DRAINAGE AND FLOOD CONTROL

DESIGN GUIDELINES & CRITERIA

CHANNELS & HYDRAULIC STRUCTURES ON SANDY SOIL

URBAN DRAINAGE AND FLOOD CONTROL DISTRICT-69

UD

SIMONS, LI & ASSOCIATES

City of Aurora Colorado

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DESIGN GUIDELINES AND CRITERIA

FOR

CHANNELS AND HYDRAULIC STRUCTURES

ON SANDY SOILS

Prepared for

Urban Drainage and Flood Control District
Denver, Colorado

and

The City of Aurora, Colorado

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I. INTRODUCTION

1.1 General

A sand-bed channel generally is continually changing its position and shape as a consequence of hydraulic forces acting on its bed and banks. Natural and man-induced changes in rivers frequently set in motion responses that can be propagated for long distances. The response of a river to natural and man-induced changes often occurs in spite of attempts to control the river environment. Unfortunately, the best methods to quantitatively predict the river response are difficult, time consuming, and require specialized knowledge of the subject for proper application. This makes them impractical for use by the design engineer and thus, river response due to man-induced changes is either ingnored or determined by inappropriate methods. This points to the need for a method of analysis that is better suited to the design engineer's needs.

1.2 Authorization

This report was authorized by the Urban Drainage and Flood Control District (DISTRICT) in joint sponsorship with the City of Aurora, Colorado, through an agreement dated September 2, 1980, with Simons, Li & Associates, Inc. The Board of Directors of the DISTRICT adopted a Policy Resolution, Number 41, Series of 1978, which set forth a DISTRICT policy of allocating funds to help local public bodies maintain facilities owned by DISTRICT or cooperatively financed by DISTRICT and the local public body, but owned by the local public body.

Monies were budgeted by the DISTRICT in the 1980 adopted Special Revenue Fund Budget, Resolution Number 55, Series of 1979, for maintenance in 1980 of facilities which were cooperatively financed by DISTRICT and local public bodies and for limited maintenance of non-DISTRICT financed major drainageway facilities.

The development of design standards for major drainageway structures built on fine-grained sandy soils commonly found in Toll Gate Creek, Sand Creek, and Cherry Creek basins was determined by the DISTRICT and City of Aurora, Colorado, to be vital for an orderly maintenance program. Such standards are also intended to assist with final design and design review of drainageway facilities on sandy soils.

1.3 Objective

The objective of this report is to prepare design standards for major drainageway facilities located in sandy soils. Since the prediction of the response to channel development is a very complex task, there are a large number of variables involved in the analysis. A great deal of effort has been devoted to the development of methodologies for predicting the response to channel development and an understanding of stable channels formed in noncohesive materials of all sizes. The application of methods developed for design of stable channel systems requires an understanding of the steady-state transport of sand-sized sediments.

This report presents accurate and efficient methods for evaluating alluvial channels and provides equations and graphs for designing major drainageway facilities. Because of the complex nature of the response to channel development, applying a design standard without understanding the characteristics of sand-bed channels may lead to an improper design of major drainageway facilities. This report also includes general descriptions of characteristics of sand-bed channels, design considerations for major drainageway facilities located in sandy soils, and a design standard which provides design engineers with minimum design procedures to follow and typical details for the design of major drainageway facilities.

1.4 Use of Design Standards

The design standards presented herein are applicable for major drainageway facilities located on noncohesive, sand-sized soils. In analyzing the response to channel development, one should evaluate drainageway morphology and drainageway response qualitatively before the quantitative determination of river response is made. This procedure will ensure that the prediction of river response is reasonable.

II. CHARACTERISTICS OF SAND-BED CHANNELS

2.1 General

An alluvial river (or sand-bed channel) generally is continually changing its position and shape as a consequence of hydraulic forces acting on its bed and banks and related biological forces interacting with these physical forces. These changes may be slow or rapid and may result from natural environmental changes or from changes caused by man's activities. When a river channel is modified locally, the change frequently causes alterations in channel characteristics both up and downstream. The response of a river to man-induced changes often occurs in spite of attempts to control the river environment.

Natural and man-induced changes in rivers frequently set in motion responses that can be propagated for long distances. In spite of the complexity of these responses, all aluvial rivers are governed by the same basic forces, but to varying degrees. It is necessary that river system design be based on adequate knowledge of: (1) geologic factors, including soil conditions; (2) hydrologic factors, including possible changes in flow and runoff, and the hydrologic effects of changes in land use; (3) geometric characteristics of the stream, including the probable geometric alterations that developments will impose on the channel; (4) hydraulic characteristics such as depth, slope, velocity of streams, sediment transport, and the changes that may be expected in these characteristics in space and time; and (5) ecological/ biological changes that will result from physical changes that may in turn induce or modify physical changes.

Documented effects of river development, flood control measures, and channel structures built during the last century have proven the need for considering immediate, delayed, and far-reaching effects of alterations man imposes on natural alluvial river systems. Variables affecting alluvial river channels are numerous and interrelated. Their nature is such that, unlike rigid-boundary hydraulic problems, it is not possible to isolate and study the role of each individual variable. Because of the complexity of the processes occurring in natural flows that influence the erosion and deposition of material, a detached analytical approach to the problem may be difficult and time consuming. Most relationships describing river process have been derived empirically. Most factors affecting alluvial stream channel geometry are: (1) stream discharge; (2) sediment load; (3) longitudinal slope; (4) characteristics of bed and bank material; (5) bank and bed resistance to flow; (6) vegetation; (7) geology, including type of sediment; and (8) works of man.

Considerable study and analysis of the hydraulics of rigid-boundary open channels provide the required knowledge to assure excellent results. The steady-state sediment transport of sand-sized sediments is well understood. Similarly, there is an adequate understanding of stable channels formed in noncohesive materials of all sizes. The theory is adequate to support design of stable channel systems, or if necessary, designs can be formulated to assure the stability of otherwise unstable reaches of river.

2.2 River Morphology and River Response

2.2.1 Stream Form and Classifications

Rivers can be classified broadly in terms of channel patterns; that is, the configuration of the river as viewed on a map or from the air. Patterns include straight, meandering, and braided systems, or some combination of these patterns. These typical river channel patterns are shown in Figure 2-1.

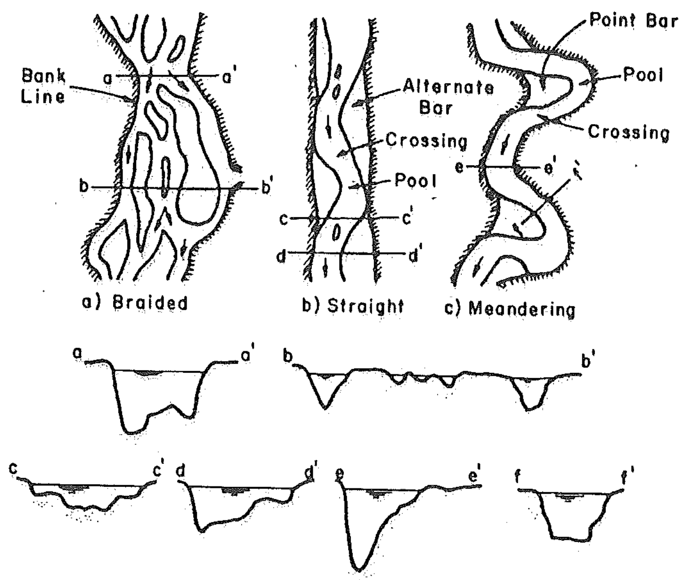


Figure 2-1. River channel patterns.

2.2.1.1 Straight Channels

A straight channel can be defined as one that does not follow a sinuous course. Truly straight channels are rare in nature. Although a stream may have relatively straight banks, the thalweg, or path or greatest depth and velocity along the channel, is usually sinuous. Leopold, Wolman, and Miller (1964) assumed a sinuosity of 1.5 as the division between meandering and straight channels. It is suggested that a better value is about 1.3. The sinuosity of a river is defined as the ratio of thalweg length to valley distance. Sinuosity varies from a value of unity to a value of three or more.

2.2.1.2 The Braided Stream

A braided river is generally wide with poorly defined unstable banks. It is characterized by a relatively steep, shallow water course with multiple channels divided by bars and islands. Braided rivers are generally caused by (1) overloading, that is, the stream may be supplied with more sediment than it can carry resulting in deposition of part of the load, and (2) steep slopes which produce a wide shallow channel where bars and islands form readily. Channel braiding is one of many river patterns that can be in quasi-equilibrium depending upon discharge, sediment load, and transporting ability. The braided stream is difficult to control. It is unstable, changes its alignment rapidly, carries relatively large quantities of sediment, is wide and shallow even at flood stages, and it alters its braiding pattern in a somewhat unpredictable manner.

2.2.1.3 The Meandering Channel

A meandering channel is one that consists of alternating bends giving a continuous S-shape appearance to the plan view of the river. The meandering river consists of a series of deep pools in the bends and shallow crossings in the short straight reaches connecting the bends. At low flows, the local slope is steeper and velocities are larger in the crossings than in the pool. At low stages, the thalweg is located very close to the outside of the bend. At higher stages, the thalweg tends to straighten. More specifically, the thalweg moves away from the outside of the bend encroaching on the point bar to some degree. In the extreme case, the shifting of the current causes chute channels to develop across the point bar at high stages.

2.2.1.4 The Continuum of Channel Patterns

Because of the physical characteristics of straight, braided, and meandering streams, all natural channel patterns intergrade. Although braided and meandering patterns are strikingly different, they actually represent extremes in a continuum of channel patterns.

A number of studies have quantified this concept of a continuum of channel patterns. Lane (1957) investigated the relationship among slope, discharge, and channel pattern in meandering and braided streams and observed that an equation of the form

$$SQ^{1/4} = k \tag{2-1}$$

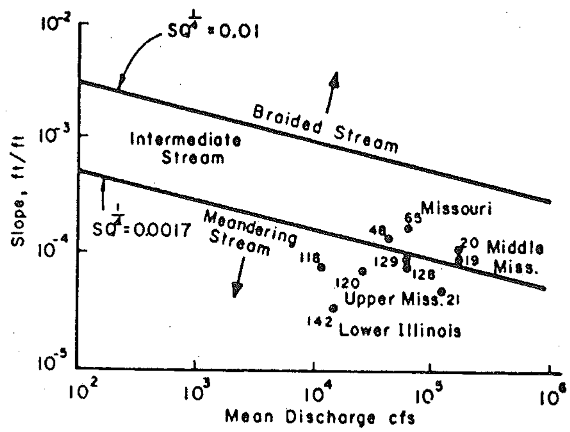
fits a large amount of data from meandering sand streams. Here, S is the channel slope, Q is the mean annual discharge which can be approximated by a two-year peak discharge, and k is a constant. Figure 2-2 summarizes Lane's plots and shows that if

$$SQ^{1/4} \leq 0.0017 \tag{2-2}$$

a sand-bed channel will normally exhibit a meandering pattern. Similarily, when

$$SQ^{1/4} \geq 0.01 \tag{2-3}$$

a river tends toward a braided pattern. The region between these values of



Identification of Reaches Plotted

19	St. Louis to Chester	Middle Mississippi	118	St. Paul to Redwing	Upper Mississippi
20	Chester to Cape Girardeau	Middle Mississippi	120	La Crosse to Lansing	Upper Mississippi
21	Ohio River		128	Hannibal to Louisiana	Upper Mississippi
48	Lower Arkansas River		129	Louisiana to Grafton	Upper Mississippi
65	Missouri River		142	Lower Illinois River	

Figure 2-2. Slope-discharge relation for braiding or meandering in sand-bed streams (after Lane, 1957).

$SQ^{1/4}$ can be considered a transitional range when streams are classified as intermediate or transitional.

2.2.1.5 The Longitudinal Profile

The longitudinal profile of a stream shows its slope or gradient. Since a river channel or river system is generally steepest in its upper regions, most river profiles are concave upward. Shulits (1941) provided an equation for the concave horizontal profile in terms of distance along the stream

$$S_x = S_o \, e^{-\alpha x} \tag{2-4}$$

where, S_x = the slope at any station a distance x downstream of reference station,

S_o = the slope at a reference station,

α = a coefficient of slope change.

Similarily, the bed material normally reduces in size downstream. The change in particle size with distance downstream can be expressed as

$$D_{50x} = D_{50o} \, e^{-\beta x} \tag{2-5}$$

where, D_{50x} = median size of bed material at distance x downstream of a reference station, feet,

D_{50o} = median size of bed material at the reference station, feet,

β = a wear or sorting coefficient that is dependent on river characteristics and flow conditions.

It is clear that these exponential functions can only give a very general idea of the longitudinal bed profile. The general curve is disturbed by the presence of bends. Hence, the longitudinal slope of the thalweg will never completely follow a simple functional relationship. Moreover, the presence of confluences and tributaries influences the main river as far as bed level and grain size are concerned.

2.2.2 Qualitative Response of River Systems

Many rivers achieve a state of approximate equilibrium throughout long reaches. Conversely, many streams contain long reaches that are actively aggrading or degrading. Regardless of the degree of channel stability, man's local activities may produce major changes in river characteristics both locally and throughout an entire reach.

Predicting the response to channel development is a very complex task. There are a large number of variables involved in the analysis that are interrelated and can respond to changes in a river system and in the continual evolution of river form. To predict the response to channel development, an understanding of the direction and magnitude of change in channel characteristics caused by actions of man and nature is required. This understanding can be obtained by: (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; and (4) applying to these, a knowledge of geology, soils, hydrology, and hydraulics of alluvial rivers. Because such a prediction is necessary, useful methods have been developed to predict both qualitative and quantitative response of channel systems to change.

2.2.2.1 Prediction of General River Response to Change

Prediction of response can be made if all of the required data are known with sufficient accuracy. Usually, however, the data are not sufficient for quantitative estimates, only for qualitative estimates. Past studies and investigations conclude that:

1. Depth of flow Y is directly proportional to water discharge Q.
2. Channel width W is directly proportional to both water discharge Q and sediment discharge Q_s.

3. Channel shape, expressed as width to depth ratio W/Y, is directly related to sediment discharge Q_s.
4. Channel slope S is inversely proportional to water discharge Q and directly proportional to both sediment discharge Q_s and grain size D_{50}.
5. Sinuosity s is directly proportional to valley slope and inversely proportional to sediment discharge Q_s.
6. Transport of bed material Q_s is directly related to stream power $\tau_o V$ and concentration of fine material C_f and inversely related to the fall diameter of bed material D_{50}. Here, the fall diameter is defined as the diameter of a sphere that has a specific gravity of 2.65 and has the same terminal uniform settling velocity as the particle when each is allowed to settle alone in quiescent-distilled water of infinite extent at a temperature of 24° C.

A very useful relation for predicting system response was developed by Simons et al. (1975) establishing a proportionality between bed material transport and several related parameters.

$$Q_s \sim \frac{(\tau_o V)\ W\ C_f}{D_{50}} \tag{2-6}$$

where, τ_o = bed shear stress, pounds/foot,

V = cross-sectional average velocity, fps,

W = channel width, feet,

C_f = concentration of fine material load, ppm.

Equation 2-6 can be modified by substituting γYS for τ_o and $Q = AV = WYV$ from continuity, yielding

$$Q_s \sim \frac{\gamma QS}{D_{50}/C_f} \tag{2-7}$$

If the specific weight γ is assumed constant and the concentration of fine material C_f is incorporated in the fall diameter, this relation can be expressed simply as

$$QS \sim Q_s D_{50} \qquad (2\text{-}8)$$

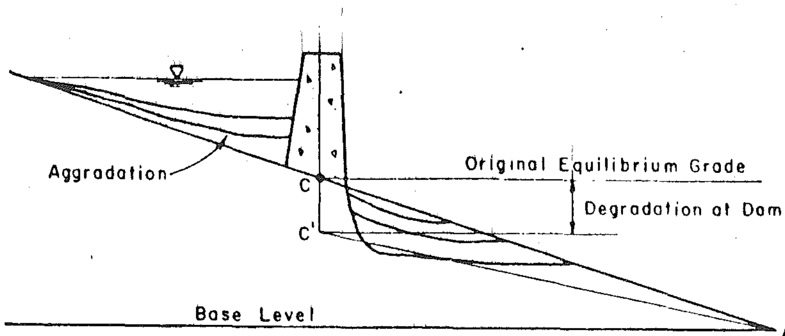
which is the relation proposed by Lane (1955).

2.2.2.2 Qualitative Analysis of River Response

Equations 2-7 and 2-8 are most useful for qualitative prediction of channel response to natural or imposed changes in a river system. Good engineering design must be based on a qualitative understanding of river response to natural processes and man's acitivties. Considerations should be given not only to the local effects, but also upstream and downstream effects resulting from changes in the river system.

Following are several relatively simple situations commonly encountered by engineers and geologists in the river environment. Each case is introduced by a sketch which shows the physical situation prior to a selected natural or man-induced change. Below the sketch, some of the major local effects, upstream effects, and downstream effects resulting from natural processes or development activities are given. It is necessary to emphasize that only the gross local, upstream, and downstream effects are identified.

2.2.2.2.1 The Construction of a Dam. Aggradation in the reservoir upstream of the dam will result in relatively clear water being released downstream of the dam. From Equation 2-8, slope must decrease downstream of the dam to balance the reduction of sediment discharge if fall diameter and water discharge remain constant resulting in degradation (Figure 2-3).



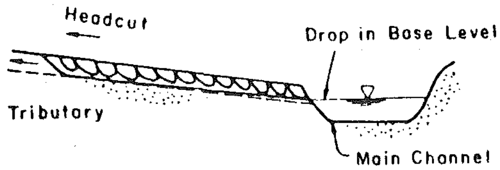
Local Effects	Upstream Effects	Downstream Effects
1. Channel degradation 2. Possible change in river form 3. Local scour 4. Possible bank instability 5. Possible dam failure	1. Aggradation of bed 2. Loss of waterway capacity 3. Change in river geometry 4. Increase flood stage	1. Degradation 2. Reduce flood stage 3. Reduce base level for tributaries, increased velocity and reduced channel stability causing increased sediment transport to main channel

Figure 2-3. Channel adjustment above and below a dam.

2.2.2.2.2 Lowering of Base Level for Tributary Stream. The average water surface elevation in the main channel acts as the base level for the tributary. The lowering of base level will increase the energy gradient in the tributary. This increased energy gradient induces headcutting and causes significant increase in water velocities in the tributary stream. Headcutting is the erosive process by which a drop in gradient moves upstream. Applying Equation 2-8 to the tributary stream, it can be seen that the increase in slope must be balanced by an increase in sediment transport if fall diameter and water discharge remain constant (Figure 2-4).

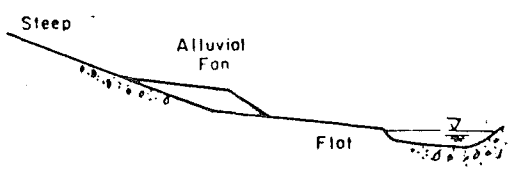
2.2.2.2.3 Raising Base Level for Tributary Stream. The raising of stream base level will decrease the energy gradient in the tributary. This decreased energy gradient, in most cases, causes significant deposition. This can be seen utilizing Equation 2-8 where a decrease in slope is accompanied by a decrease in transport capacity assuming constant conditions of water discharge and size of bed material. For example, an alluvial fan develops which in time can divert the river or reduce the waterway. A similar situation occurs naturally where a steep tributary stream draining an upland region reaches the flatter floodplain of the parent stream (Figure 2-5).

2.2.2.2.4 Straightening of a Reach by Cutoffs. Straightening a channel will significantly increase the channel slope. In general, this causes higher velocities, increases bed material transport, degradation, and possible headcutting of the reach. This can result in unstable river banks and a braided stream form. Straightening of a reach by cutoffs is very common in urban development. In order to design a straightened channel so that it behaves essentially as the natural channel in terms of velocities and magnitude of bed material transport, it is necessary, in general, to build a



Local Effects	Upstream Effects	Downstream Effects
1. Headcutting	1. Increased velocity	1. Increase transport to main channel
2. General scour	2. Increased bed material transport	2. Aggradation
3. Local scour	3. Unstable channel	3. Increased flood stage
4. Bank stability	4. Possible change of form of river	4. Possible change of form of river
5. High velocity		

Figure 2-4. Lowering of base level for tributary stream.



Local Effects	Upstream Effects	Downstream Effects
1. Alluvial fan reduce waterway	1. Erosion of banks	1. Aggradation
2. Channel location is uncertain	2. Unstable channel	2. Flooding
	3. Large transport rate	3. Development of tributary bar in the main channel

Figure 2-5. Rising base level for tributary stream.

wider shallower section, provide a sinuous low-flow section within the major channel, or use grade control structures (Figure 2-6).

More illustrations in regard to the qualitative response of river systems can be found in Sediment Transport Technology by Simons and Senturk, 1977. In engineering design, a qualitative analysis can usually point the way toward satisfying results.

2.3 Physical Properties of Sediments

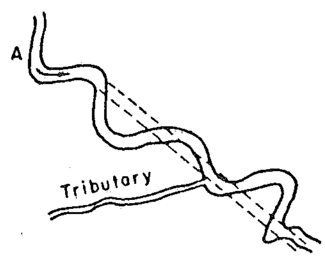
2.3.1 General

"Sedimentology" is a branch of science dealing with the properties of solid particles considered singly or as a mixture. In hydraulic engineering, the size of sediments, the fall velocity of a single particle or of a group of particles, the specific weight of a single particle, and characteristics of deposited sediment are of importance.

Sediments are broadly classified as cohesive and noncohesive. With cohesive sediment, the resistance to erosion depends on the strength of the cohesive bond binding the particles. The noncohesive sediments generally consist of larger discrete particles than the cohesive soils. Noncohesive sediment particles react to fluid forces and their movement is affected by the physical properties of the particles. The treatment herein will be limited to noncohesive sediments.

2.3.2 Sediment Size

Of the various sediment properties, size has the greatest significance to the hydraulic engineer, not only because it is the most readily measured property, but also because other properties such as shape and specific gravity tend to vary with particle size. The size of an individual particle is not



Local Effects	Upstream Effects	Downstream Effects
1. Steeper slope 2. Higher velocity 3. Increased transport 4. Degradation and possible headcutting 5. Banks unstable 6. River may braid 7. Degradation in tributary	1. See local effects	1. Deposition downstream of straightened channel 2. Increased flood stage 3. Loss of channel capacity

Figure 2-6. Straightening a reach by cutoff.

a primary factor in sedimentation study, but the size distribution of the sediment that forms the bed and banks of a stream is of great importance.

A size classification embracing and expanding the Wentworth scale, which has been recommended by the Subcommittee on Sediment Terminology of the Committee on Dynamics of Streams of the American Geophysical Union, contains six consecutive size classes; boulders, cobbles, gravel, sand, silt, and clay (Table 2-1).

2.3.2.1 Boulders

The boulder class has been of little interest when considering sediment transport problems. In this range, size measurement of a single particle is a relatively simple matter. Conversely, it is a difficult problem to establish the size distribution of a bed of boulders due to the large volume of the sample required and the size of the particles. Here, indirect methods are commonly used, such as photographing the material through a grid.

2.3.2.2 Cobbles and Gravel

These two classes play an important role in problems of local scour and resistance to flow and to a lesser extent considering bed-load transport. Size of particles in this range may be determined by direct measurement of either the volume or one or more diameters of typical particles (usually with conversion of results to the normal diameter), or by passing the material through sieves of the proper mesh size.

2.3.2.3 Sand

The sand class is very important considering sediment transport. Sand sizes are almost invariably determined by sieving or by the visual accumulation tube.

Table 2-1. Sediment Grade Scale (Rouse, 1950).

Size in Millimeters		Microns	Inches	Approximate Sieve Mesh Openings per Inch		Class
				Tyler	U.S. Standard	
4000-2000	....	....	160-80	.....	....	Very large boulders
2000-1000	....	....	80-40	.....	....	Large boulders
1000-500	....	....	40-20	.....	....	Medium boulders
500-250	....	....	20-10	.....	....	Small boulders
250-130	....	....	10-5	.....	....	Large cobbles
130-64	....	....	5-2.5	.....	....	Small cobbles
64-32	....	....	2.5-1.3	.....	....	Very coarse gravel
32-16	....	....	1.3-0.6	.....	....	Coarse gravel
16-8	....	....	0.6-0.3	2 1/2	....	Medium gravel
8-4	....	....	0.3-0.16	5	5	Fine gravel
4-2	....	....	0.16-0.08	9	10	Very fine gravel
2-1	2.00-1.00	2000-1000	....	16	18	Very coarse sand
1-1/2	1.00-0.50	1000-500	....	32	35	Coarse sand
1/2-1/4	0.50-0.25	500-250	....	60	60	Medium sand
1/4-1/8	0.25-0.125	250-125	....	115	120	Fine sand
1/8-1/16	0.125-0.062	125-62	....	250	230	Very fine sand
1/16-1/32	0.062-0.031	62-31	....	.....	....	Coarse silt
1/32-1/64	0.031-0.016	31-16	....	.....	....	Medium silt
1/64-1/128	0.016-0.008	16-8	....	.....	....	Fine silt
1/128-1/256	0.008-0.004	8-4	....	.....	....	Very fine silt
1/256-1/512	0.004-0.0020	4-2	....	.....	....	Coarse clay
1/512-1/1024	0.0020-0.0010	2-1	....	.....	....	Medium clay
1/1024-1/2048	0.0010-0.0005	1-0.5	....	.....	....	Fine clay
1/2048-1/4096	0.0005-0.00024	0.5-0.24	....	.....	....	Very fine clay

2.3.2.4 Silt and Clay

The silt and clay classes are of considerable importance in the evaluation of total sediment load when treating such problems as density currents, consolidation, channel stability, and the development of berms. Since silt sizes lie below sieve range, size determinations must be made by a bottom withdrawl tube, microscopically, or indirectly by a technique involving the relative motion between the sediment particles and a suspending fluid.

2.3.3 Fall Velocity

The primary variable defining the interaction of sediment transport with the bed, banks, or suspended in the fluid, is the fall velocity of the sediment particles. The value of the fall velocity depends on several factors such as the concentration, the presence of a wall, and the influence of turbulent velocity fluctuations. Frequently, the standard fall velocity is used. This is defined as the fall velocity of a particle in motionless distilled water of infinite extent at 24° C. Fall velocity is an important variable used to estimate resistance to flow and the rate of sediment transport. It has been shown that the bed configuration, resistance to flow, and rate of bed material transport in a sand channel may change significantly when the fall velocity of the bed material changes.

Fall velocity may be evaluated using various formulas and diagrams which can be found in Sediment Transport Technology by Simons and Senturk (1977).

2.3.4 Specific Weight and Specific Gravity

Specific gravity is an important factor extensively used in the analysis of hydraulic and sediment transport problems. The specific gravity is the ratio of the weight of a solid or a fluid to the specific weight of water at 4° C. The specific gravity of a sediment particle ranges from 2.3 for coal to 7.6 for galena. Fortunately, water-borne sediments are mainly quartz and felspathic minerals with a specific gravity of about 2.65. For this reason, the specific gravity of water-borne sediments is often assumed to be 2.65. However, this specific gravity must be assumed with caution. Different kinds of specific weights and their determination are described in Simons and Senturk (1977).

2.3.5 Size Distribution of Sediment

Sediment consists of many particles differing in size, shape, specific gravity, and fall velocity. It is necessary to make statistical analyses of these characteristics, as determined from an adequate number of samples of sufficient size, in order to define fully the representative particle characteristics of any sediment mixture as a whole. The most commonly used method is size frequency distribution. Sieve analysis is the most commonly used method for size frequency distribution. In general, the results are presented as cumulative size frequency curves. The fraction or percentage by weight of a sediment that is smaller or larger than a given size is plotted against particle size. From the size frequency curve, it is possible to obtain representative sizes of sediment for use in describing the hydraulic properties of a sand mixture related to: (1) the resistance to flow due to grain roughness, and (2) the threshold condition which defines the beginning of transport.

The sediment size frequency distribution is essentially a probabilistic approach used to help describe the transported sediment and the sediment mixture forming the bed of a river. Experimental analysis shows that a cumulative size distribution curve is similar to the cumulative distribution function (CDF) of the standardized normal distribution. The characteristics of the CDF are such that between the ordinates 0.8143 and 0.1586, the normal distribution can be approximated by a straight line. In general, the straight line representing the CDF crosses the sieve analysis curve at a diameter slightly different from 84.13 percent or 15.86 percent finer. To compensate for this discrepancy, a gradation coefficient is defined as

$$G = (1/2)\left(\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}} \right) \tag{2-9}$$

where, G = gradation coefficient,

D_x = the size of sediment for which x percent of the sample is finer.

When D_{50} and G are determined, it is possible to locate the representative line on probability paper and determine the representative sizes of sediment between D_{16} and D_{84}, such as D_{35}, D_{65}, and etc. that are utilized in various equations. The method explained above is only applicable to "S" normal distribution curves.

2.4 Forms of Bed Roughness

2.4.1 General

It is known that for flow in channels composed of sandy soils, a strong physical interrelationship exists between the friction factor, the sediment transport rate, and the geometric configuration assumed by the bed surface. The changes in bed form result from the interaction of the flow, fluid, and bed material. Thus, the resistance to flow and sediment transport are functions of the slope and depth of the stream, the viscosity of the fluid, and the size distribution of the bed material. The analysis of flow in alluvial sand-bed streams is extremely complex. However, with an understanding of the different type of bed forms that occur, the resistance to flow and sediment transport associated with each bed form and how the variables of depth, slope, viscosity, and etc. affect bed form, the engineer can analyze, anticipate, eliminate, or alleviate problems that occur when working with alluvial rivers and channels.

2.4.2 Bed Configuration, Flow Phenomena, and Resistance to Flow

The bed configuration is the array of bed forms, or absence thereof, generated on the bed of an alluvial channel by the flow. The bed configurations that may form in an alluvial channel are plane bed without sediment movement, ripples, dunes, plane bed with sediment movement, antidunes, and chutes and pools. These bed configurations are listed in their order of occurrence with increasing values of stream power ($\tau_o V$, as defined in Equation 2-6) or similar parameters for bed materials having D_{50} less than 0.6 mm. Sketches of typical bed configurations are shown in Figure 2-7.

2.4.2.1 Plane Bed Without Sediment Movement

If the bed material of a stream moves at one discharge but not at a smaller discharge, the bed configuration at the smaller discharge will exist as remnant of the bed configuration formed when sediment was moving. For bed configuration without sediment movement, the problem of resistance to flow is one of rigid-boundary hydraulics. Plane bed without sediment movement has been studied to determine the flow conditions for the beginning of motion and the bed profiles that would form after beginning of motion. For a plane bed without bed material transport, the Manning's roughness coefficient varies from 0.012 to 0.016.

2.4.2.2 Ripples

In fine sand, ripples will usually occur as soon as particles show appreciable movement. The separation zone downstream from a ripple causes very little disturbance on the water surface. The concentration of sediment is small ranging from 10 to 200 ppm. Resistance to flow is large with the Manning's n varying from 0.018 to 0.05. As the depth of the flow increases, resistance to flow due to bed roughness decreases.

