

## CHAPTER VI: EROSION AND SEDIMENTATION

### 6.1 Introduction

The hydrology and hydraulics of floodwaters are not the only concern of floodplain-management administrators and/or drainage design engineers who work in arid or semi-arid environments which contain alluvial rivers such as those that exist both within and around the City of Tucson, Arizona. The transport of sediment by floodwaters is also a major concern because of the potential for rapid bank erosion and changes in channel bed elevations. Bank erosion can often be so severe that it causes much more damage than inundation by floodwaters. Aggradation or degradation of the channel bed can rapidly change flood limits, or cause bank protection and other channel improvements to fail over a very short period of time.

The study of fluvial geomorphology and the analysis of sediment transport are usually undertaken in an attempt to quantify the broad effects of erosion and sedimentation and the impacts of sediment-transport capacity upon channel morphology. Sediment-transport analysis is a relatively specialized field of study. Predictions based upon its application are often expensive to produce, and can be highly variable in nature. Therefore, as an aid to the user, this chapter of the Manual presents some design and predictive guidelines that can be used within the City of Tucson in the absence of a more detailed sediment-transport analysis.

### 6.2 Purpose

The purpose of this chapter is to provide guidelines for the estimation of erosion, sedimentation, and channel bed scour when designing drainage channels and hydraulic structures which are to be located within the City of Tucson. These design guidelines and procedures are to be used when normal design situations are encountered. Deviations from these guidelines may occur, provided that the user has experience in sediment-transport technology; and provided that the deviation is technically justified, through detailed sediment-transport analysis, to the satisfaction of the City Engineer.

### 6.3 Fluvial Geomorphology

The study of fluvial geomorphology normally involves analyses which encompass entire drainage systems. This is so because the response of an individual channel to change within a watershed can often have an effect upon the entire drainage system. Conversely, the fluvial system, as a whole, will ultimately dictate the response of an individual channel to overall change within a watershed. Rarely is it possible to understand the fluvial processes which occur within even a short reach of an alluvial channel in isolation from its upstream and downstream system controls.

The fluvial system is generally divided into three zones (Schumm, 1977). Zone 1 is characterized as the drainage basin, watershed, or source area for sediment. This is the area from which water and sediment are derived. Storage of sediment is not significant in this zone. Zone 2 is characterized as the transport zone; where, for a stable channel, sediment input can equal sediment output. For those reaches where the sediment-transport capacity exceeds the upstream supply, it can be assumed that the sediment deficit will be made up out of the channel bed or banks. Channel bed

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degradation or erosion of channel banks will be the result. Zone 3 is characterized as the sediment sink or area of deposition.

Obviously, the division between these three zones is indiscrete. Each zone has characteristics of the other two, which are subordinate to the primary characteristic of the zone. Zone 2 is of major concern to the hydraulic and river-control engineer, and to geomorphologists concerned primarily with river-channel morphology. It is this zone with which this chapter deals.

### 6.3.1 Channel Morphology

Sediment and water moving through alluvial channels are the independent variables that determine the size, shape, and pattern of the channel. Numerous empirical relations have been developed that relate channel morphology to water and sediment discharge.

#### 6.3.1.1 Hydraulic Geometry of Alluvial Channels

As a general rule, the greater the quantity of water that moves through a channel, the larger is the cross-sectional area of that channel. Preceded by numerous studies of canal morphology and stability, Leliavsky (1955) and Leopold and Maddock (1953) demonstrated that, for most rivers, the water surface width,  $T$ , and depth,  $Y$ , increase with mean-annual discharge,  $Q_a$ , in a downstream direction such that:

$$T = k_1 Q_a^b \quad (6.1)$$

and,

$$Y = k_2 Q_a^c \quad (6.2)$$

Both the coefficients and exponents of Equations 6.1 and 6.2 (i.e., the " $k_1$ ," " $k_2$ ," " $b$ ," and " $c$ " values) are different for each river and, when data from a number of rivers are plotted against discharge, the scatter covers an entire log cycle. For a given discharge, there is an order of magnitude range of width and depth. Therefore, other variables apparently influence the hydraulic geometry of channels as well.

#### 6.3.1.2 Influence of Sediment Load

A primary variable which significantly controls river morphology is sediment load. Bed-material load is defined as that part of the stream's sediment load that consists of sediment sizes comprising a significant part of the streambed. The other component of total sediment load is wash load, which is part of the total load not significantly represented in the bed. In and around the Tucson area, wash load is generally composed of sediments smaller than sand (i.e., smaller than about 0.06 mm to 0.07 mm). Wash load is held in suspension by the turbulence of the flowing water, and therefore is transported at the same velocity as the water. Bed-material load is composed of sands and larger sediments, and therefore is generally transported at an average velocity less than the velocity of flowing water.

