause waves that may overtop the canal or the lining and eventually cause damage, even though the normal water depth is within the canal prism. As discussed in Section 2.3, to estimate the risk associated with extreme events, consideration should be given to the flow rate at which the channel overtops.

The Natural Resources Conservation Service (formerly Soil Conservation Service) (SCS 1977) recommends that the freeboard be at least 20% of the maximum flow depth in a canal. This allows for the expected variation in normal depth in an operating canal, which can be considerable as channel roughness changes. However, when we have a flume in place, there is less uncertainty regarding the upstream depth for a given flow rate since the flume head-discharge relationship is well known. Thus for small, lined trapezoidal irrigation canals, we recommend that the canal freeboard be at least 20% of h_{1max}, thus $F_1 \geq 0.2h_{1max}$. This typically allows for an overage in discharge of about 40%, not including wave action, before the canal is overtopped. Naturally, if wave action or large operational surges are anticipated, then these must control the design freeboard. However, such conditions are not conducive to accurate flow measurement and should be avoided.

In some cases, the flume designer wants to consider a fixed freeboard depth (e.g., 0.1 m or 4 in.). This freeboard requirement does not vary as the upstream head varies with changes in the flume design. Setting a fixed freeboard allows one to set the upstream water depth for Q_{max}. It should be recognized that freeboard is based on the upstream water level, while the energy head of the flow may be higher. It is possible to design a flume so that it has adequate freeboard, but the energy head is actually above the top of the canal. In such cases, a disturbance to the surface of the flow could cause the canal to overtop, since the water depth, locally, can rise to the level of the energy head. Larger canals should have an engineering evaluation of the freeboard requirements, since the consequences of overtopping can be severe.

The freeboard requirement is only a concern at Q_{max}, while submergence of a flume is a concern at both Q_{min} and Q_{max}. Sometimes, to avoid submergence of a flume at low flows (e.g., Q_{min}), a design is required that causes violation of the freeboard requirements at Q_{max}. In many cases the decision must be made to provide adequate freeboard at high flows and allow the flume to become submerged at low flows. This effectively reduces the range of flows that can be measured in order to ensure the safety of the canal.

5.4 Flume Design Objectives and Issues

As discussed previously, the primary tradeoff in flume design is between enough contraction to avoid submergence, but not so much contraction that the freeboard requirement is violated. Where a canal is already flowing relatively full, it may not be possible to place a flume in the canal without overtopping at high flows or becoming submerged. In such cases, another site should be chosen for flow measurement or the canal banks must be raised. More frequently, there is a range of contraction amounts that will satisfy these two criteria. Then, within this range of contractions, other design criteria and objectives come into play.

5.4.1 Method of contraction change

One of the essential decisions in flume design is how to provide the contraction required to create critical flow within the control section. For existing lined trapezoidal irrigation canals, the most cost-effective method is to construct a broad-crested weir by placing a bottom contraction in the existing lining. However, if the properties of the resulting cross section do not satisfy one or more of the design considerations, then an alternative contraction is needed. Other options include creating a contraction from the side by bringing the sloping sidewalls in (e.g., reducing the bottom width), contracting from the side by changing the sidewalls to vertical walls (i.e., making the section rectangular), providing both a contraction from the side and bottom, or creating a complex shape that has different relative amounts of contraction over the range of discharges.

The flume design process starts with an assumed initial shape for the control section. The default is the same shape as the upstream approach channel. Then the designer must decide how the shape is to be modified to achieve the needed contraction. There are several common methods for changing the amount of contraction (Figure 5.3):

- Add contraction from the bottom (raise height of sill),
- Raise or lower the entire section,
- Raise or lower the inner section (applies only to complex shapes), or
- Vary the amount of side contraction.

If these methods of contraction change do not provide a satisfactory design, one can combine some of the methods, for example making both a side and bottom contraction, or one can make changes to other throat cross-section dimensions, such as the side slope. The concerns here are functionality and constructability. One wants to construct the simplest, least expensive structure that will perform as needed. Thus, the general strategy is to start with the simplest shape to construct and then modify that shape if the design criteria cannot be met.

Raise or lower height of sill

With this option, the bottom of the control section is moved vertically while all the other parts remain in the same position relative to the approach channel. This, is the option to use for designing a broad-crested weir in a lined canal. In some cases, the shape actually changes with this option. For example, a sill in a U-shape could become a rectangle (bottom moved above the circular part of the U), or a trapezoid

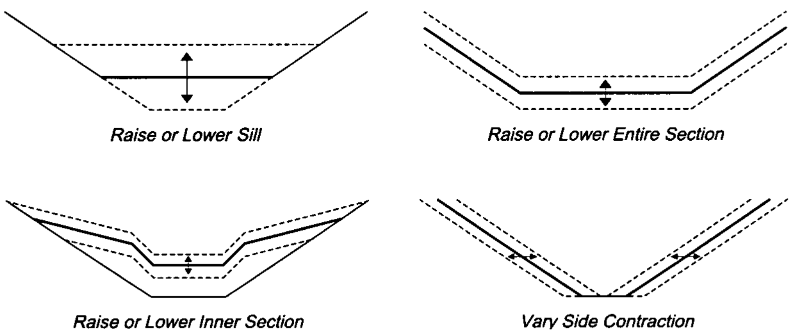


Figure 5.3 Methods for changing contraction amount for a flume.

in a circle or in a parabola could become a sill in a circle or parabola, respectively. Although the newest flume design software can both raise and lower the sill, it is usually best to start with a low sill and raise it as needed.

Raise or lower entire section

With this option, the entire control section is moved up or down relative to the approach channel bottom to provide the desired amount of contraction. The shape and the dimensions of the control section do not change with this option. Hence, the amount of contraction is changed by raising or lowering the sill height, p_1 (p_1 will not be reduced below zero). This option is useful if a standard design (i.e., prefabricated) flume is to be placed in a channel. Examples include fiberglass rectangular flumes placed in an unlined channel, or a section of pipe used to make a flume throat. Such flumes are common for flow-survey work. This option could be used to test the feasibility of using standard-sized structures in an irrigation system, as opposed to designing custom structures for each site.

Raise or lower inner section

For some situations, it is necessary to construct a throat inside of an existing channel shape to provide the necessary contraction. An example would be a small trapezoid inside a pipe (i.e., Palmer-Bowlus flume). Under this contraction-change strategy, the inner trapezoid (the part that moves up and down) remains intact, with no changes in dimension values. The outer shape remains fixed, relative to the approach channel, while the inner trapezoid moves up and down. This option is useful for designing a complex flume in an existing channel, where the existing channel becomes the outer shape, or for designing a trapezoidal control within an existing channel that is circular, U-shaped, or parabolic. This option allows an efficient design without disturbing the existing channel section. (Note that the WinFlume program will not reduce the inner sill height below zero). A small trapezoid is sometimes placed within an existing trapezoidal channel to provide a wide flow measurement range, with the small trapezoid providing good measurement accuracy at low flows.

Vary side contraction

For trapezoidal flumes, as opposed to broad-crested weirs, very often the side slope angle is fixed and the design is based on changing the bottom width in the control section. Under this strategy, the vertical location of the throat does not move, and the width is adjusted to provide the needed contraction. Some shapes do not have a defined bottom width, and another appropriate dimension is changed instead to vary the width of the throat; for example, the diameter of U-shaped and circular cross sections, or the focal length of a parabola. For a trapezoid in another shape, the outer shape is assumed to match the upstream channel, so it typically is not adjustable, and the bottom width of the inner trapezoid is changed. For triangular control-section shapes, the only adjustable parameter is the side slope. For weirs with vertically movable crests as described in Section 3.5, varying the side contraction is the only design strategy that makes sense.

5.4.2 Head loss design aims

For many channels, there is sufficient capacity so that at Q_{max} there is more available drop across the proposed flume than necessary. Under such a situation, there may be a wide range of contraction amounts (sill heights, bottom widths, etc.) that will produce an acceptable design. Assuming that other design criteria do not control the design (e.g., the Froude number limitation at maximum flow), there will be a maximum contraction amount that produces the minimum freeboard, but more head loss than needed, and there will be a minimum contraction amount that requires minimum head loss and provides more than enough freeboard (Figure 5.4). The design can be fine-tuned between these extremes by examining the tradeoffs among the various design objectives and selecting the contraction amount that provides the best overall performance.

The minimum amount of contraction often provides a good design, since under these conditions, the flume causes the least amount of change to the existing flow conditions, while still providing accurate measurement. At the same time, it provides the most safety against overtopping. If deposition of sediment being transported in the channel is a concern, this is one of the preferred options. (see Section 2.6.3 for additional options on design for sediment-laden channels). However, designing for the minimum amount of contraction also means that the design is on the verge of being submerged (unless the approach channel Froude number is controlling the design). Thus, such a design requires accurate knowledge of the tailwater conditions. Since such conditions are often not well known, such a design is a bit risky; the actual tailwater conditions may be slightly higher, causing non-modular flow and inaccurate flow measurement. Furthermore, in channels with relatively high Froude numbers, the minimum contraction amount may be limited by the approach channel Froude number requirement, so a minimum contraction design will produce a flume with a Froude number that is on the high end of the recommended range. A design with more contraction may produce a more satisfactory structure.

The maximum amount of contraction is a useful design alternative when the flow conditions in the channel are highly uncertain and the maximum flow is accurately known and controllable (as it is in many irrigation canals). Since the head-discharge relationship of the flume is accurately known, adequate freeboard can be ensured at Q_{max}, and thus the design is relatively safe and requires less certain knowledge of the downstream depth-discharge relationship. For this amount of contraction, if the

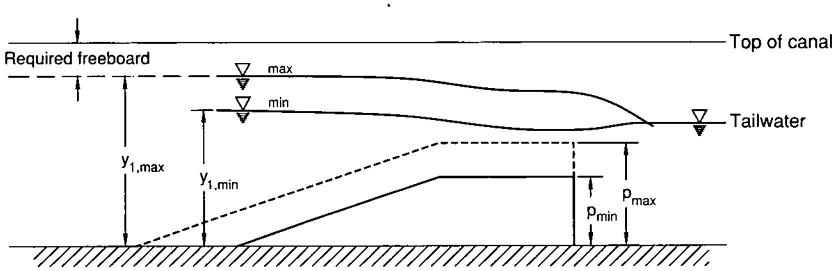


Figure 5.4 Tradeoffs in selection of contraction amount.

Froude number is still too high, this is not a suitable site for flow measurement, unless the upstream water surface can be raised (e.g., raising the upstream sidewalls).

The usual course is to select a contraction amount that falls between these two extremes. Often the contraction amount is chosen so that the dimensions of the flume are whole numbers, if possible, or fractional numbers that do not complicate construction or imply a much higher degree of precision than needed. The exact value selected within this range depends upon the factors discussed above. However, it is usually much easier to add more contraction later than to take contraction away, especially when the contraction is made of concrete.

Another strategy for design of flumes can be used when there is a natural drop in the channel bottom (Figure 5.5). With such a drop, it may be possible for the head loss across the flume to match the drop in the channel. In this case, the water depths on the upstream and downstream sides of the structure will approximately match the existing conditions. This can be used to maintain flow velocities, thus minimizing disruption of flow in the channel and keeping sediment moving through the flume (see Figure 2.8).

5.4.3 Accuracy considerations

The accuracy with which flows can be measured with long-throated flumes and broad-crested weirs is related to the accuracy of the theoretical head-discharge relationship and the accuracy of the upstream head reading. Within the range of H_1/L values recommended, the accuracy of the head-discharge relationship does not

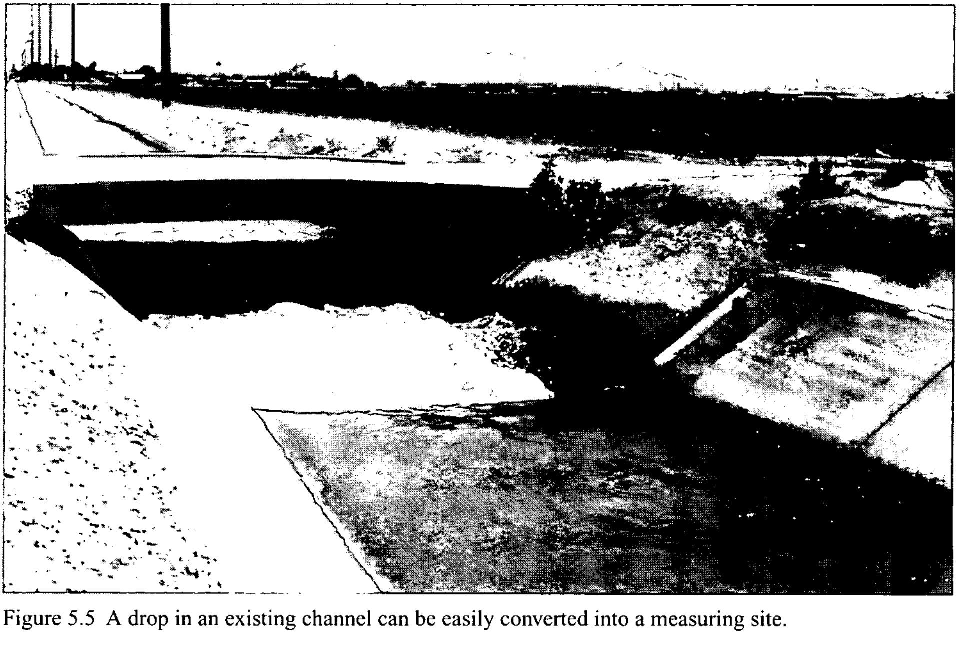


Figure 5.5 A drop in an existing channel can be easily converted into a measuring site.

change and cannot be changed by the flume designer. Those factors affecting accuracy that are under the designer's control are

- The method of upstream water level measurement, which influences the accuracy of head measurement;
- The amount of upstream sill-referenced head for a given discharge, which affects the relative errors in head measurement; and
- The value of u, the exponent in the head-discharge relation, which affects the sensitivity of the computed discharge to errors in head measurement.

The first factor is under the most direct control of the designer, while the other two factors are affected indirectly by the choice of throat section shape and dimensions. If better accuracy is desired, it can be improved by selecting a more accurate head detection method or by altering the flume shape to make the upstream sill-referenced head large, and the exponent, u, small. This is usually accomplished by making the contraction narrower and thus the sill-referenced head larger. For field use, a common target is 5% accuracy at Q_{max} and 10% accuracy at Q_{min}. In some settings, greater accuracy is desired. Because of scale effects, high accuracy is more difficult to obtain for small flumes. For large flumes, the heads are generally large enough that any head-detection method provides sufficient accuracy. Accuracy is typically a secondary criteria, when compared to submergence, freeboard and Froude number, since it can be improved by selecting a more accurate head-detection method, independent of any other change to the design of the structure.

For most channel shapes, the top width is greater at larger depths. Thus, improved accuracy is obtained by lowering the sill height and reducing the base width of the section. While this produces a lower upstream water depth, it produces a higher sill-referenced head, which directly influences the accuracy of discharge prediction. This is especially true at Q_{min}, where it is often the most difficult to achieve the desired accuracy.

5.5 Standard Flumes for Common Canal Sizes

5.5.1 Trapezoidal-throated structures

Lined trapezoidal irrigation canals are used worldwide for water conveyance and irrigation. Because of the large potential for retrofitting these canals with broad-crested weirs for flow measurement, a number of standard weir designs have been developed to simplify application. Figure 5.6 shows the general shape of these weirs. Standard weir sizes were selected and pre-computed by the methods of Chapter 6 for use in selected slip-formed canals in convenient metric and English unit dimensions. In selecting standard-sized canals and the related flow rates, consideration was given to proposals by the International Commission on Irrigation and Drainage (ICID, 1979), to the construction practices of the U.S. Bureau of Reclamation, and to design criteria for small canals used by the U.S. Natural Resources Conservation Service (formerly Soil Conservation Service).

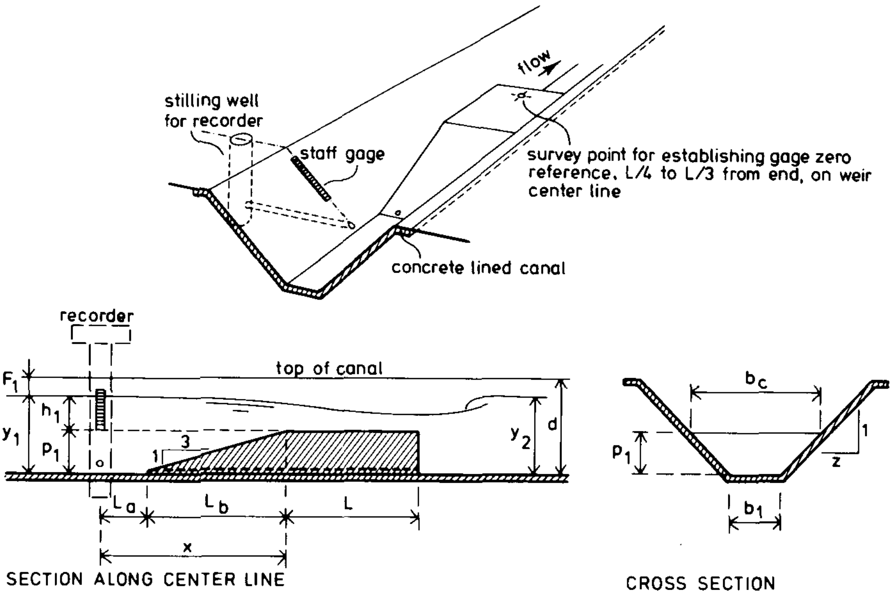


Figure 5.6 Broad-crested weir in concrete lined channel.

Present practice tends toward the use of side slopes of 1:1 for small, monolithic, concrete-lined canals with bottom widths less than about 0.8 m and depths less than about 1 m. Deeper and wider canals tend toward side slopes of 1.5:1 (horizontal : vertical). When the widths and depths are greater than about 3 m, the trend is more toward 2:1 side slopes, particularly if canal-operating procedures may allow rapid dewatering of the canal, which, in some soil conditions, can cause unbalanced hydrostatic pressures on the underside of the canal walls that lead to wall failure.

Most of the lined canals used in tertiary irrigation units or on large farms are of the smaller size, having 0.3- to 0.6-m widths, 1:1 side slopes, and capacities below 1 m^3/s (35 ft^3/s). The larger canal sizes are sometimes constructed with 1.25:1 side slopes, 0.61-m bottom widths (2 ft), and up to 1.22-m depths (4 ft). We attempt to accommodate subdivisions in metric units that can be anticipated for usual selections in metric dimensions and simultaneously cover tendencies to match equipment designed to English unit dimensions. In Table 5.2, precomputed broad-crested weir selections are given for canals with bottom widths at quarter-meter increments, with special insertions for 0.3 m (approximately 1 ft) and 0.6 m (approximately 2 ft). It is hoped that the offering of so many precomputed sizes will aid in retrofitting older canal systems and yet not prevent the adoption of the standard-sized canals proposed by ICID (1979). The approximate relationship between canal size and capacity is shown in Figure 5.7.

Table 5.3 provides similar information for canals sized in English units. Precomputed broad-crested weir selections are given for extra canal sizes with 1.25:1 side slopes that have no counterpart in the metric Table 5.2. Of course, sizes from either unit system may readily be converted to the alternate units if desired.

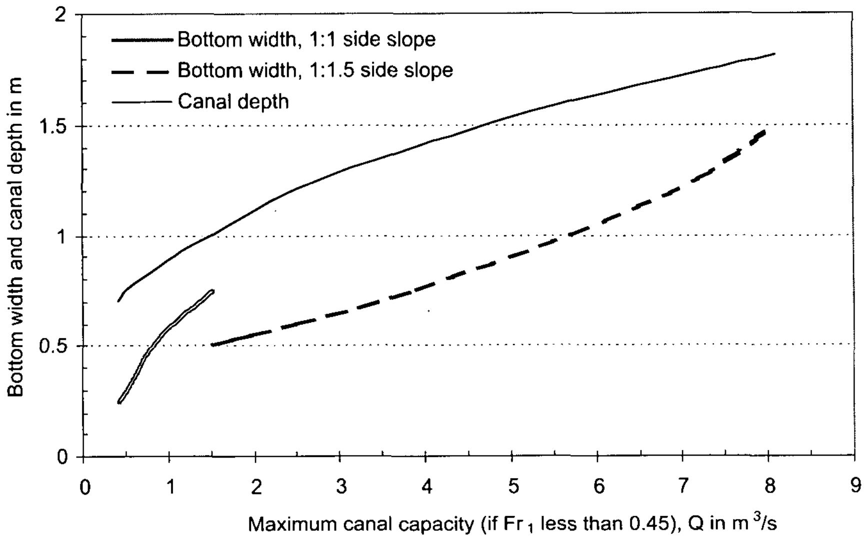


Figure 5.7 Relationship between bottom width and constructed depth of concrete lined trapezoidal irrigation canals. If Fr_1 is less than 0.45, the canal capacity is less than the shown Q-value.

For both unit systems, canal sizes with bottom widths in excess of 1.5 m or 5 ft, respectively, are avoided in the precomputed tables on the assumption that these sizes deserve special design consideration and would be best served by applying the methods of Chapters 6 and using the computer program described in Chapter 8.

Tables 5.2 and 5.3 show a number of precomputed weirs that may be used for the various combinations of bottom widths and side slopes given in the first two columns. The third column gives recommended values of maximum canal depth for each side-slope and bottom-width combination. For each canal size, a number of standard weirs can be used. The limits on canal capacity are given for each canal-weir combination. These limits on canal capacity originate from three sources:

1. The Froude number in the approach channel, Fr_1, is limited to 0.45 to assure water surface stability, where (Equation 2.18)

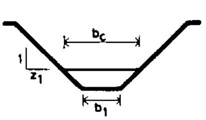
$$Fr_1 = \frac{v_1}{\sqrt{gA_1/B_1}}$$

2. The canal freeboard upstream from the weir, F_1, should be greater than 20% of the upstream sill-referenced head, h_1. In terms of canal depth, this limit becomes $d \geq 1.2\, h_1 + p_1$.
3. The sensitivity of the weir at maximum flow should be such that a 0.01 m change in the value of the sill-referenced head, h_1, causes less than 10% change in discharge. (See Equation 2.15).

Table 5.2 Choices of weir sizes and rating tables for lined canals in metric units.[a]

Canal Shape		*Maximum*	*Range of Canal Capacities*			*Weir Shape*		*Minimum Head*
Side Slopes	*Bottom Width*	*Canal Depth*[b]	*Lower*[c]	*Upper*	*Weir Selections*	*Crest Width*	*Sill Height*	*Loss*
z_1	b_1	d	Q_{min}	Q_{max}	*(See Table R.1)*	b_c	p_1	ΔH
	(m)	*(m)*	*(m³/s)*	*(m³/s)*		*(m)*	*(m)*	*(m)*
1.0	0.25	0.70	0.08	0.14[d]	A_m	0.50	0.125	0.015
			0.09	0.24[d]	B_m	0.60	0.175	0.018
			0.10	0.38[d]	C_m	0.70	0.225	0.022
			0.11	0.43[d]	D_{m1}	0.80	0.275	0.026
			0.12	0.37	E_{m1}	0.90	0.325	0.030
			0.13	0.32	F_{m1}	1.00	0.375	0.033
1.0	0.30	0.75	0.09	0.21[d]	B_m	0.60	0.150	0.017
			0.10	0.34[d]	C_m	0.70	0.200	0.021
			0.11	0.52	D_{m2}	0.80	0.250	0.025
			0.12	0.52	E_{m1}	0.90	0.300	0.029
			0.13	0.44	F_{m1}	1.00	0.350	0.033
			0.16	0.31	G_{m1}	1.20	0.450	0.039
1.0	0.50	0.8	0.11	0.33[d]	D_{m2}	0.80	0.150	0.019
			0.12	0.52[d]	E_{m2} or E_{m1}	0.90	0.200	0.024
			0.12	0.68[d]	F_{m1} or F_{m2}	1.00	0.250	0.029
			0.16	0.64	G_{m1}	1.20	0.350	0.037
			0.18	0.46	H_m	1.40	0.450	0.043
			0.20	0.29	I_m	1.60	0.550	0.048
1.0	0.60	0.9	0.12	0.39[d]	E_{m2}	0.90	0.150	0.021
			0.13	0.62[d]	F_{m2}	1.00	0.200	0.025
			0.16	1.09	G_{m1}	1.20	0.300	0.035
			0.18	0.86	H_m	1.40	0.400	0.043
			0.20	0.64	I_m	1.60	0.500	0.050
			0.22	0.43	J_m	1.80	0.600	0.049
1.0	0.75	1.0	0.16	0.91[d]	G_{m2}	1.20	0.225	0.030
			0.18	1.51	H_m	1.40	0.325	0.038
			0.20	1.22	I_m	1.60	0.425	0.047
			0.22	0.94	J_m	1.80	0.525	0.053
1.5	0.60	1.2	0.20	1.3[d]	K_m	1.50	0.300	0.031
			0.24	2.1[d]	L_m	1.75	0.383	0.038
			0.27	2.5	M_m	2.00	0.467	0.044
			0.29	2.2	N_m	2.25	0.550	0.050
			0.32	1.8	P_m	2.50	0.633	0.056
			0.35	1.4	Q_m	2.75	0.717	0.059
1.5	0.75	1.4	0.24	1.8[d]	L_m	1.75	0.333	0.036
			0.27	2.8[d]	M_m	2.00	0.417	0.042
			0.29	3.9[d]	N_m	2.25	0.500	0.049
			0.32	3.5	P_m	2.50	0.583	0.055
			0.35	3.1	Q_m	2.75	0.667	0.062
			0.38	2.6	R_m	3.00	0.750	0.066
1.5	1.00	1.6	0.29	3.4[d]	N_m	2.25	0.417	0.046
			0.32	4.7	P_m	2.50	0.500	0.052
			0.35	5.7	Q_m	2.75	0.583	0.059
			0.38	5.1	R_m	3.00	0.667	0.065
			0.43	3.9	S_m	3.50	0.833	0.081
1.5	1.25	1.7	0.32	4.1[d]	P_m	2.50	0.417	0.048
			0.35	5.6[d]	Q_m	2.75	0.500	0.055
			0.38	7.2	R_m	3.00	0.583	0.061
			0.43	5.9	S_m	3.50	0.750	0.074
			0.49	4.5	T_m	4.00	0.917	0.084
			0.55	3.3	U_m	4.50	1.083	0.089
1.5	1.50	1.8	0.35	4.8[d]	Q_m	2.75	0.417	0.051
			0.38	6.5	R_m	3.00	0.500	0.058
			0.43	8.1	S_m	3.50	0.667	0.071
			0.49	6.6	T_m	4.00	0.833	0.083
			0.55	5.1	U_m	4.50	1.000	0.092

[a] $L_a \geq H_{1max}$; $L_b = 3\,p_1$, $X = L_a + L_b > 2$ to $3\,H_{1max}$
$L > 1.5\,H_{1max}$, but within range given in Table R.1
$d > 1.2\,h_{1max} + p_1$
$\Delta H > 0.1\,H_1$
[b] Maximum recommended canal depth.
[c] Limited by sensitivity.
[d] Limited by Froude number; otherwise limited by canal depth.



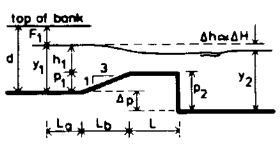


Table 5.3 Choices of weir sizes and rating tables for lined canals in English units.

Canal Shape		*Maximum*	*Range of Canal Capacities*		*Weir*	*Weir Shape*		*Minimum*
Side Slopes z_1	*Bottom Width* b_1 *(ft)*	*Canal Depth*[b] d *(ft)*	*Lower*[c] Q_{min} *(ft³/s)*	*Upper* Q_{max} *(ft³/s)*	*Selections (see Table R.2)*	*Crest Width* b_c *(ft)*	*Sill Height* p_1 *(ft)*	*Head Loss* ΔH *(ft)*
1.0	1.0	2.5	1.9	8[d]	A_e	2.0	0.50	0.06
			4.2	16[d]	B_e	2.5	0.75	0.08
			4.8	19	C_e	3.0	1.00	0.10
			5.6	15	D_e	3.5	1.25	0.12
			6.2	11	E_e	4.0	1.50	0.13
1.0	2.0	3.0	5.6	27[d]	D_e	3.5	0.75	0.10
			6.2	40	E_e	4.0	1.00	0.12
			6.8	33	F_e	4.5	1.25	0.14
			7.4	27	G_e	5.0	1.50	0.15
			8.2	22	H_e	5.5	1.75	0.16
1.25	1.0	3.0	5.0	19[d]	I_e	3	0.80	0.08
			6.4	35	J_e	4	1.20	0.11
			7.6	26	K_e	5	1.60	0.14
1.25	2.0	4.0	6.4	31[d]	J_e	4	0.80	0.10
			7.6	64[d]	K_e	5	1.20	0.13
			8.9	78	L_e	6	1.60	0.16
			10.1	62	M_e	7	2.00	0.18
			11.4	46	N_e	8	2.40	0.20
1.5	2.0	4.0	8	49[d]	P_e	5	1.00	0.11
			9	82[d]	Q_e	6	1.33	0.13
			11	86	R_e	7	1.67	0.16
			12	72	S_e	8	2.00	0.18
			13	60	T_e	9	2.33	0.20
1.5	3.0	5.0	9	66[d]	Q_e	6	1.00	0.12
			11	108[d]	R_e	7	1.33	0.14
			12	140[d]	S_e	8	1.67	0.17
			13	160	T_e	9	2.00	0.20
			14	140	U_e	10	2.33	0.22
			17	95	V_e	12	3.00	0.25
1.5	4.0	5.5	12	135[d]	S_e	8	1.33	0.15
			13	200[d]	T_e	9	1.67	0.18
			14	235	U_e	10	2.00	0.21
			17	175	V_e	12	2.67	0.26
			19	125	W_e	14	3.33	0.28
1.5	5.0	6.0	14	235[d]	U_e	10	1.67	0.20
			17	285	V_e	12	2.33	0.25
			19	220	W_e	14	3.00	0.29
			22	160	X_e	16	3.67	0.32

[a] $L_a \geq \Delta H_{1max}$; $L_b = 3\,p_1$; $x = L_a + L_b > 2$ to $3H_{1max}$

$L >$ 1.5 H_{1max}, but within range given in Table R.2

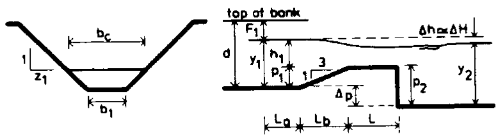
$d >$ 1.2 $h_{1max} + p_1$

$\Delta H > 0.1\,H_1$.

[b] Maximum recommended canal depth.

[c] Limited by sensitivity.

[d] Limited by Froude number; otherwise limited by canal depth.



Although Tables 5.2 and 5.3 are primarily intended for the selection of these standard weirs, they are also useful for the selection of canal sizes. The Froude number in the canal is automatically limited to 0.45, and selecting the smallest canal for a given capacity will give a reasonably efficient section. For instance, if the design capacity of the canal is to be 1.0 m³/s, the smallest canal that can incorporate a measuring structure has b_1 = 0.60 m, z_1 = 1.0, and d = 0.90 m. Larger canals can also be used. The hydraulic grade line of the channel should be checked to assure an adequate design.

Figure 5.8 demonstrates how Tables 5.2 and 5.3 can be used to select a number of weirs that may be appropriate for a given canal size and discharge. For a canal with b_1 = 0.30 m, d = 0.75 m, and z_1 = 1.0, six weirs are available for use (B_m, C_m, D_{m1}, E_{m1}, F_{m1}, G_{m1}). For the flow rate, Q = 0.36 m³/s, only weirs D_{m1}, E_{m1}, and F_{m1} can be used since their maximum capacities are above 0.36 m³/s. Further selection among these three depends on the hydraulic design, which is discussed in Section 5.6.2. If the maximum design flow rate to be measured at the site is less than the lower canal capacity limit shown in the table, then the sensitivity of the measuring structure will not be adequate, and the rectangular flumes of Section 5.5.2 should be considered.

Each standard weir can be used for several different bottom widths. This is possible because the change in flow area upstream from the weir causes only a small change in velocity of approach and thus energy head. We have limited the error in discharge caused by the change in flow area to about 1%. This is a systematic error for any particular approach area, and the value of this error varies with discharge. If a weir can be used for several bottom-width canals, it can also be used for any intermediate

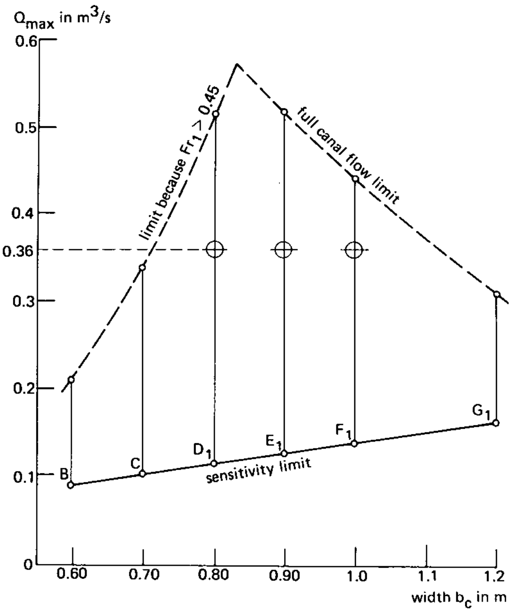


Figure 5.8 Limits for Q_{max} for a canal with b_1 = 0.30 m, d = 0.75 m, and z_1 = 1.0.

width. For example, in Table 5.2, weir G_{m1} can be used in canals with bottom widths of 0.30, 0.50, and 0.60 m, or any width in between, say b_1 = 0.40 m. However, the user is then responsible for determining the sill height, head loss, and upper limit on design discharge.

The rating tables for each of the standard weirs are given in Tables R.1 and R.2 (Appendix 4) and were computed using the following criteria:

- Each weir has a constant bottom width b_c and a sill height p_1 that varies with the canal dimensions.
- The ramp length can be set at 2.5 to 4.5 times the sill height. The 3:1 ramp slope is preferred.
- The gage is located a distance at least H_{1max} upstream from the start of the ramp. In addition, it should be located a distance of roughly two to three times H_{1max} from the entrance to the throat.
- The throat length should be 1.5 times the maximum expected sill-reference head, h_{1max}, but should be within the limits indicated in Tables R.1 and R.2.
- The canal depth must be greater than the sum of $p_1 + h_{1max} + F_1$, where F_1 is the freeboard requirement, as discussed in Section 5.6.2.

Tables R.1 and R.2 contain the specific calibrations for the canal-weir combinations specified in Tables 5.2 or 5.3. These rating tables are prepared with discharge rate expressed as the independent variable in the first column and the upstream head h_1 as the dependent variable in the second column. This is the reverse of the usual style for tables for flumes and weirs. This method allows simple calculations of values for marking sidewall gages in direct discharge units for the canals by multiplying the listed h_1 depth for each selected unit of discharge by a function of the side-slope ratio. Figure 4.2 lists these multiplier values for the h_1 value (see Section 4.3).

Also indicated in the last column of Tables 5.2 and 5.3 is a minimum head loss, ΔH, that the weir must provide (Figure 5.9). Excessive downstream water levels may prevent this minimum head loss, which means that the weir exceeds its modular limit and no longer functions as an accurate measuring device. The required head losses for the various broad-crested weirs were evaluated by the method described in Section 6.6 and, for design purposes, are listed for each weir size with the restriction that the computed modular limit shall not exceed 0.90. Thus, the design head loss is either $0.1H_1$ or the listed value for ΔH, whichever is greater. For these calculations, it was assumed that the weir was placed in a continuous channel with a constant cross section (e.g., $p_1 = p_2$, $b_1 = b_2$, and $z_1 = z_2$) and that the diverging transition was omitted (rapid expansion, m = 0). For other conditions, refer to Sections 6.6. Technically, the modular limit is based on the drop in total energy head through the weir (i.e., including velocity head), but the velocity head component is usually of the same order of magnitude upstream and downstream from the structure when $p_1 \approx p_2$ so that Δh may be satisfactorily substituted for ΔH.

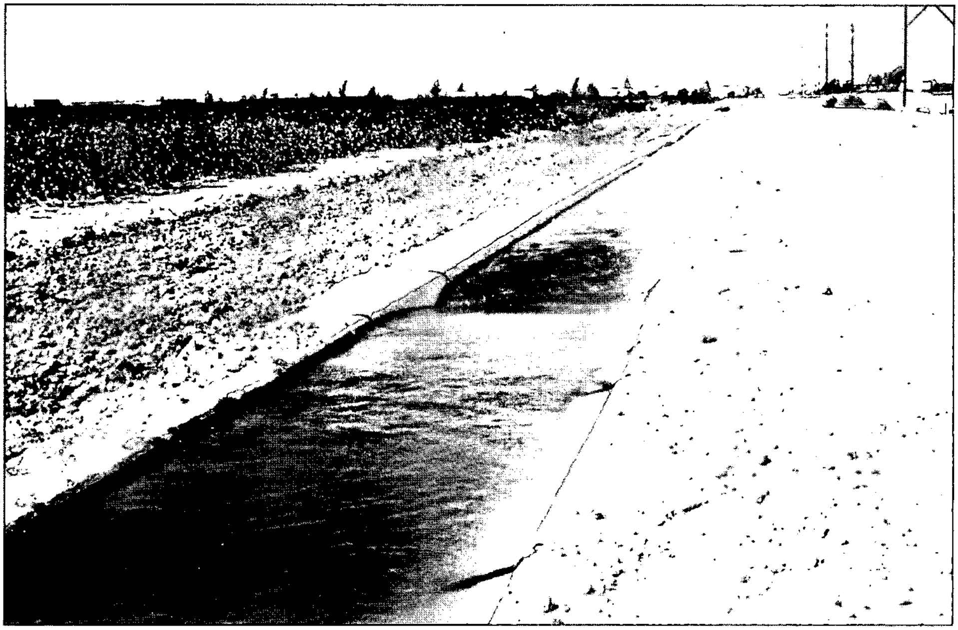


Figure 5.9 Broad-crested weir requires little head loss for satisfactory operation (Arizona).

5.5.2 Rectangular-throated structures

For a variety of channels, it is convenient to construct the flume or weir control section with a rectangular shape (see examples in Section 3.3). Most such structures have a bottom contraction, although many are contracted from both the sides and the bottom. For a given upstream head, a rectangular control section with a gradual converging transition discharges nearly equal quantities of water over equal widths. The major differences are associated with the friction along the sidewalls. Thus, the flow is nearly one-dimensional over the weir so that rating tables can be made that give the unit flow rate, q, in cubic meters per second per meter width of sill for each value of h_1. This allows an almost unlimited variety of sizes for rectangular broad-crested weirs, since for each width b_c of the weir, an accurate rating table can be made by multiplying the unit discharges shown in the tables by b_c. Hence the discharge is given by

$$Q = b_c q \tag{5.6}$$

Tables R.3 and R.4 (Appendix 4) give a series of rating tables for rectangular weirs. The groupings of weir width were selected to keep the error due to the effects of the sidewalls to less than 1%. Ratings are given for a number of sill heights to aid in design. Discharges in these tables are limited to keep the approach channel Froude number below 0.45. Interpolation between sill heights will give reasonable results.

If the approach area is larger than that used to develop these rating tables, either because of a higher sill or a wider approach, the ratings must be adjusted for the approach velocity coefficient, C_v, by the method in Section 6.4.5. (This applies to both lined and unlined channels). To simplify this process, the discharge over the weir for a C_v value of 1.0 is given in the far right column of each grouping. This discharge column is labeled as $p_1 = \infty$, since for $C_v = 1.0$ the velocity of approach is zero, as would be the case if the weir were the outlet of a deep reservoir or lake. Under this circumstance, the weir has the lowest discharge for a given upstream head. Note that at the very low heads, the discharge for the weirs with rectangular approach channels approaches that for $p_1 = \infty$ because the approach velocities are small.

The ratings given in Tables R.3 and R.4 are for the throat lengths, L, given at the head of each group of columns. When the maximum design discharge of a structure is much less than the maximum discharge shown in the rating table, the aforementioned throat length may be longer than necessary. A value of $L = 1.5H_{1max}$ is a reasonable compromise between providing a long enough throat to avoid the effects of streamline curvature and minimizing the size of the structure. The throat length may be reduced to this value provided that it does not become shorter than about two-thirds of the L value in the table heading. Such a length reduction causes the weir discharge to increase by less than 1%. The length of the converging transition, L_b, should be roughly 3 times p_1. The distance between the gaging station and the start of the throat ($L_a + L_b$) should be 2 to 3 times H_{1max}, and the distance between the gaging station and the start of the converging transition, L_a, should be greater than H_{1max} (see Figure 5.10).

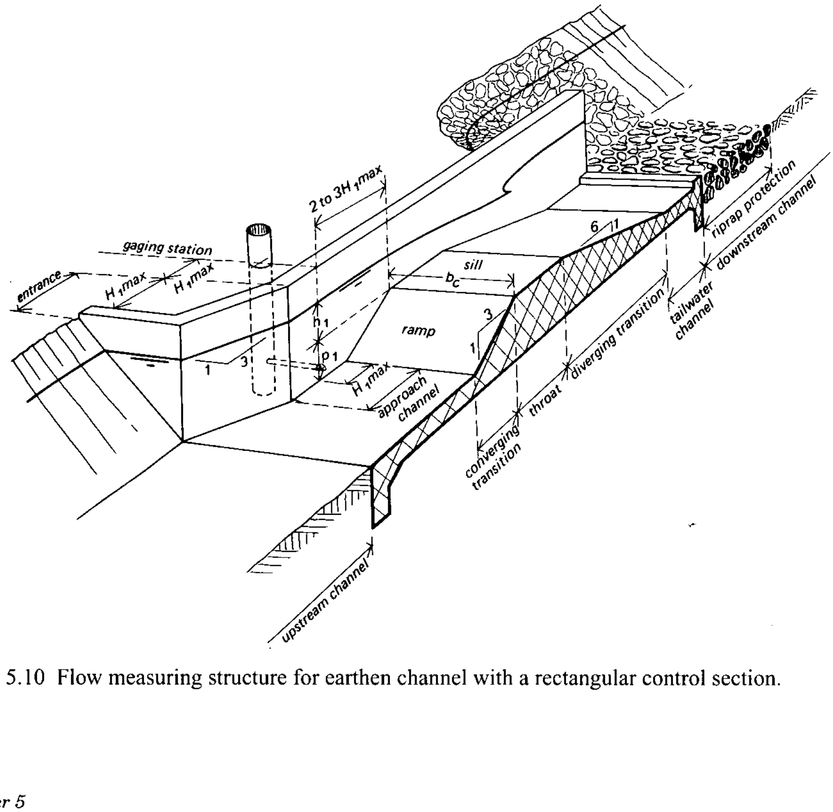


Figure 5.10 Flow measuring structure for earthen channel with a rectangular control section.

As discussed in Chapter 6, there must be a loss of energy head across the flume in order for critical flow to occur in the throat. This is required to maintain a unique head-discharge relation for the flume, unaffected by the downstream tailwater level. When this occurs, the flow is said to be modular. The value of the head loss required to maintain modular flow depends on a number of factors, including the flow velocity in the tailwater channel and the downstream expansion ratio. The head-loss values given at the bottom of the columns in Tables R.3 and R.4 are for a rectangular tailwater channel with the same width as the throat, but with a rapid expansion (i.e., no diverging transition). For a diverging transition with a 6:1 (horizontal:vertical) expansion, use one half of the head-loss values for the rapid expansion. In both cases, the length of the structure should be roughly the same (i.e., when the diverging transition is omitted, the structure is not shortened, but that length is added to the tailwater channel). To reduce the risk of non-modular flow, we do not recommend the use of head-loss values less than $0.1H_1$. Thus, use either the head-loss values shown at the bottom of the tables or $0.1H_1$, whichever is greater.

Rectangular weir in earthen channel

Generally, earthen channels are much shallower and wider than the concrete-lined canals of Section 5.5.1. For a given size of irrigation canal, the discharge is usually less for the earthen canal because the permissible flow velocities are less. Side slopes are normally in the ratio 2:1 (horizontal to vertical) or flatter, for canals deeper than about 1 m. To save on the cost of land expropriation and excavation, however, earthen irrigation canals now tend to be much narrower. Figure 5.11 is based on data of the U.S. Bureau of Reclamation (USBR 1967). It shows minimum recommended values for the bottom width and bottom width versus depth ratio as a function of maximum design discharge. Flumes and weirs placed in such channels tend to be much narrower than the earthen section.

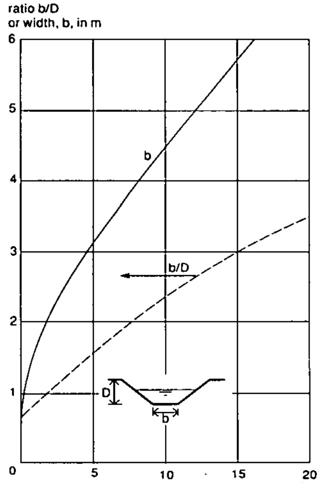


Figure 5.11 Minimum recommended values for earthen canal dimensions

The rating tables for the rectangular weirs, Tables R.3 and R.4, assume that the approach section is rectangular and of the same width as the throat. The application of these tables to a structure for which the upstream sill-referenced head is not measured in the assumed approach canal, but in the wider earthen upstream channel, causes an error in Q because of a large change in the approach velocity and the related approach velocity coefficient, C_v. Consequently, the Q values for a weir without the rectangular approach channel of Figure 5.10, must be corrected using the method in Section 6.4.5.

A truncated rectangular structure in an earthen canal often has no diverging transition and a tailwater channel that is wider than the flume throat, as shown in Figure 3.25. Because of the large number of possible sizes of tailwater channels, no precomputed values are given. The worst case is for a rapid expansion into a pool or reservoir with zero velocity (or $p_2 = \infty$). The head loss for this situation is $0.4H_1$ (see Table 6.7). This represents a much larger (estimated) required head loss than for a rectangular tailwater channel as shown in Figure 5.10. This amount of head loss is often not available. An alternative is to compute the head loss for the actual tailwater channel using the theory in Sections 6.6 or with the software of Chapter 8.

5.5.3 Triangular-throated structures

For monitoring return flows and operational spillages from irrigation systems or for measuring flows in natural streams, a structure is needed that can measure a wide range of flows. As shown in Section 2.4, a structure with a triangular control section is very suitable for this purpose because (Equation 2.3)

$$\gamma = \frac{Q_{max}}{Q_{min}} = 335$$

for this group of structures (see Table 2.2).

Table R.5 (Appendix 4) lists the calibration for several sizes of these triangular throated structures. The head-loss values given in Table R.5 are for a rapid expansion into a tailwater channel of the same size as the approach channel. A gradual transition into a wide channel is more common, as shown in Figures 3.36 and 3.56. For a triangular control section, the differences are not too significant, because the head loss shown in Table R.5 will be about $0.1H_1$, while the theoretical head loss without velocity head recovery is $0.24H_1$ (see Table 6.7).

5.5.4 Weirs in culverts

For many unlined channels, culverts at road crossings and other locations provide an excellent site for a measuring structure. A broad-crested weir can typically be placed in such a structure at a fraction of the cost of a similar structure in the unlined channel. However, care should be taken in designing the structure so that the capacity of the

Table 5.4 Properties of weirs made from placing sill in circular conduit.

Pipe Diameter D (m)	Sill Height p_1 (m)	Approach Length L_a (m)	Ramp Length L_b (m)	Throat Length L (m)	Head Range h_1 (m)	Discharge Range Q (m^3/s)	Crest Width b_c (m)	Maximum Head Loss Δh_L (m)
	0.20	0.50	0.60	0.70	0.08-0.44	0.03 — 0.56	0.800	0.030
	0.25	0.65	0.75	1.00	0.07-0.60	0.03 — 0.93	0.866	0.047
	0.30	0.60	0.90	0.90	0.07-0.55	0.03 — 0.79	0.917	0.053
1.0	0.35	0.55	1.05	0.80	0.07-0.50	0.02 — 0.66	0.954	0.059
	0.40	0.50	1.20	0.70	0.06-0.45	0.02 — 0.56	0.980	0.065
	0.45	0.45	1.35	0.60	0.06-0.40	0.02 — 0.45	0.995	0.067
	0.50	0.40	1.50	0.52	0.05-0.35	0.02 — 0.35	1.000	0.069
D (ft)	p_1 (ft)	L_a (ft)	L_b (ft)	L (ft)	h_1 (ft)	Q (ft^3/s)	b_c (ft)	Δh_L (ft)
	0.20	0.50	0.60	0.70	0.08-0.43	0.06 — 0.98	0.800	0.030
	0.25	0.65	0.75	1.00	0.07-0.60	0.05 — 1.69	0.866	0.045
	0.30	0.60	0.90	0.90	0.07-0.55	0.05 — 1.44	0.917	0.052
1.0	0.35	0.55	1.05	0.80	0.07-0.50	0.05 — 1.21	0.954	0.058
	0.40	0.50	1.20	0.70	0.06-0.45	0.04 — 1.00	0.980	0.063
	0.45	0.45	1.35	0.60	0.06-0.40	0.04 — 0.81	0.995	0.067
	0.50	0.40	1.50	0.52	0.05-0.35	0.03 — 0.64	1.000	0.069

culvert is not reduced below acceptable limits by addition of the weir. For a rectangular box culvert, the weirs of Section 5.5.2 can be used. For circular culverts, a weir sill can still be placed in the culvert, as discussed in Section 3.2.1. To keep the water surface free, we recommend that the maximum upstream water level be limited to 90% of the pipe diameter. If the upstream water surface reaches the top of the pipe, the weir will no longer serve as a viable critical-depth measuring structure.

Design of a weir sill in a circular culvert amounts to selecting the sill height as a fraction of the diameter. Table 5.4 gives the dimensions and capacities for weirs in circular culverts with a 1.0-m diameter and a 1.0-ft diameter. Several sill heights are listed, expressed as a fraction of the pipe diameter. Tables R.6 and R.7 (Appendix 4) provide rating tables for these weirs. Approximate rating tables for other pipe diameters can be obtained by adjusting the head and discharge values using hydraulic similitude (Section 6.4.6). The weir dimensions and rating table values (head and discharge) are scaled based on the ratio of the actual diameter and the reference diameter of the pipes included in the tables. Alternately, a new rating can be computed with the software discussed in Chapter 8.

To demonstrate the hydraulic similitude adjustment procedure, consider the adjustments needed to develop dimensions and rating tables for a weir constructed in a 1.25-m diameter pipe:

- Compute the length ratio by dividing the actual diameter by the reference diameter, in this case the length ratio is (1.25 m / 1.0 m) = 1.25.
- Multiply each length in Table 5.4 by the length ratio. For example, a 0.2 m sill becomes 0.25 m.

- Multiply the ranges of discharges by the length ratio raised to the 5/2 power, or $1.25^{5/2} = 1.58$.
- Multiply each head in the rating table by the length ratio, being 1.25 for this example.
- Multiply each discharge in the rating table by the length ratio raised to the 5/2 power, or 1.58 as computed above.

For ratios of sill height to pipe diameter that are not specifically listed in the tables, approximate ratings can be obtained by interpolating between adjacent rating tables.

5.5.5 Movable weirs

Design of movable weirs amounts to determining the width of the weir, since the control section is always rectangular, and the weir height is adjustable. A given flow can be regulated by a small-width weir discharging under a high head or by a wide but shallow-flowing weir. The optimum combination of head and width for movable weirs depends on a variety of factors, such as

- Total weir discharge;
- Width of the incoming main, or lateral canal in case of an in-line structure, and the width of the canal downstream from the weir functioning as an offtake;
- Water depth in the incoming main or lateral canal;
- Available head loss over the weir;
- Construction limitations, such as groove arrangement, lifting gear, and gate weight;
- Whether the weir moves behind a bottom gate or a drop in the canal bottom;
- The desired accuracy of the flow measurement; and
- The number of structures needed in the irrigated area and the desire to standardize sizes.

These factors, in practice, limit the maximum value of H_1 for movable weirs to 1.00 m. The width of the weir b_c varies from 0.30 m to as much as 4 m, the larger widths used in conjunction with high H_{1max} values. In general, the design is accomplished by selecting a head, H_1, that will give a reasonable accuracy and then selecting an appropriate width for the design discharge. A set of rating tables was developed for these movable weirs, Tables R.8 and R.9 (Appendix 4). These rating tables are valid for the maximum energy head H_{1max} being equal to the crest length. The designer then need only select the appropriate table to give a reasonable width (see Section 5.5.2). The reader should note that all dimensions of both the movable weir and its abutments are related to the maximum value selected for the total energy head over the weir crest (H_{1max}).

For a movable weir that supplies water to one large farm or to a group of small farms, an upper limit on H_1 of about 0.50 m is commonly used. Minimizing the length of the movable crest produces a crest length of L = 0.50 m and an upstream rounded nose of $0.2H_{1max}$ = 0.10 m. Other dimensions of this size offtake are shown in

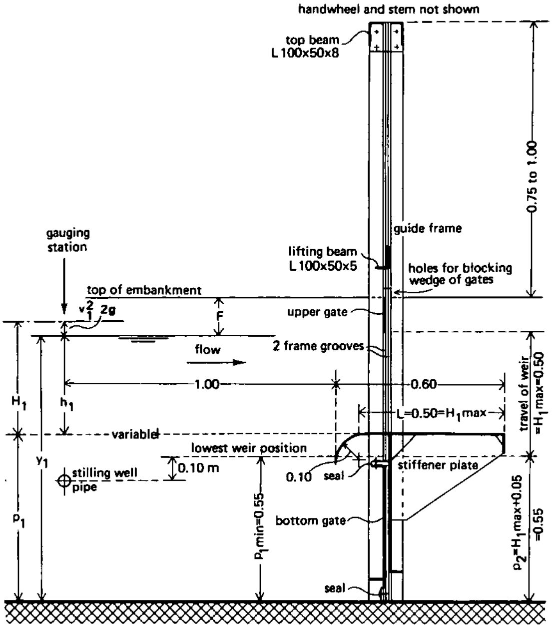


Figure 5.12. Dimensions of a commonly used offtake structure with movable weir.

Figure 5.12. With a width of $b_c = 0.30$ m (1 ft) as a minimum for practical use, this weir can measure and regulate flows between 5 and 170 liters/s (0.2 and 6 ft^3/s). It is often practical to use a width less than $b_c = 1.50$ m (5 ft) since a central lifting gear can then be used to move the weir while the groove arrangement can be a relatively simple one (see Section 3.5.2). If the width exceeds 1.50 m, lifting gears should be used on both sides of the weir to avoid binding the gate in the grooves.

Depending on the type and the operational procedures of structures in the continuing main canal or lateral irrigation canal, the water level upstream from a movable offtake will remain nearly constant or will fluctuate with the flow rate in the continuing canal. The water depth, y_1, in the approach canal thus can differ while the diverted flow, and hence h_1 is held constant. The resulting variation in the approach velocity, v_1, is greater if the upstream head, h_1, is large with respect to the water depth y_1, as is the case with the bottom-drop-type weir.

Occurrence of different values of v_1 (and thus of $v_1^2/2g$) with the same h_1 value causes a slightly larger error in the discharge measurement (see Section 6.4.3). If the water level in the continuing supply canal fluctuates by more than about $0.15H_{1max}$, use the bottom-gate-type weir or the alternate bottom for the bottom-drop-type weir. In either case, the rating table for the bottom-gate-type weir should be used (see Figure 3.61). Many irrigation canal systems, however, are operated so that the water level in the main and lateral canals is checked between narrow limits by movable control structures. Under such operational conditions, the water depth, y_1, in the approach canal remains rather constant. Using the software described in Chapter 8, we have assumed this operation in producing the rating tables (Tables R.8 and R.9) and used,

for the bottom-gate type (see Figure 3.50),

$y_1 = 2\ H_{1max} + 0.05$ m, and

for the bottom-drop type (see Figure 3.51),

$y_1 = 1.33\ H_{1max}$

If the y_1 value deviates from the preceding values, a suitable rating table may be derived by adjusting for the change in approach velocity by the method given in Section 6.4.5 or by interpolating between the listed heads. Six sets of rating tables are given: for $L = 0.50$, 0.75, and 1.00 m in the metric system; and $L = 1.0$, 2.0, and 3.0 ft with the related flow rate in ft^3/s. To minimize the size of the weir, the upper limit $H_1/L \leq 1.0$ was used.

The required head loss ΔH over the movable weir is given at the bottom of Tables R.8 and R.9. These values can be used if the weir has a short section of rectangular tailwater channel, as in the example design of Section 3.5.4. If a wider tailwater channel is used, the head loss should be calculated by use of Section 6.6 or taken as $0.4H_{1max}$ for the worst possible case (see also Table 6.7).

5.5.6 Portable structures

Frequently, little information exists on the flow conditions of an existing channel, at least insufficient information for flume design. By placing a temporary or portable structure into a channel, one can quickly learn the relative performance of that structure and obtain information on existing flow conditions for design. One approach to flume design is to place several different standard sizes into the canal and select the one that works best. For smaller channels, these portable structures can be placed or adjusted with flowing water. Portable structures are also useful when intermittent measurement needs do not justify the time or cost to construct a permanent structure, or when permission for such a structure is not obtained. For many of these portable structures, the rating tables for the permanent structures can be used.

RBC flumes

Designs for portable structures are generally based on selecting the size required for the discharge expected. Table 5.5 gives the properties of five RBC flumes for small earthen channels, including furrows. This table includes the approximate flow ranges that can be measured and the head-loss requirement. The rating tables for these structures are given in Tables R.10 and R.11 (Appendix 4). (See Section 3.3.3 and Figures 3.39 through 3.43 for details on these flumes).

Table 5.5 Properties of five RBC flumes.

Throat Width, b_c (exact size) (mm)	(inches)	*Flume Throat Length, L* (mm)	(inches)	*Approximate Discharge Range Q_{min} to Q_{max}* (liters/s)	(ft^3/s)	*Head Loss Required* (mm)
50	1.97	75	2.95	0.03 - 1.5	0.001 - 0.051	10
75	2.95	112.5	4.43	0.07 - 4.3	0.003 - 0.15	15
100	3.95	150	5.90	0.16 - 8.7	0.006 - 0.31	20
150	5.90	225	8.86	0.40 - 24.0	0.014 - 0.87	30
200	7.87	300	11.81	0.94 - 49.0	0.033 - 1.70	40

Rectangular flumes for lined and unlined canals

Portable rectangular flumes are available from several manufacturers, or can be custom made (see Figure 3.44 through 3.47). Design is primarily a matter of selecting the size that has sufficient capacity for the range of flows to be measured. Limited available head may suggest a wider flume, since a wide shallow flow requires less head loss. However, this potentially also decreases the accuracy, which is already worse for a portable device compared to a permanent one. Portability also suggests a relatively small size. Common field sizes for irrigation studies have 15-cm, 30-cm and 45-cm widths and 30-cm height. The standard rating tables for rectangular weirs can be used, Tables R.3 and R.4, or new tables computed from the computer software described in Chapter 8. The rating table for a commercially available 30.5 cm wide rectangular flume is given in Table R.12 (Appendix 4) (See Section 3.3.3 for details on these flumes).

Adjust-A-Flume

Movable throats make portable structures easier to use. As long as the flume can handle the maximum flow, no design is necessary since the throat can be adjusted to match the flow conditions. The Adjust-A-Flume (Figure 3.48 through 3.50) is a rectangular-throated flume with an adjustable sill height, available in a range of sizes; the properties of each are given in Table 5.6. Selection of size is based primarily on capacity. Rating tables for these flumes are given in Tables R.13 and R.14 (Appendix 4) (See Section 3.3.3 for details on these flumes).

Table 5.6 Properties of Adjust-A-Flumes.

Throat Width (mm)	(inches)	*Discharge Range Q_{min} to Q_{max}* (liters/s)	(ft^3/s)	*Weight* (kg)	*Length Without Flare* (m)	*Height (inside)* (m)
152	6	0.4 - 14	0.014 - 0.5	10	0.8	0.24
305	12	2.8 - 57	0.1 - 2.0	37	1.4	0.38
610	24	7.1 - 113	0.25 - 4.0	46	1.4	0.38
914	36	14.2 - 170	0.5 - 6.0	56	1.4	0.38
762	30	14.2 - 312	0.5 - 11.0	187	2.3	0.71
762	30	28.3 - 425	1.0 - 15.0	232	2.3	0.86
965	38	42.5 - 708	1.5 - 25.0	317	2.8	1.03
965	38	56.6 - 991	2.0 - 35.0	360	2.8	1.03

5.6 Flume Design Procedure

The intent of the design procedure is to determine the appropriate dimensions of a flow-measuring flume that will perform according to the criteria described in Section 5.2. That is, it will accurately measure discharge over the full range of flows to be measured. Design is often an iterative process and in many cases, there are a wide variety of flumes that will function adequately. This design procedure is aimed at providing a flume that is simple, easy to construct, and accurate.

The design process varies slightly with the channel conditions and with the source of rating tables and flume information (i.e., when design is being done with theory and equations, with rating tables for standard sizes, or with the computer program of Chapter 8, WinFlume. The basic steps in the process are given briefly below and discussed in more detail in subsequent sections. This is basically a trial and error process, although the methods for determining the amount of contraction required, described in Section 6.3.3, can speed up the process considerably.

5.6.1 Flume design steps (trial and error)

1. Obtain data on the channel and the flow condition within the channel, including the range of flows to be measured (Q_{min} and Q_{max}) and the associated tailwater levels (y_{2min} and y_{2max}). (Fill out the information in Figure 2.21. See, for example, Table 5.7).
2. Decide how much freeboard is required (F_1).
3. Decide on the allowable errors (X_{Qmin} and X_{Qmax}) in flow measurement at the minimum and maximum flow rates to be measured and determine the rating table errors (X_{Cmin} and X_{Cmax}).
4. Decide on the method of head detection, and its associated accuracy (δ_{h1}), and determine the head required to satisfy the accuracy criteria.
5. Decide on an initial shape for the control section and determine how that shape will be changed initially during the design process.
6. Select a trial contraction amount and initial flume longitudinal dimensions (if needed).
7. Determine the upstream head and the required head loss at Q_{min} and Q_{max} for this trial contraction (h_{1min} and h_{1max}, ΔH_{min} and ΔH_{max}).
8. Compare the results of this trial with the design criteria. If design criteria are not met, select a different contraction amount and repeat Steps 7 and 8 until design criteria and design aims are satisfied. (see Section 5.6.6 for recommendations).
9. Finalize flume or weir longitudinal dimensions according to criteria in Table 5.8, in Section 5.6.3.

When selecting from among standard flumes such as those in Table 5.2, the trial and error process is fast and straightforward. In fact, we recommend that the designer determine the full range of flume sizes that satisfy the design criterion. Then one can evaluate the tradeoffs between the various options. However, for general design, there may be too many options. An example is the selection of a rectangular flume

NAME OF SITE: Kodak, Colorado DATE: April 30 2001

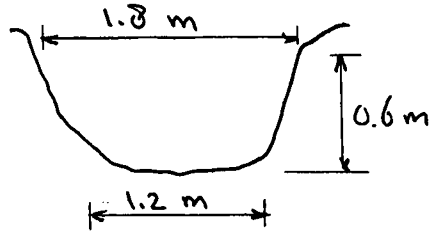
HYDRAULIC DEMANDS:

Range of flow to be measured, Q	Present water depth in channel, y_2	Minimum permitted error in measurement, X_Q
Q_{min} = 0.085 m^3/s Q_{max} = 0.340 m^3/s	y_{2min} = 0.25 m y_{2max} = 0.46 m	X_{Qmin} = 7.0 % X_{Qmax} = 5.0 %

HYDRAULIC DESCRIPTION:

Channel bottom with	b_1 = ± 1.2	m
Channel side slope	z = –	m
Channel depth	d = –	m
Maximum allowable water depth	y_{1max} = 0.60	m
Manning's n	n = 0.050	
Hydraulic gradient	S_f = 0.001	
Available drop in water surface at site	Δh = 0.15	m
Drop in channel bottom at site	Δp = 0.0	m

Sketch of channel cross section:

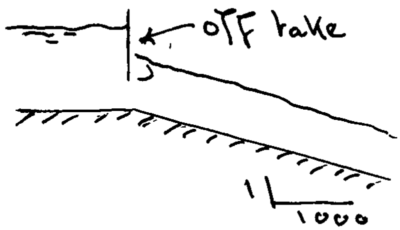


Concrete lined: ☐
Earthen channel: ☒

FUNCTION OF STRUCTURE

Measurement only	☒
Regulation and measurement of flow rate	☐

Profile along bottom of channel over length of $100b_1$



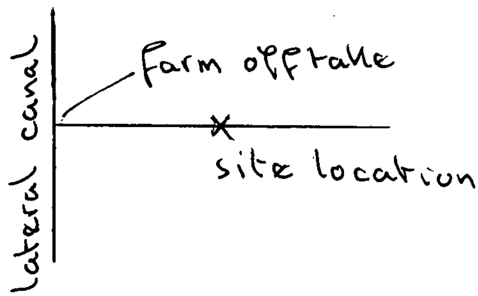
PERIOD OF STRUCTURE SERVICE

Day	☐	Season	☒
Month	☐	Permanent	☐

DESCRIPTION OF ENVIRONMENT

Irrigation System		Drainage System	
Main	☐	From irrigated area	☐
Lateral	☐	Artificial drain	☐
Farm ditch	☒	Natural drain	☐
In field	☐		

Plan of site:



FURTER DESCRIPTION
(attach photo)

Hypothetical site based on information obtained from Kodak Farms, Windsor, Colorado, USA

Table 5.7 Data for design example.

for an unlined canal where both width and sill height have a wide range of possible values. Generally, the choice between side and bottom contractions is made on the basis of flow measurement accuracy versus constructability of the device.

5.6.2 Design equations

The design requirements can be put into equation form, as follows:

Modular flow at Q_{max} 5.7

$$H_{1max} > H_{2max} + \Delta H_{max}$$

or approximately

$$h_{1max} > h_{2max} + \Delta h_{max}$$

Modular flow at Q_{max} 5.8

$$H_{1min} > H_{2min} + \Delta H_{min}$$

or approximately

$$h_{1min} > h_{2min} + \Delta h_{min}$$

Freeboard 5.9

$$h_{1max} < d - p_1 - F_1$$

Froude number 5.10

$$\mathrm{Fr}_1 = \frac{Q_{max}/A_{1max}}{\sqrt{gA_{1max}/B_{1max}}} < 0.5$$

or

$$\frac{A_{1max}{}^3}{B_{1max}} > \frac{4Q_{max}{}^2}{g}$$

Accuracy at Q_{max} 5.11

$$h_{1max} > \frac{u\delta_{h1}}{\sqrt{X_{Qmax}{}^2 - X_{Cmax}{}^2}}$$

Accuracy at Q_{min} 5.12

$$h_{1min} > \frac{u\delta_{h1}}{\sqrt{X_{Qmin}{}^2 - X_{Cmin}{}^2}}$$

Equations 5.7 through 5.12 are written so that the upstream head, h_1, and/or variables that are known functions of h_1, are on the left hand side. All of these inequalities prefer larger heads except the relationship for freeboard (Equation 5.9), which requires a smaller upstream head. The initial tradeoffs are between Equation 5.9 and Equations 5.7, 5.8 and 5.10.

Design starts with assuming an initial contraction amount. A good starting point is a contraction that is roughly one half of the approach section area (not counting the freeboard). The heads, h_{1min} and h_{1max}, for the design flows, Q_{min} and Q_{max}, can then be determined from rating tables, from the equations presented in either Section 6.4 or 6.5, or from the software in Chapter 8. This implies that initial values for the flume longitudinal dimensions have been chosen. Once these heads have been determined, Equations 5.7 through 5.12 can be evaluated. If the above design criteria are not satisfied, a higher or lower contraction amount can be tried.

In Equations 5.11 and 5.12, values for X_C depend on how the head-discharge relationship is determined. If the standard flumes and rating tables given in this book are used, the rating table errors are provided in the tables. They range from 2 to 3% (additional error is added if interpolation between columns is needed). If the head-discharge equations of Section 6.4 are used, then X_C can be found from Equation 6.28. If the head-discharge relationship is determined from the model of Section 6.5 or the program of Chapter 8, X_C can be found from Equation 6.44. Note that X_C from these equations depends upon H_1/L, and thus depends on flume dimensions. Initial values for X_C can be taken as 5% or 2% for Equation 6.28 and 6.44, respectively.

5.6.3 Requirements for flume longitudinal dimensions

The flumes and weirs presented in this book can provide accurate flow measurements only if constructed with appropriate dimensions so that they fit the

Table 5.8 Requirements for Flume Longitudinal Dimensions.