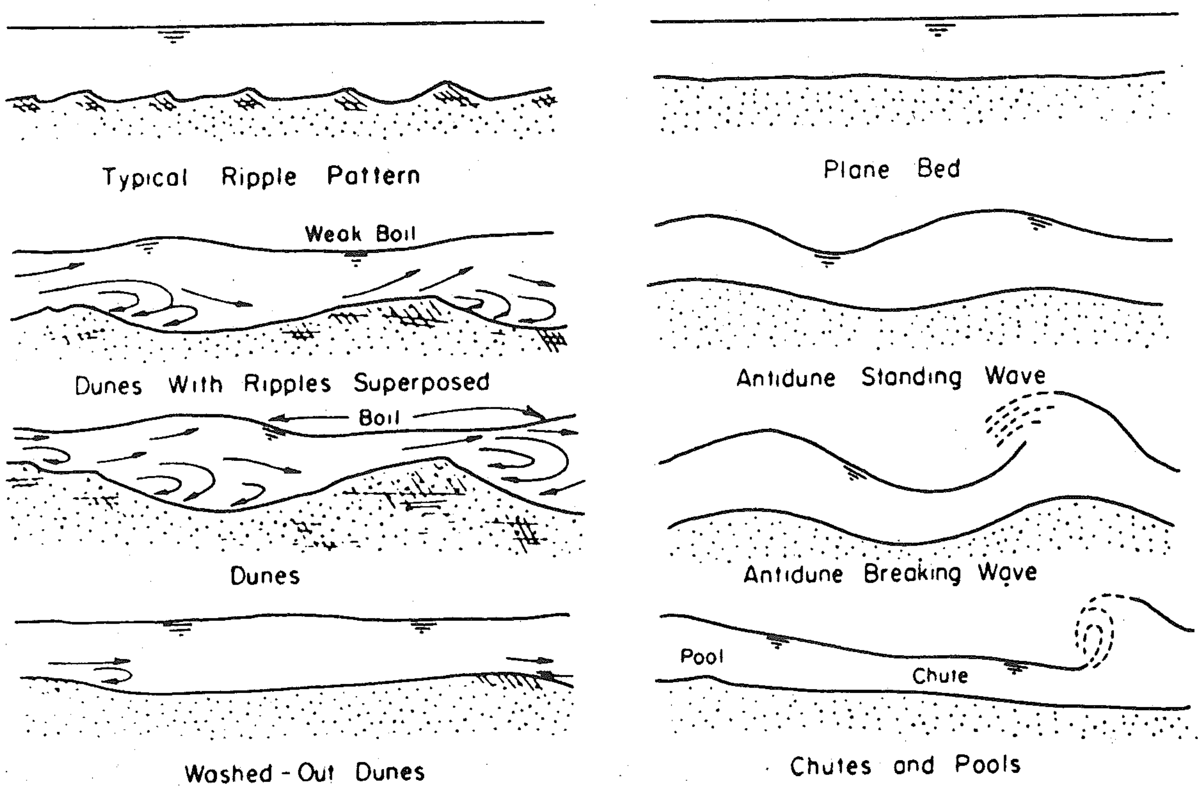


Figure 2-7. Forms of bed roughness in sand channels.

2.4.2.3 Dunes

When the shear stress and the stream power are gradually increased over a bed of ripples, or if the bed material is coarser than 0.6 mm over a plane bed, a new flow condition will be achieved that causes dunes to form. Dunes cause large separation zones in the flow. With dunes, as with tranquil flow over an obstruction, the water surface is always out of phase with the bed surface. The flow accelerates over the crest of the dunes and decelerates over the troughs. The sediment transport rate is relatively small. The concentration of bed materials is approximately 100 to 200 ppm. Resistance to flow caused by dunes is large, but not as large as that caused by ripples formed of finer sand and at shallow depth. For dunes, the Manning's n varies from 0.018 to 0.035. Resistance to flow increases with an increase in depth for coarser sands ($D_{50} \geq 0.3$ mm) and decreases with an increase in depth for finer sands ($D_{50} < 0.3$ mm). This is because the amplitude of dunes can increase with the increasing depth so that the relative roughness can remain essentially constant or even increase with increasing depth of flow. However, a decrease in angularity of the dunes in finer sands as they increase in size reduces their form roughness and their resistance to flow as well.

2.4.2.4 Plane Bed With Sediment Movement

If the shear stress or the stream power is continously increased, the size of dunes increases until the dunes reach a maximum height at a certain stream power. Thereafter, a transition regime is reached when the dunes start to diminish in amplitude with the further increase of the stream power. Finally, the dunes completely disappear and a flatbed is formed. In this case, the concentration of bed material ranges from about 1500 to 3000 ppm.

Resistance to flow is relatively low with the Manning's n varying from 0.014 to 0.022.

2.4.2.5 Antidunes

When the shear stress of the stream power is further increased, sand and water waves gradually build up from a plane bed and from a plane water surface. The waves may grow in height until they become unstable and break like the sea surf, or they may gradually subside and subsequently reform. The former have been called breaking antidunes, or antidunes, and the latter standing waves.

With antidune flow, the water and bed surface waves are in phase. This is a positive indication that the local flow is rapid (Froude number > 1.0). Resistance to flow with antidunes depends on how often the antidunes form, the area of the reach they occupy, and the violence and frequency of their breaking. If the antidunes do not break, resistance to flow is about the same as for a plane bed with sediment movement. The Manning's n value varies from 0.012 to 0.028.

2.4.2.6 Chutes and Pools

At very steep slopes, sand-bed channel flow changes to chutes and pools. This type of flow consists of a long chute in which the flow accelerates rapidly, a hydraulic jump at the end of the chute, and then a long pool in which the flow is tranquil, but accelerating. Resistance to flow is large. Manning's n varies from 0.015 to 0.031.

2.4.2.7 Recommended Values of Manning's n

Floodplain study and sediment transport analysis are two completely different analyses. For conservative estimation, it is recommended, within the range of Manning's roughness coefficients specified for each bed form, that higher roughness coefficients should be used for flood studies and lower roughness coefficients should be used for sediment transport analysis. Because of the relatively wide variation in Manning's coefficient considering all possible flow conditions and the full range of sand sizes, recommended values for design are given in Table 2-2.

2.4.3 Regime of Flow in Alluvial Channels

The flow in sand-bed channels is divided into two flow regimes with a transition zone between. Each of these two flow regimes are characterized by similarities in the shape of the bed configurations, mode of sediment transport, process of energy dissipation, and phase relation between the bed and water surfaces. The two regimes and their associated bed configurations are:

A. Lower flow regimes

   1. Ripples
   2. Dunes

B. Transition zone: bed configurations range from dunes to plane beds or to antidunes.

C. Upper flow regimes

   1. Plane bed with sediment movement
   2. Antidunes

      a. standing waves
      b. breaking antidunes

   3. Chutes and pools

In Table 2-3, variations of different variables with flow regimes and bed forms are given for two sand sizes. A relationship (Figure 2-8) was developed by Simons and Richardson (1966) that relates stream power ($\tau_0 V$), median fall diameter of bed material, and form roughness. If the depth,

Table 2-2. Values of Manning's Coefficient n for Design of Channels with Fine to Medium Sand Beds.

	Manning's Coefficient n	
Bed Roughness	For Depth and Flood Control	For Sediment Transport and Bank Stability
Ripples	0.025	0.022
Dunes	0.035	0.030
Transition	0.030	0.025
Plane Bed	0.022	0.020
Standing Waves	0.025	0.020
Antidunes	0.030	0.025

Table 2-3. Variation of Different Variables with Regimes of Flow and Forms of Bed Roughness, (Simons and Richardson Flume Experiments, 1971).

		0.28 mm Sand					0.45 mm Sand				
	Forms of Bed Roughness	Total Load Concentration (ppm)	f	n ($ft^{1/6}$)	F_r	$Sx10^2$	Total Load Concentration (ppm)	f	n ($ft^{1/6}$)	F_r	$Sx10^2$
Lower Flow Regime	Plane	0	0.0301	0.016	0.17	0.011	0	0.0359	0.016	0.18	0.015
	Ripples	1 to 150	0.0635 to 0.1025	0.02 to 0.027	0.17 to 0.37	0.023 to 0.11	1 to 100	0.0521 to 0.1330	0.020 to 0.028	0.14 to 0.28	0.016 to 0.11
	Dunes	150 to 800	0.0612 to 0.0791	0.021 to 0.026	0.32 to 0.44	0.09 to 0.16	100 to 1,200	0.0489 to 0.1490	0.019 to 0.033	0.28 to 0.65	0.06 to 0.30
Transition		1,000 to 2,400	0.0250 to 0.0344	0.014 to 0.017	0.55 to 0.67	0.13 to 0.17	1,400 to 4,000	0.0415 to 0.0798	0.016 to 0.022	0.61 to 0.92	0.37 to 0.49
	Plane	1,500 to 3,100	0.0244 to 0.0262	0.013 to 0.014	0.71 to 0.92	0.15 to 0.28					
Upper Flow Regime	Standing Waves						4,000 to 7,000	0.0200 to 0.0406	0.011 to 0.015	1.0 to 1.6	0.36 to 0.62
	Antidunes	5,000 to 42,000	0.0281 to 0.0672	0.014 to 0.022	1.0 to 1.3	0.33 to 1.0	6,000 to 15,000	0.0247 to 0.0292	0.012 to 0.014	1.4 to 1.7	0.66 to 1.0

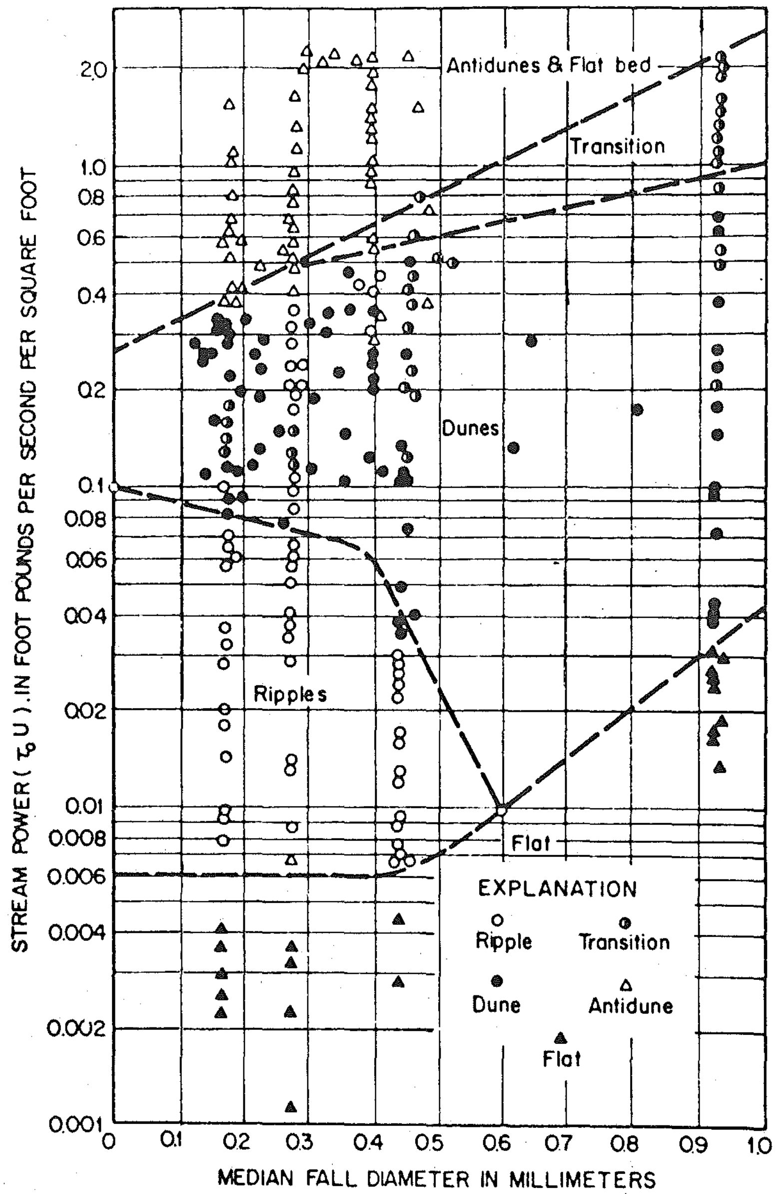


Figure 2-8. Relation of bed forms to stream power and median fall diameter of bed sediment (after Simons and Richardson, 1966).

slope, velocity, and fall diameter of bed materials are known, one can predict the form of bed roughness by using this relationship.

2.4.3.1 Lower Flow Regimes

Lower flow regime begins with the beginning of motion. The resistance to flow is large and sediment transport small. The bed form is either ripples or dunes or some combination of the two. The water surface undulations, if they exist, are out of phase with the bed surface and there is a relatively large separation zone downstream from the crest of each ripple or dune. Resistance to flow is caused mainly by form roughness.

2.4.3.2 Upper Flow Regimes

In the upper flow regime, resistance to flow is relatively small and sediment transport is large. The usual bed forms are plane bed or antidunes. The water surface is in phase with the bed surface except when an antidune breaks and normally the fluid does not separate from the boundary. A small separation zone may exist downstream from the crest of an antidune prior to its breaking. Resistance to flow is the result of grain roughness with the grains moving, sand waves and their subsidence, and energy dissipation when the antidune breaks.

2.4.3.3 Transitions

The transition zone encompasses the bed forms that occur during the passage from lower regime to upper regime. The bed configuration in the transition zone is erratic. It may range from that typical of the lower flow regime to that typical of the upper flow regime, depending mainly on antecedent conditions. The bed configuration may also oscillate between dunes and plane bed due to changes in resistance to flow and consequently, the changes in depth and slope as the bed form changes.

2.5 Sediment Transport

2.5.1 General

The amount of material transported or deposited in a channel reach is the result of the interaction of two processes. The first is the transport capacity of the reach. This is determined in part by the hydraulic conditions which are a direct result of the water discharge, channel configuration, and channel resistance. The other major factor is the sediment size present. The second process is the supply of sediment entering a channel reach. This is determined by the nature of the channel and watershed above the reach and development that it may be subject to.

2.5.2 Type of Sediment Movement

Sediment particles are transported by flow in one or a combination of the following ways: (1) rolling or sliding on the bed, surface creep; (2) jumping into the flow and then resting on the bed, saltation; and (3) supported by the surrounding fluid during its entire motion, suspension.

There is no sharp line between saltation and suspension. However, this distinction is important as it serves to delimit two methods of hydraulic transportation which follow different laws (i.e., traction and suspension). Of course, sediment may be transported partially as saltation and then suddenly be caught by the flow turbulence and transported in suspension. Sediments which move as surface creep or saltation and are supported by the bed are called bed load. Sediments which are suspended and supported by flow are called suspended load.

2.5.3 Sediment Transport Capacity

Many equations have been developed to predict the sediment transport rate under certain hydraulic conditions with certain sediment sizes. Among them, the Meyer-Peter, Muller (MPM) equation is a simple and commonly used bed-load transport equation for fine and medium sand-bed channels; the Einstein method is one of the most widely recognized methods used to compute suspended sediment loads. With the integration of these two methods, the estimation of total sand transport capacity can be made.

2.5.3.1 The Meyer-Peter, Muller Equation

The equation is

$$q_b = \frac{12.85}{\sqrt{\rho}\,\gamma_s} (\tau_o - \tau_c)^{1.5} \tag{2-10}$$

in which,

$$\tau_o = (1/8)\ \rho f_o\ V^2 \tag{2-11}$$

$$\tau_c = F_* \ (\gamma_s - \gamma)\ D_s \tag{2-12}$$

where, q_b = the bed-load transport rate in volume per unit width for a specific size of sediment,

τ_o = the boundary shear stress acting on the grain,

τ_c = the critical tractive force necessary to initiate particle motion,

ρ = the density of water,

γ_s = the specific weight of sediment,

F_* = the dimensionless shear stress depending on flow conditions,

D_s = the size of sediment,

f_o = the Darcy-Weisbach friction factor (usually assumed 0.066),

V = the mean flow velocity.

The equation is based on the theory of beginning of motion and the tractive force exerted by the flow on the bed of a channel. The value of F_* can be found from Shields' diagram presented in Figure 2-9. In the diagram, R_*

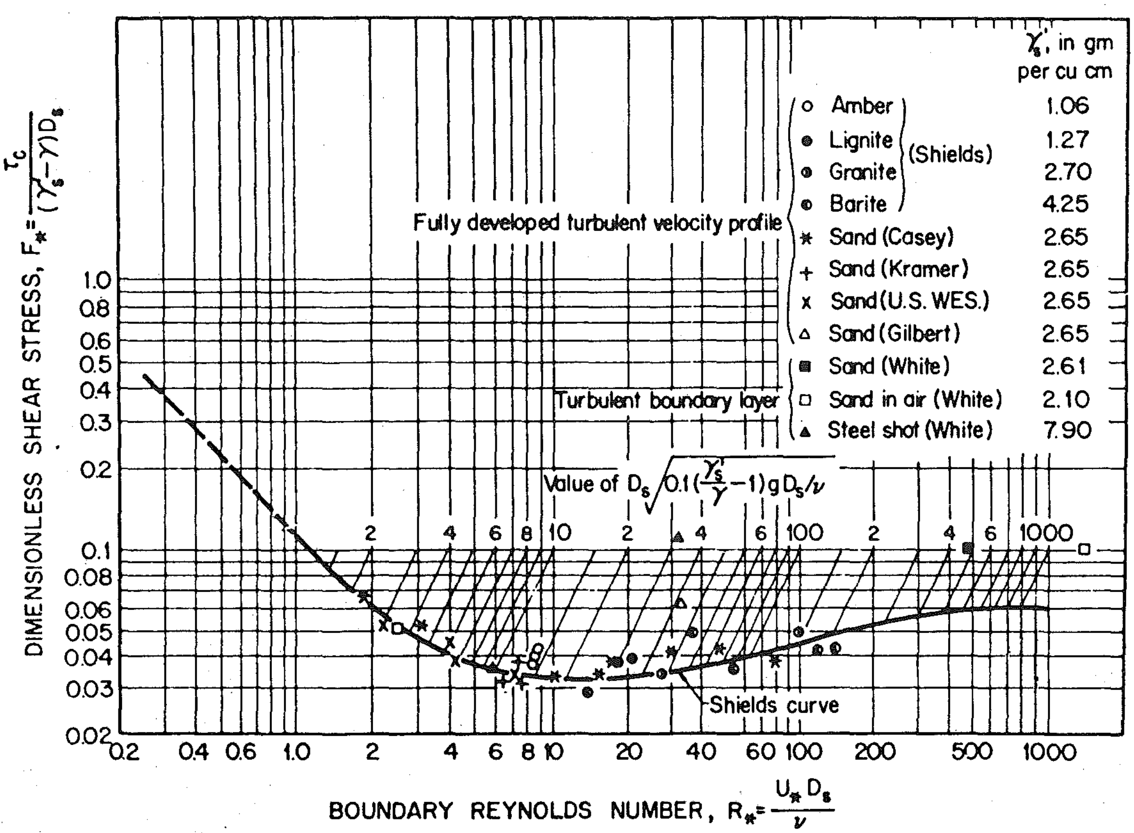


Figure 2-9. Shields' diagram: dimensionless critical shear stress.

is the boundary Reynolds number, D_s is the size of sediment, U_* is the shear velocity ($U_* = \sqrt{\tau_o/\rho}$), and ν is the kinemetic viscosity of the fluid.

2.5.3.2 The Einstein Method for Suspended Sediment Load Estimation

This method relies upon an integration of the sediment concentration profile as a function of depth. The nature of the profile is determined using turbulent transport theory. The sediment profile is assured to be in equilibrium and therefore, the rate at which sediment is transported upward due to turbulence and the concentration gradient are exactly equal to the rate at which gravity is transporting sediment downward. If the sediment concentration is known at one point, then the entire concentration is determined. The point of known concentration is assumed to be the upper limit of the bed-load layer. The resulting equation is

$$q_s = \frac{q_b}{11.6} \frac{K^{w-1}}{(1 - K)^w} [(\frac{V}{U_*} + 2.5) I_1 + 2.5 I_2] \quad (2\text{-}13)$$

in which, q_s = the suspended load,

q_b = the bed load determined by using Equation 2-10 or other appropriate bed-load equations,

K = the relative depth of the bed layer,

U_* = the shear velocity,

V = the mean velocity of flow,

I_1 and I_2 = Einstein integrals, and

w = a dimensionless parameter given by

$$w = \frac{V_s}{\kappa U_*} \quad (2\text{-}14)$$

In Equation 2-14, V_s = the fall velocity of the sediment particle, and

κ = the Karman constant (usually 0.4 is used).

I_1 and I_2 are integrals which cannot be evaluated directly. One must either use tables or numerical techniques.

III. DESIGN CONSIDERATIONS

3.1 General

A predominant characteristic of alluvial channels is the change in location, shape, and hydraulics that the channel and cross sections experience with time. These changes are particularly significant during periods when alluvial channels are subjected to comparatively high flows. In most instances, when considering the stability of alluvial channels, it can be shown that approximately 90 percent of all river changes occur during that five to ten percent of the time when large flows occur, if duration is significant. Regardless of the fact that the majority of changes occur during comparatively short time periods, there may also be regions within a river in which some degree of instability is exhibited for all flow conditions. Also, any modification to a channel can significantly alter the system.

Urbanization will always alter the natural river system, such as increasing peak flow rates, decreasing sediment supply, encroachment into the floodplain, and etc. Design of stable channels and hydraulic structures must be considered and evaluated at the planning and design stages to protect from flood damages including human lives and property.

The large number of variables which affect the river channel are interdependent. Major hydraulic variables affecting channel design and sediment transport are velocity, depth, and slope. Major factors which cause instability of channels and hydraulic structures are degradation and aggradation, general scour, local scour, lateral migration, subsurface flow, uplift force, seepage, and freeboard.

3.2 Velocity

Velocity is often accepted as the most important factor when designing stable alluvial channels. A general rule is that sediment transport increases with flow velocity to the fourth power at low discharges and to larger powers at high-flow discharges. Design velocity, during major flows, for channel improvements shall follow the recommendations as presented in Section 2.3.1A of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2.

3.3 Depth

In a natural river system, depth is an important indicator of: (1) the size of the channel; (2) the stability of the channel; and (3) the shear stress exerted on the channel boundary by the flow. Also, scouring power of water increases in proportion to a third to fifth power of depth. Design depth, during major flows, for various types of channel improvements are recommended in "Major Drainage" of the Urban Storm Drainage Criteria Manual, Volume 2.

3.4 Slope

The slope of the energy gradient plays an extremely important role in the hydraulics of river channels. Slope is utilized in velocity equations such as the Manning's equation to estimate velocity; it is also utilized in the tractive force equation to estimate the tractive force exerted on the bed and banks of open channels. A long reach of river channel may be subjected to a general lowering or raising of the bed level over a long period of time due to changing incoming sediment supply caused by activities such as urbanization, construction of a reservoir, and etc. An equilibrium channel slope is defined as the slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply. Under this condition, the channel neither aggrades nor degrades. The method and procedure for determining equilibrium channel slopes for natural channels is shown in the design example in Section 5.2.2.2. Normally, a slope of from 0.2 to 0.6 percent for grass lined channels as recommended in Section 2.3.1C of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2, will be satisfactory. Slope could be steeper if the channel is lined with concrete or riprap.

3.5 Degradation and Aggradation

A long reach of river channel may be subjected to a general degradation or aggradation of the bed level over a long period of time. Degradation and aggradation must be accurately anticipated; otherwise, foundation depths may be inadequate or excessive, depending on the magnitude of degradation or aggradation.

The basic principle of degradation and aggradation is to compare, in a reach, the sediment supply and the sediment transport. When sediment supply is less than sediment transport, the flow will remove additional sediment from the channel bed and banks to eliminate the deficit. This results in degradation of the channel bed and possible failure of the banks. If the supply entering the reach is greater than the capacity, the excess supply will be deposited.

Degradation or aggradation can be evaluated qualitatively as described in Section 2.2.2 of this report. Unfortunately, the best methods to quantitatively predict sediment transporting characteristics of a waterway are difficult, time consuming, and require specialized knowledge of the subject for proper application. This makes them impractical for use by the design engineer and thus, the sediment transport is either ignored or determined by inappropriate methods. This points to the need for a method of analysis that is better suited to the design engineer's needs. Application of such a methodology must be both accurate and efficient.

The design standard presents a sediment transport analysis method which meets the above criteria. The determination of sediment transport as presented in this design standard is based on easy to apply power relationships between sediment transport rate and velocity and depth.

The sediment transport rate in a river can generally be presented by a simplified equation relating to flow depth and velocity

$$q_s = C_1 Y^{C_2} V^{C_3} \tag{3-1}$$

where, q_s = sediment transport rate in cfs/foot,

Y = flow depth in feet,

V = flow velocity in feet per second,

C_1, C_2, C_3 = constants.

Values of C_1, C_2, and C_3 for sand materials are presented in Table 3-1 with limitations of hydraulic parameters noted. Values of C_1, C_2, and C_3 are also presented in Figure 3-1. The sediment transport rate should be determined using the Meyer-Peter, Muller equation and the Einstein method as presented in Section 2.5.3; however, the above simplified equation with the recommended values for C_1, C_2, and C_3 will provide the design engineer with a reasonable first-order estimate of sediment transport as long as it is used within the specified limits of particle size and flow velocity. The sediment transport rate should be determined for a variety of flow conditions and sediment sizes likely to occur in the study reach.

The equilibrium channel slope is defined as the slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply. Equilibrium slope should be determined for frequent flood flows,

Table 3-1. Constants for Sediment Transport Equation.

Class	Size (mm)	Geometric Mean (mm)	C_1	C_2	C_3
Very fine sand	0.062 - 0.125	0.088	58.50×10^{-6}	1.040	3.20
Fine sand	0.125 - 0.250	0.177	21.40×10^{-6}	0.837	3.59
Medium sand	0.250 - 0.500	0.354	6.47×10^{-6}	0.535	4.05
Coarse sand	0.500 - 1.000	0.707	2.90×10^{-6}	0.239	4.36
Very coarse sand	1.000 - 2.000	1.410	2.37×10^{-6}	- 0.044	4.44

Limitations: (1) Sediment sizes from 0.062 to 2.00 mm

(2) Channel bed slope from 0.002 to 0.010

(3) Flow velocities less than 25 fps.

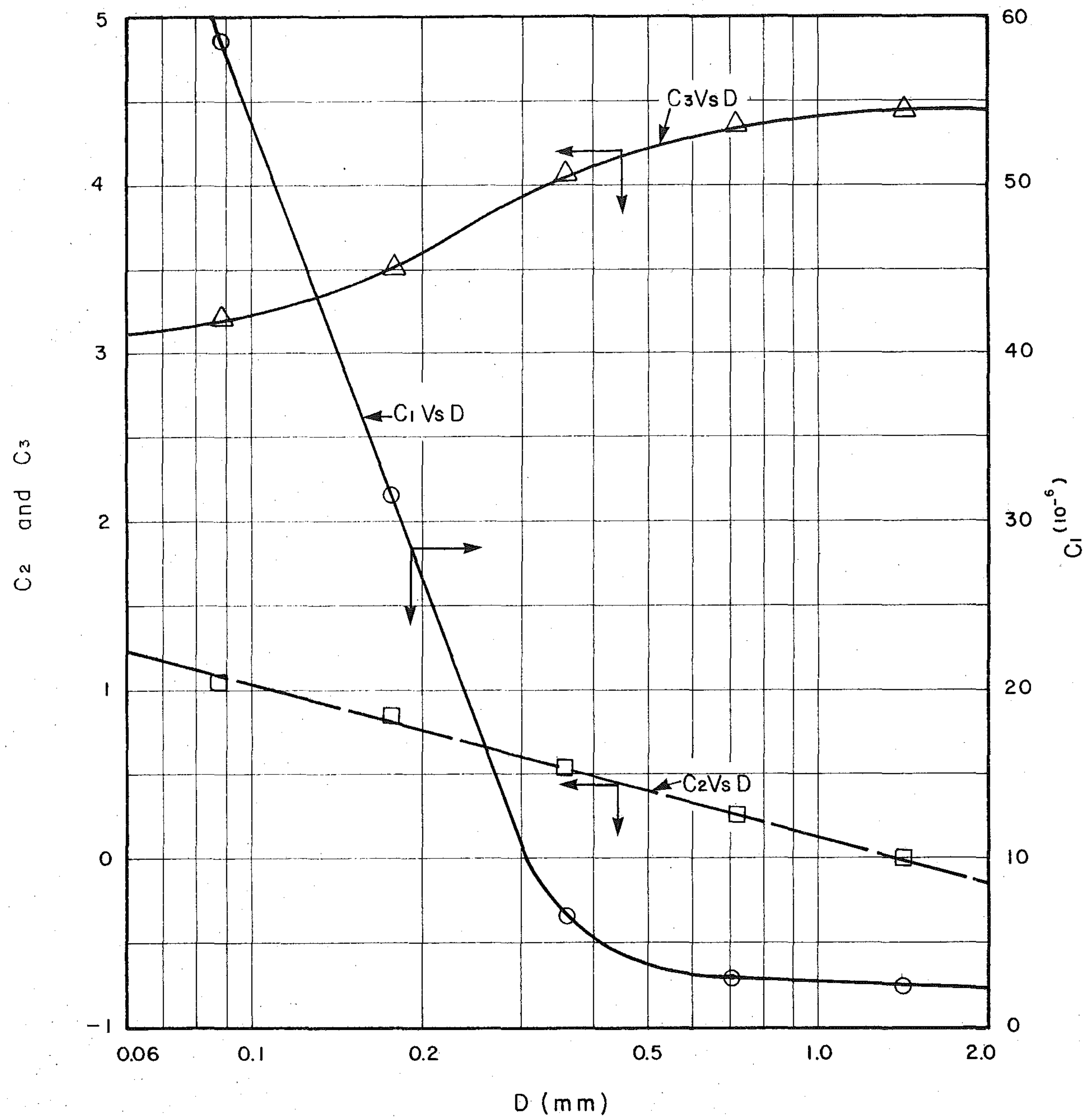


Figure 3-1. Parameters for sediment transport equation (Equation 3-1).

such as an annual flood, because it is the frequent flood discharges that dominant the long-term degradation or aggradation process. A design example for determining equilibrium slope is given in Section 5.2.2.2. In addition to the prediction of a long-term equilibrium channel slope, degradation or aggradation from a rare flood event, such as a 100-year flood, should also be evaluated because the maximum short-term degradation or aggradation usually will occur during a rare storm. The maximum degradation or aggradation may not be evident after the storm event. Prediction of ultimate degradation or aggradation of a storm can generally be made using mathematical models.

For projects where a detailed quantitative analysis can be justified, a series of programs have been developed by Simons, Li & Associates, Inc. to determine the river response to a flood event. The sediment routing model utilizes the hydraulic conditions determined by the U.S. Army Corps of Engineers' HEC-2 program, but recognizes that the channel bed and banks are movable and adjustable, responding to scour and deposition. Application of the model requires knowledge of the characteristics so the model can be calibrated and/or adjusted for each individual application.

3.6 General Scour

General scour usually occurs when flow area is contracted by embankments, channelization, and accumulation of debris. Scour at contractions occurs because the flow area becomes smaller than the normal stream and average velocity and bed shear stress increase. Hence, there is an increase in stream power at the contraction and more bed material is transported through the contracted section than is transported into the section. As bed level is lowered, velocity decreases, shear stress decreases, and equilibrium is restored when the transport rate of sediment through the contracted section is equal to the incoming rate.

3.6.1 Flow Confined to the Channel

Consider a situation where a normal river channel is narrowed by a contraction. Scour due to the contraction may be determined in the following manner (Nordin, 1971). The approach flow depth Y_1 and average approach flow velocity V_1 result in the sediment transport rate q_{s1} (Equation 3-1). Total transport rate to the contraction is $W_1 q_{s1}$ in which W_1 is width of the approach channel. If the water flow rate $Q = W_1 q_{s1}$ in the upstream channel is equal to the flow rate at the contracted section, then by continuity,

$$q_2 = \frac{W_1}{W_2} q_1 \qquad (3\text{-}2)$$

Here, $q_1 = Y_1 V_1$ and $q_2 = Y_2 V_2$, and the subscript 2 refers to conditions in the contracted section. The sediment transport rate at the contracted section after equilibrium is established (i.e., the sediment transport rate at the contracted section is equal to the sediment supply from the approach channel) must be

$$q_{s2} = \frac{W_1}{W_2} q_{s1} \qquad (3\text{-}3)$$

Knowing q_2 and q_{s2}, Y_2 and V_2 can be determined using $q_2 = Y_2 V_2$ and the sediment transport equation (Equation 3-1). Depth of scour due to the contraction can be determined as

$$Y_s = Y_2 - Y_1 \qquad (3\text{-}4)$$

3.6.2 Overbank Flow with Flow in the Channel

Laursen (1960) developed an equation for scour at a contraction where, in addition to channel flow, there is overbank flow concentrating the contracted channel (designated by subscript 2). The equation to predict depth of flow at section 2 is

$$\frac{Y_2}{Y_1} = \left(\frac{Q_t}{Q_c} \right)^{6/7} \left(\frac{W_1}{W_2} \right)^{c_1} \left(\frac{n_2}{n_1} \right)^{c_2} \tag{3-5}$$

where, $c_1 = \dfrac{6(2 + f)}{7(3 + f)}$,

$c_2 = \dfrac{6f}{7(3 + f)}$,

Q_c = the approach channel flow rate,

Q_t = the contracted channel flow rate that is greater than the approach channel flow rate by the amount of flow on the floodplain,

Y = the flow depth,

W = the channel width,

n = the Manning's roughness coefficient,

exponent f is given below

U_*/V_s	f
< 0.5	0.25
1.0	1.00
> 2.0	2.25

Here, U_* = the shear velocity ($\sqrt{\tau/\rho}$) in the approach channel,

V_s = the fall velocity of the bed material.