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From an analysis of data from regime canals, Lacey (1930) concluded that the wetted perimeter of a channel is directly dependent upon discharge; but that channel shape reflects sediment size. It is also generally recognized that coarse sediment produces channels with high width/depth ratios, while fine sediment produces channels with narrow and deep cross sections.

In addition to the size of the transported sediment, relative amounts of bed-material load and wash load significantly influence the morphology of sand-bed streams. Large bed-material loads are associated with wide channels, and large wash loads are associated with narrow widths.

The type of sediment load is considered to be a more important control on stable channel shape than the total quantity of sediment transported through a channel. For example, in one channel a certain quantity of bed-material load may exert the dominant control if it is the total load, whereas in another channel the same amount of bed-material load may exert much less influence on channel shape because it is only a small part of the total sediment load (i.e., wash load and bed-material load). Therefore, when load and discharge are constant, an increase in the quantity of bed-material load will cause an increase in channel width, and a corresponding increase in the width/depth ratio. This phenomenon is probably related to the high gradient and velocity of flow generally associated with large bed-material loads.

In summary, for alluvial channels which occur in the Tucson area, the type and amount of sediment load exerts a major control on their shape. Therefore, for a single channel under the ideal assumption of a constant discharge and a fixed amount of wash load, a change in bed-material load would be reflected by a change in both the shape and gradient of the channel.

### 6.4 Sediment-Transport Theory

Sediment particles are transported by flowing water in one or more of the following ways: (1) surface creep, (2) saltation, and (3) suspension. Surface creep is the rolling or sliding of particles along the bed. Saltation (jumping) is the cycle of motion above the bed, with resting periods on the bed. Suspension involves the sediment particle being supported by the water during its entire motion. Sediments transported by surface creep, sliding, rolling, and saltation are referred to as bed load, and those transported by suspension are called suspended load. The suspended load consists of sands, silts, and clays. Total sediment load is defined as the sum of the bed load and suspended load. Generally, the amount of bed load transported by a large river is on the order of five to twenty-five percent of the suspended load. Although the amount of bed load may be relatively small compared with total sediment load, it is important because it shapes the bed, influences channel stability, determines the form of bed roughness, and affects various other hydraulic factors as well.

As presented earlier, the total sediment load in a channel can be more simply defined as the sum of bed-material load and wash load; where the bed-material load is the sum of bed load and suspended bed-material load, representing that part of the total sediment discharge which is composed of grain sizes found in the bed; and the wash load is that part of the sediment discharge which is composed of particle sizes

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finer than those found in appreciable quantities in the bed (Simons and Senturk, 1977). The presence of wash load can increase bank stability, reduce seepage, and increase bed-material transport. Wash load can be easily transported in large quantities by the stream, but is usually limited by availability from the watershed. The bed-material load is more difficult for the stream to move, and is normally limited in quantity by the transport capacity of the channel. Figure 6.1 summarizes the various definitions of the components of sediment load, and their contribution to total sediment load.

There is no clear size distinction between wash load and bed-material load. As a rule of thumb for the Tucson area, it should be assumed that the size of bed-material particles is equal to or larger than 0.0625 mm, which is the division point between sand and silt. The sediment load consisting of grains smaller than this size is then considered as wash load. It is generally assumed that most of the wash load is transported through the system by stream flow, and that little wash load is deposited on or in the stream bed. Wash load deposited with coarse material is usually only a very small fraction of the total bed material within the channel.

The amount of material transported, eroded, or deposited in an alluvial channel is a function of both the sediment supply and the sediment-transport capacity of the channel. Sediment supply includes the quality and quantity of sediment brought to a given reach. Sediment-transport capacity is a function of the size of bed material, flow rate, and geometric and hydraulic properties of the channel. Generally, the single most important factor determining sediment-transport capacity is flow velocity. Additionally, since sediment-transport capacity is generally proportional to the third to fifth power of the velocity, small changes in velocity can cause large changes in sediment-transport capacity (Simons, Li & Associates, 1982, 1985). Either the sediment supply or sediment-transport capacity may limit the actual sediment-transport rate in a given reach.