Dimension	Requirements
Approach section length, L_a	$L_a \approx H_{1max}$ $2\,H_{1max} < L_a + L_b < 3\,H_{1max}$
Converging transition length, L_b	Provide transition angle between 2.5:1 and 4.5:1, with 3:1 preferred. $L_b \approx 3\,p_1$ for bottom contraction. $L_b \approx 3(B_1 - B)/2$ for symmetrical side contraction (see Section 6.3.3). Use the larger of these two for a combined contraction.
Throat length, L	$1.43\,H_{1max} < L < 14.3\,H_{1min}$ for model or computer rating. $1.0\,H_{1max} < L < 10\,H_{1min}$ for rating based on experiments.
Diverging transition length, L_d	$L_d < 10\,p_2$, $L_a = 6p_2$ recommended.
Diverging transition slope, m:1	m equal to either 0 or 6 is recommended.
Tailwater channel length, L_e*	$L_e = 10(p_2 + L/2) - L_d$

* The length L_e is often not a part of the actual structure; WinFlume uses this length for calculation of friction losses in the diverging transition. When constructing a full-length rectangular-throated weir like that shown in Figure 5.10, L_e is the length of the additional energy recovery section downstream from the diverging transition.

requirements for analysis as long-throated flumes. Primarily this involves the lengths of the various flume components in the direction of flow. These length requirements ensure that the desired flow conditions occur so that the hydraulic theory described in Chapter 6 can be applied. The length requirements are summarized in Table 5.8. The reasoning behind these requirements includes the following:

- The gaging station should be located far enough upstream from the crest and converging section to be out of the drawdown zone that is created as the flow accelerates toward critical velocity at the control section. However, the gaging station should not be so far upstream that unnecessary head loss occurs between the gaging station and the control section. The gaging station should be at least H_{1max} upstream from the start of the converging transition, and about 2 to 3 times H_{1max} upstream from the sill or throat.
- The converging transition should be gradual, without offsets or sudden changes in wall alignment that might cause flow separation as the flow contracts toward the control section. The converging transition should not be too long, or the structure will be unnecessarily expensive. Transition slopes of 2.5:1 to 4.5:1 are recommended.
- For best measurement accuracy, a throat length should be selected so that $0.07 \leq H_1/L \leq 0.7$ at all measured flow rates.
- If a diverging transition is used, the recommended slope is 6:1 (horizontal:vertical) for good energy recovery and economical construction. The slope should not be flatter than 10:1.

The WinFlume computer program described in Chapter 8 checks flume designs against these requirements. If a device does not meet the requirements, warning messages will be generated, and the program will suggest lengths needed to correct the problem. Resolving these warning messages is straightforward in most cases, although the length of the converging transition can be problematic in a few unusual cases. Details and some suggestions for resolving such problems are provided in Section 8.8.9.

5.6.4 Selection of standard broad-crested weirs for lined trapezoidal channels

For lined trapezoidal canals, broad-crested weirs are an attractive option since they can be easily retrofitted into the existing lined section. The selection procedure follows the flume design steps given in Section 5.6.1 as follows:

1. Determine the range of discharges to be measured, Q_{max} and Q_{min}. Then, estimate or determine by separate means, the canal flow depth (y_2, Figure 5.2) without the weir in place for the maximum design discharge Q_{max}. This tailwater depth will be used to evaluate the weir design for submergence.

The portable and temporary weirs described in Section 3.3.3 are useful for determining the flow properties of the canal. For small canals, it is often possible to select a weir size either by observing the suitability of the temporary structure or through trial and error using several temporary structures. If y_2 cannot be measured at Q_{max}, the normal depth in the canal downstream from the weir at Q_{max} can be determined using the procedures given in Section 5.3.2. For lined channels where the flow depth is determined by channel friction (i.e., channels flowing at normal depth as opposed to flow depth resulting from the backwater effect of a downstream structure), weir design based on Q_{max} is sufficient. However, if other factors affect the flow depth so that the tailwater level drops more slowly with discharge than the flow depth upstream from the weir, submergence must also be checked at the minimum flow rate Q_{min}, and thus, y_2 at Q_{min} will need to be determined.

2. Determine the required freeboard, F_1. We recommend freeboard in the amount of 20% of the upstream sill-referenced head, $F_1 = 0.20\ h_{1max}$.
3. Select the desired flow measurement accuracy and determine the rating table uncertainty, X_C. For these weirs, we recommend X_{Qmin} = 8% and X_{Qmax} = 5%, with X_C = 2%. (X_C = 3% for rating Tables R.1 and R.2, but 2% when the rating is computed with WinFlume).
4. Choose a head measurement method and determine its measurement error, δ_{h1}. For example, select δ_{h1} = 10 mm for head reading with a staff gage with an approach section Froude number, Fr_1, of roughly 0.3 (roughly interpolated from Table 4.1). These data will be used to compute the needed upstream heads to satisfy the accuracy requirements using Equations 5.11 and 5.12. For these weirs, u is approximately 1.8.
5. Consult Table 5.2 (metric) or Table 5.3 (English) and locate the canal shape that fits the canal in question.
6. Select a weir for that canal shape so that the maximum design discharge, Q_{max} falls within the range of canal capacities (Columns 4 and 5).

If the canal shape does not appear in Table 5.2 or 5.3, one may still be able to obtain a flume design. If the canal bottom width is between two specified values, use the wider bottom and recalculate the sill heights based on the side slope with b_c for each weir and b_1 for that canal. If the side slopes differ from those given such that the area of the control section A^* varies by more than 1 or 2%, these rating tables cannot be used. (For further explanation, see Section 6.3.3 and Figure 6.11.) If the discharge falls below the ranges given, this style of weir is not applicable to your situation. The rectangular weirs of Section 5.5.2 may be more appropriate. These weirs form a list of trial structures; one or more may meet the design requirements. Evaluate them in Step 7 through 9, starting with the weir with the lowest sill. The lowest sill is recommended because it is the least expensive to construct, it has the least effect on upstream flow conditions, it has the lowest potential for sediment deposition, and it can be raised far easier than a higher sill can be lowered.

7. Determine the sill-referenced heads h_1 from the rating table for the weir selected in Step 6 (Tables R.1 or R.2, Appendix 4), and determine the required head loss ΔH for maintaining modular flow. Use either the value given for the weir selected in Table 5.2 or 5.3, or use $0.1H_1$, whichever is larger. Since h_1 is usually close to H_1, a value of $0.1h_1$ may be used as a first approximation.

8. Start by checking the submergence at Q_{max}. If that is not satisfied, choose the weir with the next highest sill height and repeat Step 7. If the submergence check is satisfied, check the freeboard. If the freeboard criterion is not satisfied, choose the next lowest sill height and repeat Step 7. If that sill height has already proven unsuccessful, then these standard flume sizes will not work for this site or one or more restrictions will have to be relaxed. If the submergence criteria at Q_{max} and the freeboard criteria are met, continue on and check the design criteria for Froude number, submergence at Q_{min}, and accuracy at Q_{min} and Q_{max}.
9. If design criteria are not met, select a higher or lower sill height and repeat Steps 7 and 8 (see Section 5.6.6 for recommendations).
10. Determine the appropriate weir dimensions from Table 5.7. We recommend a 3:1 ramp, except where the sill is relatively high compared to the flow depth. We recommend $L > 1.5\ H_{1max}$, but not less than the values given in the heading of either Table R.1 or R.2.

Example selection of standard broad-crested weirs for lined trapezoidal channels

Given: the data from the example in Section 5.3.2, where the bottom width $b = 0.3$ m, side slopes are 1:1, canal depth $d = 0.55$ m, bottom slope $S_b = 0.00050$ m/m, Manning roughness coefficient $n = 0.015$, and the range of discharges is from $Q_{min} = 0.05$ m^3/s to $Q_{max} = 0.15$ m^3/s;

Task: Follow the design procedure given above to select a suitable structure.

1. Note from the example in Section 5.3.2 that the tailwater levels at Q_{min} and Q_{max} are 0.240 m and 0.412 m, respectively.
2. Choose the recommended freeboard equal to 20% of h_1 at Q_{max}.
3. Choose accuracy requirements of $X_{Qmin} = 8\%$ and $X_{Qmax} = 5\%$. Use $X_C = 2\%$.
4. Assume head to be measured with a wall-mounted staff gage, roughly $\delta_{h1} = 7$ mm. (Assume $u = 1.8$ for Equations 5.11 and 5.12).
5. Find canal shape in Table 5.2. Note maximum canal depth is much greater than depth of this canal (0.75 versus 0.55 m).
6. Note that all the weirs listed, B_m through G_{m1}, have sufficient capacity, but the desired lower limit of flow rate is lower than all the lower limits listed. This implies that the method of head detection and desired accuracy need to be further examined.
7. For each weir, B_m through G_{m1}, determine the head at minimum and maximum discharge from Table R.1 and estimate the head-loss values from Table 5.2. The required freeboard F_1 is computed as 20% of h_{1max}.

Weir	h_{1max} (m)	h_{1min} (m)	p_1 (m)	ΔH (m)	y_{1max} (m)	h_{2max} (m)	h_{2min} (m)	F_1 (m)
B_m	0.219	0.118	0.15	0.017	0.369	0.262	0.09	0.044
C_m	0.208	0.110	0.20	0.021	0.408	0.212	0.04	0.042
D_{m1}	0.197	0.103	0.25	0.025	0.447	0.162	-0.01	0.039
E_{m1}	0.187	0.097	0.30	0.029	0.487	0.112	-0.06	0.037
F_{m1}	0.178	0.091	0.35	0.033	0.528	0.062	-0.11	0.036
G_{m1}	0.163	0.083	0.40	0.039	0.563	0.012	-0.16	0.033

8. Examine the design criteria for each weir. The WinFlume program actually determines the range of sill heights that will satisfy the submergence and freeboard requirements. For this example, the range of sill heights is 0.24 m to 0.33 m. Thus, of the standard weirs, only the 0.25-m and 0.3-m high sill are acceptable, and the 0.25-m high sill only barely satisfies the submergence criteria at maximum flow.

Weir	Modular flow at Q_{max}, Equation 5.7 $H_{1max} > H_{2max} + \Delta H_{max}$ or approximately $h_{1max} > h_{2max} + \Delta h_{max}$	Modular flow at Q_{min}, Equation 5.8 $H_{1min} > H_{2min} + \Delta H_{min}$ or approximately $h_{1min} > h_{2min} + \Delta h_{min}$	Freeboard Equation 5.9 $h_{1\,max} < d - p_1 - F_1$
B_m	0.219 $\not>$ 0.262 + 0.017	0.118 > 0.09 + 0.017	0.219 < 0.55–0.15–0.044
C_m	0.208 $\not>$ 0.212 + 0.021	0.110 > 0.04 + 0.021	0.208 < 0.55-0.20-0.042
D_{m1}	**0.197 > 0.162 + 0.025**	**0.103 > -0.01 + 0.025**	**0.197 < 0.55-0.25-0.039**
E_{m1}	**0.187 > 0.112 + 0.029**	**0.097 > -0.06 + 0.029**	**0.187 < 0.55-0.30-0.037**
F_{m1}	0.178 > 0.062 + 0.033	0.091 > -0.11 + 0.033	0.178 $\not<$ 0.55-0.35-0.036
G_{m1}	0.163 > 0.012 + 0.039	0.083 > -0.16 + 0.039	0.163 $\not<$ 0.55-0.40-0.033

Weir	Froude Number Equation 5.10 $Fr_1 = \frac{Q_{max}/A_{1max}}{\sqrt{gA_{1max}/B_{1max}}} < 0.5$	Accuracy at Q_{max} Equation 5.11 $h_{1max} > \frac{u\delta_{h1}}{\sqrt{X_{Qmax}^2 - X_{Cmax}^2}}$	Accuracy at Q_{min} Equation 5.12 $h_{1min} > \frac{u\delta_{h1}}{\sqrt{X_{Qmin}^2 - X_{Cmin}^2}}$
B_m	0.399 < 0.5	0.219 $\not>$ 0.275	0.118 $\not>$ 0.163
C_m	0.327 < 0.5	0.208 $\not>$ 0.275	0.110 $\not>$ 0.163
D_{m1}	**0.272 < 0.5**	**0.197 $\not>$ 0.275**	**0.103 $\not>$ 0.163**
E_{m1}	**0.228 < 0.5**	**0.187 $\not>$ 0.275**	**0.097 $\not>$ 0.163**
F_{m1}	0.193 < 0.5	0.178 $\not>$ 0.275	0.091 $\not>$ 0.163
G_{m1}	0.165 < 0.5	0.163 $\not>$ 0.275	0.083 $\not>$ 0.163

9. Select weir E_{m1}, since it satisfies both modular flow and freeboard requirements, and provides a slight safety margin in case of higher tailwater levels in the future (e.g., due to weed growth or concrete deterioration). Note that none of these weirs meet the accuracy criteria. Actually, for the lower sills the accuracy is worse since the head detection error value we used was based on the assumption of the approach channel Froude number being less than 0.2; the actual Froude number reaches nearly 0.4. Reasonable accuracy at maximum flow can be obtained by reading the water level in a stilling well (δ_{h1} = 5 mm with h_{1max} = 0.187 m gives 5.2% accuracy). This still does not quite satisfy our accuracy requirement at minimum flow (9.5% rather than 8% accuracy), but is close enough that we will accept this result. If the rating tables are used directly (X_c=3%), then the accuracies reduce to 5.7% and 9.8% for Q_{max} and Q_{min}, respectively. If greater accuracy is required, then either a narrower flume is needed or the water depth must be measured with a more accurate gage (e.g., point gage).
10. Determine the longitudinal dimensions of the structure from Table 5.7: L_a = 0.2 m, L_b = 0.9 m, L = 0.35 m, L_d = 0 m, and the downstream slope expansion factor is m = 0 since there is adequate head drop available and we do not need a downstream ramp.

5.6.5 Selection of rectangular broad-crested weirs for earthen channels

Because of the wide variety of shapes that can be encountered in earthen channels and the range of discharges to be measured, it is rather complicated to determine the interrelated values of h_{1max}, p_1 and b_c for a structure. Rectangular weirs have proven to be an effective option for these unlined canals since a rectangular section can be constructed fairly easily out of block, stone, or masonry. (For channels deeper than about 1.5 m (5 ft), custom designed structures are recommended, often trapezoidal or triangular rather than rectangular.) Although this situation makes the design process somewhat more complicated, it allows the designer greater flexibility and expands the applicability of the weirs. In particular, because earthen channels are often very inefficient sections, flow velocities tend to be low, requiring considerable contraction. Often a side contraction is required to produce sufficient upstream head to achieve reasonable accuracy. Criteria for accuracy are useful for reducing the range of structures to be considered.

The design steps for a rectangular weir in an earthen channel are similar to those given above for a lined trapezoidal channel:

1. Determine the range of discharges to be measured, Q_{max} and Q_{min}. Then, estimate or determine by separate means, the canal flow depth (y_2) without the weir in place for the maximum design discharge, Q_{max}, and the minimum design discharge, Q_{min}. These tailwater levels will be used to evaluate the weir for submergence at Q_{min} and Q_{max}. The portable and adjustable flumes of Section 3.3.3 or temporary weirs described in Section 3.2.3. are useful for determining the flow properties of the canal. For earthen canals, submergence should be checked at the minimum flow rate Q_{min}. This is because the upstream head created by a rectangular contraction often drops faster with declining discharge than do the levels in a downstream trapezoidal or rough earthen tailwater channel. As a result, submergence is often a greater issue at Q_{min} for rectangular-throated structures, so a higher sill may be needed to avoid submergence at minimum flow. This may make it more difficult to meet freeboard requirements at high flows.
2. Select the required freeboard. For earthen channels. we recommend freeboard as a percentage, at least 10%, of the channel depth, $F_1 = 0.10(d)$. This effectively gives a maximum water level, $y_{1max} = 0.9(d)$.
3. Select the desired flow measurement accuracy. For these weirs, we recommend $X_{Qmin} = 8\%$ and $X_{Qmax} = 5\%$, with $X_C = 2\%$.
4. Select $\delta_{h1} = 10$ mm for a head reading with a staff gage in a channel with Fr_1 roughly equal to 0.3 (see Table 4.1 for adjustment if necessary). This will be used to compute the needed upstream heads, h_{1min} and h_{1max}, to satisfy the accuracy requirements with Equation 5.11 and 5.12. For these weirs, u is approximately 1.5.
5. Consider a rectangular cross section for the entire flume (including approach and tailwater channels) that is narrower than the earthen channel top width but wider than the bottom width. For very low velocity canals, consideration should be given to placing the bottom of the approach channel above the bottom of the earthen canal.

Table 5.9 Options for satisfying design requirements.

Design Requirement	*Options to Consider if Requirement is not Satisfied*
Modular Flow at Q_{max} (submergence)	• Raise the crest • Narrow the control section at Q_{max} • Add a downstream ramp (6:1 slope recommended) • Choose a location where more drop is available
Modular Flow at Q_{min} (submergence)	• Raise the crest • Narrow the control section at Q_{min} • Add a downstream ramp (6:1 slope recommended) • Choose a location where more drop is available
Freeboard at Q_{max}	• Lower the crest • Widen the control section at Q_{max}, or • Raise the maximum allowable water level (decrease required freeboard or increase height of canal banks)
Froude Number at Q_{max}	• Increase approach channel depth at Q_{max} (by increasing contraction) • Deepen the approach channel • Increase approach channel top width
Accuracy at Q_{max}	• Narrow the control section at Q_{max} • Use a more accurate head-detection method • Increase the allowable measurement error at Q_{max}
Accuracy at Q_{min}	• Narrow the control section at Q_{min} • Use a more accurate head-detection method • Increase the allowable measurement error at Q_{min}

6. Enter Tables R.3 or R.4 (Appendix 4) with values of h_{1min} and h_{1max} from Step 4. Find a table containing both h_1 values. Read over to one of the columns and select a unit discharge q. Calculate the required widths from $b_c = Q/q$. Use the smaller b_c computed from Q_{min}/q_{min} and Q_{max}/q_{max}. If the b_c from q_{min} is smaller, recompute $q_{max} = Q_{max}/b_c$ and be sure that it is still in the column from which the q values were chosen. If not, go to the table for the next widest set of weirs. If the weir is wider than the average channel width, use a narrower weir, if possible. Also, check to be sure that the width chosen is within the width range for that set of ratings. If the width is too narrow, go to the next widest grouping and repeat. If the width is too wide, go to the next narrowest grouping. If the head range for this grouping is too small, you will have to use a wider weir and allow more error in the measurement, or use the methods of Chapter 6 or 8 to develop a new rating. Start by selecting the lowest sill height in the table or the sill height for the lowest sill for which the range of heads is included.
7. Determine the sill-referenced heads h_1 from the rating table for the weir selected in Step 6 (Tables R.3 or R.4). And determine the required head loss ΔH for maintaining modular flow. Use the value given for the weir selected in Table R.3 or R.4 or $0.1H_1$, whichever is larger. Since h_1 is usually close to H_1, $0.1h_1$ may be used as a first approximation. For a structure discharging into a wide channel, use $0.4H_1$ or compute the actual head loss (see Section 6.6).
8. Evaluate the design criteria, starting with the submergence at Q_{max}. If that is not

satisfied, choose the next highest sill height and repeat Step 7. If that is satisfied, check the freeboard. If the freeboard criteria is not satisfied, choose the next lowest sill height and repeat Step 7. If that sill height has already proven unsuccessful, then either these standard sill heights will not work in that canal, the structure width will have to be changed, or one or more design requirements will have to be relaxed. Continue on and check the design criteria for Froude number, submergence at Q_{min}, and accuracy at Q_{min} and Q_{max}.

9. Alter the structure width or sill height according to the criteria that are not met, or relax the design criteria. (see Section 5.6.6 for recommendations).
10. Determine the appropriate longitudinal dimensions from Table 5.7. We recommend a 3:1 ramp, except where the sill is relatively high compared to the flow depth. We recommend $L > 1.5\ H_{1max}$, but not less than the values given in the heading of either Table R.3 or R.4.

Example selection of rectangular broad-crested weirs for earthen channels

1. The data for this example are shown in Table 5.8. Initially, the full-length structure of Figure 5.10 will be assumed.
2. Choose a freeboard of 10% of the channel depth. In this case, the maximum allowable water depth is 0.6 m. For Equation 5.9, we assume the channel depth is 0.6 m and the freeboard amount is zero.
3. Choose accuracy requirements of $X_{Qmin} = 7\%$ and $X_{Qmax} = 5\%$. Use $X_C = 2\%$.
4. Assume head to be measured with a wall-mounted staff gage, roughly $\delta_{h1} = 7$ mm for a Froude number of 0.2. (Assume $u = 1.5$ for Equations 5.11 and 5.12). Compute the minimum heads required to provide these accuracies at Q_{min} and Q_{max} from Equations 5.11 and 5.12.

$$h_{1max} > \frac{u\delta_{h1}}{\sqrt{X_{Qmax}{}^2 - X_{Cmax}{}^2}} = \frac{1.5 * 0.007\text{ m}}{\sqrt{0.05^2 - 0.02^2}} = 0.229\text{ m}$$

$$h_{1min} > \frac{u\delta_{h1}}{\sqrt{X_{Qmin}{}^2 - X_{Cmin}{}^2}} = \frac{1.5 * 0.007\text{ m}}{\sqrt{0.07^2 - 0.02^2}} = 0.157\text{ m}$$

5. Choose a rectangular cross section for the entire structure.
6. Enter Table R.3 with the above head values, roughly 0.16 and 0.23 m. Starting with the narrowest width range, choose the first table that contains both head values. This is the table for widths of 0.2 m to 0.3 m. The largest width is 0.3 m, which would produce a maximum unit discharge of 0.34 m^3/s divided by 0.3 m, or 1.13 m^3/s per meter width. The rating in this table does not go that high, so a larger width range is tried. At 0.5 m width, the maximum discharge would be 0.34/0.5 or 0.68 m^3/s per meter width. Again, this table does not go that high. The next table has a 1 m width, giving 0.34 m^3/s per meter width, which occurs in this table. So we start with widths in the range 0.5 to 1.0 m. For all these tables at the head of 0.23 m, the discharge is on the order of 0.2 m^3/s per meter of width. Since the maximum flow rate is 0.34 m^3/s, the width must be less than

0.34/0.2 or 1.7 m. Since the channel is effectively 1.2 m wide, the accuracy at maximum flow is not a limiting constraint. At a head of 0.16 m, flow rates in Table R.3 are on the order of 0.11 m^3/s per meter width. Since the minimum flow is 0.085 m^3/s, the width must be narrower than 0.085/0.11 or 0.77 m. The first trial will assume a 0.75 m width and it will be adjusted from there depending on other design limitations.

7. For a weir width of 0.75 m, the maximum unit discharge is 0.34 m^3/s divided by 0.75 m, or 0.453 m^3/s per meter width, while the minimum unit discharge is 0.113 m^3/s per meter width. From Table R.3, only the higher two sill heights are acceptable, with the lower sill height causing the Froude number to be too high in the approach.

p_1 (m)	h_{1max} (m)	h_{1min} (m)	ΔH (m)	y_{1max} (m)	h_{2max} (m)	h_{2min} (m)
0.20	0.386	0.162	0.048	0.586	0.26	0.05
0.30	0.395	0.167	0.063	0.695	0.16	-0.05

8. The design criteria for these two designs are examined.

Sill Height p_1 (m)	Modular flow at Q_{max}, Equation 5.7 $H_{1max} > H_{2max} + \Delta H_{max}$ or approximately $h_{1max} > h_{2max} + \Delta h_{max}$	Modular flow at Q_{min}, Equation 5.8 $H_{1min} > H_{2min} + \Delta H_{min}$ or approximately $h_{1min} > h_{2min} + \Delta h_{min}$	Freeboard Equation 5.9 $h_{1max} < d - p_1 - F_1$
0.2	0.386 > 0.26 + 0.048	0.162 > 0.05 + 0.048	0.368 < 0.6–0.20–0.0
0.3	0.395 > 0.16 + 0.063	0.167 > -0.05 + 0.063	0.395 $\not<$ 0.6-0.30-0.0

Sill Height p_1 (m)	Froude Number Equation 5.10 $\mathrm{Fr}_1 = \dfrac{Q_{max}/A_{1max}}{\sqrt{gA_{1max}/B_{1max}}} < 0.5$	Accuracy at Q_{max} Equation 5.11 $h_{1max} > \dfrac{u\delta_{h1}}{\sqrt{X_{Qmax}^2 - X_{Cmax}^2}}$	Accuracy at Q_{min} Equation 5.12 $h_{1min} > \dfrac{u\delta_{h1}}{\sqrt{X_{Qmin}^2 - X_{Cmin}^2}}$
0.2	0.323 < 0.5	0.386 > 0.229	0.162 > 0.157
0.3	0.250 < 0.5	0.395 > 0.229	0.167 > 0.157

9. Choose the sill height of 0.2 m, since the higher sill causes the upstream water level to become too high. This design will satisfy the accuracy requirement. However, if rating table R.3 is used directly, X_C increases to 3% and the accuracy at Q_{max} and Q_{min} are 4.4% and 8.3%, respectively.

10. Dimensions are determined from Table 5.8: L_a = 0.4 m, L_b = 0.6 m, L = 0.6 m, L_d = 0 m, m = 0, L_e = 5 m. The requirement for the tailwater section makes this structure extremely long, while there is considerable extra head loss available. Without making detailed calculations for the specific channel, the head loss for a truncated structure (i.e., with L_d = 0 m and L_e = 0 m) is 0.4 H_{1max}, or 0.154 m. Since only 0.126 m are available, the tailwater channel cannot be eliminated without a more detailed evaluation.

Methods for making the head loss calculations are given in Section 6.6, or the calculations can be made with the WinFlume software of Chapter 8 for the tailwater channel defined by the cross section of the earthen channel. In this case, the calculations would show that only 0.079 m of head loss is required for a truncated structure; thus there is plenty of head available.

5.6.6 What to do when design criteria are not met

For each of the design requirements, if the criteria are not met, the options for changes to satisfy the criteria are straightforward. These options are presented in Table 5.9. If more than one of the criteria are not met, the options may be conflicting, sometimes suggesting that a design is not possible for those criteria, or one or more criteria need to be relaxed, if feasible. However, sometimes the options only *appear* to conflict and a feasible design is possible. Take, for example, the requirements for modular flow and freeboard at Q_{max}. One suggests raising the crest; the other suggests lowering the crest. One suggests narrowing the control section; the other suggests widening the control section. Individually, these seem in direct conflict. However, it may be possible to both raise the crest and widen the control section and satisfy both requirements, because a wider control will require less head at Q_{max}, and thus less head loss (since ΔH is generally proportional to H_1). The reduced head loss may make it possible to satisfy both criteria. Until all the options are explored it is difficult to conclude that a design is not possible.

In many cases, design will be an iterative process, with many trials before a final design is selected. This procedure appears to be fairly complex; however, once the designer becomes familiar with the important features, the design becomes quick and easy. The difficult (but important) part is accurately estimating the flow conditions prior to placement of the structure. These, more than anything else, determine the constraints on the design. The following example should be helpful. It makes use of the WinFlume computer program described in Chapter 8.

Examples

Consider the design example given in Section 5.6.4. This is a concrete-lined trapezoidal channel with bottom width b = 0.3 m, side slopes of 1:1, canal depth d = 0.55 m, bottom slope S_b = 0.00050 m/m, and Manning roughness coefficient n = 0.015. The range of discharges is from Q_{min} = 0.05 m^3/s to Q_{max} = 0.15 m^3/s. We wanted to achieve measurement accuracy of ±8% at minimum flow and ±5% at maximum flow and maintain freeboard of at least 20% of h_1. We determined the tailwater levels for this site in the example given in Section 5.3.2.

Recall that a weir was chosen with a crest height of 0.30 m, which for this channel shape produces a crest width of 0.90 m and a maximum sill-referenced head of 0.187 m. This original weir (Weir-0 in the table below) was constructed by using only a bottom contraction. While this weir satisfied the Froude number, freeboard, and submergence criteria, it did not satisfy the accuracy criteria. We determined that use of a stilling well rather than a wall gage would provide sufficient accuracy, but suppose the designer wanted better accuracy with a wall gage. The flow cross section over this weir is very wide and shallow (0.187 m deep by 0.9 to 1.27 m wide, at maximum flow), making it difficult to accurately measure the small sill-referenced head. A combined side and bottom contraction may be possible that will increase the head at maximum flow and satisfy all the criteria.

A) First, we will try simply raising the trapezoidal section (0.30 m wide with 1:1 side slopes) vertically. WinFlume has an option for raising the entire shape. WinFlume also has a search procedure that determines how far the sill must be raised to yield an acceptable design. This analysis shows that acceptable flume designs can be found over a range of sill heights of 0.152 m to 0.189 m. Over this range of sill heights, because of the side contraction, the accuracy criteria are also met at both minimum and maximum flow. The middle of this range is a sill height of 0.17 m. This flume is listed as Flume-1 in the table below. A variety of other combinations of side and bottom contraction may also produce a satisfactory design. Because the accuracy criteria are barely met by the Flume-1 design, other acceptable designs would probably need a lower sill and narrower throat section.

	Design Requirements					
	Froude No.	Freeboard	Submergence	Submergence	Accuracy	Accuracy
Trial	Fr_1	F_1	y_{1max}	y_{2min}	h_{1max}	h_{1min}
	Actual < 0.5	Actual > Req.	Actual < Max.	Actual < Max.	Actual > Min.	Actual > Min.
Weir-0	0.228 < 0.5	0.064 > 0.037	0.412 < 0.458	0.240 < 0.375	0.186 ≯ 0.247	0.097 ≯ 0.141
Flume-1	0.245 < 0.5	0.080 > 0.060	0.412 < 0.429	0.240 < 0.307	0.300 > 0.292	0.169 > 0.161
Flume-2	0.267 < 0.5	0.049 ≯ 0.060	0.412 ≯ 0.412	0.240 < 0.290	0.298 > 0.292	0.168 > 0.161
Flume-3	0.281 < 0.5	0.061 > 0.059	0.412 ≯ 0.403	0.240 < 0.281	0.297 > 0.292	0.168 > 0.161
Weir-4	0.272 < 0.5	0.054 > 0.039	0.412 < 0.420	0.240 < 0.332	0.197 ≯ 0.250	0.103 ≯ 0.143
Flume-5	0.281 < 0.5	0.061 > 0.059	0.412 < 0.417	0.240 < 0.293	0.297 > 0.292	0.168 > 0.161

B) Now we will assume that the canal depth is 0.50 m instead of 0.55 m. In this case, neither the original Weir-0 nor our new Flume-1 design will satisfy the freeboard criteria (i.e., actual freeboard would decrease by 0.05 m). We then move the entire throat cross section down to satisfy the freeboard requirement. At $p_1 = 0.152$ m and below, the submergence requirement is no longer satisfied, as shown by Flume-2 (Figure 5.13). This flume is right on the edge of submergence (i.e., less than 1 mm difference between actual and maximum allowed y_{2max}). If we continue to reduce the sill height, we eventually satisfy the freeboard requirement at $p_1 = 0.142$ m (Flume-3). In between these two sill heights, neither submergence nor freeboard is satisfied. This overlapping range of unsatisfied design criteria shows that no design is possible by raising or lowering the entire section and suggests the need to alter the shape. A shallower and wider flow through the throat would reduce the head, require less head loss (because ΔH is proportional to H_1), increase the available freeboard, and reduce the freeboard requirement (which is related to the head). To test this, we arbitrarily raise the sill to 0.25 m, then have WinFlume vary the amount of side contraction to search for an acceptable design. Both freeboard and submergence are satisfied over a narrow range of throat widths; 0.724 to 0.800 m. The throat width of 0.8 m is a broad-crested weir design (Weir-4). Unfortunately, although the freeboard and submergence criteria are met, the accuracy requirements are not met. In fact, meeting the accuracy criteria would require a throat bottom width of 0.35 m or less. Between widths of 0.724 m and 0.35 m, neither freeboard nor accuracy is met. Since the unsatisfied criteria overlap, we have run out of options for improving the design in this manner. (Note that the overlapping range of widths will change with different sill heights. A few more sill heights would eventually verify that no design is possible, as they overlap with the overlapping ranges of unsatisfied design criteria for Flume-2 and Flume-3).

C) Another option for resolving the problem we faced in (B) is to add a diverging transition to the Flume-3 structure. This would reduce the head loss, which might allow us to reduce the sill height enough to meet the freeboard requirement. Since the flume was already narrow enough to satisfy the accuracy requirement, we would then have a design that meets all of the objectives. To test this in WinFlume, we add a 6:1 downstream ramp to Flume-3, since all criteria other than submergence are met. Adding the downstream ramp is sufficient to provide a satisfactory design (Flume-5). Analysis would show that sill heights of 0.137 m to 0.142 m will satisfy all criteria. When this flume is constructed, in the diverging transition we would provide both a 6:1 floor ramp and a 6:1 transition from the sidewalls of the throat section back to the sidewalls of the downstream channel.

To summarize, Figure 5.13 shows the cross sections for the various flumes and weirs. For the 0.55-m canal depth, the broad-crested weir was too high, causing the flow to be too wide and shallow to meet accuracy objectives. The Flume-1 design was narrower with deeper flow, which improved the accuracy of the flow measurement, meeting the design requirement. For the lower canal depth (0.50 m), a narrowed-throat design (Flume-2) that met accuracy requirements was so narrow that freeboard and submergence criteria could not be simultaneously satisfied. A broad-crested weir design (Weir-3) was found that could satisfy free-board/submergence criteria, but did not meet the accuracy requirements. Finally, by adding a diverging transition to the Flume-2 design, we were able to develop a design (Flume-4) that satisfies all of the design criteria.

These examples demonstrate most of the essential tradeoffs to be considered in flume design. The Froude number in the approach channel and the submergence at minimum flow rate were the only criteria that did not come into play in the examples, as they were easily satisfied by all of the alternatives we considered. The design for the reduced canal depth was tightly constrained, as are many designs that must be retrofitted to existing canal system. Sometimes, a number of different options must be considered before an acceptable design is found. Designs for new canals are usually more straightforward because the head loss needed by the flume can be easily incorporated into the design of the canal system.

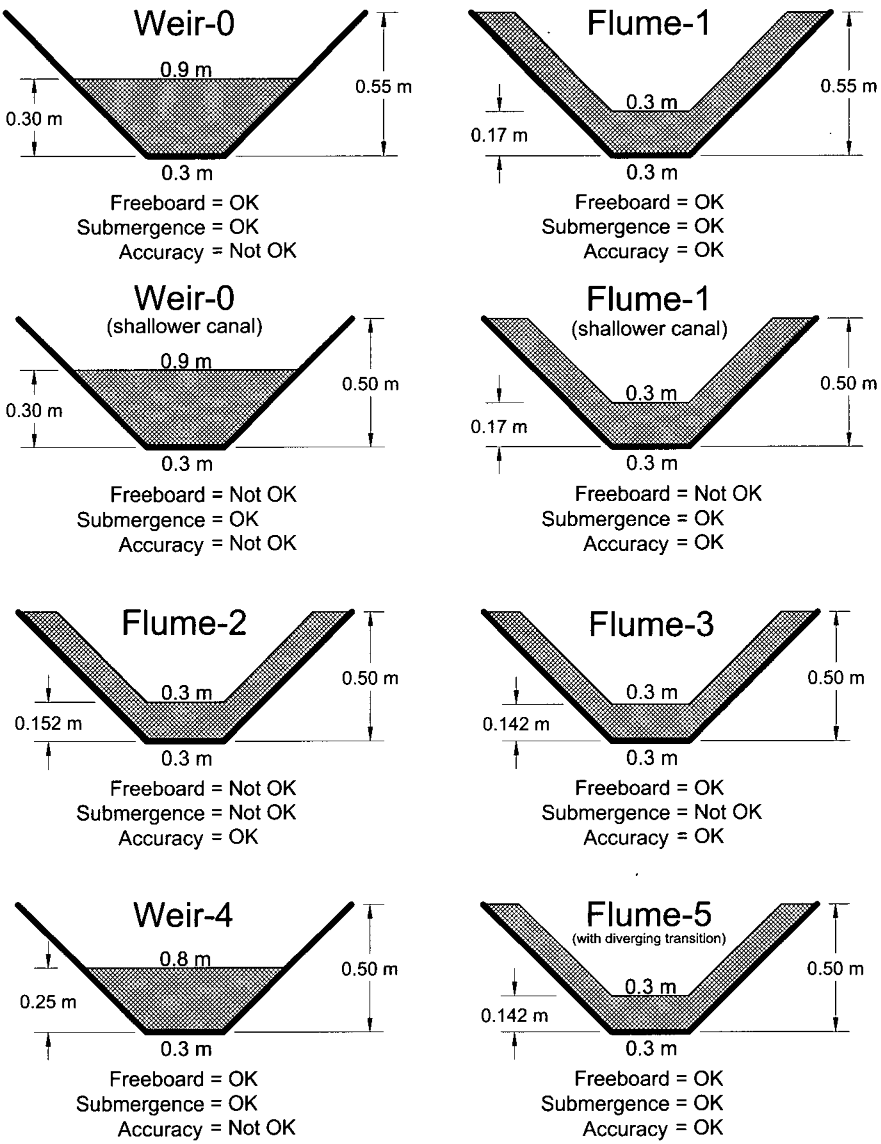


Figure 5.13. Comparison of various structures for design example.

5.7 Using WinFlume to Develop Custom Flume Designs

The procedures described in this chapter can be used to design a wide variety of flumes and weirs, using pre-computed designs for smaller structures in standard-sized canals. For design of larger structures, more detailed analysis is warranted, both to ensure that the structure satisfies the design requirements and to obtain the most accurate possible head-discharge rating. This analysis capability is provided by the WinFlume computer program (Chapter 8), which was used for the examples in

Section 5.6.6. The program evaluates a range of flume designs based on an initial throat section shape and a method of contraction change specified by the user. A report summarizing the acceptable designs is produced, and the user can then consider the tradeoffs among these designs before choosing a preferred structure. For tightly constrained problems, the user may need to examine several different methods of contraction change before arriving at a suitable design. Once a design has been selected, rating tables for the structure can be computed. Chapter 8 provides detailed information on the use of the program.

6. Hydraulic Theory and Computations for Long-Throated Flumes

The purpose of this chapter is to explain the fundamental principles involved in analytically modeling the flow through weirs and flumes, so that the head-discharge relationship and the modular limit of such structures can be determined. The information provided allows flumes of nearly any cross-sectional shape to be calibrated to a reasonable degree of accuracy. Software for making these calculations is presented in Chapter 8.

6.1 Continuity Equation

Figure 6.1 shows a stream tube that is a flow passage bounded by streamlines. Since there is, by definition, no flow across a streamline, and since we assume that water is incompressible, water must enter the tube through cross-section 1 and exit through cross-section 2 at the same flow rate, measured in volume per unit time. From the assumption of steady flow, it follows that the shape and position of the stream tube do not change with time. For steady flow, the rate, Q, at which water is flowing across a small section is the product of the average velocity component perpendicular to the section, v, and the area of the section, A. For cross sections 1 and 2 in Figure 6.1, we thus can write

$$Q = v_1 A_1 = v_2 A_2 \qquad 6.1$$

Equation 6.1 is the continuity equation and is valid for incompressible fluid flow through any stream tube. If Equation 6.1 is applied to a stream tube with a fixed and well-described boundary, such as a broad-crested weir with steady flow through it (the channel bottom, side slopes, and water surface being the boundaries of the stream tube, as shown in Figure 6.2), the continuity equation reads

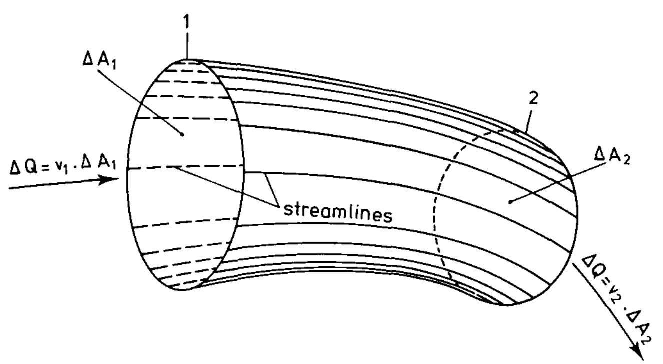


Figure 6.1 The stream tube.

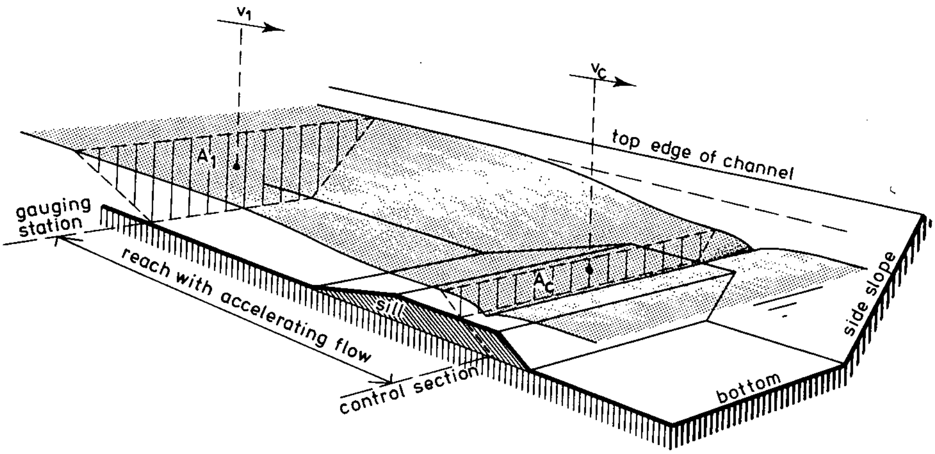


Figure 6.2 Cross-sectional area of flow at gaging station and control section.

$$Q = v_1 A_1 = v_c A_c = \text{constant} \qquad 6.2$$

where v_1 and v_c are the average velocity components perpendicular to the cross sections A_1 and A_c, respectively. In the broad-crested weir example shown, the subscript 1 indicates the gaging station upstream from the weir, and the subscript c indicates the section at which critical flow occurs.

6.2 Bernoulli's Energy Equation

Each particle of water has an individual velocity, u, an elevation, Z, a pressure, P, a heat, and a noise level. For the purpose of analyzing water flow in open channels, the heat and noise properties can be neglected. The others are related to quantities of energy that are of interest:

$\frac{1}{2}\rho u^2$ = kinetic energy per unit of volume,
P = pressure energy per unit of volume, and
$\rho g Z$ = potential energy per unit of volume,

where

ρ = mass density of the fluid and
g = acceleration due to gravity.

These energies are expressed in $kg/m/s^2$ or $Newtons/m^2$, units that do not appeal to many engineers. It is therefore common practice to assume that the mass density is constant (at $\rho = 1000\ kg/m^3$) and that the acceleration due to gravity does not change over the world ($g = 9.81\ m/s^2$), and to divide the preceding energies by the product ρg.

These energies then are written per unit of weight and are expressed in terms of water depth or head (m):

$\frac{u^2}{2g}$ = velocity head,

$\frac{P}{\rho g}$ = pressure head, and

Z = elevation head.

For a particle of water at location 1, these three heads are illustrated in Figure 6.3. In addition to the three heads already mentioned, the heads

$\frac{P}{\rho g} + Z$ = piezometric head

and

E = total energy head of the water particle

are commonly used. The total energy head and the elevation head Z are measured relative to the same reference level (see Figure 6.3). Hence, for the water particle at locations 1 and 2, we can write

$$E_1 = \frac{P_1}{\rho g} + Z_1 + \frac{u_1^2}{2g} \quad 6.3$$

$$E_2 = \frac{P_2}{\rho g} + Z_2 + \frac{u_2^2}{2g}$$

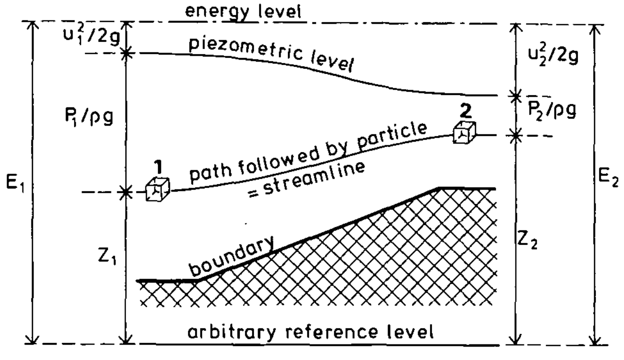


Figure 6.3 The energy of a fluid particle for steady flow.

If the distance between 1 and 2 is short and the energy loss due to friction and turbulence may be neglected, E_2 may be assumed to equal E_1 and thus

$$\frac{P_1}{\rho g} + Z_1 + \frac{u_1^2}{2g} \approx \frac{P_2}{\rho g} + Z_2 + \frac{u_2^2}{2g} \qquad 6.4$$

The reader should note that each water particle flows with a different velocity (u) at a different place and may have its own energy head. Equations 6.3 and 6.4 are alternative forms of the well-known Bernoulli equation that are valid along any streamline.

In a flow in which streamlines are straight and parallel, there is no acceleration of the flow perpendicular to the streamlines. Let us assume a flow that is horizontal as shown in Figure 6.4, and consider a thin column of water between points 1 and 2. The height of the column is Z_1-Z_2, and the cross-sectional area is dA. The forces acting on this column are the gravitational force, $\rho g(Z_1-Z_2)(dA)$ and a pressure force acting upward on the bottom of the column, $P_2(dA)$. Since the vertical acceleration of this column of water is zero, these forces must be equal and opposite, and the pressure is thus

$$P_2 = \rho g(Z_1 - Z_2) \qquad 6.5$$

This expression can be rearranged to obtain

$$\frac{P_2}{\rho g} + Z_2 = Z_1 = y = \text{constant} \qquad 6.6$$

Using this expression, the pressure can be calculated for any point, and its distribution is shown in Figure 6.4. Such a straight-line (or linear) pressure distribution is called hydrostatic. The pressure within the water column varies as a function of elevation in proportion to the acceleration of gravity.

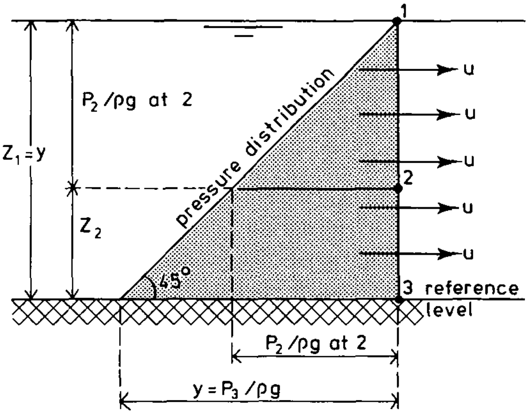


Figure 6.4 Hydrostatic pressure distribution perpendicular to straight and parallel streamlines.

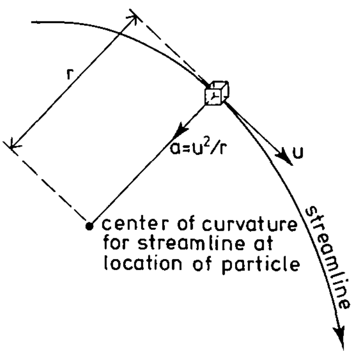


Figure 6.5 Centripetal acceleration.

If streamlines are curved so that a water particle with a unit volume follows a path with a radius r while its individual velocity equals u, this particle will undergo a centripetal acceleration u^2/r (see Figure 6.5). This centripetal acceleration is always perpendicular to the direction of the velocity and toward the center of curvature. Because the gravitational force on a volume of water is constant, the acceleration occurs due to a reduction in the pressure force. This produces a pressure gradient in which the reduction in pressure, ΔP, as one moves an incremental radial distance, Δr, toward the center of curvature is

$$\frac{\Delta P}{\Delta r} = \frac{\rho u^2}{r} \qquad 6.7$$

In the case of downward curvature, the negative pressure gradient is in the same direction as the gravitational acceleration, so the net effect is a reduction in the pressure as compared to the hydrostatic pressure distribution of flow with parallel streamlines (see Figure 6.6).

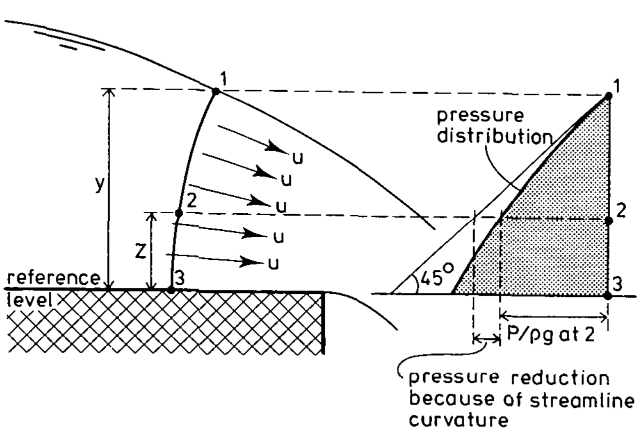


Figure 6.6 Influence of streamline curvature on pressure distribution.

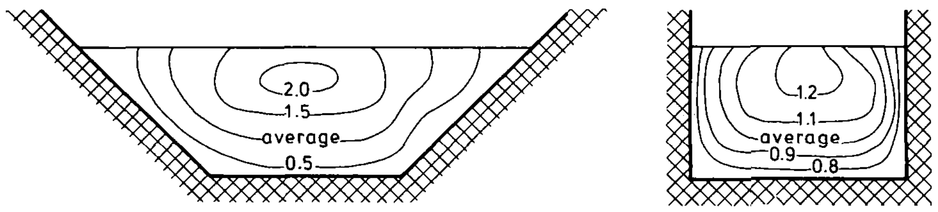


Figure 6.7 Examples of velocity distribution in channel sections.

The effect of the centripetal acceleration on the pressure and velocity distribution depends on the flow velocity, u, and the local radius of curvature, r, of the streamline at the considered location. Of these, the latter is especially difficult to measure, which makes calculations of the rate of flow through the control section 1 to 3 in Figure 6.6 time-consuming and inaccurate. If one streamline were curved as shown in Figure 6.6 and an adjacent streamline were curved in a plane perpendicular to the paper, the flow pattern would be three-dimensional, and the rate of flow could not be calculated from existing theory. Such a flow pattern occurs, for example, at a control section in a throat that is short with respect to the upstream sill-referenced head.

To be able to calculate the pressure and velocity distribution in the flume at the control section, the throat length must be sufficient for the streamlines to be practically straight and parallel to each other at this section. This may be assumed if the upstream sill-referenced head is less than half the throat length. The influence of larger heads on the head-discharge rating and on the modular limit of a flume will be shown in Sections 6.4.3 and 6.5, respectively.

We now want to determine the total energy of all the water particles that pass through the entire cross section of a channel. We therefore need to express the velocity head in Equations 6.3 and 6.4 in terms of the average velocity of all water particles in the cross section. This average velocity cannot be measured directly because the velocity is not uniformly distributed over the channel cross section. Two examples of velocity distribution for differently shaped channel sections are shown in Figure 6.7. The average velocity is a calculated velocity and is defined by the continuity equation as

$$v = \frac{Q}{A} \qquad 6.8$$

Because of the non-uniform distribution of the velocities u over the cross section, the true average velocity head $(u^2/2g)_{avg}$, will not necessarily be equal to $v^2/2g$. Therefore, a velocity distribution coefficient α is introduced so that

$$\left(\frac{u^2}{2g}\right)_{avg} = \alpha \frac{v^2}{2g} \qquad 6.9$$

The velocity distribution coefficient is 1.0 when all velocities u are equal, and it becomes greater the further the velocity distribution departs from uniform. For

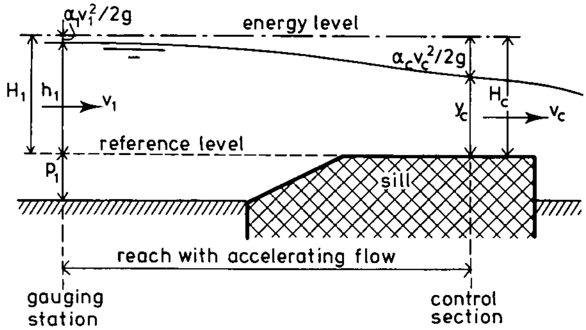


Figure 6.8 The energy level at the gaging station and the control section.

straight approach channels, α values vary between 1.03 and 1.10; for control sections in long throats, the velocity distribution is relatively uniform and α is less than 1.01. Since, in many cases, the velocity head at the approach section (i.e., the gaging station) is small with respect to the piezometric head, a value of $\alpha_1 = 1.04$ can be used without making an appreciable error in the determination of the total energy head. To simplify examples, a value of $\alpha_1 = 1.0$ will be used throughout this book with the exception of this chapter and in the computer program used to calculate the rating tables in this book (see Chapter 8).

The variation of the piezometric head terms in the energy equation depends on streamline curvature. In the two channel sections that we consider, the gaging station and the control section, streamlines are straight and parallel. Hence, according to Equation 6.6, the sum of the elevation head and pressure head is constant. Since at the water surface $P = 0$, the piezometric level of the sections coincides with the local water levels. For the gaging station, the total energy head relative to the sill of the structure, H_1, is (see Figure 6.8)

$$H_1 = h_1 + \frac{\alpha_1 v_1^2}{2g} = h_1 + \frac{\alpha_1 Q^2}{2gA_1^2} \qquad 6.10$$

For the control section, the total energy head is

$$H_c = y_c + \frac{\alpha_c v_c^2}{2g} \qquad 6.11$$

For the short reach with accelerating flow between the two sections, we may assume that energy losses due to friction and turbulence are negligible. Hence we may assume that $H = H_1 = H_c$, or

$$H_1 = h_1 + \frac{\alpha_1 v_1^2}{2g} \approx y_c + \frac{\alpha_c v_c^2}{2g} = H_c \qquad 6.12$$

Equation 6.12 is a variation of Bernoulli's energy equation valid for the channel reach described earlier (see Figure 6.8).

6.3 Critical Flow

6.3.1 Critical flow equations

Substitution of the continuity Equation 6.2 into Equation 6.12 gives

$$H_c = y_c + \frac{\alpha_c Q_c^2}{2gA_c^2} \qquad 6.13$$

where A_c, the cross-sectional area of flow at the control, can also be expressed in terms of water depth y_c. Solving this equation for Q_c produces

$$Q_c = A_c \sqrt{\frac{2g(H_c - y_c)}{\alpha_c}} \qquad 6.14$$

To calculate the flow rate, Q_c, with this equation, the values of both H_c and y_c must be measured. Since this is not very practical, we need to find an equation expressing y_c in terms of H_c. Because A_c is a function of y_c, the second term on the right-hand side of Equation 6.13 decreases with increasing y_c, and it can be shown that for a given shape of the control section and a constant discharge Q_c, there are two alternate depths of flow, y_c, for each energy head H_c (see Figure 6.9). For the large depth, y_{sub}, the flow velocity is low, and flow is called subcritical; for the shallow depth, y_{super}, the flow velocity is high and flow is called supercritical.

For one constant value of Q, Equation 6.13 can be used to plot a curve of channel-bottom-referenced energy head versus water depth, as shown in Figure 6.10. The water depths y_{sub} and y_{super} of Figure 6.9 and the related velocity heads are illustrated in the figure at an arbitrary total energy head, H.

For lower values of the total energy head, the difference between y_{sub} and y_{super} becomes smaller until they coincide at the minimum possible value of H. This corresponds to point c in Figure 6.10. The depth of flow at point c is known as *critical depth* and is denoted by y_c. All symbols that have the subscript c refer to a channel section in which flow is *critical*. This is the minimum possible energy head for the given flow, or alternately, the maximum possible flow rate for a given energy head.

We can find the relation between the depth and total energy head at the critical flow

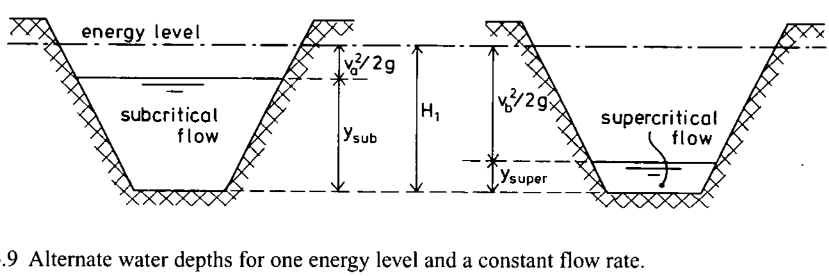


Figure 6.9 Alternate water depths for one energy level and a constant flow rate.

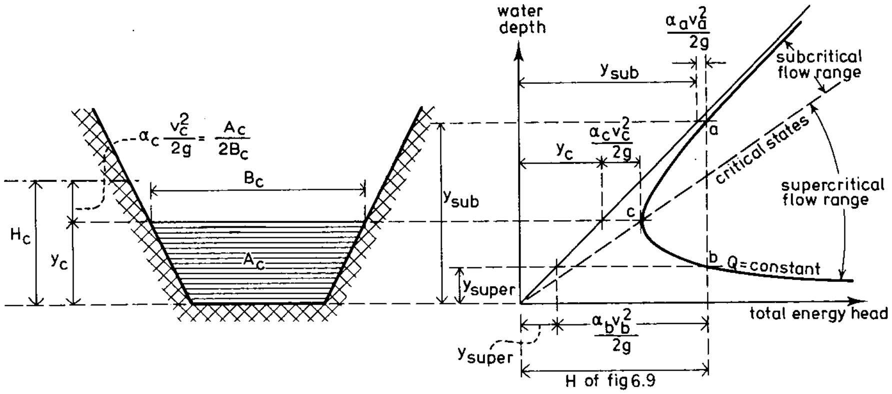


Figure 6.10 Energy curve for a constant rate of flow.

point by taking the derivative of Equation 6.13 with respect to y and setting it equal to zero (i.e., finding the local minimum on the H versus y curve). This yields $\alpha_c Q_c^2/g = A_c^3/B_c$, which can then be substituted back into Equation 6.13 to obtain

$$H_c = y_c + \frac{A_c}{2B_c} \qquad \text{or} \qquad H_c - y_c = \frac{A_c}{2B_c} \tag{6.15}$$

where

A_c = area of flow at the control section and
B_c = water surface width at the control section.

Both A_c and B_c are fully determined by y_c and it follows that the critical flow depth y_c in the control section is a unique function of the energy head, H_c. Since all of the terms on the right-hand side of Equation 6.14 are now known as a function of H_c, the discharge could be computed knowing only the value of H_c. The measurement of the upstream sill-referenced head, h_1, allows us to calculate H_1 which we noted earlier is equal to H_c when we assume no energy losses between station 1 and station c. However, we need not even directly compute H_1, since it can be eliminated from Equation 6.14 by substituting in $(H_c - y_c)$ from Equation 6.15. This gives

$$Q = \sqrt{\frac{gA_c^3}{\alpha_c B_c}} \tag{6.16}$$

This discharge equation is valid for critical flow in all arbitrarily shaped control sections. It is easily applicable if simple equations exist for both A_c and B_c. We must recall that this conclusion was reached only with the assumption that the flow is critical at station c; if the flow is not critical, then two alternate flow depths are possible and a solution cannot be obtained. For a discharge measurement structure, the flow at the control must be critical to avoid the need to measure the depth in the control section.

When the critical flow in the control section of the structure enters the tailwater channel, it must decelerate and there will be some associated energy loss. The total energy loss through the structure is the difference between the upstream sill-referenced energy head H_1 and the downstream sill-referenced energy head H_2. The ratio of these quantities is called the submergence ratio H_2/H_1. For low values of this submergence ratio, the tailwater level y_2 and the downstream energy head H_2 do not influence the relationship between upstream energy head and discharge (Equations 6.13 to 6.16), and flow is called modular. For high H_2/H_1 ratios, flow in the control section cannot become critical (there is not enough energy loss) and the upstream sill-referenced head and H_1 are influenced by the tailwater level; the flow is then non-modular. The submergence ratio at which modular flow turns into non-modular flow is named the modular limit. A method to estimate the modular limit is given in Section 6.6.

6.3.2 Calculating head-discharge relationships for ideal flow

For an ideal fluid there are no frictional energy losses. Thus, $H = H_1 = H_c$ and velocity distributions are uniform with α_1 and α_c both equal to unity. If critical flow of an ideal fluid occurs, the relationship between ideal flow, Q_i, energy head, $H_1 = H_c$, critical depth, y_c, and upstream water depth, h_1 is fixed by the geometry and the critical-flow relationships. Knowing just one of these quantities is sufficient to determine all the others.

Computing discharge when head is known

Consider the following 3 equations:

$$Q_i = \sqrt{\frac{gA_c^{\,3}}{B_c}} \qquad 6.17a$$

$$H_1 = h_1 + \frac{Q_i^2}{2gA_1^{\,2}} \qquad 6.17b$$

$$y_c = H_1 - \frac{A_c}{2B_c} \qquad 6.17c$$

There are four unknowns in these equations: Q_i, H_1, y_c, and y_1 (h_1 and y_1 are related through $y_1 = h_1 + p_1$). When any one of these is known, one can solve for the other three using straightforward trial and error procedures, provided that simple equations exist for A_c and B_c in terms of y_c and A_1 in terms of y_1. For a trapezoidal control section, for example, these equations are

$$A_c = y_c\left(b_c + z_c y_c\right) \qquad\qquad B_c = b_c + 2z_c y_c$$

where b_c is the bottom width of the control section and z_c is the side slope, (z horizontal to 1 vertical). The approach channel may also have any shape, but for the usual trapezoidal channel

$$A_1 = y_1(b_1 + z_1 y_1) \qquad\qquad y_1 = p_1 + h_1$$

If h_1 is known (and thus A_1 and y_1), an initial guess is made for y_c in terms of h_1. The value of y_c ranges from $0.67H_1$ to $0.80H_1$ for a rectangular to a triangular control section respectively. Neglecting the velocity head, $v_1^2/2g$, and thus assuming $h_1 \approx H_1$, we guess

$$y_c = 0.70\, h_1$$

It is not worthwhile making a better guess for y_c, since the trial-and-error method converges rapidly. Now, once y_c has been guessed, values of A_c, B_c, Q_i, H_1 and y_c can be computed from the above equations. If the new y_c-value equals the input y_c-value, then the computed Q_i is the flow rate for an ideal fluid matching the set h_1-value. After each trial, the new y_c-value replaces the previous y_c. Using the new y_c-value, a new series of calculations is made until the y_c values match.

Computing head when discharge is known

Alternatively, if the value for Q_i is given, Equation 6.17a is solved iteratively for y_c, since A_c and B_c are functions of y_c. Then, H_1 is found directly from Equation 6.17c, and h_1 is found from Equation 6.17b, again iteratively since A_1 is a function of h_1.

Example

Given: A trapezoidal flume with $b_c = 0.20$ m, $z_c=1.0$, $p_1 = 0.15$ m and $L = 0.60$ m is placed in a concrete lined canal with $b_1 = 0.50$ m and $z_1 = 1.0$.

Question: What is the discharge for an ideal fluid if the upstream sill-referenced head is $h_1 = 0.238$ m?

Answer: The actual upstream water depth is $h_1 + p_1 = 0.238 + 0.15 = 0.388$ m.

The upstream flow area is
$A_1 = 0.388\,[0.5 + 1.0(0.388)] = 0.345\ \text{m}^2$

First guess: $y_c = 0.7h_1 = 0.167$ m
Then: $A_c = y_c(b_c + z_c y_c) = 0.0661\ \text{m}^2$
$B_c = b_c + 2z_c y_c = 0.533$ m

$$Q_i = \sqrt{\frac{gA_c^3}{B_c}} = 0.0647\ \text{m}^3/\text{s} \qquad \text{Eq. 6.17a}$$

$$H_1 = h_1 + \frac{Q_i^2}{2gA_1^2} = 0.2398\ \text{m} \qquad \text{Eq. 6.17b}$$

$$\text{New: } y_c = H_1 - \frac{A_c}{2B_c} = 0.183\ \text{m} \qquad \text{Eq. 6.17c}$$

With the new value of y_c the above five values are recalculated until y_c remains constant. Thus

	Run 2	Run 3	Run 4	Run 5	Run 6
A_c	0.0698	0.0682	0.0677	0.0681	0.0681
B_c	0.565	0.559	0.558	0.559	0.559
Q_i	0.0769	0.0746	0.0739	0.0744	0.0744
H_1	0.2405	0.2404	0.2403	0.2404	0.2404
y_c	0.1788	0.1794	0.1796	0.1795	0.1795

Thus, the ideal flow rate is $Q_i = 0.0744$ m³/s.

6.3.3 Contraction needed for critical flow

The above equations are only valid if there is sufficient contraction to produce critical flow. Clemmens and Bos (1992) developed relationships for the amount of contraction needed in terms of upstream Froude number and control section shape. These relationships were derived from the critical depth and energy relationships given in the preceding sections. However, rather than being based on a specific control section shape, they are expressed in terms of the control section head-discharge exponent, u, and a *projected area*, A^*, defined below. If the head-discharge relationship for a structure is represented by a power function

$$Q = K_1 y_c^{\,u} \tag{6.18}$$

where K_1 is a constant and u is the head-discharge exponent, then the following relationship can be derived for the contraction needed for critical flow:

$$\mathrm{Fr}_1^{\,2} = \left(\frac{2u-1}{2u}\right)^{2u}\left(\frac{A^*}{A_1}\right)^3\left(\frac{B_1}{B^*}\right)C_v^{\,2} \tag{6.19}$$

where Fr_1 is the Froude number of the approaching flow (Equation 2.18)

$$\mathrm{Fr}_1 = \frac{Q / A_1}{\sqrt{gA_1 / B_1}}$$

and A^* is the area in the control section at a level equivalent to the water surface elevation in the approach channel (i.e., the projected area in the control section of the approaching flow, see Figure 6.11), B_1 is the approach section top width, B^* is the top width in the control section for the projected approaching flow, and C_v is the approach velocity coefficient

$$C_v = \left(\frac{H_1}{h_1}\right)^u \tag{6.20}$$

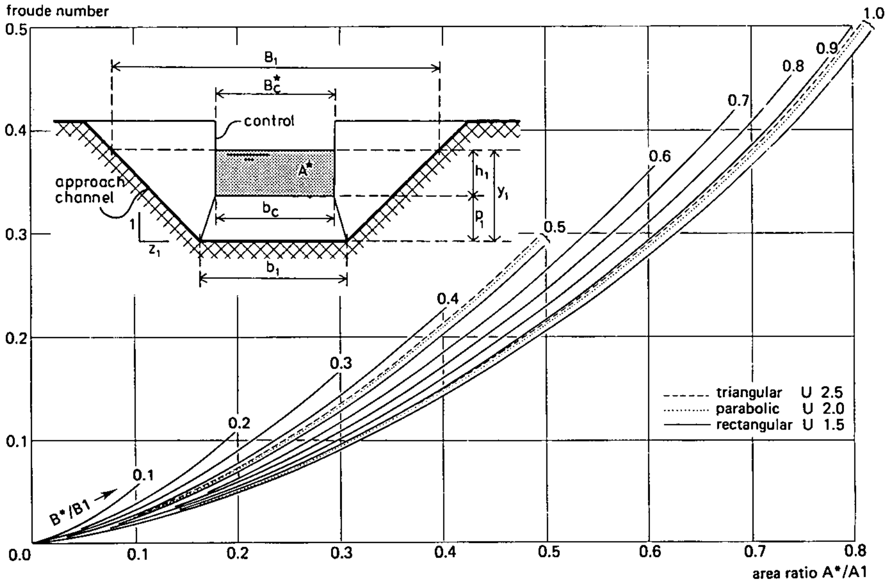


Figure 6.11 Relationship between the ratios A^*/A_1 and B^*/B_1 and the Froude number, Fr_1, with section over gaging station and view of control.

Application of Equation 6.19 can be aided by the following relationships (Equation 2.5):

$$u = 0.5 + \frac{B_c y_c}{A_c} = \frac{B_c H_c}{A_c}$$

from which one can show that

$$\frac{y_c}{H_c} = \frac{2u-1}{2u} \qquad 6.21$$

Clemmens and Bos (1992) also derived a useful relationship for the amount of area contraction needed to produce critical flow:

$$\left(\frac{A^*}{A_1}\right)^2 = (2u-1)\left(\frac{2u}{2u-1}\right)^{2u} \frac{\left(C_v^{\,1/u}-1\right)}{C_v^{\,2}} \qquad 6.22$$

Equations 6.19 and 6.22 were used in FLUME version 3 (Clemmens et al. 1993) as part of the design process in routines that searched for acceptable designs meeting specific design head-loss objectives. WinFlume (Chapter 8) uses Equation 6.19 and 6.22 in a slightly different manner. Rather than searching for a single solution, WinFlume determines the highest and lowest sills that satify submergence and freeboard requirements. This provides a range of alternatives — a more robust approach that also provides more information to the flume designer.

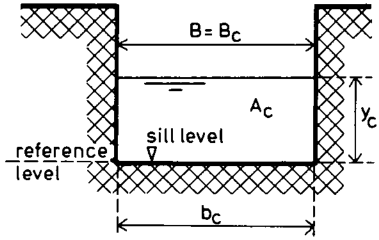


Figure 6.12 Dimensions of a rectangular control section

6.3.4 Head-discharge equations for prismatic control-section shapes

A direct relationship between energy head and discharge can be determined for many prismatic-shaped control sections. Several of the more common ones are provided in this section.

For a rectangular control section in which flow is critical (Figure 6.12), we may write $A_c = b_c y_c$ and $b_c = B_c$ and from Equation 6.15 determine for $\alpha = 1$ that

$$y_c = \frac{2}{3} H_c \tag{6.23}$$

Substitution of this relation and $A_c = b_c\, y_c$ into Equation 6.16 gives, after simplification,

$$Q = \left(\frac{2}{3}\right)^{3/2} (g)^{1/2} b_c H_c^{3/2} \tag{6.24}$$

Following the same procedure that led to Equations 6.23 and 6.24, Bos (1977a, 1978) derived head-discharge equations for commonly used shapes of the control section. Several of the simpler ones are given in Table 6.1. Results for additional shapes are provided in Section 6.4.

Table 6.1 Critical depth and discharge relationships for simple prismatic shapes.