3.6.3 Overbank Flow Only

For scour at bridges on a floodplain where there is no sediment transport upstream, Laursen (1963) proposed that

$$\frac{Y_2}{Y_1} = \left(\frac{W_1}{W_2} \right)^{6/7} \left(\frac{V_1^{\,2}}{120\ Y_1^{\,1/3}\ D_{50}^{\,2/3}} \right)^{3/7} \tag{3-6}$$

where, Y_1 = depth of the approach channel,

V_1 = velocity of the approach channel,

W_1 = width of the approach channel,

Y_2 = general scour of flow depth at the bridge,

D_{50}= the median diameter of the bed material at the bridge.

3.7 Local Scour

Local scour occurs in the bed at embankments due to the actions of vortex systems induced by obstruction of the flow. Local scour occurs in conjunction with, or in the absence of, degradation, aggradation, and general scour. 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 embankment. The action of the vortex is to erode 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 scour hole develops. As the depth is increased, strength of the vortex is reduced, transport rate is reduced, equilibrium is reestablished, and scouring ceases.

The depth of scour varies with time because sediment transported into the scour hole from upstream varies depending upon the presence or absence of dunes. A mean scour depth between the oscillation or scour depth is referred to as equilibrium scour depth.

3.7.1 Local Scour Around Embankments

Detailed studies of scour around embankments have been performed mostly in laboratories. According to the studies of Liu et al. (1961), the equilibrium scour depth for local scour in sand at a spill slope when the flow is subcritical is determined by the expression

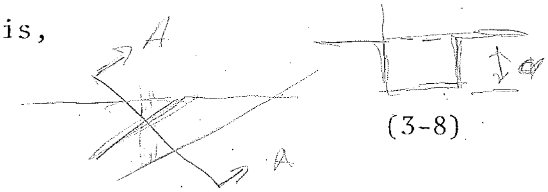
$$\frac{Y_s}{Y_1} = 1.1 \left(\frac{a}{Y_1} \right)^{0.40} Fr_1^{\;0.33} \qquad (3\text{-}7)$$

where, Y_s = the equilibrium scour depth measured from the mean bed level to the bottom of the scour hole, where, Y_1 = the upstream flow depth,

a = the embankment length measured normal to the bank,

Fr_1 = the upstream Froude number.

If the embankment terminates at a vertical wall on the upstream side, then the scour depth in sand nearly doubles. That is,



$$\frac{Y_s}{Y_1} = 2.15 \left(\frac{a}{Y_1} \right)^{0.40} Fr_1^{0.33} \tag{3-8}$$

Field data for scour at embankments for various size rivers is scarce, but data collected at rock dikes on the Mississippi River indicate that

$$\frac{Y_s}{Y_1} = 4 \ Fr_1 \tag{3-9}$$

determines the equilibrium scour depth for large a/Y_1. It is recommended that Equations 3-7 and 3-8 be applied to embankments with $0 < a/Y_1 < 25$ and Equation 3-9 be used for $a/Y_1 > 25$. If $a/Y_1 > 25$, then scour depth is independent of a/Y_1 and depends only on the approach Froude number and depth of flow.

In applying Equations 3-7 and 3-8, the embankment length a is measured from the high waterline at the valley bank perpendicularly to the end of the embankment. A definition of the embankment length for a natural channel with riprap protection is shown in Figure 3-2. It is not uncommon to find depths to be 30 percent greater than equilibrium scour depth. Lateral extent of scour can be determined from the angle of repose of the material and scour depth.

3.7.2 Local Scour Downstream of Hydraulic Structures

The scour downstream of hydraulic structures, such as stilling basins, diversion works, and etc., occurs frequently in engineering application. Figure 3-3 summarizes the possible flow conditions downstream of hydraulic

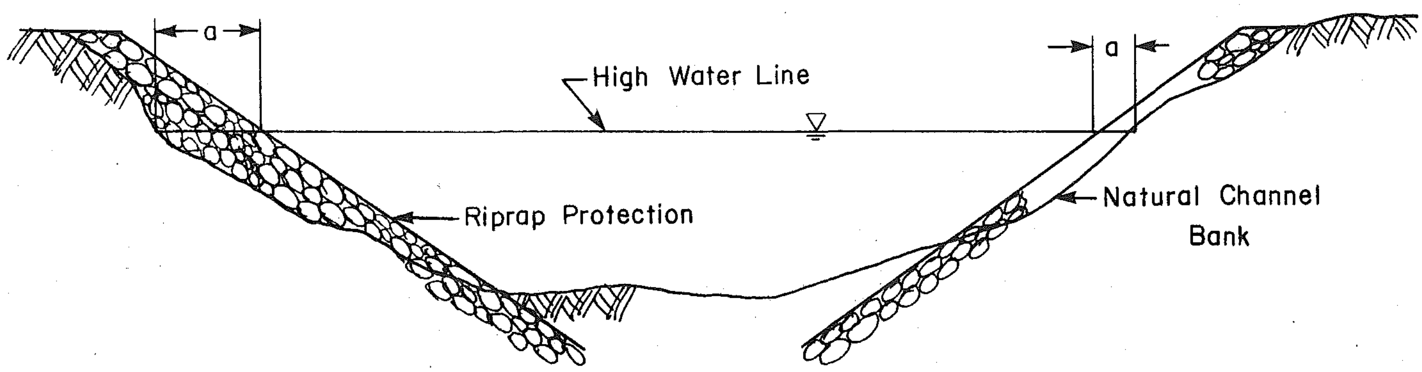


Figure 3-2. The embankment length measured normal to the flow.

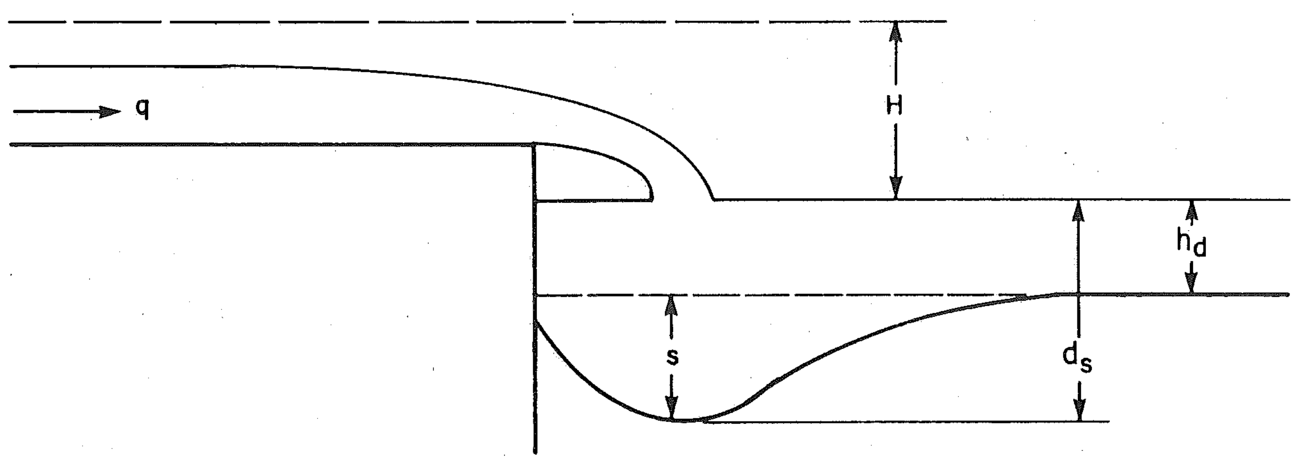


Figure 3-3. Erosion that can take place downstream of hydraulic structures.

structures which are of particular interest in major drainageway design. The term S represents the depth of scour, q is the discharge per unit width, and h_d the depth of flow downstream of the structure.

For the condition in Figure 3-3, a formula defining the scour depth was proposed by Schoklitsch in 1932. It can be written as

$$S = 4.75 \frac{H^{0.2} q^{0.5}}{D_{90}^{0.32}} - h_d \qquad \text{(3-10)}$$

Conservative

where, S = the depth of the scour hole, meters,

h_d = the downstream water depth, meters,

q = the water discharge per unit width, cubic meters per second per meter,

D_{90} = the particle size for which 90 percent of the material is finer, millimeters,

H = the vertical distance between the energy grade line and the downstream water surface, meters.

This formula is in metric units. To use the units of S, H, and h_d in feet, q in cfs, and D_{90} in millimeters, the constant should be 3.75 instead of 4.75. Also, this equation has not been adequately validated for very large particle sizes ($D_{90} \geq 12$ inches). For larger sizes of particles, physical modeling, or some type of field verification based on performance of similar structures, is advised.

3.8 Total Scour

THE TOTAL SCOUR THAT CAN OCCUR AT A STRUCTURE OR PERTINENT LOCATION IS EQUAL TO THE SUM OF DEGRADATION, GENERAL SCOUR, AND LOCAL SCOUR + ½ antidune wave height. Bank protection and protection at structures should extend to a depth below the channel bed equal to the total scour.

Ht. of Antidune = $.14 \frac{2\pi V^2}{g}$

3.9 Lateral Migration

3.9.1 General

Alluvial channels of all types deviate from straight alignment. The thalweg oscillates transversely and initiates the formation of bends. Lateral migration tendencies can be quantitatively evaluated based on a knowledge of geomorphic concepts. A meandering alluvial river has three additional degrees of freedom to adjust its geometry beyond those for a straight alluvial river (i.e., meander wavelength, wave amplitude, and radius of curvature may all change with time). The extend of possible lateral channel migration depends upon: (1) the meander belt width; (2) the rate and direction of the channel's lateral migration; and (3) the possible change of channel alignment due to the development of cutoffs upstream and downstream of the site.

The probable maximum meander wavelength, the probable maximum wave amplitude, and the probable maximum radius of curvature during the project's lifetime are estimated by the relations between meander dimension parameters. Aerial photographs of the past and present are extremely useful in assessment of the changes of all meander dimension parameters.

3.9.2 Lateral Migration Potential

The potential for lateral migration increases as a river becomes more sinuous. If the river system under consideration is straight or has a low sinuosity potential, lateral migration will be small. Figure 3-4 shows the individual meander characteristics, λ the meander wavelength, a the amplitude, R the radius of curvature, and B top width of the channel.

In order to understand the migration process of a specific river system thoroughly, a large reach of river should be studied to determine the range of meander characteristics. The lateral migration potential at a specific

$\lambda = d B$

$\lambda = b R$

$R = c B$

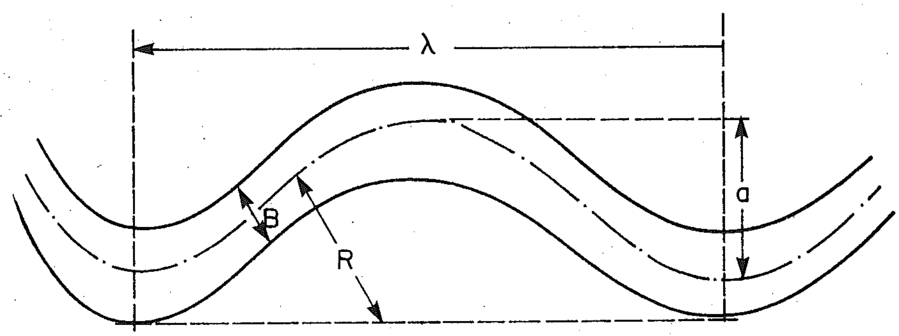


Figure 3-4. Meander characteristics.

site on the river system, such as a hydraulic structure location, is estimated by comparing the meander characteristics of the specific site to the range of meander characteristics of the river system. If the amplitude and radius of curvature at the specific site are smaller than the average of the river system, then the migration tendency will be small. However, if the amplitude and radius of curvature are greater than the average of the river system, the migration tendency will be greater.

When lateral migration potential is high, engineering control measures to prevent failure of a structure are necessary. Two possible measures to control lateral migration are bank stabilization and channelization. It is important to mention again, that the estimation of lateral migration is a qualitative engineering judgment based upon past and present river responses.

3.10 Seepage Force

Seepage forces occur whenever there is inflow or outflow through the bed material and banks of a channel formed in permeable alluvium. The inflow or outflow through the interface between water and channel wall depends on the difference in pressure across the interface and the permeability of the bed material. The seepage force acts to reduce or increase the effective weight and stability of the bed and bank materials depending on inflow or outflow. As a direct result of changing the effective weight, seepage force can influence the form of bed roughness and the resistance to flow for a given channel slope, channel shape, bed material, and discharge.

Seepage force may create an upward hydrostatic pressure on structures (uplift). The magnitude and distribution of seepage forces in a foundation and the amount of underseepage for a given coefficient of permeability can be obtained from a flow net. The weighted-creep theory as developed by Lane (1935) is suggested as a means for designing hydraulic structures on pervious foundations to be safe against uplift pressures and piping. Lane's theory defined weighted-creep ratio as

$$C_w = \frac{\Sigma L_H + 3\Sigma L_V}{3H} \tag{3-11}$$

where, C_w = weighted-creep ratio,

L_H = horizontal or flat contact distance (flatter than 45°),

L_V = vertical or steep contact distance (steeper than 45°),

H = head on structure (headwater - tailwater).

Lane's recommended weighted-creep ratios are given for various foundation materials in Table 3-2. A definition sketch of the variables in Equation 3-11 is presented in Figure 3-5 for nonporous liner and cutoffs.

Piping under the structure foundation occurs when the upward seepage force at the downstream toe of the structure exceeds the submerged weight of material. The soil would be flooded out and the erosion would progress backwards along the seepage flowline until a "pipe" would be formed, allowing rapid flow under the foundation and subsequent failure of the structure.

Cutoff walls, aprons, and drains are generally installed to control the amount of seepage under the structure and to limit the intensity of the uplift so that the stability of the structure will not be threatened.

The weighted-creep theory does not apply to porous liners such as riprap. Porous liners with proper filter underneath will prevent the liner from failure due to seepage force, provided that the submerged weight of the liner is greater than the uplift force. For porous liners, cutoff walls for the purpose of reducing seepage force are not required.

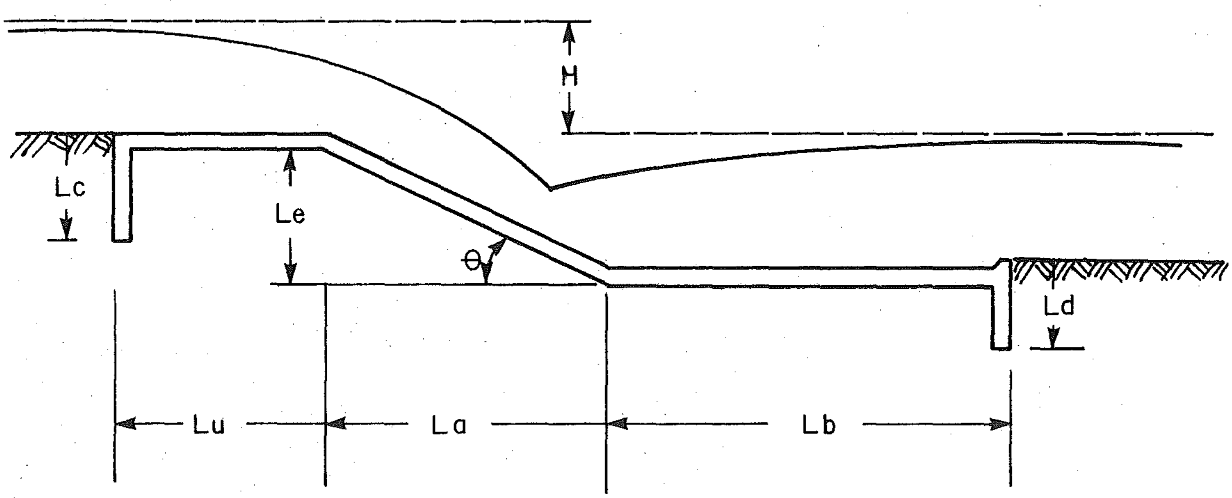
3.11 Bank Erosion

The banks of alluvial rivers experience varying degrees of flow. Forces that cause the movement of water through the bank material may be generated

$$C_w = \frac{\Sigma L_H + 3\Sigma L_V}{3H}$$

Table 3-2. Weighted-Creep Ratios.

Material	C_w
Very fine sand and silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel including cobbles	3.0
Boulder with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay or hardpan	1.6



For $\theta < 45°$, $\Sigma L_H = L_u + L_a + L_b$

$\Sigma L_V = 2\ (L_c + L_d)$

For $\theta > 45°$, $\Sigma L_H = L_u + L_b$

$\Sigma L_V = 2\ (L_c + L_d) + L_e$

Figure 3-5. Definition sketch for weighted-creep theory for nonporous liner and cutoffs.

by several factors:

1. On the rising stage of the design hydrograph, a gradient develops, sloping from the river channel into the bank material. On the falling stage of the design hydrograph, the energy gradient reverses direction and water moves through the bank toward the river channel decreasing the stability of the bank.
2. If the water table is higher than river stage, flow will be from the banks into the river. The high water table may result from many conditions: (a) a wet period during which water draining from adjacent watersheds saturates the floodplain to a higher level; (b) poor drainage conditions resulting from deterioration or failure of drainage systems; and (c) increased infiltration resulting from changes in land use causing an increase in water level.

The presence of water in the banks of rivers and its movement toward or away from the river, affect bank stability and bank erosion in various ways. The related erosion of banks resulting as a consequence of seepage force, piping, and mass wasting.

3.11.1 Piping

Piping is a phenomenon common to the alluvial banks of rivers and foundation soil of structures. For banks that are stratified with lenses of sand and coarser material sandwiched between a layer of finer cohesive materials, flow is induced in the more permeable layers by changes in river stage. With a rise in river stage, a gradient is developed that induces flow into the more permeable lenses of the banks. As the stage drops, the energy gradient is reversed and significant flow occurs toward the river in the more permeable lenses. If the flow through the permeable lenses is capable of dislodging and transporting particles from the permeable lenses, the material is slowly removed, undermining portions of the bank. Without this foundation material to support the overlying layers, a block of bank material drops down and results in the development of tension cracks that may allow surface water to enter, further reducing the stability of the affected block of bank material. Bank erosion may continue on a grain-by-grain basis or the block of bank material may ultimately slide downward and outward into the channel causing bank failure.

3.11.2 Mass Wasting

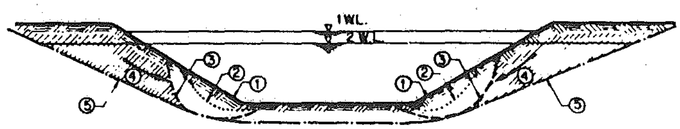
An alternate form of bank erosion is caused by local mass wasting. If the bank becomes saturated and possibly undercut by the flowing water, blocks of the bank may slump or slide into the channel. Mass wasting may be further aggravated by construction of homes on river banks, operation of equipment on the floodplain adjacent to the banks, added gravitational force resulting from trees, location of roads that may cause unfavorable drainage conditions, saturation of banks by leach fields from septic tanks, and increased infiltration of water into the floodplain as a result of changing land use practices.

3.12 Forces Causing Erosion

Erosion of river banks and modifications to channel geometry occur when the net result of all forces acting on the erodible material exceeds the net result of all forces tending to hold the material in place. In general, erosion occurs as shown in Figure 3-6. The most important force is the tractive force. The tractive force can be approximated by the relation

$$\tau_b = \gamma d S_e \qquad (3\text{-}12)$$

1W.L. = First of the Water Level
2W.L. = Second Stage of the Water Level



Stage 1: Man-made trapezoidal section.
Stage 2: Initial erosion at the toe of the side slopes.
Stage 3: Advanced stage of bank erosion.
Stage 4: Sides sliding into the channel.
Stage 5: Final shape of the section.

Figure 3-6. General erosion process.

where, τ_b = stress acts on river bed,

γ = specific weight of the water-sediment mixture,

d = depth of flow at the location where the shear stress is to be estimated, and

S_e = slope of energy gradient.

The specific weight of the water-sediment mixture is used in Equation 3-12 because the presence of suspended sediment in the flow increases the specific weight of the water-sediment mixture and increases its apparent viscosity. These characteristics of the flow directly affect the velocity, velocity distribution, shear stress, and consequently, the rate of erosion.

To estimate the stability of a river channel, the critical shear stress which is just sufficient to initiate movement of bed material can be approximated from the Shields' diagram (Figure 2-9) if there is no sediment being introduced into the channel. If the shear stress acting on the bed of the river (as determined in Equation 3-12) is greater than the critical shear stress, the bed material will be in motion.

The same principle can be used to estimate the critical shear stress and the actual shear stress action on the banks of the river. The actual shear stress on the sides of the channel relative to the shear stress on the bed is given in Figures 3-7 and 3-8. But, if the channel is curved or if the geometry is different, the coefficients 0.75 and 0.97 as shown on Figure 3-7 are not valid. Generally speaking, for trapezoidal channels of the shapes ordinarily used in design, the maximum tractive force on the bottom is close to the value given in Equation 3-12, and on the sides close to

$$\tau_s = 0.76\ \tau_b \tag{3-13}$$

The critical shear stress for the material on the river bank can be estimated utilizing Figure 2-9 to give critical bed shear stress and then, reducing this value by a factor K to allow for the gravitational component of forces on the bank particles:

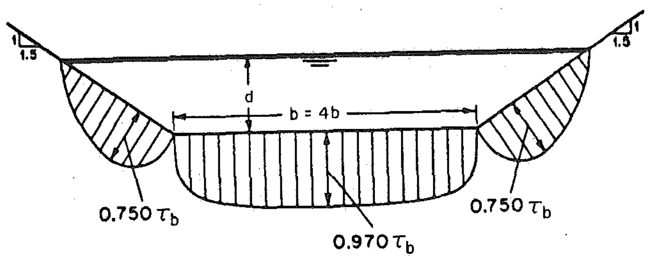


Figure 3-7. Variation of τ in a trapezoidal cross section.

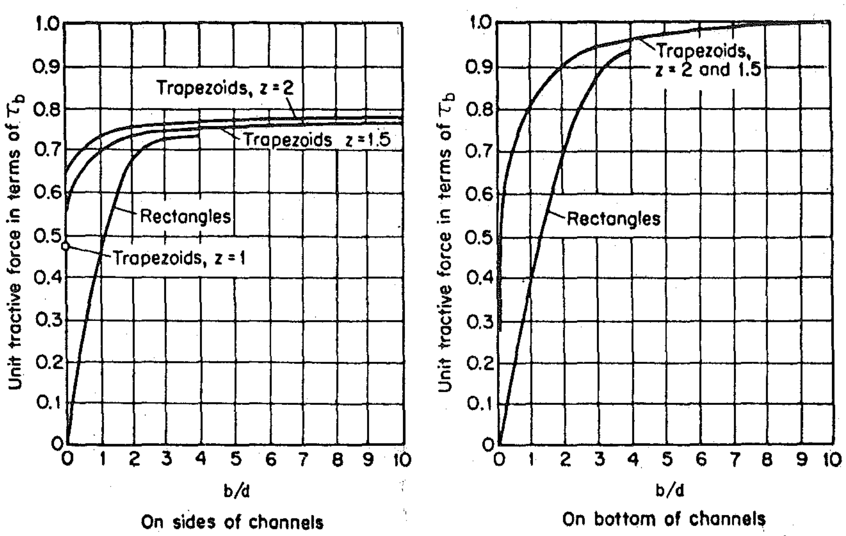


Figure 3-8. Maximum unit tractive force versus b/d.

$(\tau_b)_c \geq \tau_b = \gamma \cancel{R} S$

$$K = \frac{(\tau_s)_c}{(\tau_b)_c} = \cos\theta \sqrt{1 - (\tan^2\theta/\tan^2\phi)} \qquad (3\text{-}14)$$

$(\tau_s)_c = K(\tau_b)_c \geq \tau_s$

where, $(\tau_s)_c$ = the critical shear stress on the side,

$(\tau_b)_c$ = the critical shear stress on the bed,

θ = the angle of the side slope,

ϕ = the angle of repose of the bank material which can be estimated from Figure 3-9.

3.13 Effects of Bends

Because of the change in flow direction in the bend, there is a centrifugal force that causes superelevation of the water surface. That is, the water surface is higher at the concave bank (outer bank) than the convex bank (inner bank). There are many equations for evaluating the superelevation and the differences in the superelevations that are obtained using these equations are small. The following equation for computing the superelevation was recommended by Richardson et al. (1975):

$$\Delta Z = \frac{V^2}{g\ r_c} W \qquad (3\text{-}15)$$

where, ΔZ = the superelevation,

r_c = the radius at the center of a bend,

W = the top of the channel.

In a bend, the boundary shear stress acting on the outside bend also increases due to the uneven velocity distribution across a cross section. The velocity of flow is generally higher on the outside of the bend and smaller on the inside. These changes in velocity cause even larger changes in the boundary shear stress acting on the bed and banks of the river. The analysis presented in Section 3.12 applies to essentially straight reaches of river channel. This method of analysis can be extended to apply to the

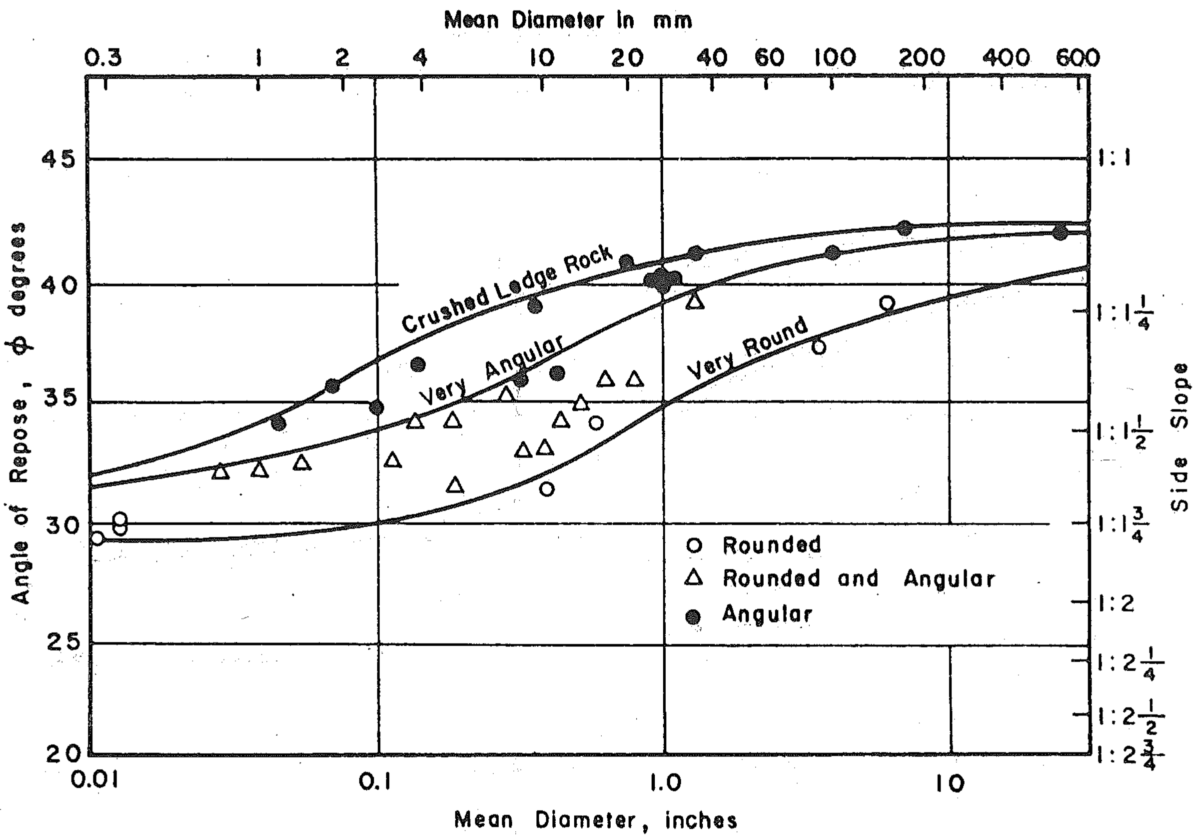


Figure 3-9. Angle of repose.

outside banks of a river system by considering the lateral velocity distribution resulting from the properties of the river bend. A graph relating the boundary shear stress in a curved reach to that of a straight reach is given in Figure 3-10. This figure shows the factor by which the mean shear stress is increased for the outside bends in curved channels. This value changes depending on the curvature of the channel. The stability of the channel bed and banks can be evaluated using the value established from Figure 3-10.

3.14 Freeboard

Freeboard is the vertical distance from the water surface elevation of the design flow to the top of channels or structures. The freeboard for a channel will depend on a number of factors, such as size of channel, velocity of water, curvature of alignment, storm water entering the channel, wind, and wave action. In channel design, the wave due to wind action is not significant. In sand-bed channels, however, the wave height due to an antidune should be considered for upper regime flows. The maximum wave height, due to antidune before breaking, can be computed as:

$$h_a = 0.14 \frac{2\pi V^2}{g} \tag{3-16}$$

where, h_a = the antidune wave height,

V = the mean flow velocity in the channel.

The minimum freeboard should be the sum of the velocity head, superelevation, and one-half of the antidune wave height, but not less than the freeboard specified in Section 2.3.1E of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2.

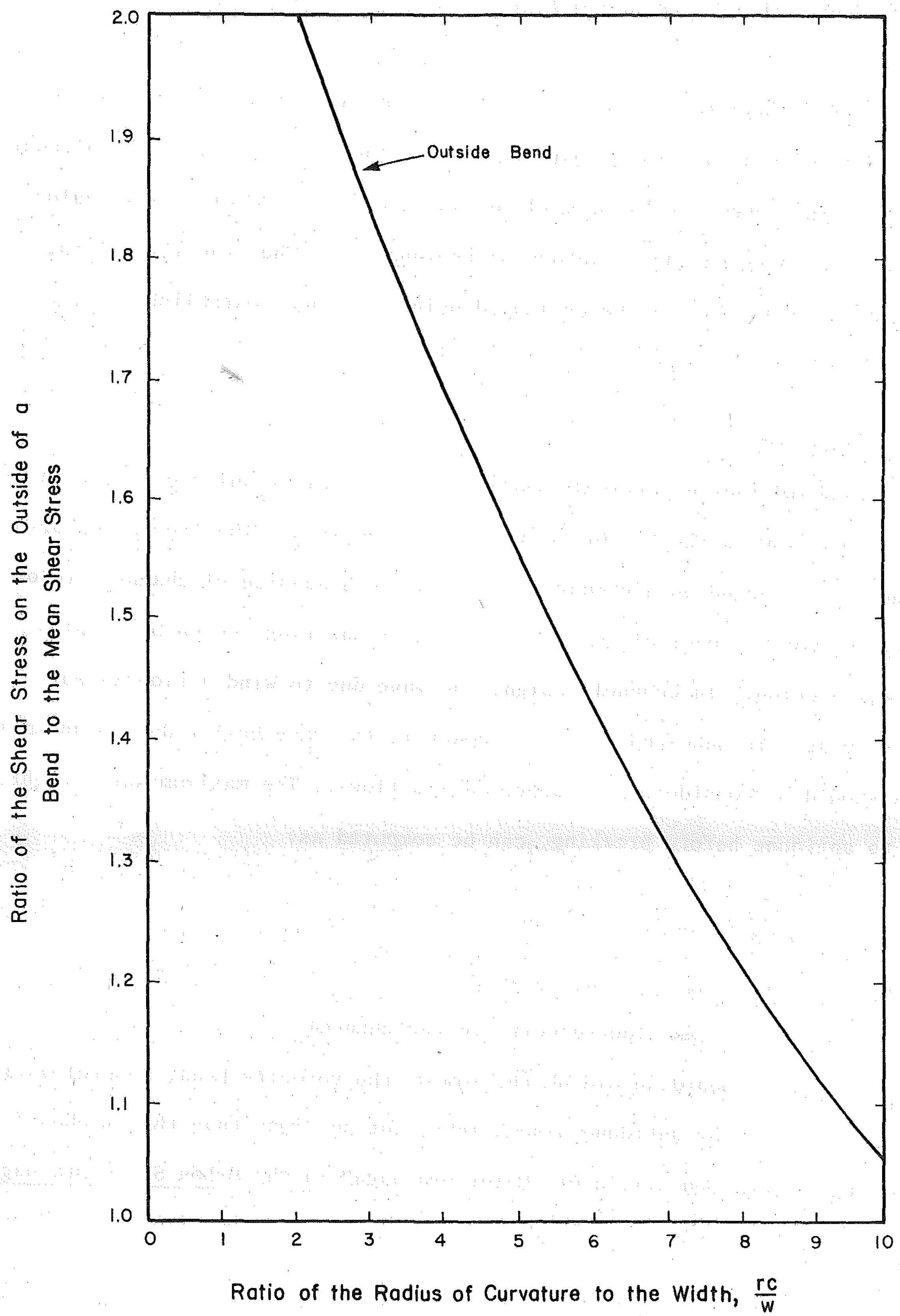


Figure 3-10. Effect of bend on boundary shear stress (after Soil Conservation Service design manual).