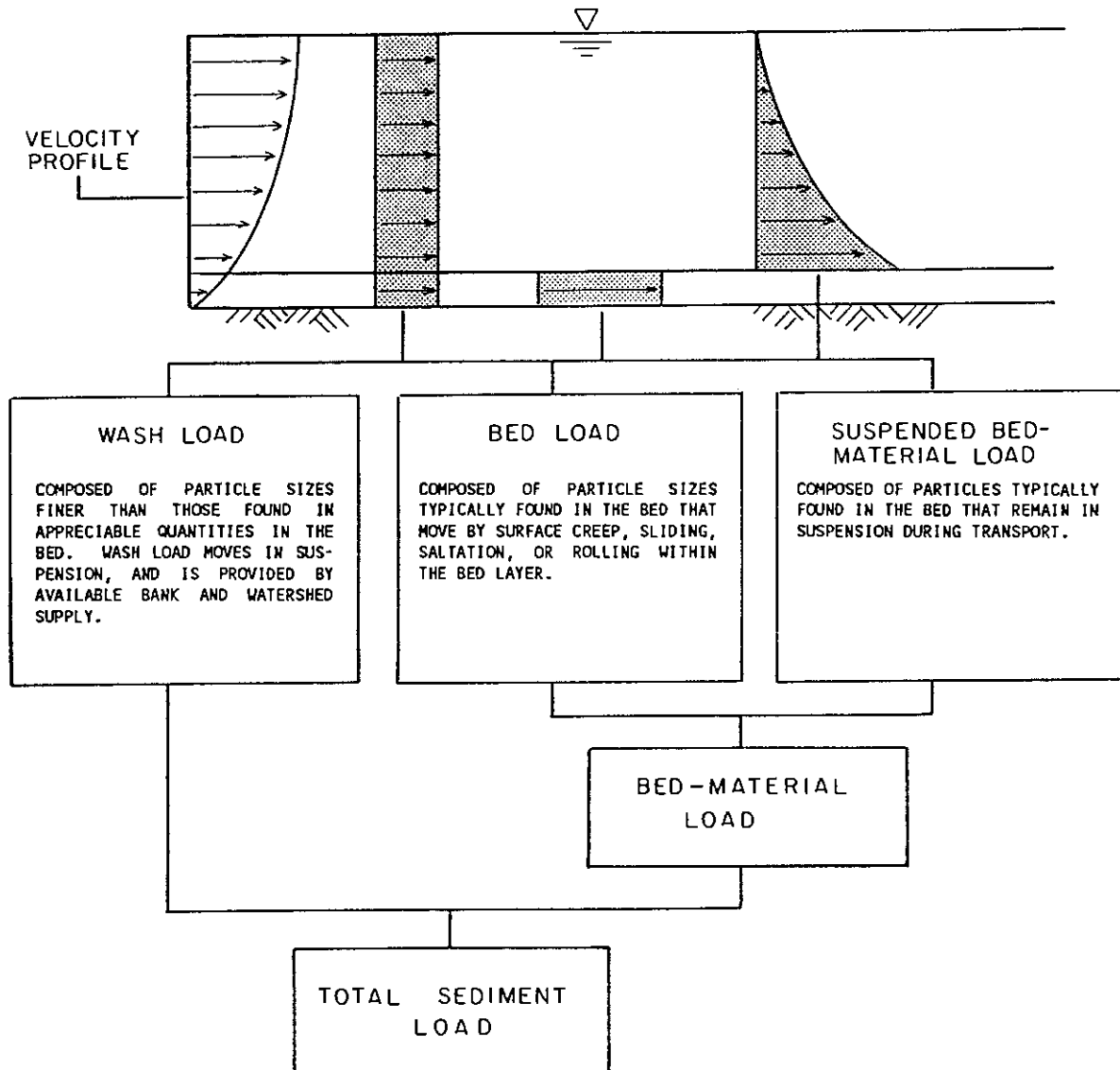
### 6.5 Sediment Routing

Supported by qualitative and quantitative analysis, a detailed evaluation of the fluvial-system response can be made based upon mathematical-modeling concepts. A mathematical model is simply a quantitative expression of the physical processes. The mathematical processes governing watershed and river responses are complicated. Computer programs can provide a means of assessing the many parameters of these complex processes within a fluvial system. There are several computer models available which are applicable to this region. For information on where to obtain these models, the user should contact the City Engineer.

#### 6.5.1 *Simplified Sediment Modeling*

After evaluating the hydraulic conditions of the river by water-routing programs such as the U.S. Army Corps of Engineers HEC-2 program, the sediment-transporting capacity can be established. Sediment-transport equations are used to determine the sediment-transport capacity for a specific set of flow conditions. Different transport capacities can be expected for different sediment sizes. For each sediment size, the transport rate includes the transport rate of the bed load and the transport rate of the suspended bed-material load.