Shape	*Exponent*	*Critical Depth*	*Discharge*
Rectangular	$u = 3/2$	$y_c = \frac{2}{3} H_c$	$Q = \left(\frac{2}{3}\right)^{3/2} (g)^{1/2} b_c H_c^{3/2}$
Parabolic	$u = 4/2$	$y_c = \frac{3}{4} H_c$	$Q = \left(\frac{3}{4} f_c g\right)^{1/2} H_c^{2}$
Triangular	$u = 5/2$	$y_c = \frac{4}{5} H_c$	$Q = \frac{16}{25}\left(\frac{2}{5} g\right)^{1/2} z_c H_c^{5/2}$

6.3.5 Limitations to simplified theory

Limitations to ideal flow assumptions

The head-discharge relationships presented above are based on idealized assumptions such as absence of energy losses between the gaging and control sections, uniform velocity distribution in both sections, and straight, parallel streamlines at the gaging and control sections (see Figure 6.8). Two approaches have been used to account for these effects in determining the head-discharge relationship for flumes and weirs. One is to determine the discharge for an ideal fluid and multiply it by an empirical discharge coefficient, C_d, which is the ratio of the actual to ideal flow (Bos 1985):

$$C_d = \frac{Q}{Q_i} \tag{6.25}$$

The discharge coefficient, C_d, is affected by

- Friction on the channel wall and bottom between the gaging station and the control section,
- The velocity profile in the approach channel and control section, and
- Changes in pressure distribution caused by streamline curvature.

The other approach is to compute the effects directly by using a mathematical theory such as the one presented here. Thus, no empirical discharge coefficient is needed. In either case, the ideal flow is calculated as a base of reference or starting point. These two approaches will be described in detail in later sections.

Limitations to critical flow assumptions

The calibration of flumes and weirs generally assumes that critical flow occurs in the flume throat or weir crest. Several conditions can prevent the expected development of critical flow. The main factors that can cause this are 1) high downstream tailwater levels that create downstream submergence, 2) curvilinear flow caused by a short throat, 3) unstable flow caused by shallow flow over a rough surface, and 4) a side contracted flume that is so wide relative to its length that the side contraction does not influence the flow in the center.

For some of the older flumes (e.g., Parshall flumes), calibrations (in the form of correction factors) were provided for conditions where downstream submergence prevented the occurrence of critical flow. While such calibration is possible, we do not recommend this practice. The flume no longer directly controls the flow in a predictable manner, but simply causes an energy loss. Since this energy loss can be influenced by a wide variety of factors and conditions, especially construction anomalies, calibration becomes rather unreliable and inaccurate. Also, the flumes and weirs developed in this book can allow higher downstream tailwater levels than many of the older flumes without preventing critical flow and influencing the head-discharge relationship, so submerged operation is rarely needed. Design procedures are included to help the user avoid the influences of downstream tailwater.

If the flume throat or weir crest is too short relative to the flow depth, flow will become curvilinear. When this occurs, hydrostatic pressure no longer exists in the flume throat and the equations used here to describe flow no longer exactly apply. The magnitude of this effect is hard to judge, but laboratory calibrations appear to drift away from theory when the upstream depth is more than half the throat length. The error in discharge is typically less than 5% when the depth equals the throat length and from there, increases rapidly with depth. This is discussed in more detail in Section 6.4.2.

When the throat is extremely long relative to the depth, the frictional effects start to dominate the flow and reliable prediction of discharge becomes extremely difficult. This condition is characterized by undular flow in the throat, with the water surface looking like a long series of sine waves and the flow oscillating between subcritical and supercritical. Although accurate predictions of flow have been achieved with a throat length to upstream head ratio of 20:1 ($H_1/L = 0.05$), the rating is very sensitive to friction. Such a calibration is likely to shift substantially if there are changes in roughness over time. As a result, we suggest limiting the throat length to upstream head ratio to 15:1, or $H_1/L = 0.07$.

In general, the theory presented here assumes that the water levels in the approach section to the flume and in the flume throat are level across the width so that flow is essentially one-dimensional. Within limits, some differences in water level across the width will not cause significant error in the assumed relationship. Velocity distribution coefficients take into account some of the transverse variations. However, in situations where the flume is extremely wide compared to the flow depth and length, large relative differences are possible in water surface elevation, velocity, etc. across the width of the flume. Consider a wide flume with a flat bottom (i.e., no raised sill in the throat) and symmetric side contractions. The side contractions can be considered disturbances at the wall. These disturbances will travel across the width of the throat at some angle to the main direction of flow. If the disturbances do not reach the centerline of the throat before reaching the end of the throat, the flow in the center will not be influenced in any way by the contraction at the wall. In this case, critical flow does not really occur there, even though theory based on "average" velocity would suggest that it should.

Critical depth in long-throated flumes generally occurs at a distance of about two-thirds of the crest length downstream from the upstream edge of the throat. At this cross-section the Froude number is unity, and since the Froude number expresses the ratios of the flow velocity and the celerity of a shallow-water wave, any disturbance of the flow at a section with a Froude number of 1 will propagate across the channel and downstream at a 45° angle to the flow direction, and cannot propagate upstream. At other Froude numbers, the disturbance will propagate across the channel at an angle $\theta = \tan^{-1}(\mathrm{Fr})$ from the flow direction. If we consider a throat section with length L and width b_c, and conservatively assume that the flow throughout the throat section has a Froude number of 1, then a crest length of $L \geq 0.75 b_c$ would be required to allow a wave to propagate from the sidewall to the centerline of the channel within a flow distance of $^2/_3 L$. In limited laboratory tests of flumes that are solely width-contracted, we have found that $L = 0.25b_c$ does not produce an accurate rating, but an accurate rating is obtained with $L = 2b_c$. At the present time the WinFlume computer program will warn the designer if a width-contracted design has a crest length-to-width ratio less than 2. The length-to-width ratio is evaluated using the average of the throat widths at the sill elevation and at the elevation corresponding to h_1. Future research may allow refinement of this limitation. In the meantime, if this length-to-width ratio cannot be provided, a significant bottom contraction should be included.

6.4 Head-Discharge Equations Based on Experimentation

The head-discharge equations based on ideal flow, such as those shown in Table 6.1, must be corrected for energy losses, velocity distributions, and streamline curvature by the introduction of a discharge coefficient C_d. Furthermore, in an open channel it is not possible to measure the energy head H_1 directly, and it is therefore common practice to relate the flow rate to the upstream sill-referenced water level (or piezometric head) by using the velocity coefficient, C_v, defined in Equation 6.20. The resulting head-discharge equation for a rectangular channel is

$$Q = C_d C_v \frac{2}{3}\left(\frac{2}{3}g\right)^{1/2} b_c h_1^{3/2} \qquad 6.26$$

The general idea is to measure the upstream water level, experimentally determine a value for C_d, estimate C_v, and compute the discharge. This approach has been applied to a variety of shapes, as shown in Figure 6.13 (Bos 1977a, Bos 1978 and Clemmens et al. 1984b). Tables 6.2 and 6.3 are used to calculate the critical depth, y_c, for trapezoidal and circular controls, respectively. Tables 6.3 and 6.4 provide useful values for computing the head-discharge relationships for circular controls and controls made from placing a bottom sill in a circular section (Clemmens et al. 1984a). Use of these relationships requires determination of C_d and in most cases C_v.

6.4.1 Effects of H_1/L on the value of the discharge coefficient, C_d

As already stated, the discharge coefficient corrects for such phenomena as the energy loss between the gaging and control sections, the non-uniformity of the velocity distribution, and the streamline curvature in these two sections. These phenomena are closely related to the value of the ratio H_1/L. Let us compare the Figures 6.14A and B. In the upper figure, the head is small with respect to the length, L, of the sill. The thin layer of water above the sill is very close to the rough boundary, and as a result, energy lost through friction is a relatively large part of H_1. In Figure 6.14B, the energy loss due to friction is a smaller percentage of H_1. To correct for this relative difference in friction losses, the C_d value of the weir of Figure 6.14A with a ratio $H_1/L = 0.1$ must be lower than that of Figure 6.14B having an H_1/L ratio of 0.33.

Comparison of Figures 6.14B and C illustrates why there is also a significant difference in their C_d values. Both weirs flow under the same head h_1 and have equal

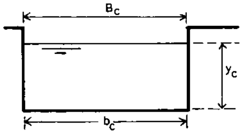
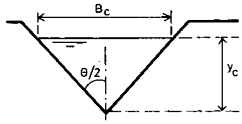
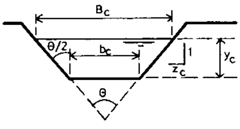
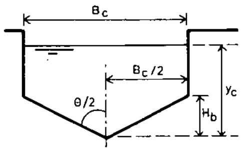
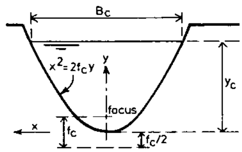
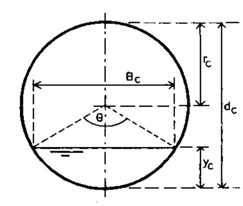
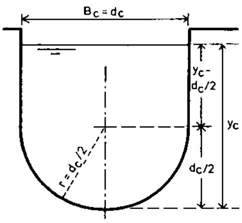
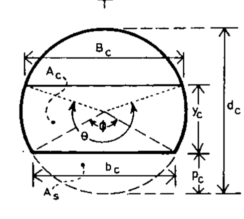
SHAPE OF CONTROL SECTION	HEAD–DISCHARGE EQ. TO BE USED	HOW TO FIND THE y_c - VALUE
	$Q = C_d C_v \frac{2}{3} (\frac{2}{3} g)^{1/2} \cdot b_c h_1^{3/2}$	$y_c = \frac{2}{3} H_1$
	$Q = C_d C_v \frac{16}{25} (\frac{2}{5} g)^{1/2} \tan\frac{\theta}{2}\, h_1^{5/2}$	$y_c = \frac{4}{5} H_1$
	$Q = C_d [b_c y_c + z_c y_c^2][2g(H_1 - y_c)]^{1/2}$	Use Table 6.2
	If $H_1 \leqslant 1.25 H_b$ $Q = C_d C_v \frac{16}{25} (\frac{2}{5} g)^{1/2} \tan\frac{\theta}{2}\, h_1^{5/2}$ If $H_1 \geqslant 1.25 H_b$ $Q = C_d C_v \frac{2}{3} (\frac{2}{3} g)^{1/2} B_c (h_1 - \frac{1}{2} H_b)^{3/2}$	$y_c = \frac{4}{5} H_1$ $y_c = \frac{2}{3} H_1 + \frac{1}{6} H_b$
	$Q = C_d C_v (\frac{3}{4} f_c g)^{1/2} h_1^2$	$y_c = \frac{3}{4} H_1$
	$Q = C_d d_c^{5/2} \sqrt{g}\, [f(\theta)]$ Use Table 6.3 to find $f(\theta)$	Use Table 6.3
	If $H_1 \leqslant 0.70\, d_c$ $Q = C_d d_c^{5/2} \sqrt{g}\, [f(\theta)]$ Use Table 6.3 to find $f(\theta)$ If $H_1 \geqslant 0.70\, d_c$ $Q = C_d C_v 2 d_c (2g)^{1/2} (\frac{1}{3} h_1 - 0.0358\, d_c)^{3/2}$	Use Table 6.3 $y_c = \frac{1}{2} H_1 + 0.152\, d_c$
	$Q = C_d d_c^{5/2} \sqrt{g}\, [f(\phi, \theta)]$ Use Table 6.4 to find $f(\phi, \theta)$	y_c is variable

Figure 6.13 Head-discharge relationships for long-throated flumes (from Bos 1977a; and Clemmens, et al., 1984b).

Table 6.2 Values of the ratio y_c/H_1 as a function of z_c and H_1/b_c for trapezoidal control section.

$\frac{H_1}{b_c}$	Side slope of channel, ratio of horizontal to vertical (z_c)									
	Vertical	0,25:1	0,50:1	0.75:1	1:1	1.5:1	2:1	2.5:1	3:1	4:1
0.00	0.667	0.667	0.667	0.667	0.667	0.667	0.667	0.667	0.667	0.667
0.01	0.667	0.667	0.667	0.668	0.668	0.669	0.670	0.670	0.671	0.672
0.02	0.667	0.667	0.668	0.669	0.670	0.671	0.672	0.674	0.675	0.678
0.03	0.667	0.668	0.669	0.670	0.671	0.673	0.675	0.677	0.679	0.683
0.04	0.667	0.668	0.670	0.671	0.672	0.675	0.677	0.680	0.683	0.687
0.05	0.667	0.668	0.670	0.672	0.674	0.677	0.680	0.683	0.686	0.692
0.06	0.667	0.669	0.671	0.673	0.675	0.679	0.683	0.686	0.690	0.696
0.07	0.667	0.669	0.672	0.674	0.676	0.681	0.685	0.689	0.693	0.699
0.08	0.667	0.670	0.672	0.675	0.678	0.683	0.687	0.692	0.696	0.703
0.09	0.667	0.670	0.673	0.676	0.679	0.684	0.690	0.695	0.698	0.706
0.10	0.667	0.670	0.674	0.677	0.680	0.686	0.692	0.697	0.701	0.709
0.12	0.667	0.671	0.675	0.679	0.684	0.690	0.692	0.701	0.706	0.715
0.14	0.667	0.672	0.676	0.681	0.686	0.693	0.699	0.705	0.711	0.720
0.16	0.667	0.672	0.678	0.683	0.678	0.696	0.703	0.709	0.715	0.725
0.18	0.667	0.673	0.679	0.684	0.690	0.698	0.706	0.713	0.719	0.729
0.20	0.667	0.674	0.680	0.686	0.692	0.701	0.709	0.717	0.723	0.733
0.22	0.667	0.674	0.681	0.688	0.694	0.704	0.712	0.720	0.726	0.736
0.24	0.667	0.675	0.683	0.689	0.696	0.706	0.715	0.723	0.729	0.739
0.26	0.667	0.676	0.684	0.691	0.698	0.709	0.718	0.725	0.732	0.742
0.28	0.667	0.676	0.685	0.693	0.699	0.711	0.720	0.728	0.734	0.744
0.30	0.667	0.677	0.686	0.694	0.701	0.713	0.723	0.730	0.737	0.747
0.32	0.667	0.678	0.687	0.696	0.703	0.715	0.725	0.733	0.739	0.749
0.34	0.667	0.678	0.689	0.697	0.705	0.717	0.727	0.735	0.741	0.751
0.36	0.667	0.679	0.690	0.699	0.706	0.719	0.729	0.737	0.743	0.752
0.38	0.667	0.680	0.691	0.700	0.708	0.721	0.731	0.738	0.745	0.754
0.40	0.667	0.680	0.692	0.701	0.709	0.723	0.733	0.740	0.747	0.756
0.42	0.667	0.681	0.693	0.703	0.711	0.725	0.734	0.742	0.748	0.757
0.44	0.667	0.681	0.694	0.704	0.712	0.727	0.736	0.744	0.750	0.759
0.46	0.667	0.682	0.695	0.705	0.714	0.728	0.737	0.745	0.751	0.760
0.48	0.667	0.683	0.696	0.706	0.715	0.729	0.739	0.747	0.752	0.761
0.5	0.667	0.683	0.697	0.708	0.717	0.730	0.740	0.748	0.754	0.762
0.6	0.667	0.686	0.701	0.713	0.723	0.737	0.747	0.754	0.759	0.767
0.7	0.667	0.688	0.706	0.718	0.728	0.742	0.752	0.758	0.764	0.771
0.8	0.667	0.692	0.709	0.723	0.732	0.746	0.756	0.762	0.767	0.774
0.9	0.667	0.694	0.713	0.727	0.737	0.750	0.759	0.766	0.770	0.776
1.0	0.667	0.697	0.717	0.730	0.740	0.754	0.762	0.768	0.773	0.778
1.2	0.667	0.701	0.723	0.737	0.747	0.759	0.767	0.772	0.776	0.782
1.4	0.667	0.706	0.729	0.742	0.752	0.764	0.771	0.776	0.779	0.784
1.6	0.667	0.709	0.733	0.747	0.756	0.767	0.774	0.778	0.781	0.786
1.8	0.667	0.713	0.737	0.750	0.759	0.770	0.776	0.781	0.783	0.787
2	0.667	0.717	0.740	0.754.	0.762	0.773	0.778	0.782	0.785	0.788
3	0.667	0.730	0.753	0.766	0.773	0.781	0.785	0.787	0.790	0.792
4	0.667	0.740	0.762	0.773	0.778	0.785	0.788	0.790	0.792	0.794
5	0.667	0.748	0.768	0.777	0.782	0.788	0.791	0.792	0.794	0.795
10	0.667	0.768	0.782	0.788	0.791	0.794	0.795	0.796	0.797	0.798
∞		0.800	0.800	0.800	0.800	0.800	0.800	0.800	0.800	0.800

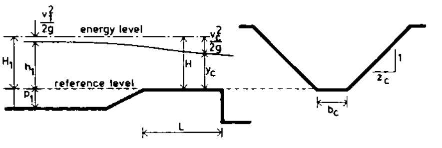


Table 6.3 Ratios for determining the discharge, Q, of broad-crested weirs and long-throated flumes with circular control sections.

$\frac{y_c}{d_c}$	$\frac{v_c^2}{2gd_c}$	$\frac{H_1}{d_c}$	$\frac{A_c}{d_c^2}$	$\frac{y_c}{H_1}$	$f(\theta)$	$\frac{y_d}{d_c}$	$\frac{y_c^2}{2gd_c}$	$\frac{H_1}{d_c}$	$\frac{A_c}{d_c^2}$	$\frac{y_c}{H_1}$	$f(\theta)$
0.01	0.0033	0.0133	0.0013	0.752	0.0001	0.51	0.2014	0.7114	0.4027	0.717	0.2556
0.02	0.0067	0.0267	0.0037	0.749	0.0004	0.52	0.2065	0.7265	0.4127	0.716	0.2652
0.03	0.0101	0.0401	0.0069	0.749	0.0010	0.53	0.2117	0.7417	0.4227	0.715	0.2750
0.04	0.0134	0.0534	0.0105	0.749	0.0017	0.54	0.2170	0.7570	0.4327	0.713	0.2851
0.05	0.0168	0.0668	0.0147	0.748	0.0027	0.55	0.2224	0.7724	0.4426	0.712	0.2952
0.06	0.0203	0.0803	0.0192	0.748	0.0039	0.56	0.2279	0.7879	0.4526	0.711	0.3056
0.07	0.0237	0.0937	0.0242	0.747	0.0053	0.57	0.2335	0.8035	0.4625	0.709	0.3161
0.08	0.0271	0.1071	0.0294	0.747	0.0068	0.58	0.2393	0.8193	0.4724	0.708	0.3268
0.09	0.0306	0.1206	0.0350	0.746	0.0087	0.59	0.2451	0.8351	0.4822	0.707	0.3376
0.10	0.0341	0.1341	0.0409	0.746	0.0107	0.60	0.2511	0.8511	0.4920	0.705	0.3487
0.11	0.0376	0.1476	0.0470	0.745	0.0129	0.61	0.2572	0.8672	0.5018	0.703	0.3599
0.12	0.0411	0.1611	0.0534	0.745	0.0153	0.62	0.2635	0.8835	0.5115	0.702	0.3713
0.13	0.0446	0.1746	0.0600	0.745	0.0179	0.63	0.2699	0.8999	0.5212	0.700	0.3829
0.14	0.0482	0.1882	0.0688	0.744	0.0214	0.64	0.2765	0.9165	0.5308	0.698	0.3947
0.15	0.0517	0.2017	0.0739	0.744	0.0238	0.65	0.2833	0.9333	0.5404	0.696	0.4068
0.16	0.0553	0.2153	0.0811	0.743	0.0270	0.66	0.2902	0.9502	0.5499	0.695	0.4189
0.17	0.0589	0.2289	0.0885	0.743	0.0304	0.67	0.2974	0.9674	0.5594	0.693	0.4314
0.18	0.0626	0.2426	0.0961	0.742	0.0340	0.68	0.3048	0.9848	0.5687	0.691	0.4440
0.19.	0.0662	0.2562	0.1039	0.742	0.0378	0.69	0.3125	1.0025	0.5780	0.688	0.4569
0.20	0.0699	0.2699	0.1118	0.741	0.0418	0.70	0.3204	1.0204	0.5872	0.686	0.4701
0:21	0.0736	0.2836	0.1199	0.740	0.0460	0.71	0.3286	1.0386	0.5964	0.684	0.4835
0 22	0.0773	0.2973	0.1281	0.740	0.0504	0.72	0.3371	1.0571	0.6054	0.681	0.4971
0.23	0.0811	0.3111	0.1365	0.739	0.0550	0.73	0.3459	1.0759	0.6143	0.679	0.5109
0.24	0.0848	0.3248	0.1449	0.739	0.0597	0.74	0.3552	1.0952	0.6231	0.676	0.5252
0.25	0.0887	0.3387	0.1535	0.738	0.0647	0.75	0.3648	1.1148	0.6319	0.673	0.5397
0.26	0.0925	0.3525	0.1623	0.738	0.0698	0.76	0.3749	1.1349	0.6405	0.670	0.5546
0.27	0.0963	0.3663	0.1711	0.737	0.0751	0.77	0.3855	1.1555	0.6489	0.666	0.5698
0.28	0.1002	0.3802	0.1800	0.736	0.0806	0.78	0.3967	1.1767	0.6573	0.663	0.5855
0.29	0.1042	0.3942	0.1890	0.736	0.0863	0.79	0.4085	1.1985	0.6655	0.659	0.6015
0.30	0.1081	0.4081	0.1982	0.735	0.0922	0.80	0.4210	1.2210	0.6735	0.655	0.6180
0.31	0.1121	0.4221	0.2074	0.734	0.0982	0.81	0.4343	1.2443	0.6815	0.651	0.6351
0.32	0.1161	0.4361	0.2167	0.734	0.1044	0.82	0.4485	1.2685	0.6893	0.646	0.6528
0.33	0.1202	0.4502	0.2260	0.733	0.1108	0.83	0.4638	1.2938	0.6969	0.641	0.6712
0.34	0.1243	0.4643	0.2355	0.732	0.1174	0.84	0.4803	1.3203	0.7043	0.636	0.6903
0.35	0.1284	0.4784	0.2450	0.732	0.1289	0.85	0.4982	1.3482	0.7115	0.630	0.7102
0.36	0.1326	0.4926	0.2546	0.731	0.1311	0.86	0.5177	1.3777	0.7186	0.624	0.7312
0.37	0.1368	0.5068	0.2642	0.730	0.1382	0.87	0.5392	1.4092	0.7254	0.617	0.7533
0.38	0.1411	0.5211	0.2739	0.729	0.1455	0.88	0.5632	1.4432	0.7320	0.610	0.7769
0.39	0.1454	0.5354	0.2836	0.728	0.1529	0.89	0.5900	1.4800	0.7384	0.601	0.8021
0.40	0.1497	0.5497	0.2934	0.728	0.1605	0.90	0.6204	1.5204	0.7445	0.592	0.8293
0.41	0.1541	0.5641	0.3032	0.727	0.1683	0.91	0.6555	1.5655	0.7504	0.581	0.8592
0.42	0.1586	0.5786	0.3130	0.726	0.1763	0.92	0.6966	1.6166	0.7560	0.569	0.8923
0.43	0.1631	0.5931	0.3229	0.725	0.1844	0.93	0.7459	1.6759	0.7612	0.555	0.9297
0.44	0.1676	0.6076	0.3328	0.724	0.1927	0.94	0.8065	1.7465	0.7662	0.538	0.9731
0.45	0.1723	0.6223	0.3428	0.723	0.2012	0.95	0.8841	1.8341	0.7707	0.518	1.0248
0.46	0.1769	0.6369	0.3527	0.722	0.2098						
0.47	0.1817	0.6517	0.3627	0.721	0.2186						
0.48	0.1865	0.6665	0.3727	0.720	0.2276						
0.49	0.1914	0.6814	0.3827	0.719	0.2368						
0.50	0.1964	0.6964	0.3927	0.718	0.2461						

Note: $f(\theta) = (A_c/d_c^2)\{2(H_1/d_c - y_c/d_c)\}^{1/2}$

$= (\theta - \sin\theta)^{1.5}/\{8[8\sin(\theta/2)]^{1/2}\}$

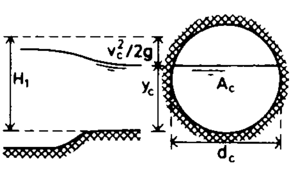
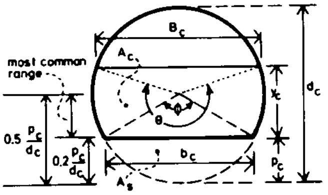


Table 6.4 Ratios for determining the discharge, Q, of broad-crested weirs in circular pipes.[a]

values of $f(\phi,\theta) = \dfrac{(\theta - \phi + \sin\phi - \sin\theta)^{1.5}}{8(8\sin\frac{1}{2}\theta)^{0.5}}$

for values of p_c/d_c

$\dfrac{p_c + H_1}{d_1}$	0.15	0.20	0.25	0.30	0.35	0.40	0.45	0.50
0.16	0.0004							
0.17	0.0011							
0.18	0.0021							
0.19	0.0032							
0.20	0.0045							
0.21	0.0060	0.0004						
0.22	0.0076	0.0012						
0.23	0.0094	0.0023						
0.24	0.0113	0.0036						
0.25	0.0133	0.0050						
0.26	0.0155	0.0066	0.0005					
0.27	0.0177	0.0084	0.0013					
0.28	0.0201	0.0103	0.0025					
0.29	0.0226	0.0124	0.0038					
0.30	0.0252	0.0145	0.0054					
0.31	0.0280	0.0169	0.0071	0.0005				
0.32	0.0308	0.0193	0.0090	0.0014				
0.33	0.0337	0.0219	0.0110	0.0026				
0.34	0.0368	0.0245	0.0132	0.0040				
0.35	0.0399	0.0273	0.0155	0.0057				
0.36	0.0432	0.0302	0.0179	0.0075	0.0005			
0.37	0.0465	0.0332	0.0205	0.0094	0.0015			
0.38	0.0500	0.0363	0.0232	0.0115	0.0027			
0.39	0.0535	0.0396	0.0260	0.0138	0.0042			
0.40	0.0571	0.0429	0.0289	0.0162	0.0059			
0.41	0.0609	0.0463	0.0320	0.0187	0.0077	0.0005		
0.42	0.0647	0.0498	0.0351	0.0214	0.0097	0.0015		
0.43	0.0686	0.0534	0.0383	0.0242	0.0119	0.0028		
0.44	0.0726	0.0571	0.0417	0.0271	0.0143	0.0043		
0.45	0.0767	0.0609	0.0451	0.0301	0.0167	0.0060		
0.46	0.0809	0.0648	0.0487	0.0332	0.0193	0.0079	0.0005	
0.47	0.0851	0.0688	0.0523	0.0365	0.0220	0.0100	0.0015	
0.48	0.0895	0.0729	0.0561	0.0398	0.0249	0.0122	0.0028	
0.49	0.0939	0.0770	0.0599	0.0432	0.0279	0.0145	0.0043	
0.50	0.0984	0.0813	0.0638	0.0468	0.0309	0.0170	0.0061	
0.51	0.1030	0.0856	0.0678	0.0504	0.0341	0.0197	0.0080	0.0005
0.52	0.1076	0.0900	0.0719	0.0541	0.0374	0.0224	0.0101	0.0015
0.53	0.1124	0.0945	0.0761	0.0579	0.0408	0.0253	0.0123	0.0028
0.54	0.1172	0.0990	0.0803	0.0618	0.0443	0.0283	0.0147	0.0044
0.55	0.1221	0.1037	0.0847	0.0658	0.0479	0.0314	0.0172	0.0061
0.56	0.1270	0.1084	0.0891	0.0699	0.0515	0.0346	0.0198	0.0080
0.57	0.1320	0.1132	0.0936	0.0741	0.0553	0.0379	0.0226	0.0101
0.58	0.1372	0.1180	0.0981	0.0783	0.0592	0.0413	0.0255	0.0123
0.59	0.1423	0.1230	0.1028	0.0826	0.0631	0.0448	0.0285	0.0147
0.60	0.1476	0.1280	0.1075	0.0870	0.0671	0.0484	0.0316	0.0172
0.62[b]		0.1382	0.1172	0.0960	0.0754	0.0559	0.0381	0.0225
0.64		0.1486	0.1271	0.1053	0.0840	0.0637	0.0449	0.0283
0.66		0.1593	0.1373	0.1149	0.0929	0.0718	0.0522	0.0346
0.68		0.1703	0.1477	0.1247	0.1020	0.0802	0.0597	0.0412
0.70		0.1815	0.1584	0.1348	0.1114	0.0888	0.0676	0.0481
0.72		0.1929	0.1692	0.1451	0.1211	0.0978	0.0757	0.0554
0.74		0.2045	0.1804	0.1556	0.1310	0.1070	0.0841	0.0629
0.76		0.2163	0.1917	0.1663	0.1411	0.1164	0.0928	0.0707
0.78		0.2283	0.2031	0.1773	0.1514	0.1260	0.1016	0.0788
0.80		0.2405	0.2148	0.1884	0.1618	0.1358	0.1107	0.0870
0.82		0.2528	0.2267	0.1997	0.1725	0.1458	0.1200	0.0955
0.84		0.2653	0.2386	0.2111	0.1833	0.1559	0.1294	0.1042
0.86		0.2780	0.2508	0.2227	0.1943	0.1662	0.1390	0.1130
0.88		0.2907	0.2630	0.2344	0.2054	0.1767	0.1487	0.1220
0.90		0.3036	0.2754	0.2462	0.2166	0.1872	0.1586	0.1311
0.92		0.3166	0.2879	0.2581	0.2279	0.1979	0.1686	0.1404
0.94		0.3297	0.3005	0.2701	0.2394	0.2087		
0.96		0.3428	0.3131	0.2823	0.2509			
0.98		0.3561	0.3259	0.2944				
1.00		0.3694	0.3387					



[a] $C_d = 1.0$; $\alpha_c = 1.0$; $H_1 = H_c$

[b] Change in increment

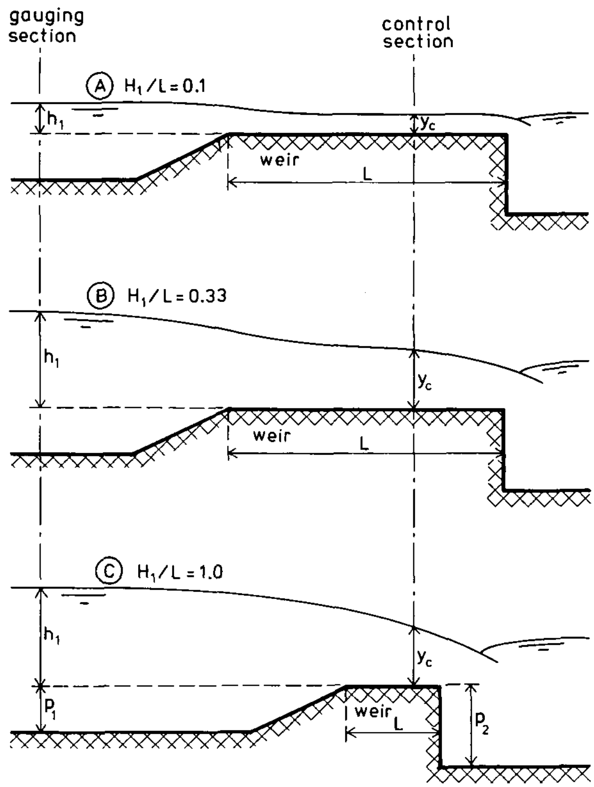


Figure 6.14 Water surface profiles over weir crests.

values of y_c. Because of the difference in H_1/L ratios, however, the pressure distribution at the control section of Figure 6.14B is hydrostatic, as shown in Figure 6.4, while, because of streamline curvature, Figure 6.14C has a modified pressure distribution similar to that of Figure 6.6. Because of the related difference in velocity distribution, the weir flowing with an H_1/L ratio of 1.0 has a much higher C_d value than the weir in Figure 6.14B, for which $H_1/L = 0.33$ (see Figure 6.15).

6.4.2 Values and accuracy of the discharge coefficient; C_d

Values of the discharge coefficient, C_d, are shown in Figure 6.16 as a function of H_1/L. The range of application for this figure is

$$0.1 \leq H_1 / L \leq 1.0 \qquad 6.27$$

The most important reasons for these limits are that for values of $H_1/L < 0.1$, minor changes in the boundary roughness of the weir sill cause an increasingly large variation of the C_d value; for values of $H_1/L > 1.0$, the streamline curvature and non-hydrostatic pressure distribution at the end of the throat exert an increasing influence that extends

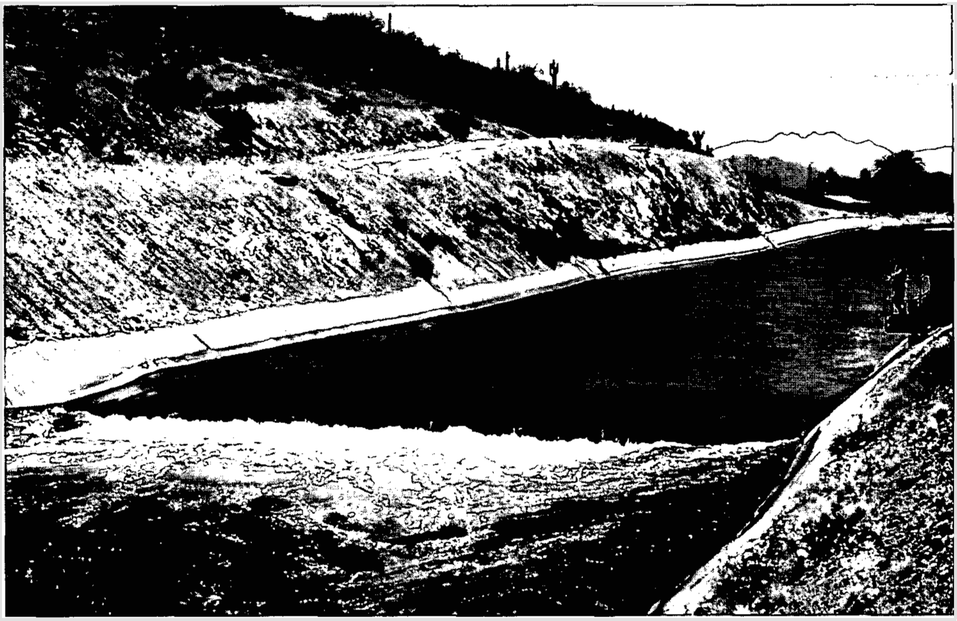


Figure 6.15 Typical water surface profile over a broad-crested weir with $H_1/L \cong 0.3$ (Arizona).

back to the control section. This makes the discharge coefficient sensitive to the slope of the downstream transition and other factors that affect the streamline curvature at the control section. One such factor is a high tailwater level, so, as a result, at values of $H_1/L > 1.0$, the allowable tailwater level is reduced, and the required head loss is increased. For more details on this head-loss requirement, see Section 6.6.

Near both the upper and lower H_1/L limits, the error in the empirical C_d value is $X_c \approx \pm 5\%$ (95% confidence level from laboratory and field data, Bos 1989). Between these limits, the error is slightly lower and can be estimated from the equation

$$X_c = \pm\left(3\left|\frac{H_1}{L} - 0.55\right|^{1.5} + 4\right)\% \qquad 6.28$$

Figure 6.14 shows the control section at a constant distance of $L/3$ from the end of the weir crest. In reality, however, flow will become critical at a variable location along the horizontal crest. For low H_1/L ratios, it is upstream of the shown location; for high H_1/L ratios, it is slightly downstream. Additionally, if the weir crest or flume throat is not horizontal in the direction of flow, the average location of the control section will differ dramatically from that shown in Figure 6.14. A downward sloping crest causes the control section to move to the upstream end of the crest, while an upward slope moves the control to the downstream edge of the crest. In both cases, the control section is in an area with streamline curvature, and this produces a higher C_d value. A slope of 2 degrees may cause a positive error in C_d of up to 5% (Bos 1989). Because it is difficult to correct for larger slopes, we recommend leveling the crest or throat rather than correcting C_d for these slopes.

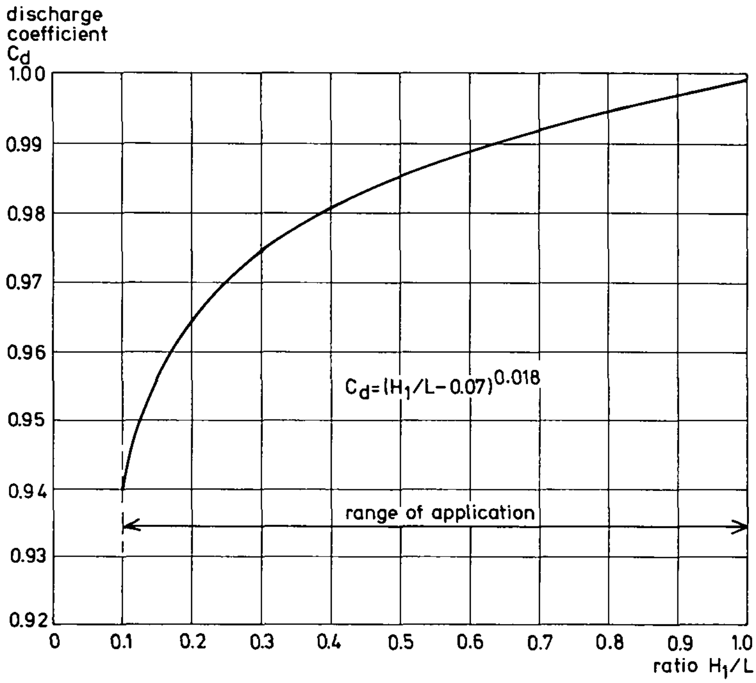


Figure 6.16 C_d as a function of H_1/L.

Anything that causes the control section to move further downstream (e.g., high H_1/L ratio or an upward sloping throat) will place the control section in a region of increasing streamline curvature and makes C_d more sensitive to changes in tailwater level that affect this streamline curvature. This can reduce the modular limit dramatically, so that the structure requires significantly more head loss to operate in the modular flow range. For example, an upward slope of 2 degrees in the throat section causes the modular limit to drop from about 0.70 to 0.30 (Bos 1989).

6.4.3 Values of the approach velocity coefficient, C_v

The head-discharge equations of Section 6.3.4 were developed in terms of energy head, assuming $H_c = H_1$, but the upstream water level, h_1, is much easier to measure. If one measures h_1 and uses the head discharge equation for H_1 , the velocity head, $\alpha_1 v_1^2/2g$, is being neglected. The approach velocity coefficient, C_v = $(H_1/h_1)u$, corrects for this difference, the magnitude of which can be seen in Figure 6.17.

Because the discharge is mainly determined by the area of flow at the control section (Equation 6.16) and the related approach velocity is determined by the area of flow at the gaging station, it was found to be convenient to correlate C_v to the area ratio $(\alpha_1)^{0.5} C_d A^*/A_1$, (Bos 1989). In this ratio, A^* is the imaginary projected area of flow at the control section if the water depth at the control were equal to h_1 (see Figure

6.11). Values of C_v as a function of the area ratio $(\alpha_1)^{0.5} C_d A^*/A_1$ are shown in Figure 6.17 for various control shapes. Because of the use of A^* in the area ratio, the C_v value is almost the same for all control shapes.

6.4.4 Calculating head-discharge relationship based on experimentation

Computing discharge when head is known

When only the upstream sill-referenced head is known, determination of discharge is an iterative process, even when using C_d and C_v, although convergence is rapid (and often ignored). Start by assuming that C_d and C_v are both unity and compute the discharge with the appropriate equation from Figure 6.13. With this discharge, compute the velocity head and add it to h_1, giving the energy head H_1 (from Equation 6.10 with $\alpha_1 = 1.0$). Compute the ratio H_1/L and read the value of C_d from Figure 6.16. Compute the ratio $(\alpha_1)^{0.5} C_d A^*/A_1$ and read the value of C_v from Figure 6.17. Next, recompute the discharge with the new values for C_d and C_v. Repeat this process until the discharge converges. The procedure converges rapidly since C_d and C_v change very little with small changes in discharge. Because C_v is weakly dependent on the head-discharge exponent, u, it is necessary to assume a value for u based on the throat section shape. For odd shapes, the procedure should be repeated at several heads so that a more refined value for u can be determined. This should have a negligible effect on the result, since the u-value has only a small influence on C_v.

Computing head when discharge is known

When discharge is known, the head-discharge equations based on experimentation can only be used to determine head by an iterative procedure. The above procedure is repeated by adjusting the head until the calculated discharge matches the known discharge. The procedure would start with a guess for h_1. The critical-flow equations

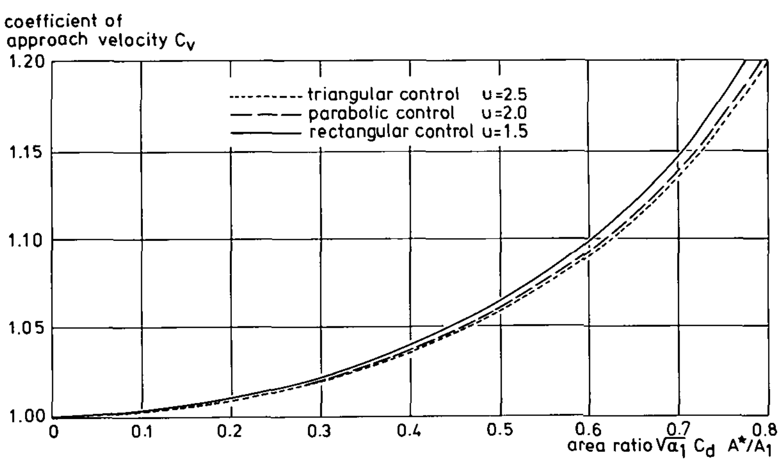


Figure 6.17 C_v as a function of the ratio $\sqrt{\alpha_1} C_d A^*/A_1$.

could be used to determine the total energy head, from which an initial estimate for h_1 could be made.

6.4.5 Adjustments to rating tables with C_v

Rating tables are often given for a particular throat cross-section and a particular approach cross-section, for example a rectangular throat and a rectangular approach, both with the same width, or a trapezoidal flume made by adding a bottom contraction in a trapezoidal channel. However, there may be situations when site conditions call for use of an approach channel section other than those assumed in the development of the rating tables. For example, a trapezoidal (or rough-form earthen) approach to a rectangular weir, or a nonstandard approach bottom or water level for a movable rectangular weir. In most cases, these alterations can be accommodated through adjustments in C_v. Tables R.3 and R.4, Appendix 4, give values for Q versus h_1 for rectangular-throated flumes, based on the relationship expressed by Equation 6.26, but with C_d and C_v computed from theory. For a given weir width and h_1 value, only C_d and C_v can change. It can be assumed that C_d does not change with minor changes in the approach channel velocity head since H_1/L changes very little. Thus we need only evaluate C_v to adjust the rating table values. The new discharge for a given value of h_1 can be found from

$$Q_{new} = Q_{rate} \frac{C_{v\,new}}{C_{v\,rate}} \qquad 6.29$$

where Q_{rate} is the value obtained from the printed rating table, $C_{v\,rate}$ is the velocity coefficient for the approach section assumed in developing the rating table, and $C_{v\,new}$ is the velocity coefficient for the actual or design approach section. Values of C_v can be obtained for each value of h_1 from Figure 6.17 or Table 6.5.

Table 6.5 Approach velocity coefficient, C_v, for rectangular broad-crested weirs and long-throated flumes ($u = 1.5$) as a function of $\sqrt{\alpha_1} C_d A^*/A_1$.

$\sqrt{\alpha_1}\dfrac{C_d A^*}{A_1}$	0.00	0.01	0.02	0.03	0.04[a]	0.05	0.06	0.07	0.08	0.09
0.0	1.000	1.000	1.000	1.000	1.000	1.001	1.001	1.001	1.001	1.002
0.1	1.002	1.003	1.003	1.004	1.004	1.005	1.006	1.007	1.007	1.008
0.2	1.009	1.010	1.011	1.012	1.013	1.014	1.016	1.017	1.018	1.019
0.3[a]	1.021	1.022	1.024	1.026	1.027	1.029	1.031	1.033	1.035	1.037
0.4	1.039	1.041	1.043	1.045	1.048	1.050	1.053	1.055	1.058	1.061
0.5	1.063	1.066	1.070	1.073	1.076	1.079	1.083	1.086	1.090	1.094
0.6	1.098	1.102	1.106	1.111	1.115	1.120	1.125	1.130	1.135	1.141
0.7	1.146	1.152	1.159	1.165	1.172	1.179	1.186	1.193	1.201	1.210

a) Example: If $\sqrt{\alpha_1}\dfrac{C_d A^*}{A_1} = 0.34$, then $C_v = 1.027$

Example

Given: A weir with rectangular control section with b_c = 1.00 m, z_c= 0 and L = 0.60 m is placed at p_1 = 0.30 m in a trapezoidal canal with b_1 = 0.60 m and z_1 = 1.0.

Question: What is the discharge if the upstream sill-referenced head is h_1 = 0.25 m?

Answer: Step 1

Find the velocity coefficient for the rectangular weir with a rectangular approach that was used to calculate Table R.3. For this approach channel b_1 = 1.0 m and z_1 = 0 so that for the "rate" situation, we know:

$y_1 = h_1 + p_1 = 0.55$ m

$A_1 = y_1(b_1 + z_1 y_1) = y_1 b_1 = 0.55$ m^2

$A^* = h_1(b_c + z_c h_1) = h_1 b_c = 0.25$ m^2

From Table R.3, for h_1 = 0.25 m, we read that q = 220 liters/s per meter, or Q_{rate} = 0.220 m^3/s because in this example b_c = 1.0 m.

For the "rate" situation, we can calculate

$$v_1 = \frac{Q_{rate}}{A_1} = \frac{0.22}{0.55} = 0.40 \text{ m / s}$$

H_1/L = 0.323 (note: h_1/L gives 0.313, which is close enough). From Figure 6.16 we read that C_d = 0.976, and subsequently calculate that (with α_1 = 1.04)

$$\sqrt{\alpha_1} C_d A^* / A_1 = 0.452$$

From Table 6.5 we find $C_{v\,rate}$ = 1.051.
Summarizing: Q_{rate} = 0.22 m^3/s and $C_{v\,rate}$ = 1.051.

Step 2

Next, find the approach velocity coefficient for the "new" trapezoidal approach canal, where b_1 = 0.60 m and z_1 = 1.0. From Step 1, we know that A^* = 0.25 m^2 and C_d = 0.976. We can calculate $A_1 = y_1(b_1 + z_1 y_1)$ = 0.633 m^2, and subsequently

$$\sqrt{\alpha_1} C_d A^* / A_1 = 0.393$$

From Table 6.5 we find $C_{v\,new}$ = 1.037. Then, from Equation 6.29,

$$Q_{new} = Q_{rate}\frac{C_{v\,new}}{C_{v\,rate}} = 0.220\frac{1.037}{1.051} = 0.217 \text{ m}^3\text{/s}$$

In this case, the adjustment would be minor. However, the procedure should be repeated at depths spanning the full range of discharges to be used.

6.4.6 Scaling flume ratings by Froude modeling

It is a common practice in the study of fluid flow to use a laboratory scale model to study performance of a larger structure. The flow properties of the large structure are inferred from the model by using the laws of hydraulic similitude. For the study of open channel flow, the most important forces are inertia and gravity, and the ratio of these forces must be the same for the model and prototype. The square root of this ratio is known as the Froude number; thus this type of hydraulic similitude is often referred to as Froude modeling.

By this hydraulic similitude concept, if the properties of one structure are known, we can determine the properties of a "*similar*" structure. That is: a weir that is 1 m high and 1 m wide, with a head of 1 m behaves *similar* to a weir 2 m high and 2 m wide, with a head of 2 m. The discharge, however, is not doubled, but is found from the following equation

$$\frac{Q_{\text{prototype}}}{Q_{\text{model}}} = \left(\frac{L_{\text{prototype}}}{L_{\text{model}}}\right)^{2.5} \qquad 6.30$$

This equation is valid when all of the physical structure dimensions and heads are of the same ratio. With this equation, the ratings for a variety of flume sizes can be determined from a given rating.

Other factors affecting fluid flow are frictional forces and turbulent energy loss resulting from fluid viscosity. These forces are accounted for in scale models through the use of the Reynolds number, but it is generally not possible to simultaneously maintain identical Froude numbers and identical Reynolds numbers in the same scale model. Fortunately, viscous effects are often negligible or similar in the model and prototype when the Reynolds numbers of each are above certain thresholds, even if the values are not identical. Thus, as long as a model is not so small that its performance becomes dominated by viscous effects, the results can be reliably scaled using Froude modeling.

The procedures in this chapter are consistent with the concept of Froude modeling; that is, all of the head-discharge equations have Q on one side and dimensions of length to the exponent 2.5 plus the units associated with $g^{0.5}$ on the other. (The units are balanced by $g^{0.5}$.) The coefficient C_d is sensitive to changes in friction and turbulent energy dissipation, effects accounted for through the Reynolds number. However, in Figure 6.16, C_d is shown as strictly a function of a length ratio. This is an average curve for a wide range of flume shapes and sizes and an average Reynolds number. Thus, the use of this curve provides only an approximate adjustment for friction within the Froude modeling concept. The mathematical model described in the next section more accurately accounts for the effects of friction through the use of friction loss calculations that vary as a function of the Reynolds number. The effects of scale on the head-discharge relationship due to friction are minor until the relative roughness becomes large. Thus reasonably accurate ratings usually can be made with the Froude model extensions to other sizes, particularly if the scale ratio is between 1:4 and 4:1.

6.5 Head-Discharge Calibrations from Computer Modeling

6.5.1 Energy losses affecting head-discharge calibration

Because no ideal fluids exist in the real world, we must account for the effects of friction. Evaluating the actual discharge through a flume requires that we consider friction in the approach channel, converging transition, and throat (see Figure 6.18). Friction loss in the diverging transition and tailwater channel (i.e., parts of the structure downstream from the critical section) does not affect the flume rating, but does affect the tailwater limit for maintaining modular flow.

Three methods are available for estimating friction losses through the flume: the Manning equation, the Chezy equation, and the boundary-layer drag method. WinFlume uses the boundary-layer drag method. Ackers and Harrison (1963) reported that the effects of friction could be replaced with a change in flow area, represented by an imaginary displacement thickness (Harrison 1967b). Replogle (1975) expanded upon their work and developed a flume model based on the boundary-layer drag theory, which, with minor modifications, is presented in this section.

Boundary layer theory

For the boundary-layer analysis, it is assumed that the throat of the flume is one side of a thin and smooth flat plate held parallel to the fluid flow. The plate causes a drag

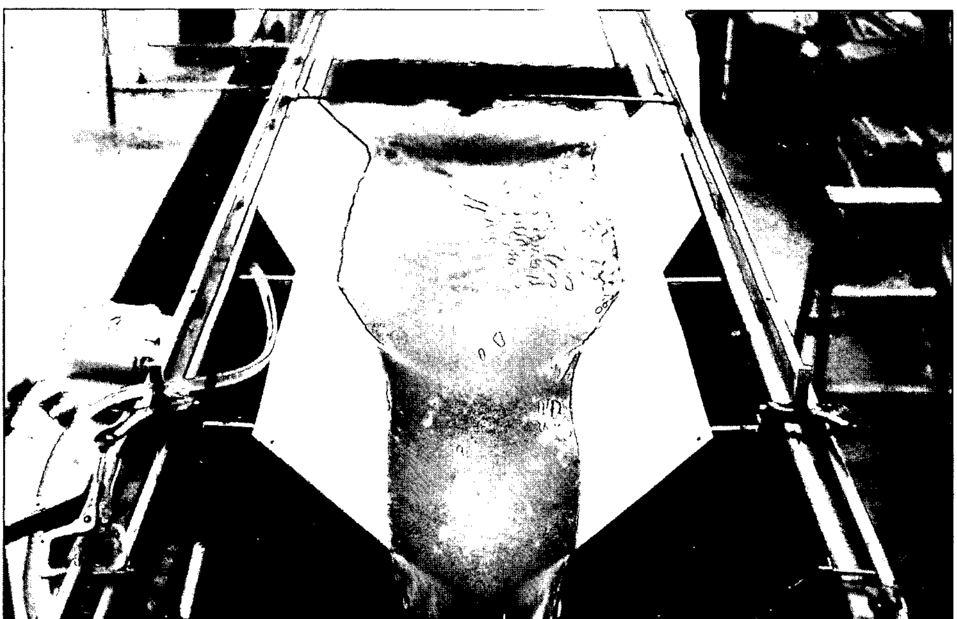


Figure 6.18 Research on the head-discharge characteristics.

on the fluid, which causes energy or head losses. The boundary layer is assumed to be "tripped" by the break between the converging transition and the throat. Boundary-layer theory indicates that the flow conditions in the boundary layer are not constant, but vary along the plate. The boundary layer starts out as laminar flow and then develops into turbulent flow, as shown in Figure 6.19. In reality, the transition from laminar to turbulent flow is gradual. For computing drag, however, the transition is assumed to be abrupt and to occur at a distance, L_x, from the entrance to the throat.

The combined drag coefficient, C_F, can be found by adding the relative drag coefficients for the laminar and turbulent parts of the boundary layer (Schlichting 1960). The drag coefficient for the turbulent part of the boundary layer is computed as if the entire boundary layer were turbulent; thus the drag coefficient for the nonexistent turbulent boundary layer over L_x, namely $C_{F,x}$, must be subtracted from the turbulent drag coefficient over the throat length L, $C_{F,L}$. The combined drag coefficient is then

$$C_F = C_{F,L} - \frac{L_x}{L} C_{F,x} + \frac{L_x}{L} C_{f,x} \qquad 6.31$$

where $C_{f,x}$ is the coefficient for the laminar boundary layer over L_x. The distance L_x can be developed from an empirical relationship for the Reynolds number of the laminar portion of the boundary layer:

$$\mathrm{Re}_x = 350\,000 + \frac{L}{k} \qquad 6.32$$

where k is the absolute roughness height of the material. This Reynolds number is related to L_x by the definition

$$\mathrm{Re}_x = v_c \frac{L_x}{\nu_i} \qquad 6.33$$

where $v_c = Q/A_c$ is the average velocity of flow, and ν_i is the kinematic viscosity of the fluid. Similarly, the Reynolds number over the entire throat length, L, is

$$\mathrm{Re}_L = v_c \frac{L}{\nu_i} \qquad 6.34$$

Values for the turbulent drag coefficients are found from the following relationship (Harrison 1967b), which was derived from Granville (1958):

$$C_{F,L} = \frac{0.544 C_{F,L}^{1/2}}{5.61 C_{F,L}^{1/2} - 0.638 - \ln\left[\left(\mathrm{Re}_L C_{F,L}\right)^{-1} + \left(4.84 C_{F,L}^{1/2} L / k\right)^{-1}\right]} \qquad 6.35$$

Equation 6.35 can also be used to determine $C_{F,x}$ by replacing $C_{F,L}$, Re_L, and L with $C_{F,x}$, Re_x, and L_x. This equation must be solved by trial and error, since $C_{F,L}$ (or $C_{F,x}$) appears several times.

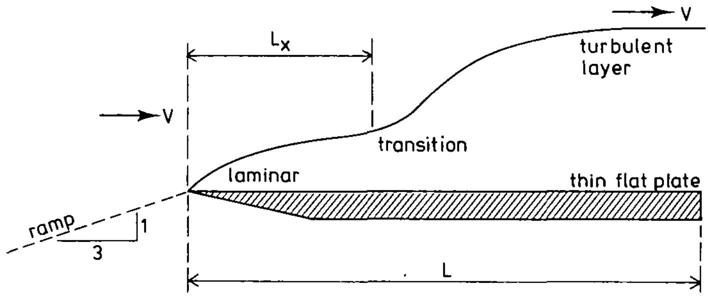


Figure 6.19 Transition from laminar to turbulent boundary layer

The drag coefficient for laminar flow can be computed by the following equation suggested by Schlichting (1960):

$$C_{f,x} = \frac{1.328}{\sqrt{\mathrm{Re}_x}} \qquad 6.36$$

If $\mathrm{Re}_L < \mathrm{Re}_x$ (Equations 6.32 and 6.34), then the entire boundary layer is laminar and $C_F = C_{f,L}$, which is found from Equation 6.36, with Re_L replacing Re_x.

For a fully developed turbulent boundary layer, as would be expected in the approach channel, converging transition, diverging transition, and tailwater channel (Figure 6.20), the drag coefficient can be taken as 0.00235. The head loss for each part of the flume is computed from

$$\Delta H_L = \frac{C_F L}{R} \frac{v^2}{2g} \qquad 6.37$$

where length and subscript L apply to each flume section considered, and R is the hydraulic radius (area divided by wetted perimeter). The combined head loss of the approach channel, converging transition, and throat is subtracted from the energy head at the gaging station to give the energy head in the critical section, $H_c = H_1 - \Delta H_1$. Equation 6.17c changes to

$$y_c = H_1 - \frac{A_c}{2B_c} - \Delta H_1 \qquad 6.38$$

where

$$\Delta H_1 = \Delta H_a + \Delta H_b + \Delta H_L \qquad 6.39$$

and the ΔH_a, ΔH_b, and ΔH_L correspond to the head losses in the approach channel, converging transition, and throat, respectively.

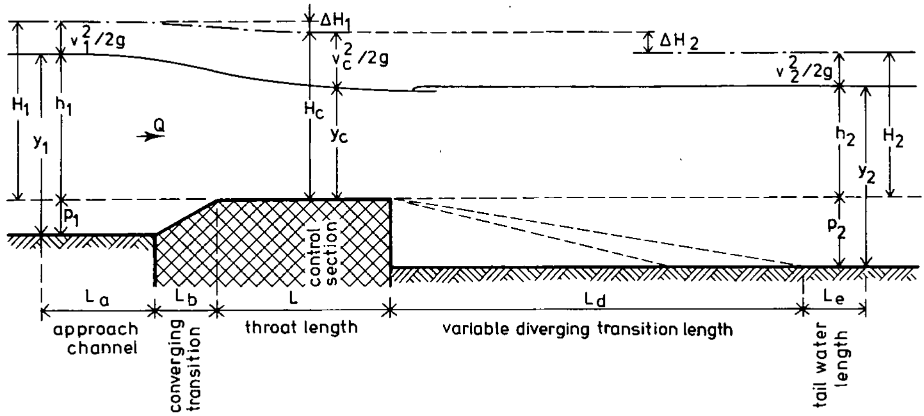


Figure 6.20 Illustration of terminology related to flume head losses.

Roughness of flow surfaces

The absolute roughness heights for a number of materials typically used for flume construction are given in Table 6.6. An analysis of the effects of roughness height showed that a change of several orders of magnitude in the value of k produces less than 0.5% (often less than 0.1%) change in discharge. Thus, a change in materials from smooth glass to rough concrete will have a minor effect on the discharge rating of the flume. This minor effect, however, should not be used as an excuse for sloppy or poor construction. If the surfaces in the control section have large undulations and irregularities, the real discharge can differ considerably from the theoretical value due to changes in control section area and the location of the critical section. Variations of material roughness and ability to maintain construction tolerances should be regarded as different sources of potential error (see Section 5.4.3).

6.5.2 Influence of velocity distribution

The equations for ideal flow developed earlier in this chapter assume that the velocity profile in the throat is uniform. It may not be uniform, however, and so a velocity-distribution coefficient, α, is introduced to take non-uniform velocity

Table 6.6 Absolute roughness height of materials used in flume construction.

Material		*Range of k, meters*		
Glass		0.000 001	to	0.000 010
Metal	Painted or smooth	0.000 020	to	0.000 10
	Rough	0.000 10	to	0.001 0
Wood		0.000 20	to	0.001 0
Concrete	Smooth troweled	0.000 10	to	0.002 0
	Rough	0.000 50	to	0.005 0

profiles into account. The value of α is the ratio of the actual velocity head of the flow divided by the velocity head based on the average velocity of the flow, and it is always greater than unity.

In long prismatic channels with a fully developed flow profile, α approaches a value of roughly 1.04 (Watts et al. 1967). For the approach channel, the velocity profile is assumed to be fully developed. This approximate value of $\alpha_1 = 1.04$ is used without further adjustment since the error in energy-head calculations resulting from an error in α_1 or in the velocity head is relatively small.

The velocity distributions for critical flow tend to be relatively uniform. However, for the control section, the velocity head is a much larger percentage of the total energy head, so some correction for α_c at the control section is still warranted. The following equation has been developed to estimate α for fully developed flow in wide channels (Chow 1959):

$$\alpha = 1 + 3\varepsilon^2 - 2\varepsilon^3 \qquad 6.40$$

where $\varepsilon = (v_m/v) - 1$, with v_m being the maximum flow velocity. For fully developed flow, ε can be approximated by

$$\varepsilon = 1.77 C_{F,L}{}^{1/2} \qquad 6.41$$

At the control section, the channel may not be sufficiently wide, and the flow profile may not be fully developed. Two additional factors are added to Equation 6.40 to take these deficiencies into account (Replogle 1974):

$$\alpha_c = 1 + \left(3\varepsilon^2 - 2\varepsilon^3\right)\left(\frac{1.5D}{R} - 0.5\right)\left(\frac{0.025L}{R} - 0.05\right) \qquad 6.42$$

with the factors limited to the ranges

$$1 \le (1.5D/R - 0.5) \le 2$$

$$0 \le (0.025L/R - 0.05) \le 1$$

In Equation 6.42, D is the average depth or the hydraulic depth (area divided by top width) and the other terms are as previously defined. This equation produces velocity distribution coefficients ranging from 1.00 to 1.04 for the ranges of conditions typically found in practice. This range is realistic, since several investigators have found nearly uniform velocity profiles at the control sections of long-throated flumes (e.g., Bos and Reinink 1981). For computing flume and weir head-discharge relationships, we use $\alpha_1 = 1.04$, and α_c is computed from Equation 6.42.

6.5.3 Accuracy of computed flow and the range of H_1/L

A limitation (Equation 6.27) was earlier placed on the range of H_1/L values for which a reasonably reliable discharge rating can be obtained when an empirical discharge coefficient is used.

$$0.1 \le H_1 / L \le 1.0 \quad \text{based on experimentation}$$

This limitation was based on extensive laboratory data on a wide variety of flumes made from a variety of construction materials (Bos 1985). Within this range, a good estimate of the discharge can be made from an empirical curve through the data. The data appear more closely grouped in the middle range (H_1/L = 0.35 to 0.70), with a range of about ±3% for the 95% confidence limits, and are more widely scattered at the extremes (H_1/L = 0.1 and H_1/L = 1.0), with a range of about ±5% for the 95% confidence limits. One of the primary reasons for the wide scatter of data in the low range is friction. The mathematical model described above thus represents an improvement over the empirical approach, since it can accurately take frictional effects into account even when the value of H_1/L is as low as 0.05. A second major reason for the wide scatter of data at the high range of H_1/L is streamline curvature. The laboratory data begin to deviate from the computer predictions above an H_1/L-value of about 0.5 because of streamline curvature, making the theoretical range of applicability of the model

$$0.05 \le H_1 / L \le 0.5 \quad \text{based on application of theory}$$

To reach a compromise between these two ranges, we consider two practical considerations. First, the roughness of flume construction materials changes over time. At low H_1/L values, these roughness changes can have a significant effect on the flume calibration. Thus, while the model can predict these effects down to H_1/L = 0.05, possible changes in roughness suggest a more realistic lower limit of $H_1/L \ge 0.07$. Second, for ratios of H_1/L up to 0.70, the effects of streamline curvature have only a slight effect on the discharge coefficient. A reasonable compromise between the two ranges for H_1/L is therefore

$$0.07 \le H_1 / L \le 0.70 \quad \text{recommended in practice} \qquad 6.43$$

which we recommend. Within this range the mathematical model calculates rating tables with an error of less than ±2% (95% confidence interval). Outside this range, this error slowly increases to about ±4% at H_1/L = 1.0. If a structure is to be designed with a high value of the ratio Q_{max}/Q_{min}, the full range of H_1/L-values should be used (see Section 2.4). In the output of WinFlume, the value of H_1/L can be given as part of the head-discharge tables. The equations for calibration accuracy used in WinFlume (Section 8.8.1) are

$$X_C = \pm\left(1.9 + 742\left(0.07 - \frac{H_1}{L}\right)^{1.5}\right) \qquad H_1/L < 0.07 \qquad 6.44a$$

$$X_C = \pm\, 1.9 \qquad 0.07 \le H_1/L \le 0.7 \qquad 6.44b$$

$$X_C = \pm\left(1.9 + 12.78\left(\frac{H_1}{L} - 0.7\right)^{1.5}\right) \qquad H_1/L > 0.7 \qquad 6.44c$$

These equations are designed to approximate the known variations in rating table uncertainties within the range of $0.05 \leq H_1/L \leq 1.0$, and to cause X_C to increase rapidly at lower and higher H_1/L values, so as to discourage design of flumes in this range.

6.5.4 Computing head-discharge relationship with model

Computing discharge when head is known

Actual flow rates are computed by the same procedures that were used for ideal flow rates except that Equations 6.16, 6.10, and 6.38 replace Equations 6.17a, 6.17b, and 6.17c, respectively. Values for ΔH_L are obtained from Equations 6.31 to 6.37, and the value for α_c is found from Equation 6.42. The ideal flow rate is computed first and then used as the initial guess for the actual flow rate. Next, the friction losses and velocity distribution coefficients are computed for the estimated discharge. Then, the actual flow rate (Equation 6.16) and the critical depth (Equation 6.38) are computed. The trial-and-error process is repeated (as for the ideal flow rate) until y_c converges. The resulting flow rate is checked against the flow rate for the previous values of ΔH_L and α_c. (The first time through, it will be compared with the ideal discharge, Q_i.) If the flow rate has not converged, ΔH_L and α_c are computed with the new discharge, Q, and the process is repeated until the flow rate converges.

Computing head when discharge is known

When Q is known, Equations 6.16 and 6.42 are solved iteratively for y_c and α_c, with equations for A_c and B_c as a function of y_c. H_c is found from Equation 6.11, H_1 is found from $H_1 = H_c + \Delta H_1$, and h_1 is found from Equation 6.10, with A_1 being a function of h_1. The ideal discharge for h_1 can only be found after h_1 is known, and is found by the ideal-flow procedure for determining Q from h_1. The ideal flow is computed only for purposes of determining the discharge coefficient, C_d.

6.5.5 Computing contraction needed for critical flow

With a known approach section, there are many ways to produce a contraction that creates critical flow. When considering the available shapes of the control section (see Section 5.1), the number of possible solutions is infinite. However, if we restrict ourselves to a particular control section shape (e.g., a trapezoid), and a method for varying the contraction in that shape (e.g., raising the sill), then for a known upstream head and discharge, the relationships given in Section 6.3.3 can be used to determine the amount of contraction required. This is the reverse of the typical calibration approach where the contraction amount (or throat shape relative to the approach section) is known and the discharge for a given head is unknown.

In Equations 6.19 and 6.22, there are four unknowns: u, C_v, A^*/A_1, and B^*/B_1. If u and B^*/B_1 were known, both equations could be solved iteratively for C_v and A^*/A_1.

This would indicate the exact amount of contraction needed to create critical flow under the given upstream conditions. For a given initial shape of the control section, B^*/B_1 is known for a given upstream head. The value of u, however, is not known and, for many shapes, is not constant with depth. It represents the slope of the tangent to a point on the head-discharge curve plotted on logarithmic paper. For common flume shapes, values range from $u = 1.5$ for a rectangular cross-section to $u = 2.5$ for a triangular cross-section, with trapezoids, parabolas, etc., falling between. (See Table 2.2).

The general solution for determining the amount of contraction uses an iterative solution for the dimensions of the control section, since both B^*/B_1 and A^*/A_1 are allowed to change and since u is unknown. The procedure for determining u, which converges very quickly, is as follows:

1. Estimate u; $u = 2$ is a good initial guess for an unknown control section.
2. Compute y_c from Equation 6.21 (with $H_c = h_1$ as a rough first estimate).
3. Compute A_c and B_c from the cross-section shape and dimensions for the control section, and then compute u from Equation 2.5, $u = B_cH_c/A_c$. This becomes the new estimate for u and the process is repeated until u converges.

Once a value for u has been determined, Equations 6.19 and 6.22 are solved for C_v and A^*/A_1. Depending on the method of contraction change, B^*/B_1 may also change, but this is governed by the throat shape and so does not really add an additional unknown. The result is a new contraction amount and thus a new throat cross-section matching the ratio A^*/A_1. Since the shape changes, u must be re-estimated and the ratio A^*/A_1 re-computed. This ratio changes slightly with u so the process converges very rapidly (e.g., two or three iterations). It is recommended that minimal contraction ($A^*/A_1 \approx 0.85$) be specified in the initial structure.

This procedure is used in the WinFlume design alternatives module (Section 8.8.3) to determine both the minimum and the maximum contraction amount that might produce workable structures for a given site. The maximum contraction amount is that which produces an upstream water level that exactly satisfies the freeboard requirement at Q_{max}. The minimum contraction amount is that needed to satisfy the Froude number criteria and the criteria for modular flow at both Q_{min} and Q_{max}. Starting from an initial throat section shape and size, WinFlume uses the user-selected method of contraction change to alter the control section to arrive at the needed ratios A^*/A_1. Once these minimum and maximum contraction amounts are determined, WinFlume checks all design requirements for these two structures, and additional possible designs between these extremes are evaluated at a contraction interval selected by the user. The user can then select from among the acceptable designs or change one or more of the criteria, dimensions, or design limitations. As a starting point, the user may wish to specify a control-section shape that is identical to the approach-section shape. WinFlume simply alters the first guess so that the solution will converge. Additional details of the algorithm are given in Section 8.8.10.