IV. DATA REQUIREMENTS

4.1 General

The primary purpose of this section is to identify data needs for the geomorphic and hydraulic analyses of river systems. Although large volumes of data relative to the morphologic and hydraulic characteristics of rivers have been collected, much of this data is not readily obtainable. Consequently, the search for data, which is a necessary preliminary step to any river system analysis, can consume a significant portion of the time and money allocated to a given study. With a view toward minimizing the investment in the data gathering effort, a checklist is provided to serve as both a guide for data gathering and as an outline of basic considerations for the analysis of the impact of historical and proposed development activities on the river environment.

4.2 Checklist for Data Needs

The type of data needed for qualitative and quantitative river analyses and the relative importance of each data type, are listed in Table 4-1. Data with a degree of importance "primary" are basic data required for any geomorphic, hydraulic, and environmental study of a river. Whenever possible, these data should be directly collected from the field. Other data with a degree of importance "secondary" are also very helpful in an analysis of a river, but are considered a secondary requirement. It must be noted that certain categories of data, including hydrologic, hydraulic, channel geometry, and hydrographic, are extremely dynamic in nature and strongly a function of past and present conditions. Therefore, available data should be validated against today's river system conditions to determine their acceptability for the analysis.

Table 4-1. Checklist of Data Needs.

Description of Data	Degree of Data Importance
Hydrology	
Design discharges with anticipated urbanization	Primary
Design hydrographs with anticipated urbanization	Primary
Flood frequency curves	Primary
Flood history	Secondary
Hydraulics	
Channel geometry	Primary
Bed slopes	Primary
Backwater calculation	Primary
Channel type (braided, meandering, straight)	Secondary
Channel controls (drops, restrictions, diversions)	Primary
Roughness coefficients	Primary
Soils	
Bed material size distribution	Primary
Bank material size distribution	Primary
Bed load	Primary
Suspended load	Primary
Wash load	Secondary
Hydraulic Structures (existing and planned structures)	
Plans and design details	Primary
Scour survey around hydraulic structures	Secondary
Alterations and repairs	Secondary
Aerial Photographs	
Recent and past photographs showing the river and surrounding terrain	Primary

Table 4-1. Checklist of Data Needs (continued).

Description of Data	Degree of Data Importance
Land Use	
Existing land use	Primary
Planned land use maps	Primary
Field Surveys	
Topographic maps	Primary
On site inspection and photographs	Primary
Observe channel changes or realignment since last maps or photographs	Primary
Sample sediments	Secondary
Measure water and sediment discharge	Secondary
Identify high waterlines or debris deposit due to recent floods	Secondary
Subsurface exploration	Secondary

V. DESIGN STANDARDS

5.1 General

The characteristics of sand bed channels, general design considerations, and data requirements are discussed in previous chapters. The primary purpose of this chapter is to establish a standard for designing major drainageway channels and hydraulic structures. The design standard for each type of structure includes its applicability, design procedures, design aids, and an illustration to assure that there will be certain uniformity in performance with respect to design and construction of major drainageway channels and hydraulic structures on sandy soils.

5.2 Design of Stable Channel

5.2.1 General

Stable channel cross sections formed in sandy soils are usually wide and shallow because the fine particles cannot withstand high velocities, turbulence, and tractive forces. Stable channel designs using maximum permissible velocity or critical shear stress criteria are frequently utilized. These methods often result in large geometric sections. In many cases, the right of way required by a wide channel is impractical and uneconomical. The design of a stable channel in the sediment laden stream usually requires bank protection. It is possible to obtain a more practical section by using a properly designed lining. Channel linings commonly used are grass, concrete, riprap, or gabion.

In sand-bed channels, stabilization of channel banks is critical because the channel banks are subjected to forces that cause lateral shifting. Channel stability may be accomplished utilizing channel bank protection only. However, the equilibrium channel slope must also be evaluated and provided so that major drainageway facilities can be properly designed.

5.2.2 Design Criteria

5.2.2.1 River Morphology and River Response

The hydraulic and geomorphic response of channels to imposed natural and man-made changes can be evaluated utilizing different relationships, methods, and levels of analysis. In all cases, it is of value to initially analyze the behavior and response of a system utilizing qualitative geomorphic and hydraulic response relationships. These methods of analysis are simple to apply and even with the most sophisticated methods of analysis, this geomorphic and hydraulic analysis provides a valuable check on the final quantitative results. The river morphology and river response is covered in detail in Section 2.2 of this report.

5.2.2.2 Equilibrium Slope

The equilibrium channel slope is defined as the slope at which the channel's sediment transporting capacity is equal to the incoming sediment supply. That is,

$$(Q_s)_{in} = (Q_s)_{out} \tag{5-1}$$

where, $(Q_s)_{in}$ = supply rate of sediment into the channel reach, and

$(Q_s)_{out}$ = supply rate of sediment out of the channel reach.

The equilibrium channel slope is used to predict the river response to man-induced changes. The evaluation will provide an understanding of the long-term effects such measures as channelization or reducing sediment supply due to urbanization will have on the channel profile.

5.2.2.3 Design Discharge

The design discharge for equilibrium channel slope determination is a discharge for which the sediment supply is to be determined. The design discharge should be a discharge which will determine the long-term response of the channel. In most cases, the mean annual flood is recommended. A peak discharge of two-year return period may be used if the mean annual flood is not available. In addition to the prediction of a long-term equilibrium channel slope, degradation or aggradation from a rare flood event, such as a 100-year flood, needs to be evaluated because the maximum short-term degradation or aggradation usually will occur during a rare storm.

5.2.2.4 Supply Reach

A major controlling factor when assessing channel response is the upstream sediment supply. Whether a channel degrades or aggrades strongly depends on the balance between the incoming sediment supply and a reach's transporting capacity. This is especially true for channels where armoring does not occur.

It is extremely important to understand that the supply system is being subjected to conditions that can drastically alter the sediment supply. The major alteration is a result of the increasing urbanization of the area.

In many regions, the urbanization process is viewed as increasing the sediment supply because of order-of-magnitude increases in erosion during construction functions. However, if the land has protective cover, added exposure to erosion is much less significant than in many other environments. The major effect in this case, is actually a reduction in sediment supply during urbanization because of clear water releases from sedimentation and flood water retention structures required by regulations. Supply is also reduced if land owners take measures to prevent erosion due to flows crossing their properties. Covering the soil with pavement, rock, houses, and vegetative landscaping such as lawns, reduces erosion. All these factors contribute to bring less sediment to the river system. Thus, as urbanization continues, there should be a corresponding decrease in sediment supply to a major drainageway.

Incoming sediment supply is very difficult to estimate. A practical way to estimate the incoming sediment supply is to select a supply reach. The supply reach must be close to its equilibrium condition. Usually, the supply reach is selected from (1) use the sediment transport capacity at an upstream reach as sediment supply using the Urban Storm Drainage Criteria Manual criteria of five feet per second maximum velocity and five feet maximum flow depth if upstream urbanization is expected; (2) a natural channel reach, upstream of the design reach, which has not been disturbed by man's activities; or (3) an upstream channelized reach which has been in existence for many years and has not experienced a recent change in profile or cross section. The sediment transport capacity of the supply reach can be calculated and used as the incoming sediment supply.

5.2.2.5 Sediment Transport Equation

The sediment transport capacity in a river can generally be roughly estimated by a simplified equation that is a function of flow depth and velocity. The sediment transport capacity should be determined using a combination of the Meyer-Peter, Muller equation and the Einstein method as presented in Section 2.5.3 of this report. The sediment transport capacity should be determined for various flow conditions and sediment sizes and distributions likely to occur in the study reach. Sediment transport equations for sand materials and their parameter ranges have been presented in Section 3.5 of this report.

The sediment transport capacity can be approximately estimated using the sediment transport equation and using only the D_{50} sediment size. However, a detailed calculation using sediment size fractions is recommended.

5.2.2.6 Sediment Size

The sediment transport equations are based on the assumption that all the sediment sizes present can be moved by the flow. If this is not true, armoring will take place. The equations are not applicable when armoring occurs. The potential for armoring can be determined using Shields' critical shear stress criteria as presented in Figure 2-9. The bed shear stress is given by two equations that are closely related. The equation $\tau_o = \gamma RS$ is usually the most simple one to utilize.

$$\tau_o = \gamma RS \quad \text{or} \quad \tau_o = (1/8)\, \rho f_o\, V^2 \tag{5-2}$$

in which, γ = the specific weight of water,

R = the hydraulic radius,

S = the energy slope,

ρ = the density of water,

f_o = the Darcy-Weisbach friction factor,

V = the mean flow velocity.

The diameter of the largest particle moving is then,

$$D = \tau_o/0.047\,(S_s - 1)\gamma \tag{5-3}$$

in which, D = the diameter of the sediment,

S_s = the specific gravity of sediment,

0.047 = the recommended value of Shields' parameter.

(All units are in feet, pounds, and seconds.) If no sediment of the computed size or larger is present, the equations are applicable.

These equations were developed for sand-bed channels and do not apply to conditions when the bed material is cohesive. The equations would over predict transport rates in a cohesive soil channel.

5.2.3 Design Procedures and Example

5.2.3.1 Equilibrium Slope Determination

1. Determine dominant discharge (mean annual flood or two-year storm peak discharge).

2. Select upstream supply reach and obtain the following pertinent information:

   a. channel geometry
   b. channel slope
   c. sediment size distribution
   d. channel resistance (n)

3. Obtain the same pertinent data as in Step 2 for the channelization under consideration.

4. Calculate the hydraulic conditions based on the dominant discharge.

5. After it has been shown that the sediment transport equation is applicable, the sediment supply from the upstream channel is computed using the equation. The calculated sediment supply is per unit width. The total sediment transport rate is obtained by multiplying the rate per unit width by the top width.

6. Determine the equilibrium slope for the downstream channel with the sediment supply rate determined in Step 5. This requires a trial and error procedure by which a given slope is chosen to compute the flow conditions and from the flow conditions, the sediment transport is calculated. When the computed rate is equal to the supply rate, the equilibrium slope has been found.

7. Based on the hydraulic conditions at equilibrium slope, estimate the largest particle size moving for armoring control check. Also, compare hydraulic parameters with the range of parameters for application of the equations.

8. Check whether the channel will be degraded or aggraded during the design storm event.

5.2.3.2 Design Example

The following is an example of the procedure by which the equilibrium slope of a channel can be calculated. The physical layout of the system is given in Figure 5-1. The upstream channelized section has been in existence for many years and has not changed significantly. It has been proposed that the channelization be carried out the remainder of the distance to the main river because of a proposed development along the unchannelized portion. The proposed downstream channel is limited by the available right of way and economic considerations. Determine the equilibrium slope and channel degradation or aggradation during a 20-year storm of 900 cfs.

1. Pertinent information:

	Upstream Channel	Downstream Channel
Dominant discharge	250 cfs	250 cfs
Channel ~~slope~~ shape	trapezoidal	trapezoidal
Sediment size distribution	--- See Figure 5-2 ---	
Channel resistance	0.025	0.025
Side slopes	4:1	4:1
Channel slope	0.00142	0.0013

2. Compute the hydraulic conditions for upstream channel assuming normal depth at $Q = 250$ cfs. Hydraulic conditions:

$Y = 2.30$ feet	$A = 76.36$ feet^2
$V = 3.27$ feet per second	$W = 42.4$ feet
$Y_n = 1.80$ feet	$Fr = 0.43$

The variable Y_n is the hydraulic depth (A/W). Since the sediment transport equations were developed for a unit width channel, the hydraulic depth is a better representation of the average channel characteristics than the thalweg depth, Y.

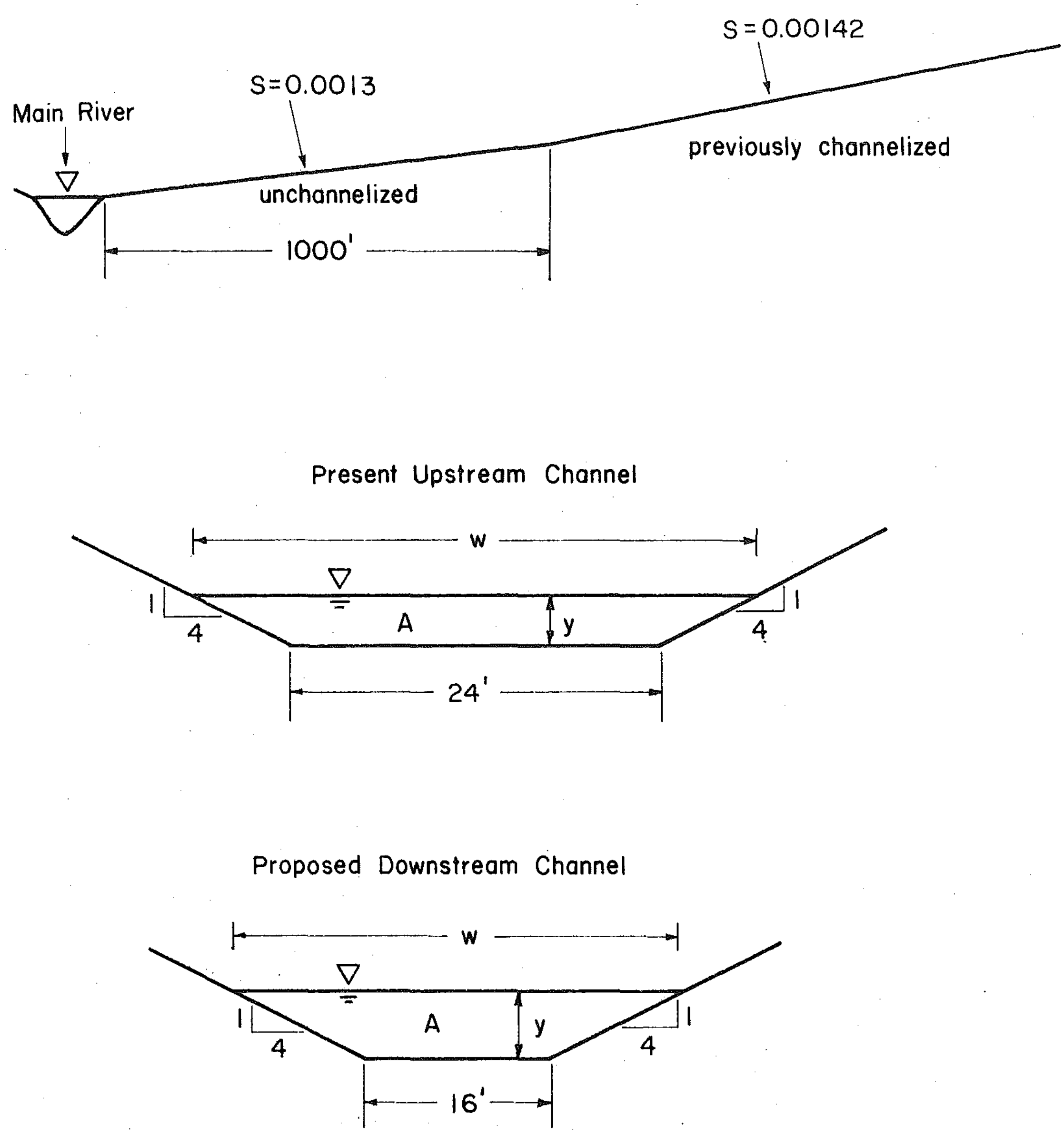


Figure 5-1. Physical layout of design example.

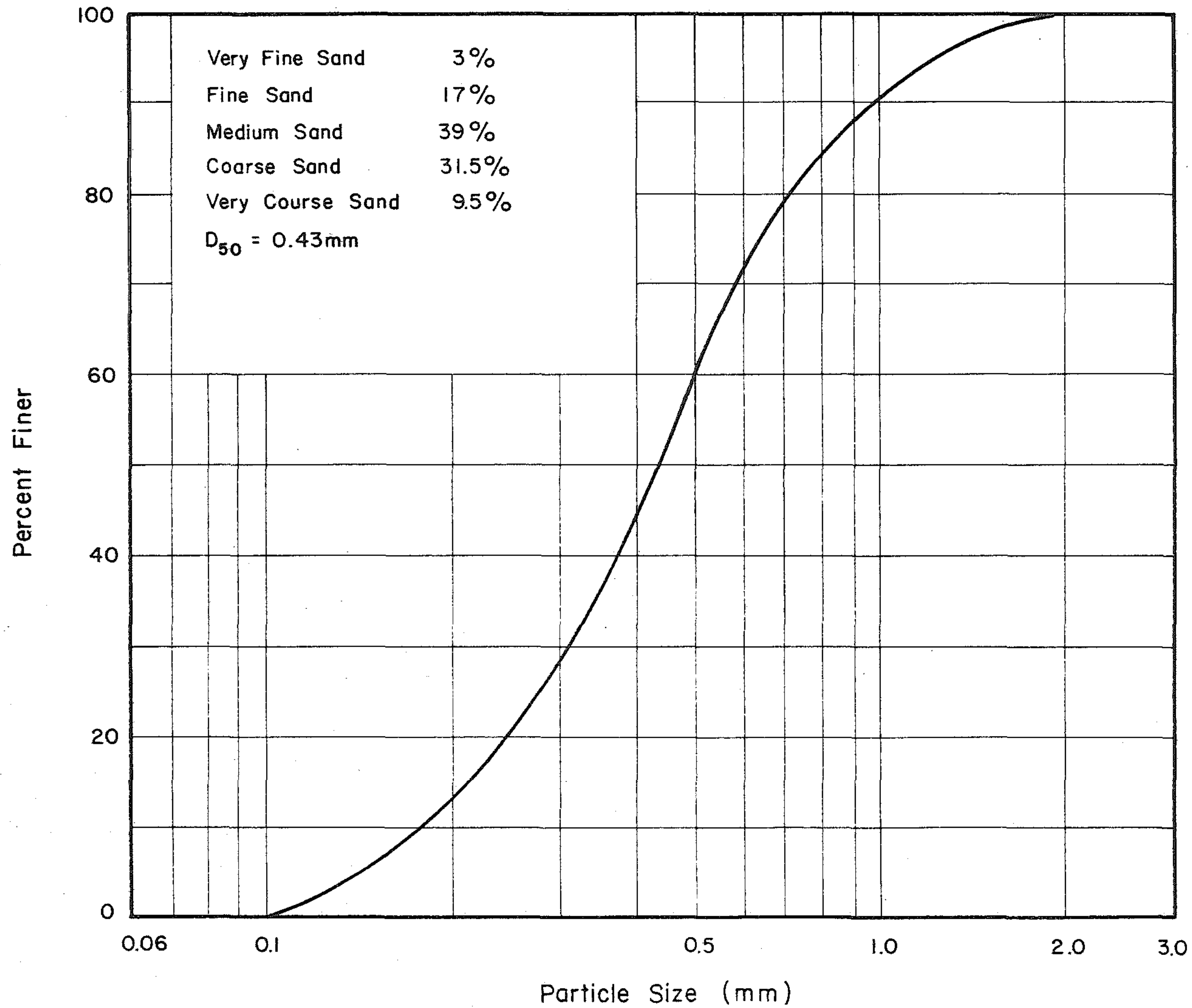


Figure 5-2. Sediment size distribution (design example).

3. Calculate sediment supply from upstream using the sediment transport equation in Section 3.5.

Very fine sand: $q_{s1} = 58.5 \times 10^{-6}(1.80)^{1.04}(3.27)^{3.20}$

$= 4.78 \times 10^{-3}$ cfs/foot.

Fine sand: $q_{s2} = 21.4 \times 10^{-6}(1.80)^{0.837}(3.27)^{3.59}$

$= 2.46 \times 10^{-3}$ cfs/foot.

Medium sand: $q_{s3} = 6.47 \times 10^{-6}(1.80)^{0.535}(3.27)^{4.05}$

$= 1.07 \times 10^{-3}$ cfs/foot.

Coarse sand: $q_{s4} = 2.90 \times 10^{-6}(1.80)^{0.239}(3.27)^{4.36}$

$= 0.58 \times 10^{-3}$ cfs/foot.

Very coarse sand: $q_{s5} = 2.37 \times 10^{-6}(1.80)^{-0.044}(3.27)^{4.44}$

$= 0.44 \times 10^{-3}$ cfs/foot.

Total sediment supply is given by:

$$Q_s = W\,(K_1q_{s1} + K_2q_{s2} + K_3q_{s3} + K_4q_{s4} + K_5q_{s5})$$

where, K_i = fraction of sediment size of q_{si}. Then,

$$Q_s = 42.4\ [(0.03 \times 4.78 \times 10^{-3}) + (0.17 \times 2.46 \times 10^{-3}) + (0.39 \times 1.07 \times 10^{-3}) + (0.315 \times 0.58 \times 10^{-3}) + (0.095 \times 0.44 \times 10^{-3})] = 0.051 \text{ cfs.}$$

4. Determine the equilibrium slope based on an upstream sediment supply of 0.051 cfs. Assume V = 3.30 fps, then,

$A = 250/3.30 = 75.76$ feet^2, $Y_n = 1.98$ feet,

$Y = 2.79$ feet, $R = 1.94$ feet.

$W = 38.32$ feet,

The sediment transport by size fraction is:

Very fine sand: $q_{s1} = 58.5 \times 10^{-6}(1.98)^{1.04}(3.30)^{3.20}$

$= 5.43 \times 10^{-3}$ cfs/foot.

Fine sand: $q_{s2} = 21.4 \times 10^{-6}(1.98)^{0.837}(3.30)^{3.59}$

$= 2.76 \times 10^{-3}$ cfs/foot.

Medium sand: $q_{s3} = 6.47 \times 10^{-6}(1.98)^{0.535}(3.30)^{4.05}$

$= 1.17 \times 10^{-3}$ cfs/foot.

Coarse sand: $q_{s4} = 2.90 \times 10^{-6}(1.98)^{0.239}(3.30)^{4.36}$

$= 0.62 \times 10^{-3}$ cfs/foot.

Very coarse sand: $q_{s5} = 2.37 \times 10^{-6}(1.98)^{-0.044}(3.30)^{4.44}$

$= 0.46 \times 10^{-3}$ cfs/foot.

Total sediment transport capacity is:

$$Q_s = 38.32\ [(0.03 \times 5.43 \times 10^{-3}) + (0.17 \times 2.76 \times 10^{-3}) + (0.39 \times 1.17 \times 10^{-3}) + (0.315 \times 0.62 \times 10^{-3}) + (0.095 \times 0.46 \times 10^{-3})] = 0.0508 \text{ cfs} \simeq 0.051 \text{ cfs.}$$

Note that $(Q_s)_{in}$ is equal to $(Q_s)_{out}$ and the channel is stable for the assumed velocity. The equilibrium slope for the dominant flow is then calculated using

$$V = \frac{1.486}{0.025} R^{2/3} S^{1/2},$$

for,

$V = 3.30$ fps,

$R = 1.94$ feet, then,

$S = 0.00127$.

If the $(Q_s)_{in}$ is not equal to $(Q_s)_{out}$, assume another velocity. The solution involves a trial and error process.

5. Compute the largest particle being transported based on equilibrium slope using Equation 5-2.

$$\tau_o = \gamma RS = 62.4 \times 1.94 \times 0.00127 = 0.15 \text{ pounds per feet}^2.$$

$$D = 0.15/0.47\ (2.65 - 1)\ 62.4 = 0.031 \text{ feet} = 9.4 \text{ mm.}$$

All sediment smaller than 9.4 mm will be moving and armoring will not control.

6. Design the downstream channel at a slope of 0.00127.

7. Check the channel conditions during a 20-year flood event.

   a. Knowing the pertinent information, determine the hydraulic conditions for $Q = 900$ cfs.

	Upstream Channel	Downstream Channel
Y	4.50 feet	5.20 feet
A	189.00 feet^2	191.36 feet^2
W	60.00 feet	57.60 feet
Y_n	3.15 feet	3.32 feet
V	4.76 fps	4.70 fps
Fr	0.47	0.45

   b. Calculate the upstream and downstream sediment transport rates.

      1. Upstream

Very fine sand: $q_{s1} = 58.5 \times 10^{-6}(3.15)^{1.04}(4.76)^{3.20}$

$= 2.84 \times 10^{-2}$ cfs/foot.

Fine sand: $q_{s2} = 21.4 \times 10^{-6}(3.15)^{0.837}(4.76)^{3.59}$

$= 1.51 \times 10^{-2}$ cfs/foot.

Medium sand: $q_{s3} = 6.47 \times 10^{-6}(3.15)^{0.535}(4.76)^{4.05}$

$= 0.66 \times 10^{-2}$ cfs/foot.

Coarse sand: $q_{s4} = 2.90 \times 10^{-6}(3.15)^{0.239}(4.76)^{4.36}$

$= 0.34 \times 10^{-2}$ cfs/foot.

Very coarse sand: $q_{s5} = 2.37 \times 10^{-6}(3.15)^{-0.044}(4.76)^{4.44}$

$= 0.23 \times 10^{-2}$ cfs/foot.

Total sediment transport capacity is:

$$Q_s = 60.00\ [(0.03 \times 2.84 \times 10^{-2}) + (0.17 \times 1.51 \times 10^{-2}) + (0.39 \times 0.66 \times 10^{-2}) + (0.315 \times 0.34 \times 10^{-2}) + (0.095 \times 0.23 \times 10^{-2})] = 0.44 \text{ cfs.}$$

2. <u>Downstream</u>

Very fine sand: $q_{s1} = 58.5 \times 10^{-6}(3.32)^{1.04}(4.70)^{3.20}$

$= 2.88 \times 10^{-2}$ cfs/foot.

Fine sand: $q_{s2} = 21.4 \times 10^{-6}(3.32)^{0.837}(4.70)^{3.59}$

$= 1.51 \times 10^{-2}$ cfs/foot.

Medium sand: $q_{s3} = 6.47 \times 10^{-6}(3.32)^{0.535}(4.70)^{4.05}$

$= 0.65 \times 10^{-2}$ cfs/foot.

Coarse sand: $q_{s4} = 2.90 \times 10^{-6}(3.32)^{0.239}(4.70)^{4.36}$

$= 0.33 \times 10^{-2}$ cfs/foot.

Very coarse sand: $q_{s5} = 2.37 \times 10^{-6}(3.32)^{-0.044}(4.70)^{4.44}$

$= 0.22 \times 10^{-2}$ cfs/foot.

Total sediment transport capacity is:

$$Q_s = 57.60\ [(0.03 \times 2.88 \times 10^{-2}) + (0.17 \times 1.51 \times 10^{-2}) + (0.39 \times 0.65 \times 10^{-2}) + (0.315 \times 0.33 \times 10^{-2}) + (0.095 \times 0.22 \times 10^{-2})] = 0.41 < 0.44 \text{ cfs.}$$

The downstream channel will be slightly aggraded during the 20-year flood event.

c. Calculate the equilibrium slope required for the upstream sediment transport capacity calculated in Step b. Assume,

$V = 4.8$ fps,

$A = 900/4.8 = 187.5$ feet^2

then,

$Y = 5.13$ feet, $Y_n = 3.28$ feet,

$A = 187.35$ feet^2, $R = 3.21$ feet.

$W = 57.04$ feet,

Very fine sand: $q_{s1} = 58.5 \times 10^{-6}(3.28)^{1.04}(4.8)^{3.20}$

$= 3.05 \times 10^{-2}$ cfs/foot.

Fine sand: $q_{s2} = 21.4 \times 10^{-6}(3.28)^{0.837}(4.8)^{3.59}$

$= 1.61 \times 10^{-2}$ cfs/foot.

Medium sand: $q_{s3} = 6.47 \times 10^{-6}(3.28)^{0.535}(4.8)^{4.05}$

$= 0.70 \times 10^{-2}$ cfs/foot.

Coarse sand: $q_{s4} = 2.90 \times 10^{-6}(3.28)^{0.239}(4.8)^{4.36}$

$= 0.36 \times 10^{-2}$ cfs/foot.

Very coarse sand: $q_{s5} = 2.37 \times 10^{-6}(3.28)^{-0.044}(4.8)^{4.44}$

$= 0.24 \times 10^{-2}$ cfs/foot.

Total sediment transport capacity is:

$$Q_s = 57.04 \times [(0.03 \times 3.05 \times 10^{-2}) + (0.17 \times 1.61 \times 10^{-2}) + (0.39 \times 0.70 \times 10^{-2}) + (0.315 \times 0.36 \times 10^{-2}) + (0.095 \times 0.24 \times 10^{-2})] = 0.44 \text{ cfs}$$

which is equal to the sediment supply rate from the upstream reach. Thus,

$$V = 4.8 = \frac{1.486}{0.025}(3.21)^{2/3} S^{1/2}$$

and the equilibrium slope for the 20-year flood peak is

$S = 0.00138.$

d. Compare the design flow equilibrium slope to the annual flood equilibrium slope. The gradient difference based on the channel length is the short-term aggradation or degradation that can occur at any point along the channelized reach during the design flow. (In this example, the aggradation is 1000 x (0.00138 - 0.00127) = 0.11 feet.) If the channel is degraded, the degradation should be added to the local scour at structures and bank protection points to determine the depth of protection required below the bed elevation.

5.3 Erosion Control Checks

5.3.1 General

Erosion control checks are often used to prevent erosion of the bed of steep-sloped small water courses. Such application can be seen as a control of the extent that a channel can degrade by providing downstream control points. As the distance between control points is decreased, less degradation will occur in the channel.

An equilibrium situation with deep erosion gullies often cannot be accepted and the construction of a series of fixed erosion control checks offers a solution. Figure 5-3 shows the natural condition erosion process generally referred to as headcutting.

The degradation process shown in Figure 5-3 can be controlled by placement of grade control structures. A grade control structure provides a pivot for the equilibrium slope. In other words, the final bed slope will be the equilibrium slope with the grade control structure as the minimum elevation downstream control. Figure 5-4 shows the effect of grade control structures on the condition presented in Figure 5-3.

It is obvious that the foundation depth of the control structures must be adapted to the elevation of the bed to be expected in the final equilibrium state. When the final equilibrium state is reached, the grade control structure is, in effect, a drop structure and should be reanalyzed as such. The local scour should be determined and adequate protection provided. Drop structures and placement of protection measures are analyzed in Section 5.5.

Use of drop structures at the outset is preferred in urban areas even though they may be buried until the stream has degraded. Drop structures provide facilities for the long-term as long as they do not require upgrading by the local governments (public sector) in the future.