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NOTE:

THE TERM "SUSPENDED LOAD" IS USED WHEN REFERRING TO THE SUM OF THE "WASH LOAD" AND "SUSPENDED BED-MATERIAL LOAD" COMPONENTS. THEREFORE, AN ALTERNATE DEFINITION OF TOTAL SEDIMENT LOAD IS THE SUM OF THE SUSPENDED LOAD AND BED LOAD.

FIGURE 6.1  
 DEFINITION OF SEDIMENT-LOAD COMPONENTS

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One modeling method uses hydraulic conditions from a rigid-boundary model such as HEC-2, or an equivalent program, and computes sediment transport based upon the Meyer-Peter, Muller bed-load equation and the Einstein suspended-load procedure for each sediment size found in the bed. The data required are the same as for HEC-2 (channel geometry, resistance, bridge constriction, etc.). Also needed are the size distribution of the bed-material and the upstream sediment supply. Using the generated hydraulic conditions, the transport capacity for each sediment size at each cross section is then determined.

Actual transport rates depend upon transport capacities as well as supply rates. The change in transport capacity between two cross sections can be used to estimate aggradation or degradation, based upon availability. For example, if sediment is in ample supply to meet the transport capacity at an upstream cross section but at the next cross section downstream the transport capacity is only one-half as much, then the other one-half of the sediment passing the upstream cross section must be deposited between the upper and lower cross sections. This comparison of transport capacities continues reach by reach and size fraction by size fraction through the entire stream segment. The drawback to this simplified approach is that the hydraulic conditions are not readjusted, due to aggradation or degradation, at frequent time increments during the passage of the flood hydrograph. However, this technique does provide "trends" in bed-elevation changes without using more complex techniques.

### 6.5.2 *Quasi-Dynamic Sediment Modeling*

The sediment-routing model previously discussed is based upon a gradually-varied-flow backwater program which assumes a rigid-boundary system. This methodology can be extended to account for unsteady flow and alluvial-channel boundaries without going to a fully unsteady water and sediment-routing model.

The quasi-dynamic sediment model uses the same gradually-varied-flow backwater program for hydraulic computations. However, the flow is assumed constant for a given time increment  $\Delta t$ . A flow event, either short-term or long-term, can be broken into a number of time increments, each with a different flow rate, but during each increment the flow is considered steady.

To account for a non-rigid or alluvial boundary, when a predetermined volume of sediment is either deposited on or eroded from the streambed, the cross section is recomputed in the following manner.

Sediment aggradation or degradation within a reach for a given time period is  $\Delta V_s = (\text{sediment supply} - \text{sediment transport}) \times BF$ , where  $\Delta V_s$  is the change in sediment volume in the reach and  $BF$  is a bulking factor. The change in sediment volume is assumed to be uniformly distributed throughout the reach. Change in area for each cross section is determined by a weighting factor based upon the conveyance in adjacent segments of the cross sections. The changes in elevation are used to generate a new HEC-2 data file for the next time period. Therefore, during any given time period the channel boundary is assumed to be rigid and the HEC-2 analysis is assumed to be valid. After evaluating the hydraulic conditions and the sediment-transport capacity, the channel boundary is modified to reflect the aggradation/

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degradation changes occurring throughout the river, and to establish the new channel configuration for the next time step.

This methodology has been successfully applied to a number of practical engineering problems. It provides a feasible and relatively cost-effective approach to design problems in alluvial rivers.

### 6.5.3 *Dynamic Mathematical Modeling*

Dynamic mathematical modeling of water and sediment routing is the next level of sophistication and complexity in determining alluvial-channel changes. It involves unsteady, non-uniform flow routing for determining the hydraulic conditions to be used to calculate sediment transport, aggradation, and degradation.

Unsteady, non-uniform flow routing solves equations governing the motion of water in open channels. These equations are mathematical descriptions of the physical phenomena. The two basic principles for water routing are continuity and momentum. Continuity states that water coming into a reach is either stored in the reach or passes downstream without gaining or losing water.

The momentum principle balances the forces and accelerations acting on flowing water. Generally, the continuity and momentum equations, along with a resistance to flow equation involving Manning's  $n$  or Chezy's  $C$ , are solved numerically in finite-difference form. The results are the hydraulic variables of velocity, depth, and width for unsteady, non-uniform flow. These are then used to route sediment. Sediment movement is controlled by the shear forces acting on the bed, transport capacity of the flow, and both availability and supply. Equations used in these calculations are described in most sedimentation textbooks. To compute aggradation and degradation, the sediment-continuity equation is used.