6.6 Head Loss Over Structures

6.6.1 Maintaining critical flow

Maintaining critical flow in the throat requires that the energy head downstream from the structure be somewhat less than the energy head in the critical section for any given discharge. When critical flow occurs, the structure is operating as a flow measurement module, hence the term modular flow. The energy head downstream from the structure is controlled by the channel conditions and structures downstream. Therefore, the flume must be designed so that the energy head in the critical section (and the approach channel) are high enough above these levels to ensure modular flow. This certainly will occur if the downstream energy level, H_2, (see Figure 6.20), is less than the critical depth, y_c, at the control section. In such a case, the available head loss (H_1 - H_2) exceeds (H_1 - y_c), and there is no need to transform the kinetic energy at the control section ($v_c^2/2g$) into potential energy downstream from the transition (h_2). In other words, there is no need for a gradual transition between the throat and the downstream channel (see Figure 6.21).

If the head loss over the structure is limited to such an extent that the downstream water level, h_2, becomes higher than the y_c level, a gradual transition can be added to regain potential energy. The amount of potential energy that can be regained depends mainly on the degree of expansion of the transition and on the ratio of cross-sectional flow areas at the control section, A_c, and at the section where h_2 is determined, A_2. The amount of potential energy that can be regained will define the limiting value of h_2, and the related H_2 value, that permit critical flow to occur in the control section. These limiting values must be determined whenever the available head is less than H_1 - y_c. The modular limit is the highest ratio between downstream and upstream energy head referenced to the flume sill or crest, H_2/H_1, at which the flow is still modular (i.e., where the upstream head-discharge relation is not affected by downstream conditions).

Determining the energy loss across the structure is needed to determine whether or not flow is modular. This energy loss can be divided into three parts:

(a) The losses between the upstream head measurement section (gaging station) and the control section in the throat, mainly due to friction;
(b) The losses due to friction between the control section and the section where h_2 is measured; and
(c) The losses due to incomplete conversion of kinetic energy into potential energy over the downstream transition (i.e., the expansion loss).

The losses described in (a) affect the rating of the structure since they change the relation between h_1 and Q. The losses described in (b) and (c) affect the modular limit of the structure.

In Section 6.5.1, methods were given for determining the head or energy loss from the gaging station to the end of the flume throat, ΔH_1. This energy loss is computed

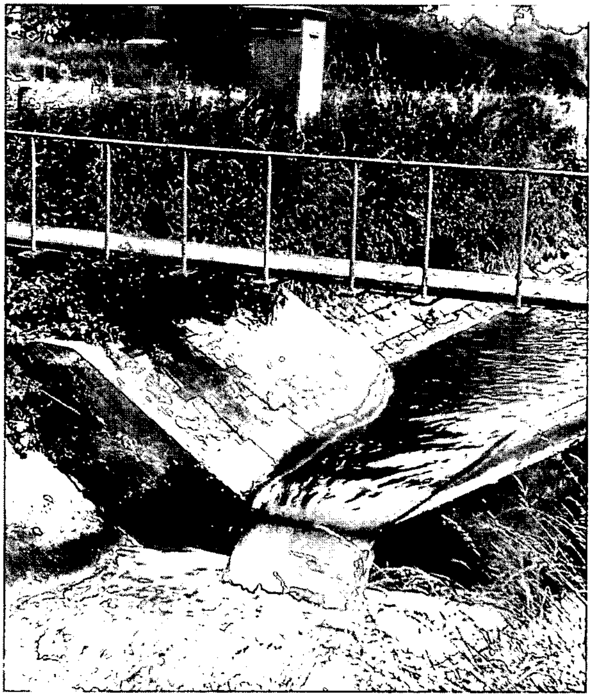


Figure 6.21 If no kinetic energy needs to be recovered, a sudden downstream expansion is adequate (The Netherlands).

iteratively using boundary-layer theory. If, however, the head-discharge relationship is determined experimentally by using a discharge coefficient, C_d, then the energy losses can be estimated from (Bos 1976 and Bos and Reinink 1981):

$$\Delta H_1 = H_1 - H_c \approx H_1\left(1 - C_d^{\,1/u}\right) \qquad 6.45$$

Within the limits of application of H_1/L as discussed in Section 6.4, Equation 6.45 gives a good estimate of the energy losses upstream from the control section.

The frictional energy losses downstream from the flume throat are relatively small compared with the turbulent energy losses. Thus, some rough approximations are sufficient. The friction losses can be estimated with sufficient accuracy by boundary-layer drag methods as discussed in Section 6.5.1. Just as for the approach channel, a constant drag coefficient of 0.00235 can be used. The head loss downstream from the flume throat is computed with Equation 6.37. No information is available for estimating the velocity distribution coefficient, α_2, and since it also has little effect compared with the turbulent energy losses, it is assumed equal to unity. (Alternately, the energy loss could be estimated with the Manning equation, but this is unnecessary considering the small energy loss when compared to the expansion loss). Equation 6.37 is applied to the reach containing the diverging transition of the bottom and sidewalls, and to the canal reach from the end of the transition to the section at which h_2 is measured. Since h_2 is not actually measured at most sites, the appropriate location for h_2 is the point at which all energy recovery has taken place. This section is estimated in the WinFlume program to have a length $L_e = 10(p_2+L/2)-L_d$, where L is the length of the throat section, p_2 is the sill height relative to the downstream channel invert, and L_d is the length of the diverging transition (see Figure 6.20).

The total energy loss due to friction over the downstream part of the structure is

$$\Delta H_f = \Delta H_d + \Delta H_e \tag{6.46}$$

where ΔH_f is the friction loss downstream from the structure, ΔH_d is the friction loss over the downstream transition, and ΔH_e is the friction loss in the tailwater channel section, L_e (see Figure 6.20). The friction losses are computed with Equation 6.37.

The energy loss for the downstream expansion (diverging transition) can be computed from

$$\Delta H_k = \xi \frac{(v_c - v_2)^2}{2g} \tag{6.47}$$

where ΔH_k is the energy loss due to the rapid expansion and ξ is an expansion energy loss coefficient that can be obtained from (adapted from Bos and Reinink 1981)

$$\xi = \frac{\log_{10}[114.59 \arctan(1/m)] - 0.165}{1.742} \tag{6.48}$$

where $\log_{10}$ is the base 10 logarithm, arctan is the inverse tangent in radians, and m is the expansion ratio of the downstream diverging transition (see Figure 6.22). For a flume with only a bottom contraction (e.g., the broad-crested weir), the expansion ratio is straightforward. It is simply the length of the transition divided by the downstream sill height, p_2. For flumes with a side contraction or a combination of a side and a bottom contraction, the proper value of the expansion ratio is less obvious. In general, the expansion of the flume bottom has a greater effect on the energy loss and recovery than the side contraction, because it affects the full width of the flow. Thus, for flumes with a sizable bottom contraction, the expansion ratio of the bottom should be used in head-loss calculations. When the contraction is primarily from the side, the expansion ratio for the sidewalls should be used. Obviously, in some cases, both play a role, and there is no definite way to determine which to use. The WinFlume program described in Chapter 8 uses only one expansion ratio (entered on the bottom profile form as the diverging transition slope) and assumes it applies equally to the bottom and side transitions. Observed data indicate that the values of ξ from Equation 6.48 are conservative and can be used for most structures. For a sudden expansion, the value of ξ is 1.2.

The total energy loss downstream from the throat can now be calculated by adding the friction losses (Equation 6.46) and expansion losses (Equation 6.47), which yields

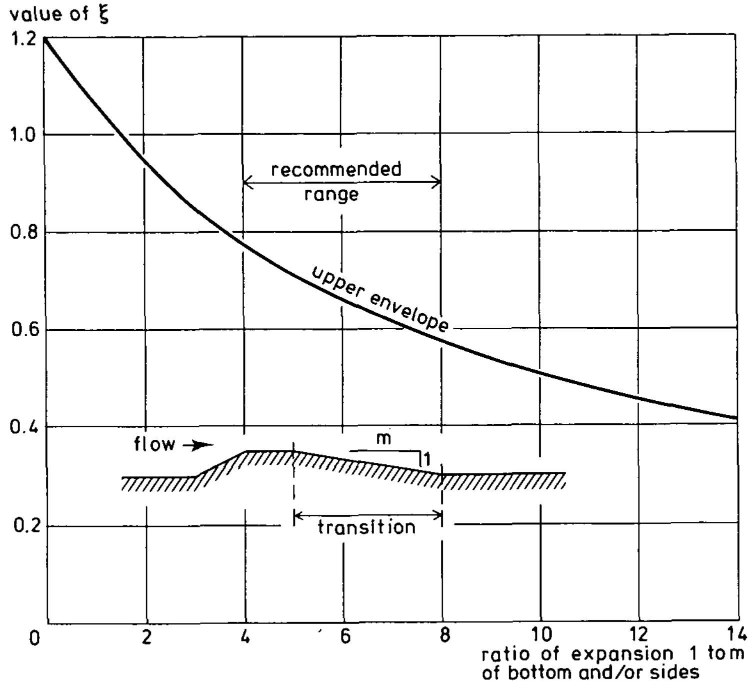


Figure 6.22 Values of ξ as a function of the expansion ratio of the downstream transition (adapted from Bos and Reinink, 1981).

$$\Delta H_2 = \Delta H_d + \Delta H_e + \Delta H_k = \Delta H_f + \Delta H_k \qquad 6.49$$

6.6.2 Determining the modular limit (allowable tailwater level)

The flume designer would usually like to find the maximum tailwater level and energy head, H_2, for which modular flow exists (Figure 6.23). These are found by combining Equations 6.39 and 6.49 to compute the minimum amount of energy loss required through the structure. The modular limit is the ratio of the associated downstream and upstream energy heads at this minimum energy loss condition.

$$\begin{aligned} H_2 &= H_c - \Delta H_2 \\ &= (H_1 - \Delta H_1) - (\Delta H_f + \Delta H_k) \\ &= H_1 - \Delta H_a - \Delta H_b - \Delta H_L - \Delta H_d - \Delta H_e - \Delta H_k \end{aligned}$$

$$ML = \frac{H_2}{H_1} \qquad 6.50$$

We can also use Equation 6.45 to a obtain a more general expression for the modular limit of any long-throated flume or broad-crested weir:

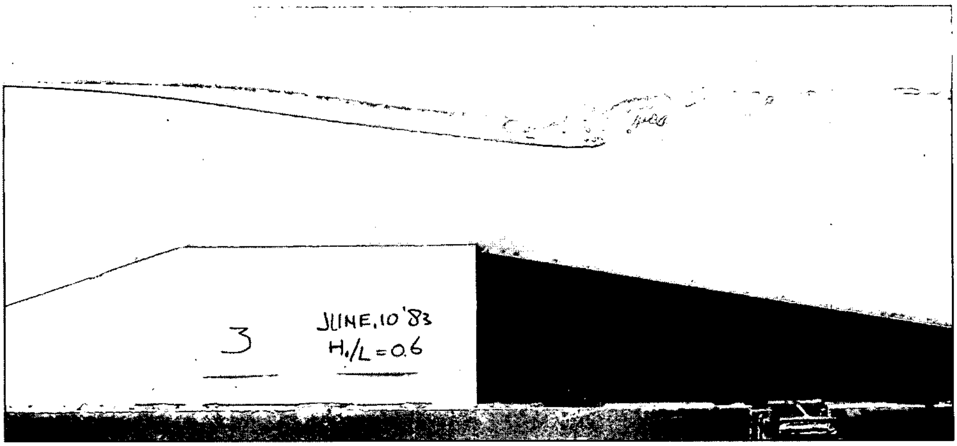


Figure 6.23 Research on submergence of a flume.

$$\begin{aligned} H_2 &= (H_1 - \Delta H_1) - (\Delta H_f + \Delta H_k) \\ &\approx H_1 - H_1\left(1 - C_d^{1/u}\right) - \Delta H_f - \Delta H_k \end{aligned}$$

$$ML = \frac{H_2}{H_1} \approx C_d^{\,1/u} - \frac{\Delta H_f}{H_1} - \xi \frac{(v_c - v_2)^2}{2gH_1} \qquad 6.51$$

That part of Equation 6.51 that expresses the sum of the energy losses due to friction becomes a large portion of the total energy loss for very gradual expansion ratios. This is mainly because the relatively high flow velocities in the downstream transition are maintained over a greater length. Long, but very gradual, downstream transitions thus have a favorable energy conversion (low ξ value) but lose some energy due to friction (high ΔH_f-value). As a result, very gradual transitions (more than 10:1) lose more energy than more rapid but shorter transitions. Because the construction cost of a very gradual transition is also higher than that of a shorter one, we advise that the ratio of expansion be no greater than about 6:1.

Rather sudden expansion ratios such as 1:1 or 2:1 are not very effective for energy conversion because the high velocity jet leaving the throat cannot change direction suddenly to follow the boundaries of the transition. In the flow separation zones that result, eddies are formed that convert kinetic energy into heat and noise. Therefore, we do not recommend the use of the expansion ratios 1:1, 2:1, or 3:1. If the length downstream from the throat is insufficient to accommodate a fully developed gradual transition, we recommend truncating the transition to the desired length rather than using a more sudden expansion ratio (see Figure 6.24). Truncating the transition to half its full length has a negligible effect on the modular limit. The truncation should not be rounded since that would guide the water into the channel bottom, causing additional energy losses and possible erosion.

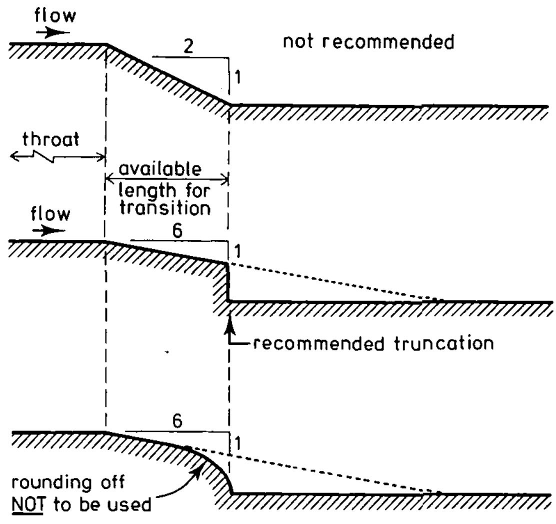


Figure 6.24 Truncation of a gradual downstream transition.

The total energy losses are maximum if a weir or flume has a sudden expansion ($\Delta H_f = 0$, and $\xi = 1.2$) and the discharge is into stagnant water ($v_2 = 0$). The energy loss can be estimated as (with $C_d \approx 1.0$ for convenience)

$$\Delta H_{\max} = 1.2 \frac{v_c^2}{2g} \qquad 6.52$$

For a rectangular control section $v_c^2/2g = H_1/3$; hence $\Delta H_{max} = 0.40H_1$. Data for other control shapes are given in Table 6.7. The value of the modular limit increases rapidly with increasing tailwater velocity and the addition of a diverging transition (decreasing ξ value). To obtain a conservative design of the structures in Chapter 3, the modular limit should be assumed to be less than 0.90.

Computational procedure for estimating the modular limit

Once the head-discharge relationship for the flume is known, for a given head-discharge (h_1, Q) pair the following are also known: the energy head at the control section, H_c; the velocity at the control section, v_c; and the upstream head loss, ΔH_1. Equation 6.48 is used to compute the expansion-loss coefficient, ξ, based on the expansion ratio, m. An estimate of the downstream water level and velocity are needed to estimate the downstream energy losses, ΔH_f and ΔH_k. A reasonable starting point is $h_2 = y_c$. Next, compute the downstream water depth, $y_2 = h_2 + p_2$, and velocity, v_2, based on h_2, Q, and the downstream cross-section, where p_2 is the downstream sill height.

The downstream distances to consider in determining head loss can be estimated from

$$L_d = p_2 m$$
$$L_e = 10(p_2 + L/2) - L_d \qquad 6.53$$

Table 6.7 Head-loss requirement under most unfavorable conditions.[a]

Shape of control section	*Power u of h_1*	*y_c/H_c*	*Minimum modular limit H_2/H_1*	*ΔH_{max}*
Rectangle	1.5	0.67	0.60	$0.40H_1$
Average trapezoid or parabola	2.0	0.75	0.70	$0.30H_1$
Triangle	2.5	0.80	0.76	$0.24H_1$

[a] $\xi = 1.2$ and $v_2 = 0$.

Equation 6.53 provides sufficient length to get away from the downstream turbulence. Then the downstream friction losses can be computed from

$$\Delta H_e = \frac{0.00235 L_e v_2^2}{2gR_2}$$

$$\Delta H_d = \frac{0.00235 L_d}{4g}\left(\frac{v_c^2}{R_c} + \frac{v_2^2}{R_2}\right) \qquad 6.54$$

where R is the hydraulic radius (area divided by wetted perimeter). Now H_2 can be computed from Equations 6.46 through 6.50. This value will likely be different from the value of H_2 implied by our initial guess of h_2. The next guess for the downstream water level can be found from

$$y_{2,new} = y_{2,guess}\frac{\left(H_{2,new} + p_2\right)}{\left(H_{2,guess} + p_2\right)} \qquad 6.55$$

where the subscript *guess* refers to the trial value and *new* refers to the new value computed from the energy calculations (Equations 6.46 through 6.50). A new value of h_2 is computed based on $y_{2,new}$ and the process is repeated until H_2 converges.

Example

The trapezoidal flume of the example in Section 6.3.2 (with b_c = 0.20 m, z_c=1.0, $p_1 = p_2$ = 0.15 m and L =0.60 m) is placed in an ongoing concrete lined canal with $b_1 = b_2$ = 0.50 m and $z_1 = z_2$ = 1.0. For Q_{max} = 0.0732 m³/s the upstream sill-referenced head is h_1 = 0.238 m.

If the weir has a rapid expansion, then $L_d = 0$, $\Delta H_d = 0$, L_e = 4.5 m, $\Delta H_f = \Delta H_e$ and $\xi = 1.2$. Substitution of these values into the above equations yields

y_2 = 0.343 m,
ML = 0.817, and
ΔH = 0.044 m.

If the tailwater level is too high to accommodate this head loss a downstream expansion may be added to the structure. Using WinFlume (Chapter 8) to calculate ML and ΔH for a 6:1 downstream expansion ($m = 6$), these values change to

$y_2 = 0.361$ m,
$ML = 0.893$, and
$\Delta H = 0.026$ m.

Thus, the required head loss decreases by about 0.02 m. One must then decide if it is better to avoid submergence by adding this expansion or raising the sill. The decision depends on such factors as standardization of structures and rating tables, available freeboard, and construction costs of the alternative structures.

In this context, the reader must note that the hydraulic roughness of a channel changes with age, the seasons, and so on. To avoid non-modular flow through the weir or flume, the design should be analyzed for the condition of maximum hydraulic roughness and associated tailwater depth, y_2.

7 The Downstream Side of the Structure

7.1 Introduction

As discussed before, the water level downstream from a structure must be lower than that upstream in order to measure a discharge. In other words, there will be a drop of energy level ΔH over the weir or flume. In relatively flat irrigated plains, ΔH usually will be less than about 0.30 m (1 ft) and the turbulence created by the weir or flume can be handled by either the concrete canal lining or by a sufficiently long riprap protection of the earthen canal. In sloping areas, it is usually necessary to limit the flow velocities in each channel section by means of drop structures in the canal bottom. For an economic canal system design, a weir or flume can be combined with these drop structures. The difference in energy head ΔH over such a two-purpose structure calls for a basin in which surplus energy is dissipated within the structure so that the channel downstream is not damaged. These energy dissipators should have rectangular cross sections.

To aid in selecting a satisfactory energy dissipator for the downstream side of the weir or flume, information must be obtained on

Q_{max} = maximum flow over the structure,
b_c = the width of the control section,
q_{max} = $Q_{\max} / b_c$, the maximum discharge per unit width over the structure,
H_{1max} = $h_1 + v_1^2/2g$, the sill-referenced energy head, and
ΔH = head loss over the structure.

Next, a number of parameters are calculated, including the Froude number Fr_u at the section of maximum velocity within the basin (for the location of section U see Figure 7.1):

H_d = the downstream energy head relative to the bottom of the energy dissipator;
ΔZ = $\Delta H + H_d - H_1$, the drop height from the weir sill to the dissipator bottom;
g = acceleration due to gravity;
v_u = $\sqrt{2g\Delta Z}$, the velocity of a free falling jet dropping a distance ΔZ;
y_u = q_{max}/v_u, the minimum water depth on the bottom of the dissipator (section U); and
Fr_u = $v_u/\sqrt{gy_u}$ the Froude number at section U. 7.1

This Froude number guides the designer to a number of structural alternatives. Although the limits between alternative designs are not sharp, from a practical viewpoint, we may state the following:

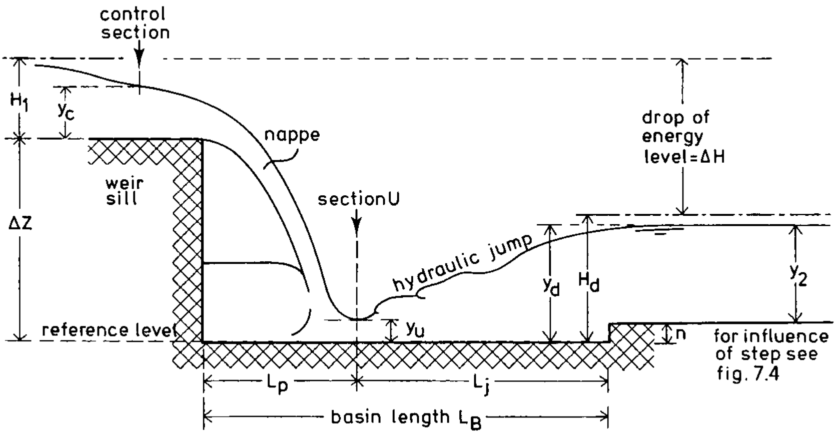


Figure 7.1 Illustration of terminology for a drop structure.

- If $Fr_u \leq 2.5$, no baffles or special devices are required but the downstream channel should be sufficiently protected over a short length (see Section 7.3).
- If Fr_u ranges between 2.5 and 4.5, the hydraulic jump is not well stabilized. The entering jet oscillates from bottom to surface and creates waves with irregular period in the downstream channel. We therefore recommend that energy be dissipated using structures that increase turbulence and not by relying solely on the naturally formed jump.
- If $Fr_u \geq 4.5$, a stable jump that can dissipate energy effectively will be formed.

Figure 7.2 shows these recommendations graphically. For a known value of the discharge per unit width, q, and an estimate of the drop height, ΔZ, Figure 7.2 gives a first indication of which dissipator type is appropriate. Upon making a more detailed hydraulic design, a better ΔZ value becomes available, which may lead to another structure.

Construction of a complex energy dissipator for a low discharge and low drop but high Froude number is impractical because the energy to be dissipated is low. Thus we have placed some limits on the minimum drop height for these structures at 0.2 and 0.4 m, as shown in Figure 7.2. Also, large straight drops often require massive structures that may be overly expensive and hydraulically unreliable. Thus we do not recommend straight drops of more than ΔH=1.5 m (see Figure 7.1) except under special circumstances. These limits on drop height ΔZ, energy drop ΔH, and Froude number Fr_u, are not absolute, but give the designer practical limits for quick decision making.

The energy dissipators described in this chapter may not be suitable for every project and they certainly do not exhaust the possibilities open to the designer. The features discussed, however, may be combined with the discharge measurement structures in most canal systems. For further information—on straight drops, end sills, baffle blocks, tapered sidewalls, to name only a few—the references Peterka (1964) and USBR (1973) are recommended.

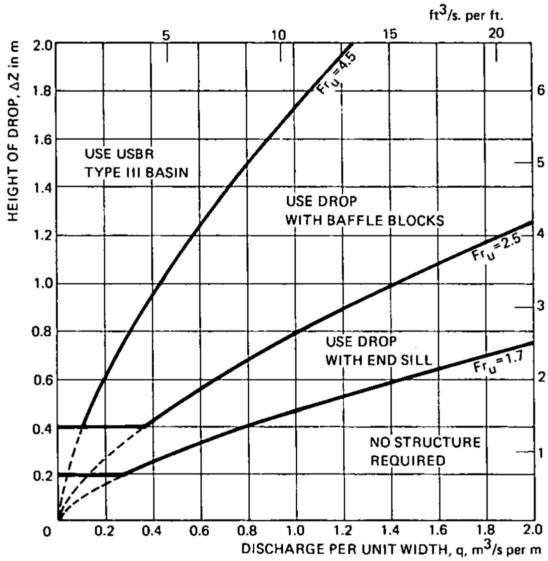


Figure 7.2 Diagram for estimating the type of energy dissipator to be used prior to detailed design.

7.2 Energy Dissipators

7.2.1 Straight drop

In a straight drop the free-falling nappe will strike the basin floor and turn downstream at section U (see Figure 7.1). Because of the impact of the nappe and the turbulent circulation in the pool beneath the nappe, some energy is dissipated. Further energy will be dissipated in the hydraulic jump downstream from section U. The remaining energy head downstream in the basin, H_d, does not vary greatly with the ratio $\Delta Z/H_1$ and is equal to about $1.67H_1$ (adapted from Henderson, 1966). This value of $1.67H_1$ provides a satisfactory estimate for the basin floor level below the energy level of the downstream canal.

As indicated in Section 7.1, the hydraulic dimensions of an energy dissipator, and thus of a straight drop, can be related to the Froude number at section U, Fr_u. This Froude number can be related directly to the straight drop geometry through the length ratios $y_d/\Delta Z$ and $L_p/\Delta H$, values of which can be read from Figure 7.3. (Also see Figure 7.1).

It is important to realize that the downstream water depths (y_d and y_2) are caused not by the drop structure, but by the flow characteristics of the downstream canal. If these characteristics are such that the required depth y_d is produced, a jump will form; otherwise it will not form and not enough energy will be dissipated within the basin. The exact location of the jump varies as a function of the flow depth in the downstream canal.

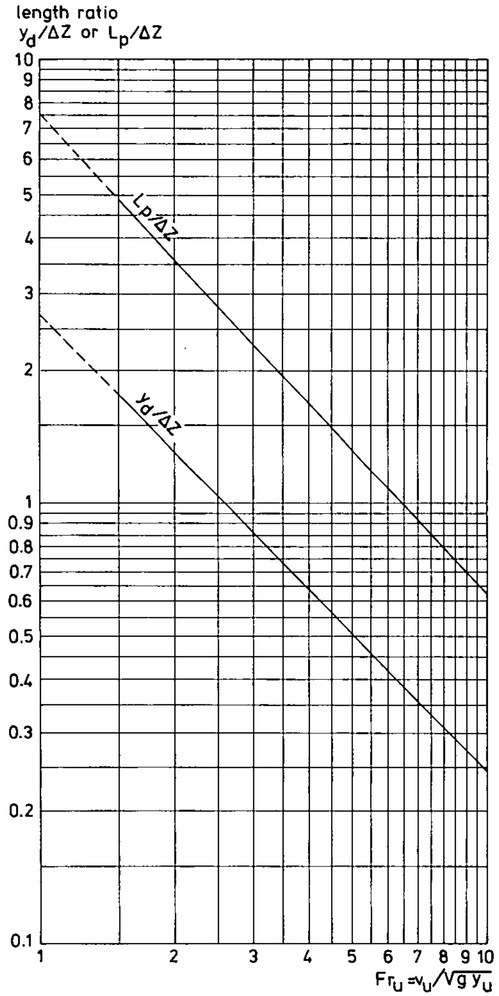


Figure 7.3 Dimensionless plot of straight drop geometry.

Because of seasonal changes of the hydraulic resistance of the canal, the flow depth as calculated by Manning's equation changes. The jump thus tends to drift up and down the canal seasonally. This unstable behavior is often undesirable, and is then suppressed by increasing the flow resistance by means of an abrupt step at the end of the basin. Usually, this step is constructed at a distance

$$L_j = 5(n + y_2) \tag{7.2}$$

downstream from section U, with n being the height of the step. For design purposes, Figure 7.4 can be used to determine the largest required value of n, if Fr_u, y_u and y_2 are known. If the drop structure discharges into a relatively wide canal or if the downstream water depth y_2 is not determined by the frictional resistance of the

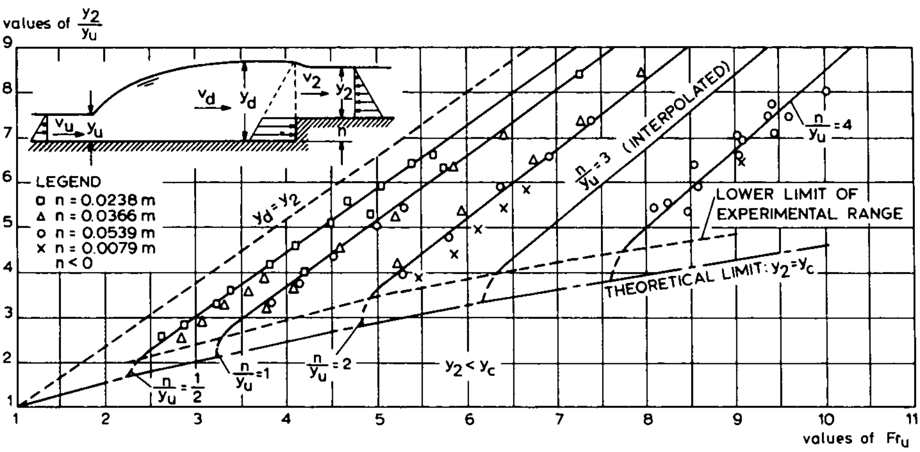


Figure 7.4 Experimental relationships between Fr_u, y_2/y_u and n/y_u for an abrupt step (after Forster and Skrinde 1950).

downstream channel but by a downstream control, the step height n must also be determined for lower flow rates and anticipated values of y_2. The highest n value must be used for the design.

The length of the straight drop with end sill is greatly influenced by the jump length L_j. As discussed with the introduction of Equation 7.2, the hydraulic jump can be stabilized and shortened by increasing the flow resistance downstream from section U. To shorten the basin downstream from section U, the hydraulic resistance can be increased further by placing baffle blocks on the basin floor.

7.2.2 Baffle-block-type basin

The baffle-block-type basin has been developed for low drops in energy level (Figure 7.5), and it gives a reasonably good dissipation of energy for a wide range of downstream water depths. The dissipation of energy is principally by turbulence induced by the impingement of the incoming flow upon the baffle blocks. The required downstream water depth, therefore, can be slightly less than with the previous basin but can vary independently of the drop height ΔZ. To function properly, the downstream water depth y_d must not be less than $1.45H_1$, while at Q_{max} the Froude number Fr_u should not exceed 4.5.

Upstream from section U, the length L_p may be determined by use of Figure 7.3. The linear dimensions of the basin downstream from section U are shown in Figure 7.6 as a function of H_1.

As mentioned earlier, the basin length of the energy dissipator shown in Figure 7.5 is much shorter than the basin having only an end sill . Although this reduction in length is a significant advantage of this type of basin, the baffle blocks have one

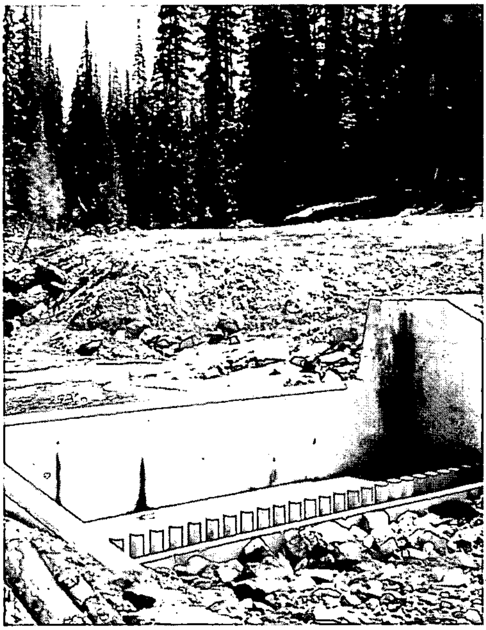


Figure 7.5 Straight drop with baffle-block-type energy dissipator (Canada).

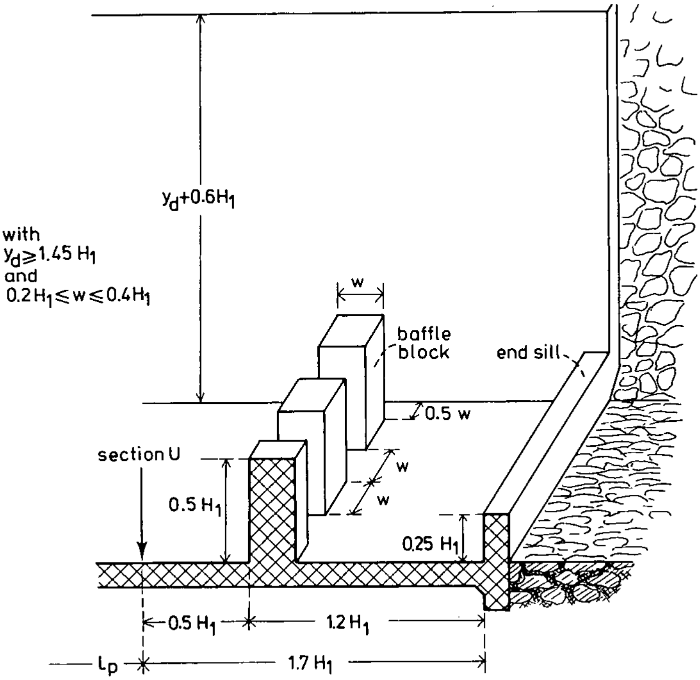


Figure 7.6 Dimensions of the baffle-block-type basin downstream from section U (Donnelly and Blaisdell 1954).

major drawback: They collect all types of floating and suspended debris, which may lead to overtopping of the basin and damaging of the baffle blocks. Therefore, to function properly, these basins require regular cleaning.

7.2.3 Inclined drop

Downstream from the control of a weir or flume, a sloping face, guiding the overfalling nappe, is a common design feature, especially if the energy drop exceeds 1.5 m. In drop structures, the slope of the downstream face often is as steep as possible. If a sharp-edged broken plane transition is used between the control and the downstream face, we recommend use of a slope no steeper than 2:1 (see Figure 7.7). The reason is to prevent flow separation at the sharp edge (development of negative pressure at the downstream end of the weir crest). If a steeper slope (1:1) is required, the sharp edge should be replaced by a transitional curve with a radius of $r \approx 0.5H_1$ (see Figure 7.7).

Values of y_u and H_u that can be used for the design of the basin downstream from section U may be determined by use of Table 7.1. The symbols used in Table 7.1 are defined in Figure 7.7. In this context the reader must note that the energy level H_u of the nappe entering the basin at section U has a much higher value if a sloping downstream face is used than if the nappe would fall free as with the straight drop. The reason is that with a straight drop, energy is dissipated because of the impact of the nappe on the basin floor and the turbulent circulation of water in the pool beneath the nappe. With the inclined drop, there is much less energy dissipation due to friction and turbulent flow over the sloping face (see Table 7.1 and Figure 7.7).

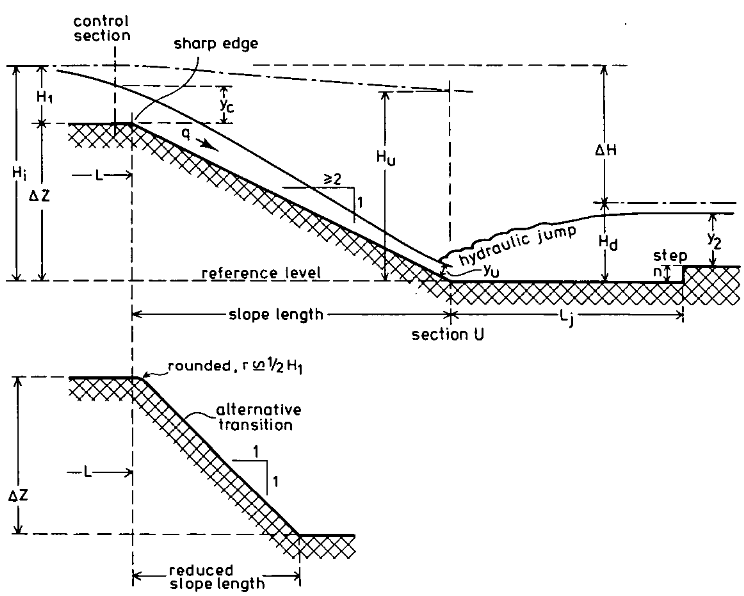


Figure 7.7 Definition sketch for Table 7.1.

Table 7.1 Dimensionless ratios for hydraulic jumps.

$\frac{\Delta H}{H_1}$	$\frac{y_d}{y_u}$	$\frac{y_u}{H_1}$	$\frac{v_u^2}{2gH_1}$	$\frac{H_u}{H_1}$	$\frac{y_d}{H_1}$	$\frac{v_d^2}{2gH_1}$	$\frac{H_d}{H_1}$
0.2446	3.00	0.3669	1.1006	1.4675	1.1006	0.1223	1.2229
0.2688	3.10	0.3599	1.1436	1.5035	1.1157	0.1190	1.2347
0.2939	3.20	0.3533	1.1870	1.5403	1.1305	0.1159	1.2464
0.3198	3.30	0.3469	1.2308	1.5777	1.1449	0.1130	1.2579
0.3465	3.40	0.3409	1.2749	1.6158	1.1590	0.1103	1.2693
0.3740	3.50	0.3351	1.3194	1.6545	1.1728	0.1077	1.2805
0.4022	3.60	0.3295	1.3643	1.6938	1.1863	0.1053	1.2916
0.4312	3.70	0.3242	1.4095	1.7337	1.1995	0.1030	1.3025
0.4609	3.80	0.3191	1.4551	1.7742	1.2125	0.1008	1.3133
0.4912	3.90	0.3142	1.5009	1.8151	1.2253	0.0987	1.3239
0.5222	4.00	0.3094	1.5472	1.8566	1.2378	0.0967	1.3345
0.5861	4.20	0.3005	1.6407	1.9412	1.2621	0.0930	1.3551
0.6525	4.40	0.2922	1.7355	2.0276	1.2855	0.0896	1.3752
0.7211	4.60	0.2844	1.8315	2.1159	1.3083	0.0866	1.3948
0.7920	4.80	0.2771	1.9289	2.2060	1.3303	0.0837	1.4140
0.8651	5.00	0.2703	2.0274	2.2977	1.3516	0.0811	1.4327
0.9400	5.20	0.2639	2.1271	2.3910	1.3723	0.0787	1.4510
1.0169	5.40	0.2579	2.2279	2.4858	1.3925	0.0764	1.4689
1.0957	5.60	0.2521	2.3299	2.5821	1.4121	0.0743	1.4864
1.1763	5.80	0.2467	2.4331	2.6798	1.4312	0.0723	1.5035
1.2585	6.00	0.2417	2.5372	2.7789	1.4499	0.0705	1.5203
1.3429	6.20	0.2367	2.6429	2.8796	1.4679	0.0687	1.5367
1.4280	6.40	0.2321	2.7488	2.9809	1.4858	0.0671	1.5529
1.5150	6.60	0.2277	2.8560	3.0837	1.5032	0.0655	1.5687
1.6035	6.80	0.2235	2.9643	3.1878	1.5202	0.0641	1.5843
1.6937	7.00	0.2195	3.0737	3.2932	1.5368	0.0627	1.5995
1.7851	7.20	0.2157	3.1839	3.3996	1.5531	0.0614	1.6145
1.8778	7.40	0.2121	3.2950	3.5071	1.5691	0.0602	1.6293
1.9720	7.60	0.2085	3.4072	3.6157	1.5847	0.0590	1.6437
2.0674	7.80	0.2051	3.4723	3.7254	1.6001	0.0579	1.6580
2.1641	8.00	0.2019	3.6343	3.8361	1.6152	0.0568	1.6720
2.2620	8.20	0.1988	3.7490	3.9478	1.6301	0.0557	1.6858
2.3613	8.40	0.1958	3.8649	4.0607	1.6446	0.0548	1.6994
2.4615	8.60	0.1929	3.9814	4.1743	1.6589	0.0538	1.7127
2.5630	8.80	0.1901	4.0988	4.2889	1.6730	0.0529	1.7259
2.6656	9.00	0.1874	4.2171	4.4045	1.6869	0.0521	1.7389
2.7694	9.20	0.1849	4.3363	4.5211	1.7005	0.0512	1.7517
2.8741	9.40	0.1823	4.4561	4.6385	1.7139	0.0504	1.7643
2.9801	9.60	0.1799	4.5770	4.7569	1.7271	0.0497	1.7768
3.0869	9.80	0.1775	4.6985	4.8760	1.7402	0.0489	1.7891
3.1949	10.00	0.1753	4.8208	4.9961	1.7530	0.0482	1.8012
3.4691	10.50	0.1699	5.1300	5.2999	1.7843	0.0465	1.8309
3.7491	11.00	0.1649	5.4437	5.6087	1.8146	0.0450	1.8594
4.0351	11.50	0.1603	5.7623	5.9227	1.8439	0.0436	1.8875
4.3267	12.00	0.1560	6.0853	6.2413	1.8723	0.0423	1.9146
4.6233	12.50	0.1520	6.4124	6.5644	1.9000	0.0411	1.9411
4.9252	13.00	0.1482	6.7437	6.8919	1.9268	0.0399	1.9667
5.2323	13.50	0.1447	7.0794	7.2241	1.9529	0.0389	1.9917
5.5424	14.00	0.1413	7.4189	7.5602	1.9799	0.0379	2.0178
5.8605	14.50	0.1381	7.7625	7.9006	2.0032	0.0369	2.0401
6.1813	15.00	0.1351	8.1096	8.2447	2.0274	0.0361	2.0635
6.5066	15.50	0.1323	8.4605	8.5929	2.0511	0.0352	2.0863
6.8363	16.00	0.1297	8.8153	8.9450	2.0742	0.0345	2.1087
7.1702	16.50	0.1271	9.1736	9.3007	2.0968	0.0337	2.1305
7.5081	17.00	0.1247	9.5354	9.6601	2.1190	0.0330	2.1520
7.8498	17.50	0.1223	9.9005	10.0229	2.1407	0.0323	2.1731
8.1958	18.00	0.1201	10.3894	10.3894	2.1936	0.0317	2.1936
8.5438	18.50	0.1180	10.7575	10.7575	2.2141	0.311/	2.2141
8.8985	19.00	0.1159	11.1290	11.1290	2.2339	0.0305	2.2339
9.2557	19.50	0.1140	11.5091	11.5091	2.2534	0.0300	2.2534
9.6160	20.00	0.1122	11.8887	11.8887	2.2727	0.0295	2.2727

Downstream from section U, Equation 7.2 and Figure 7.4 can be used for dimensioning the energy dissipator. As with the straight drop, adding chute blocks and/or baffle blocks to the basin will reduce the length of the basin.

7.2.4 USBR Type III basin

In selecting the basin layout, the reader must note that the basin with baffle blocks of Figure 7.5 was designed to dissipate energy by turbulence. Such a basin operates satisfactorily if the Froude number at maximum anticipated flow, Fr_u, does not exceed 4.5 (see Figure 7.2). For higher Froude numbers, the USBR Type III basin shown in Figures 7.8 and 7.9 may be used (Bradley and Peterka 1957).

7.3 Riprap Protection

To prevent canal bottom and bank damage by erosive currents passing over the end sill of a basin or leaving the tail of a small weir or flume (Figure 7.10), riprap is usually placed on the downstream canal bottom and banks (see also Figures 3.33 and 3.51). Several factors affect the length over which this protection is needed. As a rule of thumb, we suggest a length of riprap protection which is (1) not less than 4 times the (maximum) normal depth in the downstream canal, (2) nor less than the length of the earth transition between structure and canal, (3) nor less than 1.50 m (5 ft).

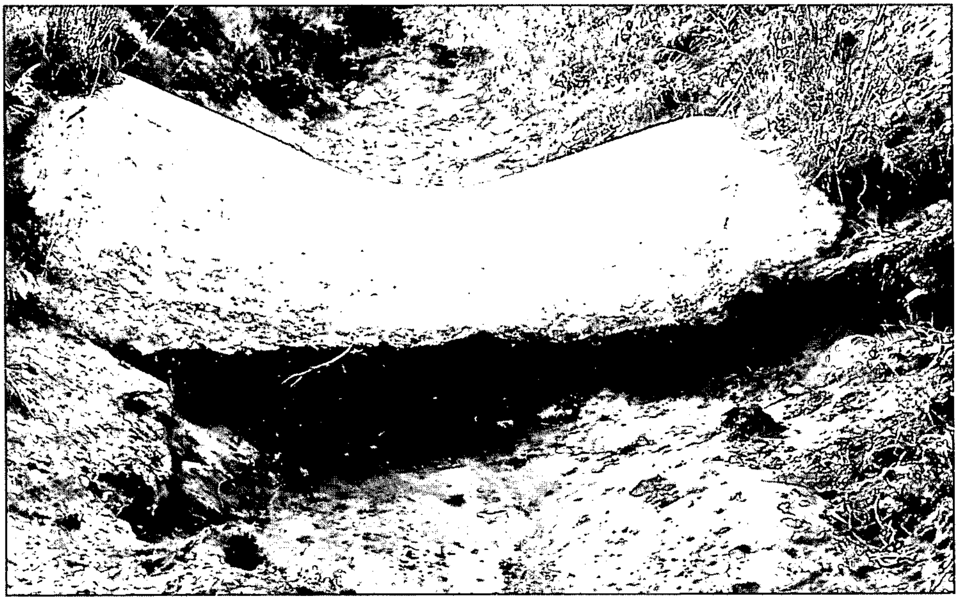


Figure 7.10 To avoid failure of the structure a cutoff wall and a riprap protection must be added to this flume (Arizona).

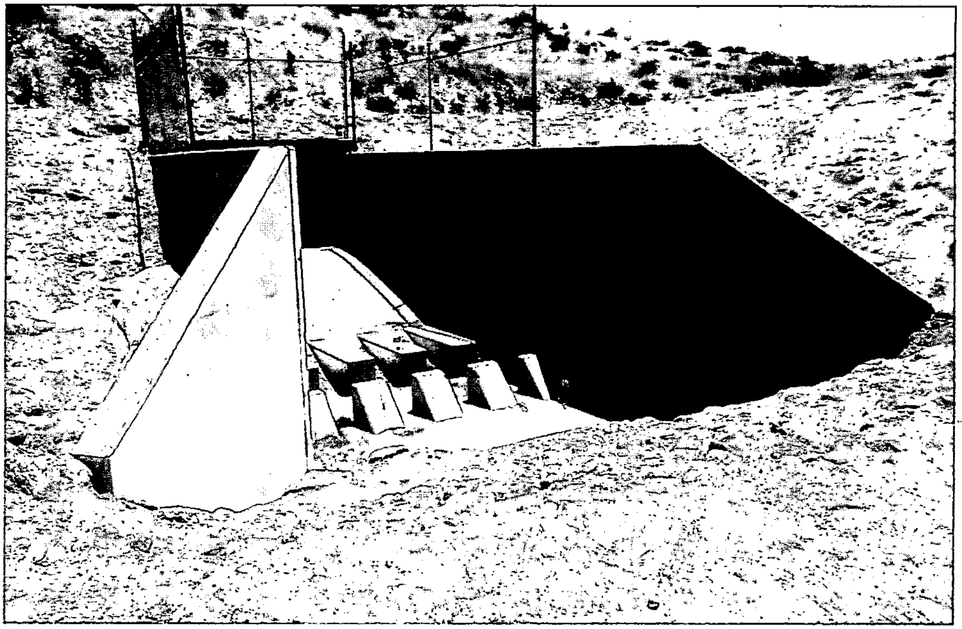


Figure 7.8 USBR Type III basin (Arizona).

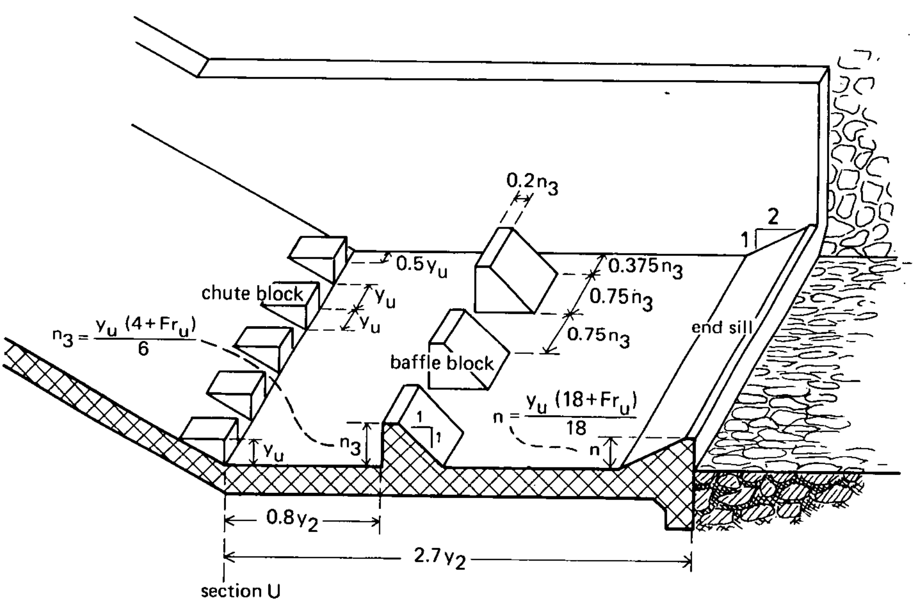


Figure 7.9 Stilling basin characteristics for use with Froude numbers above 4.5; USBR Type III basin (Bradley and Peterka 1957).

7.3.1 Determining stone size of riprap protection

Several factors affect the stone size required to resist forces that tend to move riprap. In terms of flow leaving a structure, these factors are velocity, flow direction, turbulence, and waves. Because of the variable combination of these factors, the velocity with which water will strike the riprap is rather unpredictable unless the basin is tested. For practical purposes, however, we recommend finding the stone diameter by use of Figure 7.11. To use this figure, the average velocity above the end sill of the basin can be calculated by dividing the discharge by the cross-sectional area of flow above the end sill. If no stilling basin is needed because $Fr_u \leq 1.7$ (see Section 7.1), Figure 7.11 should be entered with the impact velocity v_u, which is

$$v_u = \sqrt{2g\Delta Z} \qquad 7.3$$

Figure 7.11 gives the d_{40} size of the riprap mixture required to resist erosion. This means that 40 percent of the mixture may be finer than the dimensions shown in the figure and 60 percent must be greater than the size shown. The mixture should consist of stones that have length, width, and thickness dimensions as nearly alike as is practical (i.e., approximately spherical, although some angularity is desirable). If stones are excessively long in one dimension, then stone size no longer is a good indicator of riprap suitability, and the mixture should be sized by weight with 60 percent of the stones being of curve weight or heavier (right-hand axis of Figure 7.11) and not flat slabs.

7.3.2 Filter material placed beneath riprap

If riprap stones of a protective lining were to be installed directly on top of the fine material in which the canal is excavated, grains of this subgrade would be washed through the openings in between the riprap stones. This process is partly due to the turbulent flow of canal water in and out of the voids between the stones and partly due to the inflow of water that leaks around the structure or flows into the canal.

To avoid damage to a riprap protection because of the washing of subgrade, a filter must be placed between the riprap and the subgrade (see Figure 7.12). The protective construction as a whole and each separate layer must be sufficiently permeable to water entering the canal through its bed or banks. Further, fine material from an underlying filter layer or the subgrade must not be washed into the voids of a covering layer.

Permeability to water

To ensure that the filter layer has a sufficient permeability to water, the following d_{15}/d_{15} ratios should have a value between 5 and 40 (USBR 1973):

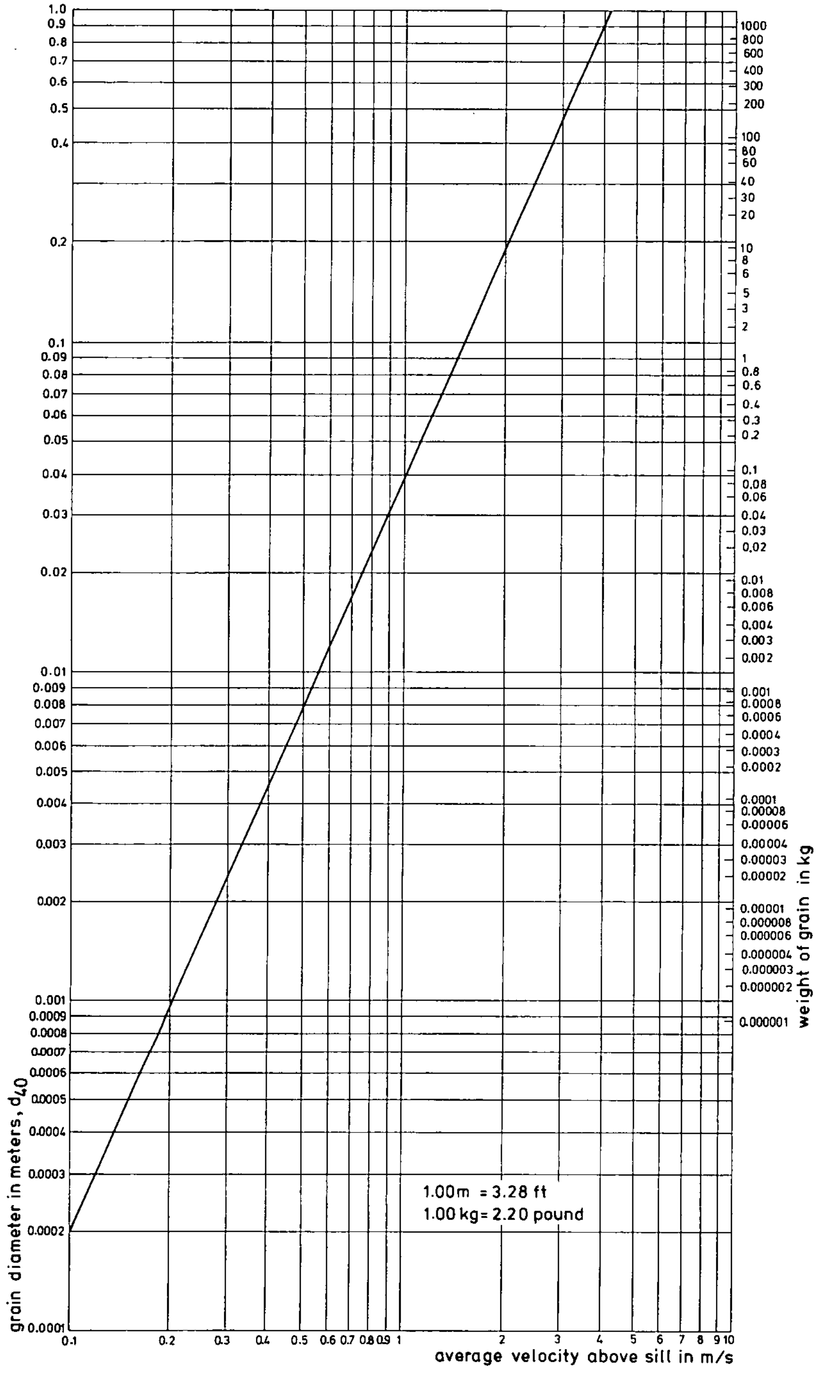


Figure 7.11 Relation between average velocity above end of sill of dissipator and stable grain size (Bos 1989).

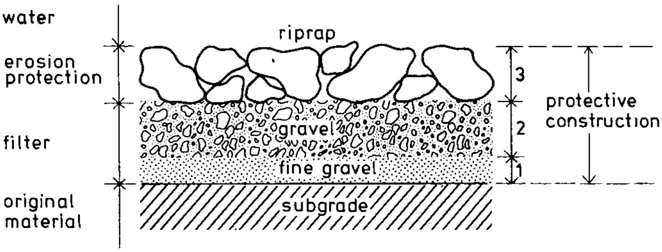


Figure 7.12 Example of filter between riprap and original material (subgrade) in which canal is excavated.

$$\frac{d_{15}\text{ layer 3}}{d_{15}\text{ layer 2}}\text{ and }\frac{d_{15}\text{ layer 2}}{d_{15}\text{ layer 1}}\text{ and }\frac{d_{15}\text{ layer 1}}{d_{15}\text{ subgrade}}=5\text{ to }40 \qquad 7.4$$

where d_{15} is the diameter of the sieve opening for which 15% of the total weight of the sample passes the sieve. Depending on the shape and gradation of the grains in each layer, the above mentioned 5 to 40 range of the ratios can be narrowed as follows (Bendegom et al. 1969):

Homogeneous round grains (gravel)	5 to 10
Homogeneous angular grains (broken gravel, rubble)	10 to 20
Well-graded grains	12 to 40

In addition, to prevent the filter from clogging it is advisable that for each layer

$$d_5 \geq 0.75\text{ mm (0.03 in.)} \qquad 7.5$$

Stability of each layer

To prevent the loss of fine material from an underlying filter layer or the subgrade through the openings in a covering layer, two requirements must be met:

- The d_{15}/d_{85} ratios should not exceed 5 (Bertram 1940)

$$\frac{d_{15}\text{ layer 3}}{d_{85}\text{ layer 2}}\text{ and }\frac{d_{15}\text{ layer 2}}{d_{85}\text{ layer 1}}\text{ and }\frac{d_{15}\text{ layer 1}}{d_{85}\text{ subgrade}}\leq 5 \qquad 7.6$$

- The d_{50}/d_{50} ratios should range between 5 and 60 (U.S. Army Corps of Engineers 1955).

$$\frac{d_{50}\text{ layer 3}}{d_{50}\text{ layer 2}}\text{ and }\frac{d_{50}\text{ layer 2}}{d_{50}\text{ layer 1}}\text{ and }\frac{d_{50}\text{ layer 1}}{d_{50}\text{ subgrade}}=5\text{ to }60 \qquad 7.7$$

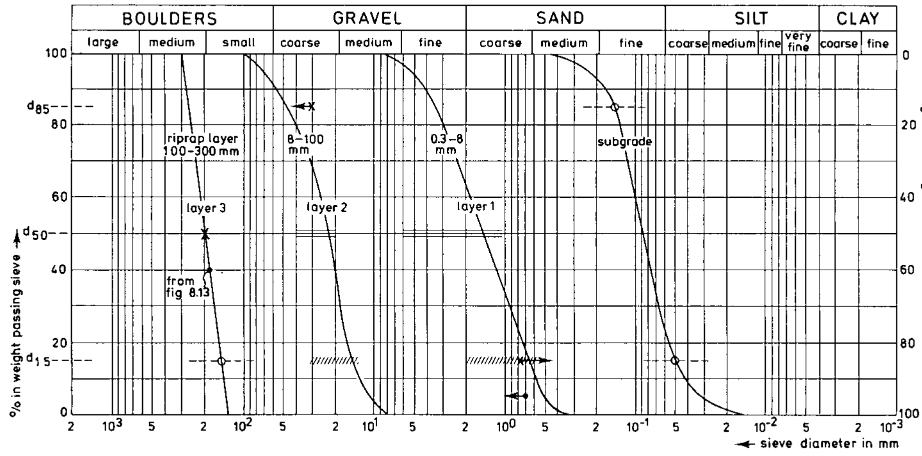


Figure 7.13 Sieve curve plotting of a protective construction.

As before, the ratio in Equation 7.7 can vary depending on the shape and gradation of the grains as follows:

Homogeneous round grains (gravel)	5 to 10
Homogeneous angular grains (broken gravel, rubble)	10 to 30
Well-graded grains	12 to 60

The requirements in this section describe the sieve curves of the successive filter layers. Provided that the sieve curves of the riprap layer and of the subgrade are known, other layers can be plotted. An example of plotting sieve curves of a construction consisting of one riprap and two filter layers is shown in Figure 7.13. In practice one should use materials that have a grain size distribution that is locally available, since it is uneconomical to compose a special mixture. To provide a stable and effective filter, the sieve curves for subgrade and filter layers should run about parallel for the small diameter grains.

Filter construction

To obtain a reasonable grain size distribution throughout a filter layer, each layer should be sufficiently thick. The following thickness must be regarded as a minimum for a filter construction made under dry conditions:

sand and fine gravel	0.05 to 0.10 m
gravel	0.10 to 0.20 m
stones	1.5 to 2 times the largest stone diameter

With filters constructed under water, these thicknesses have to be increased considerably both to compensate for irregularities in the subgrade and because it is more difficult to apply an even layer under water.

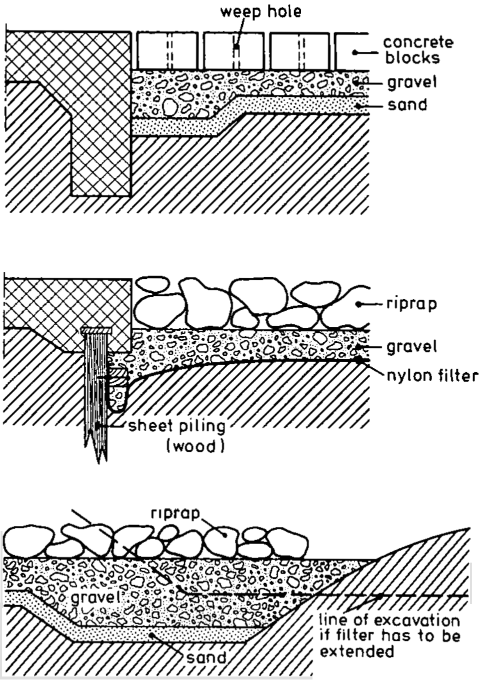


Figure 7.14 Filter construction details

Many variations can be made on the basic filter construction. One or more of the layers can be replaced with other materials. With some protective linings, only the riprap layer is maintained, while the underlying filter layers are replaced by one single layer. For example,

- Concrete blocks on a geotextile filter,
- Stone on braided hardwood strips on plastic filter,
- Gabions on fine gravel, or
- Geotextile-sand mattresses.

The usual difficulty with these variants is their perviousness to underlying material, which can lead to piping failures. As a rule, the openings in such a layer should not be greater than $^1/_2\ d_{85}$ of the underlying material. If openings are greater, one should not replace all underlying layers but maintain as many layers (usually one) as are needed to prevent the subgrade from being washed through the combined layer.

At structure-to-filter and filter-to-unprotected channel "joints," the protective construction is most subject to damage. This is because the filter layer is subject to subsidence while the (concrete) structure itself is well founded. Underlying material (subgrade) may be washed out at these joints if no special measures are taken. We recommend that the thickness of the filter construction be increased at these places. Some examples of common construction details are shown in Figure 7.14.

8. Using the WinFlume Software

8.1 Introduction

The previous chapters have emphasized that a primary advantage of long-throated flumes and broad-crested weirs is that they can be calibrated and designed using computer software based on established hydraulic theory, as opposed to most other critical-flow measurement devices which require laboratory calibration. This characteristic makes it possible to design structures for custom applications, optimize designs to meet specific operational criteria, and develop rating tables and rating equations for as-built structures.

Chapter 6 presented the hydraulic theory and calculation procedures used to develop the head-discharge relation for a long-throated flume or broad-crested weir. This chapter describes the use of the WinFlume software for developing rating tables and other output based on rating tables, and also shows how to use the program to develop designs for new structures. WinFlume is the latest in a series of flume analysis and design programs developed since 1984, initially through the cooperative research efforts of the Agricultural Research Service and the International Institute for Land Reclamation and Improvement, and now including the U.S. Bureau of Reclamation. Table 8.1 summarizes the evolution of these computer programs.

Table 8.1 Computer software for flume analysis and design.

Program	Reference	Source Code	Characteristics
Flume	Replogle, 1975. *Critical flow flumes with complex cross section.*	Basic	Developed rating table only for trapezoidal and complex trapezoidal shapes.
FLUME 1.0	Bos, Replogle and Clemmens, 1984. *Flow Measuring Flumes for Open Channel Systems*	FORTRAN IV	Develops rating tables for trapezoidal flumes at given head values. Included modular limits.
FLUME 2.0	Clemmens, Replogle and Bos, 1987b. *FLUME: A Computer Model for Estimating Flow Rates through Long-Throated Measuring Flumes* *USDA ARS-57*	FORTRAN IV	Similar to FLUME 1.0, except options for cross-section shapes greatly expanded, head-discharge equations provided, and rating tables for given discharge values provided
FLUME 3.0	Clemmens, Bos and Replogle, 1993. *FLUME: Design and Calibration of Long-Throated Measuring Flumes* *ILRI Publication 54*	Clipper, compiled for MS-DOS	Interactive, allows for calibration and design of flumes with design optimization to meet a particular head loss objective.
WinFlume 1.0	Clemmens, Wahl, Bos, and Replogle, 2001. *Water Measurement with Flumes and Weirs (This book)*	Visual Basic 4.0, for Microsoft Windows	Interactive, graphical user interface, calibration of existing structures, improved design module, enhanced output capabilities.

8.2 Computer System Requirements

The WinFlume program has been compiled in a 32-bit version for computers running the Windows 95/NT and more recent operating systems, and a 16-bit version for computers running Windows 3.1. The two program versions are nearly identical in their capabilities and use. The recommended minimum computer hardware for running WinFlume is

- Intel Pentium processor or equivalent,
- 16 MB RAM,
- VGA display, and
- Approximately 8 MB free hard disk space.

WinFlume also runs on computers with older processors or less memory, but the configuration described above is recommended for satisfactory performance.

8.3 Obtaining the Software

The current version of the WinFlume software is maintained on the U.S. Bureau of Reclamation's Water Resources Research Laboratory Internet site at *www.usbr.gov/wrrl/winflume.* The setup kits can be downloaded from this site, free of charge. As a product of the U.S. government, WinFlume is public-domain and can be copied and distributed freely to others, as long as appropriate recognition is given to its developers.

8.4 Installation

If you downloaded WinFlume from the Internet
First, be sure that you have the correct version of WinFlume for your operating system. Windows 3.1 users must use the 16-bit version. Users of Windows 95 or Windows NT (or more recent versions of Windows) should use the 32-bit version. To begin the installation, you must extract the setup kit from the compressed file that you downloaded. To do so, execute the file you downloaded by double-clicking on it. Detailed installation instructions will be available from the WinFlume web page on the Internet (see Section 8.3).

If you have the WinFlume CD-ROM
Insert the CD-ROM into your CD-ROM drive. On most computers, this will automatically start the installation process. If not, run the *INSTALL.EXE* program located in the root directory of the CD-ROM, or the appropriate *SETUP.EXE* program located in the *\SETUP32\DISK1* or *SETUP16\DISK1* directory.

8.5 Starting the Program

Once installation is completed, WinFlume may be started from the Windows Start Menu, or by double-clicking on the WinFlume icon located in the WinFlume program group.

The 32-bit version of WinFlume can also be started from a DOS prompt and accepts one optional command-line parameter:

winflume [*flume.flm*]

where *flume.flm* is the name of a flume file to be loaded on program startup. You can also drag and drop a **.flm* file onto the WinFlume program icon to start the WinFlume program with a particular flume file loaded.

In Windows 95/NT, you can define a default open action for the *FLM* file type, which will allow double-clicking on any **.flm* file to start WinFlume with that file loaded. For detailed instructions on accomplishing this, please refer to "*associating files*" in the Windows online help system.

8.6 Software Overview

Use of WinFlume is relatively straightforward. After starting the program you may load an existing flume file from the File menu, or create a new flume definition using the *File | New Flume* command. A helpful flume wizard is available to guide you through the initial data entry process, whether it be for an existing structure or an entirely new design. Once you have defined the geometric and hydraulic properties of a structure and site, you may calibrate that structure or review and refine the design using the design reports and design evaluation module.

WinFlume loads and saves files in its own **.flm* file format. One flume definition is contained in each **.flm* file. Flume designs can be shared among users simply by copying the appropriate **.flm* file to another user's computer. WinFlume is also able to import flume designs that were created using the FLUME 3.0 program. Once a flume definition is imported into WinFlume, modifications of that design can only be saved in the WinFlume **.flm* file format.

Online help is available throughout the WinFlume program, or separately by loading the help file from the icon created during program installation. Within the WinFlume program, you may press F1 at any time to obtain context-sensitive help on the program operation you are currently carrying out. You may also obtain help on the use and meaning of objects and controls on the screen by clicking the button on the toolbar (all versions), or the ? button located in the top-right corner of most dialog boxes (32-bit version only), then clicking on a particular object on the screen. A printable user's manual is also installed with the program and can be viewed or printed using the Adobe Acrobat Reader, version 4.0 or later. An installation kit for Adobe Acrobat Reader is included on the WinFlume CD-ROM.