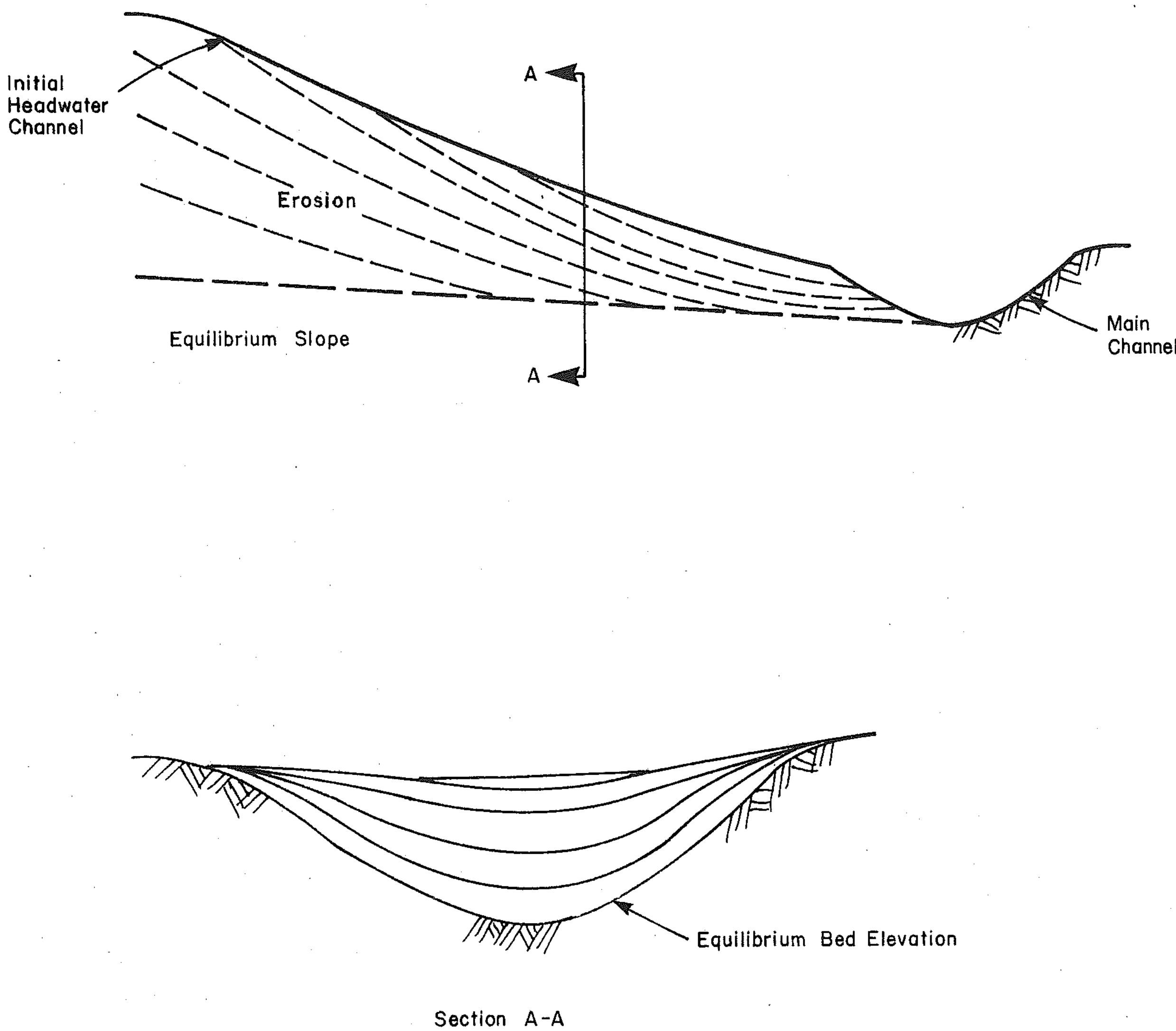
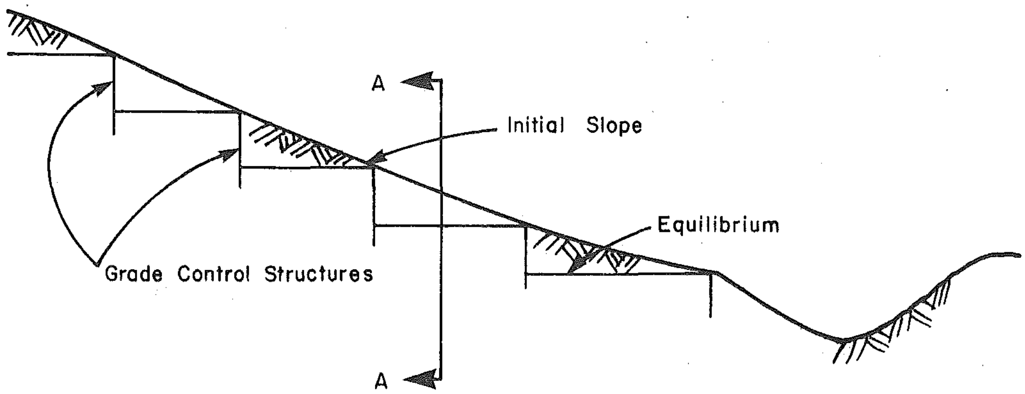
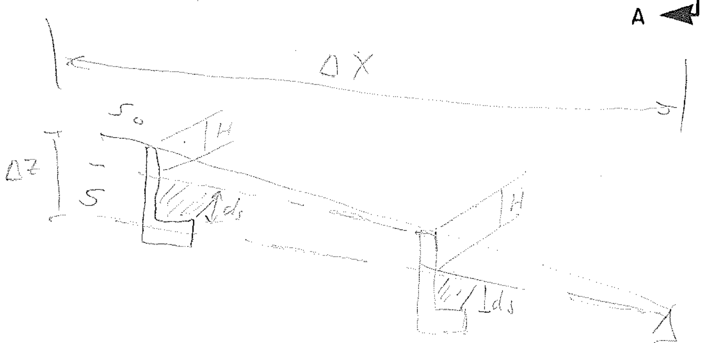


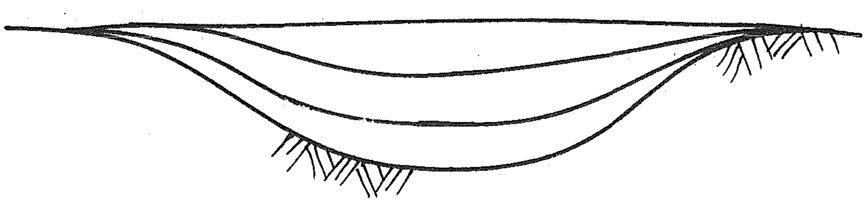
Figure 5-3. Erosion process of headcutting.





$$(S_0 - S)\,\Delta X = \Delta Z$$

$$\frac{\Delta Z}{H} = \#\text{ of drops}$$



Section A-A

Figure 5-4. Effect of grade control structures.

5.3.2 Design Criteria

5.3.2.1 Equilibrium Slope

The spatial placement and depth of the grade control structure is based on the equilibrium slope as defined in Section 5.2.2.2. The criteria for determining equilibrium slope is as defined in Section 5.2.2.

5.3.2.2 Final Equilibrium Slope

After a final equilibrium state is reached, the grade control structure should be reevaluated as a drop structure and adequate scour protection provided.

5.3.2.3 Material

The grade control structure can be formed of concrete or by placement of two rows of sheet pilings and filling between them with rock.

5.3.2.4 Size

The control structure should span the entire expected channel width. The grade control structure should be extended to a depth below the scour depth due to rare storm events after the equilibrium slope is expected. The thickness of the structure should be a minimum of five feet for sheet piling with rock filling.

5.3.3 Design Procedures

1. The equilibrium slope is determined as shown in the design example in Section 5.2.3.2.
2. Knowing the limit of degradation required and the equilibrium slope, the placement and number of control structures are determined and economically calculated.

3. Size limitations on grade control structures are given in Section 5.5.1.4.
4. Assuming the final equilibrium condition, the required structure and scour protection needed in the future is determined. A design example for drop structures is given in Section 5.5.3

5.4 Riprap

Riprap is usually used to prevent channel bottom and bank damage upstream and downstream from hydraulic structures, at bends, at bridges, and in other channel areas where erosive tendencies exist. When available in sufficient size, rock riprap is usually the most economical material for bank protection. Rock riprap has many other advantages over other types of protection. Rock riprap protection is flexible and local damage is easily repaired. Experience has shown that the usual causes of riprap failure are undersized individual rocks in the maximum size range; and improperly designed riprap gradation, thickness of layer, and bedding material. Among them, 80 percent of all riprap failure is directly attributed to bedding failure. The important factors to be considered in designing rock riprap protection on sandy soils are:

1. Rock properties.
2. Slope of riprap protection.
3. Size of riprap and its gradation.
4. Filters for riprap.
5. Thickness of riprap and filters.
6. Height of riprap protection including superelevation and antidune waves.
7. Depth of riprap protection including degradation, general scour, local scour, and antidune waves.

5.4.1 Design Criteria

5.4.1.1 Rock Properties

Rock used for riprap should be as specified in Section 5.1.1 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2.

5.4.1.2 Slope of Riprap Protection

The slope of riprap protection should be at least five degrees less than the angle of repose of bank materials, but not steeper than 2:1. The angle of repose of crushed ledge rock, very angular, and very rounded materials and mean diameter of the material is presented in Figure 3-9.

5.4.1.3 Size of Riprap

Sizing rocks for riprap is based on the stability of the rock on a slope under hydraulic forces. Riprap requirements for a stable channel side slope lining, in terms of the channel side slopes and the parameter $V^2/R^{0.33}$, should be as specified in Section 5.1.1 and 5.4.2 of "Major Drainage" in the Urban Storm Drainage Manual, Volume 2. In the parameter, V is the mean channel velocity and R is the channel hydraulic radius.

Riprap protection for channel beds on sandy soils is ususally not required if the channel is near its equilibrium slope and the bank protection is extended deep enough to cover the general scour, local scour, and antidune wave height. In some cases, when considering riprap for channel bed protection, the riprap classifications presented in Table 5-1 over an adequate bedding may be used.

5.4.1.4 Riprap Placement

Riprap placement is usually accomplished by dumping directly from trucks; however, care needs to be exercised not to damage the bedding. If riprap

Table 5-1. Riprap Requirements for Channel Bed Linings.

$V^2/R^{0.33}$	Riprap Required
< 60	Type VL *
60 - 90	Type L *
90 - 120	Type M *
120 - 180	Type H
> 180	Type VH *

* It is recommended that these riprap classifications be buried with natural soils, after the riprap has been placed and accepted by the approving agency, to prevent damage by people.

For bank protection see Section 5.4.2 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2.

is placed during construction of the embankment, rocks can be dumped directly from trucks at the top of the embankment. Rock should never be placed by dropping it down the slope in a chute or pushed downhill with a bulldozer. These methods result in segregation of sizes. With dumped riprap, there is a minimum of expensive hand work. Poorly graded riprap with slab-like rocks requires more work to form a compact protective blanket without large holes or pockets. Draglines with orange peel buckets, backhoes, and other power equipment can also be used advantageously to place the riprap.

Hand placed rock is another method of riprap placement. Stones are laid out in more or less definite patterns, usually resulting in a relatively smooth top surface. This form of placement is used rarely in modern practice because it is usually more expensive than the placement with power machinery.

5.4.1.5 Thickness of Riprap

The thickness of riprap should be sufficient to accomodate the largest rock in the riprap material. With a well-graded riprap with minimum voids, a thickness of two times the size of the median size stone ($2\ D_{50}$) should be adequate for sandy soils. The thickness of the riprap should be doubled for the part located below the streambed level. Figure 5-5 indicates a typical channel bank protection with riprap.

5.4.1.6 Filters for Riprap

Filters underneath the riprap are required to protect the fine embankment or river bank material from washing out through the riprap. Two types of filters are commonly used, gravel filters and plastic filter cloths (Figure 5-5).

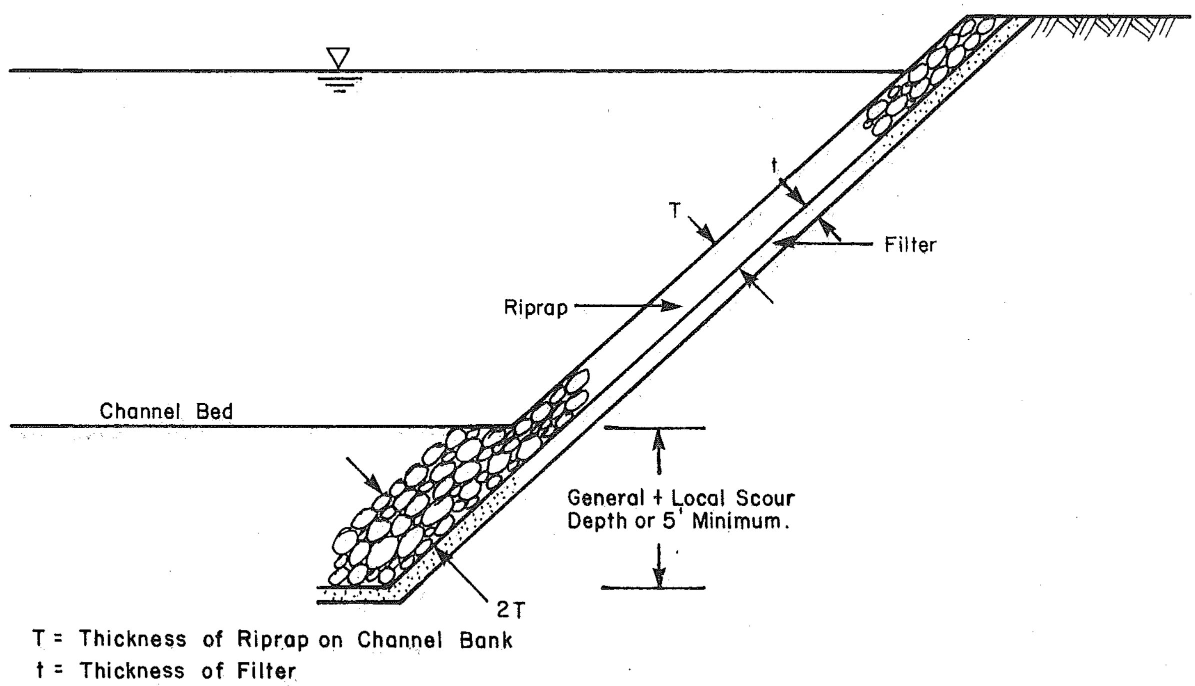


Figure 5-5. Typical channel bank protection with riprap.

5.4.1.6.1 Gravel Filters. A layer or blanket of well-graded gravel should be placed over the embankment or river bank prior to riprap placement. Sizes of gravel as specified in Section 5.3.1 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2 should be used.

5.4.1.6.2 Plastic Filter Cloths. Plastic filter cloths are being used beneath riprap and other revetment materials, such as articulated concrete blocks, with considerable success. The cloths are generally in 100 foot rolls, 12 to 18 feet wide. Overlap of 8 to 12 inches is provided with pins at two to three foot intervals along the seam to prevent separation in case of settlement of the base material. Some amount of care must be exercised in placing riprap over the plasic cloth filters to prevent damange. Experiments and results with various cloth filters were reported by Calhoun, Compton, and Strohm (1971) in which specific manufacturers and brand names are listed. Stones weighing as much as 3000 pounds have been placed on plastic filter cloths with no apparent damage.

Filters can be placed subaqueously by using steel rods as weights fastened along the edges. Additional intermediate weights would assist in sinking the cloth in place. Durability of filter cloths has not yet been established because they have been in use only since around 1967. However, inspections at various installations indicate little or no deterioration had occurred in the few (one to four) years that have elapsed since test installations. For details of plastic filter cloth installation, refer to Section 5.2.3 of "Major Drainage" in the Urban Storm Drainage Manual, Volume 2.

5.4.1.7 Height and Depth of Riprap Protection

The design height of riprap protection above the bed level must provide for freeboard, water depth, superelevation, and wave height. The methods of computing superelevation and antidune wave height were presented in Equation 3-15 and 3-16 respectively. The riprap must also extend some distance below thalweg level to provide safety against possible local socur, general degradation, and troughs of passing sand waves. The methods for estimating scour depths were presented in Section 3.6 and 3.7. In general, the riprap should extend at least five feet below thalweg level in order to protect against possible long-term degradation of the river bed.

5.4.1.8 Roughness Coefficient

The Manning's roughness coefficient (n) for hydraulic computations should range from 0.03 to 0.045 depending on D_{50}. It can be estimated using the criteria presented in Section 5.4.1 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2.

5.4.1.9 Channel Bends

The effects of bends on hydraulic design have been discussed in Section 3.13. In determining the effective shear stress acting on the bank of a bend, the effects of superelevation and uneven velocity distribution should be added to the shear stress computed for a straight reach. The total effective shear stress should be determined using Figure 3-11. The ratio in Figure 3-11 can be applied directly to the parameter $V^2/R^{0.33}$ in Section 5.4.2 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2 to the riprap for bank protection. A design example is given in Section 5.4.2.6.

The riprap protection should be placed along the outside bank and should extend upstream and downstream from the bed at least the length of the bend.

In continuous riprap bank protection, local scour should be expected at the leading portion of the revetment. The equilibrium scour depth should be determined using Equation 3-7.

5.4.2 Design Procedures and Examples

5.4.2.1 Bank Protection Design Procedures

1. Determine design discharge Q.
2. Determine average channel bed slope S.
3. Determine D_{15}, D_{50}, and D_{85} of bank material.
4. Determine the angle of repose ϕ_1 of bank material from Figure 3-9.
5. Select channel side slope $\theta \leq \phi_1 - 5°$.
6. Assume an n value to design a channel cross section or use the field surveyed cross section for bank protection on exsiting channel to compute hydraulic parameters (depth, velocity, hydraulic radius, and etc.).
7. Check the designed cross section for low-flow requirements as specified in Section 2.5.1 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2.
8. Compute the normal flow depth d and mean velocity V using Manning's equation.
9. Size the riprap using criteria presented in Section 5.4.1.3 of this report.
10. Check n value using $n = 0.0395\ D_{50}^{1/6}$ (Section 5.4.1.8).
11. Thickness of riprap layer $T = 2\ D_{50}$.
12. Select filters using criteria in Section 5.4.1.6 of this report.
13. Determine the height of riprap according to Section 5.4.1.7 of this report.
14. Determine the depth of riprap according to Section 5.4.1.7.

5.4.2.2 Bank Protection Design Example

Design a straight channel with riprap bank protection given the following data:

1. Design discharge $Q = 4000$ cfs.

2. Initial runoff discharge $Q_2 = 400$ cfs.

3. $S = 0.003$.

4. The size of bank material $D_{15} = 0.6$ mm,

   $D_{50} = 1.2$ mm,

   $D_{85} = 1.9$ mm.

5. The median size of bed material $D_{15} = 0.9$ mm,

   $D_{50} = 1.8$ mm,

   $D_{85} = 2.9$ mm.

6. The specific gravity of bank material $S_s = 2.65$.

5.4.2.2.1 Solution.

1. $Q = 4000$ cfs.

2. $S = 0.003$.

3. $D_{50} = 1.2$ mm, $D_{15} = 0.6$ mm, $D_{85} = 1.9$ mm.

4. From Figure 3-9, $\phi = 29°$.

5. $\theta = \phi - 5° = 24°$, use side slope 2.5:1, $\theta = 21.8°$.

6. Assume $n = 0.035$ and $V = 8.0$ fps,

$$A = 4000/8.0 = 500 \text{ square feet},$$

$$8.0 = (1.486/0.035)\ R^{2/3}\ (0.003)^{1/2},$$

$$R = 6.38 \text{ feet},$$

$$A = (b + 2.5\ d)\ d = 500,$$

$$R = \frac{500}{b + 2d\sqrt{1 + 6.25}} = 6.38 \text{ feet}$$

From these relations,

$b = 23.05$ feet, use $b = 24.0$ feet.

$d = 10.26$ feet, use $d = 10.2$ feet.

7. Q_2 = 400 cfs.

For,

d = 3.1 feet,	n = 0.035,
A = 98.4 feet^2,	R = 2.42 feet,
Q = 412 cfs,	V = 4.19 fps.

The value for d is greater than one foot and the value for V is greater than two fps.

8. For,

b = 24.0 feet,

d = 10.2 feet,

S = 0.003,

n = 0.035

then,

Q = 4046 cfs,

V = 8.01 fps,

R = 6.40 feet.

9. $V^2/R^{0.33} = (8.01)^2/6.40^{0.33} = 34.77$

From Table 5-5 of Section 5.4.2 of "Major Drainage" of the Urban Storm Drainage Criteria Manual, Volume 2, Type L riprap should be used for 2.5:1 size slope channels. From Table 5-1 of the same manual, D_{50} = 9 inches and D_{100} = 12 inches.

10. Check n value; $n = 0.0395\ (0.75)^{1/6} = 0.037$.

11. Thickness of riprap layer:

T = 2.0 x 12 = 18 inches.

12. Use gravel filters. For filter design, refer to the design example presented in Section 5.3.2 of "Major Drainage" of the Urban Storm Drainage Criteria Manual, Volume 2. The bank material has a medium

diameter D_{50} of 1.2 mm. According to Table 5-4 of the same manual, the bank material is classified as coarse-grained soils. Type II bedding is selected. For Type L riprap, the minimum bedding thickness is six inches.

13. Determine height of riprap.

$$D_{50} = 1.8 \text{ mm},$$

$$\tau_o V = 1.198 \times 8.01 = 9.60.$$

From Figure 2-8, the flow is in the upper regime antidune wave height and can be computed using Equation 3-16.

$$h_a = 0.14 \frac{2\pi V^2}{g} = 0.14 \frac{2\pi \times (8.01)^2}{g} = 1.75 \text{ feet.}$$

Velocity head,

$$h_v = \frac{V^2}{2g} = 1.0 \text{ foot.}$$

Height of riprap above bed level,

$$10.2 + 1.0 + (1.2) \times 1.75 = 12.075 \text{ feet, use } 12.20 \text{ feet.}$$

14. Determine the depth of riprap. According to Figure 3-2, the value a in Equation 3-7 is estimated to be 24 inches (or 2.0 feet).

$$\text{Froude number} = \frac{V}{\sqrt{gR}} = \frac{8.01}{\sqrt{32.2 \times 6.4}} = 0.56$$

From Equation 3-7:

$$Y_s = 1.1(2.25/10.2)^{0.40}(0.56)^{0.33} \times 10.2 = 4.83 \text{ feet.}$$

At the leading portion of the riprap, 4.83 feet of local scour should be expected. The antidune height has been computed to be 1.75 feet in Step 13. The depth of riprap at leading portion:

$$4.83 + (1/2) \times 1.75 = 5.71 \text{ feet, use } 6.0 \text{ feet.}$$

The remainder of the riprap depth equals five feet (minimum depth suggested). Figure 5-6 presents the design cross section and riprap bank protection.

5.4.2.3 Bed Protection Design Procedures

1. Determine design discharge Q.
2. Determine channel bed slope S.
3. Determine D_{15}, D_{50}, and D_{85} of bed material.
4. Determine hydraulic parameters of channel cross section (depth, velocity, hydraulic radius, Froude number, and etc.) by assuming a n value.
5. Size riprap using parameter $V^2/R^{0.33}$ and Table 5-1 of this report.
6. Check n value using $n = 0.0395\, D_{50}^{1/6}$.
7. Thickness of riprap layer $T = 2\, D_{50}$.
8. Select filters using criteria in Section 5.4.1.6.

5.4.2.4 Bed Protection Design Example

Design riprap protection on channel bed for the data given in the example for bank protection.

5.4.2.4.1 Solution.

1. Q = 4000 cfs.
2. S = 0.003.
3. D_{15} = 0.9 mm, D_{50} = 1.8 mm, D_{85} = 2.9 mm.
4. Use the cross section from the previous example

b = 24.0 feet,	R = 6.40 feet,
d = 10.2 feet,	Fr = 0.56.
V = 8.01 fps,	

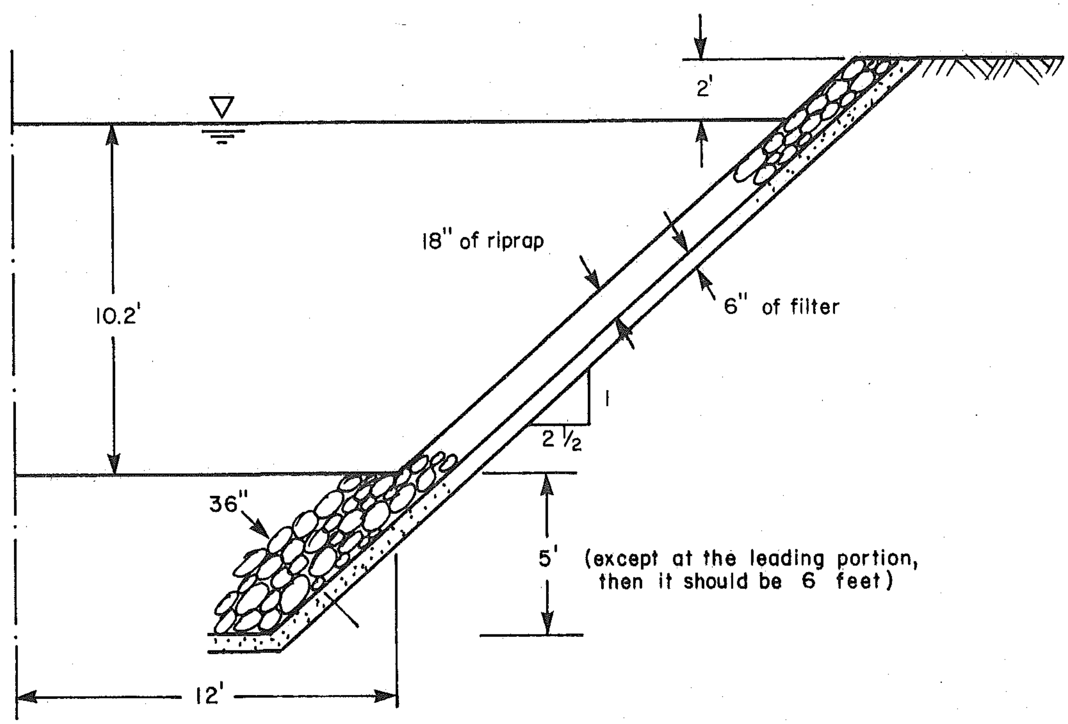


Figure 5-6. Riprap for bank protection.

5. $V^2/R^{0.33} = (8.01)^2/6.40^{0.33} = 34.77$

   From Table 5-1, Type VL riprap should be used. From Table 5-1 of "Major Drainage" of the Urban Storm Drainage Criteria Manual, Volume 2, D_{50} = 6 inches and D_{100} = 9 inches.

6. Check n value; $n = 0.0395(0.5)^{1/6} = 0.035$.

7. Thickness of riprap layer:

   T = 2 x 6 = 12 inches.

8. Use gravel filter. For filter design, refer to the example presented in Section 5.3.2 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2. The bed material has a medium diameter D_{50} of 1.8 mm. According to Table 5-4 of the same manual, the bed material is classified as coarse-grained soils. Type II bedding is selected with a thickness of six inches.

Figure 5-7 presents the designed bed and bank protection.

5.4.2.5 Design Procedures for Riprap Protection at Bends

The design procedures should be the same as the procedures for bank protection on straight channels, except the following:

1. Figure 3-10 should be used to adjust the riprap type on the outside bend.

2. Freeboard should take superelevation into account.

5.4.2.6 Design Example for Riprap Protection at Bends

Given the same flow conditions and channel geometry as the design example in 5.4.2.4, design riprap protection at outside bend with a radius to the channel centerline of 300 feet.

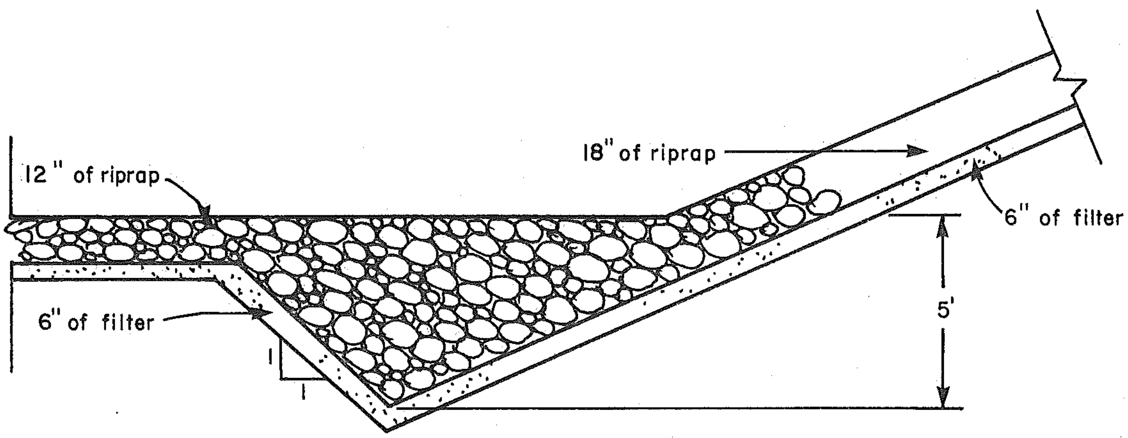


Figure 5-7. Bank and bed riprap protection.

5.4.2.6.1 Solution.

1. Top width of the channel:

$$W = b + 2 \times 2.5d = 24 + 5 \times 10.2 = 75 \text{ feet.}$$

2. $r_c/W = 300/75 = 4.0$.

3. From Figure 3-10:

$$\tau_{ob}/\tau_o = 1.69.$$

4. From previous example,

$$V^2/R^{0.33} = 34.77.$$

5. The parameter $V^2/R^{0.33}$ at bend is equal to:

$$1.69 \times 34.77 = 58.76 < 70.$$

From Table 5-5 of Section 5.4.2 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2, Type L riprap should be used.

6. Riprap size, thickness, and filter requirements are the same as the example given in 5.4.2.2 of this report.

7. Superelevation, use Equation 3-15:

$$\Delta Z = \frac{V^2}{gr_c} W = \frac{(8.01)^2}{32.2 \times 300} \times 75 = 0.5 \text{ feet.}$$

The height of riprap protection at the outside bend should be 0.5 feet higher than the straight reach presented in Section 5.4.2.2.

8. Determine the riprap depth using the same procedures as presented in the example in Section 5.4.2.2.

5.5 Drop Structures

The use of channel drops permits adjustment of a thalweg which is too steep for the design conditions. The structures can be either vertical drops or sloped drops. The materials can be sheet piling, concrete walls and footings, or riprap. In urban drainage work it is often desirable to use more low head drops in lieu of a few higher drops.

In most cases, additional bank and bottom protection will be needed after the first runoff or two when erosional tendencies are field tested. For this reason, the engineer should allow in his estimates for monies to be spent during the first two years following construction completion.

Most common failures of drop structures on sandy soils are due to undermining, piping, or flow bypassing the structure. The important factors to be considered in designing drop structures are: (1) head differential; (2) seepage flow; (3) channel degradation; (4) lateral migration; and (5) local scour.

5.5.1 Design Criteria

5.5.1.1 Site Selection

The structure should be located in a reasonably straight section of channel with neither upstream or downstream curves within 100 to 200 feet of the structure. The site selected must provide an adequate foundation for the structure. The foundation material must have the required supporting strength, resistance to sliding, and be reasonably homogenous so as to prevent differential or uneven settlement of the structure.

5.5.1.2 Type of Structure

The selection of the type of drop structure depends mainly on the hydraulic, morphologic, and soil characteristics of the channel, as well as the degree of protection against damage from storm runoff. Of course, political, environmental, and economic factors always play an important role in the selection; however, the selection of a drop structure type should also consider the range of discharges, flow depths and velocities, head differentials, soil size and its characteristics, sediment transport and storage, seepage forces, uplift forces, and channel flow regime. The types of drop structures can be classified as vertical drop structures and sloped drop structures. Typical drop structures are presented in Tables 5-2 and 5-3. Drop structures using gabions are not recommended due to fast deterioration.

5.5.1.3 Width of Structure

The width of drop structures depends mainly on the design discharge. The width should be great enough to permit discharge into the downstream channel near regime conditions. In this regard, the minimum width of a structural concrete drop structure was suggested as

$$W = 1.0 \sqrt{Q} \tag{5-4}$$

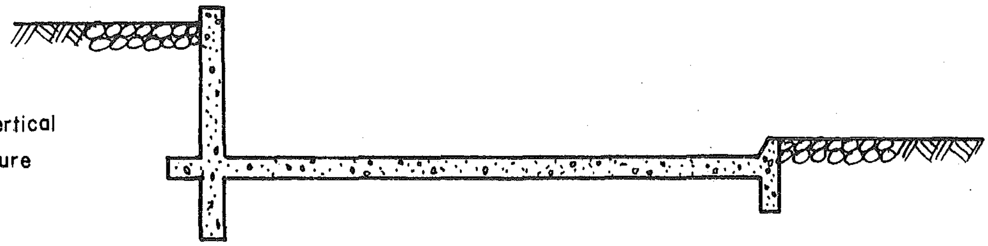
or the unit discharge should not be more than 100 cfs per foot. For riprap drop structures, the unit discharge should not be more than 35 cfs per foot.