While dynamic mathematical modeling can give excellent results, it is very complex. Fortunately, it is not often required to solve many of the more straightforward, practical problems that designers will usually encounter within the Tucson area. In fact, most aggradation and degradation problems can be solved to an acceptable degree of accuracy by the several methods previously described within this chapter of the Manual.

### 6.6 Depth of Scour

Scour, or lowering of a channel bed (excluding long-term aggradation/degradation), can be caused by discontinuity in the sediment-transport capacity of the flow during a runoff event (general scour); the formation of anti-dunes in the channel bed during a runoff event; transverse currents within the flow through a bend (bend scour) during a runoff event; local disturbances, such as abutments or bridge piers, during a runoff event; and the formation of a low-flow channel thalweg. The design depth of scour (*excluding* long-term aggradation/degradation, which must be added for toe-down design) is the sum of all these individual scour components, and can be expressed by:

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$$Z_t = 1.3 (Z_{gs} + 1/2Z_a + Z_{ls} + Z_{bs} + Z_{lft}) \quad (6.3)$$

Where:

- $Z_t$  = Design scour depth, *excluding* long-term aggradation/degradation, in feet;
- $Z_{gs}$  = General scour depth, in feet;
- $Z_a$  = Anti-dune trough depth, in feet;
- $Z_{ls}$  = Local scour depth, in feet;
- $Z_{bs}$  = Bend scour depth, in feet;
- $Z_{lft}$  = Low-flow thalweg depth, in feet; and,
- 1.3 = Factor of safety to account for nonuniform flow distribution.

The various equations for depth of scour which are to follow were developed strictly for use in conjunction with sand-bed channels in which the bed material is erodible to the depth specified by the applicable equations. However, this situation does not always exist in channels located within the City of Tucson. In some areas of the city, the channel has degraded to a point where the exposed bed is no longer composed of strictly unconsolidated alluvial material, but rather of consolidated hardpan or caliche. Channel beds composed of this type of material are not freely erodible, and thus the scour equations which follow may not strictly apply. Should such conditions be encountered, a geotechnical investigation should be submitted by an Arizona Registered Professional Civil Engineer to justify the use of a lesser scour depth than would be determined from the use of Equation 6.3.

### 6.6.1 General Scour

As previously discussed in Section 6.5 of this Manual, the depth of general scour is best estimated by performing a detailed sediment-transport analysis using the bed grain-size distribution, hydraulic conditions, sediment-transport capacity at different stages throughout the flow event, changes in bed levels throughout the event, and the sediment supply into the reach being studied. An analysis to this level of detail is beyond the scope of this Manual. However, there are several computer models commercially available to aid in making an estimate of general scour. Unfortunately, these models are very sensitive to input, and the results are best interpreted by someone with extensive experience in the field of sediment transport. A detailed discussion of sediment-transport analysis for computing general scour can be found in "Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1982), and "Arizona Department of Water Resources Design Manual for Engineering Analysis of Fluvial Systems" (Simons, Li & Associates, 1985).

General scour on regional watercourses should be estimated by undertaking a detailed sediment-transport study, as described above, when and where it is feasible to do so. However, such a study is not usually practical on smaller watercourses. Therefore, as an alternative to the above, on watercourses other than regional watercourses, the following equation (Zeller, 1981) should be used to predict general scour:

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$$Z_{gs} = Y_{\max} \left[ \frac{0.0685V_m^{0.8}}{Y_h^{0.4}S_e^{0.3}} - 1 \right] \quad (6.4)$$

Where:

- $Z_{gs}$  = General scour depth, in feet;
- $V_m$  = Average velocity of flow, in feet per second;
- $Y_{\max}$  = Maximum depth of flow, in feet;
- $Y_h$  = Hydraulic depth of flow, in feet; and,
- $S_e$  = Energy slope (or bed slope for uniform-flow conditions), in feet per foot.

NOTE: Should  $Z_{gs}$  become negative, assume that the general-scour component is equal to zero (i.e.,  $Z_{gs} = 0$ ).