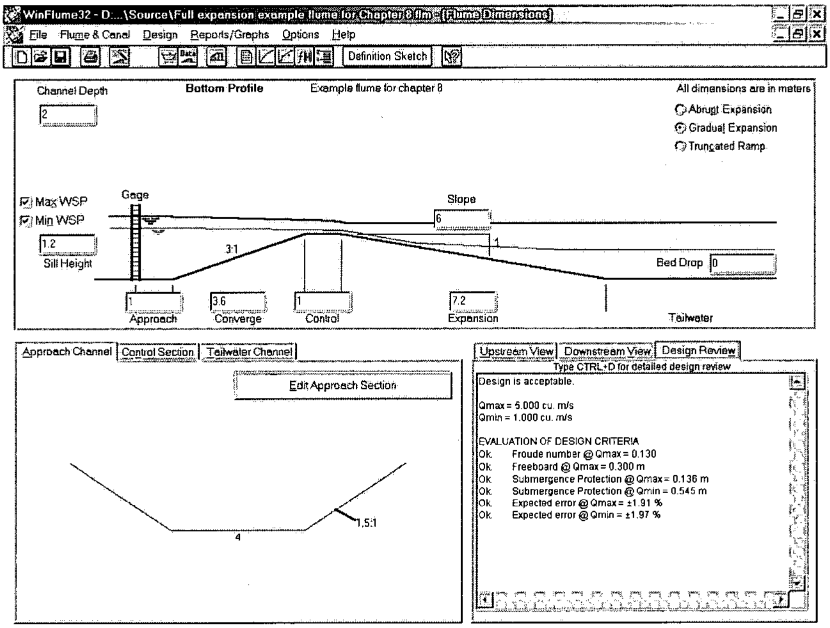


Figure 8.1 The main screen of WinFlume, displaying the bottom profile dimensions, cross-section shapes, and brief design review report.

WinFlume is operated primarily from a main screen (Figure 8.1) that provides an editable view of the bottom profile of the canal and flume structure and access to screens that display and allow editing of canal and flume cross-section shapes. Screens related to additional flume properties, design requirements, review and analysis of design alternatives, and the generation of rating tables and other output are accessed either from the menus located near the top of the screen, or from buttons located in the toolbar beneath the menus. Each toolbar button is equipped with an explanatory text tag that is displayed for 2 seconds if the mouse pointer is positioned over a toolbar button for more than 1/2 second.

8.6.1 Rating existing flumes and weirs

WinFlume can rate, or calibrate, existing structures that fit the requirements for analysis as long-throated flumes. The rating process consists of using the theory and procedures of Chapter 6 to determine the relationship between the upstream sill-reference head, h_1, and discharge, Q. The program can generate rating tables (Q versus h_1 or h_1 versus Q), curve-fit equations relating Q to h_1 for use in data loggers, and wall gage data. WinFlume can also compare field-measured Q versus h_1 data to the theoretical rating curve of a structure. All these results can be displayed in tabular or graphical form, including wall gage templates that are printed to scale. WinFlume can also be used as a design review tool to identify design deficiencies in existing or proposed structures.

The basic procedure for using WinFlume to rate an existing structure is as follows:

- Create a new flume and use the flume wizard (Section 8.7) to define the hydraulic and geometric properties of the structure. If you choose not to use the flume wizard, you may enter the dimensions, cross-section shapes and other properties of the flume and canal using the bottom-profile screen, the cross-section editing screens, and the *Flume Properties & Canal Data* form, accessed from the toolbar or the *Flume & Canal* menu.
- Open the *Rating Tables & Graphs* form using the toolbar or the *Reports/Graphs* menu. Choose the type of rating table, the range of head or discharge values to include in the table, and optional parameters to include in the rating table. Click on the *Rating Table* tab of the form to view the rating table. (Clicking on this tab actually causes WinFlume to perform the rating-table calculations. On fast computers, it appears instantaneous.) Check carefully for errors or warning messages in the rightmost column of the rating table. These may indicate errors in data entry or deficiencies in the flume design. See Section 8.9.4 for rating table details.
- Generate a curve-fit equation, wall gage plot, or comparison to measured h_1 versus Q data, if desired. (see Section 8.9)

8.6.2 Designing new flumes

The design of a new flume begins in much the same manner as the rating of an existing flume (Section 8.6.1). You define the properties and initial dimensions for the canal and flume structure, along with the design requirements for the structure. The flume dimensions may change during the design process; the initial dimensions you enter serve as a starting point for the design.

After you have defined the flume and canal geometry, design requirements, and other properties (on your own or with the aid of the flume wizard), you may use WinFlume's design evaluation reports and tools to review and refine the design to meet your design criteria and objectives. The design refinement process can be carried out manually, with the user changing flume dimensions and other properties until the design review indicates a satisfactory structure, or you can make use of the design evaluation module (Section 8.8.3). The design evaluation module will analyze a range of flume structures derived from the initial flume, and provide a report showing those flumes that are acceptable. You can then choose one of these structures and use it as is or make additional refinements to it.

Once you have obtained an acceptable structure, the various output modules of the program (Section 8.9) are used to document the design and produce rating tables, rating equations, and wall gage data or plots.

8.6.3 Input Data Requirements

For calibration of existing structures, WinFlume requires data on flume and canal dimensions and hydraulic properties. Data are entered on the bottom-profile screen, the editing screens for the three cross sections, and on the *Flume Properties & Canal Data* screen, which is under the *Flume & Canal* menu item. For design of a new structure or for review of the design suitability of an existing structure, additional data are needed, specifically the water-level measurement method, and freeboard requirement. These can be entered on the *Flume Properties, Canal Data, & Design Requirements* screen, which is under the *Design* menu item.

WinFlume requires data for some of the fields associated with design, even though the user may only be interested in the calibration of an existing flume. Some design criteria, such as accuracy and freeboard requirements, will not affect the calculation of the rating table, but may prove useful if you later wish to review the design of an existing structure.

WinFlume will not accept a tailwater level of zero, and extremely small values can be problematic (because they imply that the downstream velocity head is unreasonably large). If tailwater levels are not known, either enter a value that is about one half the upstream water depth or add a large bottom drop downstream from the throat and enter a tailwater depth approximately equal to the upstream depth. The rating table generated by WinFlume will list the highest allowable tailwater level for each flow rate.

WinFlume can accept data in several different user-selectable units systems (see Section 8.7.3). The units for data entry are generally shown to the right of the appropriate text box or list box, or at the upper right corner of the bottom-profile screen. For other items, WinFlume provides a list of possible choices and associated parameter values. For head measurement method and flume construction material roughness, a list box is provided, such that you choose items from the list or enter a value directly. In the latter case, we recommend that you enter a new description to the text field to indicate that you have not selected an item from the list. For the canal roughness coefficient (Manning's n), a special expandable box provides a wide range of choices. The list can be expanded by clicking on the + sign and values are chosen by clicking on them. Alternately a numerical value can be entered.

8.6.4 Revision tracking

WinFlume maintains a revision number for each flume defined. This revision number is printed on all program output to permit the user to confirm that output produced by the program at different times is all associated with a particular version of a flume. When a new flume is created, the revision number is set to 1, and each time the flume is modified and saved, the revision number is incremented by 1. The revision number is only incremented when properties of the flume or canal that affect the rating of the flume are modified. For example, if a flume with a revision number of 1 is loaded, and the sill height is then modified, the revision number will be incremented to 2 when the flume is saved. However, if flume revision 1 is loaded and only some details of the formatting of the wall gage are changed, then the revision number will remain 1 when the flume is saved.

You should always confirm that flume rating tables, rating equations, and wall gage data or plots display the same revision number as the flume data report and drawing printout that define the dimensions and other properties of the flume and canal. If the revision numbers are different, then the ratings are for a different flume definition; the ratings and other output should be reprinted from the most current revision of the flume design. WinFlume may at times force you to save a flume with an incremented revision number before output can be printed. This ensures that you cannot load a flume at two different times, modify it in a different manner each time, and print rating tables or other output that have the same revision number, but correspond to different flume definitions.

8.6.5 Using the undo feature

WinFlume constantly tracks modifications to the flume definition as you work with a design and maintains a list of the last ten significant modifications made to the structure. You can undo recent changes and revert back to a previous flume definition using the *Undo* command located on the *Flume & Canal* menu and the *Design* menu. The menu will describe the type of action being undone, for example, *Undo Changes to Bottom Profile* after you have changed one or more of the bottom profile dimensions.

8.6.6 Program output

Several types of output are available. A flume drawing can be printed, showing the bottom profile of the canal and measurement structure and the cross-section shapes and dimensions of the approach channel, control section, and tailwater channel. Text reports summarizing the flume design data, listing alternative designs, and reviewing the flume design relative to the design requirements are also available. Three types of flume rating tables are available, as well as a rating equation report, wall gage data report, and a report comparing the theoretical rating table to field-measured values of Q versus h_1. Finally, full-scale printed wall gages can be produced. In addition, the rating tables, rating equation report, and comparative rating table report can be presented in graphical form. All output reports and graphs (with the exception of the printed wall gages) can also be saved to a file on disk or copied to the Windows system clipboard, from which they can be pasted into word processors, spreadsheets, or other applications.

8.6.7 File handling

WinFlume stores individual flume definitions in flume files having names of the form **.flm*, where * represents the name of the flume and *flm* is the file extension. These files contain all of the geometric and hydraulic properties of the flume and the upstream and downstream canal sections, as well as all pertinent user-preferences related to design criteria and output options. These include units system settings, rating table ranges, wall gage appearance options, etc. The **.flm* file is a binary-format file, and cannot be usefully viewed with a text editor. The WinFlume program is the only useful means of viewing and editing the contents of the **.flm* file. To share a flume file with another user or transfer it to another computer, simply copy the **.flm* file to a floppy disk or onto the other computer.

Internally, the **.flm* file has three parts. The first 2 bytes of the file is a binary-coded integer that identifies the revision number of the file format. The **.flm* file format underwent several revisions during the development of WinFlume. WinFlume can read files in the current format, as well as all previous formats. Files are always saved in the latest format, so that the "upgrading" of **.flm* files to the newest format is completely transparent to the user. If you are using an older version of WinFlume, it is not possible to load a **.flm* file in a newer format. To load this file you will need to upgrade your WinFlume installation.

The next 1012 bytes of the **.flm* file is the flume data structure. This data structure is defined in the WinFlume program, and contains all geometric and hydraulic properties of the flume, as well as the flume-specific user preferences mentioned above. The third part of the **.flm* file is the array of measured h_1 versus Q data pairs used for comparing the theoretical rating curve to actual field data. If the user has not entered any h_1 versus Q data for comparison, then this third part of the **.flm* file will consist only of a single data pair with values of $h_1 = 0$ and $Q = 0$. Each pair of h_1 versus Q data adds another 8 bytes to the length of the file. Thus, the length of the **.flm* file will be at least 1022 bytes, plus the length of any additional h_1 versus Q data.

8.6.8 Loading flume designs created by FLUME 3.0

The DOS-based FLUME 3.0 program stored flume design data and rating tables in dBase format files, having an extension of **.DBF*. A catalog of flumes was maintained, and the dimensions and properties of all flumes were saved together in a single file named *FLM.DBF*. FLUME 3.0 also had a backup function that would copy one or more flumes into a backup file that could be used to transfer designs to other computer systems. This file was named *FLMBAK.DBF*. FLUME 3.0 also created other *DBF* files whose base name was derived from the name of the flume. These files only contained rating table data computed by FLUME 3.0; the flume design data was always in the *FLM.DBF* file, or its backup, *FLMBAK.DBF*.

WinFlume can import designs originally created in FLUME 3.0 from either the *FLM.DBF* or *FLMBAK.DBF* file. Once loaded into WinFlume, the designs can be

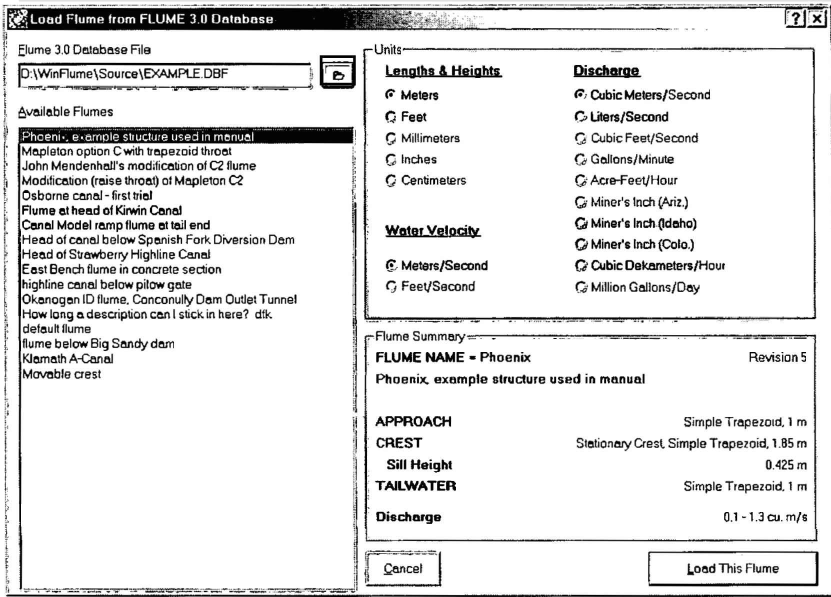


Figure 8.2 Importing flumes originally created using the FLUME 3.0 program.

modified and saved in WinFlume's standard **.flm* file format for future use. WinFlume does not make use of the rating table *DBF* files created by FLUME 3.0.

To import a flume design from FLUME 3.0, choose the *Load Flume from FLUME 3.0 Database* command on the *File* menu, or press Shift + F3. You will need to select the *DBF* file containing the FLUME 3.0 flume database. You can scroll through a list of the flumes in the *DBF* file and view a brief summary of their primary dimensions and characteristics before selecting the particular flume you wish to import. Because the FLUME 3.0 program did not store units preferences for each flume, you may need to change the units used to display the flume summary as you scroll through the list. Figure 8.2 shows the screen used to import FLUME 3.0 flume definitions.

8.7 Data Entry Using the Flume Wizard

The flume wizard leads the user through a step-by-step process that prompts for all necessary dimensions and properties of the flume and canal, as well as the user-selectable design requirements. The wizard is most useful when creating a new flume, but can be invoked at any time as a means of reviewing the input data for the flume currently in memory. The wizard is only intended to lead the user through the initial phase of entering a new flume definition into WinFlume. After the wizard has been completed, you may proceed with rating the structure, reviewing the design, or analyzing alternative designs. Figure 8.3 shows the flume wizard dialog box. While the flume wizard is active, this dialog box will stay visible on the screen as you enter data into other forms and dialog boxes.

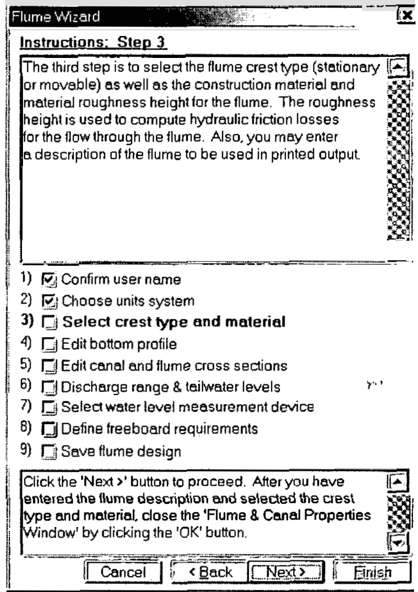


Figure 8.3 The flume wizard leads the user step-by-step through the process of defining a new flume. The upper part of the wizard dialog box describes the current step of the process, while the lower text box gives instructions. The check boxes track the user's progress.

8.7.1 Starting and using the flume wizard

The flume wizard can be started from the toolbar, the *File* menu, or the *Design* menu. If you wish to create a new flume using the wizard, choose *New Flume* from the *File* menu, and check *Use the Step-by-Step Flume Wizard* in the *Create New Flume* dialog box. If you wish to use the wizard to review the dimensions, properties and design requirements for a flume definition that is already in memory, choose *Flume Wizard* from the *Design* menu, or click the button on the toolbar.

To use the wizard, read the text in the box at the top of the form, which describes the purpose of each step, and follow the specific instructions given in the text box at the bottom of the form. As each step is completed, a check mark will appear to the left of the item. The last step of the wizard is saving the flume definition.

8.7.2 Confirm user name

The first step of the flume wizard prompts the user to enter their name. This user name becomes a setting of this WinFlume installation and will be printed on all flume reports produced by the program. The user name will be saved and recalled for use in future WinFlume sessions if the *Save Settings on Exit* menu item is checked on the *Options* menu. The user name can be changed at any time by selecting *User Name* from the *Options* menu.

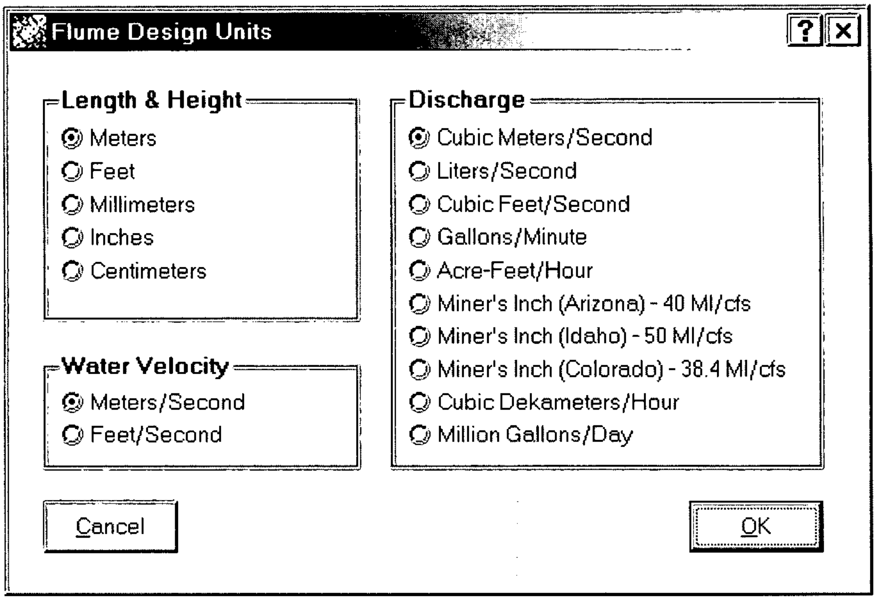


Figure 8.4 Choices for input and display units.

8.7.3 Choose units system

WinFlume works with and stores all flume designs internally using SI units, but you can choose other units for display and input of flume and canal length dimensions, water velocities, and discharge. The dialog box shown in Figure 8.4 allows you to choose the system of units used for displaying and entering length, velocity, and discharge data. To facilitate the use of consistent sets of length and velocity units, the choices made in the length unit category affect the choice in the velocity unit category. For example, if meters or millimeters are chosen for length units, the velocity units will automatically be set to meters/second, and if feet or inches are chosen for length units, the velocity units will be set to feet/second. However, to allow the use of non-consistent units if desired, the choice made in the velocity category does not affect the choice in the length category. Thus, to use feet as the unit of length and meters/second as the unit of velocity, select the length units first, then select the velocity units. The selection for discharge units is always independent of the length and velocity units choices.

Note that there are three different miner's inch units available. The miner's inch is a traditional unit of water flow in the western United States and is still used in some localities. The miner's inch originally represented the flow through an opening one inch square driven by a specified water pressure. Today, conversion factors defining the miner's inch relative to other flow units are established by statute and vary by state as follows:

- Idaho, Kansas, Nebraska, New Mexico, North Dakota, South Dakota, Utah, and Washington - 9 U.S. gallons per minute, or about 1/50 ft^3/s (0.568 liters/s);

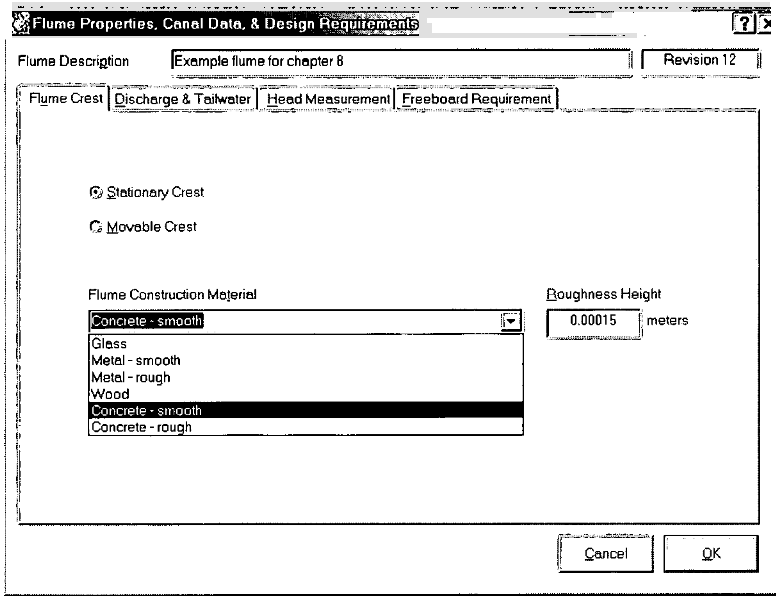


Figure 8.5 Selecting the type of crest and specifying construction material and roughness height.

- Arizona, California, Montana, Nevada, and Oregon - 1.5 ft^3/min, or 1/40 ft^3/s (0.708 liters/s); and
- Colorado - 1.5625 ft^3/min, or 1/38.4 ft^3/s (0.738 liters/s).

8.7.4 Select crest type and material

Figure 8.5 shows the dialog box used to enter the flume crest type and construction material data. The structure may be either a stationary-crest flume used only for flow measurement or a movable-crest weir used for both flow measurement and control. This choice will determine the appearance of the bottom-profile screen later used to enter the dimensions of the structure. You may also enter a text description of the flume on this screen, which will appear on printed reports, rating tables, and drawings produced by the program.

The user should specify the material that will be used to construct the flume or weir and the corresponding absolute roughness height, k, of that material. This roughness height will be used by WinFlume to compute the head losses due to friction between the gaging station and the tailwater channel. These head losses ultimately affect the relation between h_1 and Q and the total head loss through the structure. You may choose a material and roughness height from the list box, or you may type your own description of the material and enter its roughness height. The material you specify should be that used to construct the flume crest, not the material used to construct the canal.

The absolute roughness heights for a number of materials typically used for flume construction are given in Table 8.2. An analysis of the effects of roughness height

Table 8.2 Absolute roughness height, k, of materials used in flume construction.

Material	*Range of k (meters)*
Glass	0.000 001 to 0.000 010
Metal, painted or smooth	0.000 020 to 0.000 10
Metal, rough	0.000 10 to 0.001 0
Wood	0.000 20 to 0.001 0
Concrete, smooth trowelled	0.000 10 to 0.002 0
Concrete, rough	0.000 50 to 0.005 0

showed that a change of several orders of magnitude in the value of k produces less than 0.5% (often less than 0.1%) change in discharge for a given upstream head. Thus, a change in materials from smooth glass to rough concrete will have a minor effect on the discharge rating of the flume. This minor effect, however, should not be used as an excuse for sloppy or poor construction. If the surfaces in the control section have large undulations and irregularities it can affect the cross-sectional area of the section at which critical flow occurs, and the real discharge can differ considerably from the theoretical value. Material roughness variability and construction tolerances should be regarded as different sources of potential error (see Section 2.8).

8.7.5 Edit bottom profile

Once the structure type has been selected, you are ready to edit the bottom profile of the structure. The wizard window will move to the lower part of the screen to make the bottom profile visible. As you edit the bottom profile dimensions, the drawing will be updated so that you always see a properly scaled view of the structure. For a stationary-crest flume, the parameters needed to define the bottom profile are (Figure 8.6)

- Upstream channel depth measured from the invert to the top of the lining or top of channel banks,
- Approach distance from gaging station to start of converging transition,
- Length of converging transition,
- Sill height measured from the invert of the approach channel to the invert of the control section,
- Length of control section,

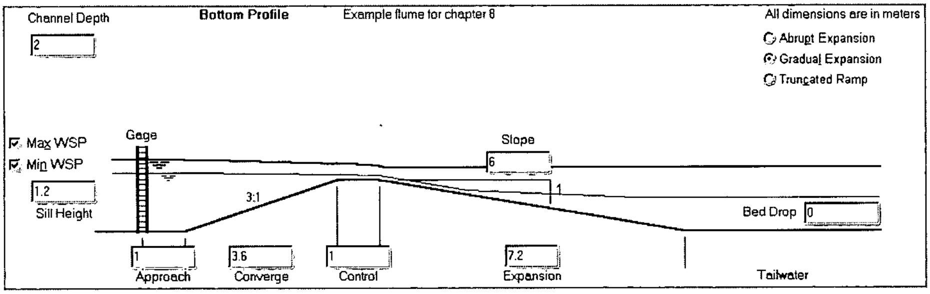


Figure 8.6 Editing the bottom profile of a stationary-crest flume.

- Slope or length of the optional diverging transition, and
- Bed drop from approach channel to tailwater channel (a negative bed drop can be entered if there is an increase in bed elevation).

For bottom-contracted flumes, either the slope or length may be entered to define the diverging transition; WinFlume will compute the associated length or slope. For structures having just a side contraction, only the slope (expansion ratio) may be entered, and WinFlume estimates the length of the transition based on the lateral expansion that must occur from the throat section to the sidewalls of the tailwater channel at the elevation of the invert of the tailwater channel.

In addition, the user may select one of three configuration options for the diverging transition:

- Abrupt expansion – a sudden expansion with no diverging transition,
- Gradual expansion – a full-length diverging transition that extends downstream until it intersects the invert of the tailwater channel, or
- Truncated ramp – the diverging transition extends half the length of a full-length transition (see Figure 6.24).

A diverging transition is needed if the total head loss through the structure is to be kept to a minimum. Hydraulically, the full-length and truncated transition ramps perform identically. A full-length transition may be desirable if it is necessary to drive equipment upstream across the structure during maintenance operations. A truncated ramp yields a reduced structure length and perhaps a construction cost savings. A truncated ramp should not be rounded at the downstream end, as discussed in Section 2.2 and Section 6.6.2 (see Figure 6.24).

For a movable-crest structure, the following information is entered on the bottom-profile screen (Figure 8.7):

- Upstream channel depth;
- Operating depth to be maintained in the upstream channel;
- Approach distance from the gaging station to the start of the movable weir;
- Radius of the leading edge of the movable weir, which ensures a smooth flow transition from the approach channel to the control section;

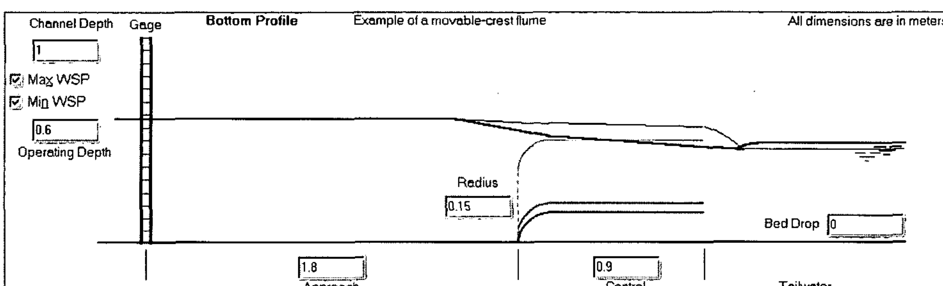


Figure 8.7 Editing the bottom profile of a movable weir.

- Length of the horizontal portion of the movable weir, which serves as the control section of the structure; and
- Bed drop from approach channel to tailwater channel (negative bed drop if there is an increase in bed elevation).

The bottom-profile screen allows the user to optionally display approximate water surface profiles through the structure at minimum and maximum flow. To enable this feature, check the *Min WSP* and *Max WSP* boxes on the bottom profile view. During initial data entry these profiles may not be very helpful, since the user has not yet completely specified discharge ranges, tailwater levels, and other data needed to compute the water surface profiles. If a water surface profile cannot be computed and shown, the check box will be grayed out until it can be computed. The display of water surface profiles can be enabled at any time.

The user should note that while editing the bottom profile as part of the flume wizard process, it is possible to also access the form used to edit the channel cross sections (see following section). When designing a broad-crested weir for a lined canal section, it may be desirable to edit the cross sections before editing the bottom profile, because doing so will simplify the later refinement of the bottom width of the control section. The simplest technique is to set the sill height to zero, then define the approach channel and control section shapes and dimensions to be identical. When the user then returns to the bottom-profile screen and increases the sill height, WinFlume will adjust the base width of the control section so that the sill spans the full width of the approach channel. WinFlume will continue to adjust the base width of the control section when the sill height is changed, as long as the user does not manually set the control section base width to be wider or narrower than the approach channel.

8.7.6 Edit canal and flume cross sections

Cross-section shapes and dimensions for the approach channel, control section, and tailwater channel are shown in the tabbed displays in the lower left corner of the main WinFlume screen. Cross-section shapes and dimensions are edited using the form shown in Figure 8.8. The list box in the upper left corner of the form is used to choose the cross-section shape. The center of the form shows a scale drawing of the cross-section, with text boxes in which dimensions can be entered. A thumbnail sketch of the basic cross-section shape and its parameters is shown in the lower right corner of the form. For some of the more complex shapes, it may be possible to enter dimensions that make some parameters of the shape unnecessary. For example, if the inner base width of a complex trapezoid is wider than the outer trapezoid, the designation of side slopes for the inner section is not needed. In this case, the values of the unused dimensions are moved to the right side of the form, where they can still be edited.

The seven shapes shown on the left hand side of Figure 8.9 are available for use in the approach and tailwater sections, and all fourteen shapes shown in the figure are available for use in the control section of the structure, except on a movable-crest

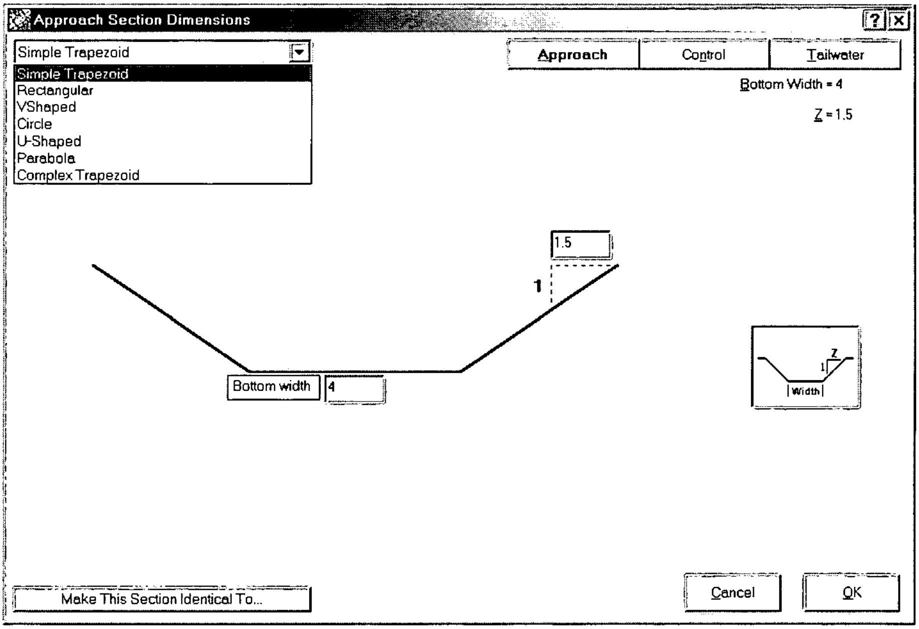


Figure 8.8 Editing cross-section shapes and dimensions.

flume, which is restricted to either a rectangular or V-shape-in-rectangle control section. Access to all three cross-sections of the structure is provided with the command buttons in the upper right corner of the section-editing form. After you have finished editing the cross-sections, you may wish to refer to the upstream and downstream views of the structure shown on the tabs located in the lower right corner of the main WinFlume screen. These views make it easy to confirm that the cross-section shapes and dimensions you have entered are physically reasonable (e.g., the control section should not be wider than the approach section, etc.).

When editing the bottom width of the control section, WinFlume will sometimes display a button labeled *Match to Approach Channel Width.* Clicking this button will cause the bottom width to be set equal to the approach channel width at the present sill height, yielding a structure that has no width contraction at the invert of the control section. Once the bottom width of the control section is set to match the approach channel width, if the user then raises or lowers the sill height (on the bottom profile drawing), WinFlume will automatically adjust the width of the control section so that it continues to match the width of the approach channel.

8.7.7 Define discharge range and tailwater levels

The range of flows to be measured and the expected tailwater conditions at the site are important parameters affecting the design of a flow measurement structure. The user enters these data on the second tab of the *Flume Properties & Canal Data* form, shown in Figure 8.10. The user should enter the minimum and maximum flow, Q_{min}

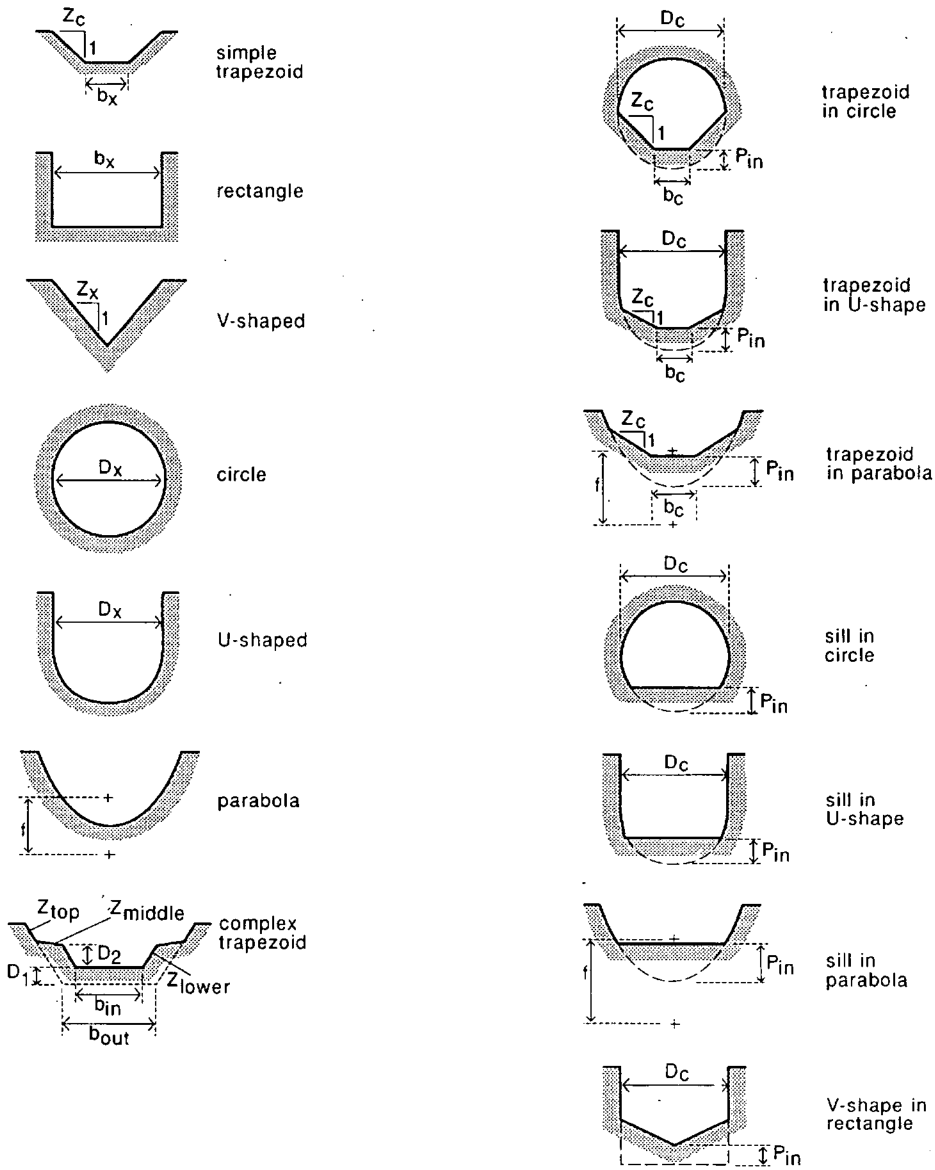


Figure 8.9. — Alternative shapes for the approach and tailwater channels (left column) and for the control section (both columns).

and Q_{max}, for which accurate discharge measurements are needed. The design of the structure will later be evaluated at these two discharges. The range of flows to be measured will have a major influence on the shape of the control section and the elevation of the sill relative to the invert of the tailwater channel. Irrigation canals will usually operate so that the ratio of Q_{max}/Q_{min} is less than 10 and a simple cross-section shape can be used. If the range of flows is large ($Q_{max}/Q_{min} > 35$, see Section 2.4), a non-rectangular control-section shape is required, so it is important to choose a reasonable range of flows. Natural channels and drains may experience a wide range of flows requiring a compound control-section shape. Existing hydrologic data should be used whenever possible to determine the range of flows to be used for the design.

Tailwater data at Q_{min} and Q_{max} are used to ensure that the flume does not become submerged and operates with modular flow (i.e., critical depth in the control section) over the full discharge range. WinFlume's design review report only evaluates the design at Q_{min} and Q_{max}, but rating tables generated by WinFlume will be checked for modular flow at all flow rates in the table, with tailwater values interpolated for intermediate flow rates using the tailwater model specified by the user. Five different methods for specifying tailwater levels are provided:

- Manning's equation using n and S,
- Manning's equation using one Q versus y_2 measurement,
- Power curve using two Q versus y_2 measurements,
- Power curve with offset using three Q versus y_2 measurements, and
- Linear interpolation/extrapolation from Q versus y_2 lookup table.

Two of the methods are based on the assumption that flow downstream from the structure is at normal depth and can thus be modeled using the Manning equation. This requirement may be met if there is a long section of uniform channel at a uniform slope and uniform roughness downstream from the site. A backwater analysis using a tool such as the HEC-RAS model may be necessary to confirm that the normal depth assumption is valid. If the flow is not at normal depth (e.g., channel slope varies, there is a change in cross-section shape or size, or there are downstream structures that create backwater), then one of the other three methods should be considered. In these cases, the designer must determine tailwater levels corresponding to a range of flows, either through field surveys or a detailed hydraulic analysis of the backwater profile in the downstream channel. These data can be used by WinFlume to extrapolate tailwater conditions over the full range of flows for which the flume will operate. When in doubt, one should attempt to obtain field measurements of tailwater levels if possible.

When applying any of the tailwater models, it is important to recognize that tailwater levels at a site may vary seasonally, rising as vegetation density in and along the channel increases during the growing season. To ensure that the measurement structure functions properly throughout the year, the tailwater data provided to WinFlume should correspond to the highest expected seasonal levels.

To specify tailwater levels, the user chooses one of the five tailwater models, and enters the appropriate data. For those methods requiring estimates of Q and y_2, it is desirable, but not necessary, to provide tailwater data at flows close to Q_{min} and Q_{max}. WinFlume will use the data provided to compute tailwater levels at Q_{min} and Q_{max} and enter the result into the text boxes in the upper part of the form (Figure 8.10). If tailwater levels cannot be calculated using the data provided, the tailwater levels will be shown in red as 0.000. The user should correct the input data until tailwater levels are properly computed.

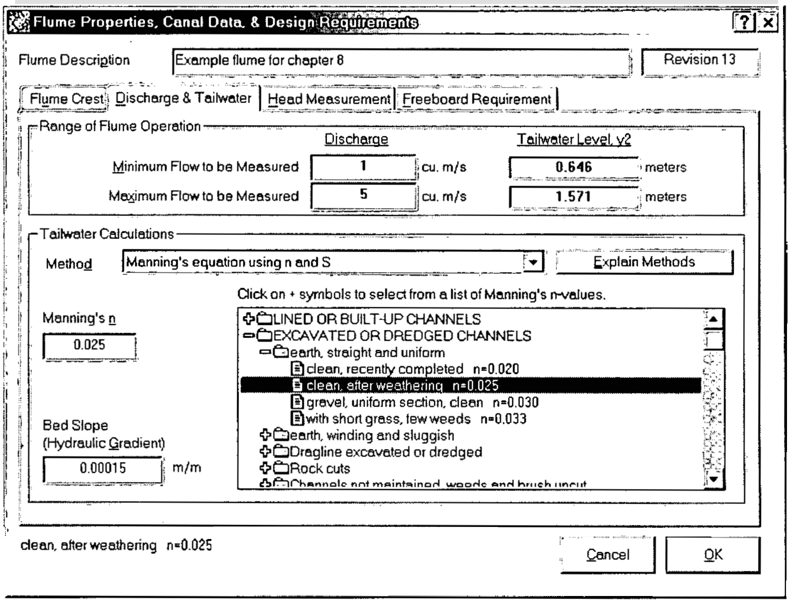


Figure 8.10 Using Manning's equation to specify tailwater levels.

Manning's equation

If the flow in the tailwater channel is at normal depth, then Manning's equation can be used to model the tailwater levels. This flow condition occurs when the downstream channel is of uniform section, slope, and roughness for a sufficient distance that the flow depth at the head of the tailwater channel (i.e., the downstream side of the flume) is solely a function of the section shape, channel slope, and the roughness coefficient (Manning's *n*) as given by (Equation 2.9)

$$Q = \frac{C_u}{n} A R^{2/3} S_f^{1/2}$$

where Q, A, R, and S_f are the discharge, wetted area, hydraulic radius, and friction slope or hydraulic gradient (equal to the bed slope if flow is at normal depth), respectively. C_u is a units coefficient that has a value of 1.0 when using units of meters and m^3/s, and 1.486 when using units of feet and ft^3/s. To apply this method the user provides values of n and the hydraulic gradient, S_f. WinFlume solves for the tailwater level, y_2, corresponding to Q_{min} and Q_{max}. The equation must be solved numerically for most channel shapes. For convenience, a database of suggested n-values for a variety of natural and constructed channels is provided in WinFlume. The list contains maximum n-values from Chow (1959). The user may choose from this list, or enter an n-value directly. To expand the list (Figure 8.10), click on the + symbol to the left of each folder symbol.

Manning's equation using one Q - y_2 measurement

This method is similar to that described above, except that values of Manning's n and the hydraulic gradient are not needed. Instead, Equation 2.9 is rewritten (Equation 5.4)

$$\frac{C_u S_f^{1/2}}{n} = \frac{Q}{AR^{2/3}}$$

The user supplies known values of the discharge and downstream tailwater level for one known flow rate. These data are used to compute the value of the ratio $C_u S_f^{1/2}/n$, which will be a constant for all flow conditions (C_u, S_f and n are each constants themselves for a given site). Once the value of $C_u S_f^{1/2}/n$ is determined, Manning's equation can be used to compute the tailwater level at any other discharge.

Power curve using two Q - y_2 measurements

The power curve using two Q - y_2 measurements is based on an empirical equation relating the discharge, Q, and the flow depth, y_2 (Equation 5.2)

$$Q = K_1 y_2^{u}$$

in which K_1 and u are empirical coefficients. The value of K_1 varies with the size of the channel, and the value of u depends on the shape of the channel, normally varying from about 1.6 for shallow, wide channels, to 2.4 for narrow, deep channels. With values of discharge and tailwater depth at two different flow rates, the values of K_1 and u can be determined, and the equation can then be used to extrapolate tailwater levels at other flow rates. Note that the use of this equation form implies that at a discharge of zero, the tailwater level is also zero.

The values of Q and y_2 can be determined from field surveys or through a detailed hydraulic analysis. If field surveys are used, the designer should ensure that the data are collected under operating scenarios and seasonal channel roughness conditions that produce the highest possible tailwater levels. To minimize the degree of extrapolation, it is best to obtain data at flows as near as possible to Q_{min} and Q_{max}. Figure 8.11 shows the data input form used when determining tailwater levels with this method.

Power curve with offset using three Q - y_2 measurements

The power curve using two Q - y_2 measurements, described above, assumes that the tailwater level is zero at a discharge of zero. In some cases, this may not be true, such as when there is a weir or other similar structure a short distance downstream from the site. In this case, the tailwater level may be non-zero when the flow rate is zero. The power curve with offset addresses this problem by modeling the downstream tailwater relationship using the empirical equation (Equation 5.3)

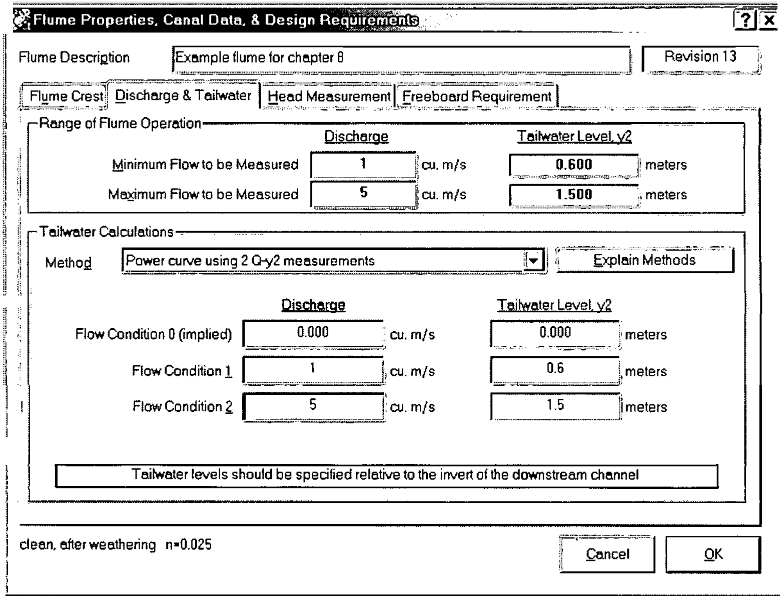


Figure 8.11 Specifying the discharge range to be measured and the tailwater levels at the site of the structure.

$$Q = K_1 (y_2 - K_2)^u$$

in which K_2 is a constant indicating the offset, or tailwater level at zero flow. To apply this method, the user must provide values of discharge and tailwater depth for three different flow conditions. To allow determination of K_2, the tailwater depth at zero flow must be one of those provided. To reduce the degree of extrapolation, tailwater levels at flows close to Q_{min} and Q_{max} are desirable for the other two data points. WinFlume uses the data to solve for K_1, u, and K_2, and can then compute tailwater levels corresponding to other flow rates.

As with the previous method, the values of Q and y_2 can be determined from field surveys or through a detailed hydraulic analysis. If field surveys are used, the designer should ensure that the data are collected under operating scenarios and seasonal channel roughness conditions that produce the highest possible tailwater levels.

Linear interpolation of Q - y_2 lookup table

The linear interpolation method can be used to specify unusual tailwater curves that the power curve methods may not approximate well, such as the unusual situation of a tailwater curve that is constant or decreasing as discharge increases. The user provides a lookup table of Q versus y_2 data points, and the program interpolates between them (or extrapolates beyond the range of the first or last point) to determine the tailwater levels at specific discharges. The values of Q and y_2 can be determined from field surveys or through a detailed hydraulic analysis. If field surveys are used,

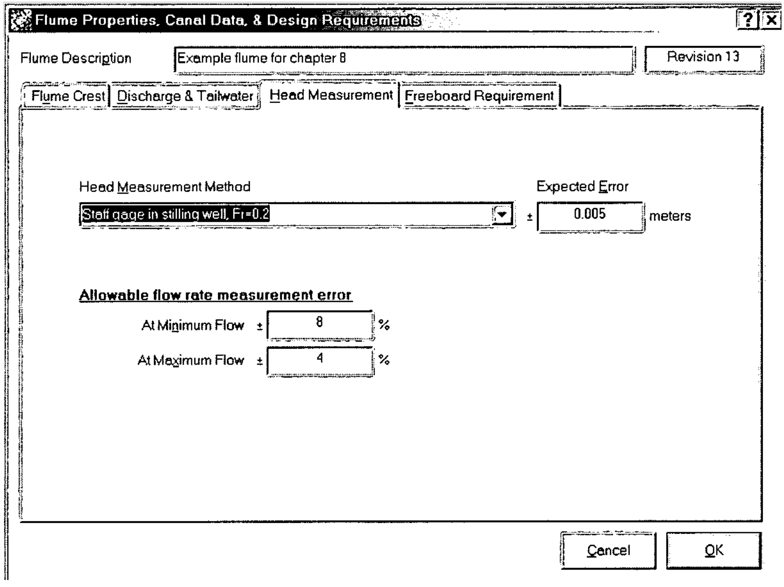


Figure 8.12 Specifying the allowable discharge measurement errors and the head measurement method.

the designer should ensure that the data are collected under operating scenarios and seasonal channel roughness conditions that produce the highest possible tailwater levels. To minimize the degree of extrapolation, it is best to collect some data at flows near Q_{min} and Q_{max}. A maximum of 20 pairs of Q - y_2 data can be entered.

8.7.8 Select water level measurement device and allowable flow measurement error

WinFlume evaluates flume designs to ensure that the combined errors due to uncertainty in the rating of the structure and uncertainty in the measurement of the upstream sill-referenced head do not exceed an allowable level specified by the user. The head measurement tab (Figure 8.12) of the *Flume Properties, Canal Data & Design Requirements* form is used to specify the allowable discharge measurement errors, the head measurement method, and its expected error. The default values of allowable discharge measurement error at Q_{min} and Q_{max} are ±8 and ±4 percent, respectively, and the user can modify the values as desired. The allowable measurement error should always be 2 percent or greater, since the uncertainty in the rating table itself is about 2 percent (see Sections 6.5.3 and 2.8).

WinFlume provides a list box containing descriptions of several commonly used head measurement methods. For each method, a default value of the expected head measurement error is provided. The user can choose a measurement method from the list, or type in their own description of the head measurement method. The user may also enter a different value of the expected error, if desired. The value should be the expected error in any one single measurement of the sill-referenced head due to factors such as waves, difficulty seeing the staff gage or water surface, electronic noise, resolution of the device, etc. This can also be considered to be the 95 percent confidence interval of the head measurement. Table 4.1 provided recommended values for commonly used head measurement methods.

The random error, X_Q, in one single measurement of the flow rate is computed from (Equation 2.14)

$$X_Q = \sqrt{X_C^2 + (uX_{h1})^2}$$

in which X_C is the relative uncertainty in the rating table generated by WinFlume, u is the exponent in the head versus discharge equation, and X_{h1} is the relative uncertainty in the measurement of upstream sill-referenced head. WinFlume assumes the expected error in the head measurement to be constant and independent of the head itself, so the relative uncertainty, X_{h1}, can become large at low flow, when h_1 is small. It is important to realize that most errors contributing to X_C and to X_{h1} have a random distribution. Hence, if many (e.g., 15 or more) discharge measurements are made to calculate the volume of water that passes a structure during a period of time (day, week, etc.), these random errors tend to cancel out and can be neglected with respect to the measurement of total volume of flow. As a result, the error in the measured volume of flow is due to systematic errors only. Of these, the errors in zero-setting are the most common (see Section 4.9). Flume construction and calibration errors also produce systematic errors in flow measurements.

8.7.9 Define freeboard requirements

Freeboard is required in channels to prevent overtopping of the channel banks due to variations in flow depth caused by waves, changes in channel roughness, and variability of flow rates. A flume or weir added to an existing channel will usually cause an increase in flow depth upstream from the site of the structure due to the added head loss across the structure, and this will reduce the available freeboard in the channel. Freeboard requirements in irrigation canals are often a percentage of the total flow depth. However, upstream from a flume or weir, it is acceptable to reduce the required freeboard to a percentage of the head on the structure, since water levels upstream from the structure will be less variable (see Section 2.3). WinFlume evaluates flume designs to ensure that the structure will not reduce the freeboard in the upstream channel to less than a required value specified by the user. The freeboard requirement can be expressed either as a percentage of the upstream sill-referenced head (20% is the default), or as an absolute distance (Figure 8.13).

8.7.10 Save flume design

The last step of the flume wizard is to save the flume definition in a **.flm* file. You will be prompted for a file name. When the flume has been saved, the wizard will end and the user can proceed with reviewing and modifying the design (Section 8.8) or rating the structure and producing output (Section 8.9).

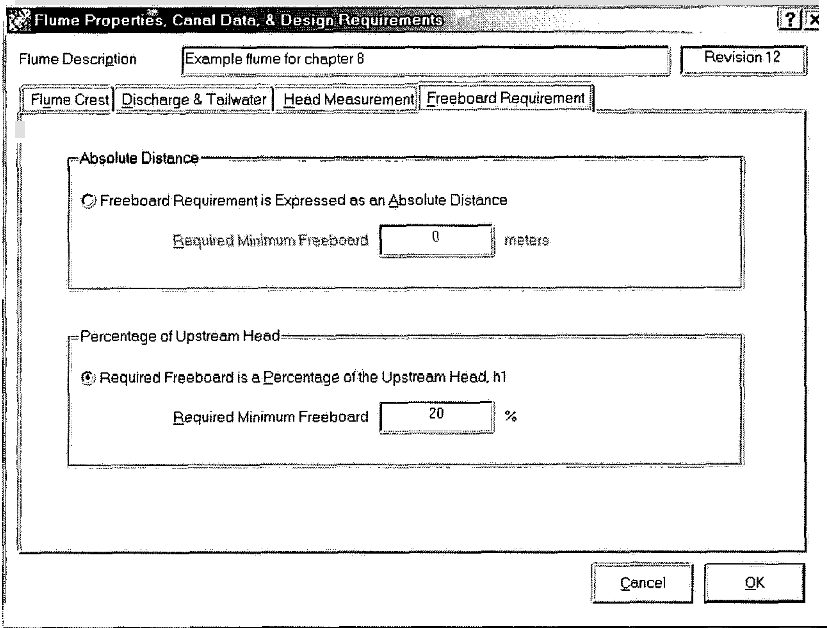


Figure 8.13 Defining freeboard requirements for the upstream channel.

8.8 Flume Design

The user may approach the problem of flume design in two ways:

- Automated analysis of alternative designs derived from the user's initial structure, and potentially some refinement of a selected design with assistance from the design review report and rating tables; or
- Iterative use of the design review report to manually change the design until the design criteria are satisfied.

Before discussing WinFlume's design tools, we will first review the design criteria and other issues common to both approaches.

8.8.1 Design criteria

WinFlume uses six design criteria to evaluate the suitability of a given structure for flow measurement. In the automated design module, four of these are considered primary criteria that must be satisfied within the design module:

- Froude number at maximum discharge ≤ 0.50,
- Freeboard at maximum discharge $\geq$ required freeboard,
- Submergence at minimum flow (actual tailwater $\leq$ allowable tailwater), and
- Submergence at maximum flow (actual tailwater $\leq$ allowable tailwater).

In addition, there are two secondary design criteria related to the flow measurement accuracy of the structure:

- Flow measurement accuracy at minimum flow meets design requirement and
- Flow measurement accuracy at maximum flow meets design requirement.

These are considered to be secondary criteria because a structure that does not meet the accuracy requirement can be improved by choosing a better head-measurement method, which does not affect the design of the structure itself. The design review report evaluates all six design criteria, without making a distinction between primary and secondary criteria.

Froude number at maximum discharge

To obtain a reasonably smooth water surface whose elevation can be measured accurately, the Froude number in the approach channel, Fr_1, should not exceed 0.5 over a distance of at least thirty times h_1 upstream from the structure. If feasible, the Froude number should be reduced to 0.2 within the approach section to the flume.

Freeboard at maximum discharge

Unless a measuring structure can be located at an existing drop in the canal system, the installation of a measuring structure in an existing canal will increase the water level upstream from the structure due to the head loss required to maintain modular flow conditions. This increased water level will reduce the available freeboard in the upstream channel. For the reasons discussed in Section 2.3, the freeboard requirement upstream from a flume or weir can usually be relaxed, but some freeboard must still be maintained. The user specifies the freeboard requirement on the *Flume Properties, Canal Data & Design Requirements* form, and WinFlume evaluates the design to ensure that this freeboard is maintained at maximum flow.

Submergence (actual versus allowable tailwater)

To maintain modular flow through a measuring structure (i.e., critical depth at the control section), there must be some head loss through the flume due to the energy losses caused by friction and the expansion of the flow in the sudden or gradual transition downstream from the throat. WinFlume computes these required head losses and uses them to determine the maximum allowable water level in the tailwater channel. WinFlume's design review compares the actual tailwater levels specified by the user (see Section 8.7.7) to the allowable tailwater levels at minimum and maximum flow. The design review assumes that if the tailwater levels are acceptable at Q_{min} and Q_{max} they will also be acceptable at intermediate flow rates. This will be true in nearly all cases, and can be verified when a detailed rating table is generated. The allowable tailwater level is computed for each flow rate in the rating tables created by WinFlume.

Required discharge-measurement accuracy

WinFlume evaluates flume and weir designs to ensure that they will meet the required discharge-measurement accuracy specified by the user (see Section 8.7.8). The accuracy of any one measurement of discharge is a function of the combined uncertainties in the Q versus h_1 rating generated by WinFlume and the uncertainty in the measurement of the upstream sill-referenced head, h_1. The uncertainty in the rating table is assumed to be ±1.9% if the ratio of H_1/L is in the range 0.07 to 0.7 and increases to ±4% at H_1/L ratios of 0.05 or 1.0. For H_1/L ratios below 0.07 or above 0.7, the rating table uncertainty is (Equation 6.44)

$$X_C = \pm\left(1.9 + 742\left(0.07 - \frac{H_1}{L}\right)^{1.5}\right) \qquad H_1/L < 0.07$$

$$X_C = \pm\left(1.9 + 12.78\left(\frac{H_1}{L} - 0.7\right)^{1.5}\right) \qquad H_1/L > 0.7$$

The rating table uncertainty increases rapidly for H_1/L values outside of the range 0.05 to 1.0, thereby discouraging the design of flumes or weirs that routinely operate outside of that range. This is consistent with the results from laboratory testing discussed in Section 6.4.1.

For a given flume and a particular upstream head, the difference between the actual discharge and predicted discharge obtained from the Q versus h_1 rating table is a systematic error. However, this systematic error is different at different heads, varies from flume to flume, varies with construction anomalies, and is otherwise unpredictable for an individual flow measurement. For this reason, the systematic rating table error is considered to be a random error when evaluating the combined uncertainty of the flow measurement.

The relative uncertainty, X_{h1}, in the measurement of h_1 is the ratio of the expected error (see Section 8.7.8) for the chosen head measurement method, and the predicted value of h_1 for a given flow rate. WinFlume uses Equations 4.1 and 4.2 to calculate an acceptable value of X_{h1} and h_1 required to meet the user's accuracy requirements at Q_{min} and Q_{max}. If the accuracy requirement is not met, WinFlume will suggest narrowing the width of the control section, which increases h_1 and reduces X_{h1}, thereby improving the accuracy.

8.8.2 Head loss objectives and tradeoffs

When designing a new measurement structure, a range of possible designs may satisfy the six design criteria discussed in Section 8.8.1. This range will be determined by two limiting design criteria out of the six. One of the design criteria will establish a lower limit on the amount of contraction required in the control section of the structure (i.e., the reduction in flow cross-sectional area from the approach channel to the control section), while the other will set the upper limit on the amount of allowable contraction. The maximum allowable contraction will always be established by the freeboard requirement, while the required minimum contraction will usually be established by one of the three other primary design criteria:

- The Froude number must be less than 0.5 at Q_{max},
- The flume must not be submerged at Q_{min}, and
- The flume must not be submerged at Q_{max}.

In some cases the measurement accuracy criteria may control the required contraction in the design, but more often these criteria only determine the shape and/or minimum required width of the control section, not the overall contraction.

The most common case is for the range of possible flume designs to be controlled by the required freeboard and the allowable tailwater at maximum flow. If we consider the characteristics of the possible designs falling between these two extremes, we find that the design having the minimum contraction will have the minimum possible head loss. This may be desirable when designing a flume to be added to an existing canal system with minimal available head. However, this design is also the most susceptible to submergence if actual tailwater levels at the site are higher than those assumed by the designer. In fact, if the lower contraction limit was established by one of the submergence criteria, then the actual tailwater and allowable tailwater are the same at either Q_{min} or Q_{max} (whichever flow controlled the design) and any increase in tailwater levels will cause the flume to be submerged. By contrast, the design having the maximum amount of contraction will have the greatest head loss of all the designs and will have no additional freeboard at Q_{max} beyond the required freeboard (see Section 8.7.9). This design will be the least susceptible to submergence in case of higher than expected tailwater levels, because the additional contraction causes the allowable tailwater to be much higher than the actual tailwater. One disadvantage of the maximum head loss design is that it creates the deepest pool upstream from the structure and the backwater influence from the structure extends for the largest distance upstream. The difference between the allowable and actual tailwater levels is listed in WinFlume's design evaluation table (see Section 8.8.3) as the *submergence protection*. It can be thought of as a safety buffer protecting the flume from submergence caused by higher than expected tailwater levels.

In addition to the minimum and maximum head loss designs, there are two other specific head loss objectives that the designer may wish to consider. The first is an intermediate head loss design, which balances the submergence protection and the additional freeboard at maximum flow so that they are equal. This design is probably the preferred design in most cases because it allows for some uncertainty in tailwater levels without creating unnecessary backwater upstream from the structure. The other specific head loss objective at a site where there is an existing drop in the channel bed is to match the head loss across the flume to the bed drop. Such a design will cause no increase in upstream water level and is thus a good choice in a channel conveying sediment. WinFlume will identify the design for which the head loss at Q_{max} matches the bed drop, if such a design exists (the bed drop must be at least as

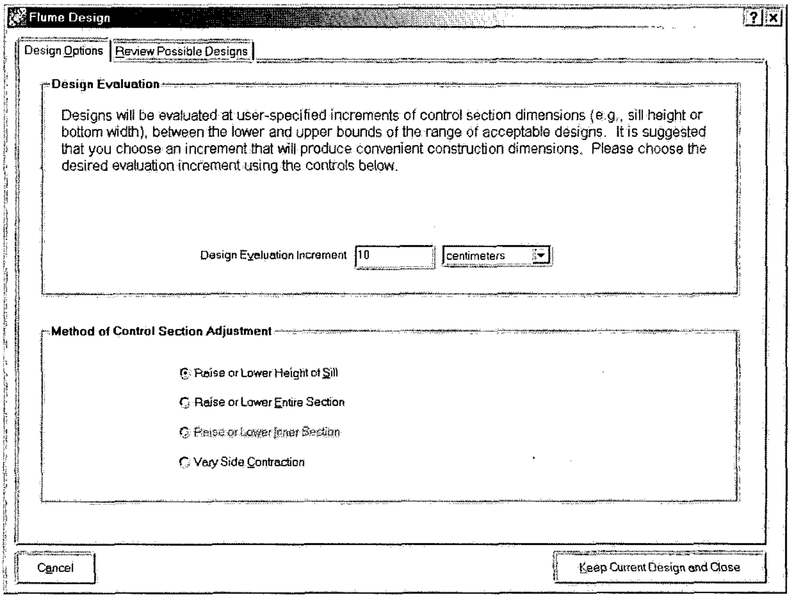


Figure 8.14 The Design Alternatives form, showing input required on the first tab of the form.

large as the minimum required head loss across the structure). If the designer wishes to also have the head loss match the bed drop at Q_{min}, this must be accomplished by trail-and-error modification of the control section shape.

8.8.3 Analyzing alternative designs

Once the user has entered initial dimensions, flume and canal properties, and design requirements, the design evaluation module of WinFlume can be used to evaluate alternative designs derived from the initial design. These alternative designs will have control section shapes similar to the initial design, but with more or less contraction. The product of the analysis will be a report listing the range of possible designs that meet the design criteria. The design module only addresses the question of what control section shape and dimensions are needed to produce a structure that meets the design criteria. Other refinements of the design are made separately (e.g., adjustments to the lengths of the approach channel, throat, and transition sections of the structure).

The *Flume Design* form (Figure 8.14) is opened by selecting *Evaluate Alternative Designs* from the *Design* menu, or by clicking the button on the toolbar. On the first tab of the form, the user must specify a method of contraction change and the length increment by which the contraction should be adjusted to create each alternative design. The second tab on the form displays the results of the analysis. If the entered value for the design evaluation increment is increased, fewer acceptable results will be shown within the range of possible designs. It is recommended to use an incremental value that corresponds with convenient construction dimensions for the flume. WinFlume will not accept increments greater than $0.05y_1$.

Methods of control section adjustment

Contraction of the flow cross-section in the form of a reduced width or raised sill height is needed to produce critical-depth flow in the throat section of a flume (see Section 5.4.1). Starting from the initial design, WinFlume's design module will further increase and decrease the control-section contraction by varying the sill height or the width of the control section. Three different methods for varying the sill height are available, making a total of four different methods of contraction change for stationary-crest flumes. For movable weirs the only contraction change option is to vary the width of the control section (vary side contraction). None of the four contraction change strategies will vary the side slopes of the control section. There may be situations where a change in side slope will be needed to find a feasible design; such a change must be made manually. The design module also will not add or modify a diverging transition, so if you have difficulty obtaining a satisfactory design, you may wish to manually add a diverging transition and repeat use of the design module.

The choice of the contraction change method will determine the configuration and dimensions of the final structure. For most design situations there is a range of dimensions and configurations that will produce acceptable designs, so the designer should choose the contraction change method with some consideration for the type of structure that is desired and that can be conveniently constructed at the site. In some cases, the designer may have to try more than one contraction method or a different initial structure design before arriving at an acceptable structure. For example, if you start with no contraction, simply raising the sill or varying the side contraction from that starting point may not provide an adequate design, but manually adding a low sill and then varying the side contraction may give an acceptable solution.

The contraction change methods and some possible applications for each method are described below, and illustrated in Figure 8.15.

Raise or lower height of sill
This option moves the bottom of the control section vertically while all the other parts (e.g., side slopes) remain in the same position relative to the approach channel. If the side slopes of the control section are not vertical, the base width of the section

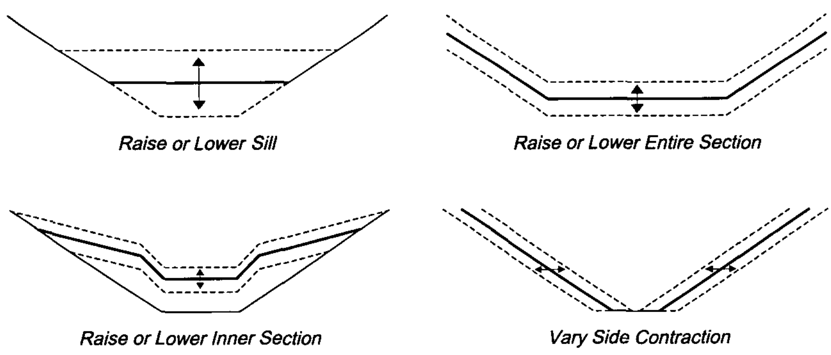


Figure 8.15 Methods of contraction change supported by the design module.

will vary as the sill height is changed. This is the option to use for designing a broad-crested weir in a lined canal. In some cases, the shape actually changes with this option. For example, a sill in a U-shape could become a rectangle (i.e., bottom moved above circular part of the U) or a trapezoid in a circle or in a parabola would become a sill in a circle or parabola, respectively.

Raise or lower entire section
This option moves the entire control section up or down relative to the approach channel bottom. The shape and the dimensions of the control section will never change under this option. Hence, the amount of contraction is changed only by raising or lowering the sill height, p_1 (p_1 will not be reduced below zero). This option is useful if a standard design (or prefabricated) flume is to be placed in a channel. Such flumes are common for flow-survey work. This option could be used to test the feasibility of using standard-sized structures at different locations throughout an irrigation system.

Raise or lower inner section
This option only applies to four shapes: a complex trapezoid, a trapezoid in a circle, a trapezoid in a U-shape, or a trapezoid in a parabola. Under this contraction-change strategy, the inner trapezoid (the part that moves up and down) remains intact, with no changes in its dimensions. The outer shape remains fixed relative to the approach channel as the inner trapezoid moves up and down. This option is useful for designing a complex flume inside of an existing channel, where the existing channel becomes the outer shape, or for designing a trapezoidal control within an existing channel that is circular, U-shaped, or parabolic. This option allows an efficient design without disturbing the existing channel section. WinFlume will not reduce the inner sill height below zero.

Vary side contraction
This option holds the bottom of the control section in a constant position relative to the approach channel, while the bottom width, b_c, changes to provide the proper amount of contraction. Some shapes do not have a defined bottom width. For those shapes WinFlume varies the diameter (for the U-shape and the circle) or the focal length (for the parabola). For the complex trapezoid and the trapezoids inside of other shapes, the inner bottom width is varied. For a sill inside of a circle, U-shape, or parabola, the diameter or focal length is varied. For the V-in-a-rectangle, the width of the rectangle is changed. This option is not allowed for V-shaped control sections, and this is the only contraction change method that can be used for movable weirs. WinFlume will not design a structure if the control section is wider than the approach section.

Choosing the increment of contraction change

WinFlume will evaluate alternative designs at a fixed increment of contraction change, specified by the user. The contraction change can be specified using several different length units, and these do not need to match the data input/output units choices selected for the structure (see Section 5.4.1 and 8.7.3). For example, the structure might be set to use units of feet for length dimensions, but a contraction change increment of 1 inch could be chosen. This allows the user to choose a contraction change increment that will yield convenient construction dimensions. The contraction change increment should be large enough that WinFlume is not forced to evaluate too many designs; an increment that might lead to evaluation of 10 to 30 designs is probably appropriate. If a very large contraction change increment is chosen, WinFlume may not find an acceptable design because the range of acceptable designs might be relatively narrow and is thus missed with a large contraction change increment. However, in this case, WinFlume will automatically reduce the contraction change increment in an attempt to find a range of acceptable designs (see Section 8.8.10).

