# VOLUME 3
# THEORY
# STORMWATER MANAGEMENT MANUAL

Prepared for

METROPOLITAN GOVERNMENT OF
NASHVILLE AND DAVIDSON COUNTY
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ENGINEERING DIVISION
NASHVILLE, TENNESSEE

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NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

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## Introduction

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# Chapter 1
# INTRODUCTION

## 1.1 PURPOSE

This volume of the Nashville Stormwater Management Manual has been prepared by the Metropolitan Department of Public Works (MDPW) as a theoretical supplement to the technical guidelines and procedures presented in Volume 2. The volume is a compilation of readily available literature relevant to stormwater management activities in Nashville and Davidson County. Although it is intended to establish uniform design practices, it neither replaces the need for engineering judgment nor precludes the use of information not presented.

## 1.2 FORMAT AND CONTENTS

The Nashville Stormwater Management Manual is presented in three volumes:

Volume 1 - Regulations
Volume 2 - Procedures
Volume 3 - Theory

Volume 3 contains the following chapters, which have titles identical to the same-numbered chapters in Volume 2:

1. Introduction
2. Hydrology
3. Open Channel Hydraulics
4. Gutter and Inlet Hydraulics
5. Culvert Hydraulics
6. Storm Sewer Hydraulics
7. Bridge Hydraulics
8. Detention/Retention Hydraulics
9. Erosion and Sediment Control
10. Outlet Protection

Tables and figures for Volume 3 are located immediately following the first textual reference. Each chapter is introduced by a detailed table of contents that includes table and figure titles. Cited literature for the entire volume is listed alphabetically by author after the last chapter. Terms and symbols are defined immediately after each equation.

## 1.3 LIMITATIONS

Theoretical information compiled in this volume is generally presented in a refresher-type context and assumes the user has some technical training related to stormwater management. The volume is not intended to function as a stand-alone reference text and the user is encouraged to utilize additional references. Since Volume 3 is a supplement to Volume 2 on procedures, it does not duplicate design aids such as charts, figures, tables, computer program information, and forms.

## 1.4 UPDATING

This volume of the Nashville Stormwater Management Manual is not expected to require significant changes. It will, however, be updated and revised as necessary to reflect up-to-date engineering practices and information applicable to Nashville and Davidson County. Registered manual users who provide a current address to MDPW will automatically be sent changes. Questions and comments should also be submitted to MDPW:

Attn: Stormwater Management
Metropolitan Department of Public Works
750 S. 5th Street
Nashville, Tennessee 37206

NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

# CHAPTER 2
## Hydrology

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# Chapter 2
# HYDROLOGY

## SYNOPSIS

Surface hydrology is a key component of stormwater management evaluations. Ideally, observed streamflow data should be used to evaluate the surface hydrology of a watershed. However, since streamflow data are usually not available, synthetic procedures based on theoretical and empirical relationships are often used in practice. Although this chapter provides information on the fundamentals of streamflow data analysis, the emphasis is on synthetic procedures for converting precipitation to runoff.

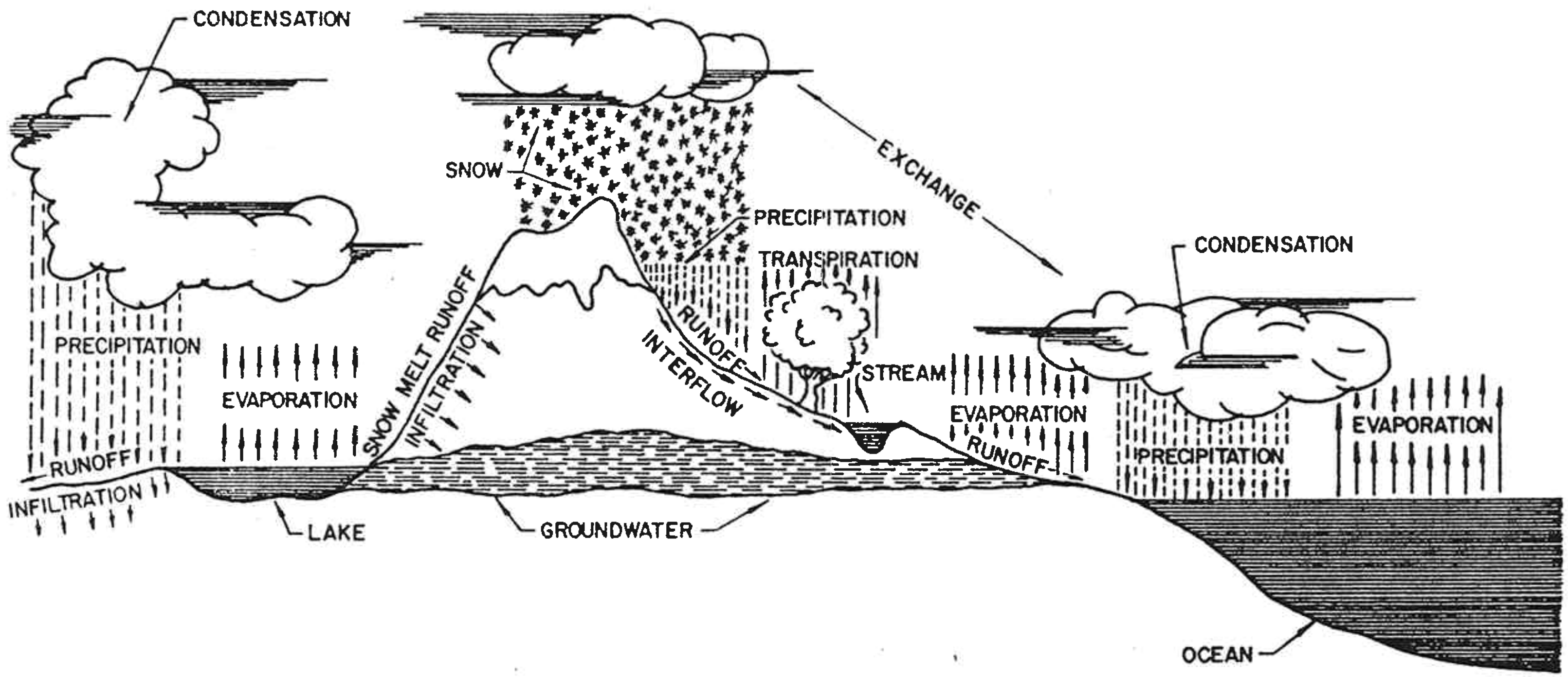
A general description of the hydrologic cycle and precipitation forms is followed by a discussion of relationships for evaluating rainfall excess. Fundamentals of synthetic procedures for the development of flood hydrographs and peak flow rates are followed by a discussion of hydrologic routing .

## 2.1 HYDROLOGIC CYCLE

The movement of water between the atmosphere and the earth's surface, and its transformation during this movement from one physical state to another, is called the hydrologic cycle. A simplified illustration of this process is provided in Figure 2-1.

During the hydrologic cycle, moisture in the warm air condenses and becomes precipitation in the form of rain, hail, sleet, or snow. The disposition of that precipitation as it reaches the earth's surface and eventually returns to the atmosphere completes the cycle.

The various processes during the hydrologic cycle that remove precipitation before it can become surface runoff are called abstractions. The relative magnitude of each of the following abstractions should be considered during stormwater management calculations.




*Reference: USDOT, FHWA, HEC-19 (1984).*

**FIGURE 2-1**
The Hydrologic Cycle

### 2.1.1 EVAPORATION

When air is in an unsaturated condition (i.e., has not reached its maximum capacity of moisture for the given temperature and pressure), water is returned to it through evaporation. This can occur both before and after precipitation reaches the ground and is a continuous process from standing waterbodies. Evaporation can usually be ignored, although it plays a major role in the long-term water balance of a watershed and may be important if long periods are required to percolate stormwater.

### 2.1.2 INFILTRATION

The flow of water into the ground is termed infiltration, and its disposition depends on the soil's field capacity, which is the moisture held by the soil after excess gravitational drainage occurs. If the moisture content is greater than field capacity, water percolates into the groundwater, from which it may move as seepage into streams and lakes or, in capillary movement, back to the root zone.

The process of infiltration is influenced by many factors, including soil type, vegetal cover, time elapsed between precipitation events, precipitation intensity, and temperature. Although infiltration is often an important element in determining the response of a watershed to a given rainfall event, there is as yet no universally acceptable model for predicting infiltration rates. Empirical methods discussed later in this chapter are often used in practice.

### 2.1.3 TRANSPIRATION

Transpiration is the process by which vegetation takes water from the soil and transmits it as vapor to the air through foliage. When the moisture content of the soil is less than the field capacity, transpiration can be a significant pathway for returning water to the atmosphere. As with evaporation, transpiration is only significant over time and can usually be ignored for stormwater management calculations.

### 2.1.4 INTERCEPTION

Interception is the process in which water falling as precipitation is trapped by objects above ground, such as buildings, trees, and vegetation, from which it subsequently evaporates. The quantity of water removed in this manner for a single storm is generally small.

### 2.1.5 DEPRESSION STORAGE

When water accumulates in puddles or depressions, with no possibility of release through overland flow or runoff, it is termed depression storage. The amount of water detained in depression storage varies greatly with land use, e.g., a paved surface has less storage than a furrowed field. The relative importance of depression storage to runoff rates for a given storm depends on the amount and intensity of precipitation.

### 2.1.6 DETENTION STORAGE

The storage incurred during a storm event by the volume of water forming the flow path that allows overland flow to enter drainage systems is called detention storage. The storage is temporary, since the water will continue to run off after precipitation ceases. The amount of water thus stored depends on land use, vegetal cover, slope, and rainfall intensity.

### 2.1.7 RUNOFF COMPONENTS

The following four main components of runoff are generally considered:

1. Overland flow
2. Channel precipitation
3. Interflow
4. Groundwater flow

Overland flow travels over the ground surface to the stream channel. Channel precipitation is direct rainfall on the water surface. Runoff that enters a stream channel by traveling laterally through the upper layer of soil is called interflow, while streamflow generated by the occurrence of water table conditions above the channel bottom is called groundwater or base flow.

The distinctions drawn between these four components are arbitrary. For convenience, a common practice is to divide the total surface runoff volume into two parts, the storm (or direct) runoff and base flow. Direct runoff includes overland flow, subsurface flow, and channel precipitation, while base flow consists of groundwater flow. Since base flow may not always be significant, a direct runoff hydrograph may be adequate for hydraulic calculations.

If base flow is a significant factor for a given watershed, an appropriate base flow component should be considered. Fundamentals of base flow separation are presented in books by Chow (1964), Linsley, Kohler, and Paulhus (1982), Viessman et al. (1977), and USDOT, FHWA (HEC-19, 1984).

## 2.2 PRECIPITATION

Precipitation occurs in a liquid state as rain, in a mixture of liquid and frozen water as sleet (melted snow), and frozen in crystalline (snow) or massive (hail) form. It results when air cools, causing moisture to condense. The most common cause of cooling is dynamic, or adiabatic, lifting of the air, in which a given parcel of air rises and cools, and moisture condenses into very small cloud droplets. If these droplets coalesce and become large enough to overcome air resistance, precipitation occurs.

### 2.2.1 TYPES

Precipitation can be classified by the origin of the lifting motion that induces it. Each type is characterized by different spatial and temporal rainfall regimes. The three major types of storms are:

1. Convective
2. Orographic
3. Cyclonic

A fourth classification often added is the hurricane or tropical cyclone, although it is basically a subtype of the cyclonic storm.

#### Convective Storms

Precipitation occurs during convective storms when warm, moist air rises from lower elevations into cooler overlying air, as shown in Figure 2-2. The characteristic form of convective precipitation is the summer thunderstorm. The surface of the earth, warmed considerably by afternoon sun, imparts heat to the adjacent air. The warmed air rises through cooler, overlying air, and if sufficient moisture content conditions are met, rapid condensation occurs. A single convective storm can produce extremely high rainfall rates in a short period of time, which is typically represented by the 8- to 60-minute portion of the appropriate rainfall intensity-duration-frequency (IDF) curves.

## Orographic Storms

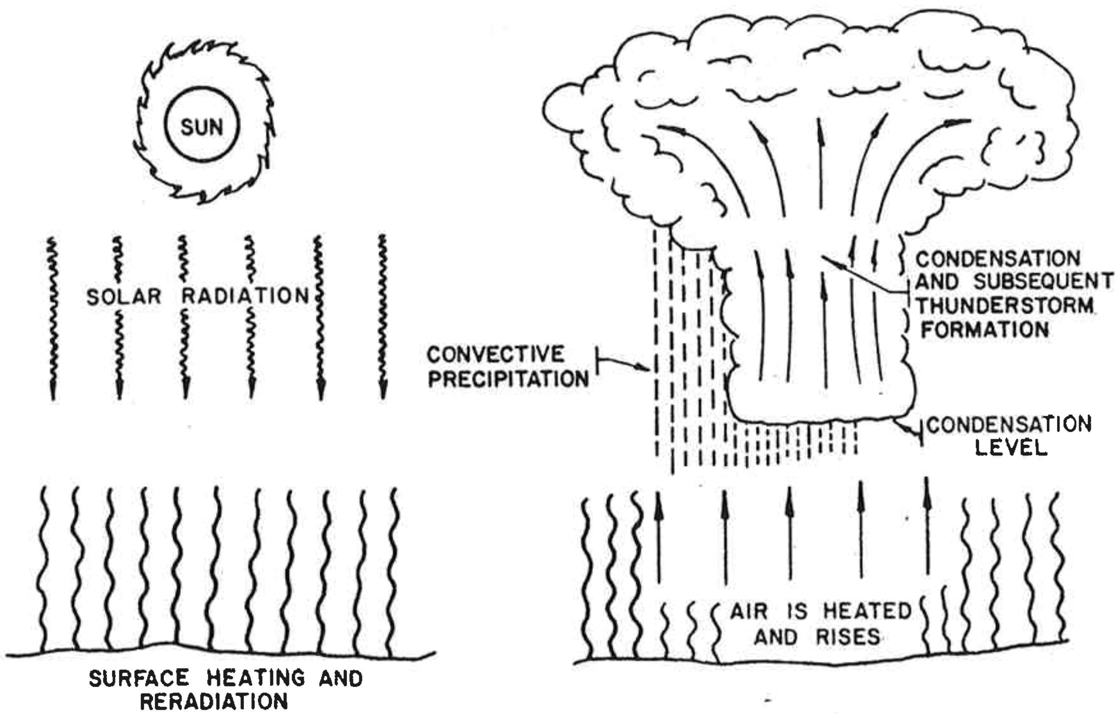
Orographic precipitation occurs when air condenses as it is forced to rise over a fixed-position geographic feature, such as is shown in Figure 2-3. Because of the relatively mild relief around Nashville and Davidson County, this type of storm is uncommon. When it does occur in more mountainous areas, however, it can generate high intensity rainfall.

## Cyclonic Storms

Cyclonic precipitation results when air rises as it moves from an area of high pressure toward an area of low pressure. In the middle latitudes, cyclonic storms generally move from west to east, and are sometimes called extra-tropical cyclones or continental storms. Continental storms occur at the boundary between air masses of significantly different temperatures. A disturbance in the boundary can grow, appearing as a wave as it travels from west to east along the boundary.

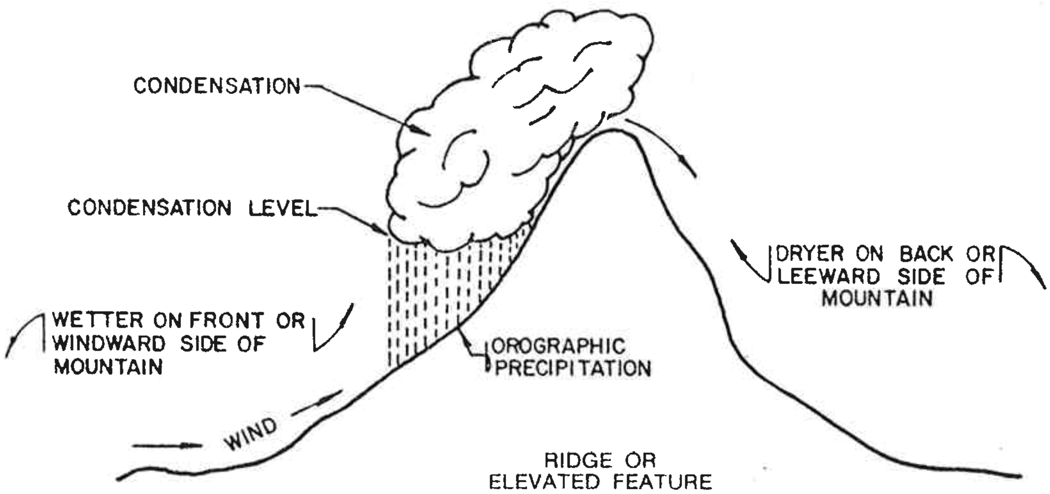
On a weather map, the cyclonic storm will generally appear as shown in Figure 2-4, with two boundaries or fronts. Warm air pushing into an area of cool air is called a warm front; when cold air is the aggressor, a cold front occurs (see Figure 2-5). The precipitation associated with a cold front is usually heavy and covers a relatively small area; warm front precipitation is more passive and lighter, but covers a much larger area. Tornadoes and other violent weather phenomena are associated with cold fronts.

Hurricanes. Hurricanes or tropical cyclones develop over tropical oceans where surface water temperatures are greater than 85°F. These storms can produce tremendous amounts of



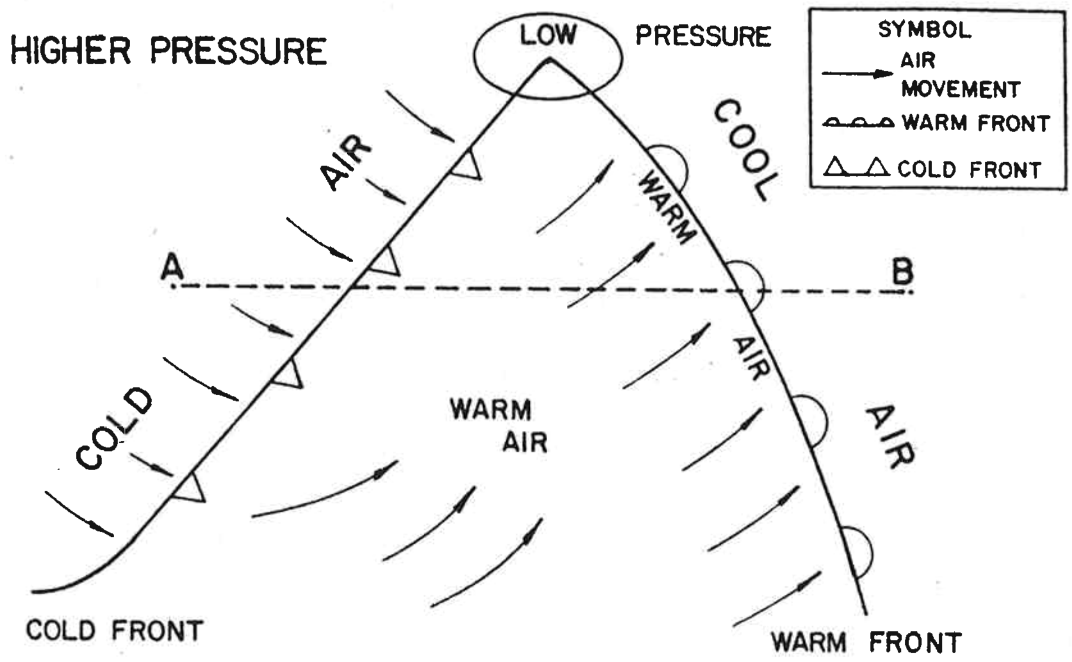

*Reference: USDOT, FHWA, HEC-19 (1984).*

**FIGURE 2-2**
Convective Storm



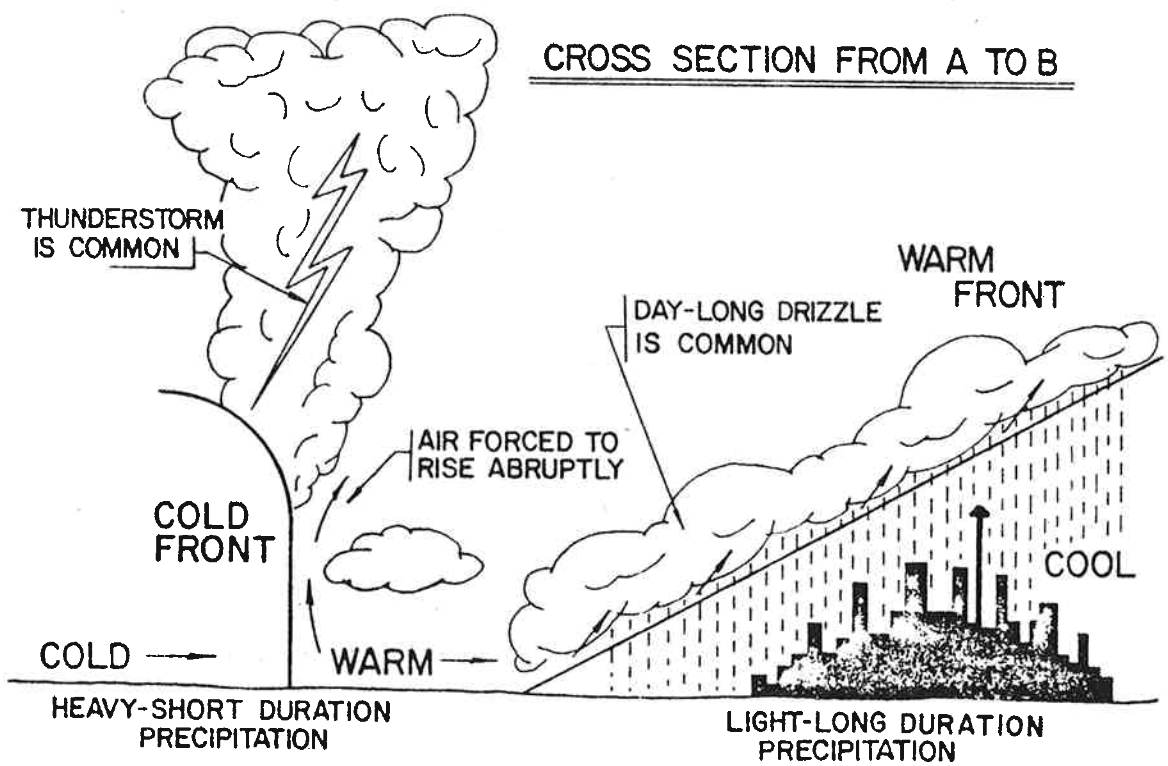

*Reference: USDOT, FHWA, HEC-19 (1984).*

**FIGURE 2-3**
Orographic Storm




*Reference: USDOT, FHWA, HEC-19 (1984).*

**FIGURE 2-4**
Cyclonic Storm




*Reference: USDOT, FHWA, HEC-19 (1984).*

**FIGURE 2-5**
Frontal Action of a Cyclonic Storm

precipitation in a relatively short time. Rainfall amounts of 15 to 20 inches in less than 24 hours are common in well developed tropical storms.

Hurricanes are very unlikely in the Metropolitan Nashville area, because hurricane forces decrease rapidly as they move across land. However, significant rainfall amounts can be expected from hurricane remnants as they pass nearby.

Northeaster. Extratropical storms that occur along the northern part of the east coast of the United States, accompanied by strong winds blowing from the northeast, are called northeasters. A typical northeaster consists of a single center of low pressure about which the winds revolve. Wind patterns are less symmetrical than those associated with hurricanes.

### 2.2.2 MEASUREMENT

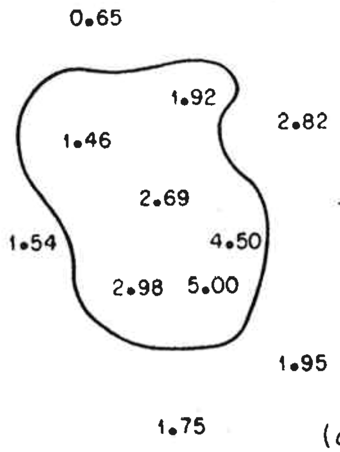
Precipitation can be measured using a variety of instruments and techniques, all of which are based on the vertical depth of water that would accumulate on a level surface if the precipitation remained where it fell.

#### Point Precipitation

Point precipitation is measured in either a recording or nonrecording rain gage. The standard non-recording gage used by the U.S. National Weather Service is an 8-inch diameter collector with a measuring tube that has a cross-sectional area one-tenth that of the collector. Recording gages in common use are the tipping bucket, the weighing gage, and the float gage. Recording gages provide information on timing, intensity, and the depth of rainfall.

#### Areal Precipitation

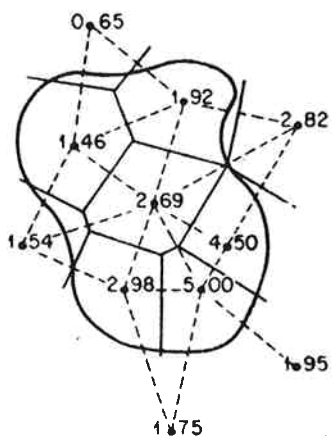
The spatial variability of precipitation can be evaluated by averaging the point measurements from several locations. Figure 2-6 provides a comparison of three averaging techniques, which are briefly described below. Point measurements can be adjusted directly to areal values by applying the areal reduction factors from Hershfield (1961) presented in Volume 2.




Arithmetic mean:

$$\frac{1.46 + 1.92 + 2.69 + 4.50 + 2.98 + 5.00}{6} = 3.09 \text{ in.}$$

(a)



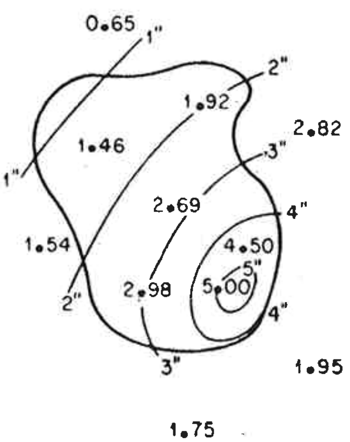

Thiessen method:

| Observed precip. (in.) | Area* (sq mi) | Per cent total area | Weighted precipitation (in.) (col.1 × col.3) |
|---|---|---|---|
| 0.65 | 7 | 1 | 0.01 |
| 1.46 | 120 | 19 | 0.28 |
| 1.92 | 109 | 18 | 0.35 |
| 2.69 | 120 | 19 | 0.51 |
| 1.54 | 20 | 3 | 0.05 |
| 2.98 | 92 | 15 | 0.45 |
| 5.00 | 82 | 13 | 0.65 |
| 4.50 | 76 | 12 | 0.54 |
| | 626 | 100 | 2.84 |

Average = 2.84 in.

*Area of corresponding polygon within basin boundary

(b)




Isohyetal method:

| Isohyet (in.) | Area* enclosed (sq mi) | Net area (sq mi) | Avg. precip. (in.) | Precipitation volume (col.3 × col.4) |
|---|---|---|---|---|
| 5 | 13 | 13 | 5.3 | 69 |
| 4 | 90 | 77 | 4.6 | 354 |
| 3 | 206 | 116 | 3.5 | 406 |
| 2 | 402 | 196 | 2.5 | 490 |
| 1 | 595 | 193 | 1.5 | 290 |
| <1 | 626 | 31 | 0.8 | 25 |
| | | | | 1634 |

Average = 1634 ÷ 626 = 2.61 in.

*Within basin boundary

(c)

*Reference: Linsley, Kohler, and Paulhus (1982).*

**FIGURE 2-6**
Areal Averaging of Precipitation by (a) Arithmetic Method, (b) Thiessen Method, and (c) Isohyetal Method

Arithmetic Average. This simple method adds the observed depths for each point precipitation measurement within a watershed and divides by the number of gages. The results are satisfactory if the gages are uniformly distributed and individual measurements do not vary widely from the average. Part A of Figure 2-6 illustrates this technique.

Thiessen Polygon. A weighting factor is developed for each point precipitation measurement. A perpendicular bisector is drawn for each line that connects gage locations. The effective area for each gage is then determined by the polygon bracketing each gage location. Multiplying the precipitation depth measured at each gage by the respective effective areas, expressed as a percentage of the total area, provides the weighted average precipitation for each gage. These weighted depths are then summed to give the precipitation depth for the total area. This technique is illustrated in Part B of Figure 2-6.

Isohyetal Method. Contours of equal precipitation are drawn to reflect the spatial variability observed over the area. The area between contours is multiplied by the average precipitation over that contour interval to obtain a weighted precipitation value. The values are then summed and divided by the total area to give the precipitation depth for the total area. Part C of Figure 2-6 illustrates this method.

### 2.2.3 CHARACTERISTICS

The following characteristics of precipitation are typically considered in drainage calculations:

1. Intensity (rate of precipitation)
2. Duration
3. Time distribution of rainfall (hyetograph)
4. Storm shape, size, and movement
5. Frequency

Brief descriptions of these characteristics are given below.

#### Intensity

Intensity is the rate of precipitation, commonly given in units of inches per hour. In any given storm, the instantaneous intensity is the slope of the mass rainfall curve at a particular time. For stormwater management, it is desirable to divide the storm into convenient time increments and to determine the average intensity over each of the selected periods. The results are plotted as rainfall hyetographs.

For drainage calculations, intensity is perhaps the most important of the rainfall characteristics, since discharge from a given watershed will increase as the intensity rises. Intensity varies from misting conditions, in which only a trace (<0.005 inch) of precipitation may fall, to cloudbursts, during which several inches per hour are common.

## Duration

The duration of observed precipitation events is related to selection of a minimum interevent period during which no rainfall occurs. Statistical procedures discussed in textbooks by Benjamin and Cornell (1970) or Haan (1977) can provide a basis for selecting an appropriate minimum interevent time when observed data are evaluated.

## Time Distribution

A hyetograph provides a distribution of incremental precipitation versus time (see Figure 2-7). In practice, depth-duration data for a particular design frequency are generally used to develop a synthetic design storm event. Theoretically, it is more accurate to use actual precipitation data and continuous real time simulation rather than a single design storm event. Current computer technology makes this approach possible (McPherson, 1978, or Marsalek, 1978), but it is often infeasible because of cost and data limitations.

Because depth-duration data do not represent a logical, chronological sequence of precipitation during a storm, the precipitation depth increments must be rearranged into a sequence that might actually occur. A balanced storm procedure can accomplish this rearrangement or, if a 24-hour duration storm is desired, a Soil Conservation Service (SCS) standard distribution could be used. The 24-hour rainfall hyetograph presented in Volume 2 should be used for Nashville and Davidson County.

A synthetic design storm developed using either the balanced storm or an SCS distribution will generally be more extreme

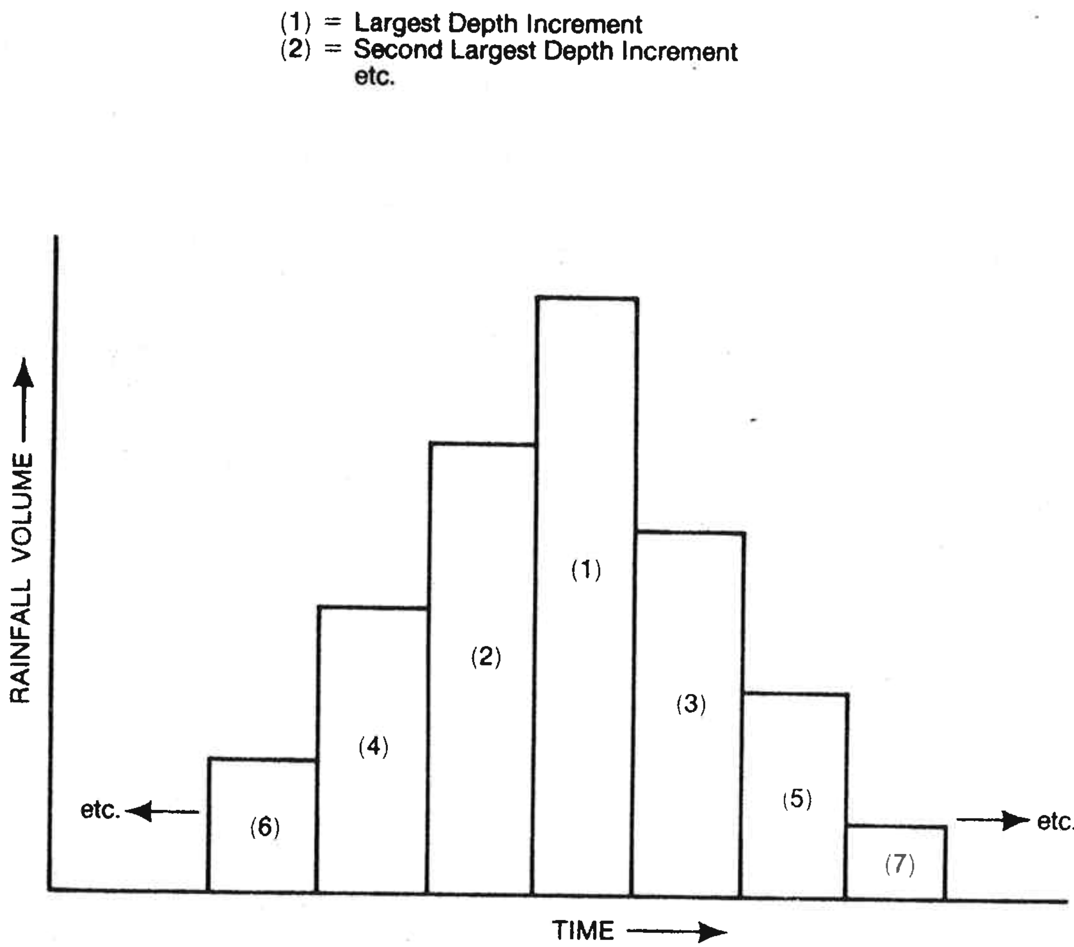



**FIGURE 2-7**
Balanced Storm Approach for Developing a Design Storm Hyetograph

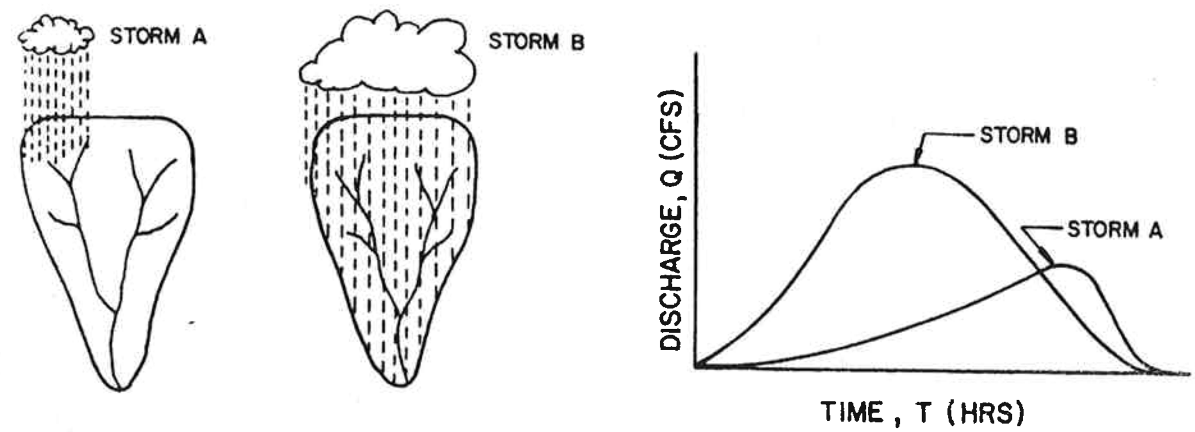
than most actual storms of the same total depth and duration, because the short-duration events are assumed to be nested within the long-duration storm. This is accepted practice, because it ensures that critical events for the small headwater areas of a watershed will not be overlooked. In addition, if the design storm duration is long enough, the critical duration for sizing a storage basin is ensured.

Balanced Storm. The balanced storm approach provides a precipitation hyetograph that has the same return period for each time interval within the total storm duration. This approach places the largest increment of rainfall for the selected time step at the midpoint of the total storm duration. For example, the midpoint for a 24-hour storm is 12 hours. Smaller depth increments are arranged symmetrically about the largest value, with the second largest value placed before and the third largest value placed after the largest. This "before and after" process continues until the entire hyetograph is developed. Figure 2-7 depicts the balanced storm approach for developing a design storm hyetograph.

SCS Distributions. Hyetographs developed by the SCS using published data were demonstrated through experience to have similar characteristics. For areas subject to both short-duration summer thunderstorms and long-duration frontal storms, the rainfall distribution versus time is termed a Type II distribution by the SCS. For areas in which intense short-duration storms are not prevalent, the rainfall distribution versus time is termed a Type I distribution.

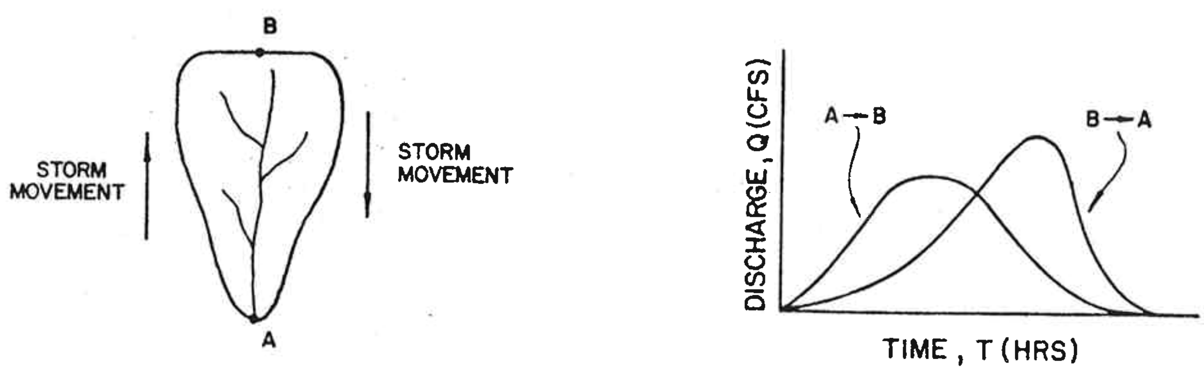
## Shape, Size, and Movement

Storm shape, size, and movement are normally determined by the type of storm. For example, storms associated with cold fronts tend to be more localized, faster moving, and of shorter duration than storms produced by warm fronts. All three factors determine the areal extent of precipitation and the size of the drainage area that contributes over time to the surface runoff. As illustrated in Figure 2-8, a small localized storm of a given intensity and duration over just a part of the drainage area will result in much less flow than if the same storm covered the entire watershed.




*Reference: USDOT, FHWA, HEC-19 (1984).*

**FIGURE 2-8**
Effect of Storm Size on Surface Runoff




*Reference: USDOT, FHWA, HEC-19 (1984).*

**FIGURE 2-9**
Effect of Storm Movement on Surface Runoff

The location of a localized storm in the drainage basin also affects the time distribution of the surface runoff. A storm near the outlet of the watershed will cause peak flow to occur very quickly and the flood to pass rapidly. If the same storm occurred in a remote part of the basin, the runoff at the outlet would last longer and the peak flow would be lowered by storage in the channel.

Storm movement has a similar effect on the runoff distribution, particularly if the basin is long and narrow. Figure 2-9 shows that a storm moving up a basin from its outlet gives a distribution of runoff that is relatively symmetrical with respect to the peak flow. The same storm moving down the basin will usually result in a higher peak flow and an unsymmetrical distribution, with the peak flow occurring later. This phenomenon is more critical for large watersheds than for small ones.

Frequency

Frequency quantifies the likelihood of the recurrence of precipitation with a given duration and average intensity. The frequency of occurrence for the surface runoff resulting from precipitation is of primary concern and, in particular, the frequency of the peak discharge. Although the assumption that a given frequency storm always produces a flood of the same frequency should be used with caution, there are a number of analytical techniques based on it, particularly for ungaged watersheds.

## 2.3 RAINFALL EXCESS

After precipitation is reduced by the various abstraction processes, the amount which is left is often called rainfall excess. In practice, rainfall excess is usually estimated using empirical methods such as a runoff coefficient or SCS curve number. Theoretical fundamentals for evaluating rainfall excess, which are not presented in this manual, may be obtained from USDOT, FHWA (HEC-19, 1984), Chow (1964), Linsley, Kohler, and Paulhus (1982), Eagleson (1970), and Viessman et al. (1977).

### 2.3.1 RUNOFF COEFFICIENT

The calculation of rainfall excess as a fixed percentage of precipitation is accomplished using a parameter called the runoff coefficient. Because the runoff coefficient must account for interception, surface storage, and infiltration, the suitability of this empirical approach decreases as the complexity of the watershed increases.

Selection of a runoff coefficient should include a consideration of soil type, land use, antecedent moisture conditions, precipitation duration, and intensity. The tables presented in Volume 2 facilitate the selection process, but their use should be based on sound engineering judgment. A comparison of the actual performance of drainage facilities to design calculations can provide a basis for making judgment decisions. In general, the best use of a runoff coefficient is for homogeneous watersheds with high percentages of impervious area and under circumstances when the areal distribution of rainfall can be assumed to be relatively uniform.

### 2.3.2 SCS RELATIONSHIP

The SCS has developed an empirical relationship for estimating rainfall excess that accounts for infiltration losses and initial abstractions by using a site-specific runoff parameter called the curve number (CN). The watershed CN is a dimensionless coefficient that reflects watershed cover conditions, hydrologic soil group, land uses, and antecedent moisture conditions. Procedures for determining curve numbers are discussed in Volume 2.

The maximum soil storage and a CN value for a watershed can be related by the following expression:

$$S = \frac{1,000}{CN} - 10 \qquad (2\text{-}1)$$

where:

S = Maximum soil storage, in inches

CN = Watershed curve number, dimensionless

When the maximum soil storage is known, the rainfall excess can be calculated using the following SCS relationship:

$$R = \frac{(P - 0.2S)^2}{P + 0.8S} \qquad (2\text{-}2)$$

where:

R = Accumulated rainfall excess (or runoff), in inches

P = Accumulated rainfall, in inches

S = Maximum soil storage, in inches

Three levels of antecedent moisture conditions are considered by the SCS relationship. Antecedent Moisture Condition I (AMC-I) is the lower limit of antecedent rainfall or the upper limit of the maximum soil storage, S. Antecedent Moisture Condition II (AMC-II) represents average antecedent rainfall conditions, and Antecedent Moisture Condition III (AMC-III) is the upper limit of antecedent rainfall or the lower limit of S. For design purposes, AMC-II is generally selected.

Additional information on the SCS relationship can be found in USDA, SCS publications TP-149(1973) and NEH-4(1972).

## 2.4 STATISTICAL ANALYSIS

Statistical analysis is a systematic way of quantifying data for purposes of generalization. Methods of statistical analysis can also be used to predict future events, based on the characteristics of past data.

Statistical analysis is based on the concepts of populations and samples. A population is defined as the entire collection of all possible occurrences of a given quantity. It may be either finite or infinite. A sample is part of a population and should have the same characteristics, or be representative, of that population.

To facilitate a typical statistical analysis, such as the probability of a certain flood event from a sample of infinite population, the concept of frequency distributions is used. A frequency distribution is an arrangement of data by classes or categories, with associated frequencies for each class. The frequency distribution is used to determine the magnitude of past events, as well as how often events of a specified magnitude have occurred.

A frequency distribution is constructed by first examining the range of magnitudes, i.e., the difference between the largest and the smallest floods, and dividing this range into a number of conveniently sized groups, usually between 10 and 20. These groups are called class intervals. The size of the class interval is the range divided by the number of class intervals selected. The following guidelines may be helpful in selecting the number of class intervals:

1. The class intervals should not overlap; for example, use 0-99, 100-199, rather than 0-100, 100-200.

2. The number of class intervals that have no events should be limited.

3. The class intervals should be of uniform size.

Guidelines for determining floodflow frequencies from observed streamflow data may be obtained from Bulletin 17B of the U.S. Water Resources Council (revised 1981). A literature review and evaluation of urban floodflow frequency procedures is presented by Rawls et al. (1980). The U.S. Geological Survey (USGS) is the primary government agency responsible for collection of streamflow data and maintenance of systematic peak discharge information. These data are reported in USGS Water Supply Papers, Annual Surface Water Records, and computer files.

### 2.4.1 TYPES OF DATA SERIES

The following types of sample data are most common:

1. Complete duration series
2. Annual series
3. Partial duration series
4. Extreme value series

The complete duration series includes all available data; the annual series consists of one value per year (typically the maximum peak flow). The partial duration series includes all peak flows above a selected base value. The extreme value series consists of the largest observation in a given time interval. Note that the annual series is a special type of extreme value series, with a time interval of 1 year.

The annual and partial duration series give similar results for return periods beyond 10 years. For return periods less than 10 years, the annual series gives higher return periods for the same peak flow rates. A comparison of the frequency curves developed using the annual and partial duration series is presented in Figure 2-10.

### 2.4.2 PLOTTING POSITION

Assigning a frequency to a data point is commonly referred to as determining the plotting position. This determination for sample data is never conclusive, as there is no assurance that a sample contains the smallest and largest values of the total population.