### 6.6.2 *Anti-Dune Trough Depth*

Anti-dunes are bed forms, in the shape of dunes, which move in an upstream rather than a downstream direction within the channel; hence the term "anti-dunes." They form as trains of waves that build up from a plane bed and a plane water surface. Anti-dunes can form either during transitional flow, between subcritical and supercritical flow, or during supercritical flow. The wave length is proportional to the velocity of flow. The corresponding surface waves, which are in phase with the anti-dunes, tend to break like surf when the waves reach a height approximately equal to 0.14 times the wave length. A relationship between average channel velocity,  $V_m$ , and anti-dune trough depth,  $Z_a$ , can therefore be developed (Simons, Li & Associates, 1982). This relationship is:

$$Z_a = \frac{1}{2} (0.14) \frac{2\pi V_m^2}{g} = 0.0137V_m^2 \quad (6.5)$$

A restriction on the above equation is that the anti-dune trough depth can never exceed one-half the depth of flow. Therefore, if the computed depth of  $Z_a$  obtained by using Equation 6.5 exceeds one-half of the depth of flow, the anti-dune trough depth should then be taken as equal to one-half the depth of flow. Figure 6.2 shows a definition sketch for anti-dune trough depth.

### 6.6.3 *Low-Flow Thalweg*

A low-flow thalweg is a small channel which forms within the bed of the main channel, and in which low discharges are carried. Low-flow thalwegs form when the width/depth ratio of the main channel is large. Rather than flow in a very wide, shallow state, low flows will develop a low-flow channel thalweg below the average channel bed elevation in order to provide more efficient conveyance of these discharges.

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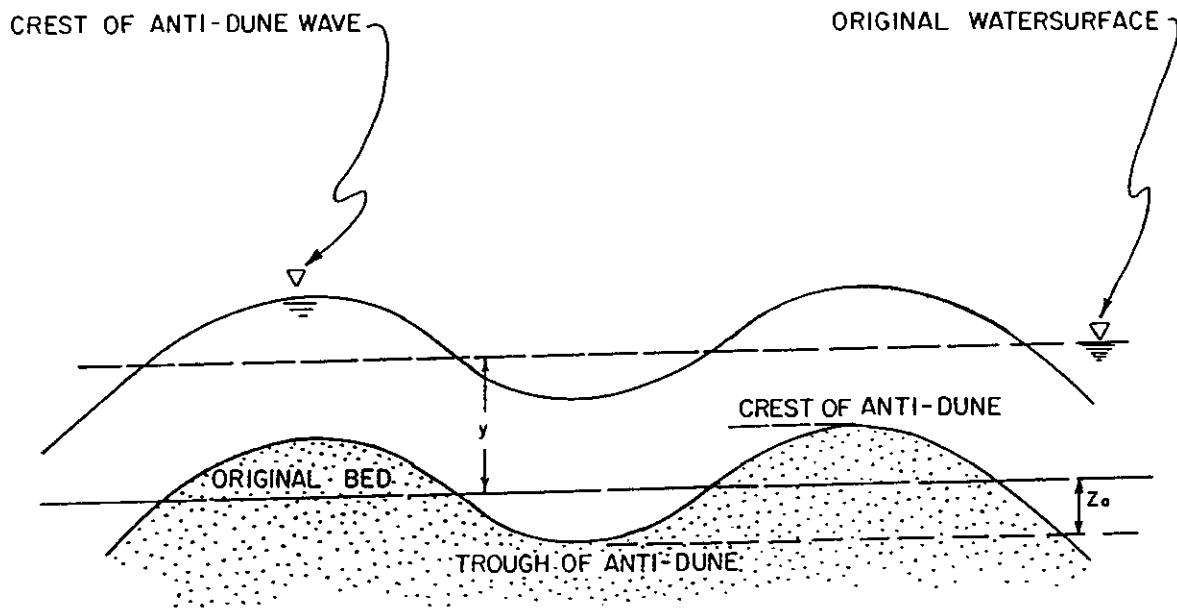


FIGURE 6.2  
DEFINITION SKETCH FOR ANTI-DUNE TROUGH DEPTH

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When the ratio of the flow width to the flow depth of a channel is greater than 1.15 times the average velocity of flow for the 100-year discharge, a low-flow thalweg must be included in all scour calculations. When the flow width or flow depth exceeds the top width and bank heights of the channel, use the top width and flow depth at bank-full conditions, instead of the actual flow width and flow depth. Presently, there is no known methodology for predicting low-flow thalweg depth. However, observation of channels in the Tucson area has revealed that low-flow thalwegs are normally one to two feet deep. Therefore, if a low-flow thalweg is predicted to be present, it should be assumed to be at least two feet deep within regional watercourses, and at least one foot deep within all other watercourses, unless field observations dictate otherwise.