5.5.1.4 Drop Height

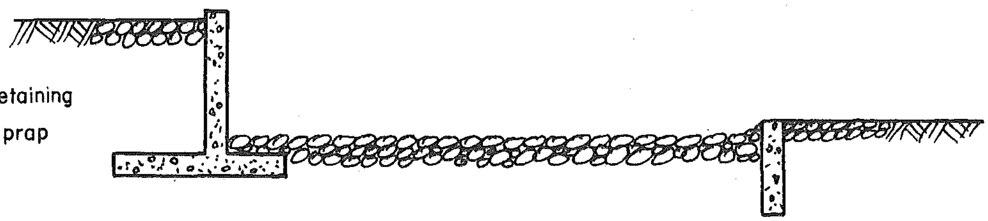
The height of drop structures is usually governed by the available construction material, required structural stability, and cost. Small structures formed of riprap are usually most economical; however, if large rock is required, it is both expensive and difficult to find. Riprap drop structures should not be utilized if drop heights exceed four feet. Vertical drop structures are usually limited to eight feet because of the difficult stability problem and expensive construction associated with high vertical retaining walls. Grouted riprap drop structures can be used for drop heights of more than four feet, however, the differential hydrostatic pressure at the face may cause the grout to crack. Hence, grouted riprap must be provided with a well-designed underdrain system.

Table 5-2. Type of Vertical Drop Structures.

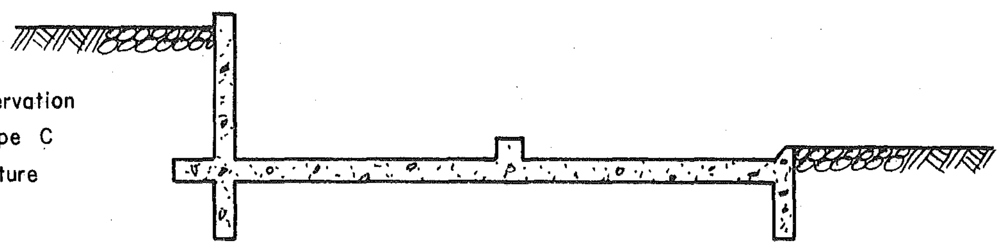
V-1. Concrete vertical drop structure



V-2. Concrete retaining wall with riprap



V-3. Soil Conservation Service Type C drop structure



V-4. Vertical drops with preshaped riprap basin

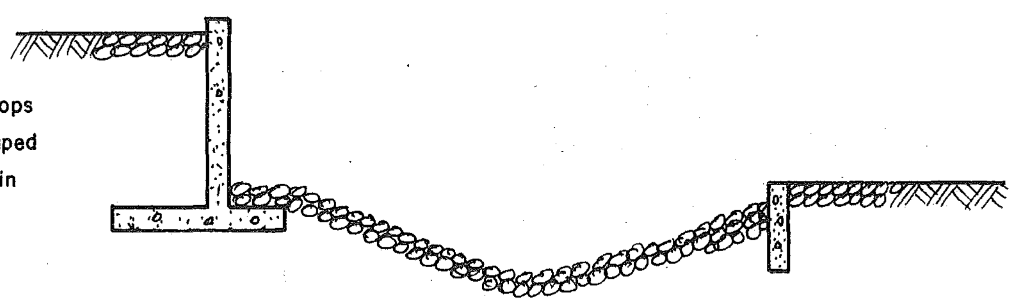
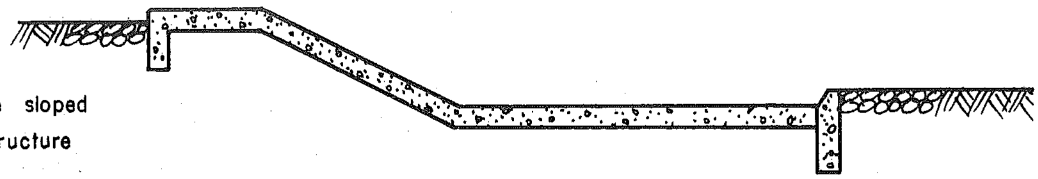
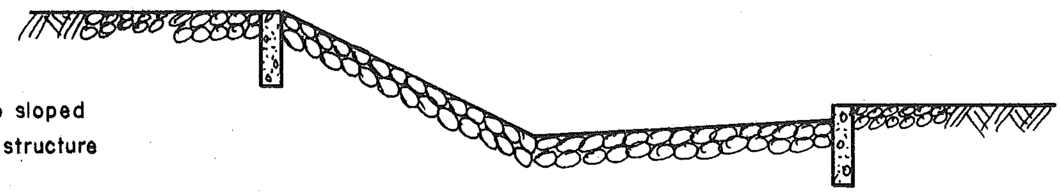


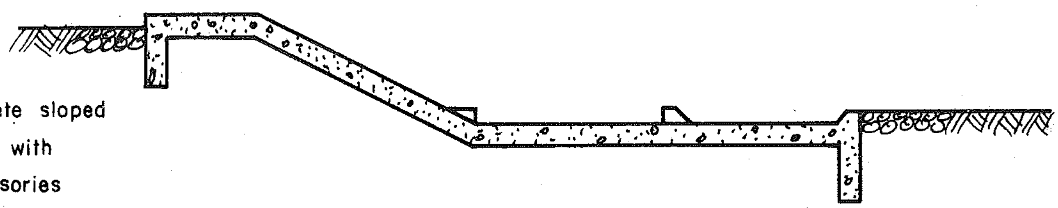
Table 5-3. Type of Sloped Drop Structures.



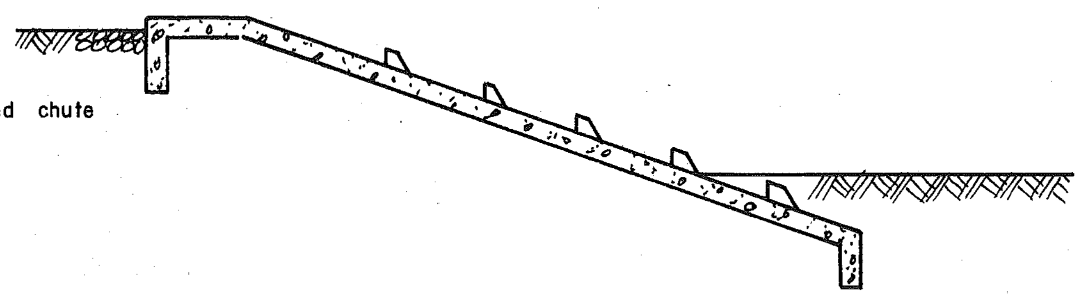
S-1. Concrete sloped drop structure



S-2. Riprap sloped drop structure



S-3. Concrete sloped drops with accessories



S-4. Baffled chute drops

5.5.1.5 Upstream Flow Conditions

If possible, the flow conditions upstream of drop structures should be in the lower regime because upper regime flow conditions result in higher approach velocities, standing waves, and possibly antidunes that vary with time. The flow regime can be estimated by knowing stream power $(\tau_o V)$ and the median fall diameter of bed sediment. The regimes of flow in alluvial channels have been discussed in Section 2.4.3.

5.5.1.6 Upstream Channel

The channel upstream of the drop structure should be reasonably straight. Erosion potential should be analyzed and steps should be taken to protect against it. Channel response to drop structures should be anticipated and proper control measures should be provided.

When constructing a rectangular drop in a trapezoidal channel, it is necessary to provide a short transition to limit the high velocities at the approach to the crest and establish essentially two-dimensional flow. An inlet length of two times the approaching head is recommended. For drop structures, a draw down water surface profile upstream will result in higher velocities in the approach channel. Therefore, increased erosion potential may require local protection of the channel bed and banks.

5.5.1.7 Downstream Channel

The downstream channel should be reasonably straight. Local scour at the toe of the structures should be analyzed and steps must be provided to control it.

If the downstream channel is near its equilibrium slope, erosion immediately below the structure due to the turbulence and surging action of the hydraulic jump can be controlled using riprap or other protective material.

A protection length of four times the jump depth is usually satisfactory. In some cases, the downstream channel may experience degrading due to channelization or a reduction of the incoming sediment supply. The stilling basin must be designed to accomodate the expected changes in channel bed elevation. However, the final stable channel bed elevation is difficult to predict, so a reasonable factor of safety must be selected. The baffle chute drop structure is recommended for this situation because a stilling basin is not required and the chute can be extended as required to compensate for unexpected changes in channel bed elevation.

5.5.1.8 Upstream Cutoff

An adequate cutoff is important to prevent piping, reduce the seepage force, and to reduce the danger of uplift pressure under the structure. The depth of the upstream cutoff can be calculated by applying Lane's weighted-creep ratio as described in Section 3.10. The upstream cutoff should extend to the channel bank and usually further, depending on the porosity and erodibility of the foundation material.

5.5.1.9 Downstream Cutoff

The upstream cutoff has been referred to as a "seepage" cutoff while the downstream one is considered the "erosion" cutoff. With adequate attention to energy dissipation, a deep erosion cutoff is not required. The downstream cutoff is needed to reduce the hydrostatic uplift under the stilling basin and to prevent undermining of the structure. The wingwalls, if used, also must provide an adequate degree of cutoff protection under the structure. In the absence of wingwalls, the downstream cutoff must extend around the sides of the basin and partly up the slope across the end. In erodible material, this cutoff should extend to a depth of $0.5d_2$ (d_2 = sequent depth).

5.5.1.10 Use of Riprap

Riprap can be used to help protect a drop structure. For use of riprap at inlet channels and protection of channels below the stilling basin, the design principles and procedures in Section 5.4, for riprap, can be applied. If riprap is used to form the stilling basin floor, the riprap should be grouted or preshaped to form a plunge pool. Flood control channels in urbanized areas are usually dry. If the drop structures have a plunge pool, the pool should not be lined with waterproof plastic cloth, but should be lined with filter cloth to prevent erosion of the material supporting the riprap.

5.5.1.11 Lateral Migration

Lateral migration occurs in sand-bed channels as described in Section 3.9. Protection of drop structures can be achieved in two ways: (1) prevent channel bends from migrating downstream (i.e., provide bank protection at the bend upstream of the structure); and (2) channelization with bank protection so that meandering will not occur near the structure.

5.5.1.12 Width of the Protection

The width of the protection here is defined as the total width of the drop structure including the drop weir length and protection needed on the overbank areas in case of overflow. Designing a drop structure in a natural channel, the channel capacity may be less than the design life of the drop structure. If the drop is designed only for the natural channel width, some flow may bypass the structure and possibly cause failure of the structure by eroding the material on the overbank areas paralleling the structure. Riprap can be used for the purpose of overbank area protection. The total width of a drop structure should be designed for the 100-year flood with overbank protection extended beyond the limits where the velocity of overbank flow is less than an acceptable value. The acceptable velocity is recommended as 3.5 fps on unlined sandy soils as specified in Section 2.3.1A of "Major Drainage" of the Urban Storm Drainage Criteria Manual, Volume 2. However, a minimum width of ten feet for overbank area protection is recommended. Figure 5-8 illustrates the necessary total width of a drop structure considering the foregoing.

As shown in Figure 5-8, Section B-B, the overbank area will be sloped in the flow direction following the channel bank. The slope of the channel bank should be used as the design slope of the overbank protection using the maximum flow depth in the overbank area. The design procedures should be the same as the design of drop structure for the main channel.

5.5.2 Design Standard

5.5.2.1 Vertical Drop Structures

Vertical drop structures generally consist of a vertical retaining wall followed by a stilling basin immediately downstream. The vertical retaining wall can be of concrete supported by a footing or sheet piling. The stilling basin can be formed of concrete slab or riprap. The concrete should be reinforced.

Vertical drop structures are acceptable if:

1. The length of the structure is limited by physical conditions.
2. The drop height is relatively low (up to eight feet). The use of a vertical drop is limited by the stability problem and expense associated with the high vertical retaining walls.

A typical vertical drop structure is shown in Figure 5-9. The hydraulic design for a vertical drop structure with vertical side walls is presented in the following sections.

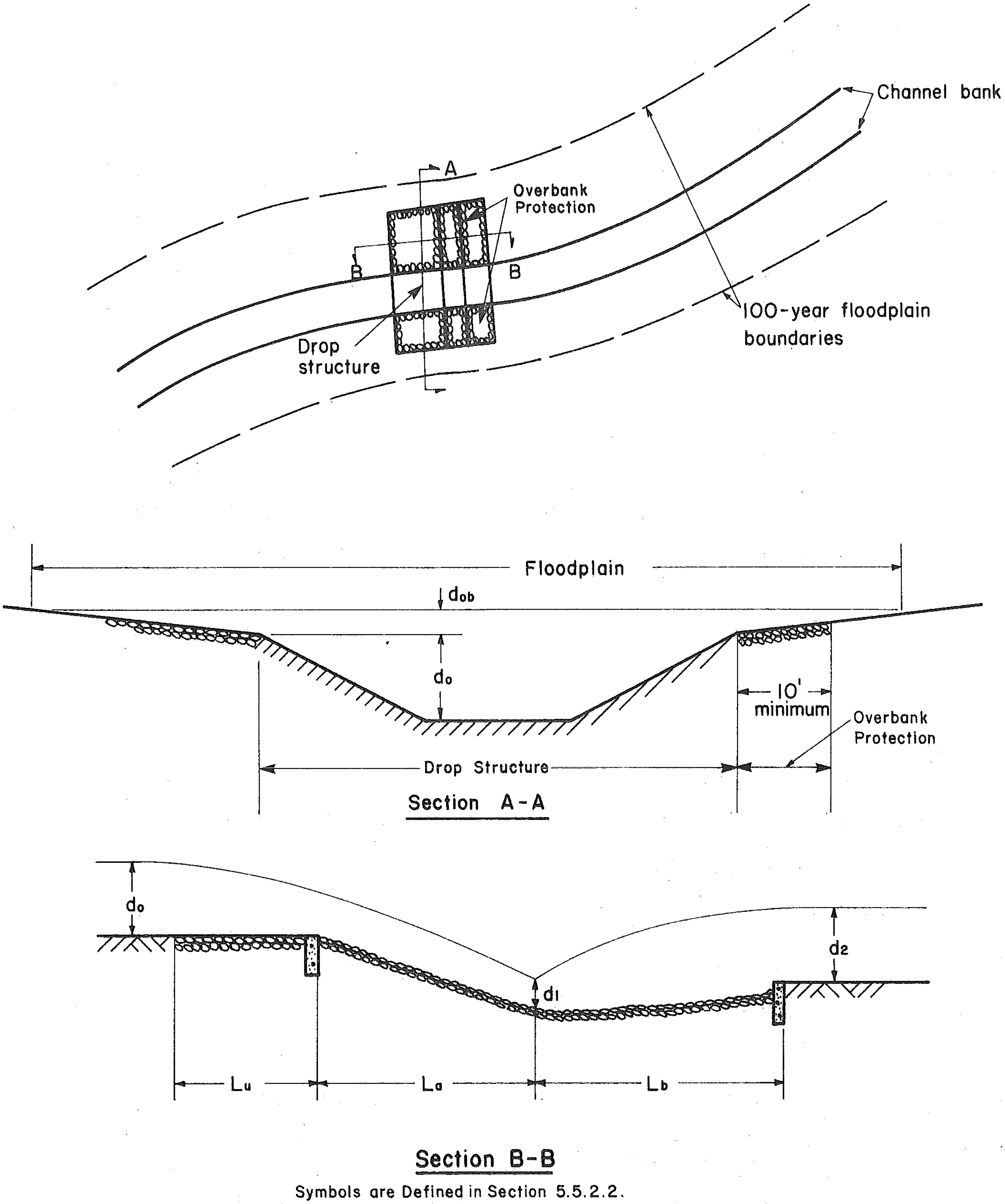


Figure 5-8. Width of a drop structure.

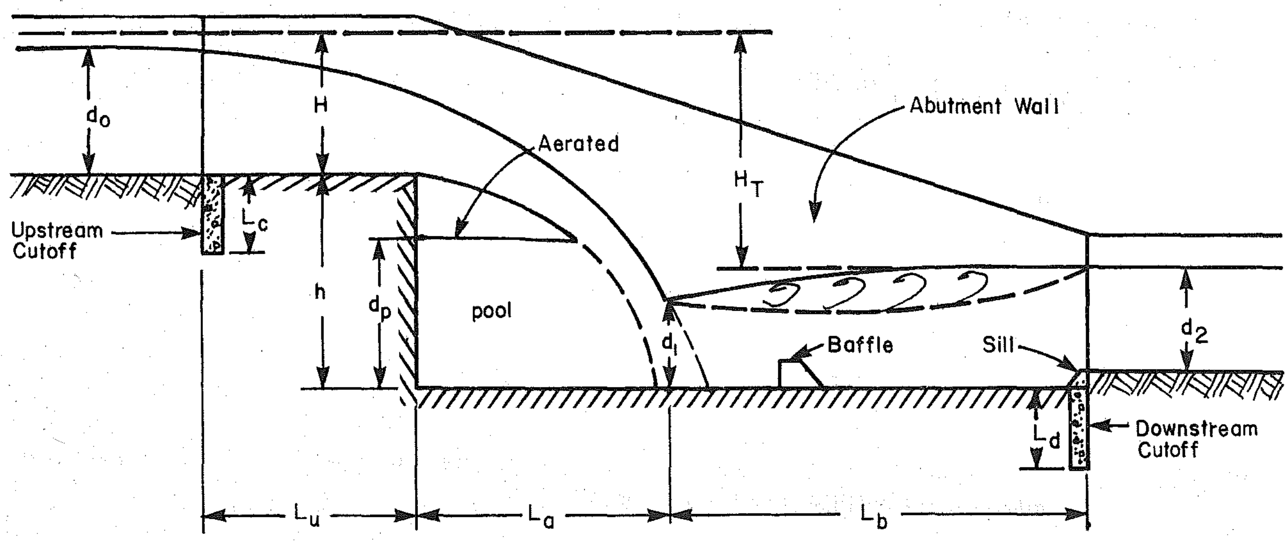


Figure 5-9. Definition sketch for a vertical drop structure with vertical side walls.

5.5.2.2 Structure Dimensions

The flow geometry at vertical drop structures can be described as functions of the drop number, which is defined as

$$D_n = q^2/gh^3 \qquad (5\text{-}5)$$

where, q = the discharge per unit width of crest of overfall,

g = the acceleration of gravity, and

h = the height of the drop.

The functions are:

$$L_a/h = 4.30\ D_n^{0.27} \qquad (5\text{-}6)$$

$$d_p/h = 1.00\ D_n^{0.22} \qquad (5\text{-}7)$$

$$d_1/h = 0.54\ D_n^{0.425} \qquad (5\text{-}8)$$

$$d_2/h = 1.66\ D_n^{0.27} \qquad (5\text{-}9)$$

where, L_a = the drop length (the distance from the drop wall to the position where the nappe strikes the floor),

d_p = the pool depth under the nappe,

d_1 = the depth at the toe of the nappe or the beginning of the hydraulic jump,

d_2 = the tailwater depth sequent to d_1.

An inlet length is recommended as follows:

$$L_u = 2\ H \qquad (5\text{-}10)$$

where, L_u = the inlet length, and

H = the specific energy at the drop.

The length of the stilling basin must be extended at least one jump length beyond where the nappe strikes the floor, which is considered as the beginning of the jump. The jump length can be determined using Figure 2-4 of "Structures" in the Urban Storm Drainage Criteria Manual, Volume 2, for Froude numbers ranging from two to five. For Froude numbers larger than five, $L_b/d_2 = 6$ is recommended.

5.5.2.3 Aeration

A vertical drop must be aerated, otherwise a clinging nappe will result. Model tests show that the non-aerated vertical drop can sometimes result in unsteady flow and that the discharge coefficient has a wide range of variation. The nappe can be aerated by aeration holes or with a simple end contraction on the abutment wall at each end of the drop. The area of aeration holes required can be estimated by the following equation:

$$\frac{A}{L} = 5.3 \times 10^{-4} \left(\frac{H^{3.64}}{p^{1.64}} \right) \qquad (5\text{-}11)$$

where, A = required area of aeration hole or holes, inches^2,

L = length of drop weir, feet,

H = specific energy at the drop, feet,

p = differential pressure between atmosphere and pressure under the nappe, feet.

The drop should be aerated through both sidewalls. The minimum recommended diameter of aeration hole is six inches. The aeration hole should be place above d_p as defined in Figure 5-9.

The aeration will also be adequate if the end contraction on the abutment wall y projects into the flow by an amount

$$y = (0.0008\ LH)^{1/2} \qquad (5\text{-}12)$$

where, y = the end contraction on both sides of the abutment wall, feet,

L = the length of drop weir, feet,

H = specific energy at the drop, feet.

5.5.2.4 Stilling Basin Elevation

The proper elevation setting of the stilling basin floor is of greater importance than the length of the basin. The floor of the stilling basin must be set a sufficient depth below the minimum tailwater to confine the hydraulic jump to the basin at all times. If the tailwater depth D_2 is less than the sequent depth d_2, the hydraulic jump will form downstream of the basin. Therefore, the floor must be set either the sequent depth d_2 or the tailwater depth D_2 below the downstream water level, whichever is greater. A depression of one-quarter of the flow depth below the downstream channel bed at the stilling basin is common.

5.5.2.5 Stilling Basin Baffles and Sills

It is possible to reduce the jump length by the installation of accessories, such as baffles and sills, in the stilling basin. When an end sill is used, the jump length (L_b in Figure 5-9) can be reduced to $4d_2$. When both baffles and end sills are used, the jump length can be further reduced to $3d_2$.

For detailed design of baffles and sills, refer to Hydraulic Design of Stilling Basins and Energy Dissipators, U.S. Bureau of Reclamation.

5.5.2.6 Sloped Drop Structures

The use of sloped drops will generally result in lower cost installations. Sloped drops can be designed to fit the channel topography needs with little difficulty. The components of the sloped drop structure are basically the same as the vertical drop structure as described in Section 5.5.2.2. For both structures, there must be some means of controlling the flow at the top and dissipating the energy at the bottom of the structure. A definition sketch for a sloped drop structure is shown in Figure 5-10. A few of the differences are discussed as follows:

1. The aeration holes are not needed.
2. Sloped drop structures should have faces not steeper than 2:1.
3. Concrete sloped drop structures can be used for higher drops than vertical drop structures. Although, more structure length is required, this is more than compensated for by the reduction in vertical wall height.
4. The length of the structure should be

$$L = L_u + L_a + L_b \tag{5-13}$$

where, $L_u = 2H$,

$L_a = Zh$,

L_b = the hydraulic jump length,

H = the specific energy at the drop,

h = the height of the drop.

5. Chute blocks, baffles, and end sills may be added at concrete sloped drop structures to reduce the length of the structure. For the design of stilling basins with chute blocks, baffles, and end sills, refer to <u>Hydraulic Design of Stilling Basins and Energy Dissipators</u>, U.S. Bureau of Reclamation.

5.5.2.7 <u>Design of Drop Structures</u>

The principles of hydraulic design of drop structures are the same for all drop structures. Special features of different drop structures (see Tables 5-2 and 5-3) are presented in the following sections.

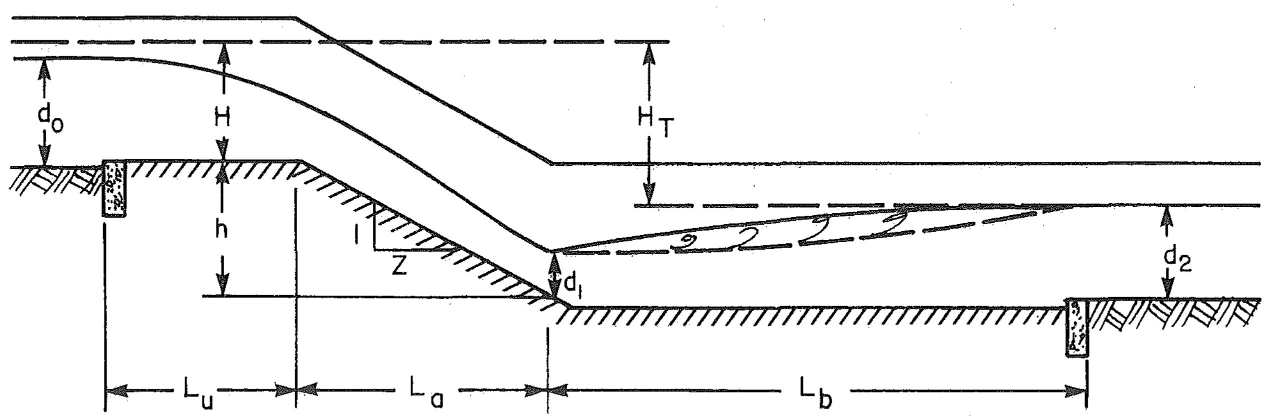


Figure 5-10. Definition sketch for a sloped drop structure.

5.5.2.8 Type V-3, Soil Conservation Service Type C Drop Spillway

The Type C drop spillways were developed utilizing theory supported by model studies as presented in the St. Anthony Falls Hydraulic Laboratory, Technical Paper No. 15, Series B, "Straight Drop Spillway Stilling Basin," by Charles A. Donnelley and Fred W. Blasidell. The design of Type C drop spillways should conform to the limitations of the tested range. Structural configuration, design limitations, design procedures, and design examples can be found in The Engineering Handbook, Section 11 by the U.S. Soil Conservation Service.

5.5.2.9 Type V-4, Vertical Drops with Preshaped Riprap Basin

The riprap forming the stilling basin floor is preshaped to form a plunge pool. The depth of the plunge pool can be estimated using Equation 3-10.

Design should conform to dimensions obtained by model testing of the pool because of the serious problems which could occur with an improperly designed pool.

5.5.2.10 Type S-2, Riprap Sloped Drop Structure

The area in the vicinity of the drop crest is the most critical area for stabilizing the riprap. A crest wall is required and should be placed at an elevation higher than the upstream channel bed to ensure uniform flow distribution over the drop crest and to increase the stability of the drop crest. The riprap below the crest wall should be thicker than the riprap at the slope face. The thickness of riprap below the crest wall depends on drop height and riprap size. The crest wall should extend to a minimum depth of the thickness of riprap plus filter. Figure 5-11 shows the crest wall on a riprap sloped drop structure.

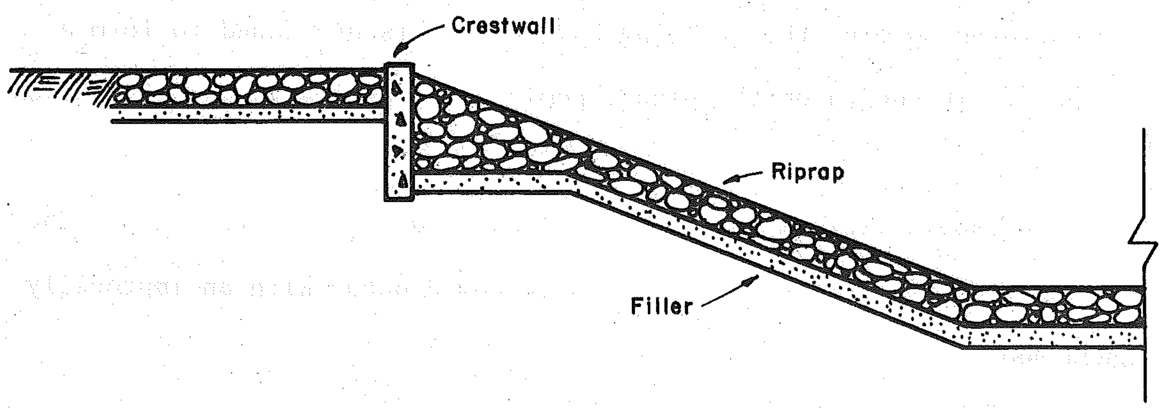


Figure 5-11. Crest wall on riprap drop structures.

5.5.3 Design Example - Sloped Drop Structure

Design a sloped drop structure in a natural or grass lined channel given the following data:

1. Channel bank full capacity $Q = 950$ cfs.
2. Channel geometry:

   Bottom width $b = 20$ feet,

   Channel depth $d_o = 5$ feet,

   Side slope $= 4:1$.
3. Design life of the drop structure is 100 years, $Q_{100} = 1800$ cfs.

   Floodplain width = 100 feet;

   Flood depth = 7 feet.
4. Drop height equals four feet.
5. Downstream flow depth equals five feet.
6. Bed material: $D_{15} = 0.09$ mm, $D_{50} = 0.18$ mm, $D_{85} = 0.29$ mm.

   Bank material: $D_{15} = 0.06$ mm, $D_{50} = 0.12$ mm, $D_{85} = 0.19$ mm.
7. Channel cross section is shown in Figure 5-12.

5.5.3.1 Solution

1. Use trapezoidal lined drop. The cross section is the same as the channel cross section.
2. Use 4:1 slope at drop. Assume no head loss through the sloped drop.

   $$V = 950/(20 \times 5 + 20 \times 5) = 4.75 \text{ fps},$$

   $$H = d_o + \frac{V^2}{2g} = 5 + \frac{(4.75)^2}{2g} = 5.35 \text{ feet}.$$

   Head available at toe of the sloped drop is:

   $$H_T = 4 + 5.35 = 9.35 \text{ feet}.$$

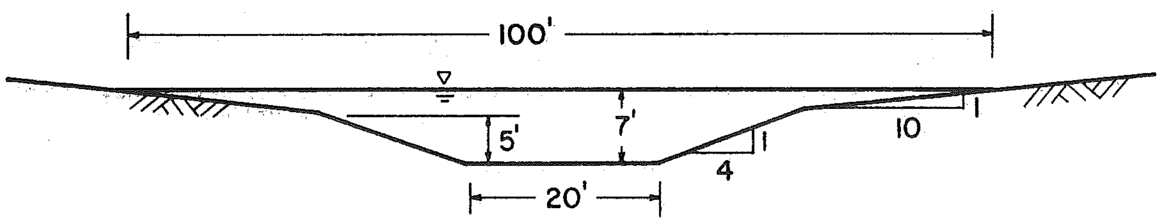


Figure 5-12. Cross section of natural channel.

Let d_1 be the depth at the toe of the slope, solving $H_T = d_1 + (V_1^2/2g)$ by trial and error.

$d_1 = 1.61$ feet;

$V_1 = 22.32$ fps.

Froude number at the toe of the sloped drop is:

$$Fr = \frac{V_1}{\sqrt{gd_1}} = \frac{22.32}{\sqrt{1.61g}} = 3.10$$

3. Determine d_2:

$$\frac{d_2}{d_1} = \frac{1}{2} (\sqrt{1 + 8 \ Fr^2} - 1) = 3.91,$$

$d_2 = 3.91 \times 1.61 = 6.30$ feet.

4. Determine jump length. From Figure 2-4 of "Structures" in the <u>Urban Storm Drainage Criteria Manual, Volume 2</u>:

$$Fr = 3.10; \quad \frac{L_b}{d_2} = 5.15;$$

$L_b = 5.15 \times 6.30 = 32.45$ feet, use 32 feet-6 inches.

5. The basin should be set at 6.30 - 5 = 1.30 (use 1 foot-4 inches) below the downstream channel bed. The basin length can be further checked for jump location to determine the exact length.
6. $L_a = 4 \times 5.33 = 21.32$ feet, use 21 feet-4 inches.
7. $L_u = 2H = 2 \times 5.35 = 10.70$ feet, use 10 feet-9 inches.
8. Drop structure dimensions are presented in Figure 5-13.
9. The structure can be either concrete or riprap. For the purpose of illustration, concrete lined drop is used. The structural design of the structure is not included in the example. The example only considers the stability of the structure from the erosion point of view.

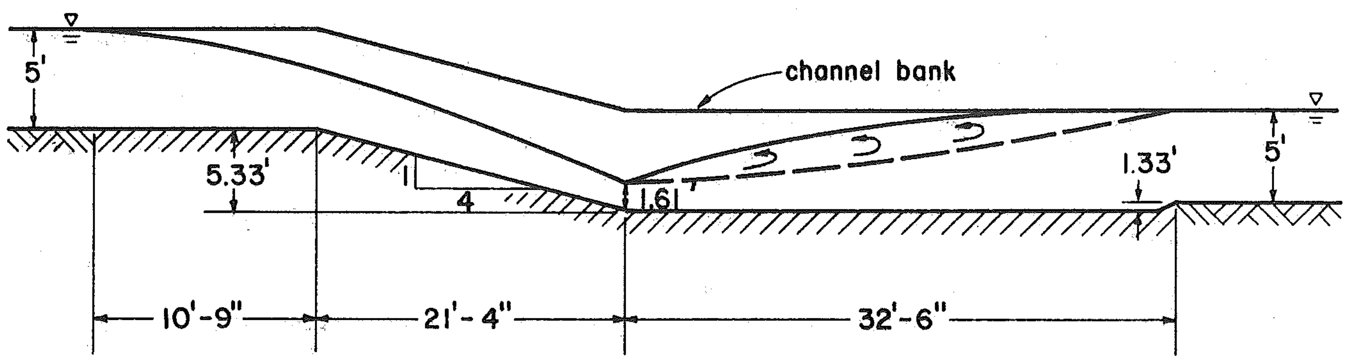


Figure 5-13. Design example - main channel.