Several plotting position relationships are discussed by Chow (1964) and general criteria for plotting position relationships are presented by Gumbel (1958). In a comparative study of plotting position relationships, Benson (1962) found that the Weibull relationship provided estimates that were consistent with experience. The Weibull relationship is expressed as:

$$P = \frac{m}{n + 1} \tag{2-3}$$

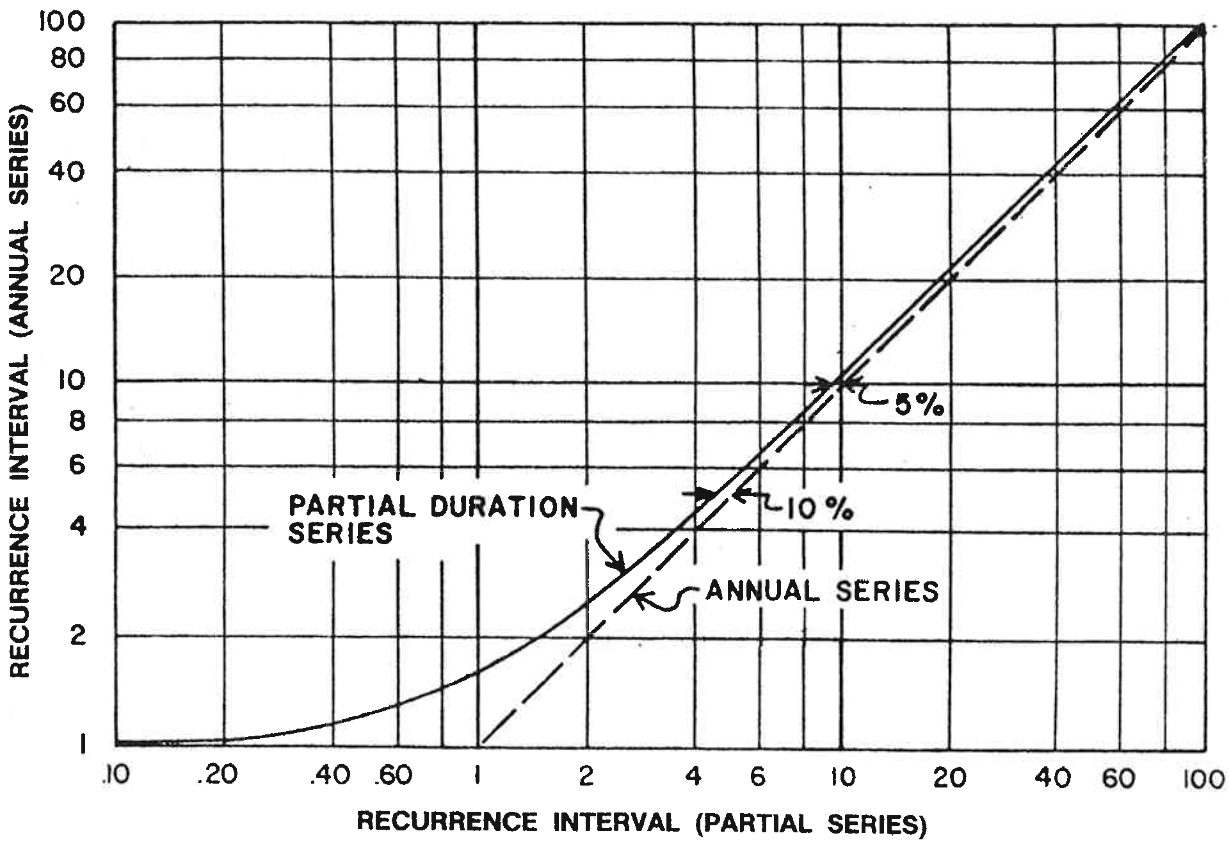
where:

P = Plotting position

m = Order number of the event from highest to lowest

n = Number of years of record

For partial duration series in which the number of floods exceeds the number of years of record, Beard (1962) recommends the expression:




*Reference: USDOT, FHWA, HEC-19 (1984).*

**FIGURE 2-10**
Relationship Between Annual and Partial Duration Series

$$P = \frac{2m - 1}{2n} \qquad (2\text{-}4)$$

where:

P = Plotting position

m = Order number of the event from highest to lowest

n = Number of years of record

## 2.4.3 STANDARD DISTRIBUTIONS

Several frequency distributions are commonly applied to the analysis of streamflow data and, as a result, have been extensively studied and standardized. The following standard frequency distributions are most useful:

1. Normal distribution
2. Log-normal distribution
3. Gumbel extreme value distribution
4. Log-Pearson Type III distribution

Mathematical fundamentals for these standard frequency distributions can be found in publications by Haan (1977), Riggs (1968a and b), Benjamin and Cornell (1970), and USDOT, FHWA (HEC-19, 1984).

### Normal Distribution

The normal, or Gaussian, distribution is a classical mathematical distribution occurring in the analysis of natural phenomena. The normal distribution is a symmetrical, unbounded, bell-shaped curve with the maximum value at the central point and extending from $-\infty$ to $+\infty$.

For the normal distribution, the maximum central value occurs at the mean flow. Because of absolute symmetry, half of the flows are below the mean and half are above. The median, therefore, corresponds to the mean value. Another characteristic of the normal distribution curve is that 68.3 percent of the events will fall between ± one standard deviation, 95 percent will fall within ± two standard deviations, and 99.7 percent will fall within ± three standard deviations.

A disadvantage of the normal distribution is that it is unbounded in the negative direction, whereas most hydrologic variables are bounded and can never be less than zero. For this reason, and because many hydrologic variables exhibit a pronounced skew, the normal distribution usually has limited application. Because the logarithms of hydrologic variables are often normally distributed, a log transformation of the data can sometimes overcome this limitation.

## Log-Normal Distribution

The log-normal distribution has the same characteristics as the normal distribution except that the independent variable is replaced with its logarithm. The log-normal distribution is bounded on the left by zero and has a pronounced positive skew. These characteristics are common to many frequency distributions obtained from analysis of hydrologic data.

## Gumbel Extreme Value Distribution

The Gumbel extreme value distribution, sometimes called the double exponential distribution of extreme values, can also be used to describe the distribution of hydrologic variables, especially peak discharges. The characteristics of the Gumbel extreme value distribution are occurrence of mean flow at a return period of 2.33 years and a positive skew, i.e., toward the high flows or extreme values.

## Log-Pearson Type III Distribution

Another distribution that has found wide application in hydrologic analysis is the log-Pearson Type III distribution, which is a three-parameter gamma distribution with a logarithmic transform of the independent variable. It is one of a number of standard distributions that have been developed, more or less empirically, for application to statistical problems. Its advantages are its ability to conform with available data and its flexibility, which allows its use in a variety of distributions. This flexibility has led the U.S. Water Resources Council to recommend its use as the standard distribution for flood frequency studies by all U.S. government agencies.

Mean flow, standard deviation, and coefficient of skew are necessary to describe the log-Pearson Type III distribution. Judicious selection of these three parameters makes it possible to fit almost any shape of distribution. An extensive description of the use of this distribution in determining flood frequency distributions is presented in Bulletin 17B by the U.S. Water Resources Council (revised 1981).

### 2.4.4 STANDARD ERROR AND CONFIDENCE LIMITS

Since more than one distribution may fit given streamflow data, quantitative measures should be used to decide which distribution is most suitable. Two of the most common measures are the standard error of estimate and confidence limits. Mathematical relationships for these parameters can be obtained in publications by Haan (1977), Benjamin and Cornell (1970), or USDOT, FHWA (HEC-19, 1984).

#### Standard Error of Estimate

The standard error of estimate is inversely proportional to the square root of the period of record; that is, the shorter the period of record, the larger the standard error. For example, standard errors for a short record will be approximately twice as large as those for a record four times as long.

#### Confidence Limits

Confidence limits are used to estimate the uncertainties associated with the determination of floods of specified return periods from frequency distributions. Since a given frequency distribution is only an estimated determinant from one sample, it is probable that another sample of equal length but taken at a different time from the same stream would yield a different frequency curve. Confidence limits, or more correctly, confidence intervals, define the range within which these frequency curves could be expected to fall, with specified confidence or levels of significance (see U.S. Water Resources Council, revised 1981).

## 2.5 TERMINOLOGY

### 2.5.1 HYDROGRAPHS

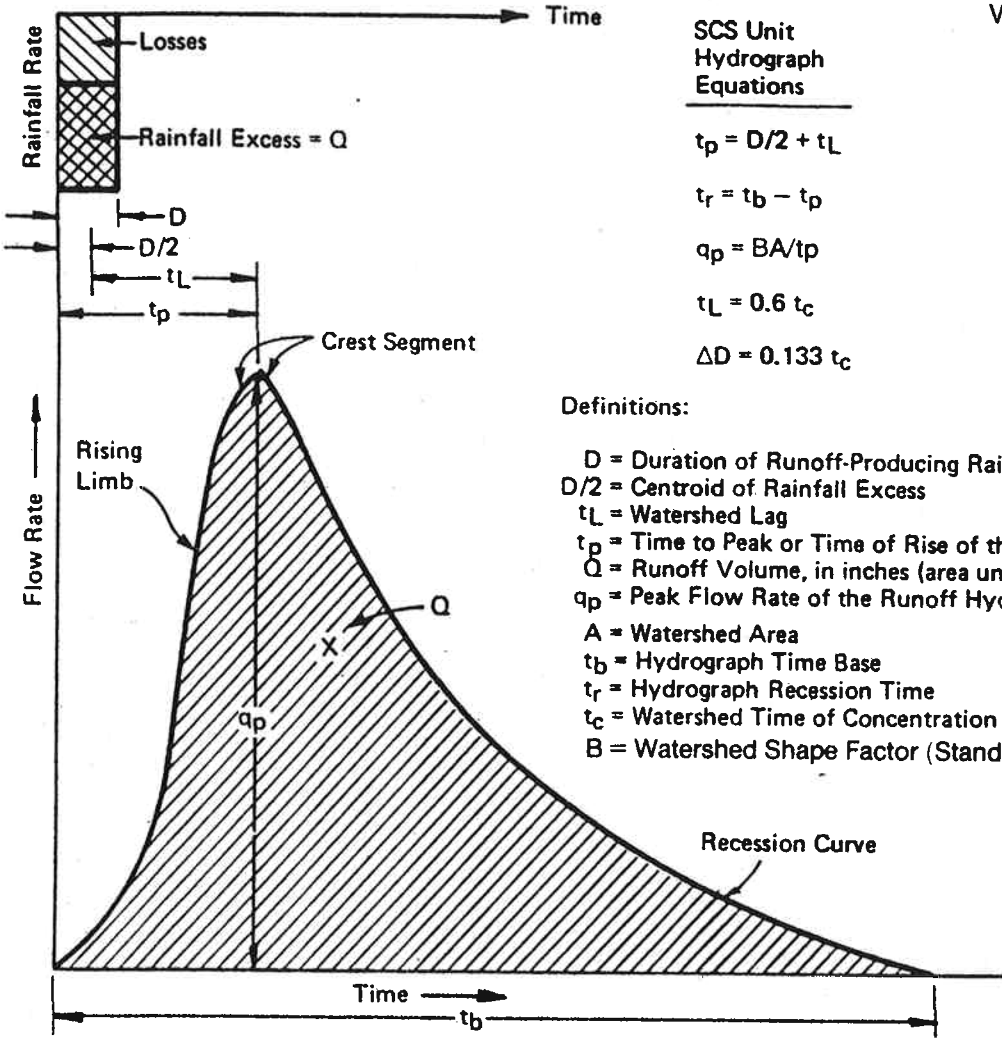
A flood hydrograph is a continuous plot of the surface runoff flow rate versus time. It is produced when a particular watershed is subjected to an input of rainfall excess. By definition, the volume of water contained in the flood hydrograph is equal to the volume of water contained in the rainfall excess hyetograph. Although historical streamflow data are preferred, they are not generally collected for the typical watershed; thus, synthetic methods for developing flood hydrographs are often required. If historical data are not available for a given basin, then data from a similar basin may be appropriate.

A typical hydrograph resulting from an isolated period of rainfall consists of a rising limb, a crest segment, and a falling or recession limb. These components are illustrated in Figure 2-11. The shape of the rising limb is influenced primarily by the characteristics that produced the surface runoff. The point of inflection on the recession limb of the hydrograph is commonly assumed to mark the time at which surface inflow to the channel system ceases (see Figure 2-12). Thereafter, the recession curve represents the withdrawal of water from storage within the watershed. As a result, the recession limb is largely independent of the storm and is influenced instead by watershed characteristics, such as channel slope and storage availability.

Typical hydrograph components are presented in Figure 2-11, along with appropriate SCS hydrograph equations. A rainfall excess hyetograph, which in this case is a single block of rainfall excess over duration, D, is shown in the upper part of Figure 2-11. The runoff hydrograph is presented directly below the rainfall excess hyetograph. The area enclosed by the hyetograph and by the runoff hydrograph represents the same volume, Q, of direct runoff. The maximum flow rate of the hydrograph is the peak flow, $q_p$.

### 2.5.2 TIME PARAMETERS

The time from the start of the hydrograph to $q_p$ is the time to peak, $t_p$. The total time duration of the hydrograph is known as the time base, $t_b$. The watershed lag time, $t_L$, is




**SCS Unit Hydrograph Equations**

$t_p = D/2 + t_L$

$t_r = t_b - t_p$

$q_p = BA/tp$

$t_L = 0.6\ t_c$

$\Delta D = 0.133\ t_c$

Definitions:

- $D$ = Duration of Runoff-Producing Rainfall in Hours
- $D/2$ = Centroid of Rainfall Excess
- $t_L$ = Watershed Lag
- $t_p$ = Time to Peak or Time of Rise of the Runoff Hydrograph
- $Q$ = Runoff Volume, in inches (area under the curve)
- $q_p$ = Peak Flow Rate of the Runoff Hydrrograph in cfs
- $A$ = Watershed Area
- $t_b$ = Hydrograph Time Base
- $t_r$ = Hydrograph Recession Time
- $t_c$ = Watershed Time of Concentration
- $B$ = Watershed Shape Factor (Standard SCS B = 484)

**FIGURE 2-11**
General Hydrograph Terminology

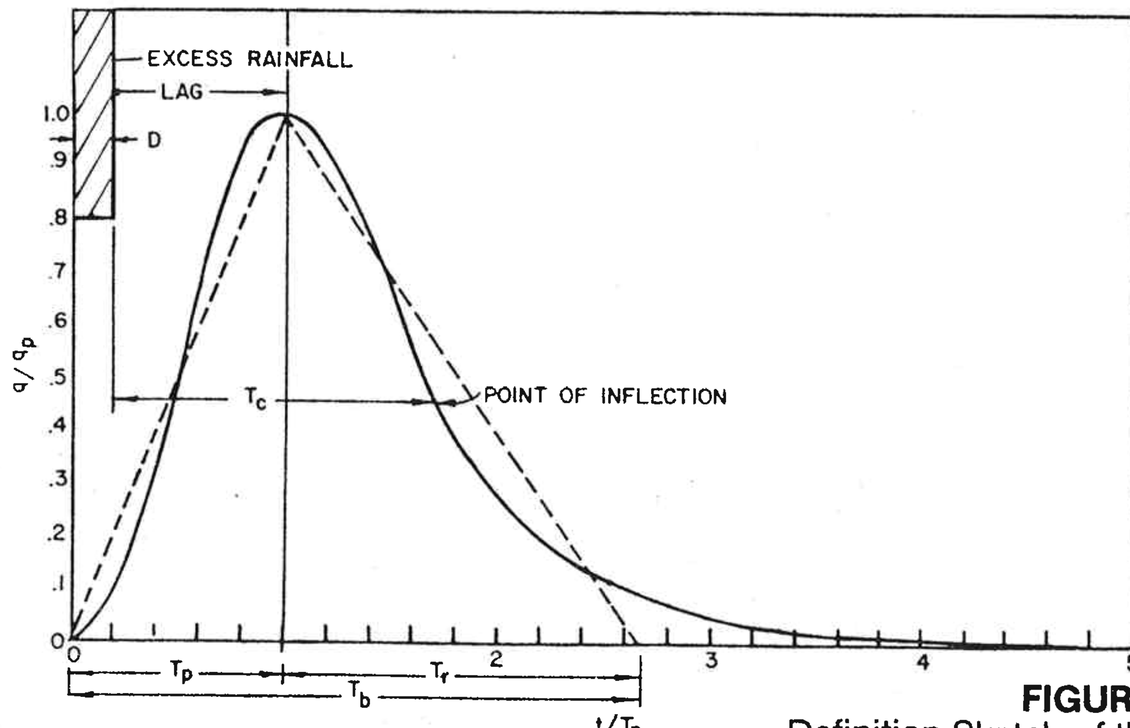



**FIGURE 2-12**
Definition Sketch of the SCS Dimensionless Unit Hydrographs

*Reference: USDA, SCS, NEH-4 (1972).*

defined as the time from the center of mass of the rainfall excess to the runoff hydrograph peak. The following equation summarizes the relationship between the time parameters for the rising limb of a direct runoff hydrograph:

$$t_p = \frac{D}{2} + t_L \qquad (2\text{-}5)$$

where:

$t_p$ = Time to peak or time of rise of the runoff hydrograph

D = Duration of runoff-producing rainfall

$t_L$ = Watershed lag time

The recession time for a hydrograph is the difference between the time base and the time to peak and can be expressed as:

$$t_r = t_b - t_p \qquad (2\text{-}6)$$

where:

$t_r$ = Hydrograph recession time

$t_b$ = Hydrograph time base

$t_p$ = Time to peak or time of rise of the runoff hydrograph

The time of concentration, $t_c$, is the time required for a hydraulic wave to travel across the watershed. It is often approximated as the time required for runoff to travel from the hydraulically most remote part of the watershed to the point of reference.

Many empirical relationships are available for estimating the value of $t_c$ from site-specific watershed data. Since these relationships can be developed to estimate different components of the total watershed time of concentration, an understanding of the conditions for which they are applicable is essential before putting them to use. Empirical methods should not be used unless it can be demonstrated that they are suitable for actual site conditions.

A segmental approach, commonly known as the velocity method, typically requires evaluation of the following three flow components:

1. Overland flow
2. Rill, shallow channel, and street gutter flow
3. Open channel flow

Overland flow is sheet flow over plane surfaces, usually limited to a maximum length of 300 feet. After 300 feet, overland flow generally becomes concentrated into rills, small channels, or gutters. Open channel flow is appropriate when the main conveyance system is encountered.

The travel time for any flow segment can be estimated as the ratio of flow length to the velocity of flow. The number of flow segments or flow components considered should best represent the actual flow path of the watershed being evaluated. The relationship for calculating the time of concentration is expressed as:

$$t_c = t_1 + t_2 + t_3 + \ldots t_i \qquad (2\text{-}7)$$

where:

$t_c$ = Watershed time of concentration

$t_1$ = Overland flow travel time

$t_2$ = Rill, shallow channel, and street gutter flow travel time

$t_3$ = Open channel flow travel time

$t_i$ = Travel time for the $i^{\underline{th}}$ segment

Procedures for estimating values for $t_1$, $t_2$, and $t_3$ are presented in Volume 2. In general, the Manning kinematic wave equation (Ragan, 1971) should be used for overland flow. Data presented in SCS TR-55 (1984) or Manning's Equation can be used to evaluate rill, shallow channel, and street gutter flow travel times. Manning's Equation is generally used to evaluate the open channel flow travel time for the main conveyance system.

## 2.6 PEAK RUNOFF RATES

Peak runoff rates are automatically obtained when a flood hydrograph is developed. In some situations, however, a peak runoff rate can be obtained without developing a complete flood hydrograph, through use of the Rational Method, regression equations, or USDA, SCS, TR-55 (1986).

### 2.6.1 RATIONAL METHOD

According to the Rational Method, the peak runoff rate can be estimated as the product of a runoff coefficient, a rainfall intensity, and the drainage area. The Rational Method is frequently expressed mathematically as:

$$Q = CIA \qquad (2-8)$$

where:

$Q$ = Peak runoff rate, in cubic feet per second (cfs)

$C$ = Rational Method runoff coefficient, dimensionless

$I$ = Average rainfall intensity for the design storm, in inches/hour

$A$ = Watershed drainage area, in acres

To aid in the analysis of parameters used in the Rational Method, the following form is used in this manual:

$$Q_T = C_T \, I_{t_c} \, A \qquad (2-9)$$

where:

$Q_T$ = Peak runoff rate for return period T, in cfs

$C_T$ = Rational Method runoff coefficient, expressed as the dimensionless ratio of rainfall excess to total rainfall, for the design storm return period T

$I_{t_c}$ = Average rainfall intensity, in inches/hour, during a period of time equal to $t_c$ for the design storm return period T (i.e., $I_{t_c}$ is a function of $t_c$)

A = Watershed drainage area, in acres, tributary to the design point

$t_c$ = Watershed time of concentration, in minutes, defined as the time required for a wave to travel from the hydraulically most distant point of a watershed to the design point

To be dimensionally correct, a conversion factor of 1.008 should be used to convert acre-inches per hour to cfs. In practice, this factor is usually neglected.

The basic assumptions behind the Rational Method include the following made by Rossmiller (1980):

1. Runoff is linearly related to rainfall.

2. The rainfall occurs uniformly over a given watershed.

3. The peak runoff rate occurs when the entire area is contributing flow.

4. The rainfall excess hyetograph is one of constant intensity for a duration equal to $t_c$.

5. The frequency of the peak runoff rate is the same as the frequency of the average rainfall intensity.

Because these assumptions are generally valid for only small watersheds (less than 200 acres), extra effort is warranted if larger areas are concerned. Comparison of results from the Rational Method to results from other procedures and historical observations is often appropriate.

### 2.6.2 REGRESSION EQUATIONS

A regression equation is a regional method that correlates peak discharge with physical factors such as watershed area and stream slope. The development of regional regression equations requires considerable field data and a major work effort. Streamflow data must be subjected to a statistical analysis (as discussed in this chapter); then the discharge-frequency curves must be correlated with watershed characteristics. Procedures for developing regression equations are presented in publications by Haan (1977) and Benjamin and Cornell (1970). Regression equations that have been developed for use in Metropolitan Nashville and Davidson County are presented in Volume 2.

### 2.6.3 SCS TR-55

The SCS has prepared TR-55 (1986) to provide simplified procedures for developing peak runoff rates in urban areas. These procedures are largely based on results of hydrologic calculations obtained using the SCS computer program TR-20 (1978). The methods provided in TR-55 are intended to be used only for overland and open channel flow conditions. If storm sewer systems are involved, TR-55 is not valid.

## 2.7 UNIT HYDROGRAPH THEORY

The concept of a unit hydrograph published by L. K. Sherman in 1932 provides a widely accepted basis for converting rainfall excess from a watershed to a runoff hydrograph. Although the tools and data available for developing unit hydrographs are more extensive than when Sherman first proposed unit hydrograph theory, the concept has not changed.

A unit hydrograph is defined as a runoff hydrograph that is produced by 1 inch of rainfall excess distributed uniformly over a watershed and occurring at a uniform rate during a specified period of time. Sherman originally used "unit" to denote the specified duration of rainfall excess for a particular unit hydrograph. The word "unit" is often misinterpreted as 1 inch or a "unit depth" of effective rainfall excess rather than as a "unit of time" for rainfall excess as originally intended.

The following assumptions constitute the basis of unit hydrograph theory:

1. The rainfall excess is uniformly distributed within its unit duration or specified period of time.

2. The rainfall excess is uniformly distributed in space over a particular watershed.

3. The time base for a direct runoff hydrograph due to a rainfall excess of unit duration is constant.

4. The ordinates of the direct runoff hydrographs, when a common unit duration is considered, are directly proportional to the total volume of direct runoff represented by each hydrograph (principle of linearity or superposition).

5. For a given drainage basin, the hydrograph of runoff caused by a unit duration and volume of rainfall excess is invariable (principle of time invariance).

These assumptions cannot be precisely applied to natural precipitation and drainage basin characteristics. However, experience has shown that the unit hydrograph method gives results that are sufficiently accurate for most drainage problems.

The two fundamental assumptions that must always be considered when applying unit hydrograph theory are the principle of linearity (Assumption 4) and the principle of time invariance (Assumption 5). Theoretically, each increment of rainfall excess can be routed through the subject watershed in accordance with the principle of linearity. In practice, this means that the product of a rainfall excess volume and the sequence of unit hydrograph ordinates (i.e., runoff rates in cfs per inch of rainfall excess) produces an estimate of the runoff hydrograph for that volume of rainfall excess. In addition, the principle of linearity allows individual runoff hydrographs developed from a sequence of individual rainfall excess volumes (i.e., a design storm of rainfall excess increments arranged in units of time equal to the unit duration) to be superimposed and added when estimating a total runoff hydrograph. The principle of time invariance requires that the hydrologic characteristics of the drainage basin be fixed or specified for a particular unit hydrograph. Land development and channel improvements are typical activities that violate the principal of time invariance.

The unit hydrograph method was originally devised for large drainage basins. However, Brater (1940) showed that unit hydrograph theory was also applicable to small drainage basins varying in size from 4 acres to 10 square miles. Dooge (1973) provides a good discussion of unit hydrograph theory, including the definition and application of the instantaneous unit hydrograph.

Unit hydrographs can be developed either from observed precipitation and streamflow records or from synthetic unit hydrograph procedures using site-specific watershed characteristics. Theoretical aspects of several procedures are presented below.

### 2.7.1 DERIVATION FROM OBSERVED DATA

If available, observed precipitation and streamflow data should be used to derive a unit hydrograph. Streamflow data for a reasonably uniform intensity storm of the desired duration with a relatively large runoff volume are best suited for this derivation. Base flow, if significant, must be separated from direct runoff. Additional details on the derivation of a unit hydrograph from observed streamflow data can be found in Linsley, Kohler, and Paulhus (1982), Viessman et al. (1977), Wisler and Brater (1959), Chow (1964), Dooge (1973), and USDOT, FHWA (HEC-19, 1984).

### 2.7.2 SCS DIMENSIONLESS UNIT HYDROGRAPH

The SCS has derived two dimensionless unit hydrographs from a large number of observed unit hydrographs for watersheds of various sizes and geographic locations. One type has a curvilinear shape; the second is a triangular approximation to that curvilinear shape. In both cases, once the time to peak and peak flow for a particular unit hydrograph have been defined, the entire shape of the unit hydrograph can be estimated with the appropriate dimensionless unit hydrograph.

Figure 2-12 presents the SCS dimensionless curvilinear unit hydrograph, which has 37.5 percent of its total volume on the rising side, corresponding to one unit of time and one unit of discharge. The triangular approximation to this SCS curvilinear hydrograph has the same percent of volume on the rising side of the triangle. Therefore, one unit of time, $t_p$, equals 0.375 unit of the discharge volume, and the time base of this triangle is 2.67 units of time (1.0/0.375), while the recession limb is 1.67 units of time ($t_b - t_p$).

Having established the shape and the time axis for the dimensionless unit hydrograph, only one value for the peak flow, $q_p$, will make the volume under the unit hydrograph equal to 1 inch of rainfall excess. This value can be determined by the equation:

$$q_p = 60 \frac{BA}{t_p} \qquad (2\text{-}10)$$

where:

$q_p$ = Unit hydrograph peak discharge, in cfs

B = Hydrograph shape factor (Note: The SCS dimensionless unit hydrograph B value of 484 should be used in Nashville and Davidson County)

A = Drainage area, in square miles

$t_p$ = Time to peak, in minutes (time from the beginning of rainfall excess to the peak flow rate)

The hydrograph shape factor, B, is generally considered to be a constant characteristic of a watershed. The SCS dimensionless unit hydrographs shown in Figure 2-12 are based on a B value of 484. However, since the value of B can be expected to range from 600 in steep terrain to 300 in flat swampy areas, adjustments to the unit hydrograph shape may be warranted. These adjustments are accomplished by changing the percent of volume under the rising and recession limbs of the unit hydrograph to reflect the corresponding change in the hydrograph shape factor. The standard SCS value of 484 should be used in Nashville and Davidson County unless another value is approved by MDPW.

The dimensionless unit hydrographs shown in Figure 2-12 have a time to peak at 1 unit of time and the point of inflection at approximately 1.7 units of time. Using the SCS average relationship of lag time to time of concentration, given as $t_L = 0.6\ t_c$, and the point of inflection, given as $1.7\ t_p$, the following relationships are obtained:

$$t_c + \Delta D = 1.7\ t_p \tag{2-11}$$

$$t_p = \frac{\Delta D}{2} + 0.6\ t_c \tag{2-12}$$

where:

$t_c$ = Watershed time of concentration

$\Delta D$ = Duration of rainfall excess increment

$t_p$ = Time to peak (time from the beginning of rainfall excess to the peak flow rate)

which can be solved simultaneously to give:

$$\Delta D = 0.133 t_c \tag{2-13}$$

### 2.7.3 CLARK UNIT HYDROGRAPH

The Clark method (1945) requires three parameters to calculate a unit hydrograph: $t_c$, the time of concentration for the basin, R, a storage coefficient, and a time-area curve. A time-area curve defines the cumulative area of the watershed contributing runoff to the subbasin outlet as a function of time (expressed as a proportion of $t_c$). The Clark unit hydrograph is most applicable in areas where observed streamflow indicates that the SCS shape factor of

484 is not appropriate, or in subbasins where a specific time-area curve is needed to describe an unusual shape.

A time-area curve can be developed using a dimensionless time-area curve as presented by the U.S. Army Corps of Engineers (1982) in the HEC-1 computer program and expressed as:

$$AI = 1.414T^{1.5} \qquad 0 \leq T < .5 \qquad (2\text{-}14)$$

$$1 - AI = 1.414\ (1-T)^{1.5} \qquad .5 \leq T \leq 1 \qquad (2\text{-}15)$$

where:

AI = Cumulative drainage area as a fraction of total subbasin area

T = Fraction of time of concentration

The ordinates of the time-area curve are converted to volume of runoff per second for unit excess and interpolated to the given time interval. The resulting translation hydrograph is then routed through a linear reservoir to simulate the storage effects of the basin, and the resulting unit hydrograph for instantaneous excess is averaged to produce the hydrograph for unit excess occurring in the given time interval.

The linear reservoir routing uses the general equation:

$$Q(2) = (CA)\ (I) + (CB)\ Q(1) \qquad (2\text{-}16)$$

and the routing coefficients are calculated from:

$$CA = \Delta t/(R + .5\ \Delta t) \qquad (2\text{-}17)$$

$$CB = 1 - CA \qquad (2\text{-}18)$$

$$QUNGR = .5[Q(1) + Q(2)] \qquad (2\text{-}19)$$

where:

Q(2) = Instantaneous flow at the end of period, in cfs

Q(1) = Instantaneous flow at the beginning of period, in cfs

I = Ordinate of the translation hydrograph

$\Delta t$ = Computation time interval, in hours

QUNGR = Unit hydrograph ordinate at the end of the computation interval

The computation of unit hydrograph ordinates is terminated for the HEC-1 computer program when its volume exceeds 0.995 inch or 150 ordinates, whichever occurs first.

### 2.7.4 URBAN REGRESSION EQUATIONS

Observed streamflow data for small urban watersheds have been analyzed to develop regression equations for deriving synthetic unit hydrographs. Equations for unit durations of 10 minutes (Espey et al., 1978) and 30 minutes (Hamm et al., 1973) are available. Site-specific parameters required to use the equations include main channel length, L, and slope, S, impervious cover, I, drainage area, A, and the watershed conveyance factor, $\Phi$. The watershed conveyance factor is intended to account for the reduction in the time of rise caused by channel improvements or storm sewers. The two components of this factor account for channel characteristics, $\Phi_1$, and vegetation characteristics, $\Phi_2$. The conveyance factor is calculated as the sum of these two components.

The unit hydrograph regression equations provide a basis for estimating the time of rise, $t_r$, peak runoff rate, $q_p$, time base, $t_b$, and unit hydrograph widths at one-half and three-fourths of the peak discharge, $W_{50}$ and $W_{75}$. Using these shape parameters, a unit hydrograph with 1 inch of runoff volume can be constructed.

## 2.8 HYDROLOGIC ROUTING

After a flood hydrograph has been developed for a particular watershed, it may be necessary to route that hydrograph to another point in the drainage system without allowing for additional flow. This process is commonly known as flood routing.

Hydrologic and hydraulic routing techniques are available to quantify the peak flow attenuation and time lag likely to occur when a flood travels through a drainage system. However, hydrologic routing considers only the conservation of mass, whereas hydraulic routing considers both the conservation of mass and the equations of motion. Since, in practice, hydrologic routing techniques are usually adequate for stormwater design purposes, the scope of this section will be limited to their use. For information on hydraulic routing techniques, references by Henderson (1966), Viessman et al. (1977), and Chow (1959, 1964) should be consulted.

### 2.8.1 INPUT DATA

Any flood routing technique requires the following three types of input data:

1. An inflow hydrograph
2. A stage-storage relationship
3. A stage-discharge relationship

An inflow hydrograph can be developed using the fundamentals discussed earlier. The stage-storage and stage-discharge relationships are developed to account for the characteristics of the channel system in question. In practice, it may be convenient to combine these two relationships into a single storage-discharge relationship. The storage-discharge relationships for channels are usually quite different from those for reservoirs, because storage in a channel may depend on both inflow and outflow.

### 2.8.2 CONTINUITY EQUATION

Hydrologic flood routing techniques are all based upon the continuity equation, which requires that the rate of change to storage in a drainage system must account for all mass flow into and out of that system. Mathematically, the continuity equation is expressed as:

$$I - O = \frac{\Delta S}{\Delta t} \qquad (2\text{-}20)$$

where:

I = Inflow rate to the drainage system, in cfs

$O$ = Outflow rate from the drainage system, in cfs

$\frac{\Delta S}{\Delta t}$ = The rate of change to storage in a drainage system, in cfs

As noted above, the storage within a particular channel segment will generally depend on both the inflow to and the outflow from that channel segment. In practice, it is usually acceptable to approximate this dual characteristic of channel storage by dividing the total storage volume into two components. The first component depends only on the outflow rate and is commonly known as <u>prism storage</u>. The second component is <u>wedge storage</u>, which depends on the difference between inflow and outflow rates. This two-component approximation of the channel storage relationship is illustrated in Figure 2-13.

A general mathematical relationship for expressing this two-component channel storage volume is presented by Chow (1959) as:

$$S = \frac{b}{a} \quad XI^{m/n} + (1-X)O^{m/n} \qquad (2\text{-}21)$$

where:

$S$ = Channel storage volume, in cubic feet

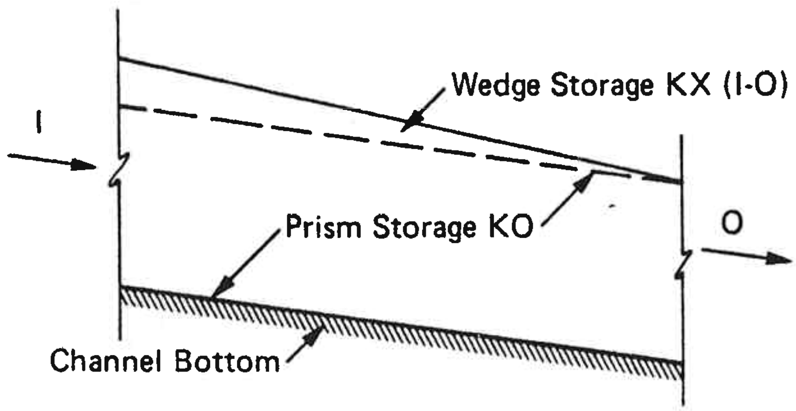
$I$ = Inflow rate to the channel, in cfs

$O$ = Outflow rate from the channel, in cfs

$X$ = Dimensionless factor that determines the relative weights of I and O on the channel storage volume

$a$ and $n$ = Constants that reflect the stage-discharge characteristics of the channel segment

$b$ and $m$ = Constants that reflect the stage-storage characteristics of the channel segment




Note: Channel Storage, S = KO + KX (I-O)

**FIGURE 2-13**
Two-Component Channel Storage Approximation

NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

# CHAPTER 3
# Open Channel Hydraulics

# TABLE OF CONTENTS

# Chapter 3
# OPEN CHANNEL HYDRAULICS

## SYNOPSIS

The conveyance capacity of most drainage facilities can be evaluated using the principles of open channel hydraulics. This chapter provides introductory material on open channel terminology; mass, energy, and momentum principles; and critical, uniform, and nonuniform flow conditions. The fundamental relationship for performing open channel capacity calculations is Manning's Equation. Key references for information presented in this chapter are Chow (1959), Henderson (1966), and French (1985). Supplemental references include Streeter (1971), Simon (1981), Rouse (1950), and USDOT, FHWA (1961) and (1965).

## 3.1 TERMINOLOGY

An open channel is defined as any conduit conveying a fluid in which the liquid surface is exposed to the atmosphere as a boundary. Pipe flow occurs in a conduit that is closed to atmospheric pressure and subject to hydraulic pressure alone. Open channel conditions are the basis for most stormwater management calculations, except for some storm sewer or culvert systems. Pipe flow fundamentals are discussed in Chapters 5 (Culverts) and 6 (Storm Sewers).

Open channel flow problems can be more complicated than similar problems in pressure pipes. This is primarily because physical conditions (such as cross section, slope, and roughness) are typically more variable in channels. Calculations for open channel flow problems tend to be more empirical than those for pipes, and there is greater uncertainty when assigning friction factors.

Terminology important to an understanding of open channel flow problems is briefly defined and discussed below.

### 3.1.1 GEOMETRIC ELEMENTS

Open channel flow problems generally require an evaluation of various geometric elements associated with the shape of the channel. For most artificial or constructed open channels, geometric elements can be determined mathematically in terms of depth of flow and other dimensions for the channel shape. For most natural channel sections, however, profile sections based on the actual variations in the depth of flow across the section are generally required. The following geometric terminology is pertinent to the fundamentals of open channel hydraulics:

Prismatic channel. An artificial channel with non-varying cross section and constant bottom slope.

Channel section. The cross section of a channel taken perpendicular to the direction of flow.

Depth of flow. The vertical distance from the lowest point of a channel section to the free surface, designated as y. The depth of flow measured perpendicular to the channel bottom is known as the depth of flow for the section and is designated as d. When the channel slope is small (less than 1 percent), the depths y and d are essentially equal. The relationship between y and d is expressed as:

$$y = \frac{d}{\cos\theta} \tag{3-1}$$

where:

$y$ = Vertical depth of flow, in feet

$d$ = Perpendicular depth of flow, in feet

$\theta$ = Slope angle of the channel bottom, in degrees

Stage. The elevation or vertical distance of the water surface relative to a datum. If the lowest point of a channel is taken as the datum, then the stage and depth of flow are equal.

Control section. Any section at which the depth of flow is known or can be controlled at a given stage for specified discharge rates.

Top width. The width of the channel section at the free surface.

Water area. The cross-sectional area of the flow perpendicular to the direction of flow.

Wetted perimeter. The length of the line of intersection of the channel wetted surface with a cross-sectional plane perpendicular to the direction of flow.

Hydraulic radius. The ratio of the water area to its wetted perimeter, which is expressed mathematically as:

$$R = \frac{A}{P} \tag{3-2}$$

where:

R = Hydraulic radius of the channel, in feet

A = Water area of the channel; in square feet

P = Wetted perimeter of the channel, in feet

Hydraulic depth. The ratio of the water area to top width, which is expressed mathematically as:

$$d_m = \frac{A}{T} \tag{3-3}$$

where:

$d_m$ = Hydraulic depth or mean depth of flow, in feet

A = Water area of the channel, in square feet

T = Top width of the channel, in feet

Equivalent depth. The depth corresponding to an area of flow having a width twice the depth, expressed mathematically as:

$$d_e = (A/2)^{1/2} \tag{3-4}$$

where:

$d_e$ = Equivalent depth, in feet

A = Cross-sectional flow area, in square feet

The equivalent depth is often used to calculate the Froude number for flow in pipes under open channel flow conditions.

Critical flow section factor. The product of the water area and the square root of the hydraulic depth, expressed mathematically as:

$$Z = A\ (\sqrt{d_m})^{1/2} = A\ \frac{A}{T}^{1/2} \qquad (3\text{-}5)$$

where:

Z = Critical flow section factor

A = Water area, in square feet

$d_m$ = Hydraulic depth (see Equation 3-3), in feet

T = Top width of channel, in feet

Uniform flow section factor. Based on Manning's Equation, the product of the water area and the hydraulic radius raised to the 2/3 power, expressed mathematically as:

$$U_F = AR^{2/3} \qquad (3\text{-}6)$$

where:

$U_F$ = Uniform flow section factor for Manning's Equation

A = Water area, in square feet

R = Hydraulic radius, in feet

### 3.1.2 STEADY/UNSTEADY FLOW

Time variations of open channel flow can be classified as either steady or unsteady. Steady flow occurs in an open channel when the discharge or rate of flow at any location along the channel remains constant with respect to time. The maintenance of steady flow in any channel reach requires that the rates of inflow and outflow be constant and equal. Conversely, open channel flow is unsteady when the discharge at any location in the channel changes with respect to time. During periods of stormwater runoff, the inflow hydrograph to an open channel is usually unsteady. However, in practice, open channel flow is generally assumed to be steady at the discharge rate for which the channel is being designed (i.e., peak discharge of the inflow hydrograph).

### 3.1.3 UNIFORM/NONUNIFORM FLOW

Spatial variations of open channel flow can be classified as either uniform or nonuniform.

Uniform flow occurs only in a channel of constant cross section, slope, and roughness, known as a uniform open channel. If a given channel segment is uniform, the mean velocity and depth of flow will be constant with respect to distance. When the requirements for uniform flow are met, the depth of flow for a given discharge is defined as the normal depth of flow. In practice, minor variations in the channel bottom or deviations from the average cross section can be ignored as long as the average values are representative of actual channel conditions.

A general assumption is that uniform flow is also steady flow. It is reasonable to conclude that when the water surface remains constant with time, depth will remain constant with distance.

True uniform flow rarely exists in either natural or artificial channels. Any change in the channel cross section, slope, or roughness with distance causes the depths and average velocities to change with distance. Flow that varies in depth and velocity when the discharge is constant, or steady, is defined as steady nonuniform flow.

Unsteady nonuniform flow, in which there are variations of both space and time, is the most complex type to evaluate mathematically. Chow (1959), Henderson (1966), or French (1985) should be consulted for theoretical information.

Nonuniform flow may be further classified as either rapidly or gradually varied. Rapidly varied flow is also known as a local phenomenon, examples of which include the hydraulic jump and hydraulic drop. The primary example of gradually varied flow occurs when subcritical flow is restricted by a culvert or storage reservoir. The water surface profile caused by such a restriction is generally referred to as a backwater curve. Additional theoretical information on nonuniform flow evaluations is presented in Section 3.5.

### 3.1.4 LAMINAR/TURBULENT FLOW

The effect of fluid viscosity relative to the inertial forces of motion is an important property to consider when evaluating open channel flow.

Laminar flow generally occurs when the viscous forces are strong relative to inertial forces. Water particles will appear to move in definite smooth paths, or streamlines, when flow is laminar. Laminar flow is known to occur in shallow overland or sheet flow conditions.

When the viscous forces are weak relative to the inertial forces, the flow can be classified as turbulent. In turbulent flow, the water particles move in irregular paths that are neither smooth nor fixed, and the result is a random mixing motion. Because turbulent flow is the most common type occurring in open channel facilities, it is the type considered for most hydraulic procedures, excluding shallow overland flow.

Operational limits for laminar and turbulent flow can be evaluated using a dimensionless parameter known as the Reynolds number, which is expressed mathematically as:

$$Re = \frac{vL}{\nu} \qquad (3-7)$$

where:

$Re$ = Reynolds number, dimensionless

$v$ = Average velocity of flow, in feet/second

$L$ = Characteristic length, in feet (Hydraulic radius as presented in Equation 3-2)

$\nu$ = Kinematic viscosity of fluid, in square feet/second

Since viscosity is in the denominator, low Reynolds number values are associated with laminar flow (high viscosity relative to inertial forces) and high Reynolds number values are associated with turbulent flow (high inertial forces relative to viscosity).

Numerous experiments have been performed to establish operational limits of the Reynolds number that will define when laminar and turbulent flow occur. When flow can be classified as neither laminar nor turbulent, it is called transitional. When the hydraulic radius is used to determine the characteristic length in Equation 3-7, then the following operational limits of the Reynolds number are reported (French, 1985):

| | |
|---|---|
| Laminar Flow | $Re \leq 500$ |
| Transitional Flow | $500 \leq Re \leq 12{,}500$ |
| Turbulent Flow | $Re \geq 12{,}500$ |

### 3.1.5 SUBCRITICAL/CRITICAL/SUPERCRITICAL FLOW

The importance of gravity as a driving force in open channel drainage systems makes its effect on the state of flow a major factor for evaluation. This can be done using a dimensionless parameter known as the Froude number, which is expressed mathematically as:

$$Fr = \frac{v}{\sqrt{gL}} \tag{3-8}$$

where:

Fr = Froude number, dimensionless

v = Average velocity of flow, in feet/second

g = Acceleration due to gravity, 32.2 feet/second$^2$

L = Characteristic length, in feet (Hydraulic depth as presented in Equation 3-3)

The Fr value is the dimensionless ratio of inertial forces to gravity forces. If Fr values are less than 1, gravity forces dominate and the open channel is said to be operating in the subcritical range of flow. This is sometimes called tranquil flow and is characterized as relatively deep, low velocity flow with respect to critical flow. Depth of flow can be controlled at a downstream location.

If Fr values are greater than 1, inertial forces dominate and the open channel is said to be operating in the supercritical range of flow. This is also called rapid or shooting flow and is characterized as relatively shallow, high velocity flow with respect to critical flow. Depth of flow can be controlled at an upstream location.

When the Fr value equals 1, inertial forces and gravity forces are balanced and the open channel exhibits critical flow. Additional information on critical flow conditions is presented in Section 3.3.

## 3.2 MASS, ENERGY, AND MOMENTUM

The following three basic principles are generally applied to open channel flow evaluations:

1. Conservation of mass
2. Conservation of energy
3. Conservation of linear momentum

### 3.2.1 MASS

The conservation of mass or continuity equation for continuous steady flow can be expressed mathematically as:

$$Q = Av \tag{3-9}$$

where:

$Q$ = Discharge, in cfs

$A$ = Cross-sectional area, in square feet

$v$ = Average channel velocity, in feet/second

For continuous unsteady flow, the continuity equation must include time as a variable. Additional information on the conservation of mass for unsteady flow can be obtained from Chow (1959), Henderson (1966), or French (1985).

### 3.2.2 ENERGY

It can be useful at times to consider the total energy head of an open channel. Because energy input must equal output, the total energy head of two points in a channel reach will equal one another. This equality is commonly known as the energy equation, which is expressed as:

$$d_1 \cos\Theta + \frac{v_1^2}{2g} + z_1 = d_2 \cos\Theta + \frac{v_2^2}{2g} + z_2 + h_{loss} \qquad (3\text{-}10)$$

where:

$d_1$ and $d_2$ = Perpendicular depths of flow at channel sections 1 and 2, respectively, in feet

$\Theta$ = Slope angle of the channel bottom, in degrees

$v_1$ and $v_2$ = Average velocities at channel sections 1 and 2, respectively, in feet/second

$g$ = Acceleration due to gravity, 32.2 feet/second$^2$

$z_1$ and $z_2$ = Bottom elevations above an arbitrary datum at channel sections 1 and 2, respectively, in feet

$h_{loss}$ = Head or energy loss between channel sections 1 and 2, in feet

For small channel slopes (less than 1 percent), the Energy Equation is expressed as:

$$y_1 + \frac{v_1^2}{2g} + z_1 = y_2 + \frac{v_2^2}{2g} + z_2 + h_{loss} \qquad (3\text{-}11)$$

where:

$y_1$ and $y_2$ = Vertical depths of flow at channel sections 1 and 2, respectively, in feet

$v_1$ and $v_2$ = Average velocities at channel sections 1 and 2, respectively, in feet/second

$g$ = Acceleration due to gravity, 32.2 feet/second$^2$

$z_1$ and $z_2$ = Bottom elevations above an arbitrary datum at channel sections 1 and 2, respectively, in feet

$h_{loss}$ = Head or energy loss between channel sections 1 and 2, in feet

When the head loss between channel sections is zero, either Equation 3-10 or 3-11 can be referred to as the Bernoulli equation.

Equations 3-10 and 3-11 ignore that the actual velocity distribution over a channel section is nonuniform (i.e., slow along the bottom and higher in the middle). When nonuniform velocity distributions are considered, the velocity head is generally greater than the value computed using the average channel velocity. Kinetic energy coefficients that can be used to account for nonuniform velocity conditions at bridges are discussed in Chapter 7.