### 6.6.4 Bend Scour

Bend scour normally occurs along the outside of bends, and is caused by spiral, transverse currents which form within the flow as the water moves around the bend. Presently, there is no single procedure which will consistently and accurately predict bend scour over a wide range of hydraulic conditions. However, the following relationship has been developed by Zeller (1981) for estimating bend scour in sand-bed channels based upon the assumption of the maintenance of constant stream power within the channel bend:

$$Z_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left[ 2.1 \left[ \frac{\sin^2(\alpha/2)}{\cos \alpha} \right]^{0.2} - 1 \right] \quad (6.6)$$

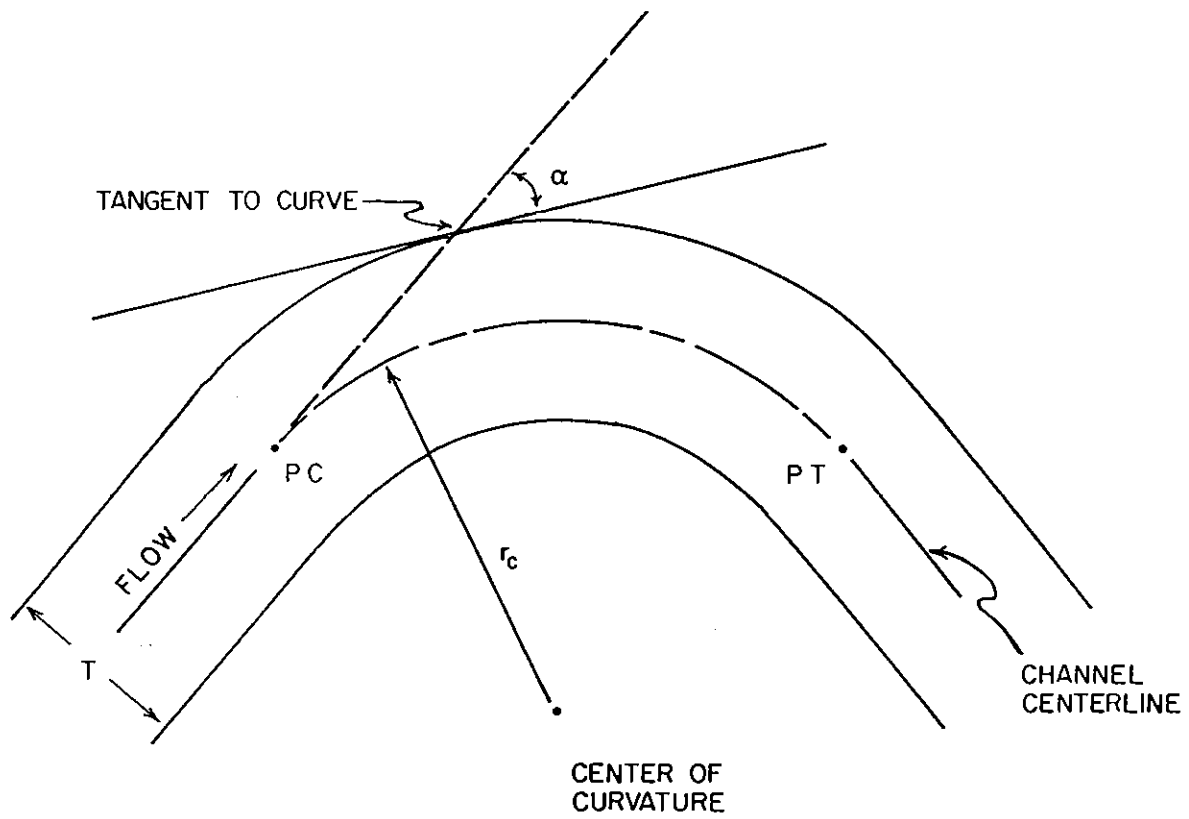
Where:

- $Z_{bs}$  = Bend-scour component of total scour depth, in feet;  
           = 0 when  $r_c/T \geq 10.0$ , or  $\alpha \leq 17.8^\circ$   
           = computed value when  $0.5 < r_c/T < 10.0$ , or  $17.8^\circ < \alpha < 60^\circ$   
           = computed value at  $\alpha = 60^\circ$  when  $r_c/T \leq 0.5$ , or  $\alpha \geq 60^\circ$
- $V_m$  = Average velocity of flow immediately upstream of bend, in feet per second;
- $Y_{max}$  = *Maximum* depth of flow immediately upstream of bend, in feet;
- $Y_h$  = Hydraulic depth of flow immediately upstream of bend, in feet;
- $S_e$  = Energy slope immediately upstream of bend (or bed slope for uniform-flow conditions), in feet per foot; and,
- $\alpha$  = Angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel, in degrees (see Figure 6.3).