10. Compute cutoff requirement using Lane's theory of weighted-creep ratio as described in Section 3.10. For $D_{50} = 0.18$ mm, from Table 3-2, the bed material is classified as fine sand. The weighted-creep ratio $C_w = 7.0$. Since the structure is concrete, for preventing piping, the critical point is located at the downstream end of the structure. The head difference will be:

$$5.35 + (5.33 - 6.33) = 4.35 \text{ feet.}$$

$$\Sigma L_H = 10.75 + 21.33 + 32.50 = 64.58 \text{ feet;} \quad \Sigma L_V = 0;$$

$$C_w = \frac{\Sigma L_H + 3\Sigma L_V}{3H} = \frac{64.58}{3 \times 4.35} = 4.95 < 7.0.$$

Let,

$$\frac{64.58 + 3\Sigma L_V}{3 \times 4.35} = 7.0; \quad \Sigma L_V = 8.92 \text{ feet.}$$

Let cutoff walls be at upstream and downstream and be the same depth as a starting assumption. Cutoff wall depth is:

$$(1/4)\ \Sigma L_V = 2.23 \text{ feet.}$$

Cutoff wall depth at downstream also must be:

$(1/2)\ d_2 = 3.15$ feet, therefore, use 3 feet-2 inches at downstream.

This satisfies the purpose for scour protection downstream. However, riprap protection at downstream of cutoff to a length of $2d_2$ is recommended. Then, the minimum depth of the upstream cutoff wall is

$(1/2)(8.92 - 2 \times 3.17) = 1.29$ feet, use 1 feet-4 inches.

11. Check the design with 100-year flood, $Q_{100} = 1800$ cfs, following steps 1 through 7.

$$V = 1800/(200 + 160) = 5.00 \text{ fps.}$$

$$H = d + \frac{V^2}{2g} = 7 + \frac{(5.00)^2}{2g} = 7.39 \text{ feet.}$$

Head available at toe of the sloped drop is:

$$H_T = 4 + 7.39 = 11.39 \text{ feet.}$$

Let d_1 be the depth at the toe of the slope, solving $H_T = d_1 = (V_1^2/2g)$ by trial and error.

$$d_1 = 2.51 \text{ feet;}$$

$$V_1 = 23.87 \text{ fps;}$$

$$Fr = \frac{V_1}{\sqrt{gd_1}} = \frac{23.87}{\sqrt{2.51g}} = 2.66;$$

$$\frac{d_2}{d_1} = \frac{1}{2}\left(\sqrt{1 + 8\ Fr^2} - 1\right) = 3.29;$$

$$d_2 = 3.29 \times 2.51 = 8.26 \text{ feet;}$$

$$Fr = 2.66;$$

$$\frac{L_b}{d_2} = 4.95;$$

$$L_b = 4.95 \times 8.26 = 40.89 \text{ feet, use 41 feet.}$$

The basin should be set at 8.26 - 7 = 1.26 (use 1 foot-4 inches) below the downstream channel bed.

$$L_a = 4 \times 5.33 = 21 \text{ feet-4 inches;}$$

$$L_u = 2H = 2 \times 7.39 = 14.78, \text{ use 14 feet-10 inches.}$$

Drop structure dimensions are presented in Figure 5-14.

Compute cutoff requirement. The head difference will be:

$$7.39 + (5.33 - 8.33) = 4.39 \text{ feet;}$$

$$\Sigma L_H = 14.00 + 21.33 + 41.00 = 76.33 \text{ feet;}$$

$$\Sigma L_V = 0;$$

$$C_w = \frac{\Sigma L_H + 3\Sigma L_V}{3H} = \frac{76.33}{3 \times 4.39} = 5.80 < 7.0.$$

Let,

$$\frac{76.33 + 3\Sigma L_v}{3 \times 4.39} = 7.0; \quad \Sigma L_v = 5.29 \text{ feet.}$$

Cutoff wall depth required at downstream is:

$(1/2)\ d_2 = 4.13$ feet, use 4 feet-2 inches.

This satisfies the purpose for scour protection downstream. However, riprap protection at downstream of cutoff to a length of $2d_2$ is recommended. Since the vertical contact distance L_v required for $Q = 950$ cfs is greater than the requirement for $Q = 1800$ cfs (Step 10), the minimum depth of the upstream cutoff wall is:

$(1/2)(8.92 - 2 \times 4.17) = 0.29$ feet, use 1 foot.

12. Determine width of protection. From Figure 5-12 for $Q = 1800$ cfs and $n = 0.03$,

$A = 360$ feet^2

$R = 3.56$ feet.

The corresponding slope from Manning's equation is $S = 0.0019$. The allowable velocity is 2.5 fps as specified in Section 5.5.1.12. Using the energy slope $(S = 0.0019)$ and velocity of 2.5 fps, solve for flow depth:

$$V = 2.5 = \frac{1.486}{0.030}\, d^{2/3}\, (0.0019)^{1/2}$$

$d = 1.25$ feet.

From Figure 5-12, the width of overbank area which needs protection is:

$10\ (2 - 1.25) = 7.5$ feet, use minimum width of 10 feet.

Riprap for overbank area protection can be determined using the design standard in Section 5.4 and Section 5.5.2.10.

Figure 5-15 presents the design of this example.

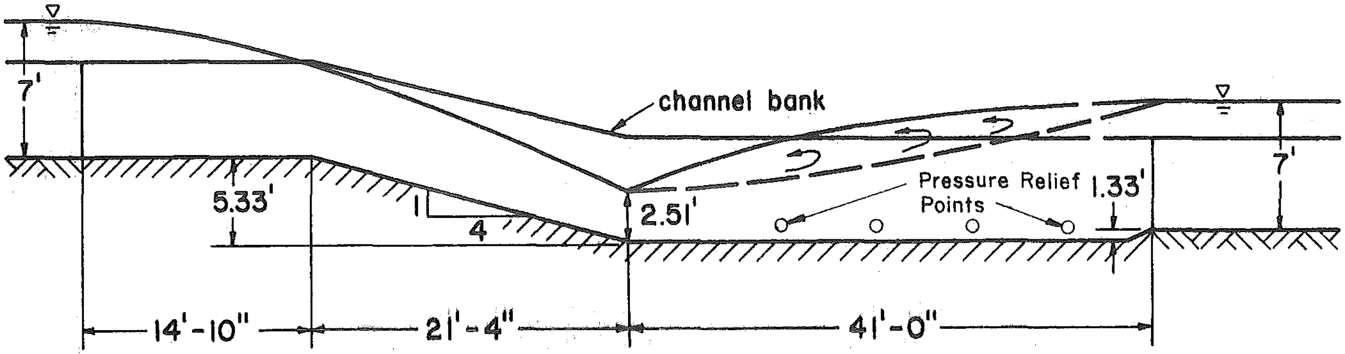


Figure 5-14. Design example - overbank area.

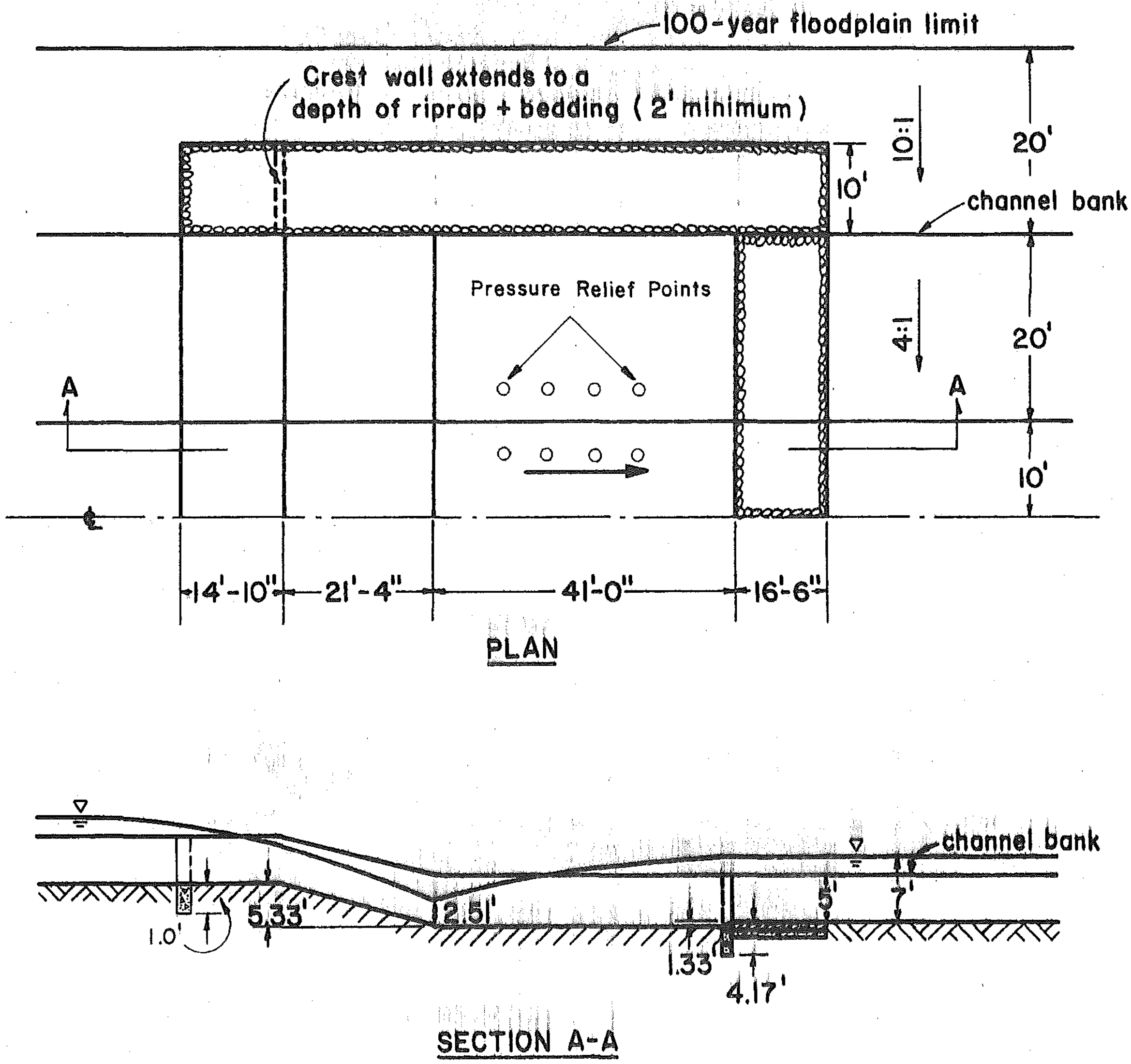


Figure 5-15. Design example - sloped riprap drop structure.

5.6 Pipe and Culvert Entrance

Design considerations and design criteria for pipe and culvert entrances are discussed in "Inlets and Culverts" of the Urban Storm Drainage Criteria Manual, Volume 2. Most common failures of pipe and culverts are: (1) grade wash out resulting from inadequate pipe or culvert capacity and consequent overtopping of the grade; (2) uplift failure at the inlet may occur for a lightweight pipe if there is inlet control. Since it only flows partly full for inlet control, the weight of the pipe and the water in it may be less than the buoyant force acting on the submerged pipe; and (3) pipe or culvert failure resulting from erosion at the entrance and consequent undermining of the pipe or culvert bedding. Channel responses should be investigated, such as degradation, aggradation, channelization, and urbanization, for pipe and culvert design. Maintenance is relatively important for pipe and culvert entrances. Debris and sediment should be removed after each storm.

Piping is not common in pipe or culvert installations because the heads involved are usually low and they are not maintained for long periods. However, if the backfill around the pipe or culvert is not carefully placed and compacted, there may be void spaces along the barrel which will permit a large rate of seepage flow.

5.6.1 Design Criteria

The design of pipe and culvert entrances should follow the criteria presented in the Urban Storm Drainage Criteria Manual, Volume 2 with the following additional considerations for erosion and sedimentation on sandy soils.

5.6.1.1 Type of Entrance

Pipe or culvert entrances shall be designed to minimize entrance and friction losses. Entrances shall also be designed to prevent pipe or culvert failure due to erosion and sedimentation. Entrances with headwalls are recommended for pipe or culvert installations on sandy soils.

5.6.1.2 Headwall

Headwalls are used for increasing the efficiency of the entrance, providing embankment stability, and providing embankment protection against erosion. The height of headwalls should be extended to the embankment slope and the embankment above the headwall should be protected against erosion to the level of the water surface plus freeboard for wave action.

5.6.1.3 Wingwalls

Wingwalls may be provided at pipe or culvert entrances to protect the embankment against erosion. The length of wingwalls is determined by embankment slope. The depth of wingwalls should extend below the expected scour depth as described in Section 3.6 and 3.7.

5.6.1.4 Headwater

The headwater shall not cause any excessive ponding and sediment deposition upstream of the entrance. The ponding will be likely to cause pipe or culvert clogging, saturation of fills, and deterimental upstream deposits of debris. Unless the culvert is used as a detention pond outlet, the headwater shall be at the lower elevation of: (1) flow velocity in the approach channel shall not be less than 2.5 feet per second; or (2) the headwater to pipe diameter ratio (culvert rise in case of rectangular culvert) shall not exceed 1.5 for major flows. When using a roadway for detention, frequent maintenance of the pipe or culvert is required.

5.6.1.5 Approach Channel

The approach channel can be the same geometry of the upstream channel or be a transitional channel. A transitional channel is best suitable for pipes or culverts with high headwater to prevent sediment deposition at the entrance. A channel apron should be provided at the toe of the headwall. This apron should extend at least 2H upstream from the entrance. Here, H is the specific energy at the entrance. The top of the apron should not protrude above the normal streambed elevation. The apron can be riprap, gabion, or concrete. Wingwalls can be used as channel banks for a transitional channel. Entrances with wingwalls should be designed with a reinforced concrete apron extending between the walls, otherwise the wingwalls should extend to the depth of total scour (see Section 3.8). The length of the transitional channel should also extend at least 2H upstream from the entrance. Transitional channels without wingwalls should be protected with riprap, gabion, or concrete. The angle of transition should not exceed 45° from the centerline of the channel. The approach channel may be sloped to accomodate large pipes or culverts for limited headwater.

5.6.1.6 Upstream Channel

If the channel upstream is degrading, the pipe or culvert with a lined entrance will serve as grade control. However, if the upstream channel is aggrading, the headwater at the pipe or culvert entrance will accelerate the silting process. Regular maintenance should be performed to keep the pipe or culvert in operational conditions.

5.6.1.7 Toe Wall

A toe wall should be provided at the upstream end of the apron. The depth of the toe wall should extend below the expected scour depth as described in in Sections 3.6 and 3.7. For a lined approach channel (riprap or concrete lining), a toe wall should be provided for both the channel bed and banks. Riprap may be used upstream of the toe wall to protect channel bed and banks from scouring.

5.6.2 Definition Sketch of Pipe and Culvert Entrances

Figures 5-16, 5-17, and 5-18 present sketches of pipe or culvert entrances that are commonly used. The applicability of each type of entrance is summarized in Table 5-4.

5.6.3 Design Procedures

1. Determine design discharge.
2. Use entrance with headwall; size the pipe or culvert according to Section 4 of "Inlets and Culverts" in the Urban Storm Drainage Criteria Manual, Volume 2 and Section 5.6.1.4 of this report.
3. Check flow velocity at the entrance and the upstream channel due to the backwater effect of the headwater using $Q = AV$.
4. Select type of entrance based on the criteria set in Table 5-4.
5. Determine the height of headwall and the length of wingwalls, if required.
6. Examine local scour potential.
7. Determine type of protection required.
8. Size riprap, if used.
9. Determine depth of cutoff.
10. Determine depth of toe wall.

5.6.4 Design Example

Given:

1. $Q = 1000$ cfs.

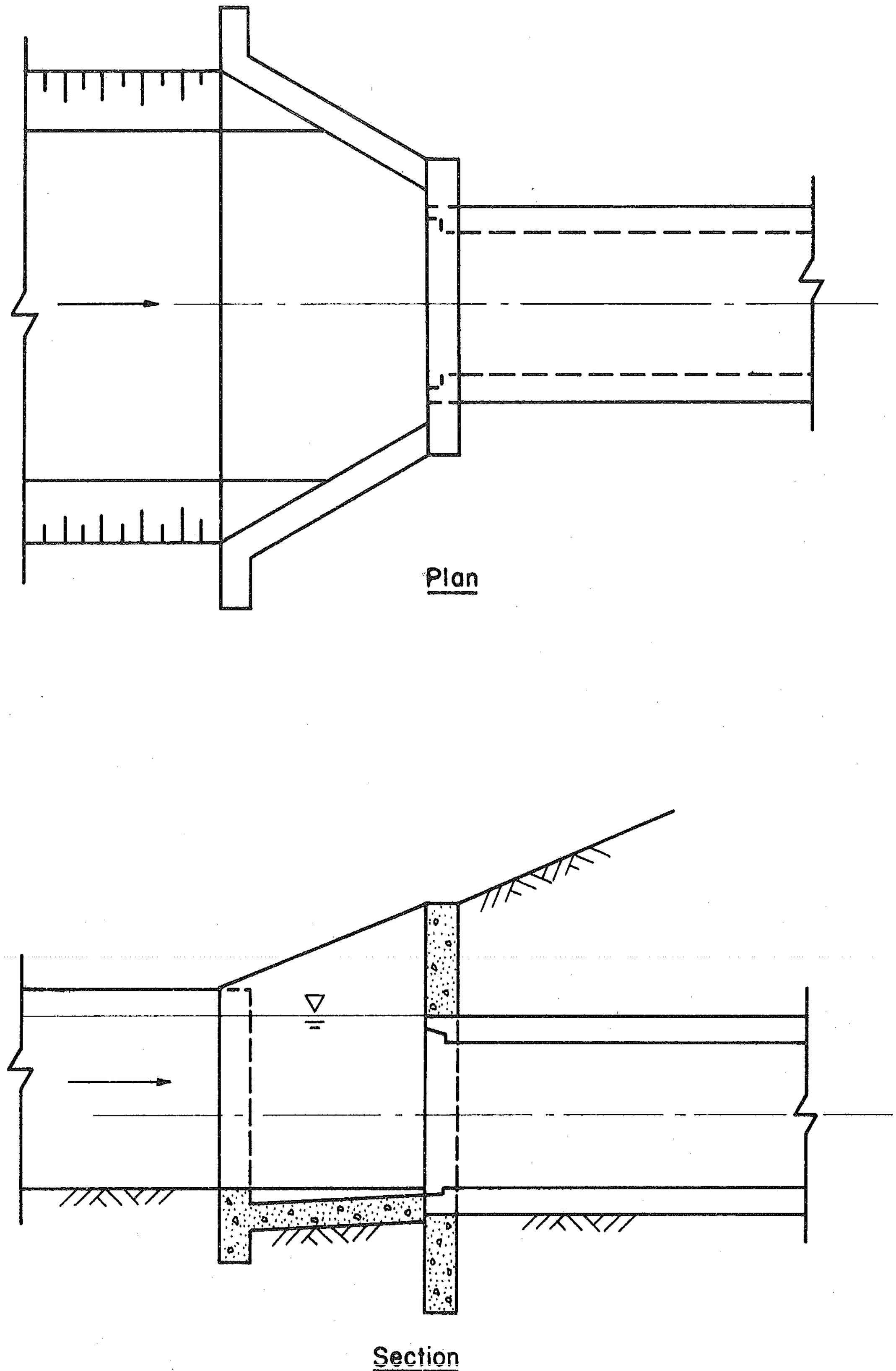


Figure 5-16. Type A - entrance with headwall, wingwalls, and toe wall.

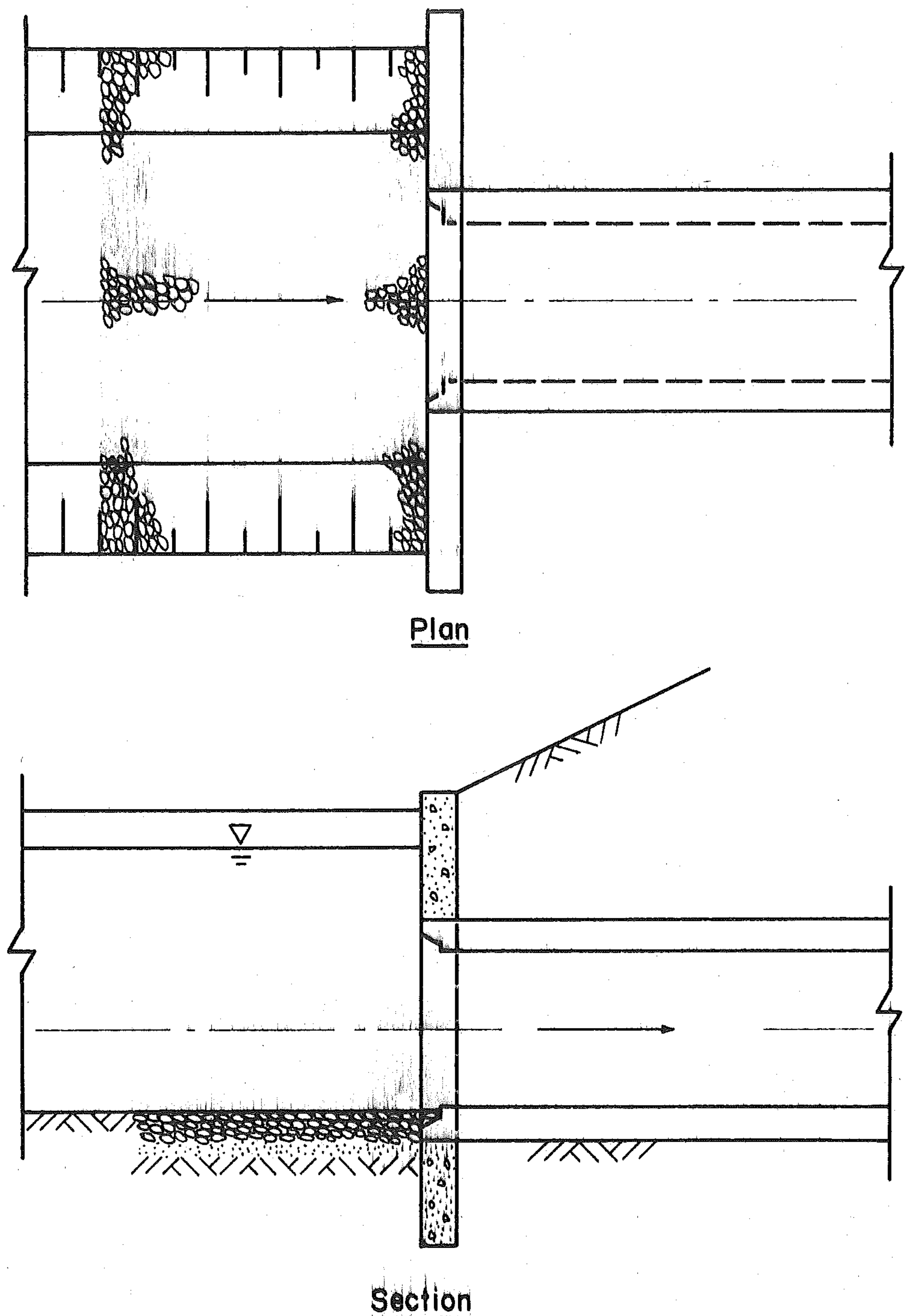
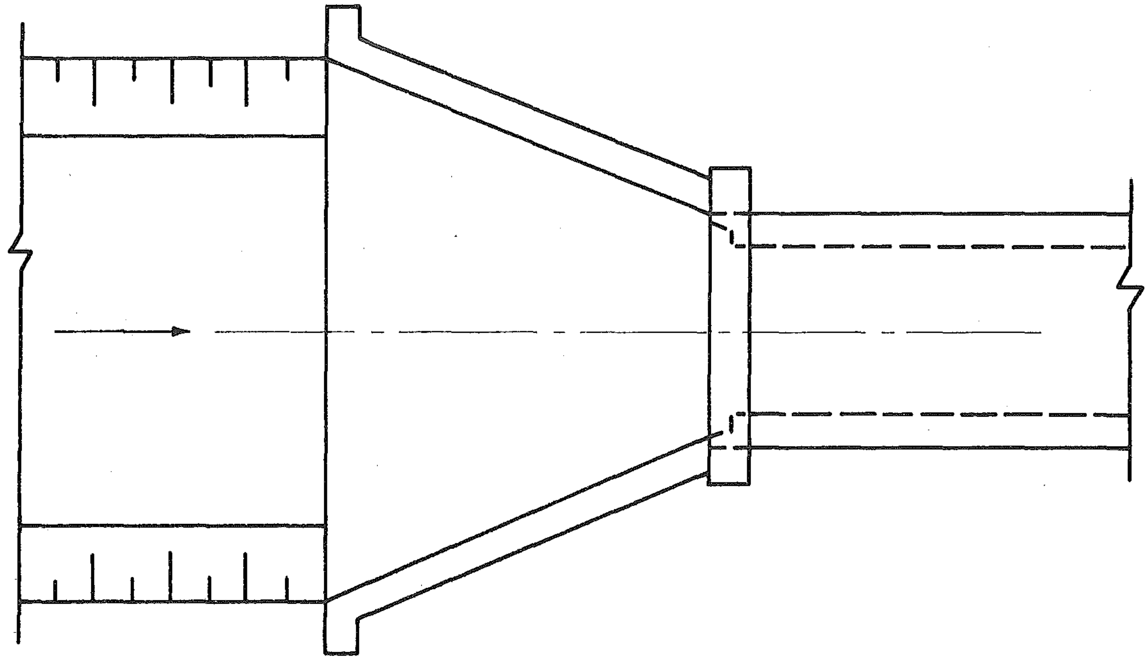
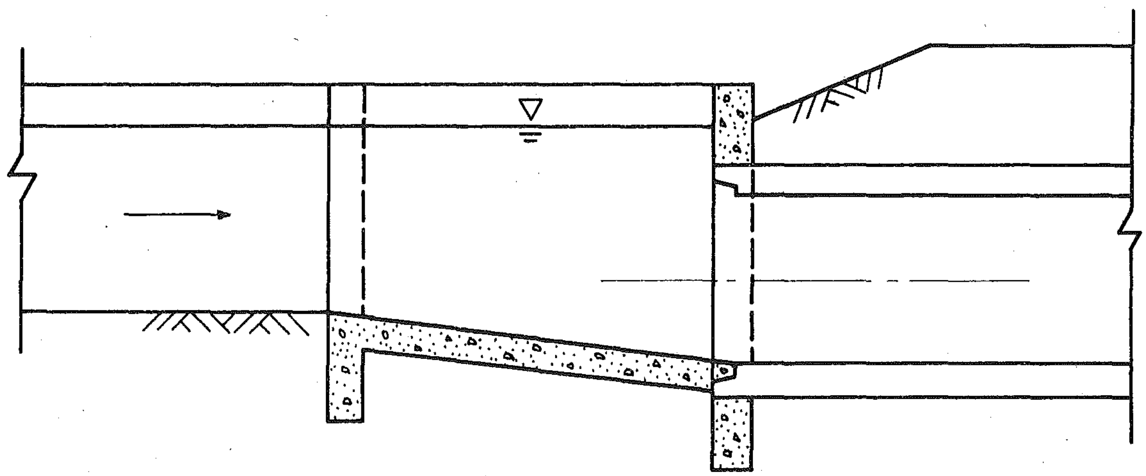


Figure 5-17. Type B - entrance with headwall and toe wall.



Plan



Section

Figure 5-18. Type C - entrance with sloped transition.

2. $S = 0.002$.

3. Approach channel cross section as shown in Figure 5-19.

4. The roadway crossing as shown in Figure 5-20.

5. Downstream tailwater depth is 4.8 feet.

6. Downstream channel has the same cross section as the upstream channel.

7. Channel bed material: $D_{15} = 0.09$ mm, $D_{50} = 0.18$ mm, $D_{85} = 0.29$ mm.

5.6.4.1 <u>Solution</u>

1. $Q = 1000$ cfs.

2. Two ten foot by five foot box culverts are used. The flow is inlet control. The headwater is 7.25 feet.

3. Velocity in the upstream channel

   $d = 7.25$ feet; $A = 273.76 + 118.16 = 391.92$ feet^2;

   $V = 1000/391.92 = 2.55$ fps > 2.5 fps.

4. For illustration, design Type A entrance:

   a. Set height of headwall six inches above the headwater, (i.e., 7.75 feet above the channel invert).

   b. Since the channel is 5.67 feet deep, the length of wingwall required is:

      $3(7.75 - 5.67) = 6.24$ feet.

   c. The ten foot by five foot culvert has a wall thickness of ten inches. The twin box culvert layout is shown in Figure 5-21.

5. Protect the approach channel to a distance of 2H upstream of the culvert inlet. The velocity at the culvert entrance is:

   $V = 1000/(7.25 \times 22) = 6.27$ fps, then,

$$H = Y_1 + \frac{V^2}{2g} = 7.25 + \frac{(6.27)^2}{2g} = 7.86 \text{ feet,}$$

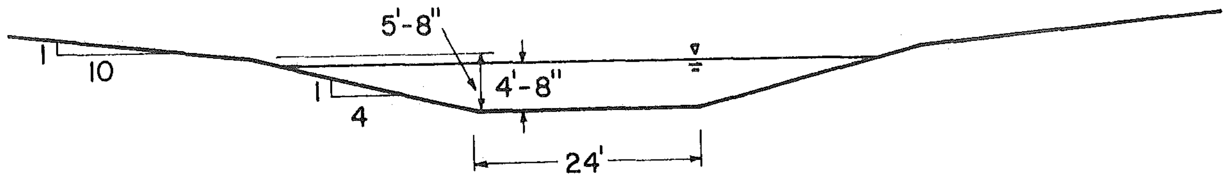


Figure 5-19. Channel cross section.

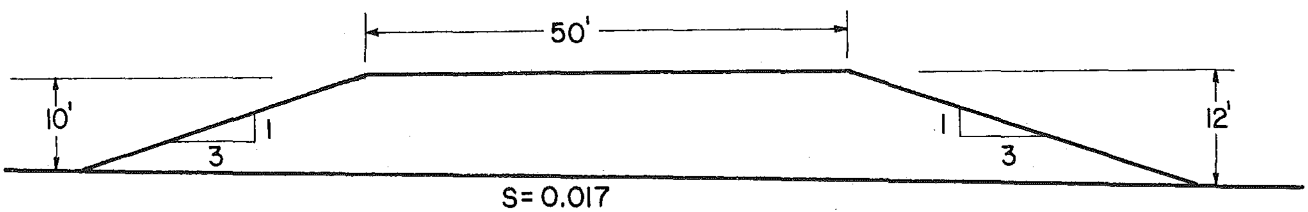


Figure 5-20. Roadway cross section.

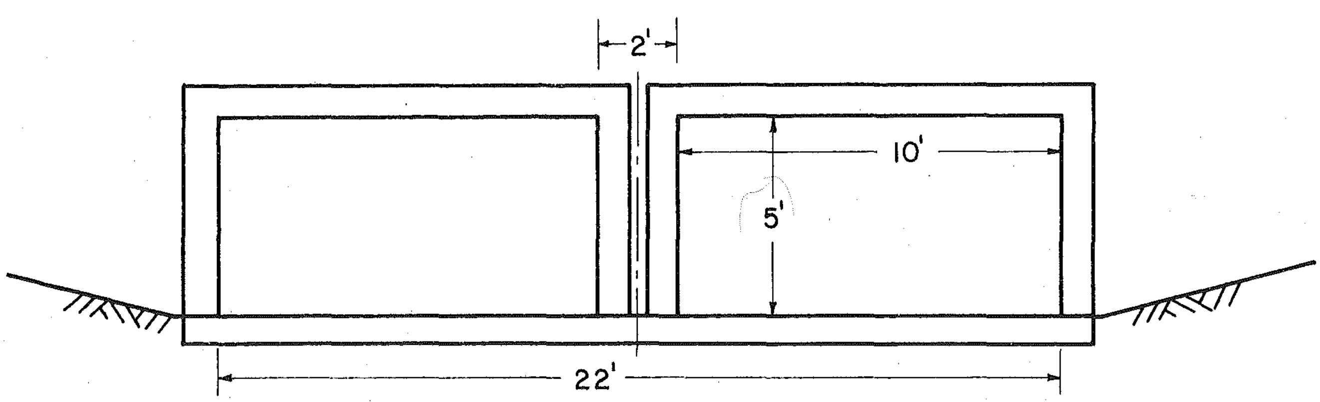


Figure 5-21. Twin box culvert layout.