For typical prismatic channels with a fairly straight alignment, the effect of disregarding the existence of a nonuniform velocity distribution is negligible, especially when compared to other uncertainties involved in such calculations. Therefore, Equations 3-10 and 3-11 are appropriate for most open channel problems. However, if velocity distributions are known or suspected to be non-typical, velocity coefficient adjustments should be considered.

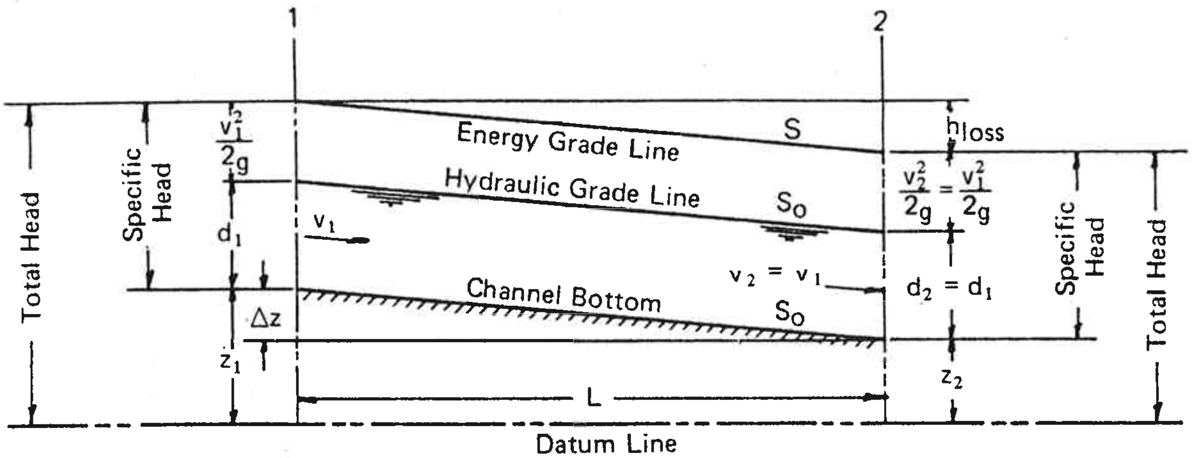
Equations 3-10 and 3-11 also assume that the hydrostatic law of pressure distribution is applicable. This law states that the distribution of pressure over the channel cross section is the same as the distribution of hydrostatic pressure; that is, that the distribution is linear with depth. The assumption of a hydrostatic pressure distribution for flowing water is valid only if the flow is not accelerating or decelerating in the plane of the cross section. Thus, these equations should be restricted to conditions of uniform or gradually varied nonuniform flow. If the flow is known to be rapidly varying, other relationships should be considered (e.g., momentum).

A longitudinal profile of total energy head elevations is called the energy grade line (gradient). The longitudinal profile of water surface elevations is the hydraulic grade line (gradient). The energy and hydraulic grade lines for uniform open channel flow are illustrated in Figure 3-1. For flow to occur in an open channel, the energy grade line must have a negative slope in the direction of flow. A gradual decrease in the energy grade line for a given length of channel represents the loss of energy caused by friction. When considered together, the hydraulic and energy grade lines reflect not only the loss of energy by friction, but also the conversion between potential and kinetic forms of energy.

For uniform flow conditions, the energy grade line is parallel to the hydraulic grade line, which is parallel to the channel bottom (see Figure 3-1). Thus, for uniform flow, the slope of the channel bottom becomes an adequate basis for the determination of friction losses. During uniform flow, no conversions occur between kinetic and potential forms of energy. If the flow is accelerating, the hydraulic grade line would be steeper than the energy grade line, while retarding flow would produce an energy grade line steeper than the hydraulic grade line.

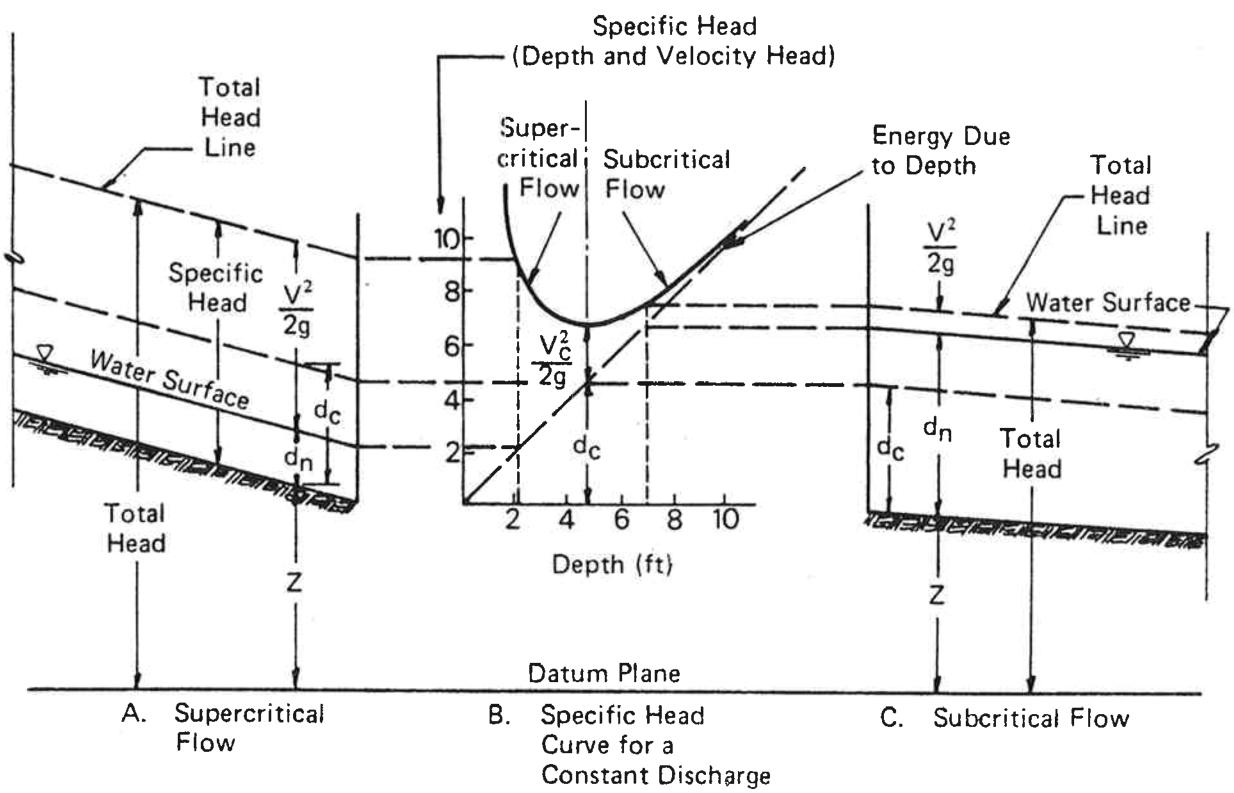
### 3.2.3 MOMENTUM

According to Newton's Second Law of Motion, the change of momentum per unit of time is equal to the resultant of all external forces applied to the moving body. Application of this principle to open channel flow produces a relationship that is virtually the same as the energy equation expressed in Equations 3-10 and 3-11. In theory, the two principles are unique, primarily because energy is a scalar quantity (magnitude only), while momentum is a vector quantity (magnitude and direction). In addition, the head loss




Reference: USDOT, FHWA, HDS-3 (1961).

**FIGURE 3-1**
Characteristics of Uniform Open Channel Flow




Reference: USDOT, FHWA, HDS-3 (1961).

**FIGURE 3-2**
Definition Sketch for Specific Head and Subcritical and Supercritical Flow

determined by the energy equation measures the internal energy dissipated in a particular channel reach, while the momentum equation measures the losses caused by external forces exerted on the water by the walls of the channel. However, for uniform flow, since the losses caused by external forces and internal energy dissipation are equal, the momentum and energy equations give the same results.

Application of the momentum principle has certain advantages for problems involving high changes of internal energy, such as a hydraulic jump. Thus, the momentum principle should be used to evaluate rapidly varied nonuniform flow conditions. The hydraulic jump is a common method for accomplishing energy dissipation at culvert, storm sewer, and channel outlets. Energy dissipation is covered in Chapter 10.

## 3.3 CRITICAL FLOW

The energy content of flowing water with respect to the channel bottom is often referred to as the specific energy head, which is expressed by the equation:

$$E = d + \frac{v^2}{2g} \tag{3-12}$$

where:

E = Specific energy head, in feet

d = Depth of open channel flow, in feet

v = Average channel velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second$^2$

Considering the relative values of potential energy (depth) and kinetic energy (velocity head) in an open channel can greatly aid the hydraulic analysis of open channel flow problems. These analyses are usually performed using a curve showing the relationship between the specific energy head and the depth of flow for a given discharge in a given channel that can be placed on various slopes. The curve representing specific energy head for an open channel is generally used to identify regions of supercritical and subcritical flow conditions. This information can be helpful for performing hydraulic capacity calculations and evaluating the suitability of channel linings and flow transition sections.

### 3.3.1 SPECIFIC ENERGY AND CRITICAL DEPTH

A typical curve representing the specific energy head of an open channel is illustrated in Figure 3-2, Part B. The straight diagonal line on this figure represents points where the depth of flow and specific energy head are equal. At such points the kinetic energy is zero; therefore, this diagonal line is a plot of the potential energy, or energy due to depth. The ordinate interval between the diagonal line of potential energy and the specific energy curve for the desired discharge is the velocity head, or kinetic energy, for the depth in question. The lowest point on the specific energy curve represents flow with the minimum content of energy. The depth of flow at this point is known as the critical depth. The general equation for determining the critical depth is expressed as:

$$\frac{Q^2}{g} = \frac{A^3}{T} \tag{3-13}$$

where:

Q = Discharge, in cfs

g = Acceleration due to gravity, 32.2 feet/second$^2$

A = Cross-sectional area, in square feet

T = Top width of water surface, in feet

Critical depth for a given channel can be calculated through trial and error with Equation 3-13. However, in practice, it may be computed more easily by using either the equations for selected channel cross sections or the critical depth charts presented in Volume 2. Chow (1959) presents a procedure for the analysis of critical flow that uses the critical flow section factor, Z, as defined by Equation 3-5.

Using the definition of the critical section factor and a velocity distribution coefficient of one, the equation for critical flow conditions is:

$$Z = \frac{Q}{\sqrt{g}} \tag{3-14}$$

where:

Z = Critical flow section factor (see Equation 3-5)

Q = Discharge, in cfs

g = Acceleration due to gravity, 32.2 feet/second$^2$

When the discharge is known, Equation 3-14 gives a value for the critical section factor, allowing Equation 3-5 to be solved for the critical depth. Conversely, when the critical section factor is known, the discharge can be calculated by rearranging Equation 3-14.

The determination of critical depth is independent of the channel slope and roughness, because critical depth represents a depth for which the specific energy head is a minimum. According to Equation 3-13, the magnitude of critical depth depends only on the discharge and the shape of the channel. Thus, any given size and shape of channel has only one critical depth for the given discharge, which is independent of the channel slope or roughness. However, if Z is not a single-valued function of depth, it is possible to have more than one critical depth. For a given value of specific energy, the critical depth results in the greatest discharge, or conversely, for a given discharge, the specific energy is a minimum for the critical depth.

### 3.3.2 CRITICAL VELOCITY

The velocity at critical depth is called the critical velocity. An equation for determining the critical velocity in an open channel of any cross section is expressed as:

$$v_c = \sqrt{g\ d_m} \qquad (3\text{-}15)$$

where:

$v_c$ = Critical velocity, in feet/second

$g$ = Acceleration due to gravity, 32.2 feet/second²

$d_m$ = Mean depth of flow or equivalent depth, $d_e$, for pipes, in feet, calculated from Equation 3-3 or 3-4

### 3.3.3 SUPERCRITICAL FLOW

For conditions of uniform flow, the critical depth, or point of minimum specific energy, occurs when the channel slope equals the critical slope (i.e., the normal depth of flow in the channel is critical depth). When channel slopes are steeper than the critical slope and uniform flow exists, the specific energy head is higher than the critical value, because of the higher values of the velocity head (kinetic energy). This characteristic of open channel flow is illustrated by the specific head curve segment to the left of critical depth in Figure 3-2, Part B, and is known as supercritical flow. Supercritical flow is characterized by relatively shallow depths and high velocities, as shown in Figure 3-2, Part A. If the natural depth of flow in an open channel is supercritical, the depth of flow at any point in the channel may be influenced by an upstream control section. The relationship of supercritical flow to the specific energy curve is shown in Figure 3-2, Parts A and B.

### 3.3.4 SUBCRITICAL FLOW

When channel slopes are flatter than the critical slope and uniform flow exists, the specific energy head is higher than the critical value, because of the higher values of the normal depth of flow (potential energy). This characteristic of open channel flow is illustrated by the specific head curve segment to the right of critical depth in Figure 3-2, Part B, and is known as subcritical flow. Subcritical flow is characterized by relatively large depths with low velocities, as shown in Figure 3-2, Part C. If the natural depth of flow in an open channel is subcritical, the depth of flow at any point in the channel may be influenced by a downstream control section. The relationship of subcritical flow to the specific energy curve is shown in Figure 3-2, Parts B and C.

### 3.3.5 SPECIFIC ENERGY CONSIDERATIONS

Several points about Figure 3-2 should be noted. First, at depths of flow near the critical depth for any discharge, a minor change in specific energy will cause a much greater change in depth. Second, velocity head for any discharge in the subcritical portion of the specific energy curve in Figure 3-2, Parts A and B, is relatively small when compared to specific energy. For this subcritical portion of the specific energy curve, changes in depth of flow are approximately equal to changes in specific energy. Finally, the velocity head for any discharge in the supercritical portion of the specific energy curve increases rapidly as depth decreases. For this supercritical portion of the specific energy curve, changes in depth are associated with much greater changes in specific energy.

## 3.4 UNIFORM FLOW

Although steady uniform flow is rare, it is practical in many cases to assume that uniform flow occurs in appropriate segments of an open channel system. The results obtained from calculations based on this assumption will be approximate and general, but can often provide satisfactory solutions.

### 3.4.1 MANNING'S EQUATION

The hydraulic capacity of an open channel is usually determined through application of Manning's Equation, which determines the average velocity when given the depth of flow in a uniform channel cross section. Given the velocity, the capacity, Q, is calculated as the product of velocity and cross-sectional area (see Equation 3-9).

Manning's Equation is an empirical equation in which the values of constants and exponents have been derived from experimental data for turbulent flow conditions. According to Manning's Equation, the mean velocity of flow is a function of the channel roughness, the hydraulic radius, and the slope of the energy gradient. As noted previously, for uniform flow, the slope of the energy gradient is assumed to be equal to the channel bottom slope. Manning's Equation is expressed mathematically as:

$$v = \frac{1.49}{n} R^{2/3} S^{1/2} \qquad (3\text{-}16)$$

or

$$Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \qquad (3\text{-}17)$$

where:

- v = Average channel velocity, in feet/second
- Q = Discharge, in cfs
- n = Manning's roughness coefficient
- R = Hydraulic radius of the channel, in feet, calculated using Equation 3-2
- S = Slope of the energy gradient, in feet/foot
- A = Cross-sectional area of the open channel, in square feet

A detailed discussion of procedures for solving Manning's Equation is presented in Volume 2.

### 3.4.2 BEST HYDRAULIC SECTION

The best hydraulic section of an open channel can be determined mathematically by using the continuity equation of open channel flow (Equation 3-9) and Manning's Equation (Equation 3-17). According to the continuity equation, if the cross-sectional area is to be a minimum, the velocity must be a maximum for any given cross-sectional area. According to Manning's Equation, the velocity is a maximum for a given cross section and channel slope when the hydraulic radius is a maximum. The hydraulic radius is a maximum when the wetted perimeter is minimized for a given cross-sectional area (see Equation 3-2).

It can be shown mathematically that the hydraulic cross section that maximizes the velocity, and thus minimizes the area required to convey a given discharge, is a semicircle (see Streeter, 1971, or Chow, 1959). Therefore, the best trapezoidal hydraulic section is one that approximates a semicircle (see Figure 3-3, Part a). For the special case in which the trapezoid is a rectangle, the best shape is that for which the width is twice the depth (see Figure 3-3, Part b).

In practice, the best hydraulic section of an open channel may be altered from the mathematically ideal cross section to account for the following factors:

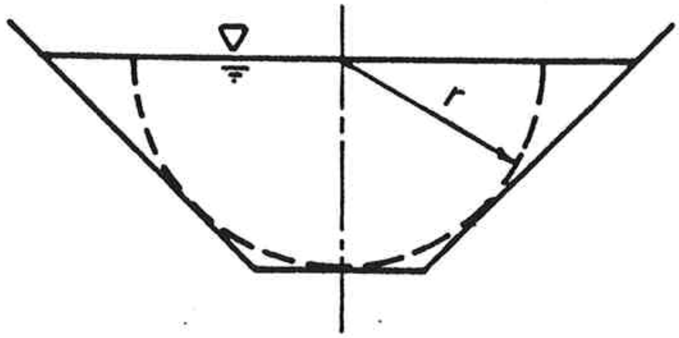
1. The average velocity of the best hydraulic section should not exceed the maximum permissible velocity for the channel bottom in question.

2. The best hydraulic section may not produce the minimum total excavation if a significant overburden must be removed.

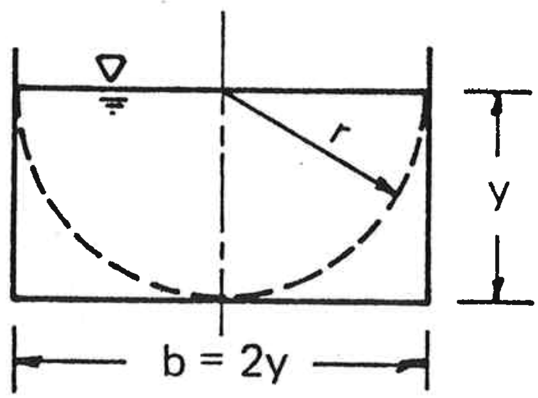
3. The proportions of an open channel may vary widely without significantly changing the required hydraulic elements.

4. The cost of excavation is not solely dependent on the amount of material removed. Considerations such as the ease of access and disposal may be more important than the volume of material excavated.

5. The method of construction or available equipment that will be used can affect the type of excavation.




(a) Trapezoidal




(b) Rectangular

**FIGURE 3-3**
Best Hydraulic Sections for Trapezoidal and Rectangular Channels

## 3.5 NONUNIFORM FLOW

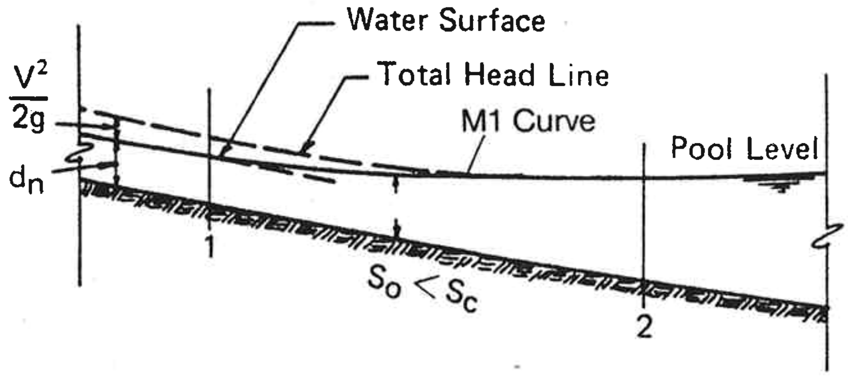
In the vicinity of changes in the channel section or slope that will cause nonuniform flow profiles, the direct solution of Manning's Equation is not possible, since the energy gradient for this situation does not equal the channel slope. Three typical examples of nonuniform flow are illustrated in Figures 3-4 through 3-6. The following sections describe these nonuniform flow profiles and briefly explain how the total head line is used for approximating these water surface profiles in a qualitative manner. For gradually varied flow, the direct step and standard step methods for performing backwater calculations are discussed. The hydraulic jump is discussed for rapidly varied flow.

### 3.5.1 GRADUALLY VARIED FLOW

A channel on a mild slope (subcritical) discharging into a reservoir or pool is illustrated in Figure 3-4. The vertical scale is exaggerated for clearer illustration.

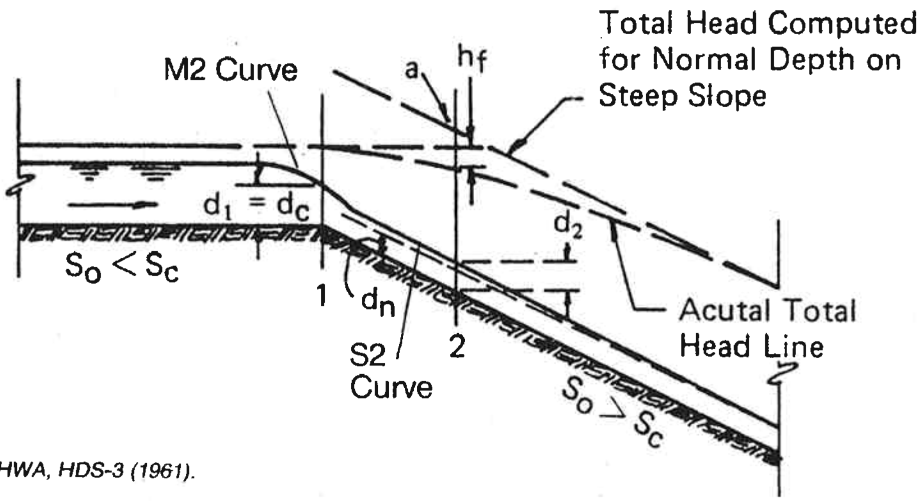
Cross Section 1 is located upstream of the pool, where uniform flow occurs in the channel, and Cross Section 2 is located at the beginning of a level pool. The depth of flow between Sections 1 and 2 is changing, and the flow is nonuniform. The water surface profile between the sections is known as a backwater curve referred to as an M1 curve, and is characteristically very long.

A channel in which the slope changes from subcritical (mild) to supercritical (steep) is illustrated in Figure 3-5. The flow profile passes through critical depth near the break in slope (Section 1). This is true whether the upstream slope is mild, as in the sketch, or the water above Section 1 is ponded, as would be the case if Section 1 were the crest of a spillway of a dam. If, at Section 2, the total head were computed, assuming normal depth on the steep slope, it would plot (Point a in Figure 3-5) above the elevation of total head at Section 1. This is physically impossible, because the total head line must slope downward in the direction of flow. The actual total head line will take the position shown and have a slope approximately equal to $S_o$ at Section 1 and approaching $S_o$ farther downstream. The drop in the total head line, $h_{loss}$, between Sections 1 and 2 represents the loss in energy due to friction.



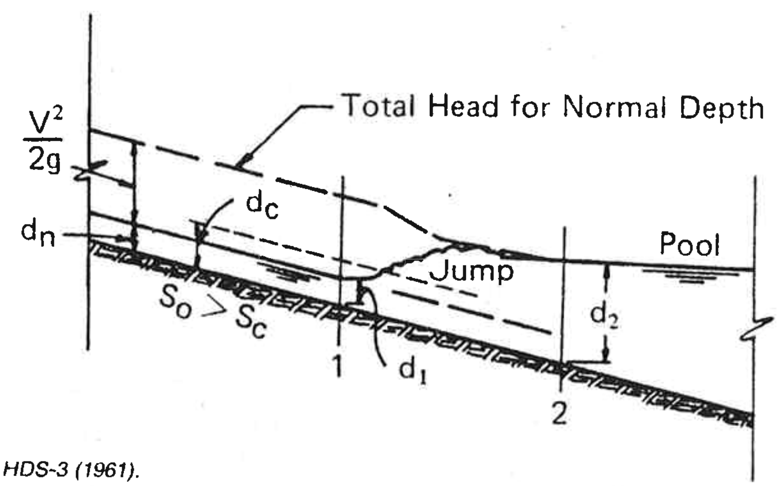

Reference: USDOT, FHWA, HDS-3 (1961).

**FIGURE 3-4**
Nonuniform Water Surface Profile for Downstream Control Caused by a Flow Restriction




Reference: USDOT, FHWA, HDS-3 (1961).

**FIGURE 3-5**
Nonuniform Water Surface Profile Caused by a Change in Slope Conditions




Reference: USDOT, FHWA, HDS-3 (1961).

**FIGURE 3-6**
Nonuniform Water Surface Profile Caused by a Hydraulic Jump

At Section 2, the actual depth, $d_2$, is greater than normal depth, $d_n$, because sufficient acceleration has not occurred, and the assumption of normal depth at this point would clearly be in error. As Section 2 is moved downstream, so that total head for normal depth drops below the pool elevation above Section 1, the actual depth quickly approaches the normal depth for the steep channel. This type of water surface curve (Section 1 to Section 2) is referred to as an S2 curve and is characteristically much shorter than the M1 backwater curve discussed previously.

Another common type of nonuniform flow is the drawdown curve to critical depth that occurs upstream from Section 1 (Figure 3-5) where the water surface passes through critical depth. The depth gradually increases upstream from critical depth to normal depth, provided that the channel remains uniform over a sufficient distance. The length of the drawdown curve, referred to as an M2 curve, is much longer than the curve from critical depth to normal depth in the steep channel.

Direct Step Method

For prismatic channels, when the resistance coefficient and shape are constant with distance, gradually varied water surface profiles can be calculated using the direct step method. This method calculates a water surface profile by determining the distance between cross sections with specified flow depths. The friction slope can be calculated using the following equation:

$$S_f = \frac{n^2 \quad v^2}{2.22 \ R^{4/3}} \qquad (3\text{-}18)$$

where:

$S_f$ = Friction slope, in feet/foot

n = Manning's n value

v = Average velocity, in feet/second

R = Hydraulic radius, in feet

The length of channel between sections with specified flow depths is calculated as:

$$\Delta x = \frac{\Delta E}{S_o - S_f} \tag{3-19}$$

where:

$\Delta x$ = Channel distance between sections with specified flow depths, in feet

$\Delta E$ = Change in specific energy between sections (depth plus velocity head), in feet

$S_o$ = Bottom slope, in feet/foot

$S_f$ = Friction slope, in feet/foot

A step-by-step procedure, tabulation form, and example problem for the direct step method are presented in Volume 2.

## Standard Step Method

A procedure suitable for calculating gradually varying water surface profiles when the channel cross section and resistance coefficient vary with distance is the standard step method. This method provides an estimate of the depth of flow at specified longitudinal distances.

The friction slope can be calculated using Equation 3-18. The friction loss in a specified channel reach length is calculated as:

$$h_f = \bar{S}_f \, \Delta x \tag{3-20}$$

where:

$h_f$ = Friction loss for a reach length of $\Delta x$, in feet

$\bar{S}_f$ = Average friction slope between sections for reach length $\Delta x$, in feet/foot

$\Delta x$ = Channel reach length, in feet

The eddy loss, which can be significant in non-prismatic channels, can be calculated using the following equation:

$$h_e = k_e \frac{v^2}{2g} \qquad (3\text{-}21)$$

where:

$h_e$ = Eddy losses, in feet

$k_e$ = Eddy loss coefficient

$v$ = Average velocity, in feet/second

$g$ = Acceleration due to gravity, 32.2 feet/second

For gradually converging and diverging reaches, $k_e$ is 0 to 0.1 and 0.2, respectively. For abrupt expansions and contractions, $k_e$ is about 0.5. For prismatic and regular channels, $k_e = 0$ (Chow, 1959).

A trial and error procedure based on an assumed depth is required to develop the water surface profile. The correct solution for a given section is obtained when the total head based on velocity and depth is equal to the total head based on friction losses (Equation 3-10) and eddy losses (Equation 3-21).

A step-by-step procedure, tabulation form, and example problem for the standard step method are presented in Volume 2.

### 3.5.2 RAPIDLY VARIED FLOW

A hydraulic jump occurs when a steep (supercritical) channel discharges into a reservoir or pool. This special case condition is illustrated in Figure 3-6. A hydraulic jump makes a dynamic transition from the supercritical flow in the steep channel to the subcritical flow in the pool. This situation differs from that shown in Figure 3-4 in that the flow approaching the pool in Figure 3-6 is supercritical and the total head in the approach channel is large, relative to the pool depth. In general, supercritical flow can be changed to subcritical flow only by passing through a hydraulic jump. The violent turbulence in the jump dissipates energy rapidly, causing a sharp drop in the total head line between the supercritical and subcritical states of flow. A jump will occur whenever the ratio of the depth in the approach channel, $d_1$, to the depth in the downstream channel, $d_2$, reaches a specific value. In Figure 3-6, normal depth in the approach channel persists beyond the point where the projected pool level would intersect the water surface of the channel at normal depth. Normal depth can be assumed to exist on the steep slope upstream from Section 1, which is located at about the toe of the jump.

The relationship between variables for a hydraulic jump can be expressed as:

$$\frac{d_2}{d_1} = \frac{\left(1 + 8Fr^2_1\right)^{0.5} - 1}{2} \qquad (3\text{-}22)$$

where:

$d_2$ = Depth below jump, in feet

$d_1$ = Depth above jump, in feet

$Fr_1$ = Froude number above jump, dimensionless (see Equation 3-8)

Additional discussion on the use of hydraulic jumps to dissipate energy is presented in Chapter 10. Detailed information on the quantitative evaluation of hydraulic jump conditions in open channels is available in publications by Chow (1959), Henderson (1966), French (1985), and Streeter (1971), and in HEC-14 from USDOT, FHWA (1983). In addition, handbooks by Brater and King (1976) and the USDA, SCS (NEH-5, 1956) may be useful.

## 3.6 CHANNEL STABILIZATION

### 3.6.1 IMPROVEMENTS

While quantification of the effects of channel improvements is difficult, qualitative assessment of possible impacts and appropriate action are required for all channel modifications. The proportionality given below provides a basis for qualitative assessment of stream modifications; the two sides will adjust to maintain a constant proportion.

$$QS \propto Q_s d_{50} \tag{3-23}$$

where:

$Q$ = Typical or dominant discharge, in cfs

$S$ = Energy slope, in feet/foot

$Q_s$ = Sediment transport, in pounds/second

$d_{50}$ = Sediment mean particle diameter, in feet

Thus, if a check dam is built across a stream, $Q_s$ decreases. Assuming $Q$ and $d_{50}$ remain constant, the slope must decrease to maintain the proportionality. The result is scour downstream from the dam and deposition upstream.

If a bend in a stream is straightened or a rough channel is paved, the slope is steepened. If $S$ increases, either $Q_s$ or $d_{50}$ or both must also increase. The result is generally advancing scour upstream, higher velocities through the reach with associated channel deterioration, scour just below the reach, and eventual deposition downstream.

A tributary carrying excess sediment resulting from construction will tend to block the main channel, raising flow levels upstream and causing deposition downstream. The cause is an increased $Q_s$ in the reach, with a resulting increase in $S$ in the downstream direction.

Urbanization tends to increase the amount of water flowing in a channel for any given storm frequency (i.e., $Q$ increases). A channel in Nashville tends to adjust its size to carry the 2- to 5-year storm within its banks. Thus, urbanization tends to cause not only increased flooding but increased channel size through bed bank erosion.

The sediment transport capacity of the stream will be approximately constant throughout the stream when neither aggradation nor deposition occurs. Bars may form and disappear and the outside of bends may show caving, but the general form of the stream is a constant. An estimate of sediment transport capacity in coarse sand and gravel bed streams can be made from the Meyer-Peter and Mueller formula (Simons and Senturk, 1977), expressed as:

$$Q_s = 600W\ (\tau - \tau_c) \qquad (3\text{-}24)$$

where:

$Q_s$ = Bedload sediment discharge, in tons/day

$W$ = Channel width, in feet

$\tau$ = Channel shear = $\gamma$ RS, in pounds/square foot

$\gamma$ = Specific weight of water, 62.2 pounds/cubic foot

$R$ = Hydraulic radius, in feet

$S$ = Channel slope, in feet/foot

$\tau_c$ = Critical shear = $4.82d_{50}$, in pounds/square foot

$d_{50}$ = Sediment mean particle diameter, in feet

This formulation assumes that the sediment has a specific gravity of 2.65, which is common for quartz rock. Additional information on channel aggradation and degradation can be found in Chapter 7.

### 3.6.2 RIPRAP

Riprap design will be influenced by one of three different types of flow conditions. In Type I flow, the velocity in the upstream channel is higher than in the riprap-lined reach. Riprap size will be determined by the faster upstream velocity. The stone roughness can be expected to slow the velocity enough within a transition length of about 75 to 100 times $d_{50}$ to design for Type II flow conditions for the remainder of the reach.

Type II flow occurs when the velocity in the riprap-lined reach is expected to be higher than the upstream channel velocity. A typical example is a broad, deep channel discharging into a shallow, narrow reach. Designing riprap for Type II flow is an iterative process. A stone size and associated roughness value are assumed, velocity calculated based on the roughness value, and a stone size determined. The calculations are repeated until the assumed and computed stone sizes are reasonabley close.

Type III flow is encountered when the turbulence acting to dislodge the stone is not generated by general boundary shear but by abrupt changes in boundary geometry. In such cases, a general riprap design procedure is not possible. Application of experienced engineering judgment, specific designs (e.g., stilling basin design), or the selection of conservative parameters are possible options.

The procedure given in Volume 2 is an adaptation of basic research performed by S. T. Maynord (1987) at the Waterways Experiment Station and developmental work by A. J. Reese (1984 and 1988). The original equation developed by Maynord (1987) is expressed as:

$$d_{30}/D = C \left[ \left( \frac{\gamma}{\gamma_s - \gamma} \right)^{0.5} \frac{v}{(gD)^{0.5}} \right]^{2.5} \qquad (3\text{-}25)$$

where:

$d_{30}$ = Diameter of stone for which 30 percent, by weight, of the gradation is finer, in feet

$D$ = Depth of flow above stone, in feet

$v$ = Mean point velocity above the stone, in feet/second

$\gamma_s$ = Specific weight of stone, in pounds/cubic foot

$\gamma$ = Specific weight of water, 62.4 pounds/cubic foot

$g$ = Acceleration due to gravity, 32.2 feet/second$^2$

The C factor is 0.30 for the bottoms of straight prismatic channels and both bottoms and sides of curved channels. The depth is the depth to the top of rock at the point in question (usually the toe of the side slope) and the velocity is the mean point velocity. The C factor for side slopes of straight prismatic channels is 0.24. With a safety factor, a general constant is given as 0.36, yielding a 1.2 safety factor for the former case and 1.5 for the latter. The C factor is a constant when $d_{30}$ is used as the representative stone size but varies with $d_{85}/d_{15}$ when $d_{50}$ is used.

If it is assumed that the specific weight of the stone is 165 (a typical value for Nashville), the following equation is developed:

$$d_{30}/D = 0.193 \left(\frac{v}{\sqrt{gD}}\right)^{2.5} \tag{3-26}$$

where:

$d_{30}$ = Diameter of stone for which 30 percent, by weight, of the gradation is fine, in feet

D = Depth of flow above stone, in feet

v = Mean point velocity above the stone, in feet

g = Acceleration due to gravity, 32.2 feet/second²

Since $d_{50}$ is the size typically recognized in design, the $d_{30}$ size from Equation 3-26 can be converted to $d_{50}$ by assuming that for typical ranges of $d_{85}/d_{15}$ (2-2.3), $d_{50}$ = 1.20 $d_{30}$.

Correction factors for a different specific weight and for placement in a bend were derived through the use of the concept of <u>effective velocity</u>. The correction factors are used to establish an effective velocity when site-specific conditions deviate from the base condition. The correction factors actually represent a ratio of the effective velocity and the actual velocity. The base conditions are for thickness equal to $d_{100}$ and mean point velocity in a straight channel.

To correct for different specific weights, the following equation is used:

$$C_g = \left(\frac{102.6}{(\gamma_s - 62.4)}\right)^{0.5} \tag{3-27}$$

where:

$C_g$ = Correction coefficient for specific weight

$\gamma_s$ = Specific weight of selected riprap, in pounds/cubic foot

A graphical solution to this equation is presented in Volume 2. Equation 3-26 requires using a point velocity over the toe. For straight channels, an adequate estimate of point velocity is the average velocity in the main portion of the channel. This is different from the mean velocity for the entire channel cross section when flow exceeds the banks of the main channel (e.g., flood plains) and if flow is slowed considerably out to about 0.5 times the main channel depth. Thus, in most cases, the channel should be broken into segments or some rule of thumb used to adjust the mean channel velocity. The ratio of mean channel velocity to point mean velocity over the toe for prismatic channels ranges from 1.1 to 1.3 and for natural channels from 1.3 to 1.5. In all cases, the greater the bank-bed roughness difference and relative flow area above the side slopes, the greater the adjustment required.

A second factor is required to account for the special conditions encountered in a bend. Typically, the point velocity in a bend is about 1.1 to 1.5 times the average velocity for a straight channel section including overbank flow. The U.S. Army Corps of Engineers (1970) provide information from various sources concerning the velocity increase for bends. A graphical approach is presented in Volume 2.

Maynord (1987) also showed that, for a range of $d_{85}/d_{15}$ from 2.0-2.32, a thickness of stone greater than the typical $d_{100}$ could allow for a smaller stone size. A graphical relationship to account for this adjustment is presented in Volume 2.

NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

# CHAPTER 4

# Gutter and Inlet Hydraulics

# TABLE OF CONTENTS

# Chapter 4
# GUTTER AND INLET HYDRAULICS

## SYNOPSIS

Variables of major concern for pavement drainage evaluations include depth of gutter flow and pavement spread. These variables and roadway features such as cross slope, grade, and gutter sections can affect the size, type, and spacing of inlets. This chapter provides fundamentals of gutter flow and inlet interception capacity. Basic terminology is defined and the fundamentals for computing the capacity of curb-opening, gutter, slotted pipe, and combination inlets are presented for continuous grade and sump location conditions. The primary reference for information presented is HEC-12 (USDOT, FHWA, 1984).

## 4.1 FLOW

### 4.1.1 TYPICAL SECTIONS

A pavement gutter is the section of a roadway normally located at its outer edge to convey stormwater runoff. It may include a portion of a travel lane or be a separate section, but it usually has a triangular shape defined by the cross slope and curb.

A typical gutter section includes the following major components:

1. Pavement cross slope, $S_x$
2. Grade, S
3. Width of flow or spread, T
4. Width of depressed gutter flow, W
5. Depth of gutter flow, d
6. Cross slope of depressed gutter, $S_w$

Sketches showing the relationship of these components for three typical gutter sections are presented in Figure 4-1. The first sketch shows a curb and gutter section with a straight cross slope; the second a v-shaped section without a curb; and the third a depressed curb and gutter section of width, W and cross slope, $S_w$.

Figure 4-1

## 4.1.2 MANNING'S EQUATION

Gutter flow is a form of open channel flow that can be analyzed using a modified form of Manning's Equation (see Chapter 3). The modification is necessary because gutter flow typically has a water surface width of more than 40 times the depth of flow. Under such conditions, the hydraulic radius does not properly describe the cross section of flow. To compute gutter flow, Manning's Equation is integrated for an increment of width across this section. The resulting equation is expressed as:

$$Q = \frac{0.56}{n} S_x^{5/3} S^{1/2} T^{8/3} \qquad (4\text{-}1)$$

where:

$Q$ = Gutter flow rate, in cfs

$n$ = Manning's roughness coefficient

$S_x$ = Pavement cross slope, in feet/foot

$S$ = Grade, in feet/foot

$T$ = Width of flow or spread, in feet

Resistance of the curb face is neglected in Equation 4-1. In practice, this omission is valid if the cross slope is 10 percent or less.

Depth of gutter flow or pavement spread are parameters that must be evaluated to properly establish the size, type, bypass, and spacing of inlets. The depth of flow and pavement spread are related mathematically as:

$$d = TS_x \qquad (4\text{-}2)$$

where:

$d$ = Depth of gutter flow, in feet

$T$ = Width of spread, in feet

$S_x$ = Pavement cross slope, in feet/foot

For conditions where the pavement cross slope is curved or parabolic instead of straight, a special adaptation of Equation 4-1 is required to evaluate gutter capacity.

The relative effects of spread, cross slope, and grade on the capacity of a gutter with a straight cross slope are presented in Figure 4-2. Each of the lines is based on the relationship between these variables, as expressed by Equation 4-1. The width of spread is shown to have the greatest impact on gutter capacity, followed by cross slope and, to an even lesser degree, by grade. For example, doubling the spread would increase gutter capacity by 6 times, while doubling cross slope or grade would result in increases of only about 3 and 1.4 times, respectively.

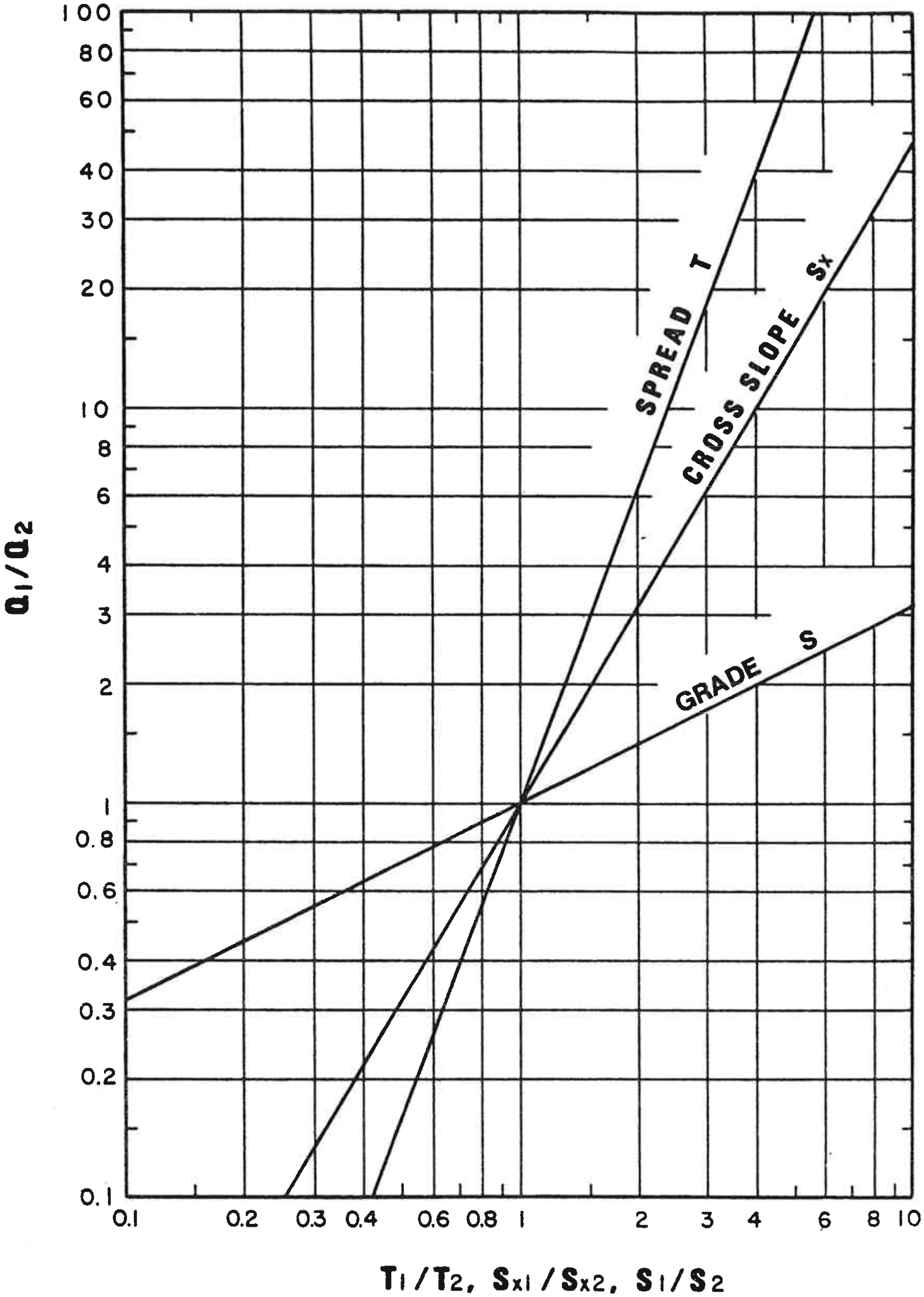
## 4.2 INLET TERMINOLOGY

Inlets for collecting pavement drainage can be divided into the following three major categories:

1. Curb-opening
2. Gutter
3. Combination

Curb-opening inlets are openings in the curb face that are generally placed in a depressed gutter section. Gutter inlets consist of a metal grate or grates placed over an opening in the gutter. A recent modification of the gutter inlet is a slotted pipe that allows pavement drainage to enter continuously along its longitudinal axis. Combination inlets are composed of both curb-opening and gutter inlets. Perspective drawings of inlet types are presented in Figures 4-3 and 4-4.

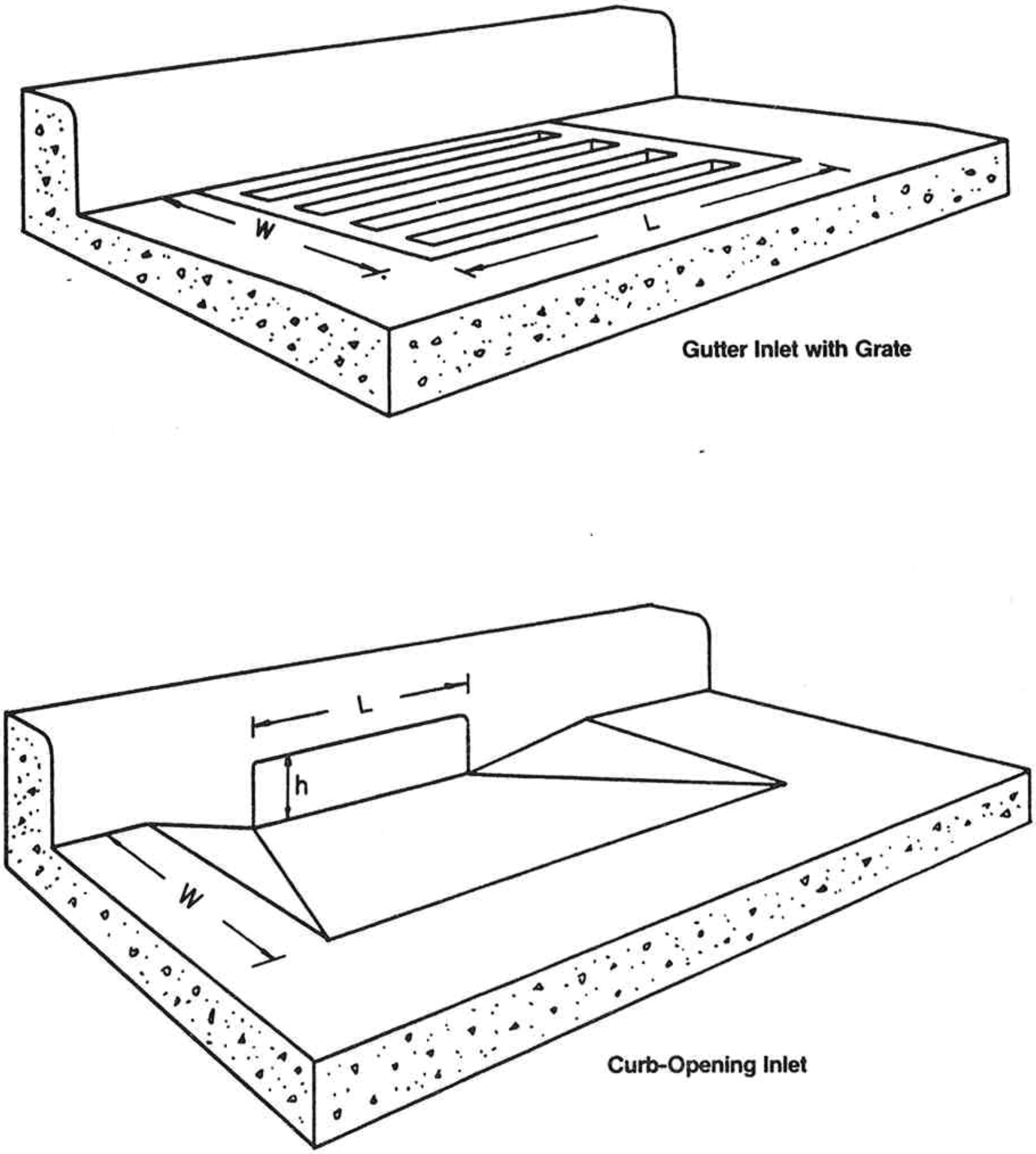
Pavement inlets can be placed either on a continuous grade or in a sump or sag condition. If pavement drainage is intended to enter the inlet from only one longitudinal direction, a continuous grade condition exists. On the other hand, if the inlet is located at a point where flow enters it from two directions, a sump condition exists.