Results of design analysis — the design evaluation report

When the *Review Possible Designs* tab on the *Flume Design* form is selected, WinFlume will execute an algorithm that generates and evaluates a family of design alternatives derived from the starting design, using the user's chosen method and increment of control section contraction change (see Section 8.8.10 for details of the algorithm). Figure 8.16 shows the results of the design evaluation as displayed on the second tab of the design evaluation form. The upper part of the form contains a spreadsheet that lists all of the evaluated designs that meet the four primary design criteria (see Section 8.8.1), or in the case where none of the designs meet the criteria, the spreadsheet lists all designs that were evaluated. The lower part of the form

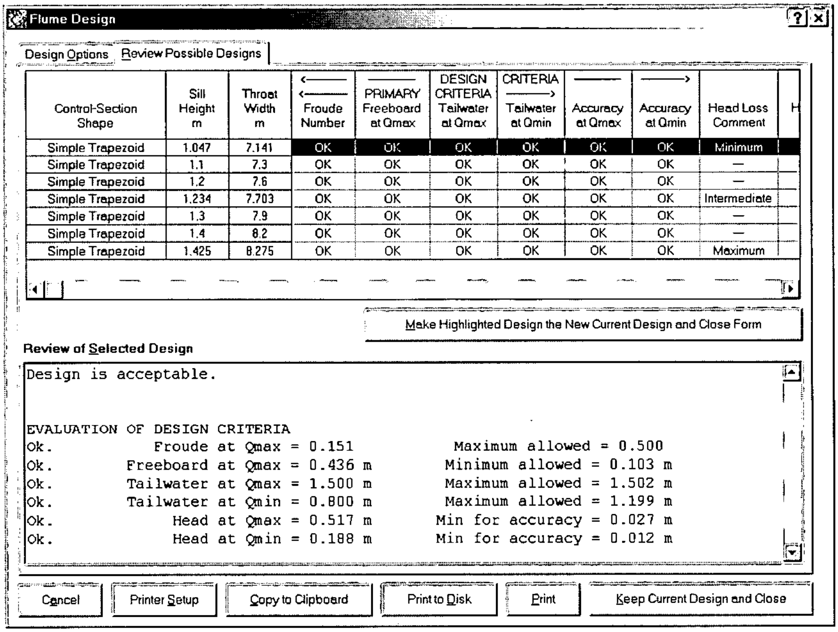


Figure 8.16 The Review Possible Designs form, showing the results of evaluating designs with varying sill heights. The bottom portion of the form shows an abbreviated design review report for the design alternative highlighted in the spreadsheet at the top.

shows an abbreviated design review report for the design alternative that the user is highlighting in the spreadsheet. Buttons on the form allow selecting one of the alternative designs as the new current design, or closing the design alternatives form and returning to the initial design. The leftmost columns of the spreadsheet show the control section shape and the key length dimension defining the amount of contraction in each design. Columns to the right indicate whether the design meets (OK) or does not meet (Not OK) each design criterion. The head loss comment column will highlight the minimum, maximum, and intermediate head loss designs, as well as the design whose head loss at Q_{max} matches the bed drop, if such a design exists. These comments will not be provided if the design does not also satisfy the two secondary design criteria for flow measurement accuracy.

By scrolling to the right side of the spreadsheet, additional columns can be seen that provide more detailed information about each design. Table 8.3 shows a printout of the complete design evaluation report. Key columns to note in the right half of the table are the extra freeboard at Q_{max} and the submergence protection. The minimum head loss design will have zero submergence protection (see Section 8.8.2), while the maximum head loss design has no extra freeboard beyond the required amount entered by the user (see Section 8.7.9).

8.8.4 Refining the design after using the design module

The evaluation of design alternatives only varies the contraction in the control section shape (i.e., the sill height, width, diameter, or focal length of the shape). After a new design has been selected from the list of alternatives, it may be necessary to adjust the lengths of the approach channel, converging transition, and control section. The rating table report or the abbreviated design review shown on the third tab at the bottom right corner of the main WinFlume screen may be helpful in determining acceptable lengths for these flume components.

Table 8.3 Design evaluation report printed by WinFlume.

User: Clemmens/Wahl/Bos/Replogle WinFlume32 - Version 1.05
D:\WinFlume Example.Flm - Revision 3
Example flume for Chapter 8
Design Alternatives Report, Printed: 2/12/2001 1:06:27 PM

			DESIGN CRITERIA											
			Primary Criteria				Secondary							
Control-Section Shape	Sill Height ft	Throat Width ft	Froude Number	Freeboard at Qmax	Tailwater at Qmax	Tailwater at Qmin	Error @ Qmax	Error @ Qmin	Head Loss Comment	Actual Head Loss ft	Actual Froude Number @ Qmax	Extra Freeboard at Qmax ft	Submergence Protection ft	Estimated Random Error @ Qmax/Qmin
Simple Trapezoid	4.047	26.141	Ok	Ok	Ok	Ok	Ok	Ok	Minimum	0.22	0.13	1.03	0.00	±1.91-2.01%
Simple Trapezoid	4.1	26.3	Ok	Ok	Ok	Ok	Ok	Ok	---	0.27	0.13	0.98	0.05	±1.91-2.01%
Simple Trapezoid	4.2	26.6	Ok	Ok	Ok	Ok	Ok	Ok	---	0.36	0.12	0.89	0.14	±1.91-2.01%
Simple Trapezoid	4.3	26.9	Ok	Ok	Ok	Ok	Ok	Ok	---	0.45	0.12	0.80	0.23	±1.91-2.01%
Simple Trapezoid	4.4	27.2	Ok	Ok	Ok	Ok	Ok	Ok	---	0.54	0.12	0.72	0.32	±1.91-2.02%
Simple Trapezoid	4.5	27.5	Ok	Ok	Ok	Ok	Ok	Ok	---	0.63	0.11	0.63	0.40	±1.91-2.02%
Simple Trapezoid	4.6	27.8	Ok	Ok	Ok	Ok	Ok	Ok	---	0.72	0.11	0.54	0.49	±1.91-2.02%
Simple Trapezoid	4.628	27.884	Ok	Ok	Ok	Ok	Ok	Ok	Intermediate	0.74	0.11	0.52	0.52	±1.91-2.02%
Simple Trapezoid	4.7	28.1	Ok	Ok	Ok	Ok	Ok	Ok	---	0.81	0.11	0.45	0.58	±1.91-2.02%
Simple Trapezoid	4.8	28.4	Ok	Ok	Ok	Ok	Ok	Ok	---	0.90	0.11	0.36	0.67	±1.91-2.02%
Simple Trapezoid	4.9	28.7	Ok	Ok	Ok	Ok	Ok	Ok	---	0.99	0.10	0.27	0.76	±1.91-2.02%
Simple Trapezoid	5.	29.	Ok	Ok	Ok	Ok	Ok	Ok	---	1.08	0.10	0.19	0.85	±1.91-2.03%
Simple Trapezoid	5.1	29.3	Ok	Ok	Ok	Ok	Ok	Ok	---	1.17	0.10	0.10	0.94	±1.91-2.03%
Simple Trapezoid	5.2	29.6	Ok	Ok	Ok	Ok	Ok	Ok	---	1.26	0.10	0.01	1.03	±1.91-2.03%
Simple Trapezoid	5.206	29.619	Ok	Ok	Ok	Ok	Ok	Ok	Maximum	1.27	0.09	0.00	1.04	±1.91-2.03%

Method of Contraction Change = Raise or Lower Sill Height
Evaluation Increment = 0.1 feet

8.8.5 Using the design review reports

Although the design module makes it convenient to quickly identify a workable flume design, there may be situations in which the designer needs to further refine the design or wishes to develop a design on their own. The general procedure is to modify the design or the design requirements and then check the design using the main design review report (*Review Current Design* command on the *Design* menu) or the abbreviated design review shown on the third tab in the bottom right corner of the main WinFlume screen. Errors or warning messages and design suggestions can be considered and the design revised until the design review report indicates a satisfactory design. Table 8.4 shows a printed design review report. The structure is evaluated against each of the six design criteria, and advice, warnings, and error messages pertaining to Q_{min} and Q_{max} are provided. The design requirements for the structure are also summarized at the bottom of the report. If fatal errors occur at minimum or maximum discharge so that hydraulic calculations cannot be completed (e.g., flow in the approach channel overtops the canal banks or tailwater submerges the control section so that a valid h_1 versus Q rating cannot be determined), the warning and error messages will help the user determine appropriate modifications that need to be made to the design.

8.8.6 Modifying the flume design to satisfy the design criteria

When any of the six design criteria are not satisfied, there may be several different design modifications that can be made in an attempt to satisfy the criteria. To assist the designer, each of the criteria are discussed below and the modifications to consider in each case are presented.

Froude number at Q_{max}

To avoid unpredictable flow conditions in the approach channel (standing waves, for example), the Froude number in the approach channel must be less than 0.5. For most channels and flumes, the Froude number increases as discharge increases. Thus, it is only necessary to check the Froude number at maximum flow. If the approach section Froude number is too high at the highest possible upstream water depth (zero freeboard), the approach channel is too small to allow accurate flow measurement with a flume or weir, and WinFlume will never find an acceptable design. The only possible way to arrive at an acceptable design in this case is to increase the cross-sectional area of the approach channel. If you have specified a non-zero required freeboard, you might find an acceptable design by decreasing the specified freeboard.

If the Froude number in the approach channel is too high, but there is additional freeboard available in the approach channel, you can reduce the Froude number by increasing the amount of control section contraction, either by raising the sill or reducing the width. If the design objective is to minimize the total head loss through the structure, it should be noted that designs that are limited by the upstream Froude number cannot achieve the minimum head loss condition (i.e., zero submergence

Table 8.4 Flume design review report.

```
User: Clemmens/Wahl/Bos/Replogle        WinFlume32 - Version 1.05
D:\WinFlume Example.Flm - Revision 3
Example flume for Chapter 8
Printed: 2/12/2001 12:22:54 PM

                       SUMMARY EVALUATION OF FLUME DESIGN
--------------------------------------------------------------------------------
Design is acceptable, but improvements are also possible.
Three errors or warnings.

             EVALUATION OF FLUME DESIGN FOR EACH DESIGN REQUIREMENT
--------------------------------------------------------------------------------
Ok.      Froude number at Qmax = 0.124            Maximum allowed = 0.500

Ok.          Freeboard at Qmax = 1.231 ft         Minimum allowed = 0.334 ft

Ok.          Tailwater at Qmax = 5.515 ft         Maximum allowed = 5.652 ft
                                   Submergence Protection at Qmax = 0.137 ft

Ok.          Tailwater at Qmin = 2.724 ft         Maximum allowed = 4.554 ft
                                   Submergence Protection at Qmin = 1.830 ft

Ok.               Head at Qmax = 1.669 ft    Minimum for accuracy = 0.104 ft
                      Expected discharge measurement error at Qmax = ±2.65 %

Ok.               Head at Qmin = 0.453 ft    Minimum for accuracy = 0.039 ft
                      Expected discharge measurement error at Qmin = ±2.01 %

                      ADVICE, WARNINGS, AND ERROR MESSAGES
--------------------------------------------------------------------------------
ERRORS AND WARNINGS AT MINIMUM DISCHARGE:
- Converging section length is too short (ramp is too steep).
    Suggest converging ramp length >= 10.50 ft

ERRORS AND WARNINGS AT MAXIMUM DISCHARGE:
- Upstream energy head / control section length exceeds 0.7.
    Suggest control length >= 2.43 ft
- Converging section length is too short (ramp is too steep).
    Suggest converging ramp length >= 10.50 ft

                             CONTROL SECTION DATA
--------------------------------------------------------------------------------
Section shape = SIMPLE TRAPEZOID
Bottom width = 26.600 ft
Side slopes = 1.50:1
Sill Height = 4.200 ft

                                DESIGN CRITERIA
--------------------------------------------------------------------------------
Structure Type:  Stationary Crest
Freeboard design criterion:  Freeboard >= 20% of upstream sill-referenced head
Allowable discharge measurement errors for a single measurement:
     At minimum discharge: ±8 %
     At maximum discharge: ±4 %
Head detection method: Transducer
     Expected measurement error = ±0.002000 ft
Design discharges and associated tailwater levels:
     Minimum discharge =  25.000 cu. ft/s     Minimum tailwater depth = 2.724 ft
     Maximum discharge = 193.000 cu. ft/s     Maximum tailwater depth = 5.515 ft
Tailwater calculation method:  Power curve using 2 Q-H measurements
     Q = 25.000 cu. ft/s   --->  y2 = 2.724 ft
     Q = 193.000 cu. ft/s  --->  y2 = 5.515 ft
```

protection or actual tailwater equal to allowable tailwater) because meeting the Froude number criteria requires some additional contraction beyond that needed to satisfy the submergence criteria. If the objective is minimum head loss and the designer finds that the design is limited by the approach channel Froude number, then the approach channel should be enlarged (e.g. widened or the invert lowered).

Freeboard at Q_{max}

If this criterion is not satisfied, the upstream water level is too high. The cross-sectional area of the control section is too small, and is thus restricting the flow at Q_{max}. The flow area at the control section can be enlarged by reducing the sill height or widening the control section. Since this criterion is only evaluated at Q_{max}, it may be possible to only widen the control section at the top by flattening the side slopes. In this manner, the flow measurement accuracy at Q_{min} can be preserved. If the control section area cannot be enlarged enough to meet the freeboard requirement, then it may be necessary to raise the maximum allowable water level by decreasing the required freeboard or increasing the canal depth.

Tailwater at Q_{max} (flume submergence)

For the flume to function properly as a flow measurement structure, critical depth must occur at the control section (i.e., modular flow). WinFlume computes the head loss through the structure at this condition and the corresponding maximum allowable tailwater depth. If the actual tailwater depth is greater than the allowable depth at Q_{max}, then there is not enough head available for the flume to operate at modular flow conditions. The flume will be submerged. Thus, the upstream sill-referenced head will be influenced by the tailwater level, and the structure will not measure the flow. To resolve this problem the designer must either increase the available head at the site by increasing the control section contraction and thereby increasing the water level upstream of the structure, or the designer must reduce the head loss through the structure. Thus, three possible solutions are: increase the sill height, narrow the control section at Q_{max}, or add a downstream ramp to reduce the total head loss.

Tailwater at Q_{min} (flume submergence)

This design criterion is similar to that described in the paragraph above, except that submergence of the flume is occurring at the minimum flow condition. Possible remedies are: increase the sill height, narrow the control section at Q_{min} (i.e., reduce the base width of the control section), or add a downstream ramp.

Flow measurement accuracy at Q_{max} (required head)

The accuracy of the flow-measuring structure is mainly influenced by the uncertainty associated with the device used to measure h_1, the upstream sill-referenced head in the approach channel. The user specifies the head-detection method and its uncertainty and a desired overall accuracy for the flow measurement structure. The overall accuracy of the structure is a combined function of the rating table uncertainty (approximately ±2%) and the relative uncertainty of head detection at any given flow rate. WinFlume calculates the minimum upstream head, h_1, required to achieve the desired accuracy for one single measurement at maximum discharge. If the control section does not constrict the flow enough, this minimum head will not be obtained and the measurement accuracy will be less than that desired. To achieve the desired flow measurement accuracy at Q_{max}, the possible remedies are narrowing the control section at Q_{max} (i.e., reducing the top width), using a more accurate head-detection method, or increasing the allowable measurement error at Q_{max}. If the H_1/L ratio for the structure is greater than 0.7, increasing the control section length may also increase the accuracy of the structure.

Flow measurement accuracy at Q_{min} (required head)

This criterion is similar to that described in the previous paragraph, except that the flow measurement accuracy is evaluated at Q_{min}. If the criterion is not satisfied, possible remedies are narrowing the control section at Q_{min} (i.e., reducing the base width, which also may require lowering the crest.), using a more accurate head-detection method, or increasing the allowable measurement error at Q_{min}. If the H_1/L ratio for the structure is less than 0.07, decreasing the control section length may also increase the accuracy of the structure.

8.8.7 Using the rating table report to refine the design

In the design module and in the design review reports, WinFlume evaluates the six design criteria at only Q_{min} and Q_{max}. In most cases this is sufficient, since the design criteria will usually be satisfied at all intermediate flow rates as well. To ensure that this is the case, a rating table can be generated, and the rightmost column of the rating table checked to ensure that there are no errors or warnings at intermediate flows. This may also reveal refinements to the design that need to be made, although they are not related to the six design criteria, such as recommended lengths of the approach channel, converging and diverging transitions, and the control section. The lengths of these flume elements need to be within the recommended ranges to ensure that the assumption of one-dimensional flow with essentially parallel streamlines is satisfied at the control section. This requirement must be met to ensure that the rating tables generated by WinFlume will have the stated accuracy of ±2 percent.

8.8.8 Issues when designing structures with compound control sections

The design of structures having compound control sections sometimes presents unique difficulties. These structures have a control section that is a complex trapezoid or any of the trapezoidal shapes inside of another section shape. These structures are intended to operate with flow only in the inner trapezoid shape at Q_{min} so that small flows can be measured accurately. At Q_{max}, the flow fills the outer part of the control section shape, which is made very wide to minimize the upstream depth increase caused by the flume at maximum discharge. In the transition zone where flow just begins to enter the outer portion of the throat shape, the accuracy of these structures is poor. They should be carefully designed so that this transition zone corresponds to a flow rate that does not occur frequently and is not of special significance.

The construction of the converging and diverging transitions of flumes having compound cross sections also presents unique problems. From the standpoint of the hydraulic analysis needed to develop the h_1 versus Q rating, the only requirement for the transitions is that they be smooth and gradual, without offsets at the upstream and downstream ends of the throat section. These requirements can be met by a multitude of different transition designs. If the flume is being constructed from concrete, the transition surfaces can often be free-formed during construction.

8.8.9 Difficulties finding an acceptable converging transition length

We have stated previously that a converging transition slope of 2.5:1 to 4.5:1 is desirable for both the floor and sidewalls of the transition. The converging transition slope is implied by the transition length entered into the flume bottom-profile screen. WinFlume evaluates the length of the converging transition when generating rating tables and design review reports. The program evaluates the length of the converging transition at both minimum and maximum flow, and for each flow rate bases the recommended transition length on the maximum of three different measures of the contraction:

- The vertical contraction from the invert of the approach channel to the invert of the control section,
- The horizontal contraction from the side wall of the approach channel to the side wall of the control section at the elevation of the control section invert, and
- The horizontal contraction from the side wall of the approach channel to the side wall of the control section, at the elevation of the water level in the approach channel (y_1).

There are some special issues to consider when one or two of these measures of contraction are dramatically different from the others or when the amount of contraction changes dramatically between minimum and maximum flow.

To measure large ranges of flow (e.g., $Q_{max}/Q_{min} > 200$), V-notched control sections or other shapes with a very narrow base width are often used (see Section 2.4). A common problem when designing such flumes is the presence of error message 22 (see Section 8.12), which states that the converging section length is too short (side contraction is too abrupt). The recommended transition length that eliminates this error message may seem unreasonably long to the user, since it will often produce a relatively flat floor ramp in the converging transition. Figure 8.17 illustrates the problem graphically. The governing contraction amount is the horizontal contraction at the invert of the control section, which is much larger than the other contractions. The converging transition length must be 2.5 to 4.5 times this length to eliminate

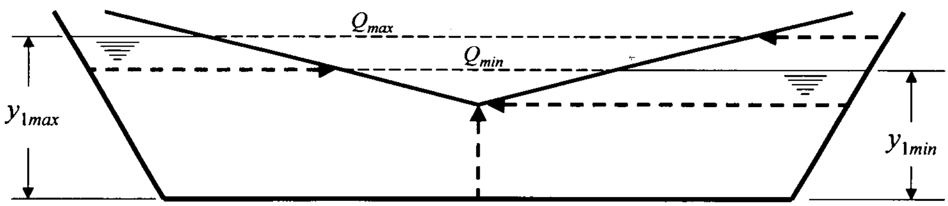


Figure 8.17 Converging transition length is based upon the largest of the contractions shown (heavy dashed lines) at minimum and maximum flow. When the control section is narrow at the base, the converging transition length is often governed by the side contraction at the invert of the control section.

error messages 22 and 23 (see Section 8.12). In this situation, we recommend making the transition length equal to the recommended amount given in connection with error 22 (2.5 times the maximum horizontal contraction). This may yield a relatively flat floor ramp, but it will ensure that there is no flow separation at the entrance to the throat section.

A related problem can occur when attempting to use an approach channel and control section of dramatically different shape, such as a rectangular control section and a trapezoidal approach channel having relatively flat side slopes, as illustrated in Figure 8.18. At maximum flow the governing contraction length is the horizontal contraction at the y_{1max} elevation, but at minimum flow the governing contraction is either the vertical contraction, or the horizontal contraction at y_{1min}, which may be much less than the value at maximum flow. In such a situation it is possible to receive error message 22 (side contraction too abrupt) at maximum flow. Then, when the converging transition length is increased, message 11 or 23 (ramp or side contraction too flat) is displayed, but the error now occurs at minimum flow. It is even possible to obtain both error messages simultaneously, with a single value of the transition length.

To resolve this problem, one should recall the reasons for which a 2.5:1 to 4.5:1 transition slope was recommended. Slopes more abrupt than 2.5:1 cause flow separation at the entrance to the throat section, which can significantly affect the

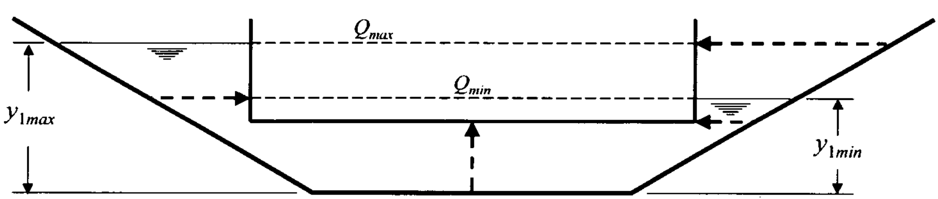


Figure 8.18 When the control section shape is significantly different from the approach channel shape, the governing (largest) contraction may be significantly different at minimum and maximum flow. This can cause conflicting warning messages that the converging transition is both too short and too long.

accuracy of the head-discharge rating of the structure. On the other hand, slopes flatter than 4.5:1 yield a longer, more expensive structure, but have little effect on the accuracy of the head-discharge rating. Thus, a transition that is too long is preferred over one that is too short. As a result, one should lengthen the transition to eliminate error 22 at maximum flow, and tolerate error 11 or 23 at minimum flow. A case where this might not be recommended is the situation in which flow measurement accuracy at maximum flow is relatively unimportant; in such a case one could tolerate a transition that is too abrupt at maximum flow.

Two other solutions are also available. One is to change the shapes of the control or approach channel sections to make them more similar, but this is not always practical. The other solution is to recall that the exact geometry of the converging transition is not critical to flume performance, as long as the transition is gradual without offsets or large breaks in slope approaching the throat section (see Section 3.1). Thus, even though WinFlume only allows the user to specify a single converging transition length, which defines the angles for both the horizontal transition and the floor ramp (vertical transition), one can use different transition lengths for the side walls and floor when the flume is constructed. Examples of this type of construction were shown in Figure 3.25 and 3.58. When using WinFlume to model a flume constructed in this manner, the most important consideration is correctly locating the gaging station. The distance from the gage to the upstream end of the throat (i.e., the sum of the approach length and the converging transition length shown on the flume bottom profile) should be the same in WinFlume and in the constructed flume.

8.8.10 Algorithm for finding and evaluating alternative designs

The algorithm used by WinFlume to find and evaluate alternative designs is essentially a trial-and-error procedure. This is a departure from the FLUME 3.0 program, which attempted to optimize the flume design to meet all six design criteria while also satisfying a single head loss objective. WinFlume simply presents the range of designs that satisfy the design criteria, then allows the user to choose a design that meets a head loss objective or other requirement that the designer may have for the structure. This design algorithm has the advantage that it is more robust when working with complex and circular control section shapes; the algorithm used by FLUME 3.0 at times could not determine which direction to change the control section contraction to move toward a satisfactory design. WinFlume simply analyzes all of the possibilities. Although this method is more computationally intensive, this is not a problem with the computational power of today's personal computers.

Once the user has chosen a method and an increment of contraction change, the program begins the design process by reducing the contraction in the initial control section shape as much as possible (e.g., reducing the sill height to zero if the method of contraction change is to raise or lower the sill). This becomes the base design for the structure, and all design alternatives to be evaluated are derived from this base design. Next, the program determines the maximum and minimum upstream water levels that an acceptable design can produce at the maximum design flow rate, Q_{max}.

The maximum allowable upstream water level, before considering freeboard requirements, is the top of the upstream channel. The minimum required upstream water level at Q_{max} is either that corresponding to an approach channel Froude number of $Fr_1 = 0.5$, or the tailwater level at the site, whichever is higher. Water levels lower than these would either violate the maximum Froude number criterion or require an energy gain through the structure, which is of course not possible. For these maximum and minimum approach channel water levels, the program can determine the corresponding contraction in the throat section shape required to produce critical flow in the control section at Q_{max}. A subroutine, using the procedure described in Section 6.3.3, is used to determine the required contraction for each case. The flume designs having these maximum and minimum amounts of contraction become the upper and lower limits for the range of flume designs that must be analyzed.

Once the range of designs has been bracketed, the program builds and evaluates designs of "virtual" flumes between the upper and lower contraction limits. The primary control section dimension for each design will be an even increment of the contraction-change interval chosen by the user. Flume designs are evaluated against the four primary design criteria (see Section 8.8.1) to determine the range of acceptable designs. A bisection search (halving the contraction change increment until the limiting case is reached) is used to further refine the minimum and maximum amounts of contraction that will yield acceptable designs. The design with the maximum contraction is the maximum head loss design (zero additional freeboard), and the design with the minimum contraction is the minimum head loss design (zero submergence protection), unless the Froude number is the limiting criterion for the minimum-contraction design. Designs having intermediate head loss (extra freeboard equal to submergence protection) and head loss equal to the bed drop at the site (if such a design is possible) are also located using a bisection search technique.

The results of the analysis are presented to the user, as described in Section 8.8.3. The user may choose to accept one of the designs as the new current design or discard the results of the analysis. Only designs meeting the four primary design criteria (freeboard, Froude number, no submergence at Q_{min} and Q_{max}) are shown to the user, unless there are no acceptable designs, in which case all of the evaluated designs are listed. Designs that meet the four primary criteria, but do not meet one of the measurement accuracy requirements, may be improved by selecting a better water level measurement method.

If the contraction-change increment specified by the user is too large, or if the design criteria are too limiting, the design algorithm may not find any acceptable designs on the first attempt. In this case, the program searches for two adjacent designs for which the unsatisfied criteria in each design are satisfied in the adjacent design. This indicates that with a smaller increment of contraction change, it may be possible to find an acceptable design between these two designs. If this is the case, the program will reduce the contraction-change increment by a factor of 10 and repeat the entire process within this reduced range. This continues until acceptable designs are found or until the program determines that no acceptable design is possible.

8.9 Producing Output

Several types of output are available from the program, including flume drawings, reports, rating tables, rating equations, and wall gages derived from rating tables. All output from the program can be printed to any Windows-compatible printer. Font options for printed reports and drawings can be changed using the Printer Setup command on the File menu. All program output except wall gages can be copied to the system clipboard for pasting into Windows-compatible word processors or spreadsheets.

When attempting to send output to the printer or the clipboard, WinFlume will sometimes require the user to save the current flume definition. This ensures that the output from the program can always be traced to a specific revision number of the flume design (see Section 8.6.4). When assembling several items of output from the program (e.g., a flume drawing and the associated rating table), the user should ensure that each is marked with the same revision number.

8.9.1 Flume drawing printout

An example of a printed flume drawing is shown in Figure 8.19. The bottom profile and cross-section shapes and dimensions of the flume are shown, as well as the upstream and downstream views and approximate water surface profiles through the

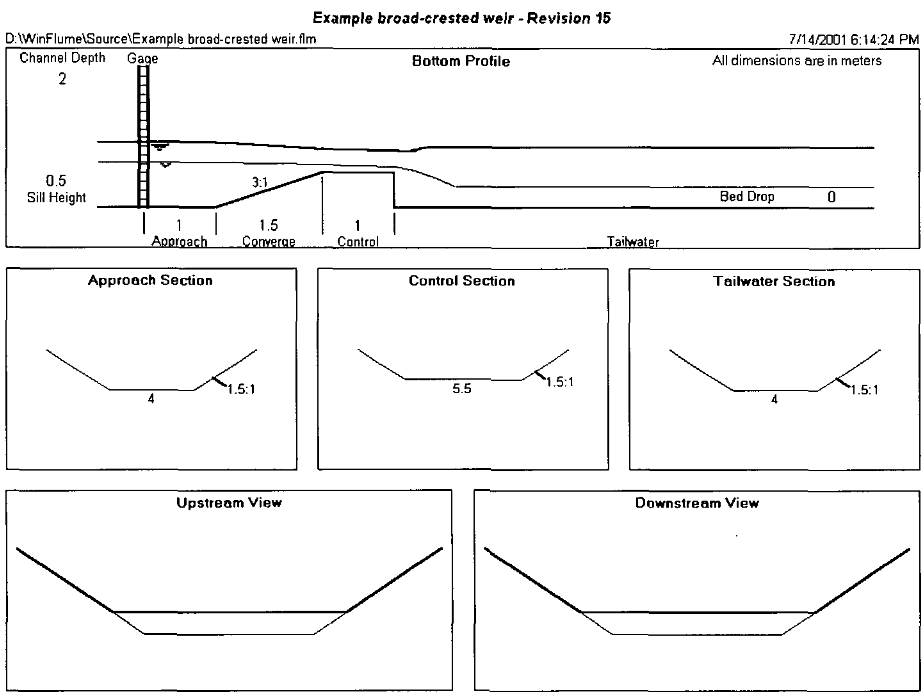


Figure 8.19 A flume drawing produced by WinFlume.

structure, if the user has chosen to show water surface profiles. Dimensions of the cross-sections are shown at their proper location on the drawing where possible, but for some of the complex shapes the dimensions are shown in the corner of the cross-section view for clarity. In some extreme cases, the text labels for the dimensions may be partially clipped. If this occurs, the user can obtain the dimensions from the flume data report described in the next section. Some printers do not properly display the control section, which is highlighted in blue on the flume drawing printout. In that case, select the option in the Printer Setup dialog box that allows the flume drawing to be printed in black and white.

8.9.2 Flume data report

The flume data report is a companion to the flume drawing, summarizing in a text form the dimensions and other properties of the structure. An example of the flume data report is shown in Table 8.5. The discharge range, tailwater conditions, and

Table 8.5 Flume data report.

```
User: Clemmens/Wahl/Bos/Replogle        WinFlume32 - Version 1.05
D:\WinFlume Example.Flm - Revision 3
Example flume for Chapter 8
Printed: 2/12/2001 4:13:29 PM
                               FLUME DATA REPORT
------------------------------------------------------------------------------
                             GENERAL DATA ON FLUME
Type of structure: Stationary Crest
Type of lining: Concrete - smooth
Roughness height of flume: 0.000492 ft

                              BOTTOM PROFILE DATA
Length per section:  Approach section, La = 2.000 ft
                Converging transition, Lb = 12.300 ft
                      Control section, L  = 3.000 ft
                 Diverging transition, Ld = 24.600 ft

Vertical dimensions: Upstream channel depth = 7.100 ft
                          Height of sill, p1 = 4.100 ft
                                    Bed drop = 0.000 ft
                 Diverging transition slope = 6.000:1

                          -- APPROACH SECTION DATA --
Section shape = SIMPLE TRAPEZOID
Bottom width = 14.000 ft
Side slopes = 1.50:1

                          -- CONTROL SECTION DATA --
Section shape = SIMPLE TRAPEZOID
Bottom width = 26.300 ft
Side slopes = 1.50:1

                          -- TAILWATER SECTION DATA --
Section shape = SIMPLE TRAPEZOID
Bottom width = 14.000 ft
Side slopes = 1.50:1
```

design requirements are not shown in this report, but are provided in the design review report, shown previously in Table 8.4.

8.9.3 Review current design

The use of the design review report (Table 8.4) was discussed in detail in Section 8.8.5. This report like all others can be copied to the clipboard or printed. Portions of the report can be copied to the clipboard by highlighting the desired text using the mouse or the Shift+arrow keys and then pressing Ctrl+C.

8.9.4 Rating tables and graphs

Several types of rating tables are available in the program. All rating tables are obtained through the tabbed rating table form shown in Figure 8.20. The standard head-discharge and discharge-head tables show one flow rate on each line of the table. A head-discharge table shows the discharges corresponding to fixed intervals of upstream sill-referenced head, while the discharge-head table shows the head at fixed discharge intervals. Additional parameters can be selected from the list shown on the right side of the form for inclusion in the table.

For either type of rating table, the user can manually enter the range of the table and the head or discharge increment of the table, or the Smart Range button can be clicked to have the program automatically select a reasonable range for the table.

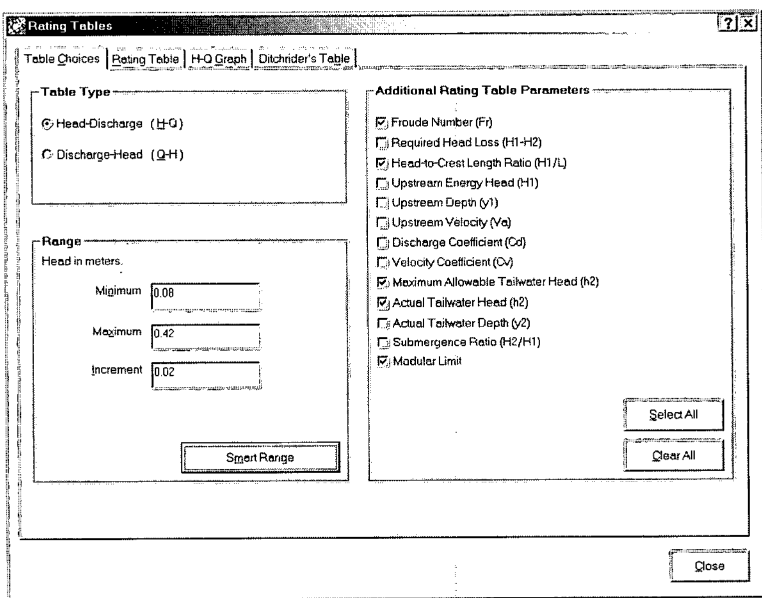


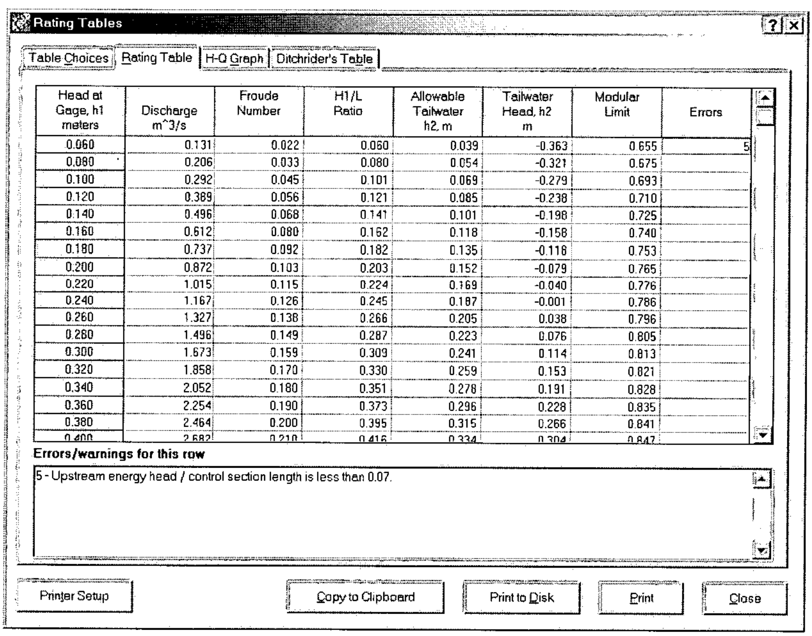
Figure 8.20 The first tab of the rating table form, used to select options for displaying standard rating tables (one flow rate per line).

The Smart Range button is only intended to provide a starting point for viewing the performance of the flume in an appropriate operational range for the structure. Some fine-tuning of the table range may be desirable.

Once the rating table type and range have been selected, the rating table can be viewed on the second tab of the form, as shown in Figure 8.21. The user can browse up and down through the table. The rightmost column of the table lists errors or warnings associated with the currently highlighted line of the table. Error descriptions keyed to the error numbers appear in the text box below the table. If the presently highlighted line does not have any errors, then the textbox will display all error messages associated with the entire rating table. These error messages can be useful when refining a flume design. The rating table will indicate the range of flows over which an error occurs.

The third tab of the *Rating Tables & Graphs* form shows the rating table data in graphical form (Figure 8.22). One to three parameters can be plotted as a function of the discharge. One of these three parameters can be assigned to a separate axis on the right side of the graph, and the right and left axes can be synchronized if desired. Portions of the rating table graph that have errors or warnings listed in the rating table will be plotted with a dashed line.

The ditchrider's rating table (Figure 8.23) is a condensed rating table that shows a wide range of flows at small head increments in a format that fits on just one or two



Head at Gage, h1 meters	Discharge m^3/s	Froude Number	H1/L Ratio	Allowable Tailwater h2, m	Tailwater Head, h2 m	Modular Limit	Errors
0.060	0.131	0.022	0.060	0.039	-0.363	0.655	5
0.080	0.206	0.033	0.080	0.054	-0.321	0.675	
0.100	0.292	0.045	0.101	0.069	-0.279	0.693	
0.120	0.389	0.056	0.121	0.085	-0.238	0.710	
0.140	0.496	0.068	0.141	0.101	-0.198	0.725	
0.160	0.612	0.080	0.162	0.118	-0.158	0.740	
0.180	0.737	0.092	0.182	0.135	-0.118	0.753	
0.200	0.872	0.103	0.203	0.152	-0.079	0.765	
0.220	1.015	0.115	0.224	0.169	-0.040	0.776	
0.240	1.167	0.126	0.245	0.187	-0.001	0.786	
0.260	1.327	0.138	0.266	0.205	0.038	0.796	
0.280	1.496	0.149	0.287	0.223	0.076	0.805	
0.300	1.673	0.159	0.309	0.241	0.114	0.813	
0.320	1.858	0.170	0.330	0.259	0.153	0.821	
0.340	2.052	0.180	0.351	0.278	0.191	0.828	
0.360	2.254	0.190	0.373	0.296	0.228	0.835	
0.380	2.464	0.200	0.395	0.315	0.266	0.841	
0.400	2.682	0.210	0.416	0.334	0.304	0.847	

Figure 8.21 A head-discharge rating table generated by WinFlume. The rightmost column in the table lists errors or warnings associated with the currently highlighted line of the table. Error descriptions appear in the text box below the table.

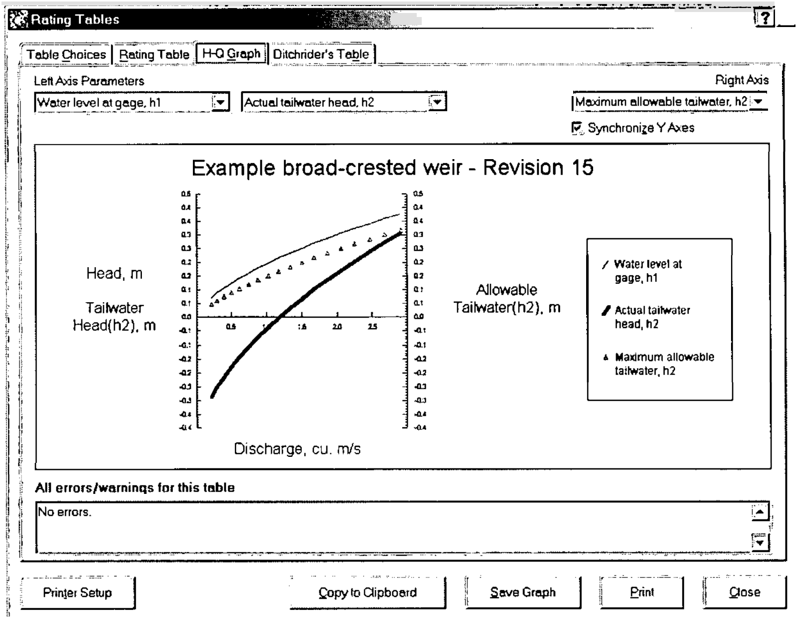
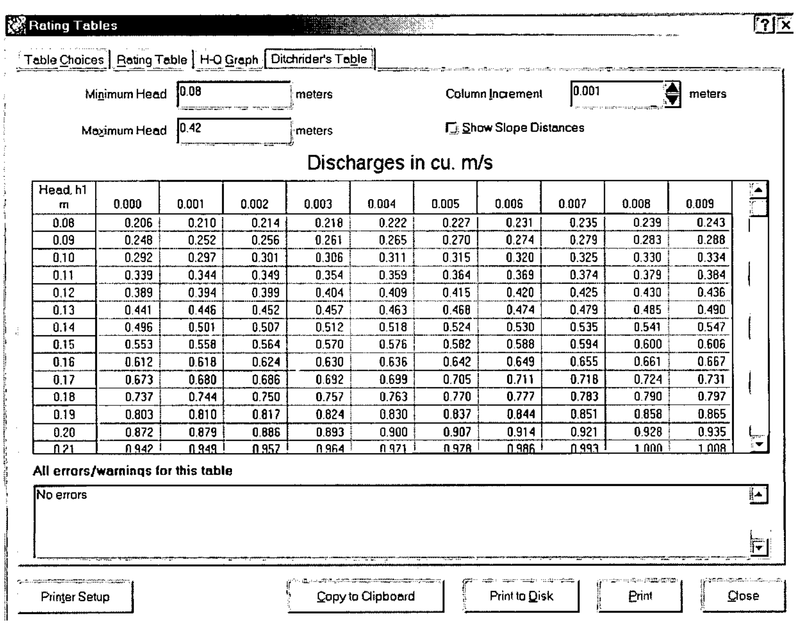


Figure 8.22 A plot of the head-discharge rating table data.



Head, h1 m	0.000	0.001	0.002	0.003	0.004	0.005	0.006	0.007	0.008	0.009
0.08	0.206	0.210	0.214	0.218	0.222	0.227	0.231	0.235	0.239	0.243
0.09	0.248	0.252	0.256	0.261	0.265	0.270	0.274	0.279	0.283	0.288
0.10	0.292	0.297	0.301	0.306	0.311	0.315	0.320	0.325	0.330	0.334
0.11	0.339	0.344	0.349	0.354	0.359	0.364	0.369	0.374	0.379	0.384
0.12	0.389	0.394	0.399	0.404	0.409	0.415	0.420	0.425	0.430	0.436
0.13	0.441	0.446	0.452	0.457	0.463	0.468	0.474	0.479	0.485	0.490
0.14	0.496	0.501	0.507	0.512	0.518	0.524	0.530	0.535	0.541	0.547
0.15	0.553	0.558	0.564	0.570	0.576	0.582	0.588	0.594	0.600	0.606
0.16	0.612	0.618	0.624	0.630	0.636	0.642	0.649	0.655	0.661	0.667
0.17	0.673	0.680	0.686	0.692	0.699	0.705	0.711	0.718	0.724	0.731
0.18	0.737	0.744	0.750	0.757	0.763	0.770	0.777	0.783	0.790	0.797
0.19	0.803	0.810	0.817	0.824	0.830	0.837	0.844	0.851	0.858	0.865
0.20	0.872	0.879	0.886	0.893	0.900	0.907	0.914	0.921	0.928	0.935
0.21	0.942	0.949	0.957	0.964	0.971	0.978	0.986	0.993	1.000	1.008

Figure 8.23 A ditchrider's rating table.

printed pages. Values of sill-referenced head are shown along the left side and top of the table, and discharge values appear in the body of the table. The ditchrider's table does not show any other parameters. To determine the flow for a given head, scan down the left side and across the top to find the sill-referenced head. The corresponding flow rate will be contained in the cell at the intersection of the row and column. The ditchrider's table can be modified to show sill-referenced slope distances rather than sill-referenced heads. This allows the table to be used with a standard staff gage installed on a sloped canal bank. Flow rates for which there are errors or warnings are indicated in the table with an asterisk.

Rating tables may be copied to the Windows clipboard in either a space- or tab-delimited format. The tab-delimited format works best when you intend to paste the table into a spreadsheet. You may use any of the following techniques to copy the rating tables to the clipboard:

- Click the Copy to Clipboard button and select the format from the popup menu,
- Right-click on the rating table spreadsheet and choose from the popup menu,
- Right-click on the Copy to Clipboard button to copy in a space-delimited format, or
- Shift+right-click on the Copy to Clipboard button to copy in a tab-delimited format.

8.9.5 Comparing the theoretical flume rating to field-measured data

In some cases it may be desirable to verify the flow measurement accuracy of a flume or weir by independently measuring the flow at the site, perhaps using current-meter discharge measurements in the field, or a weigh-tank or venturi meter in a hydraulic laboratory. WinFlume provides the measured data comparison table and graph as a means for comparing these independent discharge measurements to the theoretical rating curve for the structure. The form used to enter pairs of measured flows and corresponding sill-reference heads (Figure 8.24) is opened by selecting *Measured Data Comparison* from the *Reports/Graphs* menu, or by clicking the toolbar button. There is no limit to the number of head-discharge pairs that can be entered by the user. Once the data are entered, the user can view a rating table that compares the measured and theoretical discharges at the given heads and computes the discharge measurement error in flow rate units and as a percentage. The graph (Figure 8.25) shows the measured values as data points overlain on a graph of the theoretical rating over the range of the measured data. If there is a consistent discrepancy between the measured data and the theoretical rating, the most likely source of the error is a problem with the zero-setting of the head measurement device (see Section 4.9).

8.9.6 Developing head-discharge equations for data loggers

Flume and weir installations are often automated using continuous or periodic recording equipment, digital data loggers, and telemetry equipment that can transmit

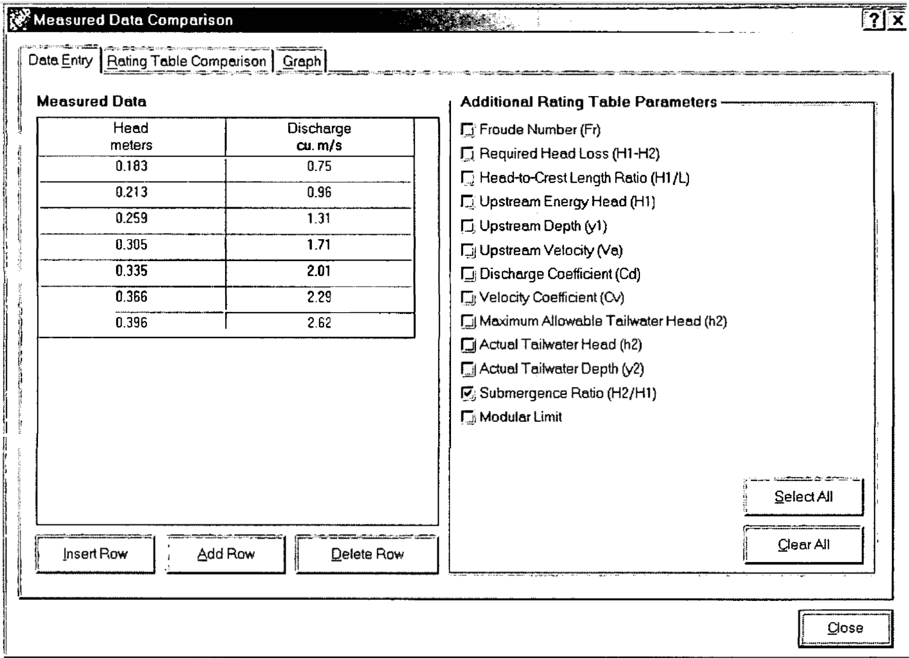


Figure 8.24 Entering measured head-discharge data for comparing to the theoretical rating.

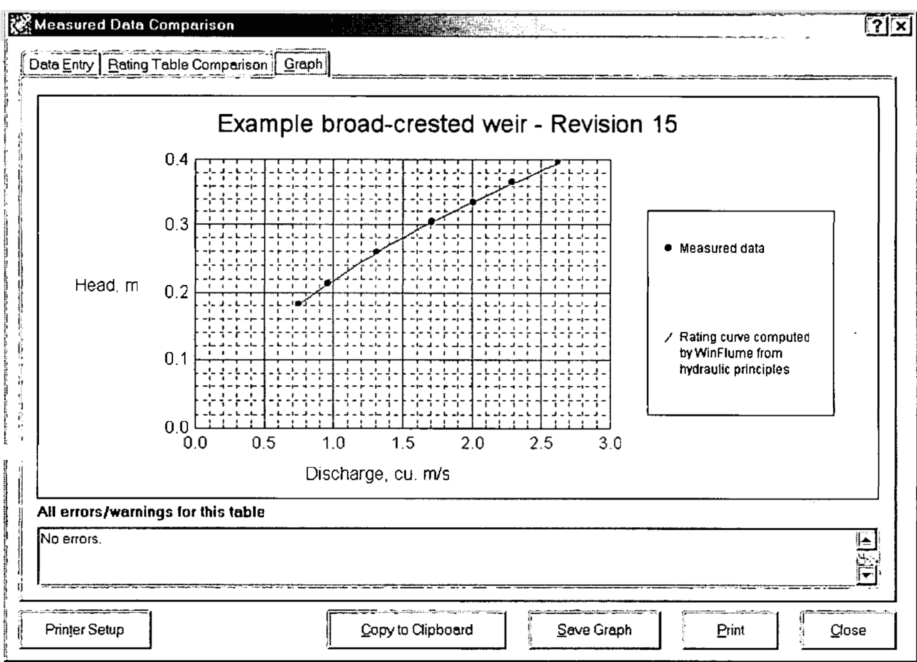


Figure 8.25 Graph comparing measured head-discharge values to the theoretical rating

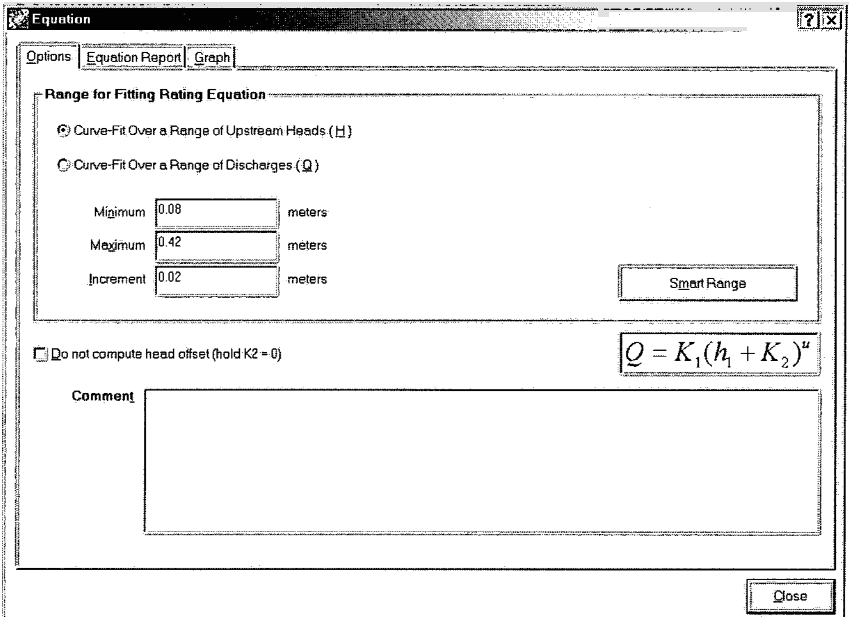


Figure 8.26 Selecting the rating table range for developing a rating curve equation.

data to a central location for real-time monitoring and long-term storage. For many of these applications, the flow rate is of primary interest, and there may not be a need to monitor or store the measured sill-referenced head. Thus, it is desirable to have a straightforward means of converting the sill-referenced head to its corresponding flow rate with a minimum of computational effort, since the capabilities of the data logger at the site may be limited. WinFlume provides a curve-fitting routine for this purpose that approximates the flume rating relationship (h_1 versus Q table) with a power-curve equation of the form

$$Q = K_1(h_1 + K_2)^u \tag{8.1}$$

in which K_1, K_2, and u are constants.

To determine a head versus discharge equation, the user begins by clicking the f(H) toolbar button or selecting the *Rating Equation* command from the *Reports/Graphs* menu. On the first tab of this form (Figure 8.26), the user selects the range of the rating table over which the curve-fitting should take place. It is recommended that the rating table contain at least 6 points to obtain a reasonable curve fit. An optional comment to be printed on the equation report can also be entered. One additional option available to the user is to force the value of $K_2 = 0$. This allows the use of a simplified power-curve equation, $Q=K_1h_1^u$, since some data loggers are not able to handle an equation of the form of Equation 8.1. When the second tab of the form is displayed, a rating table is generated and the curve-fitting routine is executed to determine the values of K_1, K_2, and u. A printed equation report is shown in Table 8.6. The results can also be shown in graphical form (Figure 8.27).

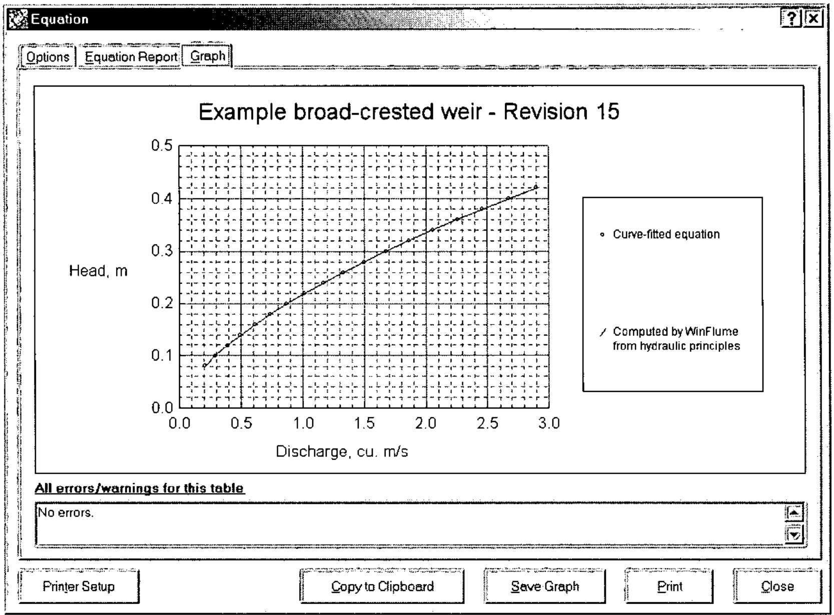


Figure 8.27 Graph showing results of curve-fitting to develop a rating curve equation.

Table 8.6 Flume rating equation report.

```
User: Clemmens/Wahl/Bos/Replogle        WinFlume32 - Version 1.05
D:\WinFlume Example.Flm - Revision 3
Example flume for Chapter 8
Rating Equation Report, Printed: 2/12/2001 1:33:20 PM

------------------------------------------------------------------------------
   Head at                    Equation      Equation      Equation
   Gage, h1     Discharge     Discharge       Error         Error       Hydraulic
     feet       cu. ft/s      cu. ft/s      cu. ft/s          %           Errors
------------------------------------------------------------------------------
      0.300        12.9          12.9        +0.020         +0.15
      0.500        28.5          28.4        -0.100         -0.35
      0.700        48.1          48.1        -0.044         -0.09
      0.900        71.3          71.4        +0.101         +0.14
      1.100        97.8          98.1        +0.234         +0.24
      1.300       127.5         127.8        +0.231         +0.18
      1.500       160.3         160.3        +0.013         +0.01
      1.700       196.2         195.6        -0.552         -0.28

  Equation:  Q = K1 * (h1 + K2) ^ u
Parameters: K1 = 82.41
            K2 = 0.01445
             u = 1.604
Coefficient of determination: 0.99999429
Optional comment

Error Summary
-------------

 No errors.
```

8.9.7 Creating wall gages

Wall gages mounted in the approach channel or on the side walls of the approach channel provide a quick and convenient means of determining the flow through a flume or weir. Even on structures where measurements are automated, a wall gage is advisable since it allows for a quick check of the automated measurements and can be conveniently read by personnel at the site without the need for additional equipment. Wall gages may be labeled to indicate the upstream sill-referenced head, with the discharge determined from the rating table, or gages can be labeled to directly indicate the flow rate. The latter eliminates the possibility of using the wrong rating table for a particular structure. Direct read-out gages can be used on both stationary-crest and movable-crest weirs. Gages can be installed vertically or placed on the sloped canal bank in sections with rigid walls of constant slope. Sloped gages must be adjusted for the increased slope length corresponding to a given vertical distance (see Figure 4.2).

WinFlume can print full-scale wall gage images or generate the data tables needed to manufacture wall gages. Gages can be labeled either with discharge values or the upstream sill-referenced head and can be adjusted for the slope of the canal bank, if necessary. Printed wall gages produced by WinFlume can be used by sign-makers to manufacture durable painted or baked-enamel wall gages suitable for field use. Gage images are corrected to account for systematic errors in the output of the printer or plotter used to produce them (see Section 8.9.7).

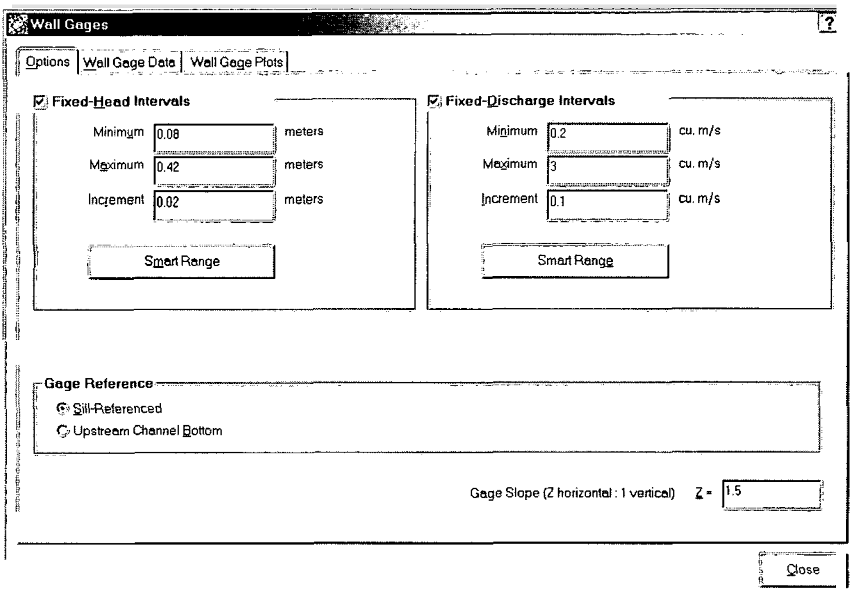


Figure 8.28 Choosing the range and tick mark increment for flume wall gages.

The wall gages produced by WinFlume are derived from a rating table. The user opens the wall gage output form by clicking the toolbar button or selecting the *Wall Gages* command from the *Reports/Graphs* menu. The first tab of the form (Figure 8.28) shows the range and increment setting for the rating table to be generated. The increment chosen for the rating table will be the smallest tick mark increment that can be used on the wall gage. Gages can be referenced either to the sill of the structure or to the bottom of the approach channel (adds the height of the sill to the length of the gage).

The second tab of the *Wall Gages* form provides rating tables and associated slope distances needed to construct a gage. The report can be printed, saved to a file, or copied to the clipboard in the form shown in Table 8.7.

Table 8.7 Wall gage data report.

```
User: Clemmens/Wahl/Bos/Replogle        WinFlume32 - Version 1.05
D:\WinFlume Example.Flm - Revision 3
Example flume for Chapter 8
Wall Gage Data, Printed: 2/12/2001 2:51:21 PM

Gage slope = 1.5:1 (horizontal:vertical distance)

Wall gage data, fixed head intervals.
------------------------------------------------------------
  Sill referenced      Slope distance
   head (vertical)      (1.5:1 slope)          Discharge
        feet                feet               cu. ft/s
------------------------------------------------------------
          0.250               0.451                  9.69
          0.300               0.541                 12.87
.
. (portion of table omitted)
.
          1.450               2.614                151.84
          1.500               2.704                160.33


Wall gage data, fixed discharge intervals.
------------------------------------------------------------
                      Sill referenced      Slope distance
      Discharge        head (vertical)      (1.5:1 slope)
      cu. ft/s              feet                feet
------------------------------------------------------------
          10.00               0.255                 0.460
          20.00               0.398                 0.718
.
. (portion of table omitted)
.
         190.00               1.667                 3.005
         200.00               1.720                 3.102

Error Summary (*'s in tables indicate lines with errors or warnings)
--------------------------------------------------------------------
No errors.
```

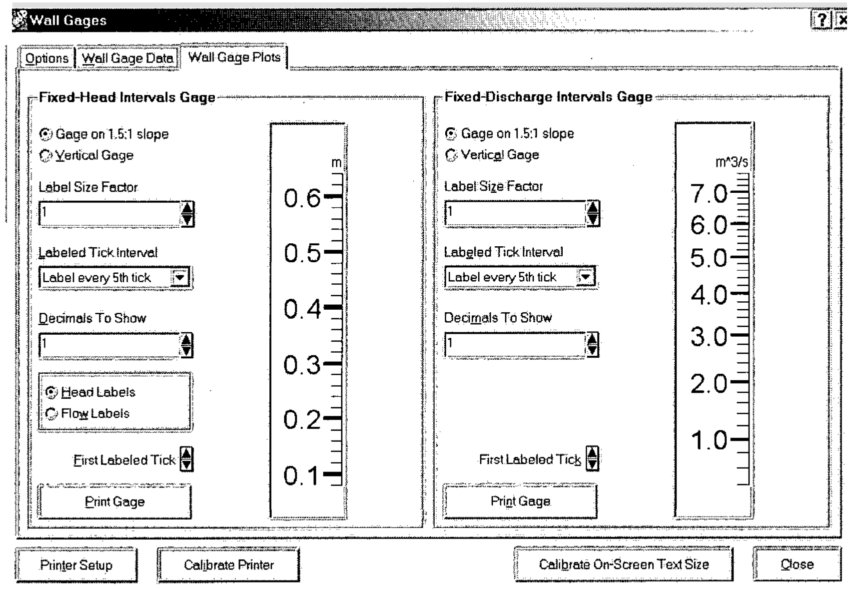


Figure 8.29 Previewing the wall gage images before printing on the system printer.

The third tab of the *Wall Gages* form (Figure 8.29) shows a preview of the wall gage images and allows the user to adjust various properties of the gages, including the relative size of the tick mark labels, the interval at which tick marks are to be labeled, and the precision of the labels. For the gage with fixed head intervals, labels may show either the head or the discharge, but if discharge labels are selected every tick mark must be labeled because the discharge increments are not constant from one tick mark to the next. For the gage with tick marks at fixed discharge intervals, discharge labels are the only option.

Wall gage images can be printed to any Windows-based printer. The width of the gage and the thickness of the printed tick marks will vary depending on the size of the gage and the paper sizes available on the printer. If a roll-feed plotter is available, it may be possible to print the entire length of long wall gages on a single sheet of paper, but if a typical office laser printer or other page printer is used, the gage will likely be longer than a single sheet of paper. In this case the gage will be printed in sections, with each section match-marked so that the sections can be accurately assembled after printing to create the complete gage. Alternate pages are printed with the match marks offset so that they are easily visible after the edges are trimmed from the printed gage section. Figure 8.30 shows the layout of the pages of a gage printed in three sections.

Calibration issues for wall gage output

At the bottom of the third tab of the wall gage form (Figure 8.29) there are two buttons labeled *Calibrate Printer* and *Calibrate On-Screen Text Size*. These

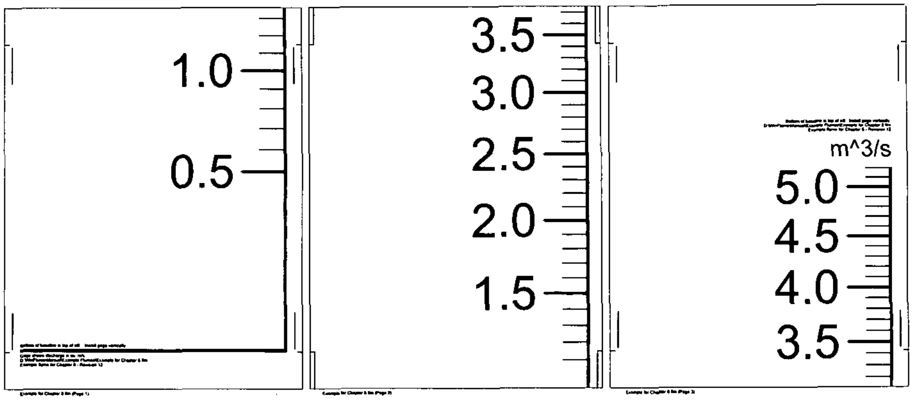


Figure 8.30 Example of a wall gage printed in three sections on a laser printer.

calibration procedures ensure that the wall gage is accurately produced on the printer and accurately previewed on the screen.

Testing during the development of WinFlume showed that printers do not always accurately reproduce the length and placement of printed lines. This seems to be especially true of laser printers and ink jet printers normally designed for office use. Although the length and placement of drawn lines is not accurate, it does seem to be highly reproducible and thus a calibration factor can be used to adjust the wall gage image to obtain accurate output. WinFlume uses a calibration procedure to determine the amount by which the wall gage should be distorted when sent to the printer to obtain a properly scaled gage. This calibration procedure should be performed on each printer used for producing wall gages. Printer calibration factors can vary significantly depending on paper size, orientation (landscape or portrait), and type (heavy or light paper stock), so the calibration procedure should be carried out for each possible combination of these factors that may be used. Calibration factors for different printers and printer configurations can be named and saved in WinFlume for later recall.

The calibration procedure can be started from the wall gage preview form or from the *Options* menu. Instructions appear on the screen. The program will print a test page and then ask the user to measure the length of the printed line. This is used to compute the calibration factor for the printer.

In addition to printer calibration, the computer screen may require calibration, since WinFlume may be run on a wide variety of computers with varying screen resolutions and font size settings. Calibration of the screen ensures that the tick mark labels in the wall gage previews are shown at the proper size relative to the length and thickness of the other gage elements. The screen calibration form is shown in Figure 8.31. Follow the instructions on the screen to adjust the size of the digit "8" until it fills the box shown on the form. When the calibration factor has been determined it will be saved by WinFlume and recalled during all subsequent WinFlume sessions. If you change the resolution or font-size setting of your computer, you should repeat the screen calibration procedure.

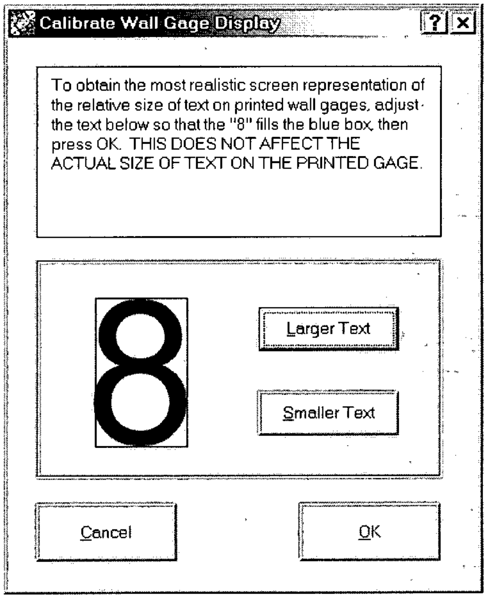


Figure 8.31 Dialog box used to calibrate the screen for accurate previewing of wall gage images.

8.10 Program Options

Several miscellaneous program options can be set from the Options menu:

- Units — Opens the units selection dialog, described in Section 8.7.3.
- User Name — Prompts the user for their name, to be printed on all reports, tables and other program output.
- Calibrate Screen for Wall Gage Preview — Opens the dialog box used to calibrate the screen for properly displaying the previews of printed wall gages. This corrects for different resolution settings on the computer that affect the relative size of text labels shown on wall gages (Section 8.9.7).
- Calibrate Printer for Wall Gage Output — Opens the dialog box used to calibrate the user's printer for most accurate printing of wall gages (Section 8.9.7).
- Show Water Surface Profiles — Allows the user to check the *Min WSP* and *Max WSP* boxes on the bottom-profile screen or select the *Show Maximum/Minimum Water Surface Profiles* commands from the *Options* menu to display approximate water surface profiles through the structure at minimum and maximum flow. If a water surface profile cannot be computed and shown due to hydraulic errors encountered while rating the structure, the check box will be grayed out until it can be computed. The water surface profiles are approximate only, based on the computation of upstream depth, tailwater levels, and critical depth through the structure.
- File-Open Dialog Options — Offers two options for displaying the file-open dialog box used to load new flume (**.flm*) files. *The Explorer File-Open Dialog* displays a standard Windows dialog box that fully supports long filenames

(32-bit version only). The *File-Open Dialog With Flume Summary* may not properly show long filenames, but provides a useful summary of each flume's characteristics as the user browses the file list.

- Save Current Settings as Defaults — Saves the user's current program options in the system registry so that the same options will be used during your next WinFlume session.
- Save Settings on Exit — Saves the current program options when the user exits WinFlume. The same options will then be used during the next WinFlume session.

8.11 Suggested Flume Dimensions

8.11.1 Length of approach channel

The gaging or head-measurement station should be located sufficiently far upstream from the structure to avoid the area of water surface drawdown, yet it should be close enough for the energy loss between the gaging station and the structure to be negligible. To meet these requirements, the gaging station should be located at a distance between two and three times H_{1max} from the leading edge of the sill or at H_{1max} from the beginning of the converging transition, whichever is greater. In addition, if a wall-mounted staff gage or stilling well port is used, there should be no offsets or sudden changes in alignment of the approach channel wall within a distance H_{1max} upstream from the gaging station.