2H = 2 x 7.86 = 15.72 feet, use 16 feet.

Since a length of 16 feet is required for the transitional channel, the wingwall is extended to 16 feet and the channel bank is protected with riprap (see Figure 5-22). The plan view and section of the inlet are shown in Figure 5-22.

6. Examine scour potential. Since the average velocity in the channel is 2.55 fps, which is less than the velocity upstream, there is no scour potential in the channel. However, at the transition, local scour should be expected. Use Equation 3-8 because of the vertical wall:

$$\frac{Y_s}{Y_1} = 2.15\left(\frac{a}{Y_1}\right)^{0.4} (Fr_1)^{0.33}$$

where, Y_1 = 7.25 feet,

a = 34.7 feet (from Figure 5-22),

V = 2.55 fps,

A = 391.92 feet^2,

p = 100.83 feet,

R = 3.89 feet,

$$Fr_1 = \frac{V}{\sqrt{gR}} = 0.23.$$

$$\frac{Y_s}{Y_1} = 2.15 \left(\frac{34.7}{7.25}\right)^{0.4} (0.23)^{0.33} = 2.48$$

$$Y_s = 2.48 \times 7.25 = 18.0 \text{ feet.}$$

If the approach channel is not protected, a local scour depth of 18.0 feet should be expected at the nose of the wingwall.

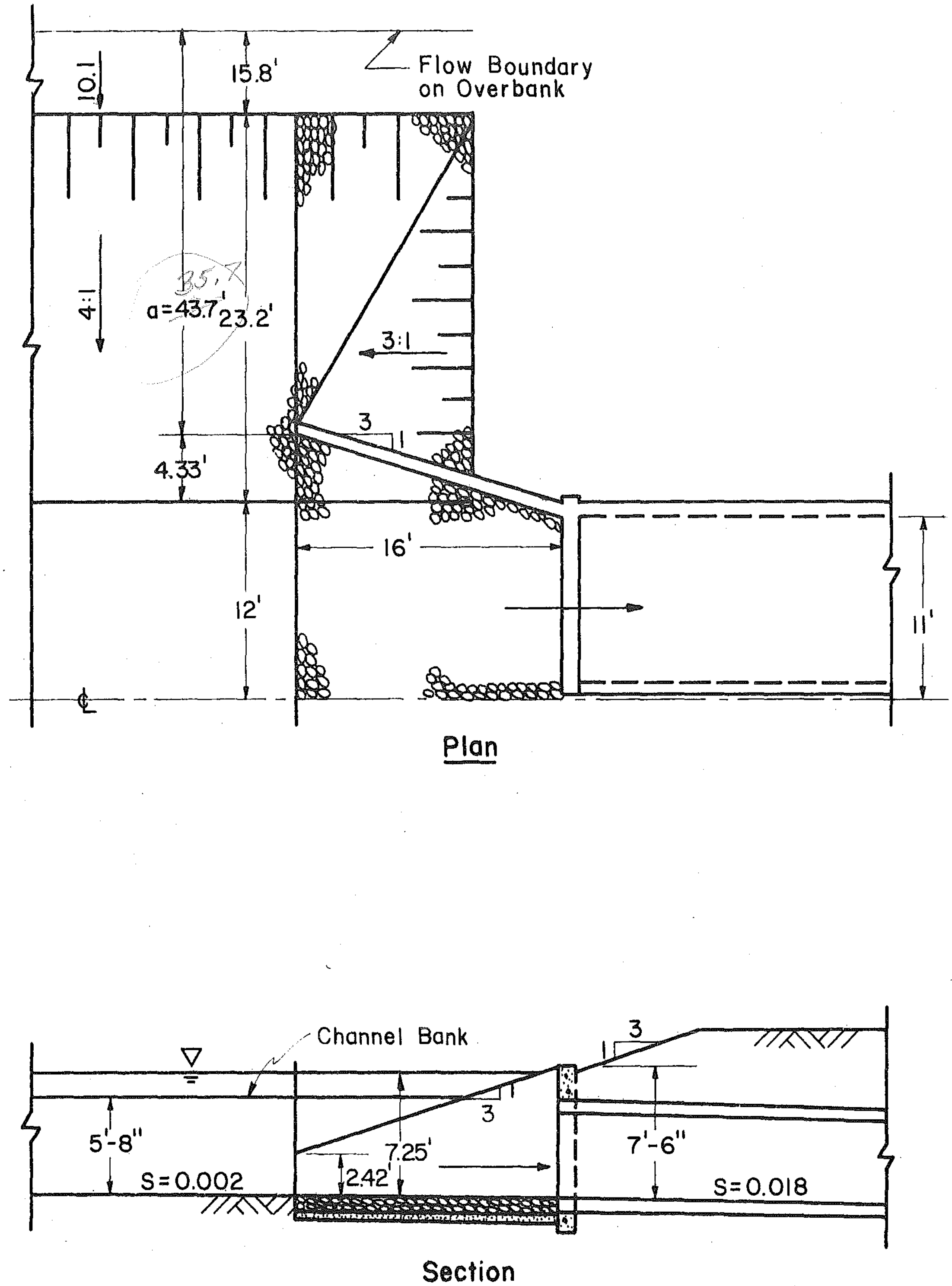


Figure 5-22. Design example - culvert entrance.

7. Size of riprap. The critical point is at the nose of the wingwall. A velocity of 1.5 times the channel velocity is used at the nose of the wingwall:

$$V = 1.5 \times 5.75 = 8.64 \text{ fps},$$

$$R = 4.52 \text{ feet},$$

$$V^2/R^{0.33} = 45.4,$$

from Table 5-1, use Type VL riprap.

8. Determine depth of cutoff. For $D_{50} = 0.18$ mm, from Table 3-2 the bed material is fine sand.

$$C_w = 7.0,$$

Head differential:

$$7.25 + 2 - 4.8 = 4.45 \text{ feet},$$

$$\Sigma L_H = 6 + 90 = 96 \text{ feet},$$

$$\Sigma L_V = 0$$

$$\frac{96 + 3\Sigma L_V}{3 \times 4.45} = 7.0,$$

$$\Sigma L_V = 0.85 \text{ feet}.$$

Use one foot upstream cutoff.

9. Since the scour potential at the upstream channel is protected by riprap, a toe wall at the end of the concrete apron should only be extended to the depth of riprap and filter, which is 12 inches.

5.7 Pipe and Culvert Outlets

The outlet is intended to contain the high velocity and minimize head loss while flow is expanded from the pipe or culvert back to the channel. At the outlet end of the pipe or culvert, the floow occurs as a highly concentrated, fast-moving jet which has considerable potential for causing damage.

The jet, with its associated eddies, will cause erosion downstream of the structure, undermine the outlet, and form a wide, deep scour hole in the downstream channel. Since a decelerating flow is inherently unstable, flow separation may occur at the outlet causing the jet to impinge against a downstream bank. In order to prevent the high outlet velocity from endangering the outlet structure and causing erosion of the downstream channel, a long outlet transition should be provided or the energy of the jet should be dissipated before the flow is released to the downstream channel.

5.7.1 Design Criteria

5.7.1.1 Outlet Transition

Experiments have shown that separation can be avoided in an outlet with straight walls diverging at a lateral expansion angle. The angle of lateral expansion can be found from Figures 5-6 and 5-7 in "Major Drainage" of the Urban Storm Drainage Criteria Manual, Volume 2. The length of protection should be determined following the criteria presented in Section 5.6.3 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2.

5.7.1.2 Energy Dissipators

Energy dissipators are very effective for protecting the channel from erosion at a pipe or culvert outlet. The energy of the jet would be dissipated before entering the downstream channel. Types of energy dissipators are presented in Section 2 of "Structures" in the Urban Storm Drainage Criteria Manual, Volume 2 and Section 5.5, entitled "Drop Structures," of this report.

5.7.1.3 Downstream Channels

The downstream channel should be reasonably straight to avoid the outlet jet impinging against the downstream bank. Downstream curves within 100 to 200 feet of the outlet should be avoided.

The pipe or culvert outlets may be the same geometry as the downstream channel. The channel bed and banks should be protected with riprap or gabion to a length depending on the ratio of tailwater to the flood depth at the outlet. The length of channel protection can be determined as described in Section 5.6.3 of "Major Drainage" of the Urban Storm Drainage Criteria Manual, Volume 2. A degradation and aggradation analysis should be performed to determine whether the downstream is a degrading or aggrading channel. An aggrading downstream channel tends to increase the tailwater depth and therefore, reduces the pipe or culvert capacity or increases the headwater. A degrading downstream channel would cause the exposure of the structure and consequent failure of the structure.

5.7.1.4 Headwall

Headwalls are used for providing embankment stability and embankment protection against erosion. The height of headwalls should be extended to the embankment slope and the embankment above the headwall should be protected against erosion to the level of the water surface plus freeboard for wave action.

5.7.1.5 Wingwalls

Wingwalls may be provided at pipe or culvert outlets to protect embankments against erosion. The length of wingwalls is determined by embankment slope. The depth of wingwalls should extend below the expected scour depth as described in Section 3.6 and 3.7 of this report.

5.7.1.6 Cutoff

A cutoff should be provided at the end of the pipe or culvert section to prevent the undermining of the structure. A cutoff should also be provided at the end of transition, riprap, natural channel, energy dissipator, and drop structures to prevent the undermining and loss of lining material. The depth of these cutoffs should extend below the scour depth using methods presented in Section 3.6 and 3.7.

5.7.1.7 Embankment Protection

At locations where roadway overflow is allowed, the downstream embankment face should be protected with riprap.

5.7.2 Definition Sketch of Pipe and Culvert Outlets

Figures 5-23, 5-24, and 5-25 present sketches of pipe or culvert outlets that are commonly used. The applicability of each type of outlet is summarized in Table 5-5.

5.7.3 Design Procedures

1. Determine design discharge.
2. Size the pipe or culvert according to Section 4 of "Inlets and Culverts" in the Urban Storm Drainage Criteria Manual, Volume 2.
3. Select type of outlet based on criteria set in Table 5-5.
4. Design the selected outlet.
5. Determine the requirement of downstream channel protection.
6. Size riprap, if required.
7. Determine the requirement of cutoff walls.

Design examples for Type C outlet can be found in Section 5.6 of "Major Drainage" of the Urban Storm Drainage Criteria Manual, Volume 2. Design example for Type B outlet can be found in Section 5.5 of this report.

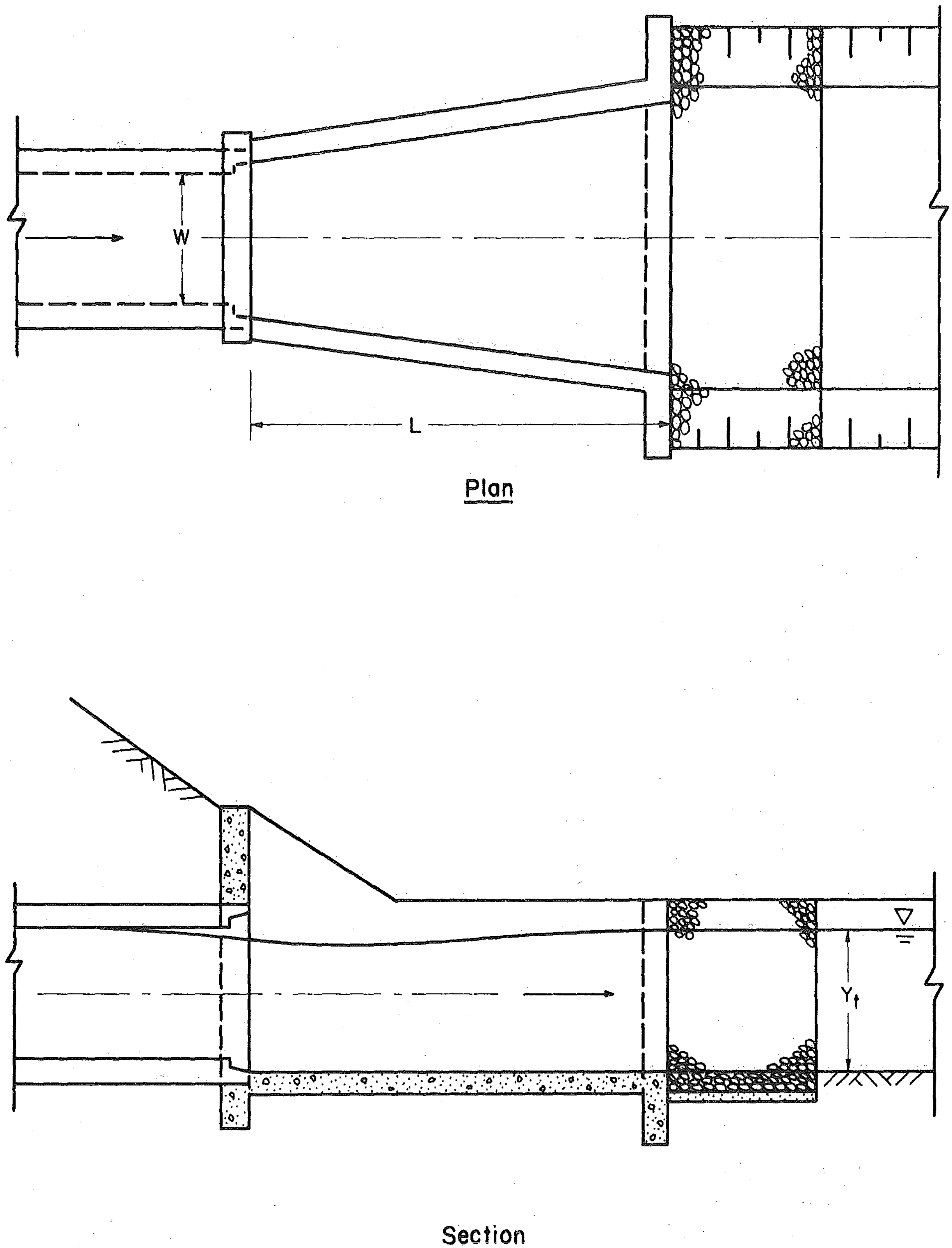


Figure 5-23. Type A - outlet with transition channel.

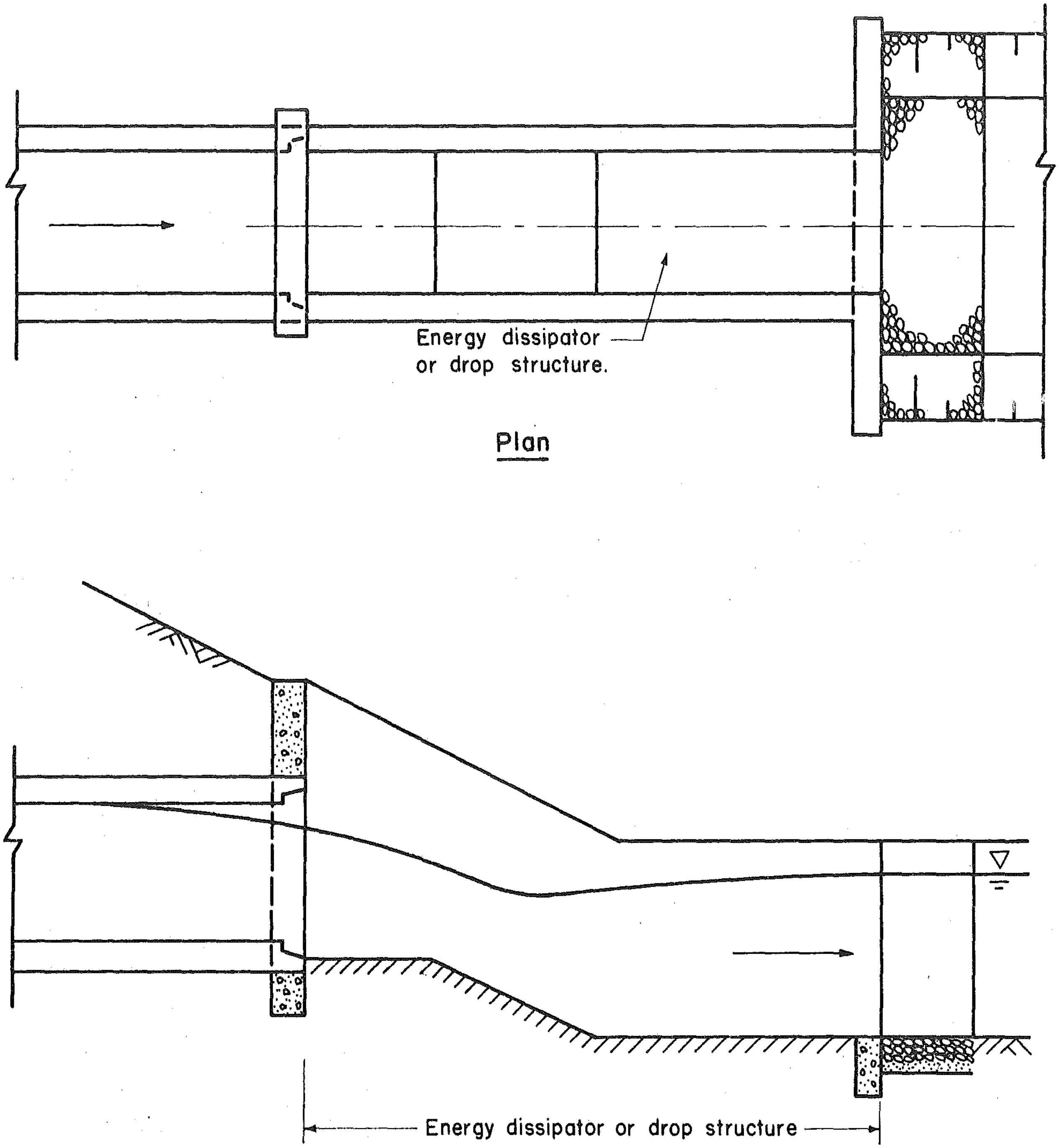


Figure 5-24. Type B - outlet with energy dissipator or drop structure.

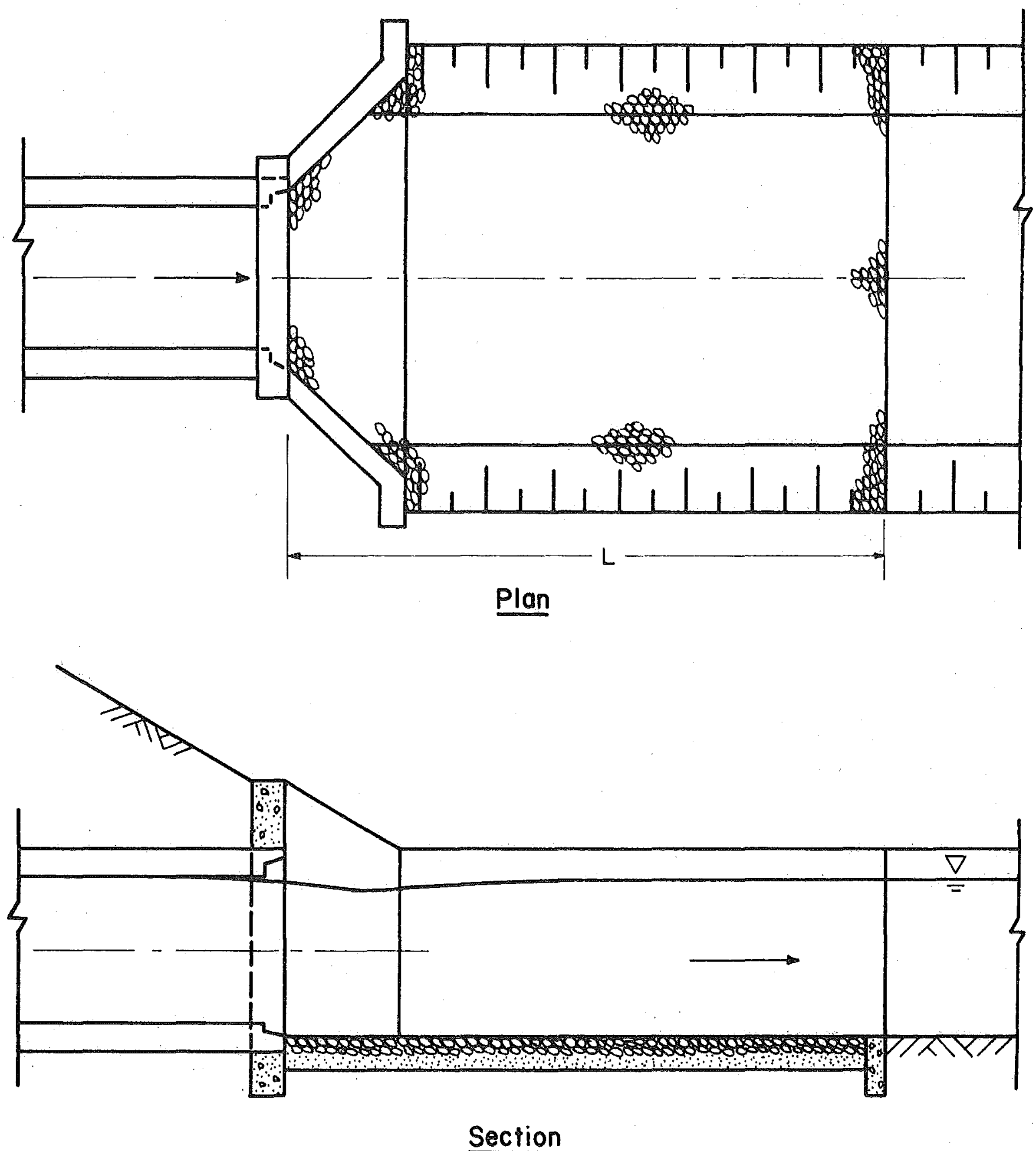


Figure 5-25. Type C - outlet with riprap channel.

5.8 Concrete Lined Channels

The general design considerations for concrete lined channels have been presented in Section 2.2 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2. Most common problems of concrete lined channels are due to bedding and liner failures. The causes of failures are: (1) liner cracking due to settlement of the subgrade; (2) liner cracking due to the removal of bed and bank material by seepage force; (3) liner cracking and floating due to hydrostatic back pressure from high groundwater; and (4) channel lining damage due to freezing and resultant heaving of the saturated subgrade.

Lack of maintenance will permit the growth of vegetation through the concrete lining and sediment deposition in the channel which will increase the flow resistance. The reduction in channel capacity can cause overflow at design discharges and, consequently, permit the erosion of overbank material and failure of concrete lining.

5.8.1 Design Criteria

5.8.1.1 Channel Geometry

For both construction and maintenance considerations, the channel side slope should not be steeper than 1.5:1. The bottom width to flow depth ratio is usually from one to two. Channels with side slopes steeper than 1.5:1 are often used; in this case, the design of channel banks should be treated as retaining walls.

5.8.1.2 Velocity

Concrete lined channels are usually used for channels having a Froude number in excess of 0.8 and/or when velocities exceed five feet per second in erosive sandy soil. The maximum velocity for unreinforced lining should be less than eight feet per second. A flow condition at Froude numbers near one is unstable and should be avoided. For flow conditions at Froude numbers larger than one, continuous two-way reinforcement is required.

5.8.1.3 Thickness

The thickness of concrete lining should be from 6 to 12 inches depending on channel capacity and stability of the channel against hydraulic force and other forces acting on the channel. Unreinforced concrete lining should have contraction joints at proper intervals to prevent cracking. Contraction joints should be spaced from 10 to 15 feet, depending on the size of channel and the thickness of lining. Reinforced concrete lining is recommended for channels on sandy soils.

5.8.1.4 Roughness Coefficient

The roughness coefficient used should be as the values listed in Table 2-3 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2 and should not be lower than 0.013.

5.8.1.5 Bedding

Long-term stability of concrete lined channels depends in part on proper bedding. Undisturbed soils often are satisfactory for a foundation for lining without further treatment. Expansive clays are usually an extreme hazard to concrete lining and should be avoided. A filter underneath the lining is recommended to protect the fine material from creeping along the lining. A well-graded gravel filter should be placed over the channel bed prior to channel lining. The specifications of filters should be as described in Section 5.3.1 of "Major Drainage" in the Urban Storm Drainage Criteria Manual, Volume 2.

5.8.1.6 Channel Transitions

Since concrete lined channels are usually used at locations where excessive seepage exists or smaller channel cross sections are required, transitions will be required both upstream and downstream of the concrete lined channel to reduce turbulence. Transitions should be lined with concrete or riprap to reduce scour potential.

5.8.1.7 Underdrains

The probability of damaging the concrete lining due to hydrostatic back pressure and subgrade erosion can be greatly reduced by providing underdrains. There are two types of artificial drainage installations. One type consists of four or six inch tile placed in gravel-filled trenches along one or both toes of the inside slopes. These longitudinal drains are either connected to tranverse cross drains which discharge the water below the channel or to pump pits, or extend through the lining and connect to outlet boxes on the floor of the channel. The outlet boxes are equipped with one-way flap valves which relieve any external pressure that is greater than the water pressure on the upper surface of the canal base, but prevent backflow. The second type consists of a permeable gravel blanket of selected material or sand and gravel pockets, drained into the canal at frequent intervals (10 to 20 feet) by flap valves in the invert. A drawing of a flap valve for use without tile pipe and in a fine gravel and sand subgrade is shown in Figure 5-26. Both the tile and pipe system and the unconnected flap valve type must be encased in a filter that will prevent piping of subgrade material into the pipe or through the valve. For detailed underdrains refer to Lining for Irrigation Canals published by the U.S. Department of the Interior, Bureau of Reclamation.

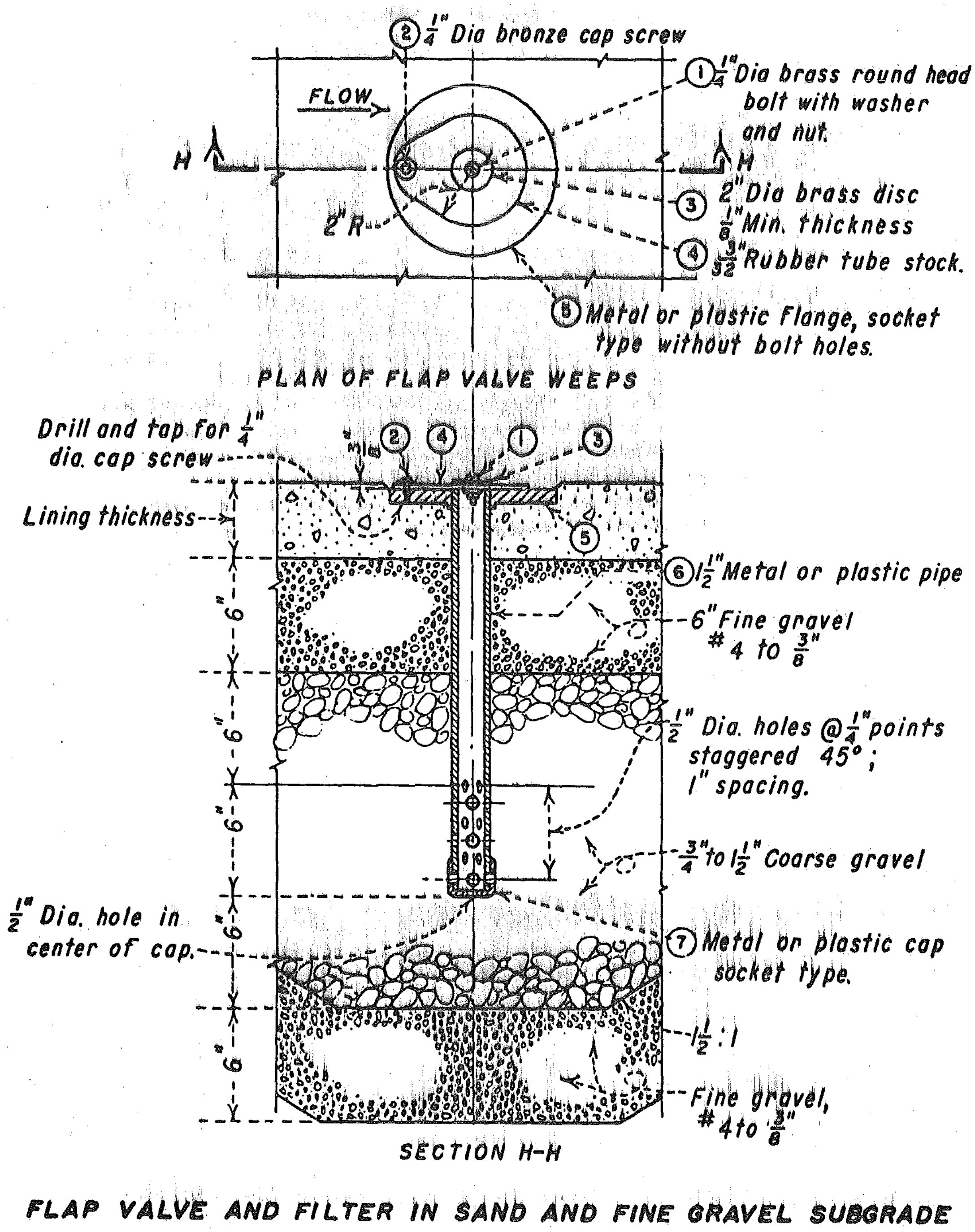


Figure 5-26. Flap valve installation for a canal underdrain.

5.8.1.8 Weep Holes

Weep holes need to be provided to reduce uplift force on the concrete lining. The spacing of weep holes should not exceed four feet.

5.8.1.9 Cutoffs

Cutoff walls should be provided at both the upstream and downstream end of the concrete lined channel to reduce seepage force and prevent lining failure due to scour, undermining, and piping. The depth of cutoff walls should extend below the expected scour depth as described in Sections 3.6 and 3.7 of this report.

5.8.1.10 Overbank Protection

Overbank areas should be protected with riprap to prevent concrete lining failure due to channel overflow. Riprap protection should extend horizontally $2d$ beyond the channel bank into the overbank area. Here, d is the depth of the channel.

5.9 Low-Flow Channels

Urbanization and encroachment on channels often result in straightening of the thalweg and also increasing the slope. As the channel steepens, the velocity increases, multiple channels can develop, and the channel is generally unstable and changes position with both time and stage. Low-flow channels can be used to confine low flows, and to reduce scouring, meandering, and silting problems. Low-flow channels are generally used in wide and shallow channels where the sediment transport capacity is low in the reach resulting in deposition of part of the incoming sediment.

5.9.1 Design Criteria

5.9.1.1 Design Discharge

Low-flow channels are generally designed for annual flooding to keep the channel from braiding. The braided stream is unstable, changes its alignment rapidly, and is difficult to work with.

5.9.1.2 Channel Slope

The equilibrium slope of low-flow channels is usually milder than the main channel because the velocities are higher and flow is more confined. Stable low-flow channels should be designed to their equilibrium slope to avoid scouring or silting problems. Grade control structures are usually used to maintain the equilibrium slope.

5.9.1.3 Channel Geometry

The width, and width to depth ratio, of low-flow channels should be designed following the past history of the stream. Low-flow channels formed in undisturbed reaches can be used as a guide for low-flow channel design because they are formed naturally and should be relatively stable.

5.9.1.4 Bank Protection

Stable low-flow channels require stabilization of the low-flow channel banks to prevent channel meandering. Concrete or riprap can be used for this purpose. The bank protection should extend to the expected scour depth as described in Section 3.6 and 3.7.

5.10 Trickle Channels

Trickle channels confine trickle flows of one-half to one percent of the 100-year flow with the intent of controlling thalweg degradation, aggradation and/or meandering, and to minimize maintenance from excessive vegetation buildup. Trickle channels should always be constructed of concrete for maintaining the required self-cleaning velocities. Trickle channels are usually small compared with the flood channel and low-flow channel. If the channel is aggrading during a flood event, the trickle channel will be silted. On the other hand, if the channel is degrading during a flood event, the trickle channel will be washed away due to the channel bed scouring. Trickle channels are not recommended for sand-bed channels unless the channel cross section is lined.

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