NOTE: A straight cross slope is assumed and Equation 4-1 provides the basis for these relationships.

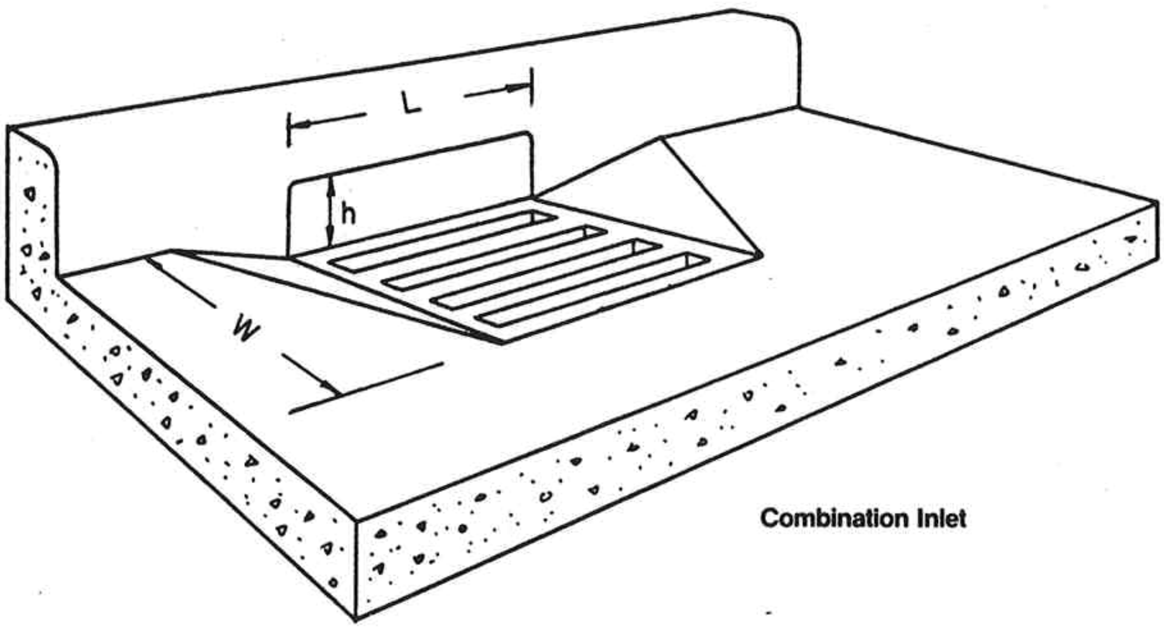
*Reference: USDOT, FHWA, HEC-12 (1984).*

**FIGURE 4-2**
Relative Effects of Spread, Cross Slope, and Grade on Gutter Flow Capacity

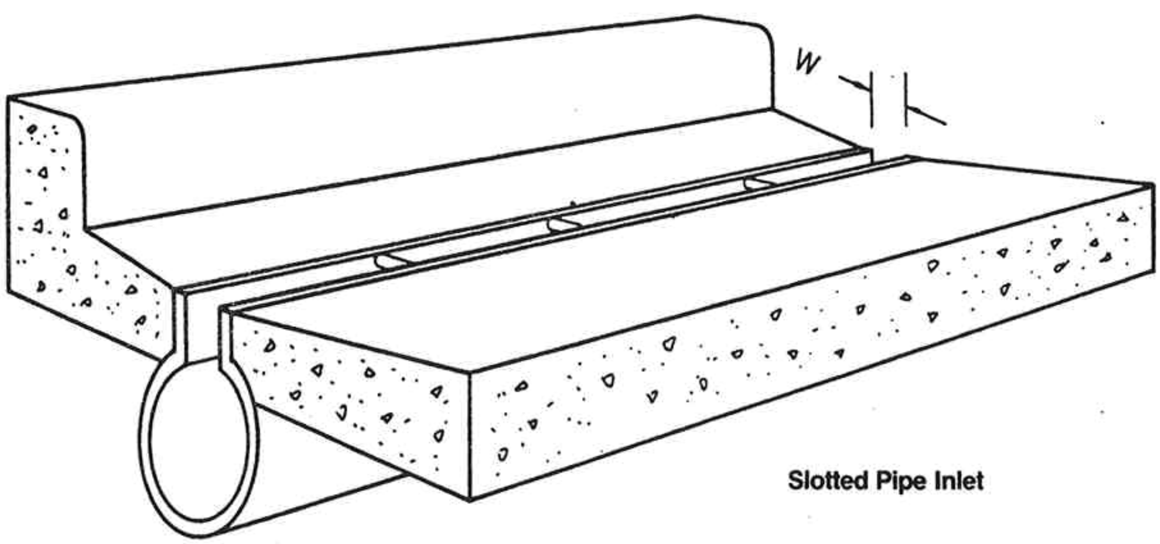



*Reference: USDOT, FHWA, HEC-12 (1984).*

**FIGURE 4-3**
Perspective Views of Gutter and Curb-Opening Inlets




Combination Inlet




Slotted Pipe Inlet

*Reference: USDOT, FHWA, HEC-12 (1984).*

**FIGURE 4-4**
Perspective Views of Combination and Slotted Pipe Inlets

The interception capacity of an inlet is the gutter flow that enters an inlet under a given set of conditions. The capacity changes as those conditions change. Factors affecting the interception capacity of curb-opening and gutter inlets are briefly discussed in Sections 4.3 and 4.4.

The efficiency of an inlet is the percent of total gutter flow that the inlet will intercept for a given set of operating conditions. In mathematical form, efficiency is defined as:

$$E = \frac{Q_i}{Q}(100) \qquad (4\text{-}3)$$

where:

$E$ = Efficiency of an inlet, in percent

$Q_i$ = Intercepted flow, in cfs

$Q$ = Total gutter flow, in cfs

Flow that is not intercepted by an inlet is called bypass, or carryover, and is expressed mathematically as:

$$Q_b = Q - Q_i \qquad (4\text{-}4)$$

where:

$Q_b$ = Bypass flow, in cfs

$Q$ = Total gutter flow, in cfs

$Q_i$ = Intercepted flow, in cfs

In most cases, an increase in total gutter flow causes an increase in the interception capacity of an inlet and a decrease in its efficiency.

Pavement inlets do not provide an efficient method for collecting large quantities of stormwater. Therefore, non-pavement drainage should be collected upstream of the pavement when possible.

## 4.3 CURB-OPENING INLETS

The advantages of curb-opening inlets are that they are less susceptible to clogging and less hazardous to pedestrians and bicyclists than gutter inlets. However, they are not always as efficient.

### 4.3.1 CONTINUOUS GRADE

A curb-opening inlet located on a continuous grade functions as a falling head weir, the efficiency of which is affected by cross slope, grade, total gutter flow, and weir length. The interception capacity of the inlet depends primarily on the flow depth at the curb and the curb-opening geometry. For a given gutter flow, interception capacity and efficiency will be lost as the grade is increased, because depth at the curb decreases as velocity increases. If the curb opening can be depressed several inches below the gutter elevation, the interception capacity of the inlet can be increased. This can be done as part of a continuous gutter depression or as a local depression at the inlet. Interception capacity increases with higher gutter flows, but efficiency decreases.

The length of curb-opening inlet required for total interception of gutter flow on a pavement section with a straight cross slope can be computed using the equation:

$$L_T = 0.6\, Q^{0.42}\, S^{0.3} \left(\frac{1}{n\, S_x}\right)^{0.6} \tag{4-5}$$

where:

$L_T$ = Length required to intercept 100 percent of gutter flow on continuous grade, in feet

$Q$ = Total gutter flow, in cfs

$S$ = Grade of the gutter, in feet/foot

$n$ = Manning's roughness factor

$S_x$ = Cross slope of pavement, in feet/foot

The efficiency of curb-opening inlets shorter than the length required for total interception is computed using the equation:

$$E = 1 - (1 - L/T)^{1.8} \tag{4-6}$$

where:

$E$ = Efficiency of the inlet on a continuous grade, expressed as a decimal

$L$ = Length of curb opening, in feet

$L_T$ = Length required to intercept 100 percent of gutter flow on continuous grade, in feet (see Equation 4-5)

The length of inlet required for total interception by depressed curb-opening inlets or curb openings in depressed gutter sections can be evaluated using Equation 4-5, once the equivalent cross slope is calculated using the equation:

$$S_e = S_x + S'_w E_o \tag{4-7}$$

where:

$S_e$ = Equivalent cross slope, in feet/foot

$S_x$ = Cross slope of pavement, in feet/foot

$S'_w$ = Cross slope of the depressed gutter measured relative to the cross slope = a/12w, in feet/foot

$a$ = Gutter depression, in inches

$w$ = Width of depressed gutter, in feet

$E_o$ = Ratio of the depressed section flow to the total gutter flow, expressed as a decimal (see Equation 4-13)

The equivalent cross slope, $S_e$, determined in Equation 4-7 replaces the cross slope, $S_x$, in Equation 4-5 to calculate the curb-opening length required to intercept 100 percent of gutter flow on continuous grades.

Top slab supports placed flush with the curb face can substantially reduce the interception capacity of curb openings. If intermediate top slab supports are required for structural support, they should be recessed several inches behind the curb face and rounded in shape.

### 4.3.2 SUMP LOCATIONS

Curb-opening inlets in sump locations operate as weirs up to a depth equal to the opening height. At depths above 1.4 times the opening height, the inlet operates as an orifice and, between these depths, a transition from weir to orifice flow occurs. The weir flow equation for a depressed curb-opening inlet is expressed as:

$$Q_i = 2.3 \ (L + 1.8W) d^{1.5} \tag{4-8}$$

where:

$Q_i$ = Interception capacity of a depressed curb-opening inlet operating as a weir and located at a sump, in cfs

$L$ = Length of curb opening, in feet

$W$ = Lateral width of depression, in feet

$d$ = Depth of gutter flow, based on width of spread and cross slope, in feet (see Equation 4-2)

Equation 4-8 is applicable for flow depths less than or equal to the curb-opening height plus the depth of the depression. This limitation is expressed mathematically as:

$$d \leq h + \frac{a}{12} \tag{4-9}$$

where:

$d$ = Depth at curb, measured from normal cross slope, in feet

h = Height of curb-opening inlet, in feet

a = Depth of depression, in inches

Since Equation 4-8 is based on a local depression, it will give conservative capacity estimates for inlets with a continuously depressed gutter.

The weir flow equation for curb-opening inlets without a depressed gutter is expressed as:

$$Q_i = 2.3\ L\ d^{1.5} \tag{4-10}$$

where:

$Q_i$ = Interception capacity of a non-depressed curb-opening inlet operating as a weir and located at a sump, in cfs

L = Length of curb opening, in feet

d = Depth of gutter flow, based on width of spread and cross slope, in feet (see Equation 4-2)

The depth limitation for weir flow represented by Equation 4-10 is expressed as:

$$d \leq h \tag{4-11}$$

where:

d = Depth of gutter flow, based on width of spread and cross slope, in feet (see Equation 4-2)

h = Height of curb opening, in feet

The orifice flow equation for evaluating the capacity of a submerged curb-opening inlet is expressed as:

$$Q_i = 0.67\ h\ L\ (2g\ d_o)^{0.5} \qquad (4\text{-}12)$$

where:

$Q_i$ = Interception capacity of a curb-opening inlet operating as an orifice and located at a sump, in cfs

h = Height of curb opening, including the depression height, a, if appropriate, in feet

L = Length of curb opening, in feet

g = Acceleration due to gravity, 32.2 feet/second²

$d_o$ = Effective head on the center of the orifice throat, in feet (see Figure 4-5)

for horizontal throat:

$$d_o = d_i - h/2$$

for inclined throat:

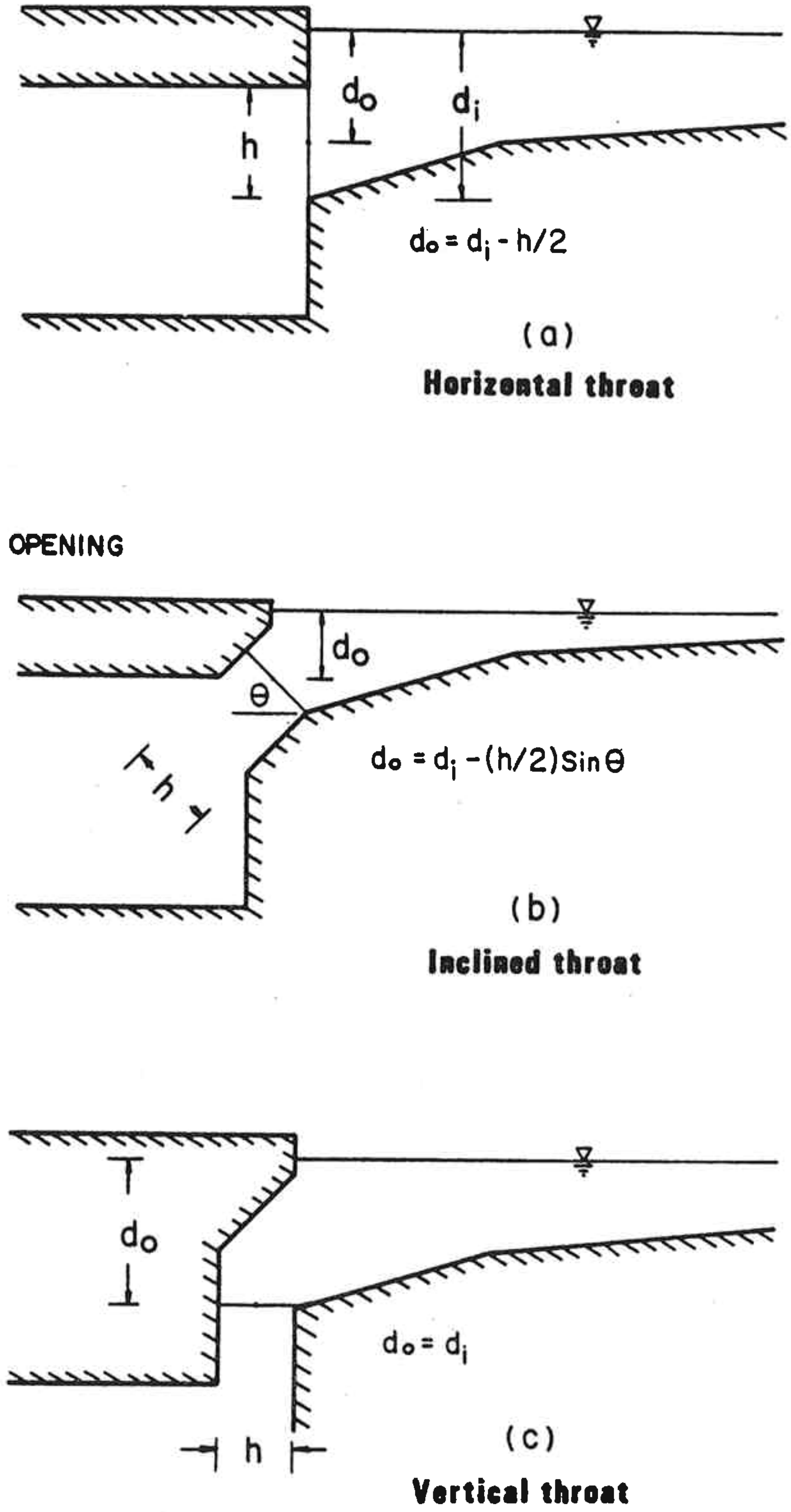
$$d_o = d_i - (h/2)\sin\theta$$

for vertical throat:

$$d_o = d_i$$

$d_i$ = Depth at lip of curb opening, in feet

As identified above, the effective head on the center of the orifice throat is dependent on the geometry of the inlet. Horizontal, inclined, and vertical throat curb-opening inlet geometries are illustrated in Figure 4-5, along with details for determining the effective depth. Equation 4-12 is applicable to depressed and undepressed curb-opening inlets, and the height of the curb opening includes the depression height, if appropriate.




$$Q = 0.87\, hL\sqrt{2gd_o}$$

L = LENGTH OF OPENING

**FIGURE 4-5**
Effective Head for Horizontal, Inclined, and Vertical Throat Curb-Opening Inlets

## 4.4 GUTTER INLETS

Gutter inlets usually have one or more metal grates covering an opening in the gutter. Such inlets may not operate effectively where a potential exists for clogging from stormwater-carried debris.

### 4.4.1 CONTINUOUS GRADE

The water flowing in the section of a gutter inlet occupied by the grate is called frontal flow. When the gutter flow velocity is low enough, the grate inlet intercepts all of the frontal flow and a small portion of the side flow, which occurs along the length of the grate. As the gutter flow velocity increases, water may begin to skip, or splash over, the grate and the efficiency of the inlet may be reduced. If splashover does not occur, the capacity and efficiency of a gutter inlet will increase with an increase of the grade (the reverse of the effect on curb-opening and slotted inlets). This is because frontal flow increases with increased grade and all frontal flow will be intercepted if splashover does not occur.

The ratio of frontal flow to total gutter flow for a straight cross slope (see Figure 4-1) is expressed as:

$$E_o = \frac{Q_w}{Q} = 1 - (1 - W/T)^{2.67} \qquad (4\text{-}13)$$

where:

$E_o$ = Ratio of frontal flow to total gutter flow

$Q_w$ = Frontal flow in width, W, in cfs

$Q$ = Total gutter flow, in cfs

$W$ = Width of gutter inlet or grate (see Figure 4-3), in feet

$T$ = Total spread of water in the gutter (see Figure 4-1), in feet

The ratio of side flow to total gutter flow is expressed as:

$$\frac{Q_s}{Q} = 1 - \frac{Q_w}{Q} = 1 - E_o \qquad (4\text{-}14)$$

where:

$Q_s$ = Side flow intercepted by gutter inlet, in cfs

$Q$ = Total gutter flow, in cfs

$Q_w$ = Frontal flow in width, W, in cfs

$E_o$ = Ratio of frontal flow to total gutter flow (see Equation 4-13)

The ratio of intercepted frontal flow to total frontal flow is expressed as:

$$R_f = 1 - 0.09\ (v - v_o) \qquad (4\text{-}15)$$

where:

$R_f$ = Ratio of intercepted frontal flow to total frontal flow

$v$ = Average velocity of gutter flow, in feet/second

$v_o$ = Average gutter velocity where splashover first occurs, in feet/second

The ratio of intercepted side flow to total side flow is expressed as:

$$R_s = 1/\left[1 + \frac{0.15\ v^{1.8}}{S_x\ \ L^{2.3}}\right] \qquad (4\text{-}16)$$

where:

$R_s$ = Ratio of intercepted side flow to total side flow

$v$ = Average velocity of gutter flow, in feet/second

$S_x$ = Cross slope of gutter, in feet/foot

$L$ = Length of grate, in feet

The operating efficiency of a gutter inlet with a grate located on a continuous grade is expressed as:

$$E = R_f E_o + R_s (1 - E_o) \qquad (4\text{-}17)$$

where:

$E$ = Efficiency of the gutter inlet on a continuous grade

$R_f$ = Ratio of intercepted frontal flow to total frontal flow (see Equation 4-15)

$E_o$ = Ratio of frontal flow to total gutter flow (see Equation 4-13)

$R_s$ = Ratio of intercepted side flow to total side flow (see Equation 4-16)

The interception capacity of a gutter inlet with a grate located on a continuous grade is equal to the efficiency of the inlet multiplied by the total gutter flow, which can be expressed as:

$$Q_i = EQ = Q [R_f E_o + R_s (1 - E_o)] \qquad (4\text{-}18)$$

where:

$Q_i$ = Interception capacity of the gutter inlet on a continuous grade, in cfs

$E$ = Efficiency of the gutter inlet on a continuous grade (see Equation 4-17)

$Q$ = Total gutter flow, in cfs

$R_f$ = Ratio of intercepted frontal flow to total flow (see Equation 4-15)

$E_o$ = Ratio of frontal flow to total gutter flow (see Equation 4-13)

$R_s$ = Ratio of intercepted side flow to total side flow (see Equation 4-16)

### 4.4.2 SUMP LOCATIONS

Gutter inlets in sump locations operate as weirs up to depths that are dependent on grate size and configuration, and as orifices at greater depths. A transition occurs between weir and orifice flow depths. In this transition, the capacity is ill-defined and may fluctuate between weir and orifice control.

The efficiency of inlets in passing debris is critical in sump locations. If the inlet plugs, hazardous ponding conditions can result. Since gutter inlets with grates tend to clog, it is generally beneficial to place a curb-opening inlet behind each grate to intercept debris upstream of the gate. This is known as a combination inlet.

The interception capacity of a gutter inlet with a grate operating as a weir in a sump location is expressed as:

$$Q_i = 3.0\ P\ d^{1.5} \tag{4-19}$$

where:

$Q_i$ = Interception capacity of a sump gutter inlet operating as a weir, in cfs

P = Perimeter of the grate, disregarding bars and the curb side, in feet

d = Depth of water above the top of the grate, in feet

The interception capacity of a gutter inlet with a grate operating as an orifice in a sump location is expressed as:

$$Q_i = 0.67\ A\ (2gd)^{0.5} \qquad (4-20)$$

where:

$Q_i$ = Interception capacity of a sump gutter inlet operating as an orifice, in cfs

A = Clear opening area of the grate, in square feet

g = Acceleration due to gravity, 32.2 feet/second²

d = Depth of water above the top of the grate, in feet

The transition zone from weir flow to orifice flow for gutter inlets in sump locations is typically between a depth of about 0.4 to 1.4 feet. However, this transition zone is known to vary depending on perimeter and clear opening area. The likely transition zone for a specific gutter inlet can be evaluated using interception capacity curves presented in Volume 2. These curves establish approximate limits of the transition zone based on experimental test results (USDOT, FHWA, HEC-12, 1984).

## 4.5 SLOTTED PIPE INLETS

Slotted pipe is a version of the gutter inlet that allows pavement drainage to enter the pipe continuously along its longitudinal axis. Slotted pipes can be used on curbed or uncurbed pavement and present minimal interference to traffic and pedestrians, but are susceptible to clogging.

### 4.5.1 CONTINUOUS GRADE

When placed on a continuous grade, slotted pipe inlets must be placed parallel to the direction of flow. If they are placed perpendicular to flow, splashover will occur and interception will be minimal. The interception capacity of slotted pipe inlets on a continuous grade has been found to be hydraulically similar to that of a curb-opening inlet.

Both inlets function as falling head weirs, with the flow subjected to lateral acceleration caused by the pavement cross slope.

The analysis of test data collected by the USDOT, FHWA (HEC-12, 1984) for slot widths greater than 1.75 inches indicates that the length of inlet required for complete interception of gutter flow can be computed using Equation 4-5, developed for curb-opening inlets.

The efficiency of a slotted pipe inlet on a continuous grade with a length shorter than that required for total interception can be computed using Equation 4-6, developed for curb-opening inlets.

### 4.5.2 SUMP LOCATIONS

Slotted pipe inlets at sump locations perform as weirs to depths of about 0.2 foot, depending on slot width and length. At depths greater than about 0.4 foot, they perform as orifices. A transition zone occurs between these two depths. The following equation can be used to estimate the capacity of a slotted pipe inlet operating as an orifice:

$$Q_i = 0.8\ L\ W\ (2gd)^{0.5} \qquad (4\text{-}21)$$

where:

$Q_i$ = Interception capacity of a slotted pipe inlet operating as an orifice, in cfs

$L$ = Length of slotted pipe inlet, in feet

$W$ = Width of slot, in feet

$d$ = Depth of water at slot, in feet ($d \geq 0.4$ foot for orifice flow)

$g$ = Acceleration due to gravity, 32.2 feet/second$^2$

For a slot width of 1.75 inches, Equation 4-21 becomes:

$$Q_i = 0.94\ L\ (d)^{0.5} \qquad (4\text{-}22)$$

For depths between 0.2 and 0.4 foot, the interception capacity of slotted pipe inlets can be computed with Equation 4-20; for depths less than 0.2 foot, Equation 4-19 is used.

## 4.6 COMBINATION INLETS

### 4.6.1 CONTINUOUS GRADE

On a continuous grade, the capacity of an unclogged combination inlet with the curb opening located next to the grate is approximately equal to the capacity of the grate inlet alone. A combination inlet with the curb opening located upstream of the grate has an interception capacity equal to the sum of the two inlets, except that the frontal flow of the grate is reduced by the amount of flow intercepted by the curb opening. By placing the curb opening upstream, debris is generally intercepted before it clogs the grate.

### 4.6.2 SUMP LOCATIONS

All debris carried by stormwater runoff that is not intercepted by upstream inlets will be concentrated at the inlet located at the low point or sump. Because this will increase the probability of clogging for grated inlets, it is generally appropriate to estimate the capacity of a combination inlet at a sump by neglecting the grate inlet capacity.

NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

# CHAPTER 5
## Culvert Hydraulics

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# Chapter 5
# CULVERT HYDRAULICS

## SYNOPSIS

A culvert is a hydraulically short conduit that conveys stormwater through a roadway embankment or past some type of flow obstruction. Culverts generally do not form a part of the traveled roadway and have a span of 20 feet or less. Conversely, the opening between inside exterior walls of a bridge generally exceeds 20 feet and the bridge span, which generally rests on abutments, is part of the traveled roadway. Whereas bridges are usually designed to provide freeboard for design event conditions that can handle boat traffic, culverts are designed to flow full or have a submerged inlet during the design flood.

During a given storm event, a culvert may operate under inlet control, outlet control, or both. This chapter provides basic theoretical information on the different variables and equations that determine the culvert capacity for each type of control. A brief discussion of improved inlets concludes the chapter. The primary reference for the information presented is HDS-5 (USDOT, FHWA, 1985).

## 5.1 FUNDAMENTALS

Theoretical analysis of culvert hydraulics is extremely complex, because flow is usually nonuniform, with regions of both gradually varying and rapidly varying flow. Exact analyses might require backwater and drawdown calculations, the balancing of energy and momentum, and the use of physical models. In practice, the results of numerous physical tests and theoretical calculations performed for the Federal Highway Administration (USDOT, FHWA, HDS-5, 1985) are presented in the form of culvert capacity nomographs.

To apply these nomographs, common types of flow are classified and analyzed on the basis of a control section. A control section is a location where a unique relationship exists between the rate of flow and depth of flow or water surface elevation. The two basic types of control sections defined by research are termed inlet and outlet control.

Inlet control exists when the culvert barrel is capable of conveying more flow than the inlet will accept. The control section for this condition is located just inside the entrance. Critical depth occurs at or near this location and the flow in the culvert is supercritical. The entrance water surface elevation and inlet geometry (barrel shape, cross-sectional area, and inlet edge) are the variables influencing culvert performance.

Flow under inlet control may be described mathematically by either the weir formula or the orifice formula, depending on the headwater depth. A weir is an edge or surface over which water flows, while an orifice is an opening with a closed perimeter through which water flows. If the perimeter of an orifice is not closed, or if the opening flows only partially full, the orifice operates as a weir.

Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet will accept. The control section for this situation is located at the barrel exit or downstream from the culvert. Either partially full subcritical flow or full pipe pressure flow conditions can occur. In addition to the variables influencing inlet performance, the slope, length, and roughness of the culvert barrel and the water surface elevation at the outlet (tailwater) can affect outlet performance. The variables influencing culvert performance are summarized in Table 5-1.

For inlet control, the tailwater elevation has no influence on performance. For outlet control, the difference between headwater and tailwater elevation represents the energy that conveys the flow through the culvert.

In most situations, the hydraulic sizing of a culvert is a trial and error process. A trial culvert size is assumed and inlet and outlet performance are evaluated to determine if they will satisfy the conditions prevailing at the proposed location. A culvert system is selected by choosing an inlet structure; barrel material, shape, and size; and an outlet structure. The inlet and outlet structures are usually the same, to achieve a symmetrical installation. If the outlet velocity is high enough to cause erosion, protection or energy dissipation is required.

Table 5-1
VARIABLES INFLUENCING CULVERT PERFORMANCE

| Variable | | Inlet Control | Outlet Control |
|---|---|---|---|
| 1. | Headwater Elevation | X | X |
| 2. | Inlet Area | X | X |
| 3. | Inlet Edge Configuration | X | X |
| 4. | Inlet Shape | X | X |
| 5. | Barrel Roughness | | X |
| 6. | Barrel Area | | X |
| 7. | Barrel Shape | | X |
| 8. | Barrel Length | | X |
| 9. | Barrel Slope | a | X |
| 10. | Tailwater Elevation | | X |

[a]Barrel slope has only a small effect on inlet control performance and is usually neglected.

## 5.2 INLET CONTROL

When a culvert is operating under inlet control, flow is supercritical and the barrel (outlet) has a greater hydraulic capacity than the inlet. For this reason, culvert capacity depends primarily on the inlet properties, with minimal effect from barrel properties. Although a steep slope may increase inlet capacity by a small amount, in practice this increase can be considered insignificant and should be neglected unless slope-tapered improvements are provided (see Section 5.6).

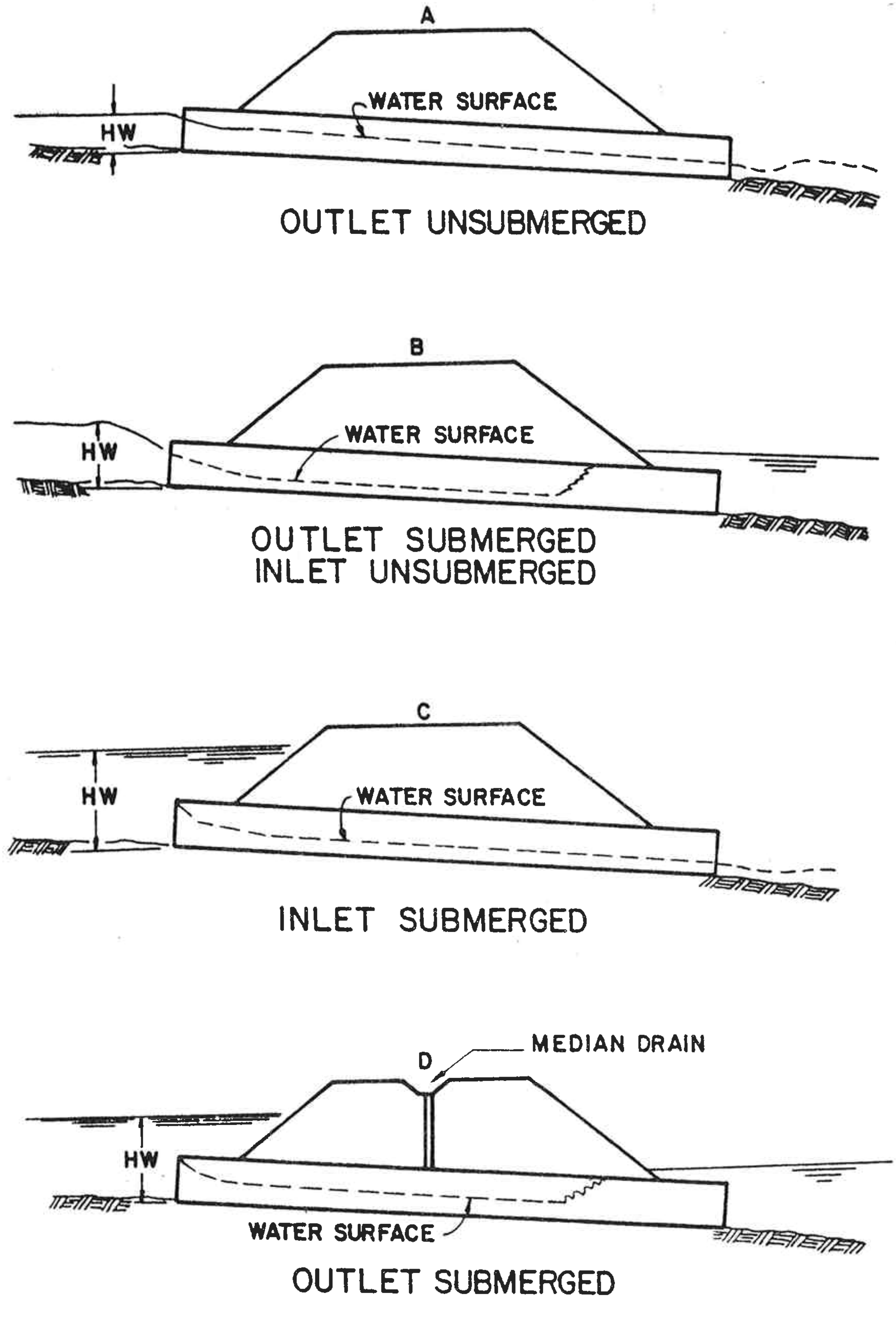
### 5.2.1 SUBMERGENCE CONFIGURATIONS

The four configurations of inlet control illustrated in Figure 5-1 present various combinations of submergence at the inlet and outlet ends of the culvert. When the depth of water approaching the culvert is less than the culvert height (Figure 5-1, Part A), the flow rate is governed by weir control. When the entrance is submerged and the control is at the inlet, the flow will be governed by orifice flow (Figure 5-1, Parts C and D).

If the culvert outlet is not submerged (Figure 5-1, Parts A and C), the barrel flows partially full over its length, and flow approaches normal depth at the outlet. As shown in Parts B and D of Figure 5-1, submergence of the outlet does not ensure outlet control. In both cases, a hydraulic jump forms in the barrel, allowing flow to pass from supercritical to subcritical conditions. If the configuration shown in Part D of Figure 5-1 were not ventilated, subatmospheric pressures could develop, possibly creating an unstable condition in which the barrel would alternate between full and partially full flow.

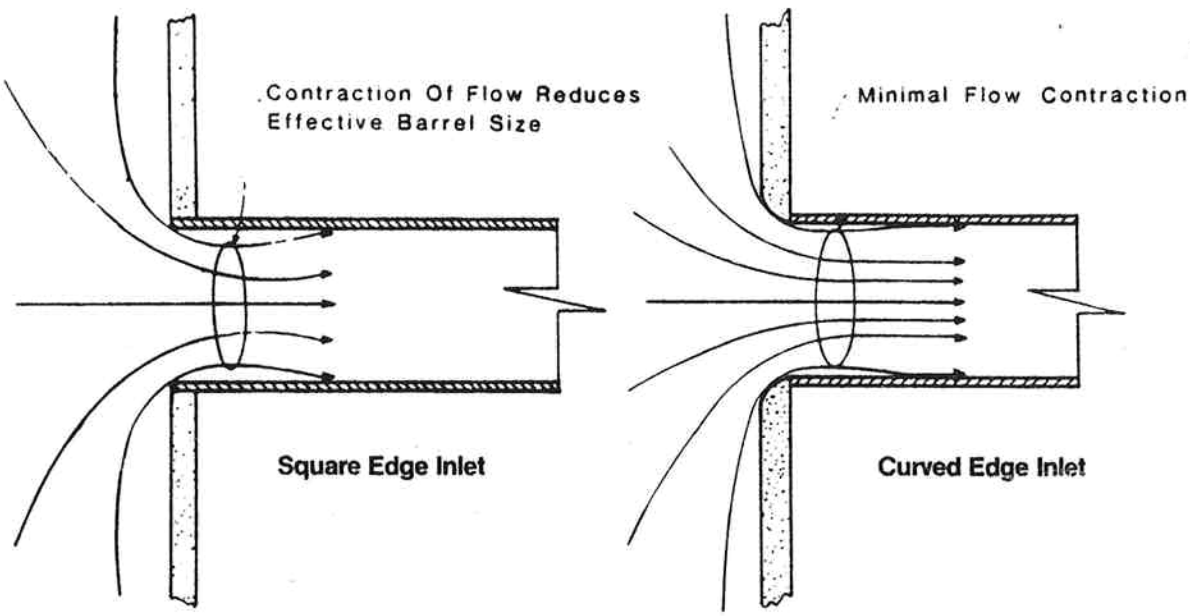
### 5.2.2 PERFORMANCE VARIABLES

As listed in Table 5-1, only the headwater and inlet configuration influence inlet control performance. Components of the inlet configuration include the area, edge configuration, and shape. The <u>inlet area</u> is the cross-sectional area of the culvert face. This area is the same as the barrel area, except when tapered inlets are used to enlarge the face relative to the barrel. The effect of the <u>edge configuration</u> is illustrated in Figure 5-2. Because the



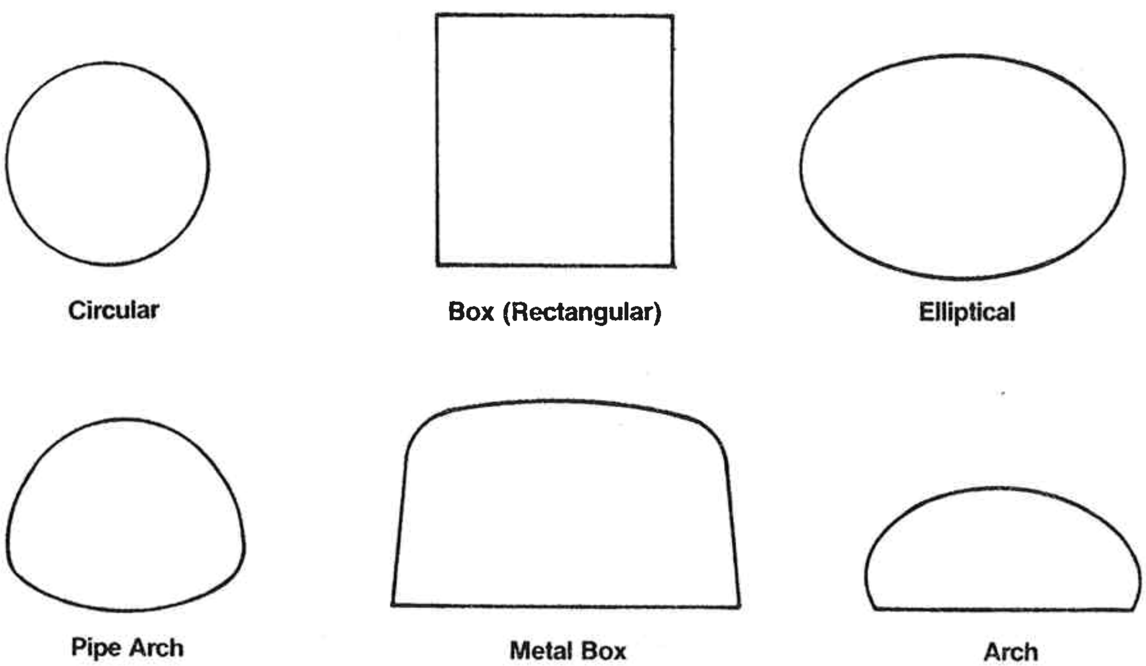

*Reference: USDOT, FHWA, HDS-5 (1985).*

**FIGURE 5-1**
Types of Inlet Control




*Reference: USDOT, FHWA, HDS-5 (1985).*

**FIGURE 5-2**
Flow Contraction at Square Edge and Curved Edge Culvert Inlets




*Reference: USDOT, FHWA, HDS-5 (1985).*

**FIGURE 5-3**
Common Culvert Shapes

contraction of flow occurring at the culvert inlet reduces the effective barrel size, edge configurations that minimize flow contraction will increase culvert capacity for both inlet and outlet control conditions.

The inlet shape is the same as the culvert barrel, except when tapered inlets are used to enlarge the inlet. The most commonly used shapes, illustrated in Figure 5-3, are circular, box (rectangular), elliptical, pipe arch, metal box, and arch. Factors affecting shape selection include cost, allowable headwater, embankment height, and hydraulic performance. If the areas for two different culvert shapes are equal, the lower profile inlet (e.g., arch vs. circular) will have more capacity for the same headwater, because the head above the culvert crown is greater.

A common method for increasing inlet performance is the use of beveled edges at the entrance. Although any beveling helps performance, the three edge configurations reported in design procedures are 33.7-degree bevels (1 inch per foot of barrel width), 45-degree bevels (1/2 inch per foot of barrel width), and grooved end (socket) of concrete pipe. All options are considered equal for design purposes, but the larger 33.7-degree bevels provide slightly better inlet performance.

### 5.2.3 FLOW VERSUS HEADWATER

The headwater or depth of ponding at the culvert entrance is a major variable affecting inlet capacity. The headwater depth, HW, is the vertical distance from the culvert invert at the entrance to the energy line of the headwater pool (depth plus velocity head). Because of the low velocities in most entrance pools and the difficulty in determining the velocity head for all flows, the approach velocity is usually ignored and the water surface and energy line at the entrance are assumed to be coincident. For the purposes of measuring headwater, the culvert invert at the entrance is the low point in the culvert opening at the beginning of the full cross section of the culvert barrel.

The three regions of flow versus headwater that occur at culvert inlets are weir (unsubmerged), transition, and orifice (submerged) flow. The transitional zone between weir and orifice flow depends on inlet geometry and normally lies between a submergence ratio of 1.2 to 1.5. (Submergence ratio is the headwater depth divided by the culvert height.) Although mathematical relationships can be used to calculate weir and orifice flow rates, experimental test results from physical models are available for most culvert inlet configurations (USDOT, FHWA, HDS-5, 1985). These results are available in the form of design nomographs, which are presented in Volume 2.

## 5.3 OUTLET CONTROL

Culverts under outlet control can flow with the culvert barrel full or partially full for all or part of the barrel length. Full flow outlet control conditions are shown in Figure 5-4, Parts A, B, and C, while partially full outlet control conditions are shown in Figure 5-4, Parts D and E.

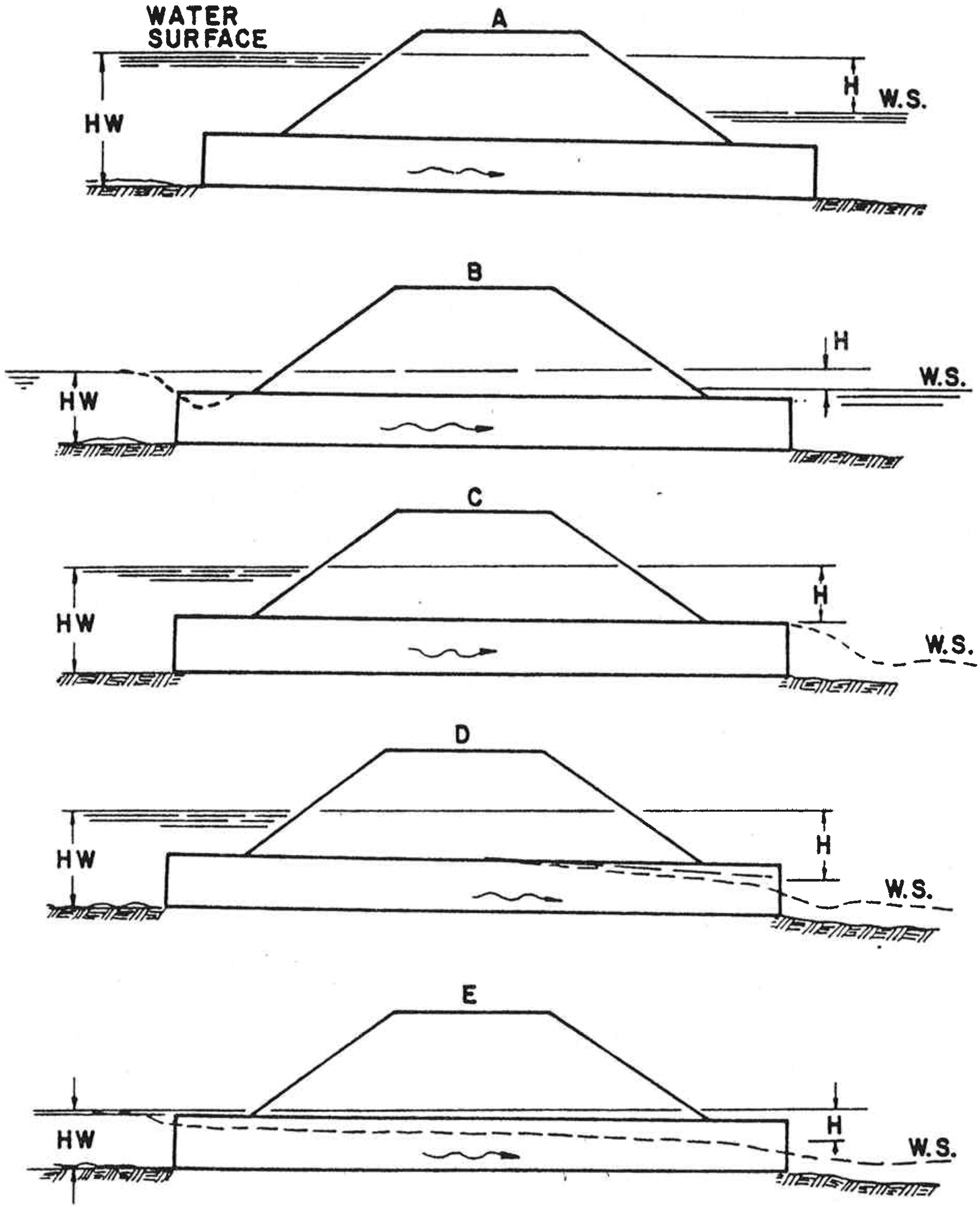
### 5.3.1 SUBMERGENCE CONFIGURATIONS

The five configurations of outlet control illustrated in Figure 5-4 present various combinations of submergence at the inlet and outlet ends of the culvert. Part A represents the classic full flow condition, with both inlet and outlet submerged. The barrel is in pressure flow throughout its length. This condition is suitable for most calculations.