8.11.2 Length of converging transition

The function of the converging transition is to provide a smooth acceleration of flow with no discontinuities or flow separation at the beginning of the throat. With stationary structures the transition commonly consists of plane surfaces. The converging transition should be flatter than 2.5 to 1 (horizontal to vertical). When the structure is viewed in plan, the angle of the line describing the intersection of the water surface with the side walls of the converging transition should also have a "slope" flatter than 2.5 to 1 (longitudinal to transverse distance) at both minimum and maximum flow conditions. With the movable weir the transition usually is rounded with a radius of $r = 0.2H_{1max}$.

8.11.3 Length of control section (i.e., throat, crest, or sill)

For accurate flow measurement the throat length should be chosen so that the ratio of the sill-referenced energy head, H_1, to the throat length, L, is in the range

$$0.070 \leq H_1/L \leq 0.70$$

Within this range WinFlume calculates rating tables with an uncertainty of less than ±2%. Outside this range, the uncertainty gradually increases to about 4% at

H_1/L = 0.05 or 1.0. If a structure is to be designed with a high value of the ratio Q_{max}/Q_{min}, the full range of H_1/L values should be used.

8.11.4 Slope of downstream expansion

If the downstream water level, y_2, is sufficiently low, there is no need for a gradual transition between the throat and the downstream channel, and a sudden expansion can be selected. If the head loss over the structure is limited to such an extent that the downstream water head, h_2, becomes higher than the critical water depth in the throat, a gradual transition with a 6-to-1 slope can be added to regain potential energy. The amount of potential energy that can be regained depends mainly on the degree of expansion of the transition. Rather sudden expansion ratios like 1-to-1 or 2-to-1 are not very effective in energy conversion because the high velocity jet leaving the throat cannot change direction suddenly to follow the boundaries of the transition. Therefore, we do not recommend the use of the expansion ratios 1-to-1, 2-to-1, or 3-to-1. Slopes flatter than 6-to-1 are not justified because of the construction expense and the fact that the additional energy recovery in the transition is offset by additional friction loss over the length of the transition.

8.12 Flume Warnings and Error Messages

Warning and error messages are displayed in rating tables and design review reports when conditions occur that adversely affect the accuracy of a flume or make it impossible to compute the flume rating. Detailed descriptions of each error message are given below:

1 - **Froude number exceeds 0.5 at the gage -** This is a primary design criterion intended to ensure that the water level in the upstream pool can be measured with reasonable accuracy. To eliminate this error, increase the size of the approach channel, reduce the control section width, or increase the height of the sill relative to the invert of the approach channel.

2 - **FATAL: Approach area too small compared to control area** - If the approach area is too small, control of the flow may occur upstream from the intended control section. To eliminate this error message, reduce the control section size or increase the size of the approach channel. Errors 2 and 7 are similar, with error 2 occurring in the routines that determine flow for a given head and error 7 occurring in the routines that determine head for a given flow.

3 - **FATAL: Maximum allowable tailwater depth is zero or less** - The calculation of allowable tailwater depth has indicated that the allowed tailwater depth is zero or less, which makes it impossible to develop any workable design.

4 - **FATAL: Tailwater area too small compared to control area -** The cross-sectional area of the tailwater channel at the maximum allowable tailwater depth must be at least 5 percent larger than the corresponding cross-sectional area of the control section. If this condition is not met, it is possible that critical

flow, and thus the control section may occur in the tailwater channel rather than in the intended control section. To eliminate this error message, increase the size of the tailwater channel, or reduce the size of the control section.

5 - **Upstream energy head / control section length is less than 0.07** - The ratio H_1/L should be in the range of 0.07 to 0.7 to obtain the most accurate flow measurement. This error message is generally associated with the low end of the discharge range of the structure. To eliminate this error, either reduce the control section length or narrow the bottom width of the control section (thereby increasing H_1 at minimum flow).

6 - **Upstream energy head / control section length exceeds 0.7** - The ratio H_1/L should be in the range of 0.07 to 0.7 to obtain the most accurate flow measurement. This error message is generally associated with the upper end of the discharge range of the structure. To eliminate this error, either increase the control section length or widen the control section (especially the top width) to reduce H_1 at maximum flow.

7 - **FATAL: Approach area too small compared to control area** - This error indicates that there is insufficient contraction from the approach channel to the control section to ensure that critical flow will take place in the control section. To eliminate this error, increase the approach channel size, or reduce the size of the control section. This error can also occur if the control section has an extremely complex shape for which the exponent u cannot be determined (numerical solution fails to converge). Errors 2 and 7 are similar, with error 2 occurring in the routines that determine flow for a given head and error 7 occurring in the routines that determine head for a given flow.

8 - **FATAL: Head is zero. This is not allowed** - Flume ratings cannot be determined for an upstream head of zero. If this error occurs, choose a non-zero head as the minimum to be shown in the rating table.

9 - **FATAL: Discharge is zero. This is not allowed** - Flume ratings cannot be determined for a flow rate of zero. If this error occurs, choose a non-zero discharge as the minimum to be shown in the rating table.

10 - **Converging section length is too short (ramp is too steep)** - In flumes whose contraction is primarily formed by a raised sill, the converging ramp slope should be in the range of 2.5:1 to 4.5:1 (horizontal:vertical) to obtain the most accurate flow measurement. If the ramp is too steep, the transition is too abrupt and there will be significant flow separation in the upstream reach of the control section. To remove this error message, increase the length of the converging section so that the ramp slope is 2.5:1 or flatter. *In unusual circumstances it may be impossible to eliminate all of the errors related to converging section length (errors 10, 11, 22, and 23). In these cases, a converging section that is too gradual (errors 11 and 23) is better than a converging section that is too abrupt (errors 10 and 22). (See Section 8.8.9).*

11 - **Converging section length may be too long (ramp is too flat)** - In flumes whose contraction is primarily formed by a raised sill, the converging ramp slope should be in the range of 2.5:1 to 4.5:1 (horizontal:vertical) to obtain the most accurate flow measurement. If the ramp is too flat, there may be excessive friction loss between the gaging station location and the control section, and the construction cost of the flume may be greater than necessary. To remove

this error message, reduce the length of the converging section so that the ramp slope is 4.5:1 or steeper. *In unusual circumstances it may be impossible to eliminate all of the errors related to converging section length (errors 10, 11, 22, and 23). In these cases, a converging transition that is too gradual (errors 11 and 23) is better than a converging transition that is too abrupt (errors 10 and 22). (See Section 8.8.9).*

12 - **Gage is too close to converging section and/or throat** - The gaging- or head-measurement station should be located sufficiently far upstream from the structure to avoid the area of water surface drawdown, yet it should be close enough for the energy loss between the gaging station and the structure to be negligible. To meet these requirements, the gaging station should be located at a distance between two and three times H_{1max} from the leading edge of the sill or at H_{1max} from the beginning of the converging transition, whichever is greater. To eliminate this error, increase the approach channel length on the flume bottom profile drawing.

13 - **Tailwater ramp is too flat** - A gradually sloped tailwater ramp helps to recover energy downstream of the flume by converting some of the kinetic energy of the flow in the critical section back into potential energy in the tailwater channel. A slope of 6:1 (horizontal:vertical) is recommended. If the slope is flatter than 10:1, the ramp becomes so long that additional friction loss along the length of the ramp probably exceeds the additional energy recovery. To eliminate this error message, increase the slope of the tailwater ramp or reduce its length to obtain a slope of 10:1 or steeper.

14 - **Upstream energy head exceeds channel depth** - This error message indicates that the approach channel energy head (water level plus velocity head) is above the banks of the upstream channel. If the velocity head is a large part of the total energy head, the water level may actually be within the channel banks, but this is still an undesirable flow condition because there is so little freeboard in the approach channel that it could conceivably spill over the banks if the flow were brought to a stop by a local offset in the canal lining or an obstruction in the approach channel. To eliminate this error message, reduce the contraction in the control section or increase the size or top elevation of the approach channel. Note that WinFlume will compute an h_1 vs. Q rating if the water level is below the top of the channel, even though the energy head may be above the top of the channel. This is considered a fatal error in WinFlume's design evaluation routines.

15 - **FATAL: Attempt to lower movable weir sill too much. Try reducing sill radius** - Movable weirs cannot be lowered to a sill height that is less than the sill radius. To eliminate this error, reduce the sill radius, or increase the width of the control section (which reduces the required head and allows the movable weir to be operated at a higher sill height).

16 - **FATAL: Movable weir water depth must be over 1.5 times sill radius** - The constant operating depth upstream from a movable weir must be at least 1.5 times the radius of the approach transition on the movable weir crest. To eliminate this error, reduce the sill radius or increase the operating depth.

17 - **FATAL: Approach channel water level exceeds 0.9 times diameter** - When the approach channel is circular, the flow depth in the approach channel cannot

be greater than 90 percent of the diameter. This flow condition is susceptible to unpredictable filling of the circular conduit due to the rapid increase in wetted perimeter as the depth increases. To eliminate this error, increase the approach channel diameter or enlarge the control section (perhaps by reducing the sill height).

18 - **FATAL: Control section water level exceeds 0.9 times diameter** - When the control section is circular, the water level in the control section cannot be in the top 10 percent of the circular section. This flow condition is susceptible to unpredictable filling of the circular conduit due to the rapid increase in wetted perimeter as the depth increases. To eliminate this error, reduce the contraction in the control section, and/or increase the control section diameter.

19 - **Tailwater channel water level limited to 0.9 times diameter** - When the tailwater channel is circular, the flow depth in the tailwater channel cannot be greater than 90 percent of the diameter. This flow condition is susceptible to unpredictable filling of the circular conduit due to the rapid increase in wetted perimeter as the depth increases. To eliminate this error, increase the tailwater channel diameter.

20 - **FATAL: Approach channel water level exceeds top of control section circle** - The water level in the approach channel cannot be higher than the crown of a circular control section, because such a condition will likely lead to pressurized flow in the control section, or an inlet-controlled orifice flow condition rather than a critical-depth controlled open-channel flow. To eliminate this error, increase the control section diameter, or enlarge the control section (thereby reducing the upstream water level).

21 - **FATAL: Submergence ratio exceeds modular limit. Flow will not be critical** - If the modular limit is exceeded, the flow in the control section will not be critical and there will not be a unique relation between upstream sill-referenced head and discharge (the relation will vary as a function of tailwater). To eliminate this error message, increase the contraction of the control section.

22 - **Converging section length is too short (side contraction is too abrupt)** - When viewed in plan, flumes that are primarily side-contracted should have a contraction angle from the approach channel to the control section that is in the range of 2.5:1 to 4.5:1 (longitudinal to lateral distance). WinFlume checks to see that this condition is met at two elevations, the invert of the control section and the level of the approach channel water surface. If the transition is too abrupt there will be significant flow separation in the upstream reach of the control section. To eliminate this error, increase the length of the converging section, or reduce the difference in width between the approach channel and the control section. *In unusual circumstances it may be impossible to eliminate all of the errors related to converging section length (errors 10, 11, 22, and 23). In these cases, a converging transition that is too gradual (errors 11 and 23) is better than a converging transition that is too abrupt (errors 10 and 22). (See Section 8.8.9).*

23 - **Converging section length may be too long (side contraction is too flat)** - When viewed in plan, flumes that are primarily side-contracted should have a contraction angle from the approach channel to the control section that is in the range of 2.5:1 to 4.5:1 (longitudinal to lateral distance). WinFlume checks to

see that this condition is met at two elevations, the invert of the control section and the level of the approach channel water surface. If the transition is too long, there will be excessive friction loss between the gaging station location and the control section, and the structure may be more expensive to construct. To eliminate this error, reduce the length of the converging section. *In unusual circumstances it may be impossible to eliminate all of the errors related to converging section length (errors 10, 11, 22, and 23). In these cases, a converging transition that is too gradual (errors 11 and 23) is better than a converging transition that is too abrupt (errors 10 and 22). (See Section 8.8.9).*

24 - For width-contracted flumes, $L/W \geq 2$ is recommended for throat section. - Preliminary laboratory tests have shown that flumes that are solely width-contracted (i.e., no sill to create a vertical contraction of flow entering the throat section), have the potential to not properly develop critical-depth flow across the full width of the throat section. At this time, a length-to-width ratio of 2:1 or greater is recommended for width-contracted flumes. The length-to-width (L/W) ratio is evaluated using the average of the throat widths at the sill elevation and at the elevation corresponding to h_1. Future research may allow refinement of this criterion.

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Appendix 1. List of Symbols

A	=	cross-sectional area perpendicular to flow (flow area)
A_1	=	cross sectional area of approach section, or factor in bed-load transport equation
A^*	=	imaginary area of flow at control section for water depth at same elevation as in approach canal
ΔA	=	incremented flow area
a	=	centrifugal acceleration of water particle
B	=	water surface or top width of flow
B^*	=	imaginary top width of flow at control section for water depth at same elevation as in approach canal
b	=	bottom width
C	=	Chezy coefficient
C_d	=	discharge coefficient
C_F	=	drag coefficient
$C_{F,L}$	=	turbulent drag coefficient for distance L
$C_{F,x}$	=	turbulent drag coefficient for distance L_x
$C_{f,x}$	=	laminar drag coefficient for distance L_x
C_u	=	units coefficient for Manning equation
C_v	=	approach velocity coefficient
D	=	average or hydraulic depth (flow area per unit length of top width of flow), or pipe diameter
D_a	=	characteristic particle diameter
D_P	=	drain pipe diameter
d	=	canal depth;
	=	diameter of pipe or float
d_x	=	diameter of sieve opening where x% of the total weight of the sample passes through the sieve
E	=	total energy head of particle with reference to an arbitrary elevation
F	=	canal freeboard;
	=	flexibility of two structures;
	=	lifting force;
	=	tensile force on float tape
Fr	=	Froude number
ΔF	=	change in tensile force on float tape
f	=	focal distance for parabolic cross section shape
f	=	friction coefficient
g	=	acceleration of gravity
H	=	energy head of flow referenced to weir sill
H_b	=	height of triangle in complex shape control
H_d	=	energy head of flow in tailwater channel relative to floor of energy dissipation structure
H_i	=	energy head of flow at control section relative to floor of energy dissipation structure

ΔH	=	loss in energy head over flume or weir
ΔH_1	=	energy loss due to friction for upstream part of structure
ΔH_2	=	energy loss for downstream part of structure
ΔH_L	=	energy loss due to friction over throat
ΔH_a	=	energy loss due to friction between gauging station and start of converging transition (L_a)
ΔH_b	=	energy loss due to friction over converging transition (L_b)
ΔH_d	=	energy loss due to friction over diverging transition
ΔH_e	=	energy loss due to friction over part of tailwater channel
ΔH_f	=	energy loss due to friction over downstream part of structure
ΔH_k	=	energy loss due to turbulence from downstream expansion
h	=	head referenced to weir sill
h_s	=	side slope distance for marking direct reading gages
Δh	=	change in sill-referenced flow depth across flume
Δh_1	=	difference between measured and true value of h_1
k	=	absolute roughness height of material
K_1	=	constant in power head-discharge equation
K_2	=	offset in power head-discharge equation
L	=	throat length;
	=	length
L_B	=	basin length for energy dissipation structure
L_{board}	=	length of divisor board
L_a	=	distance from gaging station to start of converging transition
L_b	=	length of converging transition
L_d	=	length of diverging transition
L_e	=	length of tailwater channel from end of diverging transition to fully developed flow (section 2)
L_j	=	length of hydraulic jump
L_{model}	=	length dimension in model structure
L_p	=	length from flume throat to section U in energy dissipation structure
	=	length of drain pipe
$L_{prototype}$	=	length dimension in prototype structure
L_x	=	distance to transition between laminar and turbulent boundary layers
ΔL	=	change in water level causing registration error due to changing weight of float tape
ML	=	modular limit
m	=	mass of fluid particle;
	=	ratio of lengths for downstream expansion or diverging transition (horizontal to vertical)
N	=	number of measurements
n	=	Manning roughness coefficient;
	=	height of step or end sill from floor of energy dissipation structure
n_3	=	height of baffle blocks
P	=	pressure on water particle
ΔP	=	change in pressure
p	=	sill height relative to channel bottom

Δp	=	change in sill height or drop in channel bottom across weir or flume
Q	=	flow rate or discharge
Q_{model}	=	discharge for model structure
Q_{new}	=	newly calculated or adjusted discharge
$Q_{prototype}$	=	discharge for prototype structure
$Q_{overtopping}$	=	discharge when water level is at the top of the canal just prior to overtopping
Q_{rate}	=	unadjusted discharge from rating table
Q_i	=	flow rate for an ideal fluid
$Q_{o,1}$	=	flow rate for off-take structure 1
$Q_{o,2}$	=	flow rate for off-take structure 2
$Q_{o,3}$	=	flow rate for off-take structure 3
$Q_{s,1}$	=	flow rate for continuing supply canal structure 1
$Q_{s,2}$	=	flow rate for continuing supply canal structure 2
$Q_{s,3}$	=	flow rate for continuing supply canal structure 3
ΔQ	=	discharge through stream tube (incremental flow rate);
	=	change in flow rate
$\Delta Q_{freeboard}$	=	additional discharge that can be conveyed within the freeboard
q	=	discharge per unit width
R	=	hydraulic radius (flow area per unit length of wetted perimeter of flow)
Re_L	=	length Reynolds number based on L
Re_x	=	length Reynolds number based on L_x
r	=	radius of: float wheel, circle, pipe, streamline curvature, or transition between plane surfaces
$\Delta\mathrm{r}$	=	increment in radius of curvature
S_b	=	channel bottom slope
S_f	=	hydraulic gradient
S	=	sensitivity of measuring device
T	=	sediment transport capacity;
	=	horizontal force on movable gate
T_f	=	resisting torque due to friction on float-wheel axle
t	=	temperature of water particle;
u	=	exponent or power of head-discharge equation
	=	actual velocity of water particle
ΔV	=	change in volume of submerged float section
v	=	average velocity of flow
w	=	float tape weight per unit length
W	=	weight of movable weir
X	=	distance from gauging station to start of throat
	=	dimensionless transport parameter
X_C	=	error in discharge from equations or rating tables
X_{h1}	=	error in upstream sill-referenced head
X_Q	=	error in measured flow rate
Y	=	flow parameter for sediment transport
y	=	actual water depth
y_d	=	depth of flow in tailwater channel relative to floor of energy dissipation structure

y_{sub}	=	flow depth at subcritical flow
y_{super}	=	flow depth at supercritical flow
Z	=	elevation of water particle
z	=	canal wall side slope (horizontal to vertical)
ΔZ	=	elevation difference from weir crest to floor of energy dissipation structure
α	=	velocity distribution coefficient
γ	=	ratio of maximum to minimum flow rate to be measured with a flume
$\delta_{ha} \rightarrow \delta_{hn}$	=	random errors in head measurement from miscellaneous sources a to n
δ_{h1}	=	combined error in head measurement at gaging station
ε	=	relative velocity factor
ϕ	=	angle made from center of pipe to edges of sill placed in circular channel
ξ	=	energy loss coefficient for downstream transition
μ	=	ripple factor
π	=	pi = 3.1416...
ρ	=	mass density of water
ρ_r	=	relative density
ρ_s	=	density of sediments
v_i	=	kinematic viscosity of fluid
θ	=	angle of opening for prismatic channels;
	=	angle made from center of circular channel to edges of water surface or sill crest

Subscripts

1	corresponds to head-measurement section or gaging station corresponds to location 1 of stream tube
2	corresponds to section in tailwater channel downstream from structure corresponds to location 2 of stream tube
b	corresponds to section at entrance to flume throat
c	corresponds to control section within weir or flume throat
d	corresponds to section within energy dissipation structure downstream from the hydraulic jump
o	corresponds to offtake structure
s	corresponds to continuing supply canal structure
u	corresponds to section U of energy dissipation structure
min	corresponds to minimum design or anticipated flow rate
max	corresponds to maximum design or anticipated flow rate
guess	initial value of variable
new	new computed value of variable
,	comma implies multiple subscripts

Appendix 2. Factors for Conversion of Units

Table 1. Adapted from King, H.W. and Brater, E.F. 1963. To reduce A to B, multiply A by F. To reduce B to A, multiply B by G.

Unit A	Factor F	Factor G	Unit B
Length			
Miles	63 360.[a]	0.000 015 783	Inches
Miles	5 280.[a]	0.000 189 39	Feet
Miles	1 609.34	0.000 621 37	Meters
Miles	1.609 34	0.621 37	Kilometers
Kilometers	3 280.84	0.000 304 8[a]	Feet
Meters	3.280 8	0.304 8[a]	Feet
Yards	36.[a]	0.027 778	Inches
Feet	12.[a]	0.083 333	Inches
Meters	39.370	0.025 4[a]	Inches
Inches	2.54[a]	0.393 70	Centimeters
Surface Area			
Square miles	27 878 400.[a]	0.000 000 035 870	Square feet
Square miles	640.[a]	0.001 562 5[a]	Acres
Square miles	259.000	0.003 861 0	Hectares
Acres	43 560.[a]	0.000 022 957	Square feet
Acres	4 046.9	0.000 247 10	Square meters
Hectares	2.471 05	0.404 69	Acres
Hectares	10 000.[a]	0.000 1[a]	Square meters
Square feet	144.[a]	0.006 944 4	Square inches
Square inches	6.451 6[a]	0.155 00	Square centimeters
Square meters	10.764	0.092 903	Square feet
Volume			
Cubic feet	1 728.[a]	0.000 578 70	Cubic inches
Cubic inches	16.387	0.061 024	Cubic centimeters
Cubic meters	35.314 7	0.028 317	Cubic feet
Cubic meters	1.307 95	0.764 55	Cubic yards
Cubic feet	7.480 5	0.133 68	U.S. gallons
Cubic feet	6.232 1	0.160 46	Imperial gallons
Cubic feet	28.317	0.035 315	Liters
U.S. gallons	231.[a]	0.004 329 0	Cubic inches
Imperial gallons	277.274	0.003 606 5	Cubic inches
Liters	61.023 7	0.016 387	Cubic inches
U.S. gallons	3.785 4	0.264 17	Liters
Imperial gallons	1.200 3	0.833 11	U.S. gallons
Imperial gallons	4.543 7	0.220 09	Liters

Unit A	Factor F	Factor G	Unit B
Fluid ounces	1.8047	0.55411	Cubic inches
Acre-feet	43 560.[a]	0.000 022 957	Cubic feet
Acre-feet	1 613.3	0.000 619 83	Cubic yards
Acre-feet	1 233.5	0.000 810 71	Cubic meters
Acre-inches	3 630.[a]	0.000 275 48	Cubic feet
Million U.S. gallons	3.068 9	0.325 85	Acre-feet
Velocity			
Miles per hour	1.466 7	0.681 82	Feet per second
Meters per second	3.280 8	0.304 8[a]	Feet per second
Meters per second	2.236 9	0.447 04	Miles per hour
Kilometers per hour	0.621 4	1.609 3	Miles per hour
Kilometers per hour	0.911 3	1.097 3	Feet per second
Discharge			
Cubic meter per second	35.314 7	0.028 317	Cubic feet per second
Cubic meter per second	1 000.[a]	0.001[a]	Liters per second
Cubic meter per second	86 400.[a]	0.000 011 574	Cubic meters per 24 hours
Cubic meter per second	15 850.20	0.000 063 09	U.S. gallons per minute
Cubic feet per second	28.317	0.035 31	Liters per second
Cubic feet per second	60.[a]	0.016 667	Cubic feet per minute
Cubic feet per second	86 400.[a]	0.000 001 574	Cubic feet per 24 hours
Cubic feet per second	448.83	0.002 228 0	U.S. gallons per minute
Cubic feet per second	646 317.	0.000 001 547 2	U.S. gallons per 24 hours
Cubic feet per second	1.983 5	0.504 17	Acre-feet per 24 hours
Cubic feet per second	723.97	0.001 381 3	Acre-feet per 365 days
Cubic feet per second	50.[a]	0.02[a]	Miner's inches, ID, KA, NB, NM, ND, SD, UT, WA (USA)
Cubic feet per second	40.[a]	0.025[a]	Miner's inches, AZ, CA, MT, NV, OR (USA)
Cubic feet per second	38.4[a]	0.026 042	Miner's inches, CO (USA)
Million U.S. gallons per day	1.547 2	0.646 32	Cubic feet per second
Inches depth per hour	645.33	0.001 549 6	CFS[b] per square mile
Inches depth per day	26.889	0.037 190	CFS[b] per square mile
CFS[b] per square mile	13.574	0.073 668	Inches depth per 365 days
Acre-inches per hour	1.008 3[c]	0.991 73[c]	Cubic feet per second
Cubic feet per minute	7.480 5	0.133 68	U.S. gallons per minute
Cubic feet per minute	10 772.	0.000 092 834	U.S. gallons per 24 hours
U.S. gallons per minute	1 440.[a]	0.000 694 44	U.S. gallons per 24 hours

[a]Exact values.

[b]CFS = cubic feet per second.

[c]Often approximated as 1.0.

Table 2. Conversion of inches to millimeters.

Inches	Millimeters	Inches	Millimeters	Inches	Millimeters	Inches	Millimeters
1/32	0.79	1 5/16	33.34	0.01	0.25	0.51	12.95
1/16	1.59	1 3/8	34.93	0.02	0.51	0.52	13.21
3/32	2.38	1 7/16	36.51	0.03	0.76	0.53	13.46
1/8	3.18	1 1/2	38.10	0.04	1.02	0.54	13.72
5/32	3.97	1 9/16	39.69	0.05	1.27	0.55	13.97
3/16	4.76	1 5/8	41.28	0.06	1.52	0.56	14.22
7/32	5.56	1 11/16	42.86	0.07	1.78	0.57	14.48
1/4	6.35	1 3/4	44.45	0.08	2.03	0.58	14.73
9/32	7.14	1 13/16	46.04	0.09	2.29	0.59	14.99
5/16	7.94	1 7/8	47.63	0.10	2.54	0.60	15.24
11/32	8.73	1 15/16	49.21	0.11	2.79	0.61	15.49
3/8	9.53	2	50.80	0.12	3.05	0.62	15.75
13/32	10.32	2 1/8	53.98	0.13	3.30	0.63	16.00
7/16	11.11	2 1/4	57.15	0.14	3.56	0.64	16.26
15/32	11.91	2 3/8	60.33	0.15	3.81	0.65	16.51
1/2	12.70	2 1/2	63.50	0.16	4.06	0.66	16.76
17/32	13.49	2 5/8	66.68	0.17	4.32	0.67	17.02
9/16	14.29	2 3/4	69.85	0.18	4.57	0.68	17.27
19/32	15.08	2 7/8	73.03	0.19	4.83	0.69	17.53
5/8	15.88	3	76.20	0.20	5.08	0.70	17.78
21/32	16.67	3 1/8	79.38	0.21	5.33	0.71	18.03
11/16	17.46	3 1/4	82.55	0.22	5.59	0.72	18.29
23/32	18.26	3 3/8	85.73	0.23	5.84	0.73	18.54
3/4	19.05	3 1/2	88.90	0.24	6.10	0.74	18.80
25/32	19.84	3 5/8	92.08	0.25	6.35	0.75	19.05
13/16	20.64	3 3/4	95.25	0.26	6.60	0.76	19.30
27/32	21.43	3 7/8	98.43	0.27	6.86	0.77	19.56
7/8	22.23	4	101.60	0.28	7.11	0.78	19.81
29/32	23.02	4 1/4	107.95	0.29	7.37	0.79	20.07
15/16	23.81	4 1/2	114.30	0.30	7.62	0.80	20.32
31/32	24.61	4 3/4	120.65	0.31	7.87	0.81	20.57
1	25.40	5	127.00	0.32	8.13	0.82	20.83
1 1/16	26.99	5 1/4	133.35	0.33	8.38	0.83	21.08
1 1/8	28.58	5 1/2	139.70	0.34	8.64	0.84	21.34
1 3/16	30.16	5 3/4	146.05	0.35	8.89	0.85	21.59
1 1/4	31.75	6	152.40	0.36	9.14	0.86	21.84
				0.37	9.40	0.87	22.10
				0.38	9.65	0.88	22.35
				0.39	9.91	0.89	22.61
				0.40	10.16	0.90	22.86
				0.41	10.41	0.91	23.11
				0.42	10.67	0.92	23.37
				0.43	10.92	0.93	23.62
				0.44	11.18	0.94	23.88
				0.45	11.43	0.95	24.13
				0.46	11.68	0.96	24.38
				0.47	11.94	0.97	24.64
				0.48	12.19	0.98	24.89
				0.49	12.45	0.99	25.15
				0.50	12.70	1.00	25.40

Appendix 3. Glossary

Approach Channel

The approach channel is the canal reach between the gaging station location and the beginning of the converging transition. The approach channel is necessary for the development of uniform and symmetric flow conditions and the establishment of a stable water surface whose elevation can be determined accurately—the approach channel may be lined or may be the original earthen channel.

Control Section

The control section is the location at which the flow passes through critical depth. The control section is located within the throat section of the flume, usually about one-third of the throat section length upstream of the downstream end of the throat.

Converging Transition

The converging transition connects the approach channel to the control section of the structure. In the converging transition section the subcritical approach flow must accelerate smoothly toward the control section with no discontinuities or flow separation—the transition may consist of plane surfaces or may be rounded.

Crest

The crest is the raised portion of a flume or broad-crested weir, horizontal in the flow direction. The upstream sill-referenced head is measured relative to the invert of the crest shape.

Critical Depth and the Froude Number

Critical depth occurs in open-channel flows when the Froude number is equal to 1.0. The Froude number is defined as the ratio of the flow velocity, v, to the celerity (speed) of a gravity wave:

$$\mathrm{Fr} = \frac{v}{\sqrt{gD}}$$

where g is the acceleration of gravity and D is the hydraulic depth, which is defined as the cross-sectional area of the channel normal to the flow direction divided by the width of the free surface.

When the Froude number is less than 1.0, the flow is *subcritical* and it is possible for gravity waves to propagate upstream (the wave celerity is high enough to overcome the flow velocity). When the Froude number is greater than 1.0, the flow is *supercritical* and it is impossible for a gravity wave to propagate upstream because the celerity is less than the flow velocity. Long-throated flumes and other critical-flow measurement devices create a transition from subcritical to supercritical flow. Flow conditions in the tailwater channel cannot affect flow conditions in the approach channel because gravity waves in the flow cannot propagate upstream through the critical section. As long as critical flow can be maintained in the throat of the flume, there is a unique relation between upstream sill-referenced head and discharge through the critical section; this relation is unaffected by flow conditions in the tailwater channel.

Diverging Transition

The diverging transition is the section in which the velocity of the supercritical flow exiting the throat section is reduced and energy is dissipated or partially recovered. If energy recovery is not needed, an abrupt transition can be used.

Energy Grade Line

The energy grade line is a line along the channel profile specifying the total energy head at any point in the channel. The elevation of the line at any point is the sum of the channel invert elevation, the flow depth, and velocity head, $v^2/2g$. If a stick were held in the flow so as to bring the water velocity to zero on the upstream side of the stick, the water level upstream of the stick would theoretically rise to the height of the energy grade line (neglecting losses).

Freeboard

Freeboard is the distance between the upstream water level and the top of the upstream channel, as specified in the Channel Depth text box on the flume bottom profile drawing. It should be noted that WinFlume will not allow the design of a flume for which the upstream energy grade line (water level plus velocity head) exceeds the channel depth, and will not compute rating tables for this range of flows. Thus, although you could specify an allowable freeboard of zero, WinFlume will still require a freeboard at least equal to the upstream velocity head.

Froude Number

Square root of ratio of inertia to gravity forces. See also critical depth.

Gaging Station

The gaging station is located in the approach channel and is the location at which the difference in elevation between the approach water level and the crest of the throat section will be measured. The flow rate through the flume will be computed as a function of this upstream sill-referenced head. The upstream sill-referenced head can be measured with a staff gage or any of a variety of available automated sensors. The measurement can be made directly in the canal or in a stilling well tapped into the canal at the gaging location.

Long-Throated Flume

The term *long-throated flume* is used to describe a broad class of critical-flow devices used to measure the flow of water in open channels. Long-throated flumes are hydraulically similar to broad-crested weirs. The name implies that the control section is of sufficient length in the flow direction that one-dimensional flow is produced at the critical section. This permits the application of one-dimensional hydraulic theory to the problem of determining a calibration, or rating curve for a flume. Thus, with the use of the WinFlume software, long-throated flumes can be calibrated without the need for laboratory testing. The common configuration of a ramp and sill in a trapezoidal channel has also come to be known as a *Replogle flume* or *ramp flume*.

Modular Limit

The modular limit is the maximum submergence ratio (H_2/H_1) for which a flume will operate with critical flow in the throat section. If the actual submergence ratio is less than or equal to the modular limit, there will be a unique functional relationship between the upstream sill-referenced head and the discharge.

Sill

See Crest.

Sill-Referenced Head

See Upstream sill-referenced head.

Submergence Protection

Submergence protection is the vertical distance between the flume's allowable tailwater level and the actual tailwater level at the site. It can be thought of as the designer's insurance against errors made in estimating tailwater conditions at the site. Choosing a design with more submergence protection (and thus, more head loss), allows for some error in estimated tailwater levels without causing the flume to be submerged. If a flume is being added to an existing canal system and little head is available, the designer may have no choice but to select a design with less submergence protection.

Tailwater Channel

The tailwater channel is on the downstream side of the structure. Within the tailwater channel the water level is a function of canal operations, the flow rate, and the hydraulic properties of the downstream channel and structures. The range of water levels in this channel is fundamentally important to the design of the structure because it determines the elevation and size of the control section needed to maintain critical flow conditions through the flume.

Throat Section

The control section of the flume is the region in which the flow goes through critical depth. The general term *control section* is usually used to describe this flume component, but the alternatives terms *crest, sill*, or *throat* are also used at times, depending somewhat on the particular configuration. The control section must be horizontal in the direction of flow, but in the direction perpendicular to the flow, any shape may be used.

Upstream Sill-Referenced Head

The elevation of the water level at the gaging station location, relative to the invert of the flume crest, sill, or throat section.

Appendix 4. Rating Tables

Table R.1.	Rating Tables for Weirs in Trapezoidal Lined Canals in Metric Units ($X_C = 3\%$).
Table R.2.	Rating Tables for Weirs in Trapezoidal Lined Canals in English Units ($X_C = 3\%$).
Table R.3.	Rating Tables for Rectangular Weirs in Metric Units with Discharge per Meter Width ($X_C = 3\%$).
Table R.4.	Rating Tables for Rectangular Weirs in English Units with Discharge per Foot Width ($X_C = 3\%$).
Table R.5.	Rating Tables for Triangular-Throated Flumes ($X_C = 2\%$).
Table R.6.	Rating Tables for Sills in 1-m Diameter Circular Conduits ($X_C = 2\%$).
Table R.7.	Rating Tables for Sills in 1-ft Diameter Circular Diameter ($X_C = 2\%$).
Table R.8.	Rating Tables for Movable Weirs in Metric Units ($X_C = 3\%$).
Table R.9.	Rating Tables for Movable Weirs in English Units ($X_C = 3\%$).
Table R.10.	Head-Discharge Relationships of 5 RBC Flumes for Use in Unlined Channels (Metric Units) ($X_C = 2\%$).
Table R.11.	Head-Discharge Relationships of 5 RBC Flumes for Use in Unlined Channels (English Units) ($X_C = 2\%$).
Table R.12.	Rating Tables for Commercially Available Rectangular Fiberglass Flume ($X_C = 2\%$).
Table R.13.	Rating Equations and Properties of Adjust-A-Flumes in Metric Units ($X_C = 2\%$).
Table R.14.	Rating Equations and Properties of Adjust-A-Flumes in English Units ($X_C = 2\%$).

Table R.1. Rating Tables for Weirs in Trapezoidal Lined Canals in Metric Units ($X_C = 3\%$).[a]

Weir A_m, $b_c = 0.50$ m, $0.23 \le L \le 0.34$ m		Weir B_m, $b_c = 0.60$ m, $0.30 \le L \le 0.42$ m		Weir C_m, $b_c = 0.70$ m, $0.35 \le L \le 0.51$ m		Weir D_{m1}, $b_c = 0.80$ m, $0.40 \le L \le 0.58$ m		Weir D_{m2}, $b_c = 0.80$ m, $0.30 \le L \le 0.45$ m		Weir E_{m1}, $b_c = 0.90$ m, $0.38 \le L \le 0.56$ m	
Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)
0.005	0.032	0.005	0.029	0.01	0.041	0.01	0.038	0.01	0.038	0.01	0.035
.010	.049	.010	.045	.02	.063	.02	.059	.02	.058	.02	.055
.015	.063	.015	.058	.03	.081	.03	.075	.03	.075	.03	.071
.020	.075	.020	.069	.04	.096	.04	.090	.04	.089	.04	.084
.025	.086	.025	.078	.05	.110	.05	.103	.05	.102	.05	.097
.030	.095	.030	.088	.06	.123	.06	.115	.06	.113	.06	.108
.035	.104	.035	.096	.07	.135	.07	.127	.07	.124	.07	.119
.040	.112	.040	.104	.08	.146	.08	.137	.08	.134	.08	.129
.045	.120	.045	.111	.09	.156	.09	.147	.09	.144	.09	.139
.050	.127	.050	.118	.10	.166	.10	.156	.10	.153	.10	.148
.055	.135	.055	.125	.11	.175	.11	.165	.11	.162	.11	.156
.060	.141	.060	.132	.12	.184	.12	.174	.12	.170	.12	.164
.065	.148	.065	.138	.13	.192	.13	.182	.13	.178	.13	.172
.070	.154	.070	.144	.14	.200	.14	.190	.14	.186	.14	.180
.075	.160	.075	.149	.15	.208	.15	.198	.15	.193	.15	.187
.080	.166	.080	.155	.16	.216	.16	.205	.16	.200	.16	.194
.085	.171	.085	.160	.17	.223	.17	.212	.17	.207	.17	.201
.090	.177	.090	.166	.18	.230	.18	.219	.18	.214	.18	.208
.095	.182	.095	.171	.19	.237	.19	.226	.19	.220	.19	.215
.100	.187	.100	.176	.20	.244	.20	.232	.20	.227	.20	.221
.105	.192	.105	.181	.21	.251	.21	.239	.21	.233	.21	.227
.110	.197	.110	.185	.22	.257	.22	.245	.22	.239	.22	.233
.115	.201	.115	.190	.23	.263	.23	.251	.23	.245	.23	.239
.120	.206	.120	.194	.24	.269	.24	.257	.24	.251	.24	.245
.125	.211	.125	.199	.25	.275	.25	.263	.25	.257	.25	.251
.130	.215	.130	.203	.26	.281	.26	.269	.26	.262	.26	.256
.135	.219	.135	.207	.27	.287	.27	.274	.27	.268	.27	.262
.140	.224	.140	.212	.28	.293	.28	.280	.28	.273	.28	.267
		.145	.216	.29	.298	.29	.285	.29	.278	.29	.272
		.150	.220	.30	.304	.30	.291	.30	.283	.30	.278
		.160[b]	.228	.31	.309	.32[b]	.301	.32[b]	.293	.32[b]	.288
		.170	.235	.32	.314	.34	.311	.34	.311	.34	.298
		.180	.242	.33	.319	.36	.321	.36	.321	.36	.307
		.190	.250	.34	.324	.38	.330	.38	.330	.38	.316
		.200	.256	.35	.329	.40	.339	.40	.339	.40	.325
		.210	.263	.36	.334	.42	.348	.42	.348	.42	.334
		.220	.270	.37	.339	.44	.357	.44	.357	.44	.342
		.230	.276	.38	.344	.46	.365	.46	.365	.46	.351
		.240	.282			.48	.374	.48	.373	.48	.359
						.50	.382	.50	.382	.50	.367
						.52	.390	.52	.389	.52	.375
$K_1 = 2.226$[c]		$K_1 = 2.389$		$K_1 = 2.675$		$K_1 = 2.849$		$K_1 = 2.879$		$K_1 = 2.956$	
$K_2 = 0.0083$		$K_2 = 0.0083$		$K_2 = 0.0122$		$K_2 = 0.0120$		$K_2 = 0.0089$		$K_2 = 0.0100$	
$u = 1.898$		$u = 1.872$		$u = 1.900$		$u = 1.879$		$u = 1.843$		$u = 1.832$	

(continued)

[a] See Table 5.2 for flume dimensions and head-loss values.

[b] Change in discharge increment.

[c] The following equation approximates the values in the paired columns of Table R.1 by applying the K_1, K_2, and u factors at the bottom of each column pair: $Q = K_1 (h_1 + K_2)^u$

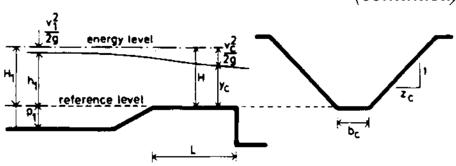


Table R.1. *(continued)*[a]

Weir E_{m2} $b_c = 0.90$ m $0.38 \le L \le 0.56$ m		Weir F_{m1} $b_c = 1.0$ m $0.42 \le L \le 0.61$ m		Weir F_{m2} $b_c = 1.0$ m $0.42 \le L \le 0.61$ m		Weir G_{m1} $b_c = 1.2$ m $0.50 \le L \le 0.75$ m		Weir G_{m2} $b_c = 1.2$ m $0.45 \le L \le 0.68$ m		Weir H_m $b_c = 1.4$ m $0.56 \le L \le 0.84$ m	
Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)
0.01	0.035	0.01	0.033	0.01	0.033	0.02	0.046	0.02	0.046	0.02	0.042
.02	.055	.02	.051	.02	.051	.04	.072	.04	.072	.04	.066
.03	.070	.03	.066	.03	.066	.06	.093	.06	.092	.06	.085
.04	.084	.04	.079	.04	.079	.08	.111	.08	.110	.08	.102
.05	.096	.05	.091	.05	.091	.10	.127	.10	.126	.10	.117
.06	.108	.06	.102	.06	.102	.12	.142	.12	.141	.12	.131
.07	.118	.07	.112	.07	.112	.14	.156	.14	.155	.14	.144
.08	.128	.08	.122	.08	.121	.16	.169	.16	.167	.16	.156
.09	.137	.09	.131	.09	.130	.18	.181	.18	.179	.18	.167
.10	.146	.10	.140	.10	.139	.20	.193	.20	.191	.20	.178
.11	.155	.11	.148	.11	.147	.22	.204	.22	.202	.22	.189
.12	.163	.12	.156	.12	.155	.24	.215	.24	.213	.24	.199
.13	.170	.13	.163	.13	.162	.26	.225	.26	.223	.26	.209
.14	.178	.14	.171	.14	.170	.28	.235	.28	.232	.28	.218
.15	.185	.15	.178	.15	.177	.30	.244	.30	.242	.30	.227
.16	.192	.16	.185	.16	.183	.32	.254	.32	.251	.32	.236
.17	.199	.17	.192	.17	.190	.34	.263	.34	.260	.34	.244
.18	.206	.18	.198	.18	.196	.36	.271	.36	.268	.36	.253
.19	.212	.19	.204	.19	.203	.38	.280	.38	.277	.38	.261
.20	.218	.20	.211	.20	.209	.40	.288	.40	.285	.40	.269
.21	.224	.22[b]	.223	.22[b]	.221	.42	.296	.42	.293	.42	.276
.22	.230	.24	.234	.24	.232	.44	.304	.44	.301	.44	.284
.23	.236	.26	.245	.26	.243	.46	.312	.46	.308	.46	.291
.24	.242	.28	.256	.28	.253	.48	.319	.48	.316	.48	.298
.25	.247	.30	.266	.30	.263	.50	.327	.50	.323	.50	.305
.26	.253	.32	.276	.32	.273	.55[b]	.344	.55[b]	.340	.55[b]	.322
.27	.258	.34	.285	.34	.282	.60	.362	.60	.357	.60	.339
.28	.264	.36	.295	.36	.292	.65	.378	.65	.373	.65	.355
.29	.269	.38	.304	.38	.300	.70	.394	.70	.389	.70	.370
.30	.274	.40	.312	.40	.309	.75	.409	.75	.404	.75	.384
.32[b]	.284	.42	.321	.42	.318	.80	.423	.80	.418	.80	.398
.34	.293	.44	.329	.44	.326	.85	.437	.85	.432	.85	.412
.36	.303	.46	.337	.46	.334	.90	.451	.90	.446	.90	.425
.38	.312	.48	.345	.48	.342	.95	.464			.95	.438
.40	.320	.50	.353	.50	.349	1.00	.477			1.00	.451
.42	.329	.52	.361	.52	.357	1.05	.490			1.05	.463
.44	.337	.54	.368	.54	.364	1.10	.502			1.10	.475
.46	.346	.56	.376	.56	.371					1.15	.486
.48	.353	.58	.383	.58	.379					1.20	.498
.50	.361	.60	.390	.60	.386					1.25	.509
.52	.369	.62	.397	.62	.392					1.30	.520
		.64	.404	.64	.399					1.35	.530
		.66	.410	.66	.406					1.40	.541
		.68	.417	.68	.412					1.45	.551
$K_1 =$	3.081[c]	$K_1 =$	3.140	$K_1 =$	2.226	$K_1 =$	3.640	$K_1 =$	3.751	$K_1 =$	4.070
$K_2 =$	0.0102	$K_2 =$	0.0097	$K_2 =$	0.0083	$K_2 =$	0.0101	$K_2 =$	0.0126	$K_2 =$	0.0129
$u =$	1.847	$u =$	1.814	$u =$	1.898	$u =$	1.815	$u =$	1.841	$u =$	1.824

(continued)

[a] See Table 5.2 for flume dimensions and head-loss values.

[b] Change in discharge increment.

[c] The following equation approximates the values in the paired columns of Table R.1 by applying the K_1, K_2, and u factors at the bottom of each column pair: $Q = K_1\,(h_1 + K_2)^u$

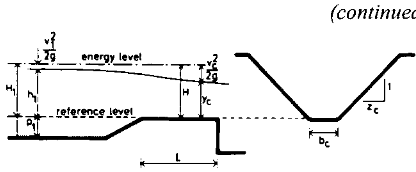


Table R.1. *(continued)*[a]

Weir I_m b_c = 1.6 m $0.48 \le L \le 0.71$ m		Weir J_m b_c = 1.8 m $0.40 \le L \le 0.60$ m		Weir K_m b_c = 1.5 m $0.48 \le L \le 0.72$ m		Weir L_m b_c = 1.75 m $0.58 \le L \le 0.87$ m		Weir M_m b_c = 2.00 m $0.65 \le L \le 0.97$ m		Weir N_m b_c = 2.25 m $0.75 \le L \le 1.10$ m	
Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)
0.02	0.039	0.02	0.036			0.05	0.065	0.05	0.061	0.10	0.088
.04	.061	.04	.056	0.04	0.062	.10	.101	.10	.094	.20	.136
.06	.079	.06	.072	.06	.080	.15	.129	.15	.121	.30	.174
.08	.094	.08	.087	.08	.096	.20	.154	.20	.144	.40	.207
.10	.108	.10	.100	.10	.110	.25	.176	.25	.165	.50	.236
.12	.121	.12	.113	.12	.122	.30	.197	.30	.184	.60	.263
.14	.134	.14	.124	.14	.134	.35	.215	.35	.202	.70	.288
.16	.145	.16	.135	.16	.146	.40	.233	.40	.219	.80	.311
.18	.156	.18	.146	.18	.156	.45	.249	.45	.234	.90	.333
.20	.166	.20	.155	.20	.166	.50	.265	.50	.249	1.00	.354
.22	.176	.22	.165	.22	.176	.55	.280	.55	.264	1.10	.374
.24	.186	.24	.174	.24	.185	.60	.294	.60	.277	1.20	.393
.26	.195	.26	.183	.26	.194	.65	.307	.65	.290	1.30	.411
.28	.204	.28	.192	.28	.203	.70	.321	.70	.303	1.40	.428
.30	.213	.30	.200	.30	.211	.75	.333	.75	.315	1.50	.445
.32	.221	.32	.208	.32	.219	.80	.345	.80	.327	1.60	.461
.34	.230	.34	.216	.34	.227	.85	.357	.85	.338	1.70	.477
.36	.238	.36	.223	.36	.234	.90	.369	.90	.350	1.80	.492
.38	.245	.38	.231	.38	.241	.95	.380	.95	.360	1.90	.507
.40	.253	.40	.238	.40	.249	1.00	.391	1.00	.371	2.00	.521
.42	.260	.42	.245	.42	.256	1.05	.401	1.10[b]	.391	2.10	.535
.44	.268	.44	.252	.44	.262	1.10	.412	1.20	.411	2.20	.549
.46	.275	.46	.259	.46	.269	1.15	.422	1.30	.429	2.30	.562
.48	.282	.48	.266	.48	.275	1.20	.432	1.40	.447	2.40	.575
.50	.289	.50	.272	.50	.282	1.25	.441	1.50	.464	2.50	.588
.55[b]	.305	.55[b]	.288	.55[b]	.297	1.30	.451	1.60	.481	2.60	.601
.60	.321	.60	.304	.60	.312	1.35	.460	1.70	.497	2.70	.613
.65	.336	.65	.318	.65	.326	1.40	.469	1.80	.512	2.80	.625
.70	.351	.70	.333	.70	.340	1.45	.478	1.90	.527	2.90	.637
.75	.365	.75	.346	.75	.352	1.50	.487	2.00	.542	3.00	.648
.80	.379	.80	.360	.80	.365	1.55	.495	2.10	.556	3.10	.660
.85	.392	.85	.372	.85	.377	1.60	.504	2.20	.570	3.20	.671
.90	.405	.90	.385	.90	.389	1.65	.512	2.30	.584	3.30	.682
.95	.417			.95	.400	1.70	.520	2.40	.597	3.40	.693
1.00	.430			1.00	.412	1.75	.528	2.50	.610	3.50	.703
1.05	.442			1.05	.422	1.80	.536	2.60	.623	3.60	.714
1.10	.453			1.10	.433	1.85	.544	2.70	.635	3.70	.724
1.15	.465			1.15	.443	1.90	.551	2.80	.647	3.80	.734
1.20	.476			1.20	.453	1.95	.559			3.90	.744
				1.25	.463	2.00	.566				
				1.30	.473	2.05	.574				
						2.10	.581				
K_1 =	4.217[c]	K_1 =	4.351	K_1 =	4.351	K_1 =	5.472	K_1 =	5.924	K_1 =	6.342
K_2 =	0.0088	K_2 =	0.0054	K_2 =	0.0054	K_2 =	0.0209	K_2 =	0.0194	K_2 =	0.0264
u =	1.751	u =	1.685	u =	1.685	u =	1.907	u =	1.881	u =	1.907

(continued)

[a] See Table 5.2 for flume dimensions and head-loss values.

[b] Change in discharge increment.

[c] The following equation approximates the values in the paired columns of Table R.1 by applying the K_1, K_2, and u factors at the bottom of each column pair: $Q = K_1\,(h_1 + K_2)^u$

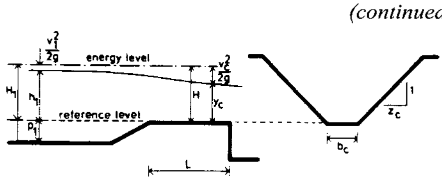


Table R.1. *(continued)* [a]

Weir P_m, $b_c = 2.50$ m, $0.80 \le L \le 1.20$ m		Weir Q_m, $b_c = 2.75$ m, $0.85 \le L \le 1.28$ m		Weir R_m, $b_c = 3.00$ m, $0.95 \le L \le 1.40$ m		Weir S_m, $b_c = 3.50$ m, $0.95 \le L \le 1.40$ m		Weir T_m, $b_c = 4.00$ m, $0.85 \le L \le 1.20$ m		Weir U_m, $b_c = 4.50$ m, $0.68 \le L \le 1.00$ m	
Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)	Q (m^3/s)	h_1 (m)
0.10	0.082	0.10	0.078	0.10	0.075	0.10	0.068	0.10	0.062	0.10	0.057
.20	.127	.20	.121	.20	.115	.20	.105	.20	.097	.20	.089
.30	.164	.30	.156	.30	.148	.30	.136	.30	.125	.30	.115
.40	.195	.40	.186	.40	.177	.40	.162	.40	.150	.40	.139
.50	.223	.50	.213	.50	.203	.50	.187	.50	.173	.50	.160
.60	.249	.60	.237	.60	.227	.60	.209	.60	.193	.60	.179
.70	.273	.70	.260	.70	.249	.70	.230	.70	.213	.70	.198
.80	.295	.80	.282	.80	.270	.80	.249	.80	.231	.80	.215
.90	.316	.90	.302	.90	.290	.90	.268	.90	.249	.90	.232
1.00	.336	1.00	.322	1.00	.309	1.00	.286	1.00	.266	1.00	.248
1.10	.355	1.10	.340	1.10	.327	1.10	.302	1.10	.282	1.10	.263
1.20	.374	1.20	.358	1.20	.344	1.20	.319	1.20	.297	1.20	.277
1.30	.391	1.30	.375	1.30	.361	1.30	.334	1.30	.312	1.30	.292
1.40	.408	1.40	.391	1.40	.377	1.40	.350	1.40	.326	1.40	.305
1.50	.424	1.50	.407	1.50	.392	1.50	.364	1.50	.340	1.50	.319
1.60	.440	1.60	.422	1.60	.407	1.60	.378	1.60	.354	1.60	.332
1.70	.455	1.70	.437	1.70	.421	1.70	.392	1.70	.367	1.70	.344
1.80	.470	1.80	.451	1.80	.436	1.80	.406	1.80	.380	1.80	.356
1.90	.485	1.90	.466	1.90	.449	1.90	.419	1.90	.392	1.90	.368
2.00	.499	2.00	.479	2.00	.463	2.00	.431	2.00	.405	2.00	.380
2.10	.512	2.10	.493	2.10	.476	2.20[b]	.456	2.10	.417	2.10	.391
2.20	.526	2.20	.506	2.20	.489	2.40	.480	2.20	.428	2.20	.403
2.30	.539	2.30	.518	2.30	.501	2.60	.502	2.30	.440	2.30	.414
2.40	.552	2.40	.531	2.40	.513	2.80	.524	2.40	.451	2.40	.424
2.50	.564	2.50	.543	2.50	.525	3.00	.545	2.50	.462	2.50	.435
2.60	.576	2.60	.555	2.60	.537	3.20	.566	2.60	.473	2.60	.445
2.70	.588	2.70	.566	2.70	.548	3.40	.586	2.70	.483	2.70	.456
2.80	.600	2.80	.578	2.80	.560	3.60	.605	2.80	.494	2.80	.466
2.90	.611	2.90	.589	2.90	.571	3.80	.624	2.90	.504	2.90	.476
3.00	.623	3.00	.600	3.00	.582	4.00	.642	3.00	.514	3.00	.485
3.10	.634	3.20[b]	.622	3.20[b]	.603	4.20	.660	3.20[b]	.534	3.20[b]	.504
3.20	.645	3.40	.643	3.40	.623	4.40	.677	3.40	.553	3.40	.523
3.30	.656	3.60	.663	3.60	.643	4.60	.694	3.60	.572	3.60	.541
3.40	.666	3.80	.683	3.80	.663	4.80	.710	3.80	.590	3.80	.558
3.50	.677	4.00	.702	4.00	.682	5.00	.727	4.00	.608	4.00	.575
3.60	.687	4.20	.720	4.20	.700	5.50[b]	.765	4.20	.625	4.20	.592
3.70	.697	4.40	.739	4.40	.718	6.00	.803	4.40	.642	4.40	.608
3.80	.707	4.60	.756	4.60	.735	6.50	.839	4.60	.658	4.60	.624
3.90	.717	4.80	.774	4.80	.753	7.00	.873	4.80	.674	4.80	.640
4.00	.726	5.00	.791	5.00	.769	7.50	.906	5.00	.690	5.00	.655
4.20[b]	.745	5.20	.807	5.50[b]	.810	8.00	.938	5.50[b]	.728		
4.40	.764	5.40	.823	6.00	.848			6.00	.764		
4.60	.782	5.60	.839	6.50	.884			6.50	.799		
				7.00	.920						
$K_1 =$	6.814[c]	$K_1 =$	7.288	$K_1 =$	7.692	$K_1 =$	8.529	$K_1 =$	9.213	$K_1 =$	9.853
$K_2 =$	0.0255	$K_2 =$	0.0240	$K_2 =$	0.0239	$K_2 =$	0.0197	$K_2 =$	0.0131	$K_2 =$	0.0089
$u =$	1.886	$u =$	1.870	$u =$	1.857	$u =$	1.812	$u =$	1.740	$u =$	1.681

[a] See Table 5.2 for flume dimensions and head-loss values.

[b] Change in discharge increment.

[c] The following equation approximates the values in the paired columns of Table R.1 by applying the K_1, K_2, and u factors at the bottom of each column pair: $Q = K_1\,(h_1 + K_2)^u$

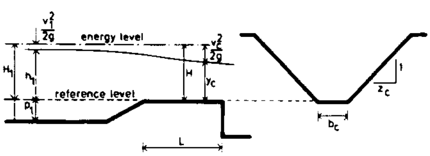


Table R.2. Rating Tables for Weirs in Trapezoidal Lined Canals in English Units (X_C = 3%).[a]

Weir A_e b_c = 2.0 ft 0.90≤L≤1.30 ft		Weir B_e b_c = 2.5 ft 1.2≤L≤1.8 ft		Weir C_e b_c = 3.0 ft 1.3≤L≤1.9 ft		Weir D_e b_c = 3.5 ft 1.6≤L≤2.1 ft		Weir E_e b_c = 4.0 ft 1.7≤L≤2.1 ft		Weir F_e b_c = 4.5 ft 1.5≤L≤2.2 ft	
Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)
						0.5	0.130				
0.2	0.102	0.4	0.139	0.4	0.125	1.0	.202	1.0	0.188	1.0	0.174
.3	.131	.6	.179	.6	.161	1.5	.260	1.5	.242	1.5	.225
.4	.157	.8	.214	.8	.193	2.0	.311	2.0	.290	2.0	.270
.5	.180	1.0	.245	1.0	.221	2.5	.357	2.5	.333	2.5	.311
.6	.201	1.2	.274	1.2	.248	3.0	.399	3.0	.373	3.0	.349
.7	.221	1.4	.301	1.4	.273	3.5	.438	3.5	.410	3.5	.384
.8	.240	1.6	.327	1.6	.297	4.0	.475	4.0	.445	4.0	.417
.9	.257	1.8	.351	1.8	.319	4.5	.509	4.5	.478	4.5	.449
1.0	.274	2.0	.374	2.0	.340	5.0	.542	5.0	.510	5.0	.479
1.1	.290	2.2	.396	2.2	.361	5.5	.574	5.5	.540	5.5	.508
1.2	.306	2.4	.417	2.4	.380	6.0	.604	6.0	.569	6.0	.535
1.3	.321	2.6	.437	2.6	.399	6.5	.633	6.5	.597	6.5	.562
1.4	.335	2.8	.456	2.8	.418	7.0	.660	7.0	.624	7.0	.588
1.5	.349	3.0	.475	3.0	.435	7.5	.687	7.5	.650	7.5	.613
1.6	.362	3.2	.494	3.2	.453	8.0	.714	8.0	.675	8.0	.637
1.7	.375	3.4	.511	3.4	.469	8.5	.739	8.5	.699	8.5	.661
1.8	.388	3.6	.528	3.6	.486	9.0	.763	9.0	.723	9.0	.684
1.9	.400	3.8	.545	3.8	.502	9.5	.787	9.5	.746	9.5	.706
2.0	.412	4.0	.562	4.0	.517	10.0	.811	10.0	.769	10.0	.728
2.2[b]	.436	4.2	.578	4.2	.532	10.5	.834	11.0[b]	.813	11.0[b]	.770
2.4	.458	4.4	.593	4.4	.547	11.0	.856	12.0	.855	12.0	.811
2.6	.479	4.6	.608	4.6	.562	11.5	.878	13.0	.895	13.0	.850
2.8	.500	4.8	.623	4.8	.576	12.0	.899	14.0	.934	14.0	.887
3.0	.520	5.0	.638	5.0	.590	12.5	.920	15.0	.971	15.0	.924
3.2	.539	5.5[b]	.673	5.5[b]	.623	13.0	.940	16.0	1.007	16.0	.959
3.4	.558	6.0	.707	6.0	.656	13.5	.960	17.0	1.042	17.0	.993
3.6	.576	6.5	.739	6.5	.687	14.0	.980	18.0	1.076	18.0	1.026
3.8	.593	7.0	.770	7.0	.716	14.5	.999	19.0	1.109	19.0	1.058
4.0	.610	7.5	.800	7.5	.745	15.0	1.018	20.0	1.141	20.0	1.090
4.2	.627	8.0	.829	8.0	.773	16.0[b]	1.055	21.0	1.173	21.0	1.120
4.4	.643	8.5	.858	8.5	.800	17.0	1.091	22.0	1.204	22.0	1.150
4.6	.659	9.0	.884	9.0	.826	18.0	1.126	23.0	1.233	23.0	1.180
4.8	.675	9.5	.911	9.5	.852	19.0	1.160	24.0	1.263	24.0	1.208
5.0	.690	10.0	.936	10.0	.876	20.0	1.193	25.0	1.291	25.0	1.236
5.5[b]	.727	11.0[b]	.986	11.0[b]	.924	21.0	1.225	26.0	1.319	26.0	1.264
6.0	.762	12.0	1.033	12.0	.970	22.0	1.256	27.0	1.347	27.0	1.291
6.5	.795	13.0	1.077	13.0	1.013	23.0	1.286	28.0	1.374	28.0	1.317
7.0	.827	14.0	1.121	14.0	1.055	24.0	1.316	29.0	1.400	29.0	1.343
7.5	.858	15.0	1.162	15.0	1.096	25.0	1.345	30.0	1.426	30.0	1.369
8.0	.888	16.0	1.202	16.0	1.135	26.0	1.374	32.0[b]	1.477	31.0	1.394
				17.0	1.172	27.0	1.402	34.0	1.526	32.0	1.418
				18.0	1.209			36.0	1.574	33.0	1.443
				19.0	1.245			38.0	1.620		
								40.0	1.665		
K_1 =	9.309[c]	K_1 =	10.40	K_1 =	11.88	K_1 =	13.62	K_1 =	14.32	K_1 =	16.04
K_2 =	0.029	K_2 =	0.045	K_2 =	0.038	K_2 =	0.039	K_2 =	0.057	K_2 =	0.043
u =	1.879	u =	1.905	u =	1.844	u =	1.843	u =	1.872	u =	1.801

(continued)

[a] See Table 5.3 for flume dimensions and head-loss values.

[b] Change in discharge increment.

[c] The following equation approximates the values in the paired columns of Table R.2 by applying the K_1, K_2, and u factors at the bottom of each column pair: $Q = K_1\,(h_1 + K_2)^u$

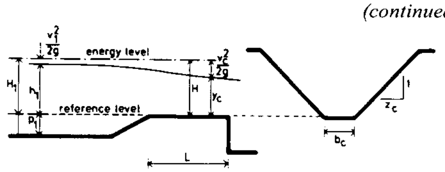


Table R.2. *(continued)*[a]

Weir G_e $b_c = 5.0$ ft $1.5 \leq L \leq 2.2$ ft		Weir H_e $b_c = 5.5$ ft $1.1 \leq L \leq 1.6$ ft		Weir I_e $b_c = 3.0$ ft $1.2 \leq L \leq 1.8$ ft		Weir J_e $b_c = 4.0$ ft $1.8 \leq L \leq 2.2$ ft		Weir K_e $b_c = 5.0$ ft $2.0 \leq L \leq 2.9$ ft		Weir L_e $b_c = 6.0$ ft $2.0 \leq L \leq 3.0$ ft	
Q *(ft³/s)*	h_1 *(ft)*	Q *(ft³/s)*	h_1 *(ft)*	Q *(ft³/s)*	h_1 *(ft)*	Q *(ft³/s)*	h_1 *(ft)*	Q *(ft³/s)*	h_1 *(ft)*	Q *(ft³/s)*	h_1 *(ft)*
1.0	0.163	1.0	0.154	0.4	0.124	1.0	0.185	2.0	0.254	2.0	0.228
1.5	.211	1.5	.199	.6	.160	1.5	.239	3.0	.327	3.0	.295
2.0	.253	2.0	.239	.8	.191	2.0	.286	4.0	.391	4.0	.353
2.5	.291	2.5	.275	1.0	.219	2.5	.328	5.0	.448	5.0	.405
3.0	.327	3.0	.309	1.2	.245	3.0	.367	6.0	.500	6.0	.454
3.5	.361	3.5	.341	1.4	.269	3.5	.403	7.0	.549	7.0	.499
4.0	.393	4.0	.371	1.6	.292	4.0	.437	8.0	.595	8.0	.542
4.5	.423	4.5	.400	1.8	.314	4.5	.469	9.0	.638	9.0	.582
5.0	.451	5.0	.427	2.0	.334	5.0	.500	10.0	.679	10.0	.620
5.5	.479	5.5	.454	2.2	.354	5.5	.529	11.0	.718	11.0	.657
6.0	.505	6.0	.479	2.4	.373	6.0	.557	12.0	.756	12.0	.693
6.5	.531	6.5	.504	2.6	.391	6.5	.584	13.0	.792	13.0	.727
7.0	.556	7.0	.527	2.8	.409	7.0	.610	14.0	.827	14.0	.760
7.5	.580	7.5	.551	3.0	.426	7.5	.635	15.0	.860	15.0	.791
8.0	.603	8.0	.573	3.2	.442	8.0	.660	16.0	.893	16.0	.822
8.5	.626	8.5	.595	3.4	.458	8.5	.683	17.0	.925	17.0	.852
9.0	.648	9.0	.616	3.6	.474	9.0	.706	18.0	.955	18.0	.881
9.5	.670	9.5	.637	3.8	.489	9.5	.728	19.0	.985	19.0	.910
10.0	.691	10.0	.657	4.0	.504	10.0	.750	20.0	1.014	20.0	.937
10.5	.711	10.5	.677	4.2	.518	11.0[b]	.792	22.0[b]	1.070	22.0[b]	.991
11.0	.732	11.0	.697	4.4	.533	12.0	.832	24.0	1.124	24.0	1.042
11.5	.751	11.5	.716	4.6	.546	13.0	.871	26.0	1.175	26.0	1.091
12.0	.771	12.0	.735	4.8	.560	14.0	.908	28.0	1.224	28.0	1.139
12.5	.790	12.5	.753	5.0	.573	15.0	.943	30.0	1.272	30.0	1.184
13.0	.808	13.0	.771	5.5[b]	.605	16.0	.978	32.0	1.318	32.0	1.229
13.5	.827	13.5	.789	6.0	.636	17.0	1.011	34.0	1.362	34.0	1.271
14.0	.845	14.0	.806	6.5	.665	18.0	1.044	36.0	1.406	36.0	1.313
14.5	.863	14.5	.823	7.0	.693	19.0	1.075	38.0	1.448	38.0	1.354
15.0	.880	15.0	.840	7.5	.721	20.0	1.106	40.0	1.488	40.0	1.393
16.0[b]	.914	15.5	.857	8.0	.747	21.0	1.136	42.0	1.528	42.0	1.431
17.0	.947	16.0	.873	8.5	.772	22.0	1.165	44.0	1.567	44.0	1.469
18.0	.980	16.5	.890	9.0	.797	23.0	1.193	46.0	1.604	46.0	1.505
19.0	1.011	17.0	.906	9.5	.821	24.0	1.221	48.0	1.641	48.0	1.541
20.0	1.042	17.5	.921	10.0	.844	25.0	1.248	50.0	1.677	50.0	1.576
21.0	1.071	18.0	.937	11.0[b]	.889	26.0	1.275	52.0	1.713	55.0[b]	1.660
22.0	1.101	18.5	.952	12.0	.932	27.0	1.301	54.0	1.747	60.0	1.741
23.0	1.129	19.0	.967	13.0	.973	28.0	1.326	56.0	1.781	65.0	1.818
24.0	1.157	19.5	.982	14.0	1.012	29.0	1.351	58.0	1.815	70.0	1.893
25.0	1.184	20.0	.997	15.0	1.050	30.0	1.376	60.0	1.847	75.0	1.964
26.0	1.211	20.5	1.012	16.0	1.087	31.0	1.400	62.0	1.879		
27.0	1.238	21.0	1.026	17.0	1.122	32.0	1.424	64.0	1.911		
		21.5	1.040	18.0	1.156	33.0	1.447				
		22.0	1.055	19.0	1.189	34.0	1.470				
						35.0	1.493				
$K_1 = 17.74$[c] $K_2 = 0.030$ $u = 1.737$		$K_1 = 19.38$ $K_2 = 0.019$ $u = 1.683$		$K_1 = 12.68$ $K_2 = 0.041$ $u = 1.898$		$K_1 = 14.91$ $K_2 = 0.063$ $u = 1.912$		$K_1 = 16.96$ $K_2 = 0.078$ $u = 1.919$		$K_1 = 19.89$ $K_2 = 0.067$ $u = 1.861$	

(continued)

[a] See Table 5.3 for flume dimensions and head-loss values.

[b] Change in discharge increment.