Part B depicts the outlet submerged with the inlet unsubmerged. For this case, the headwater is shallow, so that the inlet crown is exposed as the flow contracts into the culvert.

Part C shows the entrance submerged to such a degree that the culvert flows full throughout its entire length while the exit is unsubmerged. This condition is rare, as it requires an extremely high headwater to maintain full barrel flow with no tailwater. The outlet velocities are usually high under this condition.

In Part D, the entrance is submerged by the headwater and the outlet flows freely with a low tailwater. For this condition, the barrel flows partially full over at least part of its length (subcritical flow) and the flow passes through critical depth just upstream of the outlet.




Reference: USDOT, FHWA, HDS-5 (1985).

**FIGURE 5-4**
Types of Outlet Control

Part E shows neither the inlet nor the outlet end of the culvert submerged. The barrel flows partially full over its entire length, and the flow profile is subcritical.

### 5.3.2 PERFORMANCE VARIABLES

The variables influencing the performance of a culvert in inlet control also influence culverts in outlet control. In addition, the barrel characteristics (roughness, area, shape, length, and slope) and the tailwater elevation affect culvert performance in outlet control (see Table 5-1).

The barrel roughness is a function of the material used to fabricate the barrel. Typical materials include concrete and corrugated metal. The roughness is represented by Manning's n value. Typical values for culverts are presented in Volume 2.

The barrel area and barrel shape are the same as the inlet, unless a tapered inlet is used. The barrel length is the total culvert length from the entrance to the exit of the culvert. As the length increases, the head loss caused by friction increases. The barrel slope is the actual slope of the culvert barrel and is often the same as the natural stream slope, unless the culvert inlet is raised or lowered.

The tailwater elevation is based on the downstream water surface elevation. Backwater calculations from a downstream control, a normal depth approximation, or field observations can be used to define the tailwater elevation.

### 5.3.3 FULL FLOW PERFORMANCE

Performance calculations for full flow outlet control can be made by accounting for energy losses resulting from entrance losses, friction losses, and outlet losses. These losses are expressed mathematically as:

$$H = H_e + H_f + H_o \qquad (5\text{-}1)$$

where:

$H$ = Total head, or the elevation difference between the headwater, HW, and tailwater, TW, in feet

$H_e$ = Entrance loss, in feet

$H_f$ = Friction loss, in feet

$H_o$ = Outlet loss, in feet

To evaluate the components of Equation 5-1, the average culvert velocity and velocity head are calculated using the equations:

$$v = Q/A \tag{5-2}$$

$$H_v = v^2/2g \tag{5-3}$$

where:

v = Average velocity in the culvert barrel, in feet/second

Q = Flow rate, in cfs

A = Cross-sectional area of culvert flow, in square feet

$H_v$ = Velocity head, in feet

g = Acceleration due to gravity, 32.2 feet/second²

Using the velocity head, each component of Equation 5-1 can be calculated using the equations:

$$H_e = k_e \left(\frac{v^2}{2g}\right) \tag{5-4}$$

$$H_f = \left(\frac{29\ n^2\ L}{R^{1.33}}\right) \frac{v^2}{2g} \tag{5-5}$$

$$H_o = 1.0 \left[\frac{v^2}{2g} - \frac{v_d^2}{2g}\right] \tag{5-6}$$

where:

$k_e$ = Entrance loss coefficient (design values reported in Volume 2)

$n$ = Manning's roughness coefficient (design values reported in Volume 2)

$L$ = Length of culvert barrel, in feet

$R$ = Hydraulic radius of full culvert barrel = A/P, in feet

$A$ = Cross-sectional area of full culvert flow, in square feet

$P$ = Perimeter of culvert barrel, in feet

$v$ = Average velocity in the culvert barrel, in feet/second

$v_d$ = Downstream channel velocity, in feet/second

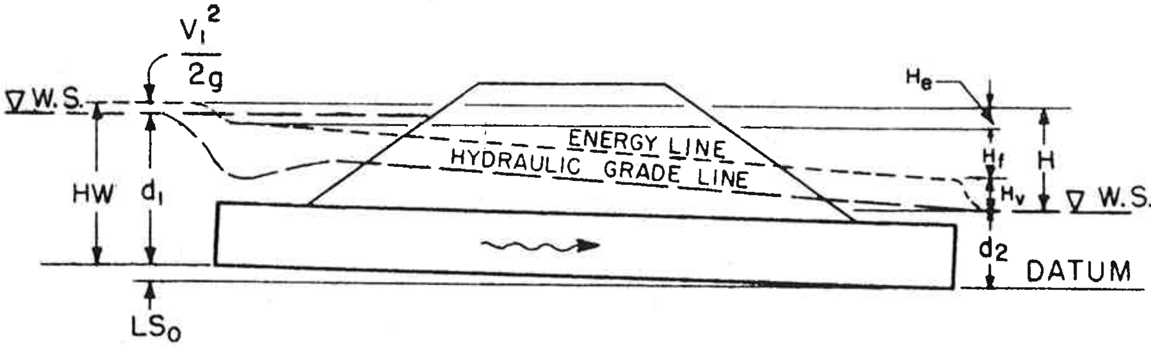
$g$ = Acceleration due to gravity, 32.2 feet/second²

Since the downstream channel velocity can usually be neglected, Equation 5-1 becomes:

$$H = \left[1.0 + k_e + \frac{29\ n^2\ L}{R^{1.33}}\right] \frac{v^2}{2g} \qquad (5\text{-}7)$$

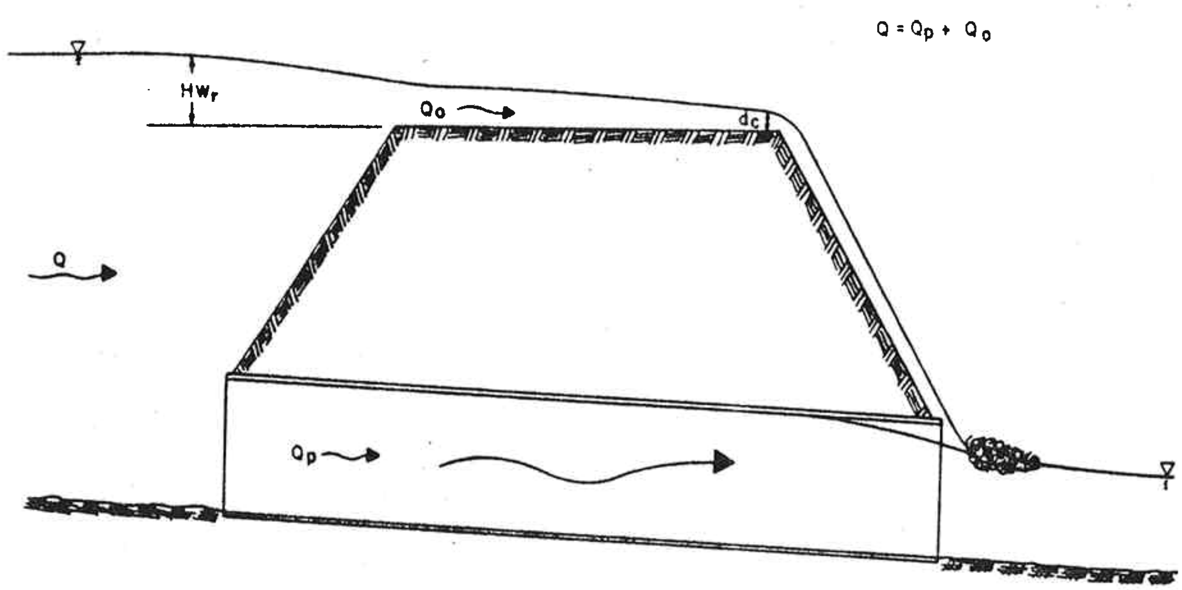
It may be necessary to build a culvert that has one or more bends in the alignment. If such a culvert is operating under outlet control, then losses caused by bends should be added to Equation 5-7. Theoretical aspects of evaluating head loss caused by bends are presented in Chapter 6.

Figure 5-5 illustrates the terms of Equation 5-7, the energy line, the hydraulic grade line, and the headwater depth, HW. The energy line represents the total energy at any point along the culvert barrel. The hydraulic grade line, or pressure line, is defined by the elevations to which water




*Reference: USDOT, FHWA, HDS-5 (1985).*

**FIGURE 5-5**
Full Flow Outlet Control Energy and Hydraulic Gradients




*Reference: USDOT, FHWA, HDS-5 (1985).*

**FIGURE 5-6**
Variables for Evaluating Roadway Overtopping

would rise in small vertical pipes attached to the culvert wall along its length. The energy line and the pressure line are parallel over the length of the barrel, except in the immediate vicinity of the inlet where the flow contracts and re-expands. The difference in elevation between these two lines is the velocity head, $v^2/2g$.

As shown in Figure 5-5, the head, H, is the difference between the elevations of the hydraulic grade line at the outlet and the energy line at the inlet (neglecting downstream velocity). Headwater depth is the vertical distance from the culvert invert at the entrance to the water surface, assuming the water surface (hydraulic grade line) and the energy line to be coincident. Since the velocity head at the inlet is usually small under ponded conditions, the water surface or headwater pool elevation is assumed to equal the elevation of the energy line. Thus, headwater depths based on a zero approach velocity are conservative.

Having established the total head loss, H, the headwater depth, HW, for outlet control can be computed as:

$$HW = H + h_o - LS_o \tag{5-8}$$

where:

HW = Headwater depth for outlet control, in feet

H = Total head, in feet (see Equation 5-7)

$h_o$ = Design tailwater, in feet

L = Length of culvert barrel, in feet

$S_o$ = Barrel slope, in feet/foot

Outlet control nomographs for full pipe flow, presented in Volume 2, provide graphical procedures to evaluate the total head loss, H, for various culvert materials, cross sections, and inlet combinations (USDOT, FHWA, HDS-5, 1985). The depth of water at the culvert outlet due to downstream conditions is termed the <u>tailwater</u>, TW. The tailwater condition that prevails during the design event is called the design tailwater, $h_o$. The design tailwater may be a function of either downstream or culvert outlet conditions.

The tailwater depth is measured from the invert of the culvert to the water surface elevation at the outlet and can be influenced by conditions downstream of the outlet. If the outlet is operating in a free outfall condition, the tailwater may be equal to critical depth for the culvert. If the culvert discharges into an open channel, the tailwater may be equal to the normal depth of flow in that channel. If the culvert outlet is located near the inlet of a downstream culvert, the headwater elevation of the downstream culvert may define tailwater depth for the upstream culvert.

## 5.3.4 PARTIALLY FULL FLOW PERFORMANCE

Backwater calculations may be required for the partially full flow conditions shown in Figure 5-4, Parts D and E. These calculations begin at the water surface at the downstream end of the culvert and proceed upstream to the entrance of the culvert. The downstream water surface is based on critical depth at the culvert outlet or on the tailwater depth, whichever is higher. If the calculated backwater profile intersects the top of the barrel, as in Figure 5-4, Part D, a straight, full flow hydraulic grade line extends from that point upstream to the culvert entrance. From Equation 5-5, the full flow friction slope is:

$$S_n = \frac{H_f}{L} = \left(\frac{29\ n^2}{R^{1.33}}\right)\frac{V^2}{2g} \qquad (5\text{-}9)$$

where:

$S_n$ = Full flow friction slope, in feet/foot

$H_f$ = Friction loss, in feet

$L$ = Length of culvert barrel, in feet

$n$ = Manning's roughness coefficient (design values reported in Volume 2)

$R$ = Hydraulic radius of full culvert barrel = A/P, in feet

A = Cross-sectional area of full culvert flow, in square feet

P = Perimeter of culvert barrel, in feet

v = Average velocity in the culvert barrel, in feet/second

g = Acceleration due to gravity, 32.2 feet/second$^2$

To avoid tedious backwater calculations, approximate methods have been developed to analyze partially full flow conditions. Based on numerous backwater calculations reported by the USDOT, FHWA, in HDS-5 (1985), it was found that a downstream extension of the full flow hydraulic grade line for the flow condition shown in Figure 5-4, Part D, pierces the plane of the culvert outlet at a point one-half way between critical depth and the top of the barrel. Therefore, it is possible to begin the hydraulic grade line at the equivalent hydraulic depth of $(d_c + D)/2$ above the outlet invert and extend the straight, full flow hydraulic grade line upstream to the inlet of the culvert at a slope of $S_n$. If the tailwater exceeds $(d_c + D)/2$, the tailwater is used to set the downstream end of the extended full flow hydraulic grade line. The inlet losses and the velocity head are added to the elevation of the hydraulic grade line at the inlet to obtain the headwater elevation.

This approximate method works best when the barrel flows full over at least part of its length (Figure 5-4, Part D). When the barrel is partially full over its entire length (Figure 5-4, Part E), the method becomes increasingly inaccurate as the headwater falls further below the top of the barrel at the inlet. Adequate results are obtained down to a headwater of 0.75D. For lower headwaters, backwater calculations are required to obtain accurate headwater elevations.

## 5.4 ROADWAY OVERTOPPING

The broad-crested weir equation is used to evaluate flow over the low point of a roadway. The equation is expressed as:

$$Q_o = C_d \, L \, HW_r^{1.5} \tag{5-10}$$

where:

$Q_o$ = Overtopping flow rate, in cfs

$C_d$ = Overtopping discharge coefficient

L = Length of the roadway crest, in feet

$HW_r$ = Upstream depth, measured from the roadway crest to the water surface upstream of the weir drawdown, in feet

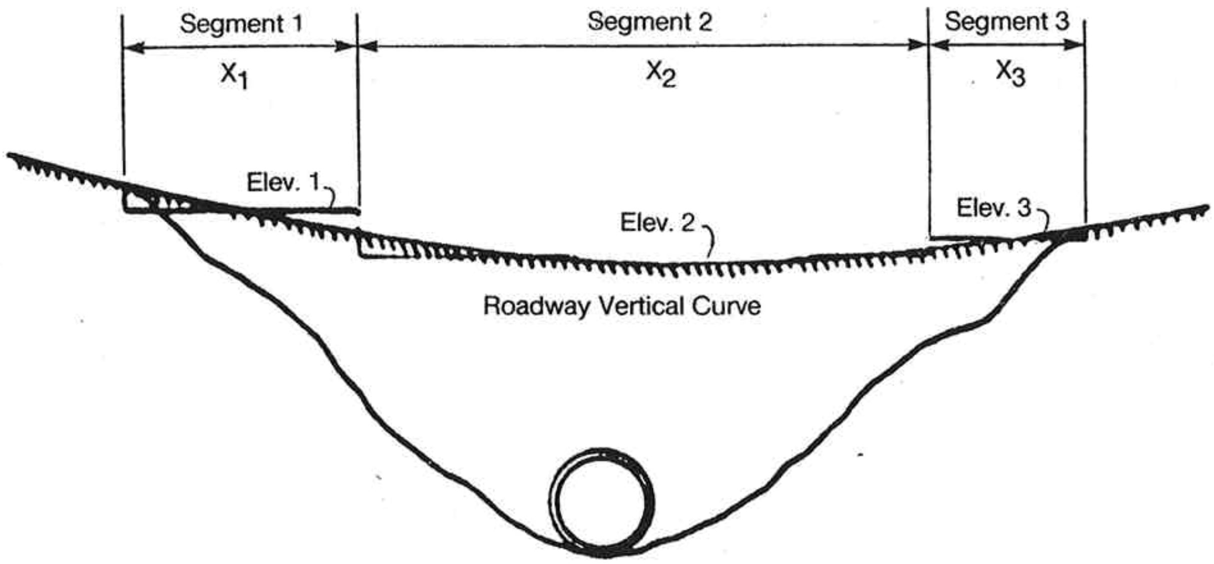
Variables of Equation 5-10, other than the discharge coefficient, are illustrated in Figure 5-6. The total flow, Q, consists of the pipe flow, $Q_p$, and the overtopping flow, $Q_o$. Graphical information is presented in Volume 2 for selecting appropriate discharge coefficient values.

When the roadway crest is defined by a sag vertical curve, the two methods illustrated in Figure 5-7 are suggested for setting the length and crest elevation. Method 1, shown in Part A of Figure 5-7, involves dividing the roadway vertical curve into a series of horizontal segments to approximate the curve of the roadway. The flow over each segment is calculated using Equation 5-10 and the incremental flows for each segment are added to give the total flow across the roadway. Method 2, shown in Part B of Figure 5-7, involves selecting a single horizontal line to represent the average depth of the upstream pool. In this case, Equation 5-10 is applied once to give an estimate of the total flow.

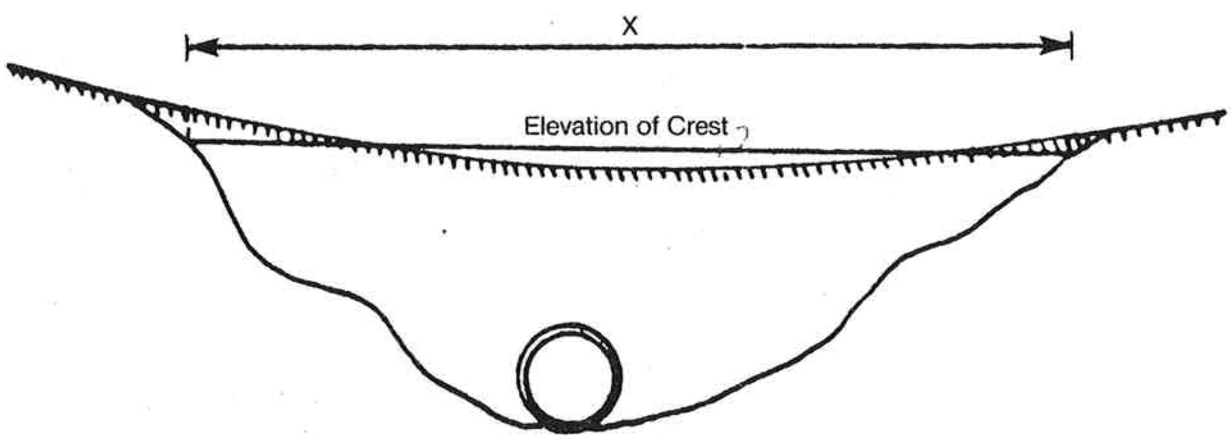
Since the total flow, Q, comprises both culvert flow and roadway overflow once the crest elevation is exceeded, a trial and error process is used to solve Equation 5-10. An approximate solution can be obtained by superimposing the culvert and roadway overflow performance curves.

## 5.5 OUTLET VELOCITY

Culvert outlet velocities, which are typically higher than natural stream velocities, may make channel stabilization or




**A. Method 1 - Subdivision into Segments**




**B. Method 2 - Use of a Single Segment**

**FIGURE 5-7**
Method for Setting Sag Vertical Curve Length and Crest Elevation

energy dissipation necessary. Because the type of flow affects the depth of flow at the outlet, different calculations are required for inlet and outlet control conditions.

In inlet control, backwater calculations may be necessary to determine the outlet velocity. As illustrated in Figure 5-8, these calculations begin at the culvert entrance and proceed downstream to the exit. The flow velocity is obtained from the flow and the cross-sectional area at the exit (use Equation 5-2).

An approximation may be used to avoid backwater calculations in determining the outlet velocity for culverts operating in inlet control. The water surface profile converges toward normal depth as calculations proceed down the culvert barrel. Therefore, if the culvert is of adequate length, normal depth will exist at the culvert outlet. Even in short culverts, normal depth can be assumed and used to define the area of flow at the outlet and obtain the outlet velocity (see Figure 5-8). The velocity calculated in this manner may be slightly higher than the actual velocity at the outlet. Normal depth can be estimated using Manning's Equation as presented in Chapter 4.

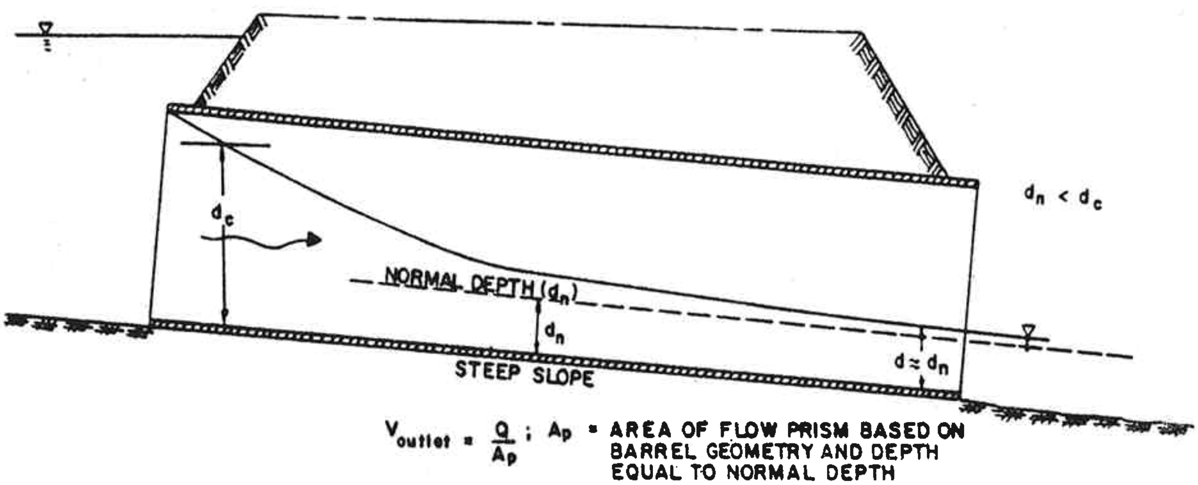
In outlet control, the depth of flow for computing the velocity is critical depth, $d_c$, tailwater depth, TW, or the height of the culvert, D, as defined in Figure 5-9. Critical depth is used when the tailwater is less than critical depth; the tailwater depth is used when the tailwater is greater than critical depth, but below the top of the barrel. The total barrel area is used when the tailwater exceeds the top of the barrel.

## 5.6 IMPROVED INLETS

In conditions of inlet control, techniques available to balance the inlet capacity with the outlet or barrel capacity include beveled inlet edges, side tapering the inlet, and slope tapering the inlet.

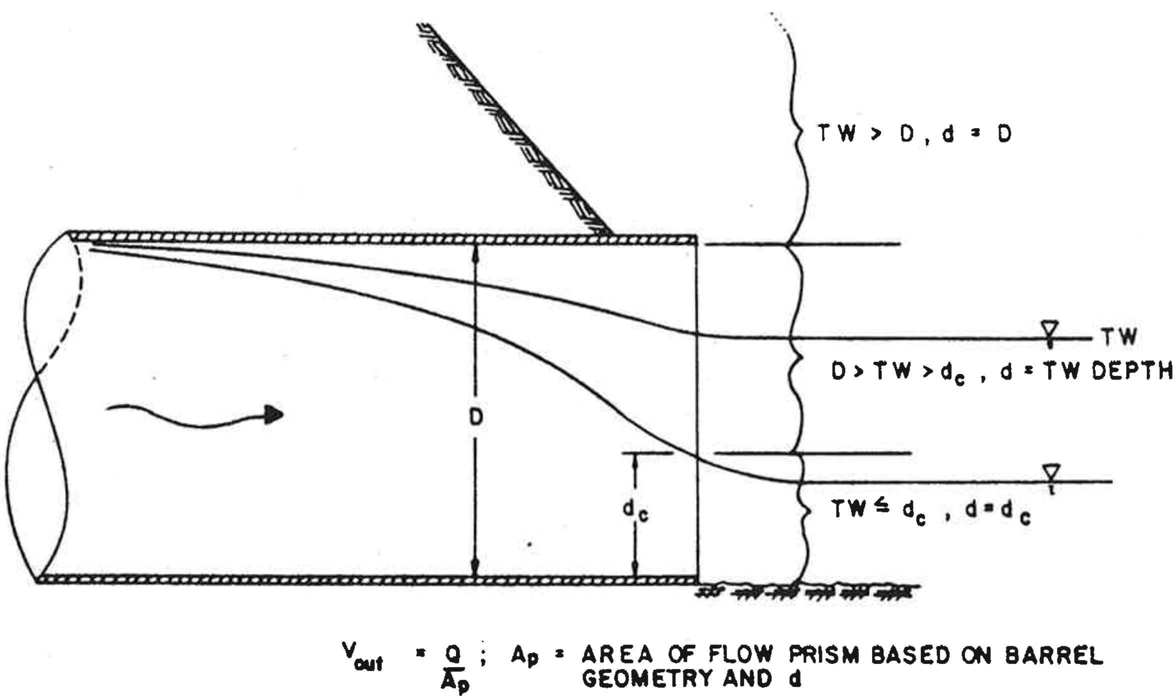
### 5.6.1 BEVELS

A bevel is similar to a chamfer, except that a chamfer is smaller and is generally used to prevent damage to sharp




*Reference: USDOT, FHWA, HDS-5 (1985).*

**FIGURE 5-8**
Inlet Control Outlet Velocity Calculations




*Reference: USDOT, FHWA, HDS-5 (1985).*

**FIGURE 5-9**
Outlet Control Outlet Velocity Calculations

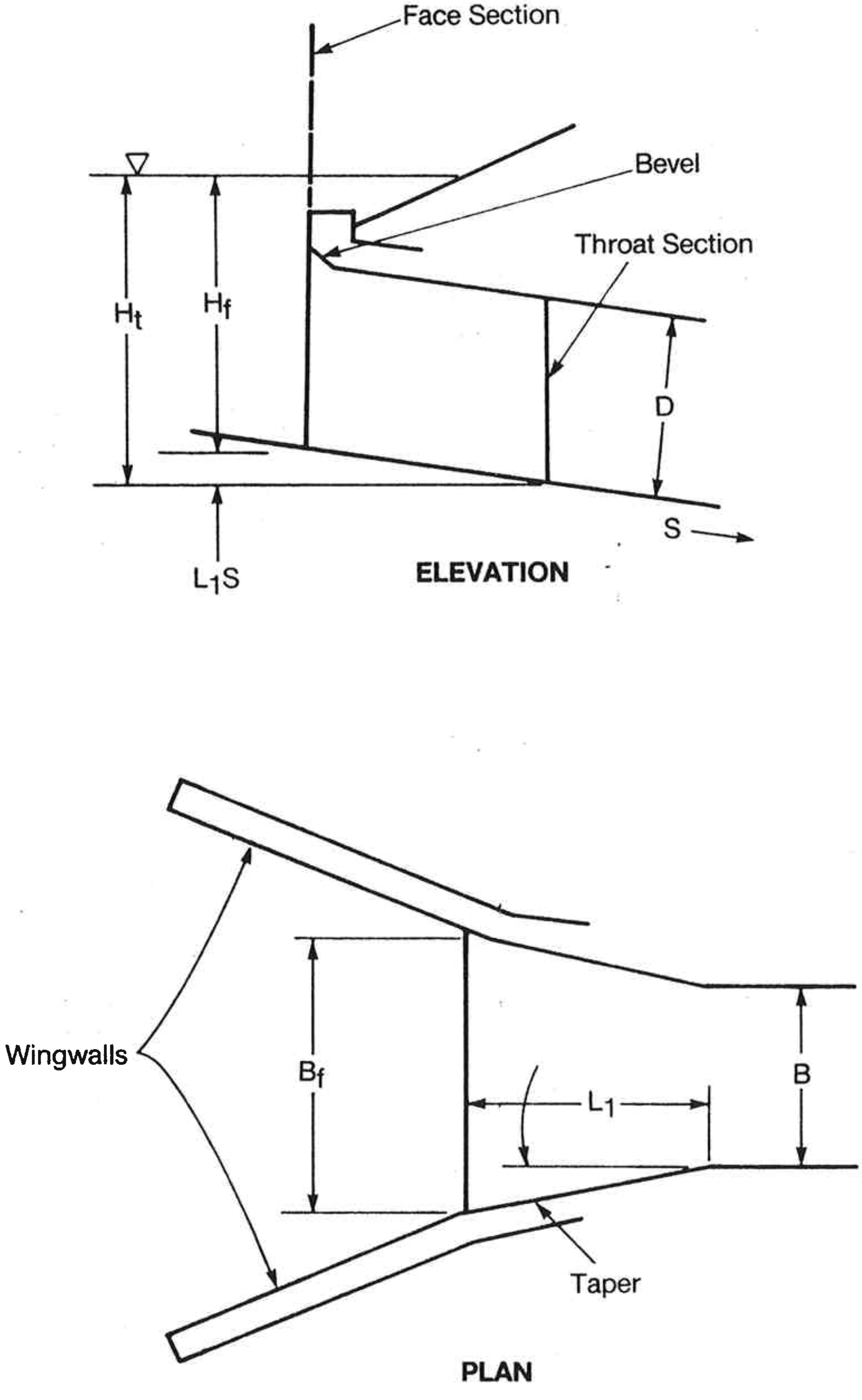
concrete edges during construction. The entrance loss coefficient, $k_e$, can be reduced from 0.7 for a square edge to 0.2 for beveled edges. Figures 5-10 and 5-11 illustrate bevels used in conjunction with other inlet improvements. It should be noted that the socket end of concrete pipe is comparable to a bevel in reducing the entrance loss coefficient and thus increases inlet capacity.

### 5.6.2 SIDE-TAPERED INLETS

Side-tapered inlets provide an enlarged culvert entrance with a transition to the original barrel dimensions. The inlet face has the same height as the barrel, and its top and bottom are extensions of the top and bottom of the barrel. The intersection of the sidewall tapers and barrel is defined as the throat section. Based on test results reported by the USDOT, FHWA, in HEC-13 (1972), the side-taper geometry shown in Figure 5-10 is recommended. The two possible control sections identified in Figure 5-10 are the face and the throat. Use of a side-tapered improvement is maximized by designing it so that the capacity is controlled by the throat.

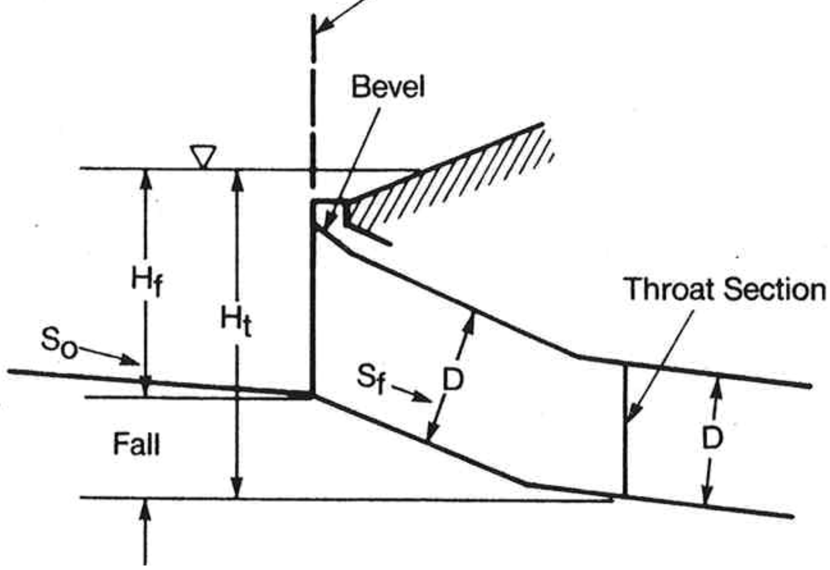
### 5.6.3 SLOPE-TAPERED INLETS

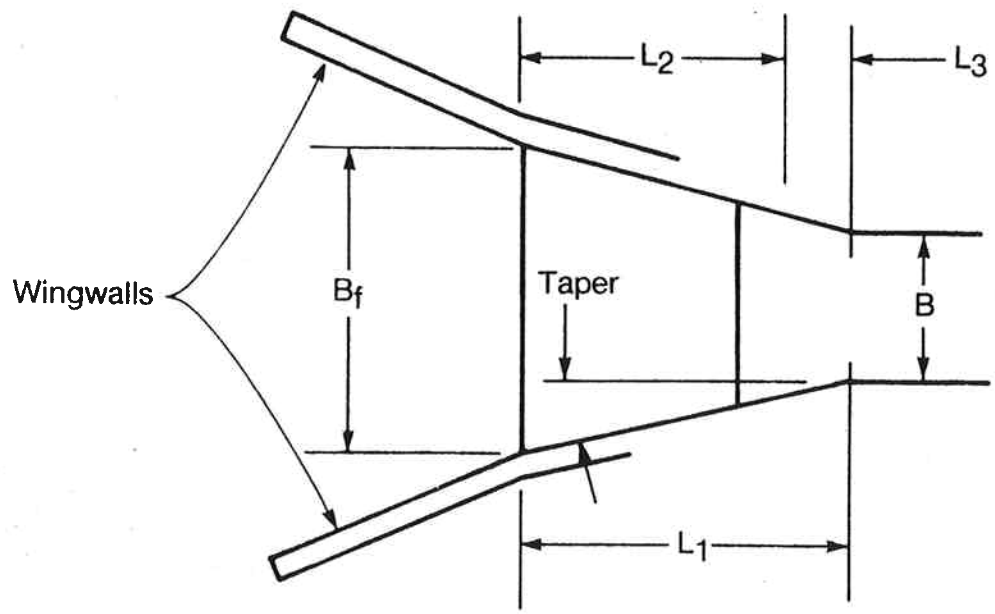
Slope-tapered inlets provide a steeper slope at the entrance than occurs throughout the remaining length of a culvert. The steeper slope increases the head on the throat section, making additional fall available. Depending on available fall, inlet capacities can be increased 100 percent or more above a conventional culvert with square edges. Based on test results reported by the USDOT, FHWA, in HEC-13 (1972), the slope-taper geometry shown in Figure 5-11 is recommended. As with side-tapered improvements, the slope-taper should be designed so that the throat section controls capacity.




*Reference: USDOT, FHWA, HEC-13 (1972).*

**FIGURE 5-10**
Typical Side-Tapered Inlet Detail




*Reference: USDOT, FHWA, HEC-13 (1972).*

**FIGURE 5-11**
Typical Slope-Tapered Inlet Detail

NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

# CHAPTER 6
## Storm Sewer Hydraulics

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# Chapter 6
# STORM SEWER HYDRAULICS

## SYNOPSIS

The sizing of a closed storm sewer system usually requires two major computations. The first is primarily a hydrologic problem and provides a determination of the peak runoff flow rate arriving at particular design points in the system. The second involves sizing the storm sewers for conveyance of these peak runoff rates and is primarily a hydraulic problem. This chapter presents a brief discussion of the hydrologic and hydraulic fundamentals associated with storm sewer systems. References include the American Society of Civil Engineers (1969), the American Concrete Pipe Association (1978, 1980), the American Iron and Steel Institute (1980), Yen et al. (1974), and Colyer and Pethick (1976).

## 6.1 HYDROLOGY

Hydrologic determination of a peak runoff rate for sizing a storm sewer system can be made using many of the procedures developed for estimating either peak runoff rates or runoff hydrographs, as presented in Chapter 2. In general, storm sewer systems are sized to carry stormwater intercepted by appropriate inlet facilities. However, if the intercepted runoff is transported through an extensive pipe network, channel storage occurring within the storm sewers can modify the peak runoff rate as it travels along the system. The evaluation of this peak flow modification can be accomplished using hydrologic channel routing procedures.

### 6.1.1 SUMMATION OF FLOWS METHOD

The hydrologic computations for this method of storm sewer design are performed exactly as its title suggests. The total peak runoff rate is estimated by adding the runoff intercepted by each inlet upstream of the design point. This sum is then used to size the conduit. Results of this method are more conservative than those of the other methods discussed in this section. This method is not acceptable for use in Nashville and Davidson County.

A situation where peak runoff from numerous inlets arrives at the design point at different points in time cannot be accounted for by the Summation of Flows Method. In addition, the method does not consider that the peak runoff rate for each inlet can be reduced by the influence of channel storage in the upstream portion of the storm sewer system. The Summation of Flows Method is not considered good engineering practice for large systems where the potential for reducing peak runoff by channel storage exists and where time of travel characteristics of the system warrant detailed study. Since attenuation is ignored, the method can over-estimate the peak runoff arriving at the lower portions of a complex storm sewer system.

### 6.1.2 RATIONAL METHOD

A peak runoff rate can be calculated for each design point in a storm sewer system by using the Rational Method. A different time of concentration, which should increase as the design proceeds in the downstream direction, is required for each design point. Thus, for large systems, the design rainfall intensity is decreased and the design flow rate will be lower than that obtained through use of the Summation of Flows Method.

Application of the Rational Method has a logical basis, and it indirectly accounts for situations in which peak runoff arrives at individual inlets at different points in time. However, this approach does not explicitly account for channel storage that may be important in the upstream portions of a storm sewer system.

There may be locations in a storm sewer system where only a portion of the area draining to that location will cause the peak discharge. This can happen for systems where a small part of the total area has a disproportionately long travel time. Thus, another larger portion of the contributing area with a shorter time of concentration (and, therefore, a higher average rainfall intensity) can cause a higher peak discharge at a given location.

### 6.1.3 INLET HYDROGRAPH METHOD

The Inlet Hydrograph Method uses the inlet and storm sewer travel times to adjust the intercepted peak runoff rate for each storm sewer segment. The composite peak runoff rate at the design point is then obtained by summing the ordinates of triangular hydrographs for each inlet. This is generally accomplished graphically by plotting triangular hydrographs on the same scale and using dividers to sum hydrograph ordinates. The procedure is detailed in Volume 2, based on the presention in Jens and McPherson (1964) and Kaltenbach (1963).

## 6.2 HYDRAULICS

The hydraulic evaluation of a storm sewer system should provide a balanced system in which all segments will be used to their full capacity consistent with the flood protection criteria for the project site. The hydraulic computations are based on the appropriate peak runoff rates, developed with the hydrologic procedures discussed above.

Two types of flow can occur in closed conduits. Gravity flow occurs when a free water surface is exposed to the atmosphere as a boundary (see Figure 6-1). When the conduit is flowing full, the pipe is considered to be flowing under pressure (see Figure 6-2).

### 6.2.1 ENERGY LOSSES

The two basic types of energy losses associated with flowing water in storm sewers are pipe friction losses, usually called major losses, and pipe form losses (minor losses). Pipe friction losses are generally evaluated by selecting an appropriate Manning's roughness coefficient, n, when pipe capacity calculations are performed. Pipe form losses require more complex calculations and depend on characteristics of the storm sewer configuration, such as changes in pipe size, branches, junctions, expansions, and contractions.

Pipe form losses are the result of fully developed turbulence, which can be evaluated using the general expression:

$$H_L = K\ (v^2/2g) \qquad (6\text{-}1)$$

where:

$H_L$ = Head loss, in feet

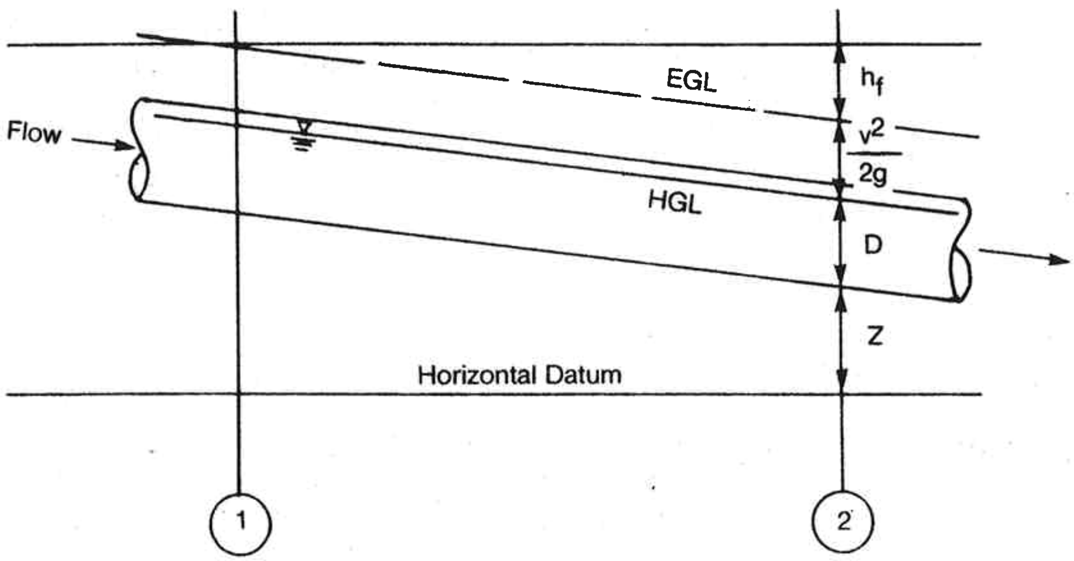


**FIGURE 6-1**
Open Channel Flow in a Closed Conduit

Z = Distance above horizontal datum
D = Depth of flow
P = Pressure head
$\frac{v^2}{2g}$ = Velocity head
$h_f$ = Friction loss between Section 1 and Section 2
EGL = Energy grade line
HGL = Hydraulic grade line

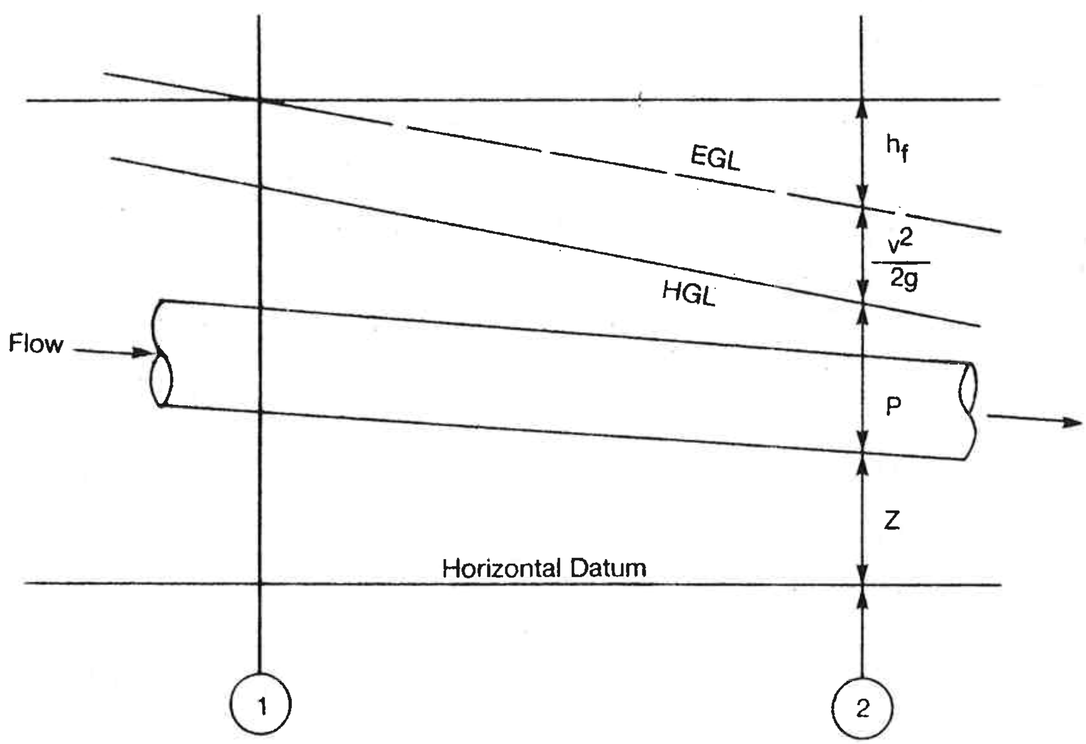


**FIGURE 6-2**
Pressure Flow in a Closed Conduit

K = Loss coefficient

v = Average velocity, in feet/second

g = Acceleration due to gravity, 32.2 feet/second$^2$

A few of the common types of pipe form losses encountered in storm sewer system design are defined below. Procedures for selecting the expansion loss coefficient for the various types of pipe form losses are presented in Volume 2. Standard references include publications by the University of Missouri (1958), the American Society of Civil Engineers (1969), and Marsalek (1985).

Expansion Losses. Expansion in a storm sewer conduit causes a shearing action between a high velocity jet from a smaller pipe and the boundary of a larger downstream pipe. As a result, much of the kinetic energy is dissipated by eddy currents and turbulence.

Contraction Losses. The reduction in cross-sectional area when flow passes from a large to a small pipe causes head losses, which are dependent on the amount of contraction and the type of entrance at the transition. Losses at contractions can be lowered by beveling the inlet edge at the contraction. This type of transition requires conversion of potential energy into kinetic energy to increase velocity.

Bend Losses. The bend loss coefficient used with Equation 6-1 has been found to be a function of the ratio of the radius of curvature of the bend to the width of the conduit; the deflection angle of the conduit; the geometry of the cross section of flow; and the Reynolds Number and relative roughness.

Junction and Manhole Losses. A junction occurs where one or more branch storm sewers enter a main storm sewer, usually at manholes. The hydraulic design of a junction is, in effect, the design of two or more transitions (e.g., expansions, contractions, and bends), one for each flow path. Allowances should be made for head loss caused by the impact at junctions.

### 6.2.2 GRAVITY FLOW

Under non-pressure or gravity flow conditions, the capacity of a closed conduit can be analyzed by applying Manning's Equation to evaluate frictional losses for uniform flow. As shown in Figure 6-1, the hydraulic grade line is the free water surface elevation and is parallel to the energy grade line under uniform flow conditions. For circular conduits flowing full, Manning's Equation can be expressed as:

$$v = \frac{0.592}{n} d_s^{2/3} S_o^{1/2} \tag{6-2}$$

or

$$Q = \frac{0.465}{n} d_s^{8/3} S_o^{1/2} \tag{6-3}$$

where:

$v$ = Full flow velocity, in feet/second

$Q$ = Design discharge, in cfs

$n$ = Manning's roughness coefficient

$d_s$ = Diameter of the circular conduit, in feet

$S_o$ = Pipe slope, in feet/foot

Non-circular and non-full flow conditions can be evaluated using the standard form of Manning's Equation discussed in Chapter 3.

Given the appropriate peak runoff rate for the design point in question, the conduit is sized to carry this peak rate as gravity flow, using Manning's Equation. For a condition in which pressure is allowed to develop in storm sewers with a design based on gravity flow conditions, the design capacity of the system will be greater than that determined using Equation 6-3, and can be evaluated as discussed below.

### 6.2.3 PRESSURE FLOW

If the hydraulic grade line, as illustrated in Figure 6-2, can be increased above the crown of the pipe, pressure flow occurs. Theoretically, pressure flow can be evaluated using appropriate forms of the energy and continuity equations. In practice, pressure flow can be evaluated using the nomographs developed for culvert flow that are discussed in Chapter 5 of this volume and presented in Volume 2. Pressure flow calculations for complex or critical systems should be performed using computer programs.

NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

# CHAPTER 7
## Bridge Hydraulics

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# Chapter 7
# BRIDGE HYDRAULICS

## SYNOPSIS

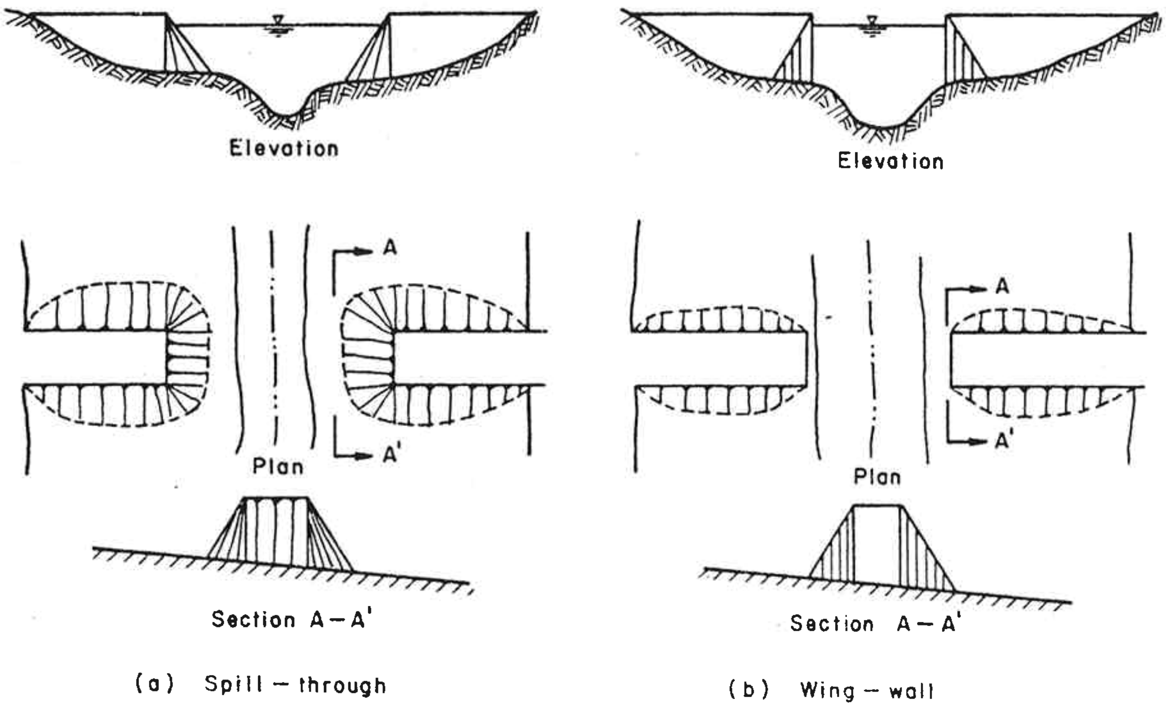
Bridge placement that constricts the normal passage of floodwaters can cause excessive backwater, which may result in flooding of upstream property, overtopping of roadways, excessive scour, or even the loss of a bridge. Bridge hydraulic computations are used to establish an appropriate hydraulic opening (length, vertical clearance, and location) within the acceptable limits of increased backwater.

Results of laboratory studies using physical models at Colorado State University (1957) and the Iowa Highway Research Board (1956 and 1958) have been combined with field data collected by the USGS to develop the bridge hydraulics procedures discussed in this chapter. Key references are HDS-1 (USDOT, FHWA, 1978) and WSPRO (HY-7) (USDOT, FHWA, 1986). Information on scour and degradation is summarized from the Training and Design Manual entitled Highways in the River Environment, Hydraulic and Environmental Design Considerations (USDOT, FHWA, 1975).

## 7.1 BRIDGE TYPES

River and stream highway crossings can sometimes be accomplished without encroachment into the flood plain by locating the bridge and its approaches far above and beyond a possible flood stage. However, since the cost of spanning an entire flood plain is usually prohibitive, some degree of encroachment is commonly imposed. An encroachment can occur when earth fill embankments are placed over the flood plain or into the channel itself, or when piers and abutments are placed in the main channel of a river.

A bridge is the most common type of facility used to minimize flood plain encroachment. The geometric properties of the bridge, illustrated in Figure 7-1, are commonly used, depending on the conditions at the site. The approach embankments may be skewed or normal (perpendicular) to the direction of flow. One approach may be longer than the other, producing an eccentric crossing. Abutments used for




*Reference: USDOT, FHWA, Training and Design Manual (1975).*

**FIGURE 7-1**
Geometric Properties of Bridge Crossings

the overbank-flow case may be set back from the low-flow channel banks to provide room to pass the floodflow; or the abutments may extend up to the banks or even protrude over the banks, constricting the base flow channel. Piers, dual bridges for multi-lane freeways, channel bed conditions, and spur dikes are additional factors to consider when selecting an appropriate type of bridge crossing.

## 7.2 RIVER DYNAMICS

In general, a river is dynamic, continually changing its position and shape as a consequence of hydraulic forces acting on its bed and banks. Changes may be slow or rapid, natural or artificially induced. When a project modifies a river channel locally, it frequently changes channel characteristics both up and down the river. The response of a river to man-induced changes often occurs in spite of attempts to keep the anticipated response under control.

An important component of river hydraulics at bridge crossings is the distribution and direction of flow velocities. The maximum longitudinal velocity is usually approximately 25 to 50 percent greater than the average velocity for the cross section. The instantaneous turbulent velocity in rivers can exceed the average velocity by as much as 70 percent or more.

When river flow encounters the change in direction caused by a bend, super-elevation of the channel bottom produces a transverse, or elliptical, velocity distribution component. Transverse velocities are caused by an imbalance of radial pressures on the flow as it travels around the bend. The magnitude of the transverse velocity component depends on the radius of curvature of the river and on the proximity of the flow to the banks. Immediately next to the banks, lateral velocity cannot occur if the river is narrow and deep.

## 7.3 FUNDAMENTALS

### 7.3.1 BACKWATER

It is seldom economically feasible or necessary to span the entire width of a flood plain. Where conditions permit, approach embankments can be extended onto the flood plain to reduce costs, recognizing that in so doing, the embankments will constrict the flow of the stream during flood stages. Normally, this is an acceptable practice provided water surface profile and scour conditions are properly evaluated.

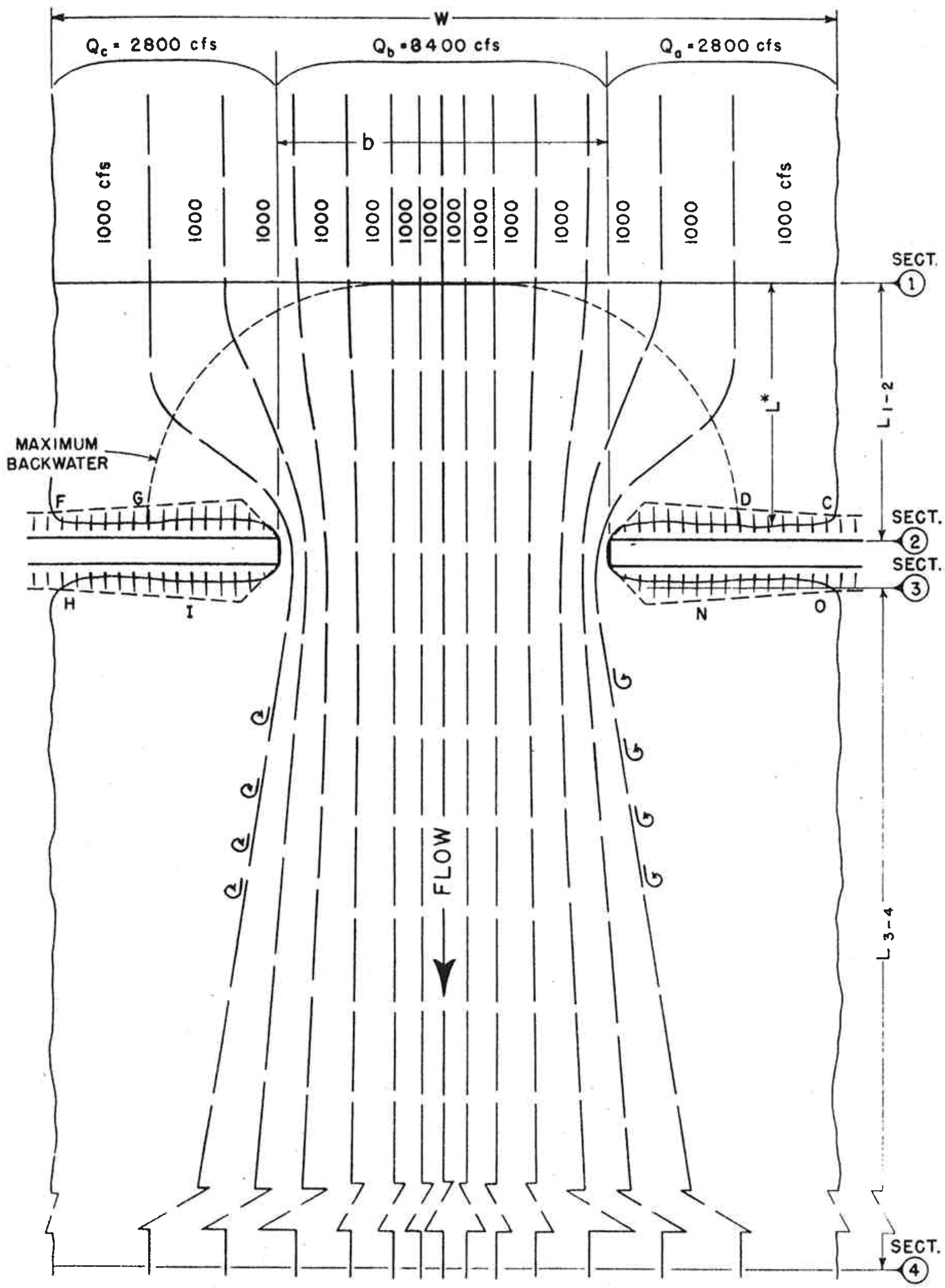
The manner in which flow is contracted in passing through a channel constriction is illustrated in Figure 7-2. For this example, the flow bounded by each adjacent pair of streamlines is the same (1,000 cfs). The channel constriction appears to produce practically no alteration in the shape of the streamlines near the center of the channel. A very marked change occurs near the abutments, however, since the momentum of the flow from both sides (or flood plains) must constrict the advancing central portion of the stream. Upon leaving the constriction, the flow gradually expands (5 to 6 degrees per side) until normal conditions are reestablished.

Constriction of the flow causes a loss of energy, with the greater portion occurring in the reexpansion downstream. This loss of energy is reflected in a rise in the water surface and in the energy line upstream from the bridge. A profile along the center of the stream illustrates this energy loss, as shown in Figures 7-3 for wingwall abutments (Part A) and 7-4 for spillthrough abutments (Part A).

The normal stage of the stream for a given discharge, before constricting the channel, is represented by the dashed line labeled normal water surface. (Water surface is abbreviated as "W.S." in the figures.) The nature of the water surface after constriction of the channel is represented by the solid line "actual water surface." Note that the water surface starts out above normal stage at Section 1, passes through normal stage close to Section 2, reaches minimum depth in the vicinity of Section 3, and then returns to normal stage a considerable distance downstream, at Section 4. Determination of the rise in water surface at Section 1 is denoted by the symbol $h_1$* and is referred to as the <u>bridge backwater</u>.

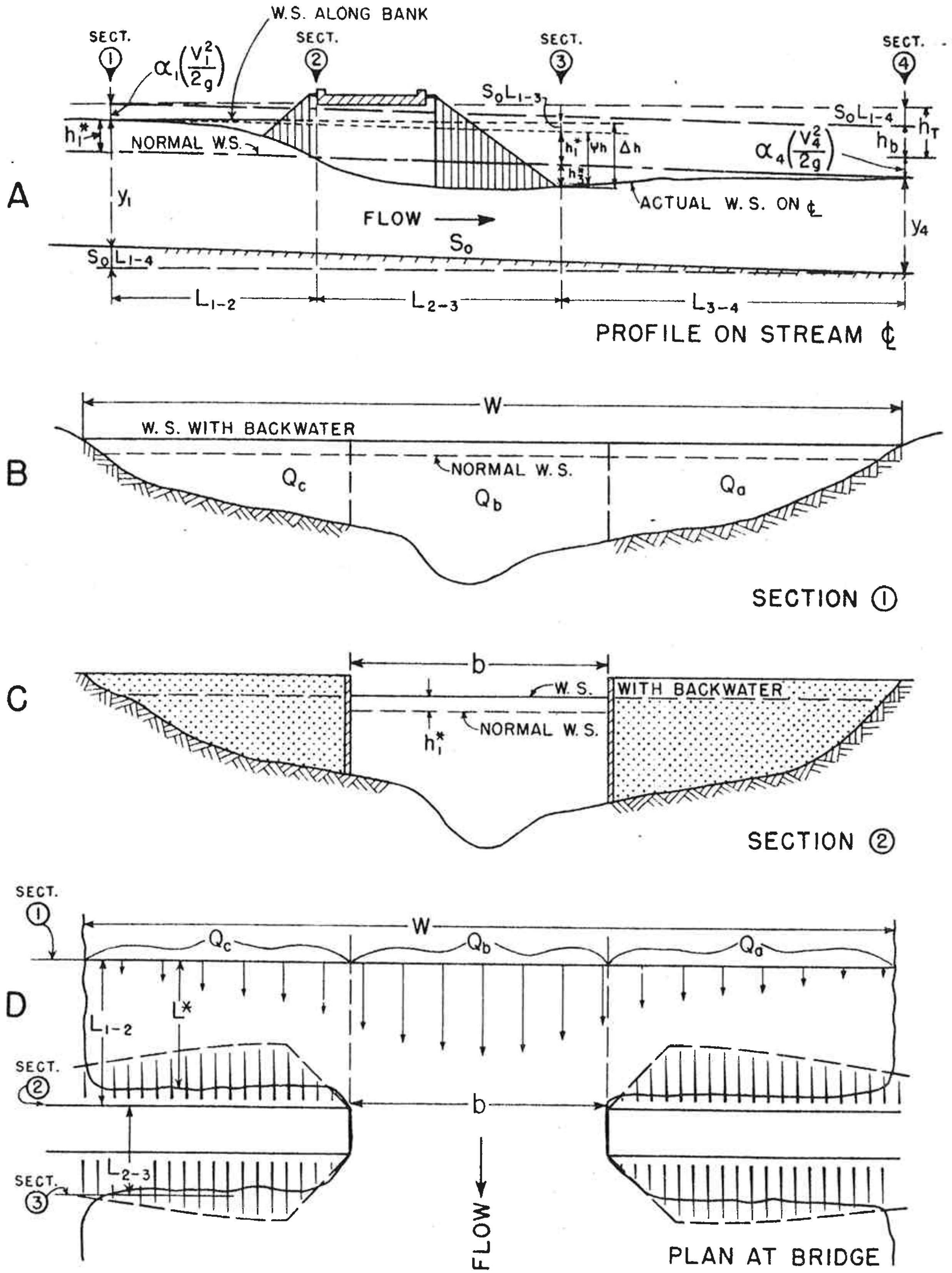
### 7.3.2 TYPES OF FLOW

The four types of flow encountered in bridge waterway design are labeled Types I, IIA, IIB, and III in Figure 7-5. The long dashed lines shown on each profile represent normal



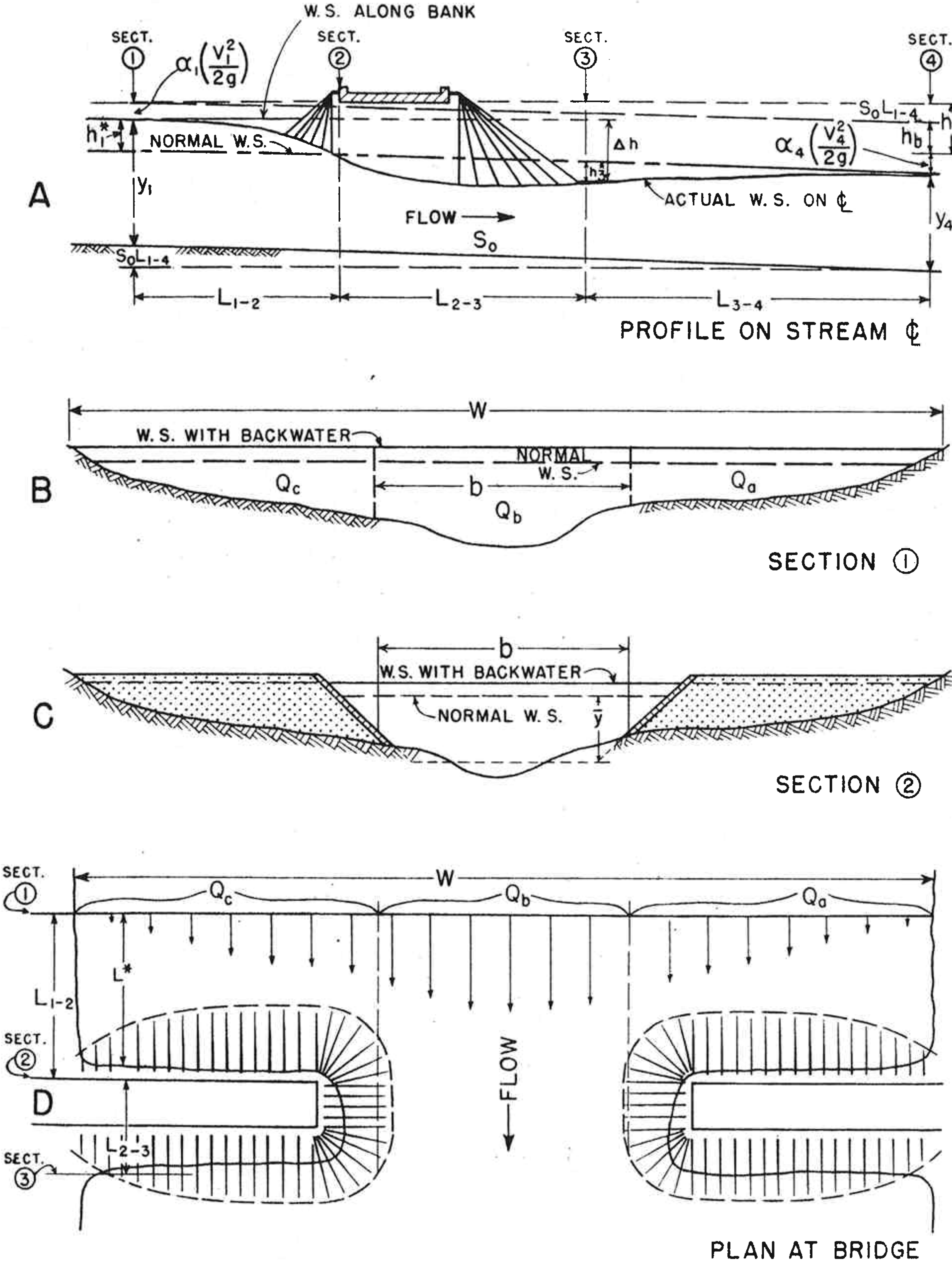

Reference: USDOT, FHWA, HDS-1 (1978).

**FIGURE 7-2**
Flow Lines for Typical Bridge Crossing



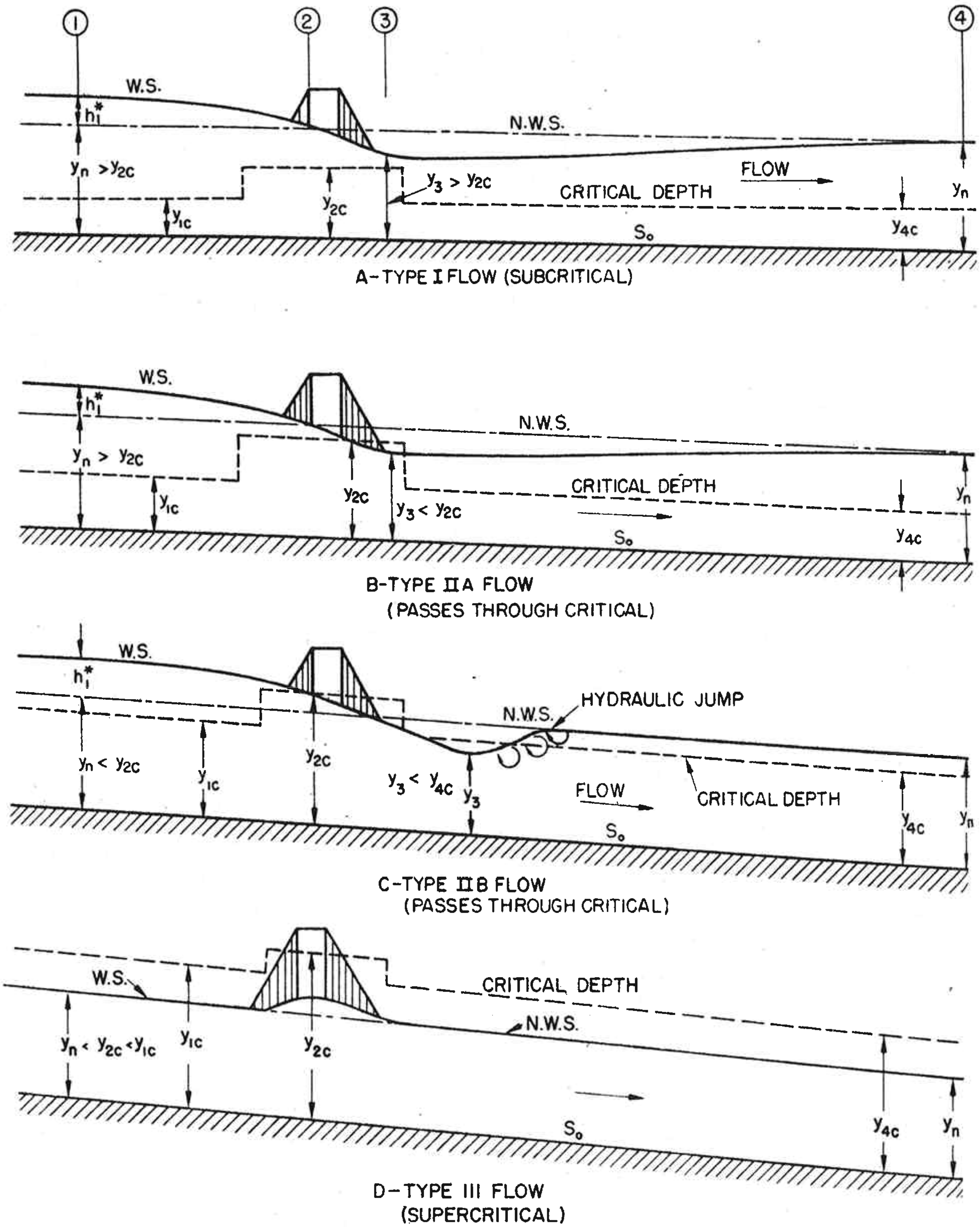

*Reference: USDOT, FHWA, HDS-1 (1978).*

**FIGURE 7-3**
Normal Crossings: Wingwall Abutments




*Reference: USDOT, FHWA, HDS-1 (1978).*

**FIGURE 7-4**
Normal Crossings: Spillthrough Abutments




*Reference: USDOT, FHWA, HDS-1 (1978).*

**FIGURE 7-5**
Types of Flow Encountered

water surface, or the stage the design flow would assume prior to placing a constriction in the channel. The solid lines represent the configuration of the water surface on the centerline of the channel after the bridge is in place. The short dashed lines represent critical depth, or critical stage, in the main channel, $y_{1c}$ and $y_{4c}$, and critical depth within the constriction, $y_{2c}$, for the design discharge. Since normal depth is shown as essentially the same in the four profiles, the discharge, boundary roughness, and slope of the channel must all increase in passing from Type I to Type IIA, Type IIB, and Type III flow.

## Type I Flow

In Figure 7-5, Part A, normal water surface is above critical depth at all times. This condition has been labeled Type I or subcritical flow, the type most often encountered in practice. The design information in HDS-1 (USDOT, FHWA, 1978) is limited primarily to Type I flow. The backwater expression for Type I flow is obtained by applying the conservation of energy principle between Sections 1 and 4.

## Type IIA Flow

There are at least two variations of Type II flow, described here as Types IIA and IIB. For Type IIA flow, shown in Figure 7-5, Part B, normal water surface again remains above critical depth in the unconstricted channel, but passes through critical depth in the constriction. Once critical depth is penetrated, the water surface upstream from the constriction, and thus the backwater, becomes independent of conditions downstream (even though the water surface returns to normal stage at Section 4). Thus, the backwater expression for Type I flow is not valid for Type IIA flow.

## Type IIB Flow

The water surface for Type IIB flow, shown in Figure 7-5, Part C, starts out above both normal water surface and critical depth upstream, passes through critical depth in the constriction, dips below critical depth for the channel downstream from the constriction, and then returns to normal. The return to normal depth can be abrupt, as seen in Figure 7-5, Part C, where it takes place in the form of a poor hydraulic jump, since normal water surface in the stream is above critical depth. A backwater expression applicable to both Types IIA and IIB flow has been developed by equating the total energy between Section 1 and the point at which the water surface passes through critical stage in the constriction.

Type III Flow

In Type III flow, shown in Figure 7-5, Part D, the normal water surface is always below critical depth and the flow throughout is supercritical. This condition, which requires a steep gradient, occurs mainly in mountainous regions. Theoretically, backwater should not occur for this type, since flow throughout is supercritical, but an undulation of the water surface in the vicinity of the constriction is quite possible, as indicated in Figure 7-5, Part D.

### 7.3.3 TERMINOLOGY

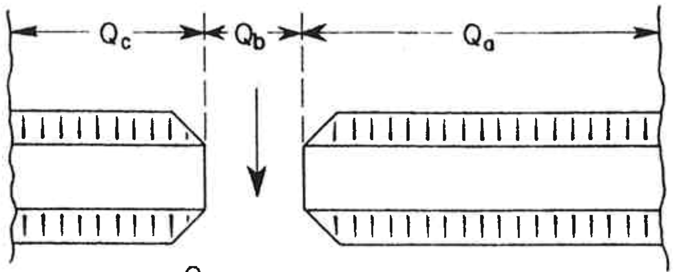
Terms and expressions frequently used in bridge hydraulics are briefly defined below.

Normal stage. Normal stage is the normal water surface elevation of a stream at a bridge site for a particular discharge, before stream constriction (see Figures 7-3, Part A, and 7-4, Part A). The profile of the water surface is essentially parallel to the bed of the stream.

Abnormal stage. When a bridge site is located upstream from, but relatively close to, the confluence of two streams, high water in one stream can produce a backwater effect extending for some distance up the other stream. This can cause the stage at a bridge site to be abnormal, i.e., higher than would exist for the tributary alone. An abnormal stage may also be caused by a dam, another bridge, or some other constriction downstream. The water surface with abnormal stage is not parallel to the bed.

Normal crossing. A crossing with alignment at approximately 90 degrees to the general direction of flow during high water (as shown in Figure 7-2).

Eccentric crossing. A crossing in which the main channel and the bridge are not in the middle of the flood plain (see Figure 7-6). Eccentricity is defined as one minus the ratio

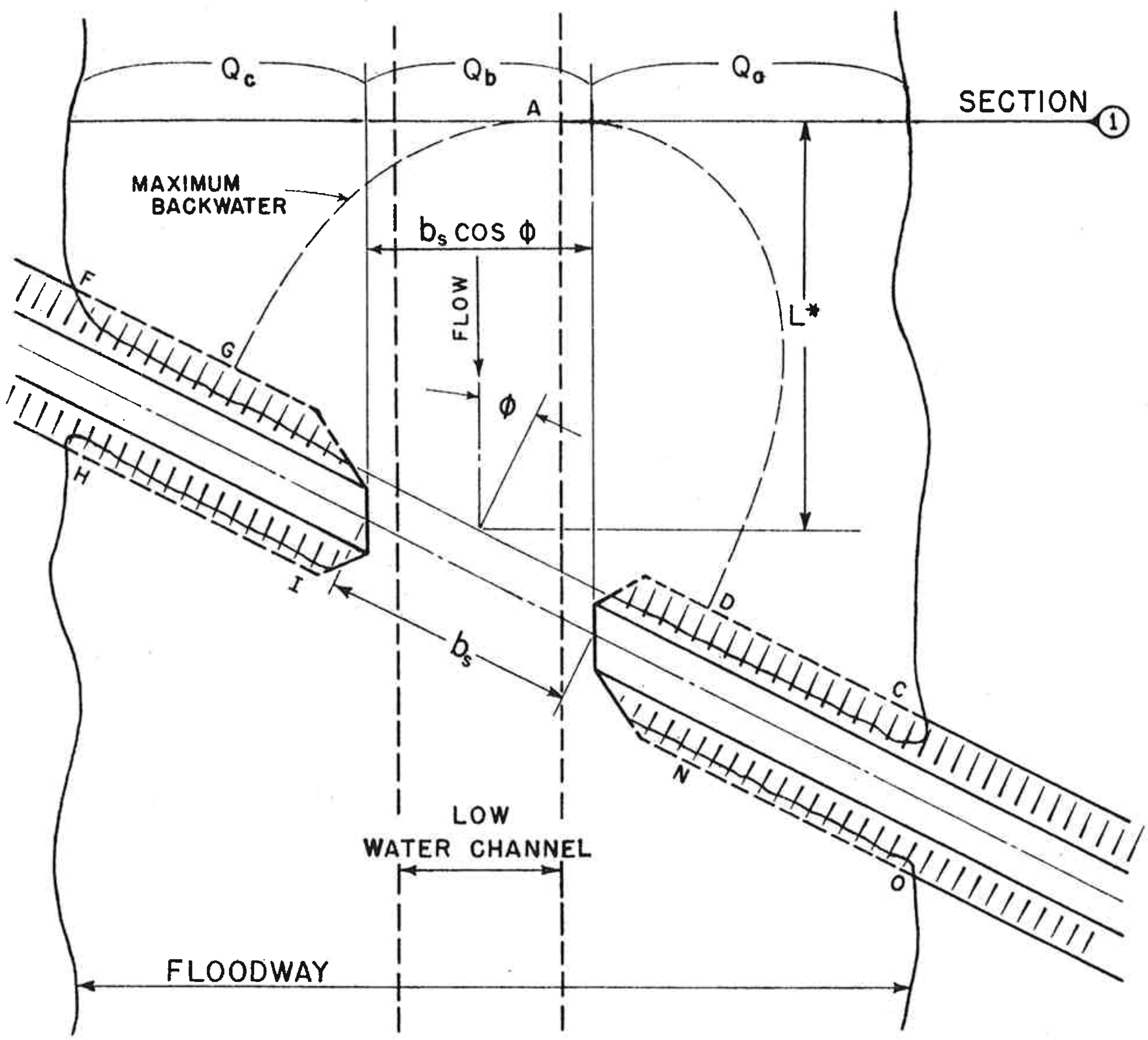



$$\text{eccentricity} = \left(1 - \frac{Q_c}{Q_a}\right) \quad \text{where} \quad Q_c < Q_a \quad \text{or}$$

$$\text{eccentricity} = \left(1 - \frac{Q_a}{Q_c}\right) \quad \text{where} \quad Q_a < Q_c$$

*Reference: USDOT, FHWA, HDS-1 (1978).*

**FIGURE 7-6**
Definition Sketch of Eccentricity at Bridge Crossings




*Reference: USDOT, FHWA, HDS-1 (1978).*

**FIGURE 7-7**
Skewed Bridge Crossings

of the lesser to the greater discharge outside the projected length of the bridge.

Skewed crossing. A crossing that is aligned other than 90 degrees to the general direction of flow during flood stage (see Figure 7-7).

Dual crossing. A pair of parallel bridges, such as for a divided highway.

Multiple bridges. Usually consisting of a main channel bridge and one or more relief bridges.

Width of constriction, b. For abutments with vertical faces, the construction width, b, is simply the horizontal distance between abutment faces. In the more usual case involving spillthrough abutments, where the cross section of the constriction is irregular, it is suggested that the irregular cross section be converted to a regular trapezoid of equivalent area, as shown in Figure 7-4, Part C. Then the length of bridge opening can be interpreted as:

$$b = \frac{A_{n2}}{\bar{y}} \tag{7-1}$$

where:

$b$ = Width of constriction, in feet

$A_{n2}$ = Gross area of constriction below normal water surface at Section 2, in square feet

$\bar{y}$ = Mean depth of flow, referenced to normal stage, in feet

Width to depth ratio. Defined as the ratio of flood plain width to mean constriction depth:

$$\frac{A_1}{\bar{y}_2} \quad \text{(for irregular cross sections)} \tag{7-2}$$

where:

$A_1$ = Area of flow, including backwater at Section 1, in square feet

$\bar{y}_2$ = Mean depth of flow at Section 2, in feet

7.3.4 CONVEYANCE

Conveyance is a measure of the ability of a channel to transport flow. In streams of irregular cross section, it is necessary to divide the water area into smaller but more or less regular subsections, assigning an appropriate roughness coefficient to each and computing the discharge for each subsection separately. By rearranging Manning's Equation as presented in Chapter 3, the following relationship is derived:

$$k = \frac{q}{\sqrt{S}} = \frac{1.49}{n} AR^{2/3} \tag{7-3}$$

where:

k = Channel subsection conveyance

q = Subsection discharge, in cfs

S = Channel bottom slope, in feet/foot

n = Manning's roughness coefficient

A = Subsection cross-sectional area, in square feet

R = Subsection hydraulic radius, in feet

Conveyance can, therefore, be expressed either in terms of flow factors or strictly geometric factors. In bridge waterway computations, conveyance is used as a means of approximating the distribution of flow in the natural river channel upstream from a bridge. Total conveyance, K, is the summation of the subsection conveyances constituting the particular section.

### 7.3.5 BRIDGE OPENING RATIO

The bridge opening ratio, M, defines the degree of stream constriction involved, expressed as the ratio of the flow that can pass unimpeded through the bridge constriction to the total flow of the river. Referring to Figure 7-2:

$$M = \frac{Q_b}{Q_a + Q_b + Q_c} = \frac{Q_b}{Q} \qquad (7\text{-}4)$$

where:

$M$ = Bridge opening ratio

$Q_b$ = Unimpeded flow, in cfs

$Q_a$ and $Q_c$ = Flow impeded by bridge obstruction, in cfs

$Q$ = Total flow, in cfs

As shown in Figure 7-2, if $Q_b$ = 8,400 cfs and Q = 14,000 cfs, M = 0.60.

The irregular cross section common in natural streams and the variation in boundary roughness within any cross section cause the velocity to vary across a river, as indicated by the stream tubes in Figure 7-2. The bridge opening ratio, M, is most easily explained in terms of discharges but is usually determined from conveyance relations. Since conveyance is proportional to discharge, assuming all subsections to have the same slope, M can also be expressed as:

$$M = \frac{K_b}{K_a + K_b + K_c} = \frac{K_b}{K} \qquad (7\text{-}5)$$

where:

$M$ = Bridge opening ratio

$K_b$ = Conveyance of unimpeded flow, in cfs

$K_a$ and $K_c$ = Conveyance of flow impeded by bridge obstruction, in cfs

K = Total conveyance of flow, in cfs

## 7.3.6 KINETIC ENERGY COEFFICIENT

As the velocity distribution in a river varies from a maximum at the deeper portion of the channel to essentially zero along the banks, the average velocity head, computed as $(Q/A_1)^2/2g$ for the stream at Section 1, does not give a true measure of the kinetic energy of the flow. A weighted average value of the kinetic energy is obtained by multiplying the average velocity head by a kinetic energy coefficient, $\alpha$, defined as:

$$\alpha = \frac{\Sigma(k^3/a^2)}{K^3/A^2} \tag{7-6}$$

where:

$\alpha$ = Kinetic energy coefficient for nonuniform velocity distribution

k = Conveyance of subsection (see Section 7.3.4)

a = Flow area of subsection, in square feet

K = Conveyance of total cross section (see Section 7.3.4)

A = Flow area of total cross section, in square feet

## 7.3.7 EFFECTIVE FLOW LENGTH

Because friction losses are directly proportional to flow length, it is important to obtain the best possible estimate of flow length. This is especially true for those cases where the friction loss is a significant component of the energy balance between two sections. The straight-line distance between sections is typically used to estimate the approach reach friction loss. For minor degrees of constriction, this is usually adequate. However, for significant constrictions, the straight-line distance is representative of only that portion of the flow that is generally in direct line with the opening. Flow farther from the opening must flow downstream and across the valley to reach the opening, thus traveling much farther than the straight-line distance. A technique has been developed to calculate an effective flow length that accounts for this increase. The optimum location of the approach section is estimated using the following equations:

$$L_{opt} = \left[\frac{b}{\pi(1 - m')}\right]\Phi \tag{7-8}$$

$$m' = 1 - \frac{b}{B} \tag{7-9}$$

$$\Phi = \frac{1}{2}\ln\left[\left(\sqrt{\frac{8}{\varepsilon^2} + 8} - \frac{3}{\varepsilon} - \varepsilon\right)\left(-\sqrt{8 + 8\varepsilon^2} - 3\varepsilon - \frac{1}{\varepsilon}\right)\right] - \ln\left(\varepsilon - \frac{1}{\varepsilon}\right) \tag{7-10}$$

$$\varepsilon = 1 + \delta + \sqrt{\delta^2 + 2\delta} \tag{7-11}$$

$$\delta = \frac{2}{\tan^2\left[1 - \left(\frac{b}{2B}\right)\pi\right]} \tag{7-12}$$

where:

$L_{opt}$ = Optimum distance between the approach section and the upstream face of the bridge opening, in feet

$b$ = Bridge opening length, in feet

$m'$ = Geometric contraction ratio

$B$ = Top width of the approach section flow area, in feet

$\Phi$, $\varepsilon$, and $\delta$ = Terms for computing the optimum location for the approach section

By definition, the optimum location for the approach section, $L_{opt}$, is in a zone of nearly one-dimensional flow, to satisfy the basic requirements of the one-dimensional energy equation. However, as long as the effective flow length is used in the computations, the approach section can be placed almost anywhere. As a convention, the approach section is generally placed one bridge-opening length upstream from the bridge opening.

The computational technique above varies depending on the relative magnitudes of $L_{opt}$ and b. For the ideal situation of a symmetric constriction with uniform, homogeneous conveyance, only one-half of the valley cross section is required. This one-half section is generally divided into ten equal-conveyance stream tubes between the edge of the water and the centerline at both the $L_{opt}$ location and the upstream face of the bridge. Equal-conveyance stream tubes are equivalent to equal-flow stream tubes for one-dimensional flow.

Figure 7-8 illustrates a case with a small geometric contraction ratio. $L_{opt}$ is less than b for lesser degrees of constriction. Since $L_{opt}$ is located in a zone of nearly one-dimensional flow, the streamlines are essentially parallel between the approach section and the $L_{opt}$ location. Between $L_{opt}$ and the bridge opening, the corresponding flow division points are connected with straight lines. In this case, the effective flow length is the average length of the ten equal-flow stream tubes, computed as:

$$L_{av} = 1/10 \left[ \sum_{i=2}^{10} s_i + (s_1 + s_{11})/2 \right] \qquad (7\text{-}13)$$

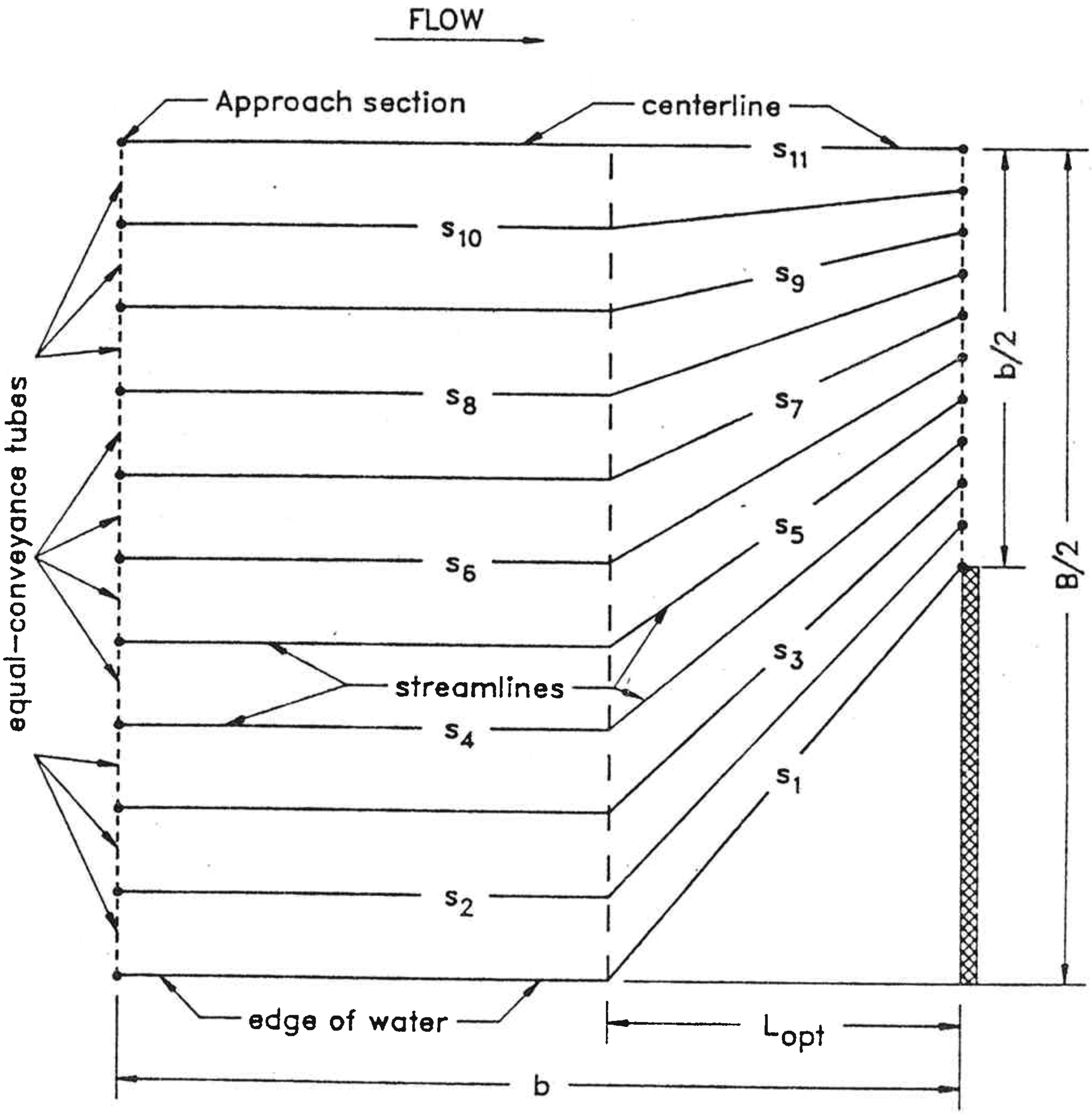
where:

$L_{av}$ = Effective flow length for a symmetric constriction with uniform, homogeneous conveyance, in feet

$s_i$ = Length of streamline, i, in feet

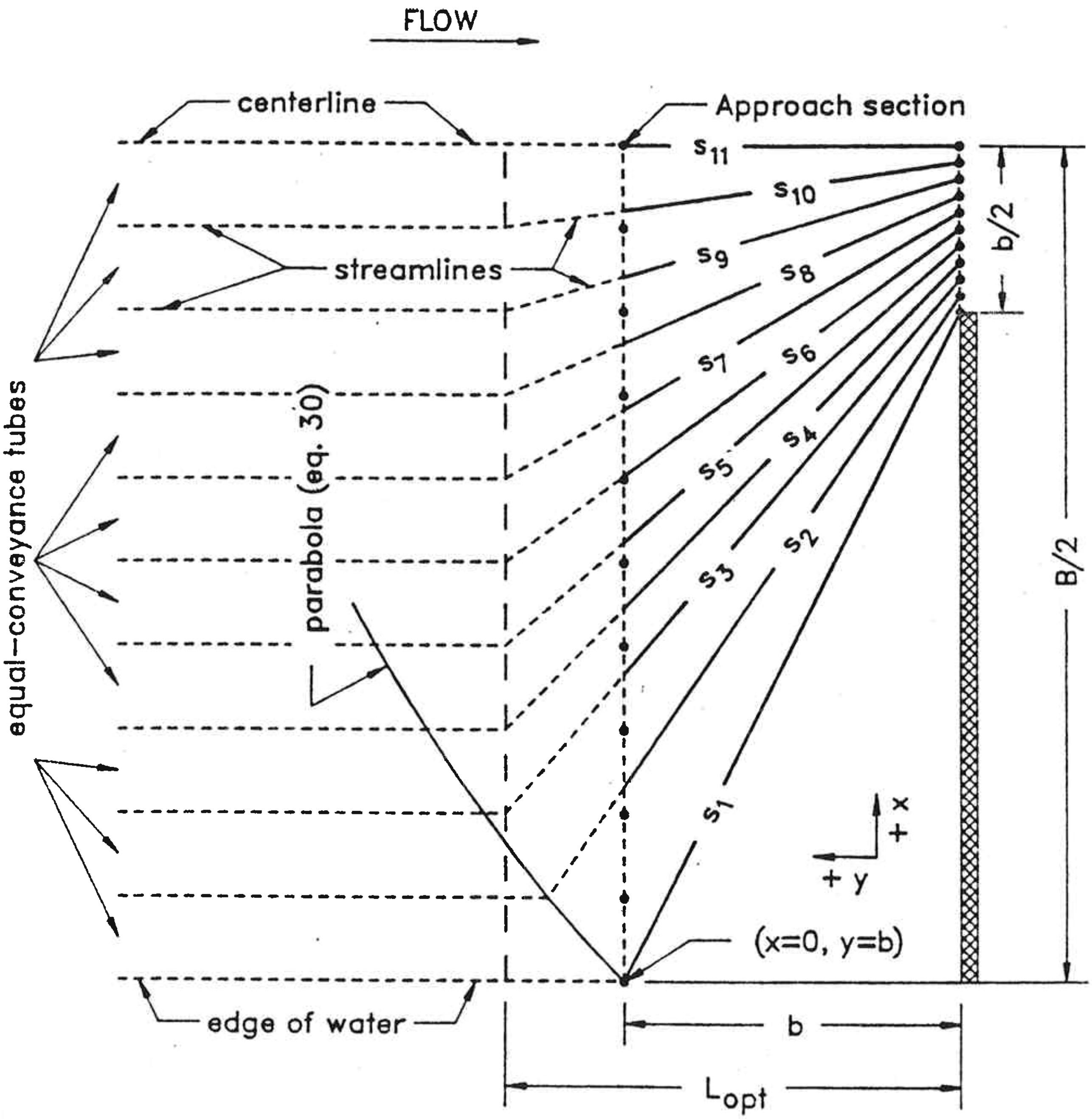
Although the straight-line pattern is a simplification of the actual curvilinear streamlines, the computed $L_{av}$ values are less than 2 percent greater than the exact solution for small geometric contraction ratios.

Figure 7-9 illustrates a relatively high degree of geometric contraction. Connecting the flow division points of the $L_{opt}$ and bridge sections does not result in representative




*Reference: USDOT, FHWA, HY-7 (1986).*

**FIGURE 7-8**
Effective Flow Length Configuration for Relatively Small Contractions




*Reference: USDOT, FHWA, HY-7 (1986).*

**FIGURE 7-9**

Effective Flow Length Configuration for a Relatively High Degree of Contraction lengths for those streamlines farthest away from the opening. Therefore, a parabola is computed as follows:

$$y^2 = 2b\left(x + \frac{b}{2}\right) \tag{7-14}$$

where:

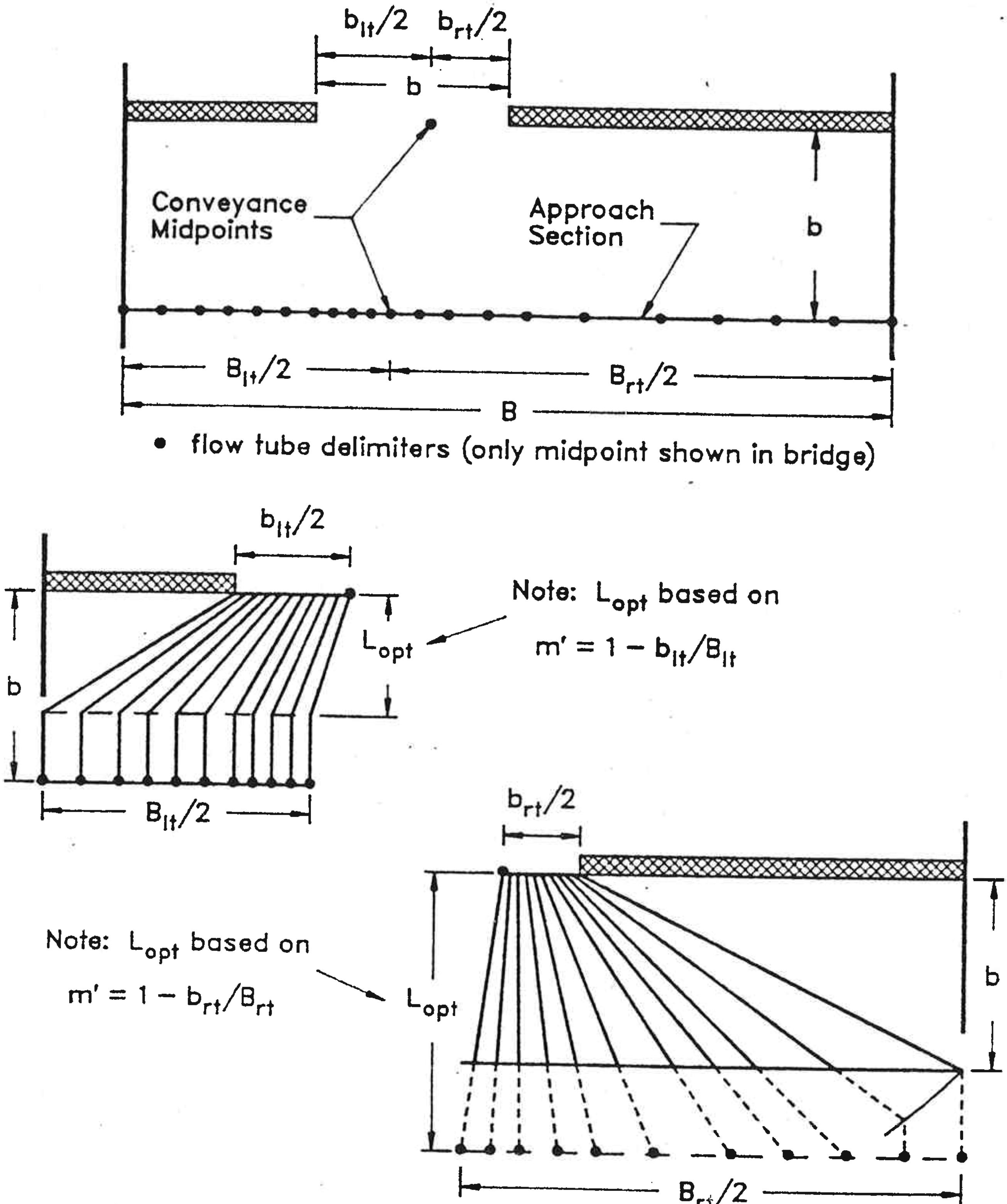
$y$ = Parabola y coordinate, in feet

$x$ = Parabola x coordinate, in feet

$b$ = Bridge opening length, in feet

This parabola has its focus at the edge of the water and its axis in the plane of the upstream face of the bridge. Positive x and y distances are measured from the edge of the water toward the stream centerline and upstream from the plane of the bridge, respectively. For portions of the section where $L_{opt}$ is upstream from this parabola, the parallel streamlines are projected to the parabola. A straight line then connects this projected point with the corresponding flow division point in the bridge opening. Flow division points of the $L_{opt}$ section at or downstream from the parabola are connected directly to their corresponding flow division point for the bridge opening. Only the distances between the approach and bridge-opening sections are used to compute the effective flow length, $L_{av}$, with Equation 7-13. This process generally produces results that are within 5 percent of the exact solution. For very severe constrictions (e.g., m' = 0.95), the differences are closer to 10 percent.

A nonuniform conveyance distribution in the approach reach is represented by defining the stream tubes on a conveyance basis. Asymmetric constrictions with nonuniform conveyances can be analyzed by treating each half of the reach on either side of the conveyance midpoints separately, then averaging the results. The effective flow length, $L_{av}$, for each side is the conveyance-weighted average streamline length. A typical asymmetric, nonuniform conveyance situation is illustrated in Figure 7-10.




Reference: USDOT, FHWA, HY-7 (1986).

**FIGURE 7-10**
Assumed Flow Pattern for a Nonsymmetric Constriction with a Nonhomogeneous Roughness Distribution

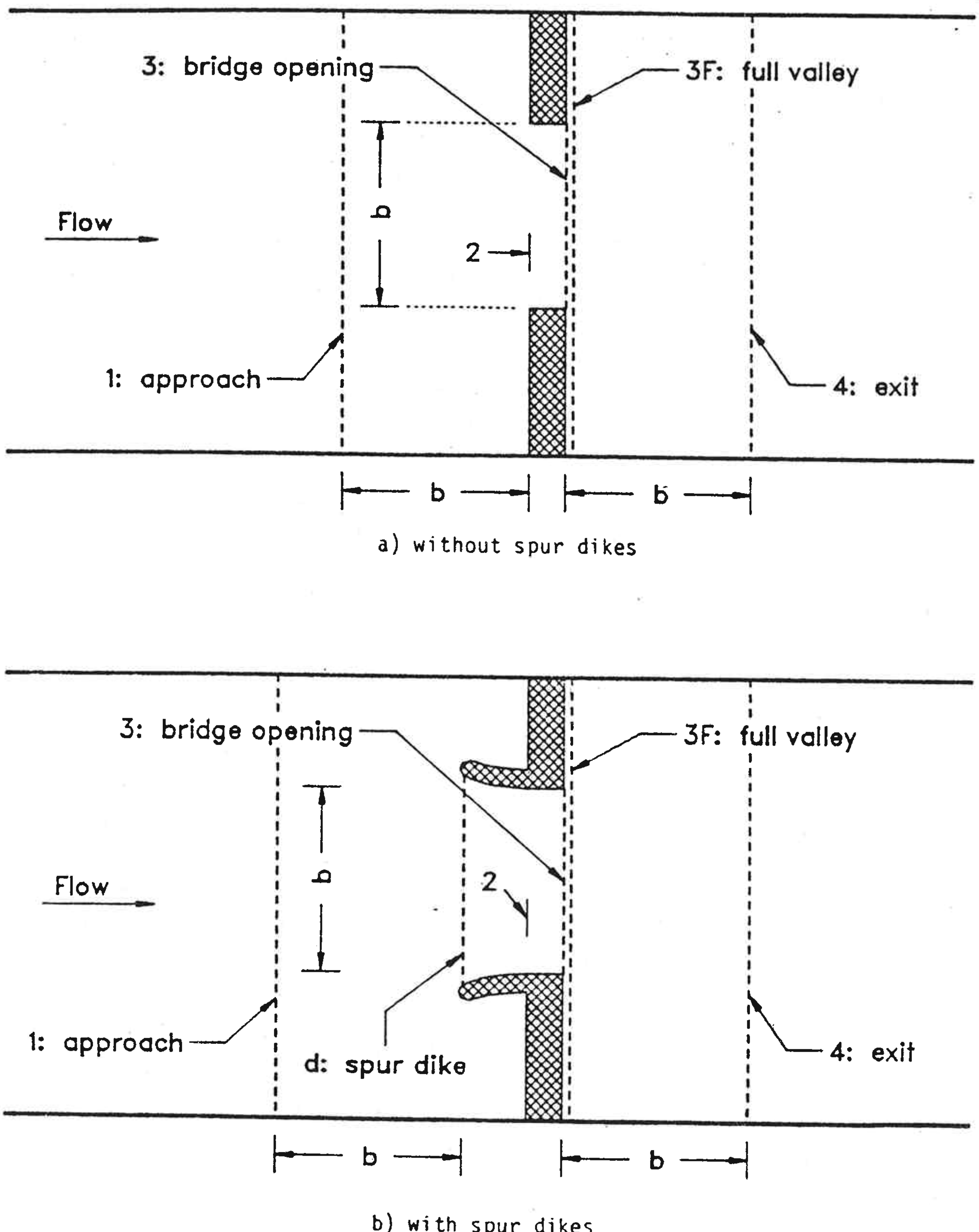
## 7.4 WATER SURFACE PROFILES

Computation of the water surface profile through a bridge having a single waterway opening requires definition of a minimum of four cross sections. The cross sections numbered 4, 3F, and 1 in Figure 7-11 are unconstricted valley sections and are referred to as exit, full-valley, and approach sections, respectively. These three sections, along with the bridge-opening section (Figure 7-11, Section 3), represent the minimum definition of a stream crossing.

The bridge-opening section is located at the downstream face of the bridge. Another location (Figure 7-11, Section 2) at the upstream face of the bridge is a control point in some cases, but requires no input data. If spur dikes are present, a dike section located at the toe of the dikes must be defined (Figure 7-11, Section d). Also, if flow over the embankment might occur, a road-grade section (not shown in Figure 7-11) is required to define the top of the embankment, which would serve as the crest of a weir.

The flow situation that exists at a single bridge opening depends on the relative elevations of the water surface both upstream and downstream of the bridge with respect to the elevations of the top of the bridge opening (referred to as low steel) and the top of the road grade. Free-surface flow exists when there is no contact or insignificant contact of water surface and low steel. Pressure flow through the bridge opening occurs as either (1) submerged orifice flow (when the water surface is in contact with low steel for the full flow length through the bridge) or (2) orifice flow (when only the upstream water surface is in contact with low steel). Any of these flows through the bridge opening can occur in conjunction with road overflow.

The type of flow depends on the elevation of the water surface relative to (1) the elevation of low steel in the bridge, which determines whether there will be free-surface or pressure flow through the opening, and (2) the minimum elevation along the top of the embankment, which determines whether there will be road overflow. Table 7-1 summarizes the flow classifications and the governing elevation relationships. The symbols used are defined as follows: $h_{ds}$ and $h_{us}$ are the water surface elevations immediately downstream and immediately upstream of the bridge,




Reference: USDOT, FHWA, HY-7 (1986).

**FIGURE 7-11**
Cross Section Locations for Water Surface Profile Computations at Single-Opening Bridges

Table 7-1
SUMMARY OF FLOW CLASSIFICATIONS FOR A SINGLE BRIDGE OPENING

a) Flow only through the bridge opening.

<table>
<tr><th>Class</th><th>Flow Class</th><th colspan="3">Relative Elevations</th></tr>
<tr><td>1</td><td>Free surface</td><td rowspan="2">$h_{ds} < Y_{ls}$</td><td>$h_{us} < Y_{ls}$</td><td rowspan="3">$h_{us} < Y_{min}$</td></tr>
<tr><td>2</td><td>Orifice</td><td rowspan="2">$h_{us} > Y_{ls}$</td></tr>
<tr><td>3</td><td>Submerged orifice</td><td>$h_{ds} > Y_{ls}$</td></tr>
</table>

b) Combination of flow through the bridge opening and weir flow over the road grade.

<table>
<tr><th>Class</th><th>Flow Class</th><th colspan="3">Relative Elevations</th></tr>
<tr><td>4</td><td>Free surface</td><td rowspan="2">$h_{ds} < Y_{ls}$</td><td>$h_{us} < Y_{ls}$</td><td rowspan="3">$h_{us} < Y_{min}$</td></tr>
<tr><td>5</td><td>Orifice</td><td rowspan="2">$h_{us} > Y_{ls}$</td></tr>
<tr><td>6</td><td>Submerged orifice</td><td>$h_{ds} > Y_{ls}$</td></tr>
</table>

---

$h_{ds}$ = Water surface elevation downstream of the bridge, in feet

$h_{us}$ = Water surface elevation upstream of the bridge, in feet

$Y_{ls}$ = Low steel elevation, in feet

$Y_{min}$ = Minimum embankment elevation, in feet respectively; $Y_{ls}$ is a low steel elevation; and $Y_{min}$ is the minimum embankment elevation.

The computation of natural profile elevations, subscripted as $h_{in}$ where i is the section number, should precede bridge hydraulic computations. Such data permit determination of the amount of backwater caused by the constriction and also provide an initial trial elevation in the iterative solution for the water surface profile through the bridge.

### 7.4.1 FREE-SURFACE FLOW

The total energy equation between the exit and approach sections (Sections 4 and 1), assuming natural profile elevation at Section 4, can be written as:

$$h_1 + h_{v1} = h_{4n} + h_{v4n} + \text{Losses}_{(1-4)} \qquad (7\text{-}15)$$

where:

$h_1$ = Water surface elevation at Section 1, in feet

$h_{v1}$ = Velocity head at Section 1, in feet

$h_{4n}$ = Natural profile elevation at Section 4, in feet

$h_{v4n}$ = Natural profile velocity head at Section 4, in feet

$\text{Losses}_{(1-4)}$ = Summation of friction losses in the subreaches between Sections 1 and 4 plus an expansion loss between Sections 3 and 4

Friction loss in a subreach is computed as:

$$h_f = L_{av} S_f \qquad (7\text{-}16)$$

$$S_f = \frac{\left[(Q_0 + Q_1)/2\right]^2}{K_0 K_1} \qquad (7\text{-}17)$$

where:

$h_f$ = Friction loss, in feet

$L_{av}$ = Effective flow length, in feet (see Section 7.3.7)

$S_f$ = Geometric mean friction slope, in feet/foot

$Q_0$ = Downstream discharge, in cfs

$Q_1$ = Upstream discharge, in cfs

$K_0$ = Downstream total cross-sectional conveyance (see Equation 7-3)

$K_1$ = Upstream total cross-sectional conveyance (see Equation 7-3)

The expansion loss in a subreach is computed as:

$$h_e = K_e \, (h_{v1} - h_{v0}) \tag{7-18}$$

where:

$h_e$ = Expansion loss, in feet

$K_e$ = Expansion coefficient (Range 0 to 1.0)

$h_{v1}$ = Velocity head at Section 1, in feet

$h_{v0}$ = Velocity head downstream, in feet

For contracting flow, the loss in a subreach is computed as:

$$h_c = K_c \, (h_{v0} - h_{v1}) \tag{7-19}$$

where:

$h_c$ = Contraction loss, in feet

$K_c$ = Contraction coefficient (Range 0 to 0.5)

$h_{v0}$ = Velocity head downstream, in feet

$h_{v1}$ = Velocity head at Section 1, in feet

Without spur dikes, the friction losses between the approach section and the upstream face of the bridge (Figure 7-11a, Sections 1 and 2) are expressed as:

$$h_{f(1-2)} = \frac{L_{av} Q^2}{K_1 K_c} \qquad (7\text{-}20)$$

where:

$h_{f(1-2)}$ = Friction losses between Sections 1 and 2, in feet

$L_{av}$ = Effective flow length, in feet (see Section 7.3.7)

$Q$ = Design discharge, in cfs

$K_1$ = Conveyance at Section 1 (see Equation 7-3)

$K_c$ = Control conveyance (see discussion below)

The value used for $K_c$ is the minimum of the following conveyances: $K_3$, $K_d$ (if dikes exist), and $K_q$. $K_q$ is the conveyance of the $K_q$ section, which is defined as that segment of the approach section that conveys discharge that can flow through the bridge opening without contraction. The horizontal limits of the $K_q$ section are determined by projecting the bridge opening to the approach section with the projection lines oriented parallel to the general direction of flow.

When spur dikes are present, friction losses upstream from the bridge are computed separately for the two subreaches (Figure 7-11b, Sections 1 to d and d to 2) upstream from the the bridge by the equations:

$$h_{f(1-d)} = \frac{L_{av} Q^2}{K_1 K_c} \qquad (7\text{-}21)$$

and

$$h_{f(d-2)} = \frac{L_{(d-2)} Q^2}{K_d K_c} \tag{7-22}$$

where:

$h_{f(1-d)}$ = Friction loss between Sections 1 and d, in feet

$L_{av}$ = Effective flow length, in feet (see Section 7.3.7)

$Q$ = Design discharge, in cfs

$K_1$ = Conveyance at Section 1 (see Equation 7-3)

$K_c$ = Control conveyance (see discussion above)

$h_{f(d-2)}$ = Friction loss between Sections d and 2, in feet

$K_d$ = Conveyance with spur dikes (see Equation 7-3)

$L_{(d-2)}$ = Straight-line flow distance from d to 2, in feet

The friction loss through the bridge is expressed as:

$$h_{f(2-3)} = L_{(2-3)}\left(\frac{Q}{K_3}\right)^2 \tag{7-23}$$

where:

$h_{f(2-3)}$ = Friction loss between Sections 2 and 3, in feet

$L_{(2-3)}$ = Straight-line flow distance from 2-3, in feet

$Q$ = Design discharge, in cfs

$K_3$ = Conveyance at Section 3 (see Equation 7-3)

The friction loss in the flow expansion reach is computed as:

$$h_{f(3-4)} = \frac{b\ Q^2}{K_c\ K_{4n}} \qquad (7\text{-}24)$$

where:

$h_{f(3-4)}$ = Friction loss between Sections 3 and 4, in feet

$b$ = Bridge-opening length, in feet

$Q$ = Design discharge, in cfs

$K_c$ = Control conveyance (see discussion above)

$K_{4n}$ = Natural profile conveyance at Section 4 (see Equation 7-3)

The expansion loss from Section 3 to 4 is computed by the equation:

$$h_e = \frac{Q^2}{2g\ A_4^2}\left[2\beta_4 - \alpha_4 - 2\beta_3\left(\frac{A_4}{A_3}\right) + \alpha_3\left(\frac{A_4}{A_3}\right)^2\right] \qquad (7\text{-}25)$$

where $\beta$ is a momentum correction factor for nonuniform flow distribution. $\alpha_4$ and $\beta_4$ are computed as:

$$\alpha_4 = \frac{\Sigma\ (K^3/a^2)}{K^3\ A^2} \qquad (7\text{-}26)$$

$$\beta_4 = \frac{\Sigma\ (k^2/a)}{K^2/A} \qquad (7\text{-}27)$$

where lower case and upper case indicate subsection and total section properties. $\alpha_3$ and $\beta_e$ are related to bridge geometry and are computed as:

$$\alpha_3 = \frac{1}{C^2} \qquad (7\text{-}28)$$

$$\beta_3 = \frac{1}{C} \qquad (7\text{-}29)$$

where:

$C$ = The coefficient of discharge for the bridge

### 7.4.2 ORIFICE FLOW

If the water surface is in contact only with the upstream girders, the water surface profile through the bridge can be determined with an orifice flow equation expressed as:

$$QBO = C_D\, A_{3net} \sqrt{2g\,(Y_u - Z/2 + h_{v1})} \qquad (7\text{-}30)$$

where:

$QBO$ = Orifice flow through the bridge opening, in cfs

$C_D$ = Discharge coefficient

$A_{3net}$ = Net area (total minus area of piers or piles) in the bridge opening for an elevation of $h_{us}$ (Equation 7-31), in square feet

$Y_u$ = Average upstream depth above the streambed, in feet

$Z$ = Height of the bridge opening above the streambed, in feet

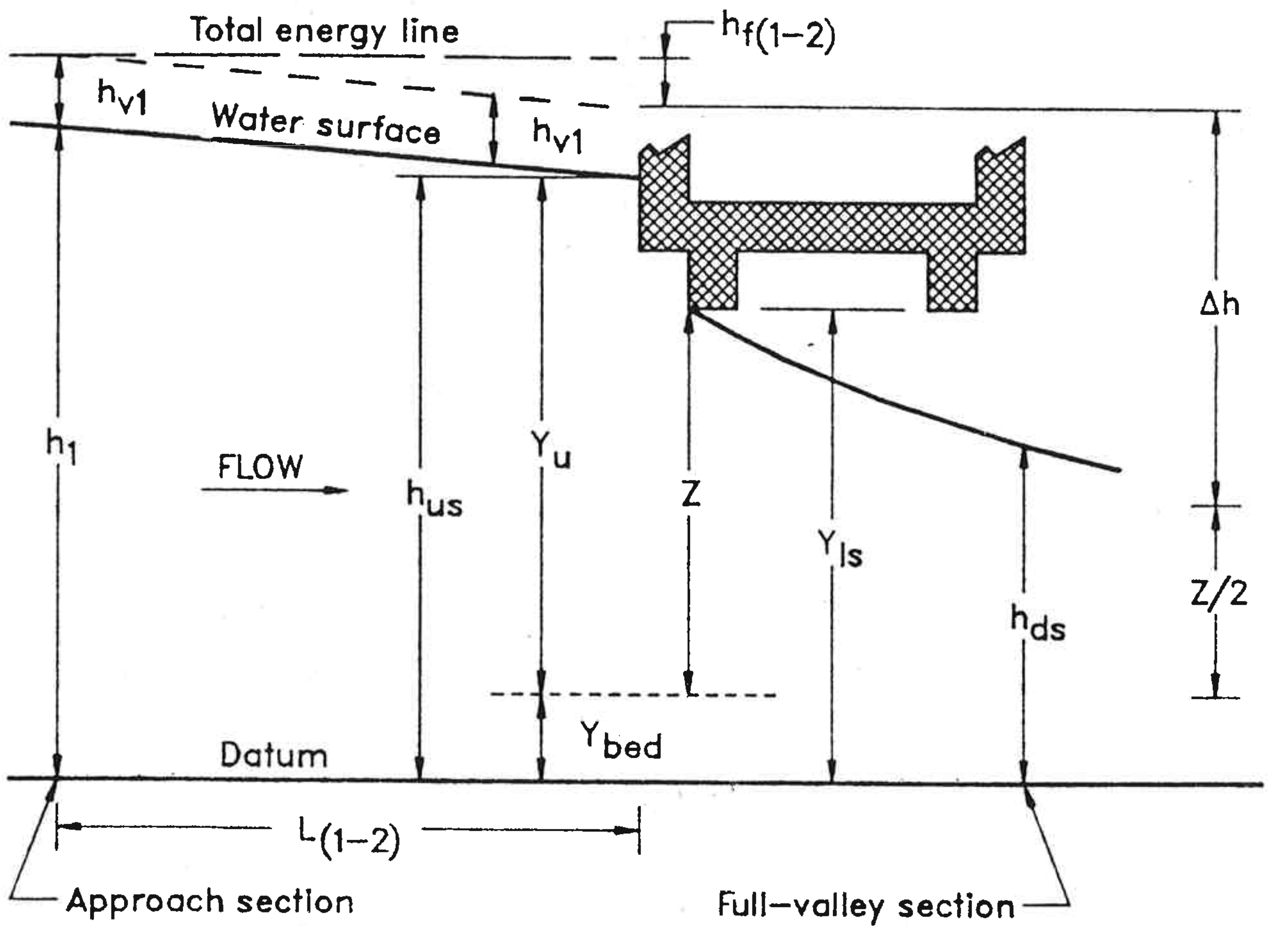
$h_{v1}$ = Velocity head at the approach Section 1, in feet

$g$ = Acceleration due to gravity, 32.2 feet/second$^2$

Figure 7-12 illustrates the definition of the variables involved in this computation. The water surface elevation immediately upstream from the bridge, $h_{us}$, is computed as:

$$h_{us} = h_1 - h_{f(1-2)} \qquad (7\text{-}31)$$

where:




*Reference: USDOT, FHWA, HY-7 (1986).*

**FIGURE 7-12**
Definition Sketch for Orifice Flow Computations

$h_{us}$ = Water surface elevation immediately upstream from the bridge, in feet

$h_1$ = Water surface elevation at the approach Section 1, in feet

$h_{f(1-2)}$ = Friction loss in the approach reach, in feet

The friction-loss term $h_{f(1-2)}$ is computed using Equation 7-20 with $K_c$ set to equal the bridge-opening conveyance, $K_3$. The velocity head at the approach section, $h_{v1}$, is assumed to be applicable throughout the approach reach. The submergence elevation, $Y_{1s}$, is either computed from input data defining bridge deck elevation and girder depth or specified by the user. The hydraulic depth, Z, within the bridge opening is computed as:

$$Z = A_{3net}/b \tag{7-32}$$

and the reference bed elevation ($Y_{bed}$) is estimated as:

$$Y_{bed} = Y_{1s} - Z \tag{7-33}$$

and the average upstream depth is:

$$Y_u = h_{us} - Y_{bed} \tag{7-34}$$

where:

Z = Height of the bridge opening above the streambed, in feet

$A_{3net}$ = Net area (total minus area of piers or piles) in the bridge opening for an elevation of $h_{us}$ (Equation 7-31), in feet

b = Bridge-opening length, in feet

$Y_{bed}$ = Reference streambed elevation, in feet

$Y_{ls}$ = Elevation of low steel for the bridge, in feet

$Y_u$ = Average upstream depth above the streambed, in feet

### 7.4.3 SUBMERGED ORIFICE FLOW

Flow through the bridge is handled as submerged orifice flow when the water surface is in contact with the girders for the entire flow length through the bridge (see Figure 7-13). Discharge through the opening for such a case is computed as:

$$QBO = C_D \; A_{3net} \; \sqrt{2g\Delta h} \qquad (7\text{-}35)$$

$$\Delta h = h_{us} + h_{v1} + h_{3n} \qquad (7\text{-}36)$$

where:

QBO = Submerged bridge opening discharge, in cfs

$C_D$ = Discharge coefficient

$A_{3net}$ = Total net flow area in the bridge opening, in square feet

$\Delta h$ = Head differential, in feet

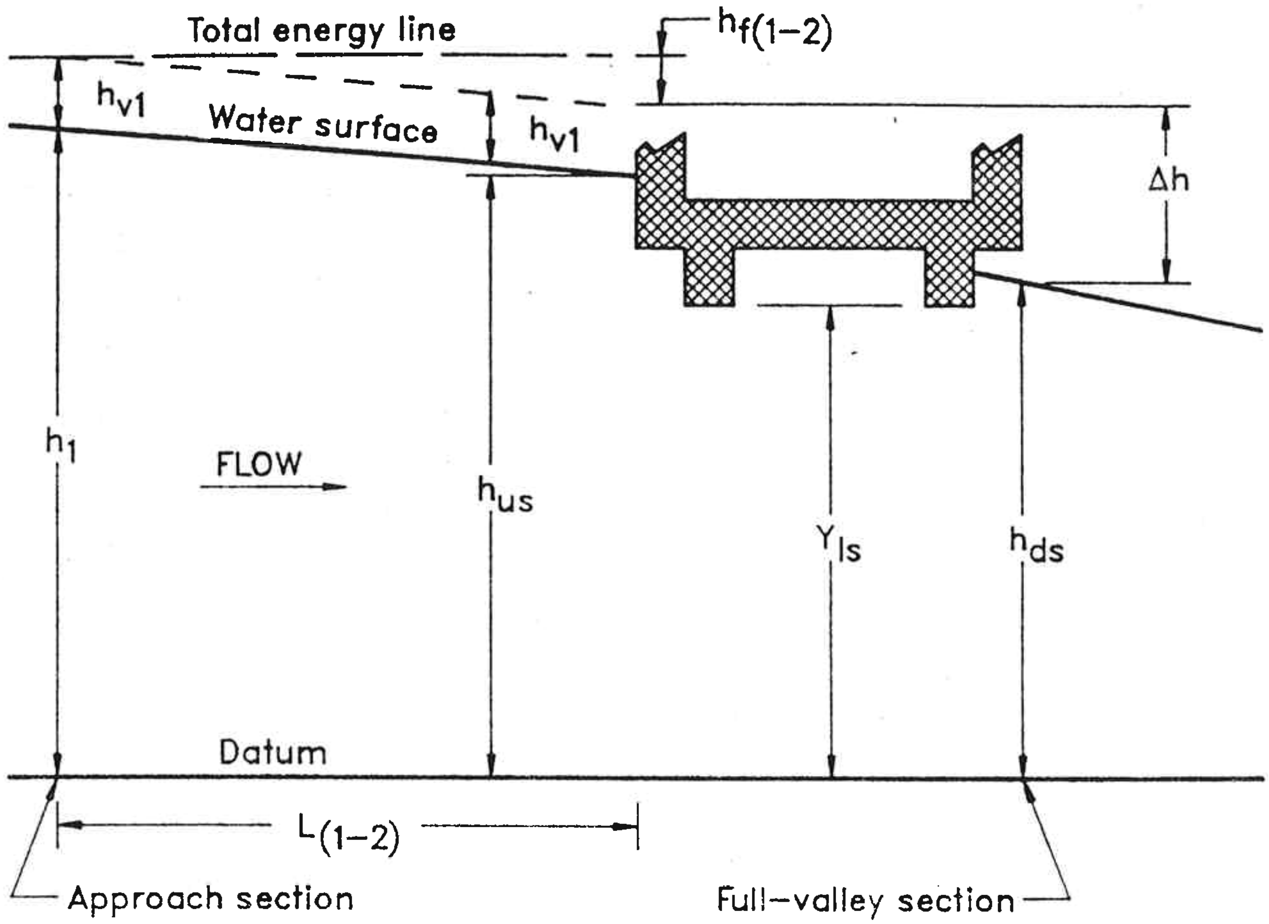
$h_{us}$ = Upstream elevation, in feet

$h_{v1}$ = Velocity head at Section 1, in feet

$h_{3n}$ = Natural profile elevation at Section 3, in feet

Based on laboratory data, a constant value of $C_D$ equal to 0.8 can be used to give the following form of Equation 7-35:

$$QBO = 6.42 \; A_{3net} \; \sqrt{\Delta h} \qquad (7\text{-}37)$$




*Reference: USDOT, FHWA, HY-7 (1986).*

**FIGURE 7-13**
Definition Sketch for Submerged Orifice Flow Computations

### 7.4.4 ROADWAY OVERTOPPING

When the water surface elevation immediately upstream from the embankment exceeds the minimum elevation along the top of the embankment, the embankment begins to function as a broad-crested weir. Fundamentals of this situation are covered in Chapter 5 on culvert hydraulics.

## 7.5 SCOUR AND DEGRADATION

Three types of interrelated phenomena should be considered when evaluating changes to bed level related to bridges: general scour, local scour, and degradation or aggradation.

### 7.5.1 GENERAL SCOUR

Scour occurs at contractions because the flow area becomes smaller than the normal stream and the average velocity and bed shear stress increase. As a result, the transport capacity of the stream increases at the contraction and more bed material is transported through the contracted section than is transported into the section. As the bed level is lowered, the velocity and shear stress decrease and equilibrium is restored when the transport rate of sediment through the contracted section is equal to the incoming rate.

General scour can be evaluated by considering the sediment transport and hydraulic properties in the vicinity of the contraction. The magnitude of general scour should be quantified before performing local scour calculations to establish an appropriate reference elevation.

Ignoring scour in water surface profile computations will give conservative backwater elevations, because scour can increase the waterway area of a bridge. A detailed procedure is presented in HDS-1 (USDOT, FHWA, 1978) for evaluating backwater elevations when the potential for scour exists. Although the scour-related reduction of backwater can be estimated, a design should not depend on scour as a means of enlarging the bridge waterway area and thereby lowering backwater.

An important consideration when the potential for scour is significant is the stability of bridge abutments and piers. A careful evaluation of velocity distributions and the stability of the streambed and embankment materials should be performed to verify that bridge abutments and piers are safe for selected design conditions. Spur dikes can be sized to provide additional stability.

### 7.5.2 LOCAL SCOUR

Local scour occurs in the bed of the channel around piers and embankments and is caused by the action of vortex systems induced by obstructions to flow. Local scour generally occurs independent of degradation, aggradation, and scour caused by contractions.

The basic mechanism causing local scour is the vortex of fluid resulting from the pileup of water on the upstream edge and subsequent acceleration of flow around the nose of the pier or embankment. The vortex erodes bed materials away from the base region. If the transport rate of sediment away from the local region is greater than the transport rate into the region, a scourhole develops. As the depth is increased, the strength of the vortex is reduced, the transport rate is reduced, equilibrium is reestablished, and scouring ceases.

The presence or absence of sand bars can affect the depth of local scour. Because the time required for sand bar motion is much longer than the time required for local scour, even if steady-state conditions exist, the depth of scour is likely to fluctuate with time when there are sand bars traveling on the channel bed. When the crest of the sand bar reaches the local scour area, the transport rate into the hole increases, the scourhole fills, and the scour depth temporarily decreases. When a trough approaches, there is less sediment supply and the scour depth increases to try to reestablish equilibrium in sediment transport rates. A mean scour depth between these oscillations is referred to as <u>equilibrium scour depth</u>. It is not uncommon (as determined in laboratory tests) to find maximum depths to be 30 percent greater than equilibrium scour depths. The depth that would be reached if no sediment was transported into the scourhole is the <u>clear water scour depth</u>.

The horseshoe vortex system is dominant at piers, causing deepest scour at the nose of the pier. The axis of this vortex, or the vortex line, is horizontal and wraps around the base of the pier in the shape of a horseshoe. The high velocities scour the bed.

A wake vortex system has vertical axes and develops because of blockage of the flow by the pier. The wake vortices are commonly seen as eddies. This vortex system suspends the scoured material and carries it downstream with the flow. Downstream of embankments, large wake vortices or eddies are set up that scour the downstream sides of the embankments, the river bank, and the streambed. Wake vortices downstream of piers may create sufficient velocities to cause bed scour if the piers are wide. For most piers, however, very little additional scouring is caused by wake vortices.

The shape of the pier is very significant with respect to scour depth because it reflects the strength of the horseshoe vortex at the base of the pier. A blunt-nose pier causes the greatest scour depth. Streamlining the front end of the pier reduces the strength of the horseshoe vortex and thus reduces the scour. Streamlining the downstream end of piers reduces the strength of wake vortices.

Detailed studies of scour around embankments have been made mainly in laboratories. There are very few case studies for scour at field installations. Empirical procedures based on the average velocity and local depth of flow can be used to estimate local scour.

### 7.5.3 DEGRADATION AND AGGRADATION

Many rivers have achieved a state of practical equilibrium throughout long reaches that can be considered stable; these rivers are known as graded streams by geologists and as poised streams by engineers. However, this does not preclude significant changes over a short period of time or over a period of years. Conversely, many streams contain long reaches that are actively aggrading and degrading. These aggrading and degrading channels may pose a definite hazard to any highway crossing or encroachment.

Regardless of natural channel stability conditions, man's local activities may produce major changes in river characteristics locally and throughout the entire reach. All too frequently, the result of a river improvement is a greater departure from equilibrium than that which originally prevailed. As an example, the channel downstream from an impoundment is likely to undergo degradation from sediment removal by storage, while the channel upstream from an impoundment is likely to experience aggradation caused by reduced velocities. These potential changes in the channel profile should be accounted for when new facilities are designed.

Good design should enhance the natural tendency of the stream toward poised conditions. To do so, an understanding of the direction and magnitude of change in channel characteristics caused by the actions of man and nature is required. This understanding can be obtained by taking the following actions:

1. Studying the river in a natural condition

2. Having knowledge of the sediment and water discharge

3. Being able to predict the effects and magnitude of man's future activities

4. Applying to these a knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers

Aggradation and degradation processes related to channel improvements are also discussed in Chapter 3. More detailed information can be found in the Training and Design Manual by the USDOT, FHWA (1975).

NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

# CHAPTER 8
## Detention/Retention Hydraulics

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# Chapter 8
# DETENTION/RETENTION HYDRAULICS

## SYNOPSIS

Retention/detention facilities are often a key component of stormwater management systems. They can control flooding levels as well as potentially adverse impacts associated with stormwater quality. This chapter provides a brief discussion of the uses for retention/detention facilities, along with fundamentals of detention routing calculations and an evaluation of land-locked retention. The Storage Indication Method is presented for performing final detention routing calculations. Key references for this chapter include publications by the American Public Works Association (1974 and 1981), the American Society of Civil Engineers (1982 and 1985), and Malcolm (1980).

## 8.1 USES

Retention refers to stormwater storage facilities without access to a positive outlet. Detention facilities offer temporary storage accompanied by controlled release of the stored water. Retention and detention can be incorporated separately or together in storage facility designs, as site conditions and management objectives require.

The use of retention and detention facilities for stormwater management has increased dramatically in recent years. The benefits of retention/detention systems can be divided into the two major control categories of quality and quantity.

### 8.1.1 QUALITY

Control of stormwater quality using retention/detention offers the following potential benefits:

1. Decrease in soil erosion

2. Control of sediment deposition

3. Improved water quality through stormwater filtration

### 8.1.2 QUANTITY

Controlling the quantity of stormwater using detention can provide the following potential benefits:

1. Mitigation of peak runoff rate increases caused by development

2. Prevention or reduction of downstream drainage capacity problems

3. Recharge of groundwater resources

4. Reduction or elimination of downstream outfall improvements

5. Maintenance of historic low flow rates by controlled discharge from storage

Design criteria for managing stormwater quantity by detention are typically based on limiting peak runoff rates to match one or more of the following values:

1. Historic rates for specific design conditions (i.e., post-development peak equals pre-development peak)

2. Discharge capacity of the downstream drainage system

3. A specified value or allowable discharge set by regulatory jurisdiction

For land-locked watershed areas, the total volume of runoff is critical and the mitigation of potential volume increases is generally accomplished by retention storage.

The use of retention and detention systems to reduce peak runoff rates or volumes to a desired value should be evaluated using a reservoir routing procedure. The two basic categories of detention facilities usually considered are <u>dry</u> and <u>wet detention</u>. Wet detention typically has a pool of water below the outlet elevation, while dry detention has an outlet elevation that is above the seasonal high water table. Definition sketches of dry and wet detention storage are presented in Figure 8-1.

## 8.2 DETENTION RESERVOIR ROUTING

The peak flow reduction obtained by a stormwater detention system can be evaluated by performing reservoir routing calculations, usually as a trial and error process. A hydrologic routing procedure called the Storage Indication Method is recommended for final routing calculations, since it explicitly accounts for site-specific conditions. Empirical relationships are also available to perform preliminary sizing calculations, but preliminary results should be confirmed using the Storage Indication Method.

### 8.2.1 STORAGE INDICATION METHOD

To use the Storage Indication Method, the following three basic relationships must be established:

1. Inflow hydrograph
2. Stage-storage curve
3. Stage-discharge curve

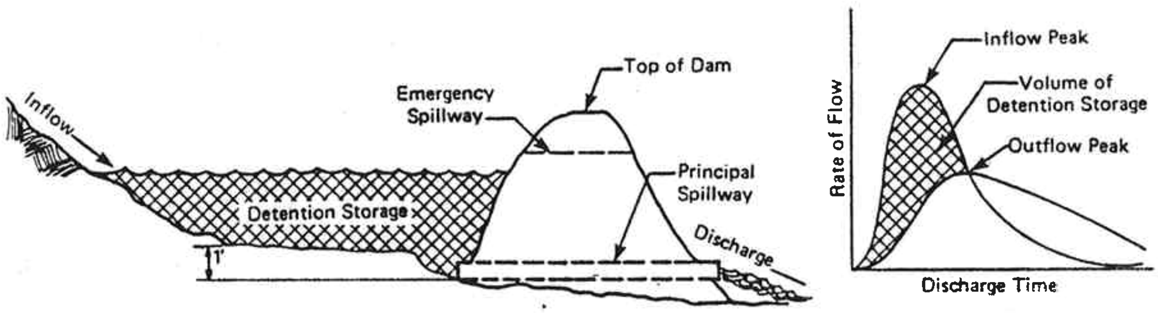
Development of each of these relationships should be based on site-specific data.

Once any two of these variables are known, the third is automatically fixed. Two categories of routing problems are often encountered that have the common factor of needing to lower the peak outflow rate to an acceptable value. In one type, the basin stage-storage curve and inflow hydrograph are known and the outlet works must be sized to obtain the minimum peak outflow from the basin. The other problem involves determining the storage volume required to obtain the desired peak outflow rate for a given design storm. Each of these problems involves a trial and error solution.

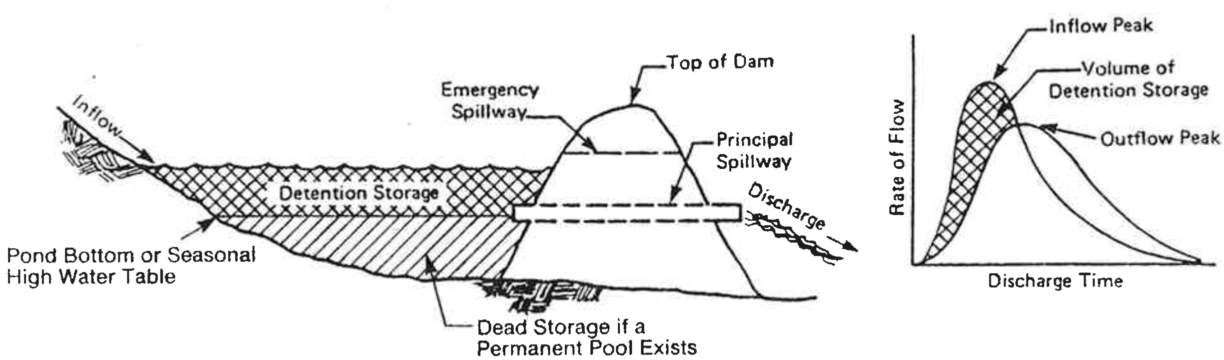
#### Inflow Hydrograph

Fundamentals for the development of an inflow hydrograph for design flood conditions are presented in Chapter 2.

NOTE: The principal spillway outlet elevation must be above the seasonal high water table elevation.




**A) Case 1—Side View of a Dry Detention Basin**




**B) Case 2—Side View of a Wet Detention Basin**

**FIGURE 8-1**
Definition Sketches of Dry and Wet Detention Storage

Procedures for developing synthetic runoff hydrographs can be found in Volume 2.

## Stage-Storage Curve

A stage-storage curve defines the relationship between the depth of water and storage volume in a reservoir. An example of a stage-storage curve is shown in Figure 8-2. The data for this type of curve are usually developed using a topographic map and the double-end area frustum of a pyramid or prismoidal formulas. The double-end area formula is expressed as:

$$V_{1,2} = \left(\frac{A_1 + A_2}{2}\right) d \tag{8-1}$$

where:

$V_{1,2}$ = Storage volume, in cubic feet, between elevations 1 and 2

$A_1$ = Surface area at elevation 1, in square feet

$A_2$ = Surface area at elevation 2, in square feet

d = Change in elevation between points 1 and 2, in feet

The frustum of a pyramid is expressed as:

$$V = \frac{1}{3}\, d \left(A_1 + \sqrt{A_1 A_2} + A_2\right) \tag{8-2}$$

where:

V = Volume of frustum of a pyramid, in cubic feet

d = Change in elevation between points 1 and 2, in feet

$A_1$ = Surface area at elevation 1, in square feet

$A_2$ = Surface area at elevation 2, in square feet

The prismoidal formula for trapezoidal basins is expressed as:

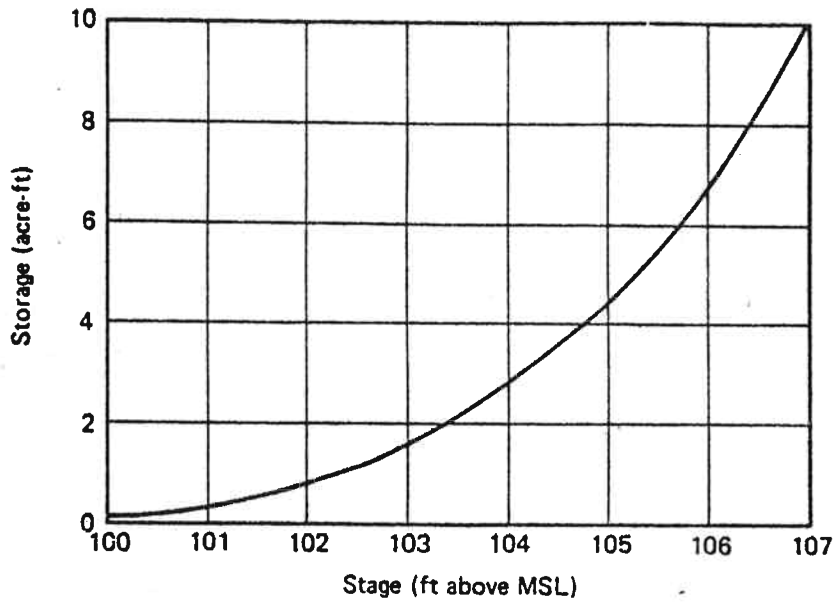



**FIGURE 8-2**
Example of a Stage-Storage Curve

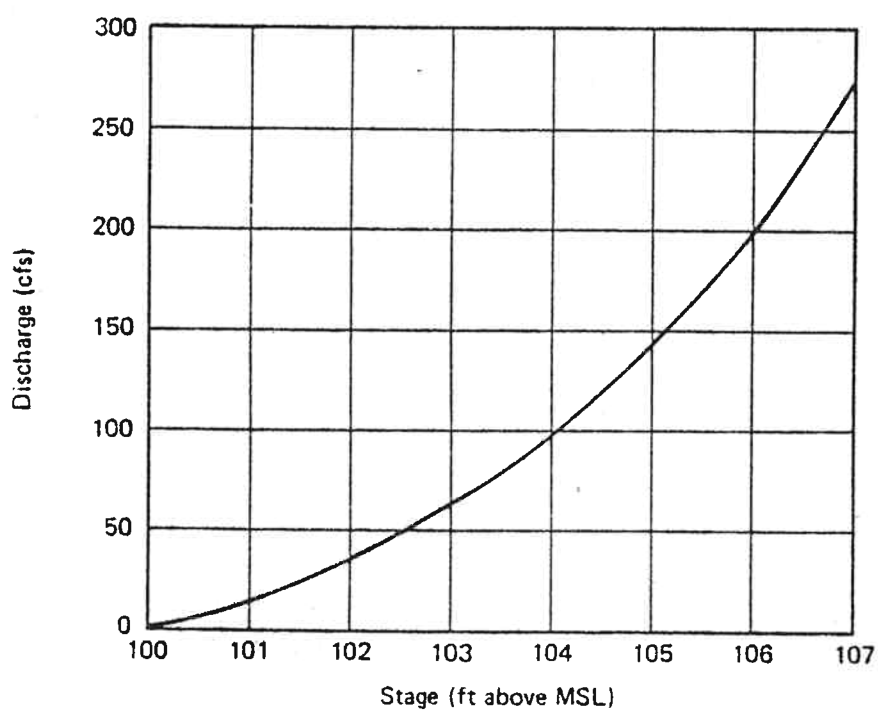



**FIGURE 8-3**
Example of a Stage-Discharge Curve

$$V = LWD + (L + W)\ ZD^2 + \frac{4}{3} Z^2 D^3 \qquad (8\text{-}3)$$

where:

- V = Volume of trapezoidal basin, in cubic feet
- L = Length of basin at base, in feet
- W = Width of basin at base, in feet
- D = Depth of basin, in feet
- Z = Side slope factor, ratio of horizontal to vertical

## Stage-Discharge Curve

A stage-discharge curve defines the relationship between the depth of water and the discharge or outflow from a storage basin. An example of a stage-discharge curve is presented in Figure 8-3. As illustrated in Figure 8-1, a typical stormwater storage basin has two spillways: principal and emergency. The principal spillway is usually designed with a capacity sufficient to convey the design storm without allowing flow to enter the emergency spillway. A pipe culvert is generally used for the principal spillway. Since this pipe culvert operates hydraulically in a manner identical to a culvert through an embankment, a stage-discharge curve for a pipe principal spillway can be developed using the culvert nomographs presented in Volume 2.