[c] The following equation approximates the values in the paired columns of Table R.2 by applying the K_1, K_2, and u factors at the bottom of each column pair: $Q = K_1\,(h_1 + K_2)^u$

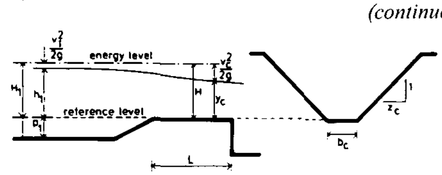


Table R.2. *(continued)*[a]

Weir M_e $b_c = 7.0$ ft $1.7 \le L \le 2.5$ ft		Weir N_e $b_c = 8.0$ ft $1.4 \le L \le 2.0$ ft		Weir P_e $b_c = 5.0$ ft $1.6 \le L \le 2.4$ ft		Weir Q_e $b_c = 6.0$ ft $2.2 \le L \le 3.3$ ft		Weir R_e $b_c = 7.0$ ft $2.6 \le L \le 3.9$ ft		Weir S_e $b_c = 8.0$ ft $2.6 \le L \le 3.9$ ft	
Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)
				1.	0.162			2.	0.208		
2.	0.207	2.	0.190	2.	.251	2.	0.226	4.	.321	4.	0.296
3.	.268	3.	.247	3.	.322	3.	.292	6.	.413	6.	.382
4.	.321	4.	.296	4.	.384	4.	.349	8.	.492	8.	.457
5.	.370	5.	.342	5.	.440	5.	.401	10.	.564	10.	.524
6.	.415	6.	.383	6.	.491	6.	.448	12.	.629	12.	.587
7.	.457	7.	.423	7.	.538	7.	.492	14.	.690	14.	.644
8.	.497	8.	.460	8.	.583	8.	.534	16.	.747	16.	.699
9.	.534	9.	.495	9.	.624	9.	.573	18.	.801	18.	.750
10.	.570	10.	.529	10.	.664	10.	.610	20.	.852	20.	.799
11.	.605	11.	.562	11.	.702	11.	.646	22.	.901	22.	.846
12.	.638	12.	.593	12.	.738	12.	.680	24.	.948	24.	.891
13.	.670	13.	.623	13.	.773	13.	.713	26.	.992	26.	.934
14.	.701	14.	.652	14.	.806	14.	.745	28.	1.036	28.	.975
15.	.731	15.	.681	15.	.838	15.	.776	30.	1.077	30.	1.016
16.	.760	16.	.708	16.	.869	16.	.805	32.	1.118	32.	1.055
17.	.789	17.	.735	17.	.899	17.	.834	34.	1.157	34.	1.092
18.	.817	18.	.761	18.	.929	18.	.862	36.	1.195	36.	1.129
19.	.843	19.	.787	19.	.957	19.	.890	38.	1.232	38.	1.165
20.	.870	20.	.812	20.	.985	20.	.916	40.	1.268	40.	1.200
22.[b]	.921	21.	.837	21.	1.012	22.[b]	.968	42.	1.303	42.	1.234
24.	.970	22.	.860	22.	1.039	24.	1.017	44.	1.338	44.	1.267
26.	1.017	23.	.884	23.	1.064	26.	1.064	46.	1.371	46.	1.300
28.	1.062	24.	.907	24.	1.090	28.	1.109	48.	1.404	48.	1.332
30.	1.106	25.	.930	25.	1.114	30.	1.153	50.	1.436	50.	1.363
32.	1.148	26.	.952	26.	1.138	32.	1.195	55.[b]	1.513	55.[b]	1.438
34.	1.189	27.	.974	27.	1.162	34.	1.236	60.	1.587	60.	1.510
36.	1.229	28.	.995	28.	1.185	36.	1.276	65.	1.658	65.	1.580
38.	1.268	29.	1.016	29.	1.208	38.	1.314	70.	1.726	70.	1.646
40.	1.306	30.	1.037	30.	1.231	40.	1.352	75.	1.791	75.	1.710
42.	1.343	32.[b]	1.078	32.[b]	1.274	42.	1.388	80.	1.854	80.	1.771
44.	1.380	34.	1.117	34.	1.316	44.	1.424	85.	1.915	85.	1.831
46.	1.415	36.	1.156	36.	1.357	46.	1.459	90.	1.975	90.	1.889
48.	1.450	38.	1.193	38.	1.397	48.	1.493	95.	2.03	95.	1.945
50.	1.484	40.	1.230	40.	1.436	50.	1.526	100.	2.09	100.	2.00
52.	1.517	42.	1.265	42.	1.473	55.[b]	1.606	105.	2.14	110.[b]	2.11
54.	1.549	44.	1.300	44.	1.510	60.	1.683			120.	2.21
56.	1.581	46.	1.334	46.	1.546	65.	1.755			130.	2.30
58.	1.613			48.	1.581	70.	1.826			140.	2.39
60.	1.644					75.	1.893			150.	2.48
62.	1.674					80.	1.958			160.	2.57
$K_1 = 23.53$[c]		$K_1 = 26.79$		$K_1 = 18.78$		$K_1 = 20.38$		$K_1 = 23.59$		$K_1 = 24.44$	
$K_2 = 0.045$		$K_2 = 0.034$		$K_2 = 0.053$		$K_2 = 0.076$		$K_2 = 0.064$		$K_2 = 0.097$	
$u = 1.772$		$u = 1.724$		$u = 1.891$		$u = 1.914$		$u = 1.873$		$u = 1.907$	

(continued)

[a] See Table 5.3 for flume dimensions and head-loss values.

[b] Change in discharge increment.

[c] The following equation approximates the values in the paired columns of Table R.2 by applying the K_1, K_2, and u factors at the bottom of each column pair: $Q = K_1 (h_1 + K_2)^u$

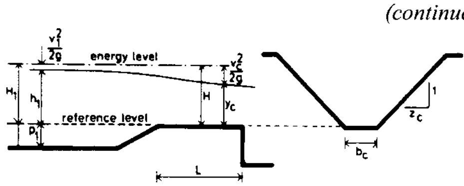


Table R.2. *(continued)*[a]

Weir T_e		Weir U_e		Weir V_e		Weir W_e		Weir X_e	
$b_c = 9.0$ ft		$b_c = 10.0$ ft		$b_c = 12.0$ ft		$b_c = 14.0$ ft		$b_c = 16.0$ ft	
$2.8 \le L \le 4.2$ ft		$3.0 \le L \le 4.4$ ft		$3.1 \le L \le 4.6$ ft		$2.6 \le L \le 3.8$ ft		$2.0 \le L \le 2.9$ ft	
Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)
5.	0.317	5.	0.301	5.	0.270	5.	0.243	5.	0.224
10.	.493	10.	.465	10.	.418	10.	.378	10.	.349
15.	.632	15.	.597	15.	.539	15.	.489	15.	.452
20.	.754	20.	.713	20.	.645	20.	.587	20.	.542
25.	.862	25.	.817	25.	.741	25.	.676	25.	.625
30.	.962	30.	.912	30.	.829	30.	.757	30.	.701
35.	1.054	35.	1.000	35.	.911	35.	.834	35.	.773
40.	1.140	40.	1.083	40.	.988	40.	.906	40.	.841
45.	1.221	45.	1.161	45.	1.061	45.	.974	45.	.905
50.	1.298	50.	1.235	50.	1.131	50.	1.039	50.	.966
55.	1.372	55.	1.306	55.	1.198	55.	1.102	55.	1.025
60.	1.442	60.	1.373	60.	1.262	60.	1.162	60.	1.082
65.	1.509	65.	1.438	65.	1.323	65.	1.220	65.	1.137
70.	1.574	70.	1.501	70.	1.382	70.	1.276	70.	1.190
75.	1.636	75.	1.561	75.	1.440	75.	1.330	75.	1.241
80.	1.697	80.	1.620	80.	1.496	80.	1.383	80.	1.291
85.	1.755	85.	1.677	85.	1.550	85.	1.434	85.	1.340
90.	1.812	90.	1.732	90.	1.602	90.	1.484	90.	1.388
95.	1.867	95.	1.785	95.	1.653	95.	1.532	95.	1.434
100.	1.921	100.	1.838	100.	1.703	100.	1.580	100.	1.479
105.	1.973	105.	1.889	110.[b]	1.800	105.	1.626	105.	1.523
110.	2.02	110.	1.938	120.	1.892	110.	1.671	110.	1.567
115.	2.07	115.	1.987	130.	1.980	115.	1.716	115.	1.609
120.	2.12	120.	2.03	140.	2.07	120.	1.759	120.	1.651
125.	2.17	125.	2.08	150.	2.15	125.	1.802	125.	1.691
130.	2.22	130.	2.13	160.	2.23	130.	1.844	130.	1.731
135.	2.26	135.	2.17	170.	2.30	135.	1.885	135.	1.771
140.	2.31	140.	2.22	180.	2.38	140.	1.925	140.	1.810
145.	2.35	145.	2.26	190.	2.45	145.	1.965	145.	1.848
150.	2.40	150.	2.30	200.	2.52	150.	2.00	150.	1.885
155.	2.44	160.[b]	2.38	210.	2.59	160.[b]	2.08	155.	1.922
160.	2.48	170.	2.46	220.	2.66	170.	2.15	160.	1.958
165.	2.52	180.	2.54	230.	2.72	180.	2.23		
170.	2.56	190.	2.61	240.	2.79	190.	2.30		
175.	2.60	200.	2.69	250.	2.85	200.	2.36		
180.	2.64	210.	2.76	260.	2.91	210.	2.43		
185.	2.68	220.	2.83	270.	2.97	220.	2.49		
190.	2.72	230.	2.90	280.	3.03				
195.	2.75								
200.	2.79								
$K_1 = 27.06$[c]		$K_1 = 29.86$		$K_1 = 35.85$		$K_1 = 43.56$		$K_1 = 50.96$	
$K_2 = 0.091$		$K_2 = 0.086$		$K_2 = 0.071$		$K_2 = 0.045$		$K_2 = 0.024$	
$u = 1.879$		$u = 1.86$		$u = 1.805$		$u = 1.726$		$u = 1.66$	

[a] See Table 5.3 for flume dimensions and head-loss values.

[b] Change in discharge increment.

[c] The following equation approximates the values in the paired columns of Table R.2 by applying the K_1, K_2, and u factors at the bottom of each column pair: $Q = K_1 (h_1 + K_2)^u$

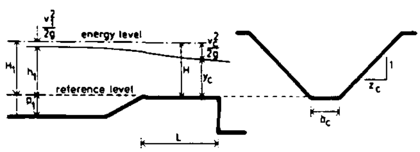


Table R.3. Rating Tables for Rectangular Weirs in Metric Units with Discharge per Meter Width ($X_C = 3\%$).[a]

	$0.10 \le b_c \le 0.20$ m $L = 0.2$ m		
h_1	q (m^3/s per meter width)		
(m)	$p_1 = 0.05$ m	$p_1 = 0.1$ m	$p_1 = \infty$
0.014	0.0026	0.0026	0.0026
.016	.0032	.0032	.0032
.018	.0039	.0039	.0039
.020	.0046	.0045	.0045
.022	.0054	.0053	.0053
.024	.0062	.0061	.0060
.026	.0070	.0069	.0068
.028	.0079	.0077	.0077
.030	.0088	.0086	.0085
.032	.0097	.0095	.0094
.034	.0107	.0104	.0103
.036	.0117	.0114	.0112
.038	.0128	.0124	.0122
.040	.0138	.0134	.0132
.042	.0150	.0145	.0142
.044	.0161	.0156	.0153
.046	.0173	.0167	.0164
.048	.0185	.0178	.0174
.050	.0197	.0190	.0186
.055[b]	.0230	.0222	.0215
.060	.0264	.0253	.0245
.065	.0300	.0286	.0277
.070	.0337	.0322	.0309
.075	.0377	.0359	.0343
.080	.0418	.0397	.0379
.085	.0461	.0436	.0415
.090	.0506	.0477	.0452
.095	.0552	.0520	.0491
.100	.0599	.0564	.0531
.105	.0649	.0609	.0571
.110	.0699	.0656	.0613
.115	.0751	.0704	.0655
.120	.0805	.0753	.0698
.125	.0860	.0803	.0743
.130	.0917	.0855	.0788
$\Delta H =$ [c]	0.011 m or $0.1H_1$	0.019 m or $0.1H_1$	$0.4H_1$

	$0.20 \le b_c \le 0.30$ m $L = 0.35$ m		
h_1	q (m^3/s per meter width)		
(m)	$p_1 = 0.1$ m	$p_1 = 0.2$ m	$p_1 = \infty$
0.025	0.0064	0.0063	0.0063
.030	.0085	.0084	.0084
.035	.0108	.0107	.0107
.040	.0133	.0131	.0131
.045	.0160	.0158	.0157
.050	.0189	.0186	.0184
.055	.0220	.0215	.0213
.060	.0252	.0246	.0244
.065	.0285	.0278	.0275
.070	.0321	.0312	.0308
.075	.0357	.0347	.0342
.080	.0396	.0384	.0377
.085	.0435	.0421	.0414
.090	.0476	.0460	.0451
.095	.0519	.0500	.0489
.100	.0563	.0542	.0529
.110[b]	.0654	.0628	.0611
.120	.0751	.0719	.0697
.130	.0854	.0815	.0786
.140	.0961	.0915	.0880
.150	.1073	.1019	.0976
.160	.1190	.1127	.1076
.170	.1311	.1240	.1179
.180	.1438	.1356	.1285
.190	.1566	.1477	.1393
.200	.1700	.1599	.1504
.210	.1838	.1726	.1618
.220	.1981	.1858	.1735
.230	.213	.1993	.1855
$\Delta H =$ [c]	0.024 m or $0.1H_1$	0.037 m or $0.1H_1$	$0.4H_1$

(continued)

[a] $L_a \ge H_{1max}$
$L_b = 3\,p_1$
$L_a + L_b \ge 2$ to 3 times H_{1max}

[b] Change in head increment

[c] ΔH values are for abrupt expansion into rectangular channel of same width as crest. Use the larger of $0.1H_1$ or the value shown. For discharge into a stagnant pool, head loss is $0.4H_1$.

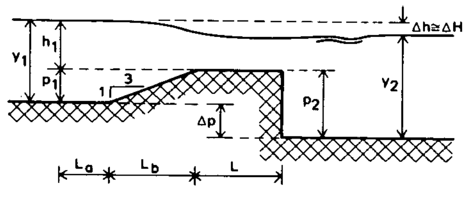


Table R.3. *(continued)*[a]

h_1 (m)	$0.30 \le b_c \le 0.50$ m, $L = 0.5$ m — q (m^3/s per meter width) $p_1 = 0.1$ m	$p_1 = 0.2$ m	$p_1 = \infty$
0.035	0.0107	0.0106	0.0106
.040	.0133	.0131	.0130
.045	.0160	.0157	.0156
.050	.0188	.0185	.0183
.055	.0219	.0214	.0212
.060	.0251	.0245	.0242
.065	.0285	.0278	.0274
.070	.0320	.0311	.0307
.075	.0357	.0347	.0341
.080	.0395	.0383	.0376
.085	.0434	.0421	.0412
.090	.0476	.0460	.0450
.095	.0518	.0500	.0488
.100	.0560	.0541	.0527
.110[b]	.0651	.0626	.0608
.120	.0748	.0716	.0693
.130	.0849	.0811	.0782
.140	.0955	.0910	.0875
.150	.1066	.1014	.0970
.160	.1182	.1121	.1069
.170	.1302	.1233	.1172
.180	.1427	.1349	.1277
.190	.1556	.1468	.1385
.200	.1690	.1592	.1496
.220[b]	.1971	.1850	.1728
.240	.227	.212	.1970
.260	.258	.241	.222
.280	.291	.271	.248
.300	.326	.303	.276
.320	.361	.336	.304
$\Delta H =$ [c]	0.027 m or $0.1H_1$	0.042 m or $0.1H_1$	$0.4H_1$

h_1 (m)	$0.5 \le b_c \le 1.0$ m, $L = 0.75$ m — q (m^3/s per meter width) $p_1 = 0.1$ m	$p_1 = 0.2$ m	$p_1 = 0.3$ m	$p_1 = \infty$
0.050	0.0186	0.0183	0.0182	0.0181
.055	.0216	.0212	.0210	.0209
.060	.0248	.0242	.0240	.0239
.065	.0281	.0274	.0272	.0270
.070	.0315	.0308	.0305	.0302
.075	.0352	.0342	.0339	.0336
.080	.0389	.0378	.0374	.0371
.085	.0429	.0416	.0411	.0407
.090	.0469	.0454	.0449	.0444
.095	.0511	.0494	.0488	.0482
.100	.0554	.0535	.0528	.0521
.110[b]	.0645	.0621	.0612	.0602
.120	.0741	.0711	.0700	.0687
.130	.0842	.0806	.0792	.0776
.140	.0948	.0905	.0889	.0869
.150	.1059	.1008	.0989	.0964
.160	.1175	.1116	.1094	.1064
.170	.1295	.1227	.1202	.1166
.180	.1420	.1343	.1313	.1271
.190	.1549	.1463	.1429	.1380
.200	.1683	.1586	.1548	.1491
.220[b]	.1963	.1845	.1797	.1722
.240	.226	.212	.206	.1964
.260	.257	.241	.234	.222
.280	.290	.271	.262	.248
.300	.325	.302	.293	.275
.320	.361	.335	.324	.303
.340	.398	.369	.357	.332
.360	.437	.405	.390	.362
.380		.442	.426	.393
.400		.480	.462	.425
.420		.519	.499	.457
.440		.560	.538	.490
.460		.601	.577	.524
.480		.644	.618	.559
.500		.688	.660	.594
$\Delta H =$ [c]	0.028 m or $0.1H_1$	0.049 m or $0.1H_1$	0.063 m or $0.1H_1$	$0.4H_1$

(continued)

[a] $L_a \ge H_{1max}$
$L_b = 3\,p_1$
$L_a + L_b \ge 2$ to 3 times H_{1max}

[b] Change in head increment

[c] ΔH values are for abrupt expansion into rectangular channel of same width as crest. Use the larger of $0.1H_1$ or the value shown. For discharge into a stagnant pool, head loss is $0.4H_1$.

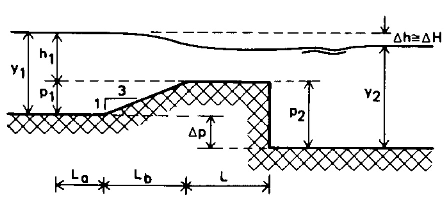


Table R.3. *(continued)*[a]

h_1 (m)	$1.0 \le b_c \le 2.0$ m, $L = 1.0$ m — q (m³/s per meter width) $p_1 = 0.2$ m	$p_1 = 0.3$ m	$p_1 = 0.4$ m	$p_1 = \infty$
0.07	0.0303	0.0301	0.0300	0.0298
.08	.0374	.0370	.0368	.0366
.09	.0450	.0445	.0442	.0439
.10	.0530	.0524	.0521	.0516
.11	.0616	.0608	.0603	.0597
.12	.0706	.0696	.0691	.0683
.13	.0801	.0788	.0782	.0771
.14	.0900	.0885	.0877	.0864
.15	.1004	.0985	.0976	.0960
.16	.1111	.1089	.1078	.1059
.17	.1223	.1198	.1185	.1161
.18	.1339	.1310	.1295	.1267
.19	.1459	.1425	.1408	.1375
.20	.1582	.1545	.1525	.1487
.22[b]	.1841	.1794	.1769	.1718
.24	.212	.206	.203	.1961
.26	.240	.233	.230	.221
.28	.271	.262	.258	.248
.30	.302	.293	.287	.275
.32	.335	.324	.318	.303
.34	.370	.357	.350	.332
.36	.405	.391	.383	.362
.38	.442	.426	.417	.393
.40	.480	.462	.452	.425
.42	.520	.500	.488	.457
.44	.560	.538	.526	.490
.46	.602	.578	.564	.524
.48	.645	.619	.603	.559
.50	.689	.661	.644	.595
.55[b]	.805	.770	.750	.687
.60	.927	.886	.861	.783
.65	1.056	1.008	.979	.884
$\Delta H =$ [c]	0.052 m or $0.1H_1$	0.068 m or $0.1H_1$	0.083 m or $0.1H_1$	$0.4H_1$

h_1 (m)	$b_c \ge 2.0$ m, $L = 1.0$ m — q (m³/s per meter width) $p_1 = 0.2$ m	$p_1 = 0.4$ m	$p_1 = 0.6$ m	$p_1 = \infty$
0.10	0.0532	0.0522	0.0518	0.0516
.12	.0708	.0692	.0687	.0683
.14	.0902	.0879	.0871	.0864
.16	.1114	.1081	.1070	.1060
.18	.1342	.1298	.1283	.1268
.20	.1586	.1529	.1510	.1489
.22	.1846	.1774	.1749	.1721
.24	.212	.203	.200	.1964
.26	.241	.230	.226	.222
.28	.271	.259	.254	.248
.30	.303	.288	.283	.275
.32	.336	.319	.312	.304
.34	.371	.351	.343	.333
.36	.406	.384	.375	.363
.38	.443	.418	.408	.394
.40	.482	.453	.442	.426
.42	.521	.490	.477	.458
.44	.562	.527	.513	.491
.46	.604	.566	.550	.526
.48	.647	.605	.588	.560
.50	.691	.646	.627	.596
.55[b]	.807	.752	.729	.688
.60	.930	.864	.836	.785
.65	1.060	.982	.948	.886
.70	1.195	1.106	1.066	.990
.75		1.236	1.190	1.098
.80		1.371	1.318	1.210
.85		1.512	1.453	1.326
.90		1.658	1.591	1.445
.95		1.810	1.735	1.568
1.00		1.966	1.883	1.693
$\Delta H =$ [c]	0.051 m or $0.1H_1$	0.091 m or $0.1H_1$	0.121 m or $0.1H_1$	$0.4H_1$

[a] $L_a \ge H_{1max}$
$L_b = 3\,p_1$
$L_a + L_b \ge 2$ to 3 times H_{1max}

[b] Change in head increment

[c] ΔH values are for abrupt expansion into rectangular channel of same width as crest. Use the larger of $0.1H_1$ or the value shown. For discharge into a stagnant pool, head loss is $0.4H_1$.

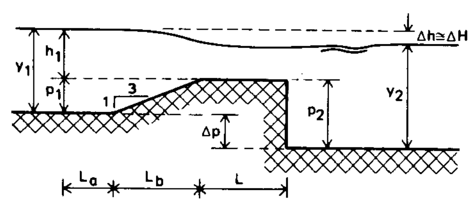


Table R.4. Rating Tables for Rectangular Weirs in English Units with Discharge per Foot Width ($X_C = 3\%$).[a]

$0.35 \le b_c \le 0.65$ ft, $L = 0.75$ ft			
h_1	q (ft^3/s per foot width)		
(ft)	$p_1 = 0.125$ ft	$p_1 = 0.25$ ft	$p_1 = \infty$
0.07	0.0554	0.0542	0.0537
.08	.0685	.0669	.0661
.09	.0827	.0806	.0794
.10	.0979	.0951	.0934
.11	.1140	.1104	.1081
.12	.1310	.1265	.1236
.13	.1489	.1434	.1397
.14	.1677	.1611	.1565
.15	.1873	.1795	.1739
.16	.208	.1987	.1918
.17	.229	.219	.210
.18	.251	.239	.230
.19	.274	.260	.249
.20	.297	.282	.269
.21	.322	.305	.290
.22	.347	.328	.311
.23	.373	.352	.333
.24	.399	.376	.355
.25	.426	.402	.378
.26	.454	.427	.401
.27	.483	.454	.424
.28	.512	.481	.448
.29	.543	.508	.473
.30	.573	.537	.498
.32[b]	.637	.595	.549
.34	.703	.655	.602
.36	.771	.718	.656
.38	.842	.783	.712
.40	.915	.850	.769
.42	.991	.919	.828
.44	1.069	.990	.888
.46	1.149	1.064	.949
.48	1.232	1.139	1.012
.50	1.317	1.217	1.076
$\Delta H =$[c]	0.04 ft or $0.1H_1$	0.06 ft or $0.1H_1$	$0.4H_1$

$0.65 \le b_c \le 1.0$ ft, $L = 1.0$ ft			
h_1	q (ft^3/s per foot width)		
(ft)	$p_1 = 0.25$ ft	$p_1 = 0.5$ ft	$p_1 = \infty$
0.12	0.1263	0.1240	0.1231
.13	.1433	.1405	.1392
.14	.1610	.1576	.1560
.15	.1795	.1754	.1733
.16	.1987	.1939	.1913
.17	.219	.213	.210
.18	.239	.233	.229
.19	.261	.253	.249
.20	.283	.274	.269
.22[b]	.328	.318	.311
.24	.377	.364	.355
.26	.428	.412	.401
.28	.481	.462	.448
.30	.537	.514	.498
.32	.596	.569	.549
.34	.656	.625	.601
.36	.719	.684	.656
.38	.784	.744	.712
.40	.851	.807	.769
.42	.920	.871	.828
.44	.992	.937	.888
.46	1.065	1.005	.950
.48	1.141	1.075	1.013
.50	1.219	1.146	1.077
.52	1.299	1.220	1.143
.54	1.380	1.295	1.210
.56	1.464	1.371	1.278
.58	1.549	1.450	1.347
.60	1.637	1.530	1.418
.62	1.726	1.612	1.490
.64	1.817	1.695	1.562
.66	1.910	1.780	1.637
.68	2.01	1.867	1.712
$\Delta H =$[c]	0.06 ft or $0.1H_1$	0.10 ft or $0.1H_1$	$0.4H_1$

[a] $L_a \ge H_{1max}$
$L_b = 3\,p_1$
$L_a + L_b \ge 2$ to 3 times H_{1max}

[b] Change in head increment

[c] ΔH values are for abrupt expansion into rectangular channel of same width as crest. Use the larger of $0.1H_1$ or the value shown. For discharge into a stagnant pool, head loss is $0.4H_1$.

(continued)

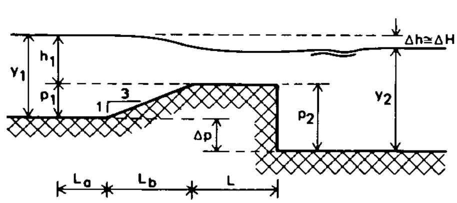


Table R.4. *(continued)*[a]

h_1 (ft)	$1.0 \le b_c \le 1.5$ ft, $L = 1.5$ ft — q (ft^3/s per foot width) $p_1 = 0.25$ ft	$p_1 = 0.50$ ft	$p_1 = \infty$
0.14	0.1600	0.1567	0.1550
.16	.1976	.1929	.1904
.18	.238	.232	.228
.20	.281	.273	.268
.22	.327	.317	.310
.24	.376	.363	.354
.26	.427	.411	.400
.28	.480	.461	.447
.30	.536	.513	.497
.32	.595	.568	.548
.34	.655	.624	.601
.36	.718	.683	.655
.38	.781	.743	.711
.40	.848	.804	.767
.42	.916	.868	.825
.44	.987	.933	.885
.46	1.060	1.001	.946
.48	1.135	1.070	1.009
.50	1.212	1.141	1.072
.55[b]	1.413	1.326	1.238
.60	1.626	1.521	1.411
.65	1.851	1.728	1.592
.70	2.09	1.944	1.780
.75	2.33	2.17	1.978
.80	2.59	2.41	2.18
.85	2.87	2.65	2.39
.90	3.14	2.91	2.60
.95		3.17	2.82
1.00		3.44	3.05
$\Delta H =$ [c]	0.07 ft or $0.1H_1$	0.11 ft or $0.1H_1$	$0.4H_1$

h_1 (ft)	$1.5 \le b_c \le 3.0$ ft, $L = 2.25$ ft — q (ft^3/s per foot width) $p_1 = 0.25$ ft	$p_1 = 0.5$ ft	$p_1 = 1.0$ ft	$p_1 = \infty$
0.24	.372	.359	.352	.350
.26	.422	.407	.399	.395
.28	.475	.456	.447	.442
.30	.530	.508	.497	.491
.32	.587	.562	.549	.542
.34	.647	.618	.602	.594
.36	.709	.676	.658	.648
.38	.774	.736	.715	.703
.40	.840	.798	.774	.760
.42	.909	.861	.834	.818
.44	.979	.927	.897	.878
.46	1.052	.994	.961	.939
.48	1.127	1.063	1.026	1.002
.50	1.204	1.134	1.093	1.065
.55[b]	1.405	1.319	1.268	1.231
.60	1.617	1.515	1.451	1.404
.65	1.841	1.721	1.644	1.585
.70	2.08	1.936	1.845	1.772
.75	2.32	2.17	2.06	1.969
.80	2.58	2.40	2.27	2.17
.85	2.85	2.64	2.51	2.38
.90	3.13	2.90	2.74	2.59
.95	3.42	3.16	2.98	2.82
1.00	3.71	3.43	3.23	3.04
1.05		3.71	3.49	3.27
1.10		4.00	3.76	3.51
1.15		4.30	4.03	3.75
1.20		4.61	4.31	4.00
1.25		4.92	4.60	4.25
1.30		5.25	4.90	4.51
1.35		5.58	5.20	4.77
1.40		5.92	5.51	5.04
1.45		6.26	5.83	5.32
1.50		6.62	6.15	5.59
$\Delta H =$ [c]	0.07 ft or $0.1H_1$	0.13 ft or $0.1H_1$	0.20 ft or $0.1H_1$	$0.4H_1$

[a] $L_a \ge H_{1max}$
$L_b = 3\,p_1$
$L_a + L_b \ge 2$ to 3 times H_{1max}

[b] Change in head increment

[c] ΔH values are for abrupt expansion into rectangular channel of same width as crest. Use the larger of $0.1H_1$ or the value shown. For discharge into a stagnant pool, head loss is $0.4H_1$.

(continued)

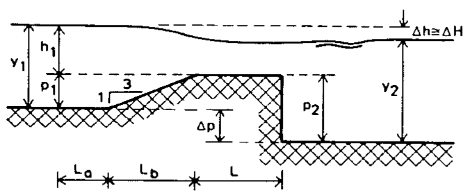


Table R.4. *(continued)*[a]

$3.0 \le b_c \le 6.0$ ft, $L = 3.0$ ft				
h_1	q (ft^3/s per foot width)			
(ft)	$p_1 = 0.5$ ft	$p_1 = 1.0$ ft	$p_1 = 1.5$ ft	$p_1 = \infty$
0.30	0.504	0.493	0.489	0.486
.35	.642	.626	.620	.615
.40	.793	.770	.762	.755
.45	.955	.924	.914	.903
.50	1.129	1.089	1.075	1.060
.55	1.314	1.263	1.246	1.226
.60	1.510	1.447	1.425	1.399
.65	1.716	1.640	1.613	1.580
.70	1.932	1.842	1.809	1.768
.75	2.16	2.05	2.01	1.963
.80	2.40	2.27	2.23	2.16
.85	2.64	2.50	2.45	2.37
.90	2.90	2.74	2.67	2.59
.95	3.16	2.98	2.91	2.81
1.00	3.43	3.23	3.15	3.03
1.05	3.71	3.49	3.40	3.27
1.10	4.00	3.76	3.66	3.50
1.15	4.30	4.03	3.92	3.75
1.20	4.61	4.31	4.19	4.00
1.25	4.93	4.60	4.47	4.25
1.30	5.25	4.90	4.75	4.51
1.35	5.58	5.20	5.05	4.77
1.40	5.92	5.51	5.34	5.04
1.45	6.27	5.83	5.65	5.32
1.50	6.62	6.16	5.96	5.60
1.60[b]	7.36	6.83	6.60	6.17
1.70	8.12	7.52	7.26	6.76
1.80	8.91	8.25	7.95	7.37
1.90		9.00	8.66	7.99
2.00		9.77	9.40	8.63
$\Delta H =$ [c]	0.14 ft or $0.1H_1$	0.22 ft or $0.1H_1$	0.29 ft or $0.1H_1$	$0.4H_1$

$b_c \ge 6.0$ ft, $L = 4.5$ ft				
h_1	q (ft^3/s per foot width)			
(ft)	$p_1 = 1.0$ ft	$p_1 = 1.5$ ft	$p_1 = 2.0$ ft	$p_1 = \infty$
0.45	0.913	0.903	0.898	0.891
.50	1.077	1.064	1.057	1.048
.55	1.251	1.234	1.225	1.213
.60	1.434	1.413	1.402	1.386
.65	1.627	1.600	1.587	1.566
.70	1.828	1.796	1.780	1.754
.75	2.04	2.00	1.982	1.949
.80	2.26	2.21	2.19	2.15
.85	2.49	2.43	2.41	2.36
.90	2.72	2.66	2.63	2.57
.95	2.96	2.90	2.86	2.79
1.00	3.22	3.14	3.10	3.02
1.10[b]	3.74	3.64	3.59	3.49
1.20	4.30	4.18	4.12	3.98
1.30	4.88	4.74	4.66	4.50
1.40	5.50	5.33	5.24	5.03
1.50	6.14	5.94	5.83	5.58
1.60	6.81	6.58	6.46	6.15
1.70	7.51	7.25	7.10	6.75
1.80	8.23	7.94	7.77	7.35
1.90	8.98	8.65	8.46	7.98
2.00	9.76	9.39	9.18	8.62
2.10	10.56	10.15	9.91	9.28
2.20	11.38	10.93	10.67	9.96
2.30	12.23	11.74	11.45	10.64
2.40	13.11	12.57	12.25	11.35
2.50	14.00	13.42	13.07	12.07
2.60	14.92	14.29	13.91	12.81
2.70	15.87	15.18	14.78	13.56
2.80	16.83	16.10	15.66	14.32
2.90	17.82	17.04	16.56	15.10
3.00	18.83	17.99	17.49	15.89
$\Delta H =$ [c]	0.25 ft or $0.1H_1$	0.33 ft or $0.1H_1$	0.40 ft or $0.1H_1$	$0.4H_1$

[a] $L_a \ge H_{1max}$
$L_b = 3\,p_1$
$L_a + L_b \ge 2$ to 3 times H_{1max}

[b] Change in head increment

[c] ΔH values are for abrupt expansion into rectangular channel of same width as crest. Use the larger of $0.1H_1$ or the value shown. For discharge into a stagnant pool, head loss is $0.4H_1$.

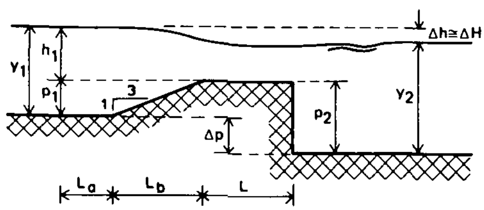
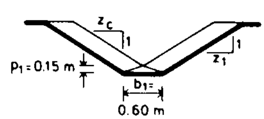


Table R.5. Rating Tables for Triangular-Throated Flumes (X_C = 2%).[a]

Metric Units				*English Units*			
h_1	Q (m^3/s)			h_1	Q (ft^3/s)		
(m)	$z_c = 1$	$z_c = 2$	$z_c = 3$	*(ft)*	$z_c = 1$	$z_c = 2$	$z_c = 3$
				0.25	0.063	0.130	0.196
				.30	.101	.207	.313
				.35	.149	.308	.465
				.40	.211	.434	.655
0.08	0.0020	0.0042	0.0063	.45	.286	.587	.885
.10	.0036	.0074	.0111	.50	.376	.769	1.159
.12	.0057	.0118	.0178	.55	.480	.981	1.480
.14	.0085	.0175	.0264	.60	.600	1.225	1.849
.16	.0121	.0247	.0372	.65	.737	1.504	2.27
.18	.0163	.0333	.0503	.70	.891	1.817	2.74
.20	.0214	.0436	.0658	.75	1.063	2.17	3.27
.22	.0273	.0556	.0840	.80	1.254	2.56	3.86
.24	.0341	.0695	.1049	.85	1.465	2.98	4.51
.26	.0419	.0853	.1288	.90	1.695	3.45	5.22
.28	.0506	.1031	.1557	.95	1.947	3.97	5.99
.30	.0604	.1230	.1858	1.00	2.22	4.52	6.83
.32	.0712	.1451	.219	1.05	2.51	5.12	7.75
.34	.0831	.1694	.256	1.10	2.83	5.77	8.73
.36	.0962	.1961	.297	1.15	3.17	6.47	9.78
.38	.1104	.225	.341	1.20	3.54	7.21	10.91
.40	.1259	.257	.389	1.25	3.93	8.01	12.12
.42	.1426	.291	.441	1.30	4.34	8.86	13.41
.44	.1607	.328	.497	1.35	4.78	9.76	14.77
.46	.1800	.368	.558	1.40	5.25	10.71	16.23
.48	.201	.410	.622	1.45	5.74	11.72	17.76
.50	.223	.456	.692	1.50	6.26	12.79	19.38
.55[b]	.285	.583	.886	1.60[b]	7.38	15.10	22.9
.60	.356	.729	1.108	1.70	8.62	17.65	26.8
.65	.437	.898	1.365	1.80	9.98	20.4	31.0
.70	.528	1.087	1.653	1.90	11.47	23.5	35.7
.75	.631	1.300	1.980	2.00	13.08	26.8	40.8
.80	.745	1.536	2.34	2.10	14.82	30.4	46.3
				2.20	16.71	34.3	52.2
				2.30	18.73	38.5	58.6
				2.40	20.9	43.0	65.5
				2.50	23.2	47.8	72.8
				2.60	25.7	52.9	80.6
$\Delta H =$ [c]	0.09 m	0.07 m or $0.1H_1$	0.06 m or $0.1H_1$	$\Delta H =$ [c]	0.30 ft	0.22 ft or $0.1H_1$	0.19 ft or $0.1H_1$

[a] $b_1 = b_2 = 0.60$ m; $b_c = 0$; $p_1 = p_2 = 0.15$ m;
$L_a = 0.90$ m; $L_b = 1.0$ m; $L = 1.2$ m; $z_1 = z_c = z_2$.



[b] Change in head increment

[c] For an abrupt expansion into a stagnant pool, $\Delta H = 0.24H_1$.

Table R.6. Rating Tables for Sills in 1-m Diameter Circular Conduits ($X_C = 2\%$).[a]

h_1 (m)	Q (m^3/s)						
	p_1=0.20 m	p_1=0.25 m	p_1=0.30 m	p_1=0.35 m	p_1=0.40 m	p_1=0.45 m	p_1=0.50 m
0.06				0.0232	0.0238	0.0242	0.0243
.08	0.0323	0.0335	0.0351	.0363	.0372	.0377	.0377
.10	.0463	.0480	.0500	.0515	.0525	.0530	.0530
.12	.0624	.0644	.0668	.0686	.0697	.0702	.0700
.14	.0804	.0828	.0855	.0875	.0886	.0890	.0885
.16	.1004	.1029	.1060	.1080	.1091	.1093	.1084
.18	.1222	.1249	.1281	.1302	.1312	.1311	.1297
.20	.1459	.1486	.1519	.1539	.1547	.1542	.1523
.22	.1714	.1740	.1773	.1791	.1796	.1786	.1760
.24	.1986	.201	.204	.206	.206	.204	.201
.26	.228	.230	.233	.234	.233	.231	.227
.28	.258	.260	.262	.263	.262	.259	.253
.30	.291	.292	.294	.294	.292	.288	.281
.32	.325	.325	.326	.326	.323	.318	.310
.34	.360	.360	.360	.359	.355	.349	.339
.36	.397	.396	.396	.393	.388	.380	.369
.38	.436	.433	.432	.429	.422	.413	
.40	.476	.472	.470	.465	.457	.446	
.42	.518	.512	.509	.503	.493		
.44	.561	.553	.549	.541	.530		
.46		.596	.590	.581	.568		
.48		.640	.632	.621			
.50		.685	.675	.662			
.52		.731	.720				
.54		.779	.765				
.56		.827	.811				
.58		.877					
.60		.928					

[a] See Table 5.4 for details on flume dimensions and head-loss values.

Table R.7. Rating Table for Sills in 1-ft Diameter Circular Conduits (X_C = 2%).[a]

h_1	Q (ft^3/s)						
(ft)	p_1=0.20 ft	p_1=0.25 ft	p_1=0.30 ft	p_1=0.35 ft	p_1=0.40 ft	p_1=0.45 ft	p_1=0.50 ft
0.06			0.040	0.041	0.042	0.043	0.043
.08	0.058	0.060	.063	.065	.066	.067	.067
.10	.083	.087	.090	.093	.094	.095	.095
.12	.112	.117	.121	.124	.126	.126	.126
.14	.145	.150	.155	.158	.160	.160	.159
.16	.181	.187	.192	.196	.197	.197	.196
.18	.221	.227	.232	.236	.237	.237	.234
.20	.264	.270	.276	.279	.280	.279	.275
.22	.311	.317	.322	.325	.325	.323	.318
.24	.360	.366	.371	.374	.373	.370	.363
.26	.413	.418	.423	.425	.423	.418	.410
.28	.469	.473	.477	.478	.475	.469	.459
.30	.527	.531	.534	.534	.530	.522	.509
.32	.589	.592	.594	.592	.586	.576	.561
.34	.654	.655	.656	.652	.645	.632	.614
.36	.722	.721	.720	.715	.705	.690	.669
.38	.792	.789	.786	.779	.767	.749	
.40	.865	.860	.855	.845	.831	.810	
.42	.941	.933	.926	.914	.896		
.44	1.019	1.008	.999	.984	.963		
.46		1.086	1.074	1.056	1.031		
.48		1.166	1.151	1.129			
.50		1.249	1.230	1.205			
.52		1.333	1.311				
.54		1.420	1.393				
.56		1.509	1.478				
.58		1.600					
.60		1.693					

[a] See Table 5.4 for details on flume dimensions and head-loss values.

Table R.8. Rating Tables for Movable Weirs in Metric Units ($X_C = 3\%$).[a]

$L = 0.50$ m $0.3\ m \le b_c \le 2.0$ m			$L = 0.75$ m $0.5\ m \le b_c \le 3.0$ m			$L = 1.00$ m $1.0\ m \le b_c \le 4.0$ m		
	q (*m³/s per meter width*)			*q* (*m³/s per meter width*)			*q* (*m³/s per meter width*)	
h_1 (*m*)	*Bottom Drop*	*Bottom Gate*	h_1 (*m*)	*Bottom Drop*	*Bottom Gate*	h_1 (*m*)	*Bottom Drop*	*Bottom Gate*
0.05	0.0185	0.0185				0.10	0.0520	0.0519
.06	.0245	.0245				.12	.0687	.0686
.07	.0310	.0310	0.07	0.0305	0.0305	.14	.0869	.0868
.08	.0380	.0379	.08	.0374	.0374	.16	.1066	.1064
.09	.0455	.0454	.09	.0448	.0448	.18	.1275	.1272
.10	.0534	.0533	.10	.0526	.0525	.20	.1498	.1493
.11	.0617	.0615	.11	.0608	.0607	.22	.1733	.1726
.12	.0704	.0701	.12	.0694	.0693	.24	.1979	.1970
.13	.0796	.0792	.13	.0784	.0783	.26	.224	.223
.14	.0891	.0886	.14	.0878	.0876	.28	.251	.249
.15	.0990	.0983	.15	.0976	.0973	.30	.278	.277
.16	.1093	.1085	.16	.1076	.1073	.32	.308	.305
.17	.1200	.1189	.17	.1181	.1176	.34	.338	.335
.18	.1310	.1297	.18	.1288	.1283	.36	.369	.365
.19	.1424	.1408	.19	.1399	.1393	.38	.401	.396
.20	.1542	.1523	.20	.1513	.1505	.40	.434	.429
.22[b]	.1789	.1761	.22[b]	.1751	.1740	.42	.468	.462
.24	.205	.201	.24	.200	.1986	.44	.503	.496
.26	.233	.227	.26	.226	.224	.46	.540	.531
.28	.262	.255	.28	.254	.251	.48	.577	.567
.30	.292	.283	.30	.282	.279	.50	.615	.603
.32	.325	.313	.32	.312	.308	.55[b]	.716	.699
.34	.359	.344	.34	.343	.338	.60	.823	.799
.36	.394	.376	.36	.375	.369	.65	.938	.905
.38	.432	.409	.38	.409	.401	.70	1.060	1.016
.40	.472	.444	.40	.443	.433	.75	1.190	1.132
.42	.514	.480	.42	.479	.467	.80	1.329	1.253
.44	.558	.516	.44	.516	.502	.85	1.478	1.380
.46	.606	.554	.46	.555	.538	.90	1.639	1.512
.48		.594	.48	.595	.574	.95		1.650
			.50	.636	.612			
			.55[b]	.746	.711			
			.60	.865	.815			
			.65	.996	.926			
			.70		1.042			
$\Delta H =$	0.13 m	0.11 m	$\Delta H =$	0.20 m	0.17 m	$\Delta H =$	0.26 m	0.22 m

[a] Weir dimensions: $L_b = 0.1H_{1max} = 0.1L$; $L_a = 2H_1max = 2L$.
Bottom-drop: $y_1 = 1.33H_{1max}$; $p_{1min} = 0.33H_{1max}$.
Bottom-gate: $y_1 = 2H_{1max} + 0.05$ m; $p_{1min} = H_{1max} + 0.05$ m.

[b] Change in head increment.

Table R.9. Rating Tables for Movable Weirs in English Units ($X_C = 3\%$).[a]

$L = 1.0$ ft $1.0 \text{ ft} \le b_c \le 4.0$ ft			$L = 2.0$ ft $1.5 \text{ ft} \le b_c \le 8$ ft			$L = 3.0$ ft $3.0 \text{ ft} \le b_c \le 12$ ft		
	q (ft^3/s per foot width)			*q* (ft^3/s per foot width)			*q* (ft^3/s per foot width)	
h_1 (ft)	Bottom Drop	Bottom Gate	h_1 (ft)	Bottom Drop	Bottom Gate	h_1 (ft)	Bottom Drop	Bottom Gate
0.10	0.094	0.094						
.12	.125	.125				0.30	0.483	0.487
.14	.158	.158				.35	.617	.617
.16	.194	.194				.40	.750	.755
.18	.232	.232	0.20	0.270	0.270	.45	.900	.905
.20	.272	.273	.25	.378	.375	.50	1.067	1.062
.22	.314	.316	.30	.498	.495	.60[b]	1.400	1.402
.24	.359	.360	.35	.628	.625	.70	1.783	1.773
.26	.405	.407	.40	.770	.768	.80	2.18	2.17
.28	.454	.456	.45	.920	.918	.90	2.62	2.60
.30	.504	.507	.50	1.080	1.075	1.00	3.08	3.05
.32	.556	.560	.55	1.250	1.243	1.10	3.57	3.53
.34	.609	.615	.60	1.428	1.418	1.20	4.08	4.03
.36	.665	.672	.65	1.615	1.603	1.30	4.62	4.56
.38	.722	.731	.70	1.810	1.793	1.40	5.20	5.11
.40	.781	.791	.75	2.02	1.993	1.50	5.78	5.68
.42	.841	.853	.80	2.23	2.20	1.60	6.42	6.27
.44	.903	.918	.85	2.45	2.41	1.70	7.07	6.89
.46	.966	.984	.90	2.68	2.63	1.80	7.75	7.52
.48	1.031	1.052	.95	2.91	2.86	1.90	8.47	8.18
.50	1.098	1.122	1.00	3.16	3.09	2.00	9.20	8.86
.55[b]	1.271	1.305	1.10[b]	3.67	3.58	2.10	9.97	9.56
.60	1.453	1.501	1.20	4.22	4.10	2.20	10.78	10.29
.65	1.645	1.710	1.30	4.81	4.64	2.30	11.63	11.03
.70	1.845	1.932	1.40	5.44	5.21	2.40	12.52	11.80
.75	2.06	2.17	1.50	6.11	5.80	2.50	13.43	12.59
.80	2.27	2.42	1.60	6.82	6.42	2.60	14.42	13.40
.84	2.51	2.70	1.70	7.59	7.07	2.70	15.43	14.24
.90	2.74	2.99	1.80	8.41	7.74	2.80		15.10
.95	2.99	3.31	1.90		8.45	2.90		15.98
$\Delta H =$	0.26 ft	0.22 ft	$\Delta H =$	0.52 ft	0.44 ft	$\Delta H =$	0.78 ft	0.66 ft

[a] Weir dimensions: $L_b = 0.1H_{1max} = 0.1L$; $L_a = 2H_{1max} = 2L$.
Bottom-drop: $y_1 = 1.33H_{1max}$; $p_{1min} = 0.33H_{1max}$.
Bottom-gate: $y_1 = 2H_{1max} + 0.164$ ft; $p_{1min} = H_{1max} + 0.164$ ft.

[b] Change in head increment.

Table R.10. Head-Discharge Relationships of 5 RBC Flumes for Use in Unlined Channels (Metric Units) ($X_C = 2\%$).[a]

b_c = 50 mm		b_c = 75 mm		b_c = 100 mm		b_c = 150 mm		b_c = 200 mm	
h_1 (mm)	Q (liters/s)	h_1 (mm)	Q (liters/s)	h_1 (mm)	Q (liters/s)	h_1 (mm)	Q (liters/s)	h_1 (mm)	Q (liters/s)
5	0.026			10	0.159			20	0.935
6	.036			12	.216			22	1.092
7	.047	7	0.066	14	.278	14	0.401	24	1.258
8	.059	8	.084	16	.347	16	.500	26	1.433
9	.072	9	.102	18	.422	18	.606	28	1.617
10	.086	10	.122	20	.503	20	.720	30	1.809
11	.101	11	.143	22	.590	22	.842	32	2.01
12	.118	12	.166	24	.682	24	.971	34	2.22
13	.135	13	.190	26	.780	26	1.108	36	2.44
14	.153	14	.215	28	.884	28	1.251	38	2.66
15	.172	15	.241	30	.994	30	1.402	40	2.90
16	.192	16	.269	32	1.109	32	1.560	42	3.14
17	.214	17	.298	34	1.230	34	1.725	44	3.39
18	.236	18	.328	36	1.357	36	1.897	46	3.65
19	.259	19	.359	38	1.490	38	2.08	48	3.92
20	.283	20	.392	40	1.628	40	2.26	50	4.19
21	.309	21	.426	42	1.773	42	2.46	55[b]	4.91
22	.335	22	.461	44	1.923	44	2.66	60	5.69
23	.363	23	.497	46	2.08	46	2.86	65	6.51
24	.391	24	.535	48	2.24	48	3.08	70	7.39
25	.421	25	.574	50	2.41	50	3.30	75	8.32
26	.451	26	.614	55[b]	2.86	55[b]	3.89	80	9.30
27	.483	27	.655	60	3.34	60	4.51	85	10.33
28	.516	28	.698	65	3.87	65	5.19	90	11.41
29	.549	29	.741	70	4.43	70	5.91	95	12.55
30	.584	30	.786	75	5.04	75	6.68	100	13.74
31	.620	32[b]	.880	80	5.68	80	7.49	105	14.98
32	.657	34	.980	85	6.37	85	8.35	110	16.28
33	.696	36	1.087	90	7.10	90	9.25	115	17.63
34	.735	38	1.197	95	7.87	95	10.21	120	19.03
35	.775	40	1.312	100	8.68	100	11.21	125	20.5
36	.817	42	1.432			105	12.26	130	22.0
37	.860	44	1.557			110	13.36	135	23.6
38	.904	46	1.688			115	14.51	140	25.2
39	.949	48	1.824			120	15.71	145	26.9
40	.995	50	1.966			125	16.96	150[b]	28.7
42[b]	1.091	55	2.34			130	18.26	160	32.3
44	1.191	60	2.75			135	19.62	170	36.2
46	1.297	65	3.20			140	21.0	180	40.3
48	1.407	70	3.69			145	22.5	190	44.7
50	1.522	75	4.21			150	24.0	200	49.4

[a] See Table 5.5 for details on flume dimensions and head-loss values.
[b] Change in head increment.

Table R.11. Head-Discharge Relationships of 5 RBC Flumes for Use in Unlined Channels (English Units) (X_C = 2%).[a]

b_c = 50 mm		b_c = 75 mm		b_c = 100 mm		b_c = 150 mm		b_c = 200 mm	
h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)	h_1 (ft)	Q (ft^3/s)
				0.035	0.0062			0.07	0.0368
		0.026	0.0029	.040	.0078			.08	.0457
0.018	0.0011	.028	.0033	.045	.0096			.09	.0553
.020	.0013	.030	.0037	.050	.0114	0.050	0.0163	.10	.0655
.022	.0015	.032	.0041	.055	.0131	.055	.0191	.11	.0766
.024	.0018	.034	.0046	.060	.0154	.060	.0220	.12	.0882
.026	.0020	.036	.0050	.065	.0174	.065	.0250	.13	.1007
.028	.0023	.038	.0055	.070	.0198	.070	.0283	.14	.1139
.030	.0026	.040	.0060	.075	.0223	.075	.0317	.15	.1275
.032	.0029	.042	.0065	.080	.0247	.080	.0352	.16	.1419
.034	.0032	.044	.0071	.085	.0274	.085	.0389	.17	.1571
.036	.0036	.046	.0076	.090	.0301	.090	.0427	.18	.1729
.038	.0039	.048	.0082	.095	.0330	.095	.0467	.19	.1892
.040	.0043	.050	.0087	.100	.0361	.100	.0508	.20	.206
.042	.0046	.055[b]	.0103	.105	.0392	.110[b]	.0595	.21	.224
.044	.0050	.060	.0119	.110	.0423	.120	.0688	.22	.242
.046	.0054	.065	.0136	.115	.0457	.130	.0787	.23	.262
.048	.0058	.070	.0154	.120	.0492	.140	.0891	.24	.281
.050	.0062	.075	.0174	.125	.0528	.150	.1001	.25	.302
.055[b]	.0074	.080	.0194	.130	.0566	.160	.1116	.26	.323
.060	.0086	.085	.0215	.135	.0604	.170	.1239	.27	.345
.065	.0099	.090	.0238	.140	.0644	.180	.1366	.28	.367
.070	.0112	.095	.0261	.145	.0684	.190	.1500	.29	.391
.075	.0127	.100	.0286	.150	.0726	.200	.1638	.30	.414
.080	.0142	.105	.0311	.160[b]	.0813	.210	.1785	.32[b]	.464
.085	.0159	.110	.0338	.170	.0907	.220	.1934	.34	.517
.090	.0175	.115	.0365	.180	.1005	.230	.209	.36	.572
.095	.0194	.120	.0394	.190	.1107	.240	.226	.38	.631
.100	.0212	.125	.0423	.200	.1214	.250	.242	.40	.692
.110[b]	.0253	.130	.0454	.210	.1328	.260	.260	.42	.756
.120	.0297	.135	.0486	.220	.1446	.270	.278	.44	.823
.130	.0345	.140	.0519	.230	.1569	.280	.297	.46	.893
.140	.0397	.145	.0553	.240	.1698	.290	.316	.48	.966
.150	.0452	.150	.0588	.250	.1832	.300	.336	.50	1.042
.160	.0512	.160[b]	.0662	.260	.1970	.320[b]	.378	.52	1.121
		.170	.0740	.270	.212	.340	.423	.54	1.203
		.180	.0822	.280	.227	.360	.470	.56	1.289
		.190	.0909	.290	.242	.380	.519	.58	1.377
		.200	.1001	.300	.258	.400	.572	.60	1.469
		.210	.1098	.310	.275	.420	.627	.62	1.563
		.220	.1199	.320	.292	.440	.684	.64	1.661
		.230	.1305			.460	.745	.66	1.763
		.240	.1416			.480	.808		
		.250	.1532			.500	.874		

[a] See Table 5.5 for details on flume dimensions and head-loss values.
[b] Change in head increment.

Table R.12. Rating Tables for Commercially Available Rectangular Fiberglass Flume ($X_C = 2\%$).[a]

Metric Units		English Units	
h_1 (mm)	Q (liters/s)	h_1 (ft)	Q (ft^3/s)
20	1.73	0.08	0.084
25	2.46	.10	.119
30	3.28	.12	.158
35	4.19	.14	.202
40	5.16	.16	.249
45	6.21	.18	.300
50	7.34	.20	.354
55	8.53	.22	.412
60	9.78	.24	.473
65	11.10	.26	.537
70	12.48	.28	.604
75	13.93	.30	.674
80	15.43	.32	.747
85	16.99	.34	.823
90	18.61	.36	.902
95	20.3	.38	.983
100	22.0	.40	1.068
105	23.8	.42	1.155
110	25.6	.44	1.244
115	27.5	.46	1.337
120	29.5	.48	1.432
125	31.5	.50	1.529
130	33.5	.52	1.629
135	35.6	.54	1.731
140	37.8	.56	1.836
145	40.0	.58	1.943
150	42.2	.60	2.05
160	46.8	.62	2.17
170	51.7	.64	2.28
180	56.7	.66	2.40
190	61.9	.68	2.52
200	67.2		

[a] Flume Dimensions:

$b_1 = b_c = 1.25$ ft = 381 mm
$L = 1.00$ ft = 305 mm
$L_a = 0.75$ ft = 229 mm
$L_b = 0.75$ ft = 229 mm
$p_1 = 0.25$ ft = 76 mm
Diverging transition expansion factor, $m = 0$ (abrupt expansion)
$d = 1.0$ ft = 305 mm

Table R.13. Rating Equations and Properties of Adjust-A-Flumes in Metric Units ($X_C = 2\%$).

Flume Width (mm)	*Sill Height (mm)*	*Approach Length (mm)*	*Converging Transition Length (mm)*	*Throat Length (mm)*	*Head Range (mm)*	*Discharge Range (liters/s)*	*Discharge Equation (h_1 in meters) (m^3/s)*
154	25	102	192	685	13 - 115	0.4 - 13	$Q = 0.4190\,(h_1)^{1.624}$
154	51	102	187	685	13 - 126	0.4 - 13	$Q = 0.4462\,(h_1)^{1.594}$
154	76	102	178	685	13 - 126	0.4 - 13	$Q = 0.4673\,(h_1)^{1.576}$
154	102	102	165	685	13 - 126	0.4 - 13	$Q = 0.4836\,(h_1)^{1.564}$
305	51	229	387	320	21 - 198	2 - 55	$Q = 0.7449\,(h_1)^{1.610}$
305	76	229	383	320	21 - 204	2 - 55	$Q = 0.6895\,(h_1)^{1.592}$
305	102	229	377	320	21 - 207	2 - 55	$Q = 0.6549\,(h_1)^{1.579}$
305	127	229	369	320	21 - 207	2 - 54	$Q = 0.6311\,(h_1)^{1.569}$
305	152	229	360	320	21 - 210	2 - 54	$Q = 0.6144\,(h_1)^{1.562}$
305	178	229	348	320	21 - 201	2 - 50	$Q = 0.6004\,(h_1)^{1.555}$
305	203	229	334	320	21 - 177	2 - 40	$Q = 0.5883\,(h_1)^{1.549}$
610	51	229	387	320	21 - 198	3 - 111	$Q = 1.500\,(h_1)^{1.610}$
610	76	229	383	320	21 - 204	3 - 111	$Q = 1.387\,(h_1)^{1.593}$
610	102	229	377	320	21 - 207	3 - 111	$Q = 1.317\,(h_1)^{1.579}$
610	127	229	369	320	21 - 207	3 - 111	$Q = 1.268\,(h_1)^{1.569}$
610	152	229	360	320	21 - 210	3 - 109	$Q = 1.234\,(h_1)^{1.562}$
610	178	229	348	320	21 - 201	3 - 100	$Q = 1.206\,(h_1)^{1.555}$
610	203	229	334	320	21 - 177	3 - 81	$Q = 1.181\,(h_1)^{1.549}$
914	51	229	387	320	21 - 198	5 - 176	$Q = 2.256\,(h_1)^{1.611}$
914	76	229	383	320	21 - 204	5 - 167	$Q = 2.085\,(h_1)^{1.593}$
914	102	229	377	320	21 - 207	5 - 166	$Q = 1.978\,(h_1)^{1.580}$
914	127	229	369	320	21 - 207	5 - 162	$Q = 1.905\,(h_1)^{1.569}$
914	152	229	360	320	21 - 210	5 - 163	$Q = 1.853\,(h_1)^{1.562}$
914	178	229	348	320	21 - 201	5 - 150	$Q = 1.811\,(h_1)^{1.555}$
914	203	229	334	320	21 - 177	5 - 122	$Q = 1.774\,(h_1)^{1.549}$
762	76	385	758	685	49 - 357	14 - 340	$Q = 1.786\,(h_1)^{1.614}$
762	102	388	755	685	50 - 417	14 - 425	$Q = 1.716\,(h_1)^{1.607}$
762	127	392	751	685	50 - 424	14 - 425	$Q = 1.658\,(h_1)^{1.589}$
762	152	396	747	685	50 - 430	14 - 425	$Q = 1.690\,(h_1)^{1.590}$
762	178	402	741	685	50 - 435	14 - 425	$Q = 1.575\,(h_1)^{1.585}$
762	203	409	734	685	50 - 438	14 - 425	$Q = 1.545\,(h_1)^{1.577}$
762	229	416	727	685	50 - 442	14 - 425	$Q = 1.520\,(h_1)^{1.572}$
762	254	425	718	685	50 - 445	14 - 425	$Q = 1.499\,(h_1)^{1.567}$
762	279	434	709	685	50 - 447	14 - 425	$Q = 1.482\,(h_1)^{1.563}$
762	305	445	698	685	50 - 449	14 - 425	$Q = 1.466\,(h_1)^{1.559}$
762	330	456	687	685	50 - 451	14 - 425	$Q = 1.453\,(h_1)^{1.556}$
965	102	439	755	1219	88 - 613	42 - 991	$Q = 2.125\,(h_1)^{1.619}$
965	127	442	751	1219	88 - 622	42 - 991	$Q = 2.136\,(h_1)^{1.622}$
965	152	447	747	1219	88 - 631	42 - 991	$Q = 2.082\,(h_1)^{1.615}$
965	178	453	741	1219	88 - 640	42 - 991	$Q = 2.037\,(h_1)^{1.609}$
965	203	459	734	1219	88 - 640	42 - 991	$Q = 2.000\,(h_1)^{1.604}$
965	229	467	727	1219	91 - 640	42 - 991	$Q = 1.969\,(h_1)^{1.598}$
965	254	475	718	1219	91 - 640	42 - 991	$Q = 1.942\,(h_1)^{1.594}$
965	279	485	709	1219	91 - 640	42 - 991	$Q = 1.918\,(h_1)^{1.589}$
965	305	495	698	1219	91 - 640	42 - 991	$Q = 1.898\,(h_1)^{1.585}$
965	330	507	687	1219	91 - 640	42 - 991	$Q = 1.880\,(h_1)^{1.582}$
965	356	520	674	1219	91 - 640	42 - 991	$Q = 1.863\,(h_1)^{1.578}$
965	381	534	660	1219	91 - 640	42 - 991	$Q = 1.842\,(h_1)^{1.573}$
965	406	549	645	1219	91 - 640	42 - 991	$Q = 1.826\,(h_1)^{1.569}$

Table R.14. Rating Equations and Properties of Adjust-A-Flumes in English Units ($X_C = 2\%$).

Flume Width (in.)	*Sill Height (in.)*	*Approach Length (in.)*	*Converging Transition Length (in.)*	*Throat Length (in.)*	*Head Range (ft)*	*Discharge Range (ft^3/s)*	*Discharge Equation (h_1 in feet) (ft^3/s)*
6.08	1	4.0	7.6	7.5	0.04 - 0.38	0.013 - 0.45	$Q = 2.153\ (h_1)^{1.624}$
6.08	2	4.0	7.4	7.5	0.04 - 0.40	0.013 - 0.45	$Q = 1.927\ (h_1)^{1.594}$
6.08	3	4.0	7.0	7.5	0.04 - 0.41	0.013 - 0.45	$Q = 1.821\ (h_1)^{1.576}$
6.08	4	4.0	6.5	7.5	0.04 - 0.42	0.013 - 0.45	$Q = 1.760\ (h_1)^{1.564}$
12	2	9.0	15.2	12.6	0.07 - 0.65	0.055 - 1.95	$Q = 3.886\ (h_1)^{1.610}$
12	3	9.0	15.1	12.6	0.07 - 0.67	0.054 - 1.96	$Q = 3.673\ (h_1)^{1.592}$
12	4	9.0	14.8	12.6	0.07 - 0.68	0.054 - 0.68	$Q = 3.543\ (h_1)^{1.579}$
12	5	9.0	14.5	12.6	0.07 - 0.68	0.053 - 0.68	$Q = 3.455\ (h_1)^{1.569}$
12	6	9.0	14.2	12.6	0.07 - 0.69	0.053 - 0.69	$Q = 3.393\ (h_1)^{1.562}$
12	7	9.0	13.7	12.6	0.07 - 0.66	0.053 - 0.66	$Q = 3.342\ (h_1)^{1.555}$
12	8	9.0	13.1	12.6	0.07 - 0.58	0.053 - 0.58	$Q = 3.298\ (h_1)^{1.549}$
24	2	9.0	15.2	12.6	0.07 - 0.65	0.11 - 3.93	$Q = 7.820\ (h_1)^{1.610}$
24	3	9.0	15.1	12.6	0.07 - 0.67	0.11 - 3.93	$Q = 7.584\ (h_1)^{1.593}$
24	4	9.0	14.8	12.6	0.07 - 0.68	0.11 - 3.93	$Q = 7.119\ (h_1)^{1.579}$
24	5	9.0	14.5	12.6	0.07 - 0.68	0.11 - 3.82	$Q = 6.939\ (h_1)^{1.569}$
24	6	9.0	14.2	12.6	0.07 - 0.69	0.11 - 3.84	$Q = 6.813\ (h_1)^{1.562}$
24	7	9.0	13.7	12.6	0.07 - 0.66	0.11 - 3.54	$Q = 6.710\ (h_1)^{1.555}$
24	8	9.0	13.1	12.6	0.07 - 0.58	0.11 - 2.86	$Q = 6.621\ (h_1)^{1.549}$
36	2	9.0	15.2	12.6	0.07 - 0.65	0.16 - 5.90	$Q = 11.75\ (h_1)^{1.611}$
36	3	9.0	15.1	12.6	0.07 - 0.67	0.16 - 5.91	$Q = 11.10\ (h_1)^{1.593}$
36	4	9.0	14.8	12.6	0.07 - 0.68	0.16 - 5.86	$Q = 10.69\ (h_1)^{1.580}$
36	5	9.0	14.5	12.6	0.07 - 0.68	0.16 - 5.74	$Q = 10.42\ (h_1)^{1.569}$
36	6	9.0	14.2	12.6	0.07 - 0.69	0.16 - 5.77	$Q = 10.23\ (h_1)^{1.562}$
36	7	9.0	13.7	12.6	0.07 - 0.66	0.16 - 5.31	$Q = 10.08\ (h_1)^{1.555}$
36	8	9.0	13.1	12.6	0.07 - 0.58	0.16 - 4.29	$Q = 9.943\ (h_1)^{1.549}$
30	3	15.2	29.8	27.0	0.016 - 1.17	0.5 - 12	$Q = 9.266\ (h_1)^{1.614}$
30	4	15.3	29.7	27.0	0.163 - 1.17	0.5 - 15	$Q = 8.985\ (h_1)^{1.607}$
30	5	15.4	29.6	27.0	0.164 - 1.17	0.5 - 15	$Q = 8.771\ (h_1)^{1.598}$
30	6	15.6	29.4	27.0	0.165 - 1.17	0.5 - 15	$Q = 8.609\ (h_1)^{1.590}$
30	7	15.8	29.2	27.0	0.165 - 1.17	0.5 - 15	$Q = 8.483\ (h_1)^{1.583}$
30	8	16.1	28.9	27.0	0.165 - 1.17	0.5 - 15	$Q = 8.381\ (h_1)^{1.577}$
30	9	16.4	28.6	27.0	0.165 - 1.17	0.5 - 15	$Q = 8.289\ (h_1)^{1.572}$
30	10	16.7	28.3	27.0	0.165 - 1.17	0.5 - 15	$Q = 8.229\ (h_1)^{1.567}$
30	11	17.1	27.9	27.0	0.165 - 1.17	0.5 - 15	$Q = 8.170\ (h_1)^{1.563}$
30	12	17.5	27.5	27.0	0.166 - 1.17	0.5 - 15	$Q = 8.120\ (h_1)^{1.559}$
30	13	18.0	27.0	27.0	0.166 - 1.17	0.5 - 15	$Q = 8.077\ (h_1)^{1.556}$
38	4	17.3	29.7	48.0	0.29 - 2.01	1.5 - 35	$Q = 10.97\ (h_1)^{1.619}$
38	5	17.4	29.6	48.0	0.29 - 2.04	1.5 - 35	$Q = 10.98\ (h_1)^{1.622}$
38	6	17.6	29.4	48.0	0.29 - 2.07	1.5 - 35	$Q = 10.79\ (h_1)^{1.615}$
38	7	17.8	29.2	48.0	0.29 - 2.1	1.5 - 35	$Q = 10.63\ (h_1)^{1.609}$
38	8	18.1	28.9	48.0	0.29 - 2.1	1.5 - 35	$Q = 10.51\ (h_1)^{1.604}$
38	9	18.4	28.6	48.0	0.3 - 2.1	1.5 - 35	$Q = 10.41\ (h_1)^{1.598}$
38	10	18.7	28.3	48.0	0.3 - 2.1	1.5 - 35	$Q = 10.32\ (h_1)^{1.594}$
38	11	19.1	27.9	48.0	0.3 - 2.1	1.5 - 35	$Q = 10.25\ (h_1)^{1.589}$
38	12	19.5	27.5	48.0	0.3 - 2.1	1.5 - 35	$Q = 10.19\ (h_1)^{1.585}$
38	13	20.0	27.0	48.0	0.3 - 2.1	1.5 - 35	$Q = 10.14\ (h_1)^{1.582}$
38	14	20.5	26.5	48.0	0.3 - 2.1	1.5 - 35	$Q = 10.09\ (h_1)^{1.578}$
38	15	21.0	26.0	48.0	0.3 - 2.1	1.5 - 35	$Q = 10.04\ (h_1)^{1.593}$
38	16	21.6	25.4	48.0	0.3 - 2.1	1.5 - 35	$Q = 10.00\ (h_1)^{1.569}$

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