The emergency spillway is sized to provide a bypass for stormwater during a storm that exceeds the design capacity of the principal spillway. Selecting a magnitude for sizing the emergency spillway depends on the potential threat to downstream life and property if the storage basin embankment were to fail. A broad-crested weir is the type of structure often used for an emergency spillway. The stage-discharge curve of a broad-crested weir is expressed as:

$$Q = C\ L\ H^{3/2} \qquad (8\text{-}4)$$

where:

Q = Discharge, in cfs

C = Weir coefficient

L = Length of the weir, in feet

H = Height or head of water above the weir elevation, in feet

A typical value of the weir coefficient for a broad-crested weir is 3.0. Detailed information for determining specific values of the weir coefficient for various weir configurations is presented by Brater and King (1976).

In cases where culvert and broad-crested weir hydraulic relationships are not appropriate, more detailed information on stage-discharge relationships should be obtained. Reliable sources include the hydraulics handbook by Brater and King (1976) and a report by the American Society of Civil Engineers (1985).

## Routing Fundamentals

Routing techniques can be classified into the following two main categories:

1. Hydrologic
2. Hydraulic

Hydrologic routing techniques are based entirely on the continuity equation, while hydraulic techniques use both the continuity equation and the dynamic equation of motion. In practice, hydrologic routing techniques are usually adequate for most stormwater systems. For information on hydraulic routing techniques, references by Linsley, Kohler, and Paulhus (1982), Viessman et al. (1977), Chow (1959, 1964), Henderson (1966), or French (1985) are suggested.

The continuity equation, on which hydrologic routing techniques are based, requires that the rate of change of storage account for all mass flow into and out of the facility being evaluated. Mathematically, the continuity equation is expressed as:

$$I - O = \frac{\Delta S}{\Delta t} \qquad (8\text{-}5)$$

where:

$I$ = Inflow rate, in cfs

$O$ = Outflow rate, in cfs

$\frac{\Delta S}{\Delta t}$ = Time rate of storage, in cfs

A finite difference approximation to Equation 8-5 can be expressed as:

$$\left[\frac{I_1 + I_2}{2}\right] - \left[\frac{O_1 + O_2}{2}\right] = \frac{S_2 - S_1}{\Delta t} \qquad (8\text{-}6)$$

where:

$I_1$ and $I_2$ = Inflow rates at times 1 and 2, respectively, in cfs

$O_1$ and $O_2$ = Outflow rates at times 1 and 2, respectively, in cfs

$S_1$ and $S_2$ = Storage volumes at times 1 and 2, respectively, in cubic feet

$\Delta t$ = Time change between periods 1 and 2, in seconds

By rearranging Equation 8-6, the following equation is used to perform a reservoir routing using the Storage Indication Method:

$$S_2 + \frac{O_2}{2}\Delta t = \left[S_1 - \frac{O_1}{2}\Delta t\right] + \left[\frac{I_1 + I_2}{2}\Delta t\right] \qquad (8\text{-}7)$$

where:

$S_1$ and $S_2$ = Storage volumes at times 1 and 2, respectively, in cfs

$O_1$ and $O_2$ = Outflow rates at times 1 and 2, respectively, in cfs

$I_1$ and $I_2$ = Inflow rates at times 1 and 2, respectively, in cfs

$\Delta t$ = Time change between periods 1 and 2, in seconds

The only unknown of Equation 8-6 for any time increment is the left-hand side of the equation. To simplify calculations, a pair of storage characteristics curves can be developed that provide a direct determination of $O_2$ and $S_2$, given the value of the right-hand side of the equation. The storage characteristics curves are developed using appropriate storage-discharge data for the basin and outlet control configuration being considered.

### 8.2.2 PROBLEM TYPES

The most common type of detention routing problem requires knowing the design storm return period or inflow hydrograph and peak outflow or allowable discharge from the detention basin. A trial and error procedure is used to calculate the storage volume required.

A less common routing problem involves preventing storage basin overflow during the design storm for a given basin size and return period. In such a case, the magnitude of the peak flow reduction is fixed. A trial and error procedure will be required to find a solution, as only the stage-storage curve is known explicitly.

## 8.3 LAND-LOCKED RETENTION

Karst topography occurs in the Nashville area and can result in watershed areas tributary to land-locked retention. Such retention is typically located in depressions that can exhibit highly variable outflow characteristics, depending on local hydrogeologic conditions. Historical measurements provide the best information for evaluating the outflow characteristics of a specific depression. A graphical mass flow routing procedure, which involves using site-specific measurements to develop inflow and outflow curves, can provide a basis to evaluate the performance of such systems.

A mass inflow curve is fairly simple to develop with appropriate hydrologic procedures. The development of a mass outflow curve, however, can be quite complex and often requires substantial judgment and local experience.

NASHVILLE STORMWATER MANAGEMENT MANUAL

# VOLUME 3—THEORY

# CHAPTER 9
# Erosion and Sediment Control

TABLE OF CONTENTS

# Chapter 9
# EROSION AND SEDIMENT CONTROL

## SYNOPSIS

Soil erosion is caused primarily by the forces of water and wind. Because wind erosion is not a concern for most stormwater management projects, theoretical aspects are not presented. Fundamentals of soil detachment and transport are discussed along with the Universal Soil Loss Equation (USLE) and sediment delivery ratios.

This chapter classifies control practices into sediment control and erosion control measures. Sediment control measures are generally structural in nature and are intended to prevent detached soil particles from leaving a particular project site. Erosion control measures can be either structural or non-structural and are intended to prevent soil particles from becoming detached. Key references include Wischmeier and Smith (1978), Schwab et al. (1981), the Transportation Research Board (1980), Gray and Leiser (1982), Graf (1971), and Goldman et al. (1986).

## 9.1 FUNDAMENTALS

The soil erosion process can be broken into two parts: (1) detachment and (2) transport. Qualitative descriptions of soil detachment and transport processes are given below.

### 9.1.1 DETACHMENT

Soil detachment is caused by raindrop impact and by sheet, rill, gully, and stream runoff. Each of these processes also acts to transport soil away from the point of detachment. In general, soil detachability increases as soil particle size increases, and the potential for soil transport increases with a decrease in particle size. For example, clay particles are initially more difficult to detach than sand, but are more easily transported. Cohesive soils are also less erodible than noncohesive soils.

Raindrop erosion is soil detachment resulting from the impact of raindrops on soil particles or shallow water surfaces, such as puddles or sheet flow. From an energy standpoint, raindrop erosion is more important than runoff erosion, because raindrop velocities typically range from 20 to 30 feet per second, while runoff velocities typically range from 1 to 6 feet per second.

Factors affecting the magnitude of soil detachment include slope, wind, soil particle size and aggregation, and soil cover. Mulch or vegetative cover reduces soil detachment from rainfall by absorbing erosive energy. Raindrop impact on bare soil not only causes detachment, but also decreases aggradation and causes deterioration of soil structure. Raindrop erosion becomes more serious as the land slope increases.

Runoff erosion can occur as a result of sheet, rill, gully, or stream flow. The eroding and transporting capability of sheet flow are functions of the depth and velocity of runoff for a given size, shape, and density of soil particle or aggregate. Erosion from sheet flow rapidly converts to rill erosion as small but well-defined channels, or streamlets, are formed. In practice, rills are defined as small channels that can be removed by normal earth-working operations. Rill erosion can become a significant source of soil loss if runoff control measures are not installed.

Gully erosion is an advanced stage of rill erosion, just as rill erosion is an advanced stage of sheet erosion. The rate of gully erosion depends primarily on the runoff-producing characteristics of the watershed; the drainage area; soil characteristics; the alignment, size, and shape of the gully; and the slope of the channel. Evaluation and prediction of gully erosion is difficult, because contributing factors are not well-defined and field records are generally inadequate. As gullies work upstream, the most severe erosion occurs at the upper end or gully head. This is often referred to as head cutting. The most stable section of the gully is generally at the lower end below the area of active head cutting.

The distinction between stream channel and gully erosion is generally based on location within the watershed and duration of flow. Stream channel erosion is generally a result of continuous flow occurring in the relatively flat lower end of a watershed, while gully erosion generally occurs in intermittent channels located in the steeper upstream reaches of a watershed.

Two types of erosion are associated with stream channels. Bank erosion occurs when runoff flows over the side of stream banks and erodes the stream bank itself (also called sloughing). This type of erosion can be increased by the removal of vegetation or by performing earthwork too near the stream banks. Scour erosion, which is generally more significant than bank erosion, occurs below the water surface and is influenced by the velocity and direction of flow, depth and width of the channel, and soil texture. Improper alignment of channel encroachments and the presence of obstructions to flow can increase meandering, which is generally caused by scour erosion. Scour erosion can also be aggravated by the improper placement and protection of pipe outlet facilities.

### 9.1.2 TRANSPORT

Soil that has been eroded can be transported by suspension, by saltation, and by bed load movement. The dominant action of these processes is observed in stream flows; they can also occur in runoff prior to entering a main stream channel. Important variables that control sediment transport processes include velocity of flow; turbulence; grain size and distribution, cohesiveness, and specific gravity of transported materials; channel roughness; obstructions to flow; and the availability of materials for transport.

Suspended sediment transport occurs when sediment travels without contacting the channel bottom. Most measurements of sediment transport are limited to suspended sediment, because other modes of transport are difficult to measure.

Saltation refers to the process of sediment particles bouncing or skipping along the stream bed. The height of bounce is directly proportional to the ratio of particle density to fluid density. The quantity of sediment transported by saltation is generally small when compared to total sediment transport.

Bed load refers to sediment that is rolled or pushed along the bottom by the force of flowing water. Bed load sediment is in almost continuous contact with the stream bed and, although saltation and bed load are two distinct types of transport, saltation is generally included in bed load.

### 9.1.3 UNIVERSAL SOIL LOSS EQUATION (USLE)

The USLE is an empirical procedure for estimating soil losses from upland slopes. Although it was developed for agricultural purposes, the USLE has been successfully adapted to construction sites. The equation contains factors that relate to rainfall, soils, runoff, and erosion control practices. The gross erosion produced by rill and inter-rill erosion from a field-sized upland area can be estimated using the USLE expressed as:

$$A = RKLSCP \tag{9-1}$$

where:

A = Soil loss, in tons per acre, for the time period selected for R

R = Rainfall factor

K = Soil erodibility factor, in tons per acre per R unit

LS = Length-slope factor, dimensionless

C = Cropping management factor, dimensionless

P = Erosion control practice factor, dimensionless

To estimate the sediment yield at some point beyond a field-sized upland area, additional erosion from gullies and stream banks must be added and deposition subtracted. The gross erosion from an upland area can be estimated for average annual, average monthly, return period annual, or return period single-storm time scales. Although it is not suitable for predicting the actual soil loss from specific design storms, a realistic estimate of the average soil loss for a number of storms with a specified return frequency can be obtained.

Numerical values for each of the parameters in the USLE must be determined for each problem considered. Guiding principles and data for determining these parameters in Nashville are presented in Volume 2.

## Rainfall Factor, R

The numerical value used for the rainfall factor, R, in the USLE must quantify the effect of raindrop impact and provide relative information on the amount and rate of runoff likely to be associated with the rainfall. Higher R values indicate a higher potential for rainfall to cause erosion.

Research has indicated that the value of R can be determined directly as the product of the total storm energy, E, and the maximum 30-minute rainfall intensity, I. The relation of this product parameter, EI, to soil loss has been found to be linear, such that values determined from individual storms are additive. Thus, the quantitative measure of the erosion potential for a given period of rainfall is the sum of the product EI values within that period of rainfall.

Wischmeier and Smith (1978) have analyzed 22 years of rainfall data at various locations across the United States to determine appropriate rainfall factors for various time scales. Storm events of less than 1/2 inch and those separated from other storm events by more than 6 hours were omitted from the rainfall factor calculations unless as much as 0.25 inch fell in 15 minutes. The results of this rainfall analysis are presented for average annual, average monthly, return period annual, and return period single-storm time scales.

## Soil Erodibility Factor, K

The soil erodibility factor, K, in the USLE is defined as the rate of soil loss per erosion index unit, measured on a unit plot for a given type of soil. The unit plot, which is used to determine K values experimentally, has been defined arbitrarily to match those field conditions under which erosion measurements have been made. A unit plot is 72.6 feet long, with a uniform lengthwise slope of 9 percent, in continuous fallow tilled up and down the slope. Continuous fallow, for this purpose, is land that has been tilled and kept free of vegetation for more than 2 years.

More than 25 characteristics of a soil affect its response to water erosion. A few of the most important characteristics include the texture and organic matter of the surface layer, size and stability of structural aggregates in the surface layer, permeability of the subsoil, and depth to slowly permeable layers. Several K factors may be determined for a soil series depending on the profile characteristics.

## Length-Slope Factor, LS

Theoretically, the effect of slope length and steepness on soil loss are considered separately. However, in practice, these two factors are combined in a single length-slope topographic factor, LS. The LS value is defined as the ratio of soil loss per unit area from a given site to that from a site with a 72.6-foot length and uniform slope of 9 percent under otherwise identical conditions. A site-specific value of LS can be estimated using the following empirical relationship presented by Wischmeier and Smith (1978):

$$LS = \left(\frac{\lambda}{72.6}\right)^m (65.41 \sin^2\theta + 4.56 \sin\theta + 0.065) \qquad (9\text{-}2)$$

where:

$LS$ = Length-slope factor, dimensionless

$\lambda$ = Slope length, in feet

$\theta$ = Angle of slope

$m$ = 0.5 if the percent slope is 5 or more; 0.4 on slopes of 3.5 to 4.5 percent; 0.3 on slopes of 1 to 3 percent; and 0.2 on slopes less than 1 percent

Equation 9-2 will underestimate the soil loss to the foot of a convex slope and overestimate the soil loss from a concave slope. Although concave and convex slopes may have the same average slope and length, soil losses will be different. Other factors being equal, the convex slope will have the higher sediment production, because the steepest slope is closer to the receiving water.

Control Practice Factor, CP

Wischmeier and Smith (1978) originally defined two factors for evaluating erosion in agricultural applications. The cover factor, C, was defined as the ratio of soil loss from an area with specified cover and crop management conditions to the soil loss of an identical area in tilled continuous fallow. The support practice factor, P, was defined as the ratio of soil loss from an area with a support practice such as contouring, stripcropping, or terracing, to that of an area with straight-row farming up and down the slope.

Since conditions of a cleared construction site are similar to those of a tilled field with no vegetation, a combination of these two factors, known as the control practice factor, CP, can be used to evaluate construction erosion control practices. Standard control practices include surface stabilization, runoff control, and exposure scheduling. Since sediment trapping devices act to control sediment loss downstream from the point of erosion, sediment retained by these devices is generally accounted for by developing a sediment delivery ratio.

Unprotected soil has a CP value of 1.0. If the conditions being considered vary significantly from unprotected soil conditions, a CP value other than 1.0 must be determined to establish baseline conditions for developing an erosion and sediment control plan. Having established a baseline, the USLE can be used to evaluate various control alternatives.

### 9.1.4 SEDIMENT DELIVERY RATIOS

The USLE provides a practical method for estimating the gross soil loss from a field-sized upland area. Many natural or manmade opportunities for sediment deposition can exist from the point of origin to the design point in question. It is also possible that additional soil may be eroded from the gullies or stream banks that transport stormwater from the point of origin to the design point in question. Therefore, the sediment yield at a particular point in a stormwater conveyance system can be greater or smaller than the gross soil loss estimated using the USLE. A sediment delivery ratio is usually used to quantify the sediment transported to a particular design point. The sediment delivery ratio is defined as:

$$D = \frac{Y}{A_T} \tag{9-3}$$

where:

$D$ = Sediment delivery ratio without manmade controls

$Y$ = Sediment yield from a watershed without manmade controls, in tons/acre for the specified time period

$A_T$ = Total gross erosion from the watershed, which includes upland sheet and rill erosion, gully erosion, and stream erosion, in tons/acre for the specified time period

The total gross erosion, $A_T$, includes additional erosion that is not accounted for by the USLE. The sediment yield, Y, is the total gross erosion minus deposition that occurs in the watershed. Very limited information is available to quantify sediment delivery ratios without actual field data.

To evaluate the need for erosion and sediment control practices at a construction site, the sediment delivery ratio with controls can be estimated as:

$$D_C = \frac{A - T}{A_T} \tag{9-4}$$

where:

$D_C$ = Sediment delivery ratio with controls

$A$ = Sediment yield with controls, in tons/acre/year (from USLE, Equation 9-1)

$A_T$ = Target total erosion from a project site, in tons/acre/year

$T$ = Sediment trapped onsite, in tons/acre/year

By selecting a desirable value for $A_T$, a proposed erosion plan should be designed to keep $D_C$ at or below 1.0.

## 9.2 EROSION CONTROL

The control practices considered in this chapter are classified as either erosion control or sediment control measures. In general, erosion control practices are designed to prevent soil particles from being detached, whereas sediment control practices prevent the detached particles from leaving the site or from entering a receiving water. Sediment control measures are generally structural in nature, while erosion control measures can be either structural (such as diversions) or non-structural (such as mulches).

Three general classifications of erosion control measures are presented in this section: (1) surface stabilization, (2) exposure scheduling, and (3) runoff control. Appropriate control measures for each classification are defined in sufficient detail to identify applicable conditions for employing each of these measures. Quantitative performance data for use with the USLE are presented in Volume 2, along with general information related to design considerations.

Construction erosion control measures are usually temporary, i.e., intended to function for only the duration of construction. However, the long-term or permanent stabilization of a site should be considered concurrently with the development of a temporary plan to best use the resources available for a project.

### 9.2.1 SURFACE STABILIZATION

Surface stabilization control measures include mulches, seeding and vegetation, chemical binders or tacks, coats, and other materials.

#### Mulches (Temporary)

A mulch is a layer of material applied to the soil surface for temporary soil stabilization and to help establish plant cover by holding in moisture and preventing the loss of seed. The major types of mulching material include straw or hay and wood chips. Mulches are practical on graded or cleared areas for 6 months or less where seedings may not have a suitable growing season to produce an erosion-resistant cover. Final grading is not required prior to mulching; however, mulch may be applied after final grade is reached. Whenever structural erosion control features are used, they should be installed prior to mulching.

Seeding and Vegetation (Temporary and Permanent)

Surface stabilization by vegetation includes temporary seeding, permanent seeding, sod, vines, shrubs, and trees. The types of seeding and vegetation, application rates, site preparation, and fertilizer and water requirements are discussed in Volume 2.

Chemical Binders and Tacks (Temporary)

Synthetic binders and tacks are sprayed on bare soils or mulches to bind soil particles or mulch material, reduce moisture loss, and enhance plant growth. A chemical binder or tack is a temporary erosion control measure and may be applied with seed, lime, and fertilizers. Chemical binders and tacks provide a viable alternative to seeding if construction occurs at a time when seeding is not feasible.

Other Materials

Other stabilization materials include nettings and plastic filter sheets for temporary control, and dumped-riprap for permanent control. Nettings of fiberglass, plastic, and paper yarn can be used to anchor straw, hay, wood chips, or grass and sod in drainageways and in other areas subject to concentrated runoff. Plastic filter sheets consist of porous fabric woven from polypropylene monofilament yarns. They are lightweight, porous, strong, abrasion-resistant, and unaffected by saltwater. Dumped-riprap is stone or broken concrete dumped in place on a filter blanket or prepared slope to form a well-graded mass with minimum voids.

### 9.2.2 EXPOSURE SCHEDULING

The sequence and duration of exposure of cleared land to the erosive forces of rainfall can have a significant effect on the gross soil loss from a site. The impact of exposure scheduling can be evaluated with the USLE by using the monthly distribution of the rainfall factor, R, which can be obtained from data presented in Volume 2.

Exposure scheduling can be evaluated by determining the appropriate surface stabilization factors for the sequence of land covers being considered. For example, a site may begin the year as an undisturbed woodland, followed by clearing for construction, temporary seeding, and then permanent seeding. Using the length of time and season during which each of these land cover types will exist, a weighted surface stabilization factor can be calculated using the monthly distribution of the rainfall factor, R.

### 9.2.3 RUNOFF CONTROL

Runoff control measures include diversions, down drains, level spreaders, and check dams.

#### Diversions (Temporary or Permanent)

Any structure that slows runoff or diverts runoff away from downslope areas can reduce erosion. Types of diversion structures include dikes, swales, and channels, which can function as temporary or permanent facilities.

A diversion dike is a ridge of compacted soil placed above, below, or around a disturbed area to intercept runoff and divert it to a stable area. Generally, a diversion dike is the least durable diversion structure and is best used to provide protection for short periods of time and when relatively small amounts of runoff must be handled. It is often used above a newly constructed cut and fill slope to prevent excessive erosion of the slope until more permanent control features are established. Where the ground slope is not steep, it is also used before graded slopes to divert sediment-laden runoff into sediment traps or basins. Once the slope is stabilized, the diversion dike is removed.

A diversion swale is an excavated, temporary drainageway used above and below disturbed areas to intercept runoff and divert it to a safe disposal area. A diversion swale can be constructed at the perimeter of a disturbed area to transport sediment-laden water to a sediment trap or sediment basin. The swale is left in place until the disturbed area is permanently stabilized. A diversion swale can be constructed in conjunction with a dike to prevent stormwater from entering a disturbed area. Although it is generally a temporary feature, with careful planning a swale could also become part of the permanent drainage system.

A diversion channel is a permanent or temporary drainageway constructed by excavating a ditch along a hillside and possibly placing a soil dike along the downhill edge of the ditch with the excavated soil. In other words, it can be a combination of a ditch and a dike. Although diversion channels can be used in place of temporary structures such as diversion dikes and swales, they are mainly used to provide more permanent runoff control on long slopes subject to heavy flow concentrations. Diversion channels can be used to intercept runoff upgradient of a roadway to prevent off-site flow from entering roadway gutter and inlet facilities.

## Slope Drains and Down Drains (Temporary or Permanent)

Down drain structures (pipes) and slope drains (paved or sodded channels) can be used as temporary or permanent structures to conduct concentrated runoff safely down a slope. Such structures are often used to help dispose of water collected by diversion structures.

A paved slope drain is a channel lined with bituminous concrete, portland cement concrete, or comparable nonerodible material (such as grouted riprap), placed to extend from the top to the bottom of a slope. A paved slope drain is generally used where a concentrated flow of surface runoff must be conveyed down a slope without causing erosion. When flow is supercritical, pipe down drain should be used to prevent flow from leaving the channel.

A down drain generally consists of either corrugated metal pipe or flexible tubing, together with a prefabricated entrance section, and is temporarily placed to extend from the top to the bottom of a slope.

## Level Spreaders (Permanent)

A level spreader is generally a permanent outlet constructed at zero percent grade across the slope to convert a concentrated flow of sediment-free runoff (e.g., from diversion outlets) into sheet flow and to discharge it at nonerosive velocities onto undisturbed areas stabilized by existing vegetation. The level spreader should be used only in those situations where the following conditions apply:

1. The spreader can be constructed on undisturbed soil.

2. The area directly below the level lip is stabilized by existing vegetation.

3. The drainage area above the spreader is stabilized by existing vegetation.

4. The materials used are rigid and nonerodible, such as concrete or asphalt, and a fixed grade can be maintained.

5. The water will not be reconcentrated immediately below the point of discharge.

### Check Dams (Permanent)

A check dam is generally a permanent structure used to maintain subcritical flow and thus stabilize the grade or control head cutting in natural or artificial channels. Check dams are used to reduce or prevent excessive erosion by reducing velocities in watercourses or by providing partially lined channel sections or structures that can withstand high flow velocities. Check dams are used where the following conditions apply:

1. The capacity of earth and/or vegetative measures is exceeded in the safe handling of water at permissible velocities.

2. Excessive grade or overall conditions occur.

3. Water must be lowered from one elevation to another.

## 9.2.4 OUTLET PROTECTION

Outlet protection entails providing de-energizing devices and erosion-resistant channel sections between drainage outlets and stable downstream channels. The channel sections may be rock-lined, vegetated, paved with concrete, or otherwise made erosion-resistant. The purpose of outlet protection is to convert pipe flow to channel flow and reduce the velocity of the water consistent with the channel lining. The flow of water can then be conveyed to a stable existing downstream channel without causing erosion.

This practice is applicable to storm sewer outlets, road culverts, and paved channel outlets discharging into natural or constructed channels, which, in turn, discharge into existing streams or drainage systems. The appropriate treatment should be provided along the entire length of the flow path from the end of the conduit, channel, or structure to the point of entry into an existing stream or publicly maintained drainage system. More detailed information on outlet protection is presented in Chapter 10.

## 9.3 SEDIMENT CONTROL

Sediment control practices are designed to prevent detached soil particles from leaving a particular site. The three general types of sediment control measures defined and discussed in this section are straw bale dikes, sediment traps, and sediment basins. The stabilization of construction entrances to control sediment is considered as well.

### 9.3.1 STRAW BALE DIKES (TEMPORARY)

This temporary measure is a dike constructed of straw bales, normally installed across or at the toe of a slope, with a life expectancy of 3 months or less. A straw bale dike is used to intercept and detain small amounts of sediment from unprotected areas of limited extent; it is a stop-gap measure applicable only under the following conditions:

1. No other practice is feasible.

2. There is no concentration of water in a channel or other drainageway above the barrier.

3. Sheet and rill erosion would occur.

4. Contributing drainage area is less than 1/2 acre and the length of slope above the dike is less than 100 feet.

Straw bales should be placed in a row with ends tightly abutting adjacent bales. Each bale should be embedded a minimum of 4 inches into the soil. In addition, bales should be securely anchored in place by stakes or rebars driven through the bales.

### 9.3.2 TRAPS (TEMPORARY OR PERMANENT)

A sediment trap is generally a temporary basin formed by an excavation and/or an embankment to intercept sediment-laden runoff and to trap and retain the sediment. In so doing, drainageways, properties, and rights-of-way below the trap are protected from sedimentation. If maintained by periodic cleaning, traps can sometimes be used as permanent facilities.

An earth outlet sediment trap consists of a basin formed by excavation and/or an embankment. The trap has a discharge point over or cut into natural ground. A pipe outlet sediment trap consists of a basin formed by an embankment or a combination of an embankment and excavation. The outlet for the trap is through a perforated riser and a pipe through the embankment.

### 9.3.3 BASINS (TEMPORARY OR PERMANENT)

A sediment basin is usually a temporary facility constructed along a flow path to capture sediment carried by runoff from a cleared construction site. The basin is formed by placing an earthen dam across the watercourse, by excavating a depression, or by a combination of the two. The purpose of a sediment basin is to protect drainageways, properties, and rights-of-way below the sediment basin from sedimentation.

Sediment basins are installed below construction sites, on or adjacent to the major watercourses. They act as a last line of defense against offsite sediment damage. If maintained by periodic cleaning, they can sometimes be used as permanent facilities.

A sediment basin should be constructed only under the following conditions:

1. When failure would not result in loss of life, damage to buildings, or interruption of service from public roads or utilities

2. When it will be removed within a specified period of time or maintained at regular intervals to restore its sediment storage capabilities

### 9.3.4 CONSTRUCTION ENTRANCES (TEMPORARY)

All points of access to a construction site should be stabilized to reduce or eliminate the tracking or flowing of sediment onto public rights-of-way. A stabilized pad of crushed stones is to be placed at entrances of construction sites for this purpose. Maintenance of such entrances may require periodic top dressing with additional stone as conditions demand or cleanout of any facilities used to trap sediment.

In some cases, wheels of construction vehicles should be cleaned prior to leaving the construction site. When appropriate, a stabilized area (one with crushed stone) that drains into a sediment trap or basin should be washed.

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# CHAPTER 10
## Outlet Protection

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# Chapter 10
# OUTLET PROTECTION

## SYNOPSIS

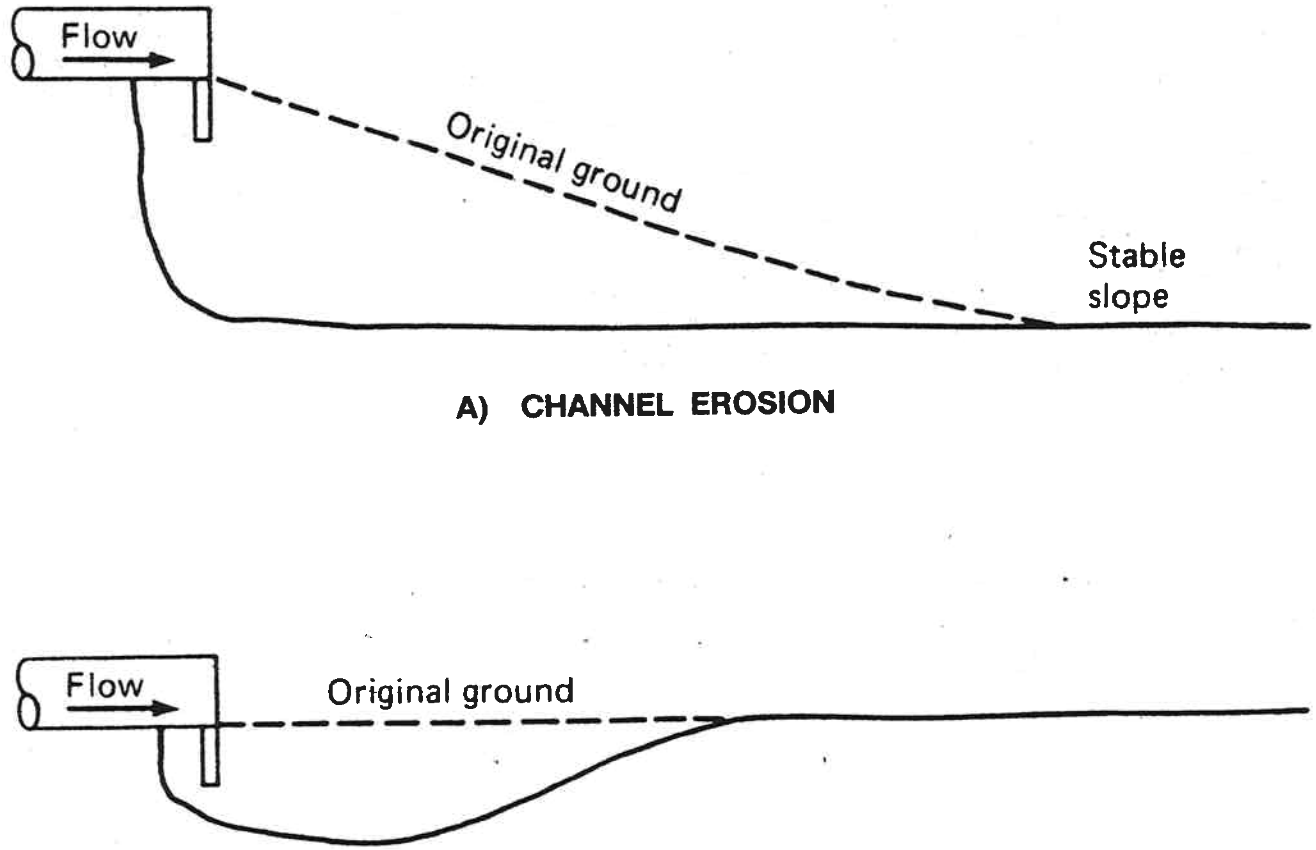
When a pipe or culvert discharges into a natural channel, potential exists for erosion at the outlet. The ground surface immediately downstream of the conduit may be eroded and the conduit itself can be undermined. To control such erosion, outlet protection devices can be used to reduce the velocity of the flow before it discharges into the natural channel. The type of device used depends on the flow characteristics, site layout, cost, and suitability for a specific location.

## 10.1 TYPES OF SCOUR

Figure 10-1 illustrates the two types of outlet erosion: gully scour and scourholes. Gully scour occurs when the stability limits of the soil are exceeded because of the flow velocity and the natural ground slope. In such a case, outlet protection should be provided to the point where the slope becomes gentle enough to be stable. For a steep slope downstream of a discharge structure, gully scour should be considered in the design of outlet protection.

Scourholes are a more localized form of erosion, caused by flow expansion from the conduit into the wider natural channel. Even when the channel slope is negligible, scourholes can form from the impact of the high velocity outflow on the ground and the turbulence of the flow expansion.

For both types of outlet erosion, the solution is to reduce the flow velocity or energy before flow is discharged onto the natural ground surface. In the case of a scourhole, the hole itself is an energy dissipator and will only enlarge to a certain size as a function of the flow conditions (see Volume 2). If the expected scourhole size is acceptable, it may simply be allowed to develop. Because a scourhole is usually undesirable, however, some type of outlet protection structure is required. Protection should always be provided if there is potential for gully scour.




B) SCOURHOLE

*Reference: Bohan (1970).*

**FIGURE 10-1**
Types of Outlet Erosion

## 10.2 TYPES OF OUTLET PROTECTION

One major type of outlet protection is an impact structure, which dissipates energy through the impact of the outflow stream on a wall, impact blocks, or other obstacle. Some energy is also dissipated by the resulting turbulence, but most is lost during impact. The second major type of outlet protection dissipates energy by forcing the flow to go through critical depth and a hydraulic jump. Energy is lost when the jump occurs and in the resulting turbulence. In general, impact type dissipators are more efficient and less expensive then hydraulic jump dissipators.

The United States Bureau of Reclamation (USBR) has developed descriptions for selected types of energy dissipators, listed in Table 10-1. In addition to the USBR structures defined, riprap aprons or basins are effective energy dissipators. Riprap aprons are the simplest impact structures; energy is lost through the impact of the flow on angular rocks and the resulting turbulence. In a riprap basin, the flow drops into a rock-lined basin, where a hydraulic jump occurs. Additional energy is lost through turbulence and impact with the rocks. Riprap structures are generally a cost-effective choice for many small applications.

Suggested guidelines for selecting the type of outlet protection to use for various Froude number and outlet velocity conditions are presented in Volume 2.

## 10.3 HYDRAULIC JUMPS

The hydraulic jump is a natural phenomenon that occurs when supercritical flow changes to subcritical flow. This abrupt change in flow conditions can be accompanied by considerable turbulence and dissipation of energy. The effectiveness of a hydraulic jump for outlet erosion protection depends on the flow conditions, which can be evaluated using the Froude number, and the structure designed to contain the jump.

### 10.3.1 JUMP CATEGORIES

Critical flow exists with an upstream Froude number of 1.0, and a jump cannot occur. When the Froude number is greater

Table 10-1
SUMMARY OF OUTLET PROTECTION STRUCTURE TYPES

| Type | Primary Means Of Energy Dissipation | Description |
|---|---|---|
| USBR Type II | Hydraulic jump | Used on high spillways and large canal structures for $Fr > 4.5$. Contains row of chute blocks at basin inlet and a dentated end sill. |
| USBR Type III | Hydraulic jump | Used on small spillways, outlet works, etc., for $Fr > 4.5$. Contains row of inlet chute blocks, row of baffle piers, and solid end sill. |
| USBR Type IV | Hydraulic jump | Used when $2.5 < Fr < 4.5$. Uses chute blocks and solid end sill and may also be followed by rafts or other wave suppressors. |
| USBR Type VI (Baffled Outlet) | Impact | Used for pipe or open channel outlets. Uses a small box and vertical hanging baffle wall, with flow energy reduced by impact with wall and flow emerging beneath the baffle. |
| Riprap apron | Impact | Consists of a flat rock-lined apron. Used for low Froude numbers. |
| Riprap basin | Hydraulic jump | Similar to a riprap apron, but lowered so that discharging flow passes critical depth; rocks cause turbulence and help induce jump. |

Fr = Froude number.

than 1.0 but less than 1.7, the upstream flow is slightly below critical depth and an undulating water surface occurs at the transition to subcritical flow. As the Froude number increases to the range of 1.7 to 2.5, a rolling transition begins to appear, signaling the conditions generally considered as the weak jump range. The energy loss for these conditions is generally no more than 20 percent.

An oscillating form of jump can occur for Froude numbers between 2.5 and 4.5, with a jet alternating from flow near the bottom to along the surface. Erosion can become a problem if surface waves are allowed to form.

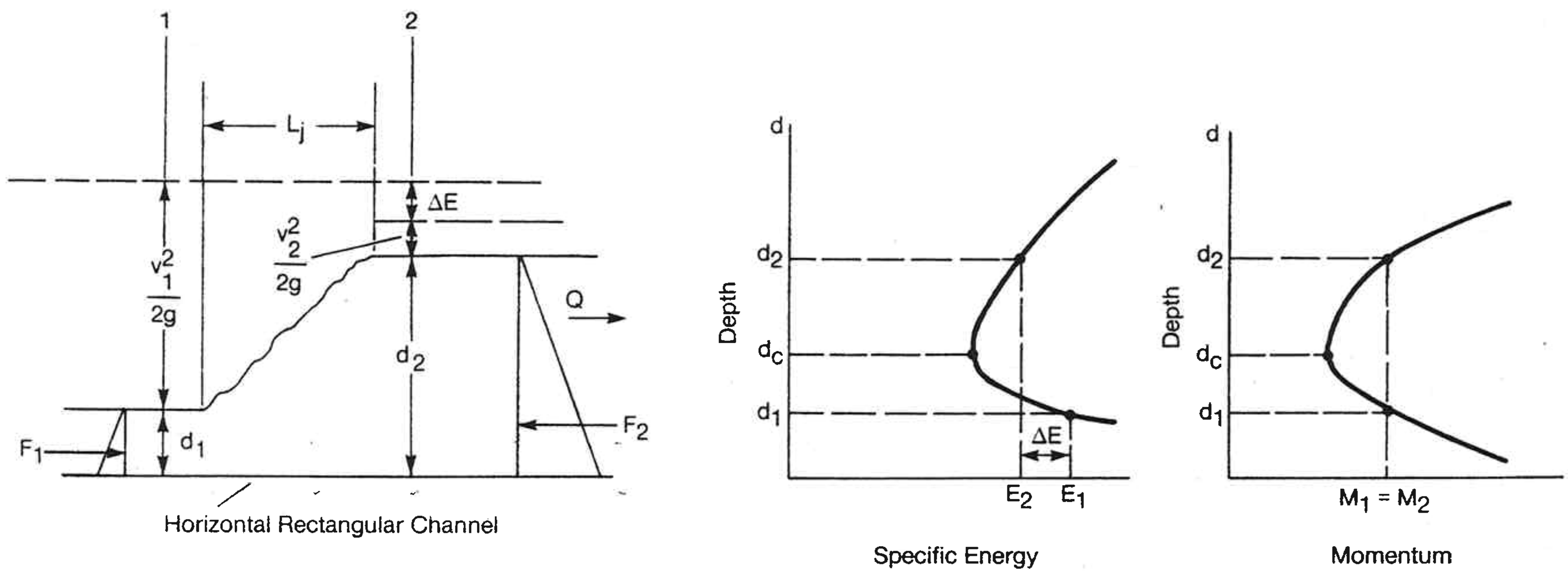
A well balanced and stable jump occurs when the incoming Froude number exceeds 4.5. Turbulence can generally be confined to the jump for this type of flow and the downstream water surface is comparatively smooth up to a Froude number of 9.0. Energy dissipation for this type of steady jump generally ranges from 45 to 70 percent.

When the Froude number exceeds 9.0, energy losses up to 85 percent are possible but downstream erosion may result from the unstable water surface. The high velocity incoming jet intermittently grabs slugs of water that roll down the face of the jump.

### 10.3.2 DEPTH EQUATIONS

The idealized sketch of a hydraulic jump presented in Figure 10-2 defines a control volume and the forces to consider. Control section 1 is supercritical flow upstream of the jump where the water surface is relatively undisturbed. Control section 2 is far enough downstream of the jump for flow to be again considered as longitudinal.

A hydrostatic pressure distribution can be assumed to occur at both control sections. The momentum entering and exiting the control volume is balanced by the resultant of the pressure and boundary friction forces acting on the control volume. Since the jump length is relatively short, the boundary friction forces can be neglected and the conservation of momentum gives the following general relationship:




Q = Discharge
F = Hydrostatic force
$d_1$ = Flow depth upstream of jump
$d_2$ = Flow depth downstream of jump
$d_c$ = Critical depth
E = Specific energy = $d + \frac{v^2}{2g}$
M = Momentum = $\frac{\gamma}{g}Qv$
$\Delta E$ = Head loss through jump = $E_1 - E_2$
v = Velocity
$\gamma$ = Specific weight of water
$L_j$ = Length of jump

*Reference: Morris and Wiggert (1972).*

**FIGURE 10-2**
Hydraulic Jump Variables with Energy and Momentum Diagrams

$$F_1 - F_2 = \frac{\gamma}{g} Qv_2 - \frac{\gamma}{g} Qv_1 \qquad (10\text{-}1)$$

where:

$F_1$ = Upstream hydrostatic force, in pounds

$F_2$ = Downstream hydrostatic force, in pounds

$\gamma$ = Density of water, 62.4 pounds/cubic foot

Q = Design flow rate, in cfs

g = Acceleration due to gravity, 32.2 feet/second²

$v_1$ = Upstream velocity, in feet/second

$v_2$ = Downstream velocity, in feet/second

For the case of a horizontal rectangular channel, the following depth relationships can be derived from the general momentum equation (French, 1985):

$$\frac{d_2}{d_1} = \frac{1}{2}\left[(1 + 8Fr_1^{\,2})^{0.5} - 1\right] \qquad (10\text{-}2)$$

$$\frac{d_1}{d_2} = \frac{1}{2}\left[(1 + 8Fr_2^{\,2})^{0.5} - 1\right] \qquad (10\text{-}3)$$

where:

$d_1$ = Upstream flow depth, in feet

$d_2$ = Downstream flow depth, in feet

$Fr_1 = v_1\ (gd_1)^{0.5}$ = Upstream Froude number, dimensionless

$Fr_2 = v_2\ (gd_2)^{0.5}$ = Downstream Froude number, dimensionless

$v_1$ = Upstream, velocity in feet/second

$v_2$ = Downstream velocity, in feet/second

$g$ = Acceleration due to gravity, 32.2 feet/second$^2$

Because Equations 10-2 and 10-3 contain three independent variables, two must be known before a third can be estimated. It is important to note that the downstream depth is not a result of upstream conditions but of downstream control. In other words, if the downstream control causes a depth equal to $d_2$, then a jump will form.

For non-rectangular prismatic and horizontal channels, depth equations analogous to Equations 10-2 and 10-3 cannot be deriven. The general form of the momentum equation could be solved by trial and error or semi-empirical approximations, and other analytical techniques, as presented by French (1985), are available.

The hydraulic jump dissipator structures listed in Table 10-1 could also be analyzed using the general momentum equation. For these structure, a solution can become difficult because the forces acting on baffle blocks and end sills must be included in the analysis. Designs of these structures, as presented in Volume 2, are usually based on empirical relationships between the structure dimensions and the Froude number derived from physical model studies.

### 10.3.3 ENERGY LOSS

In a horizontal channel, the energy loss across the jump, as shown on the simplified sketch in Figure 10-2, is expressed as:

$$\Delta E = E_1 - E_2 \qquad (10\text{-}4)$$

where:

$\Delta E$ = Energy loss from section 1 to 2, in feet

$E_1 = d_1 + \frac{v_1^2}{2g}$ = Specific energy at section 1, in feet

$E_2 = d_2 + \frac{v_2^2}{2g}$ = Specific energy at section 2, in feet

$d_1$ = Depth at section 1, in feet

$d_2$ = Depth at section 2, in feet

g = Acceleration due to gravity, 32.2 feet/second²

For a rectangular channel, Equation 10-4 can be expressed as (French, 1985):

$$\Delta E = \frac{\left(d_2 - d_1\right)^3}{4d_1 \, d_2} \qquad (10\text{-}5)$$

where:

$\Delta E$ = Energy loss in the jump, in feet

$d_1$ = Upstream water depth, in feet

$d_2$ = Downstream water depth, in feet

The efficiency of a jump in a rectangular channel can be estimated as follows (French, 1985):

$$\frac{E_2}{E_1} = \frac{\left(8Fr_1^{\,2} + 1\right)^{3/2} - 4Fr_1^{\,2} + 1}{8Fr_1^{\,2} \left(2 + Fr_1^{\,2}\right)} \qquad (10\text{-}6)$$

where:

$E_1$ = Specific energy at section 1, in feet

$E_2$ = Specific energy at section 2, in feet

$Fr_1$ = Upstream Froude number, dimensionless

For non-rectangular channel sections, either the general specific energy equation must be solved on a case-by-case basis or a general graphical solution must be obtained.

## 10.3.4 JUMP LENGTH

The length of a hydraulic jump is generally defined to be the distance from the front face of the jump to a point on the surface of the flow immediately downstream of the roller created by the jump (see Figure 10-2). In general, the jump length cannot be derived from theoretical considerations and experimental testing is required.

For horizontal rectangular channels, Silvester (1964) has demonstrated the suitability of the following relationship:

$$\frac{L_j}{d_1} = 9.75\ (Fr_1 - 1)^{1.01} \qquad (10\text{-}7)$$

where:

$L_j$ = Length of jump, in feet

$d_1$ = Upstream depth, in feet

$Fr_1$ = Froude number at section 1, dimensionless

An alternate approach is to use a graphical relationship presented by Bradley and Petraka (1957).

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