NOTE: Mathematically, it can be shown that, for a simple circular curve, the following relationship exists between  $\alpha$  and the ratio of the centerline radius of curvature,  $r_c$ , to channel top width,  $T$ .

$$\frac{r_c}{T} = \frac{\cos \alpha}{4 \sin^2(\alpha/2)} \quad (6.7)$$

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PT = Downstream point of tangency to the centerline radius of curvature.  
PC = Upstream point of curvature at the centerline radius of curvature.

FIGURE 6.3  
ILLUSTRATION OF TERMINOLOGY FOR BEND-SCOUR CALCULATIONS

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Where:

- $r_c$  = Radius of curvature along centerline of channel, in feet; and,  
 $T$  = Channel top width, in feet.

If the bend deviates significantly from a simple circular curve, the curve should be divided into a series of circular curves, and the bend scour computed for each segment should be based upon the angle  $\alpha$  applicable to that segment.

Equation 6.6 can be applied to obtain an approximation of the scour depth that can be expected in a bend during a specific water discharge. The impact that other simultaneously occurring phenomena such as sand waves, local scour, long-term degradation, etc., might have upon bend scour is not known for certain, given the present state of the art. Therefore, in order that the maximum scour in a bend not be underestimated, it is recommended that bend scour be considered as an independent channel adjustment that should be added to those adjustments computed for long-term degradation, general scour, and sand-wave troughs.

The longitudinal extent of the bend-scour component is as difficult to quantify as the vertical extent. Rozovskii (1961) developed an expression for predicting the distance from the end of a bend at which the secondary currents will have decayed to a negligible magnitude. This relationship, in a simplified form, can be expressed as:

$$x = \frac{0.6}{n} Y^{1.17} \quad (6.8)$$

Where:

- $x$  = Distance from the end of channel curvature (point of tangency, PT) to the downstream point at which secondary currents have dissipated, in feet;  
 $n$  = Manning's roughness coefficient;  
 $g$  = Acceleration due to gravity, 32.2 ft/sec<sup>2</sup>; and,  
 $Y$  = Depth of flow (to be conservative, use maximum depth of flow, exclusive of scour, within the bend), in feet.

Equation 6.8 should be used for determining the distance downstream of a curve that secondary currents will continue to be effective in producing bend scour. As a conservative estimate of the longitudinal extent of bend scour, both through and downstream of the curve, it would be advisable to consider bend scour as commencing at the upstream point of curvature (PC), and extending a distance  $x$  (computed with Equation 6.8) beyond the downstream point of tangency (PT).

### 6.6.5 Local Scour

Local scour occurs whenever there is an abrupt change in the direction of flow. Abrupt changes in flow direction can be caused by obstructions to flow, such as bridge piers or abrupt contractions at bridge abutments.

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The depth of scour at bridge piers is highly dependent upon the shape of the pier. Figure 6.4 gives several common pier shapes. A square-nosed pier causes the deepest scour. The depth of scour caused by a square-nosed pier is computed from (Richardson et al., 1975):

$$Z_{lsp} = 2.2 Y \left[ \frac{b_p}{Y} \right]^{0.65} F_u^{0.43} \quad (6.9)$$

Where:

- $Z_{lsp}$  = Local scour depth due to pier, in feet;
- $Y$  = Flow depth, in feet;
- $b_p$  = Pier width normal to the flow direction, in feet; and,
- $F_u$  = Upstream Froude number.

Table 6.1 can be used for computing the reduction in the depth of pier scour for the various types of piers shown in Figure 6.4.

TABLE 6.1: REDUCTION FACTORS TO BE USED WHEN APPLYING SCOUR FORMULAS FOR SQUARE-NOSED PIERS TO OTHER SHAPES (assuming equally projected widths of piers)	
Type of Pier	Reduction Factor
Square Nose	1.0
Cylinder	0.9
Round Nose	0.9
Sharp Nose	0.8
Group of Cylinders	0.9

Scour is reduced if the pier is streamlined in the direction of flow. However, many watercourses transport significant amounts of debris during large floods. Such debris can become impaled upon bridge piers, leading to an increase in the pier-width component,  $b_p$ , found in Equation 6.9. Therefore, in instances where significant debris

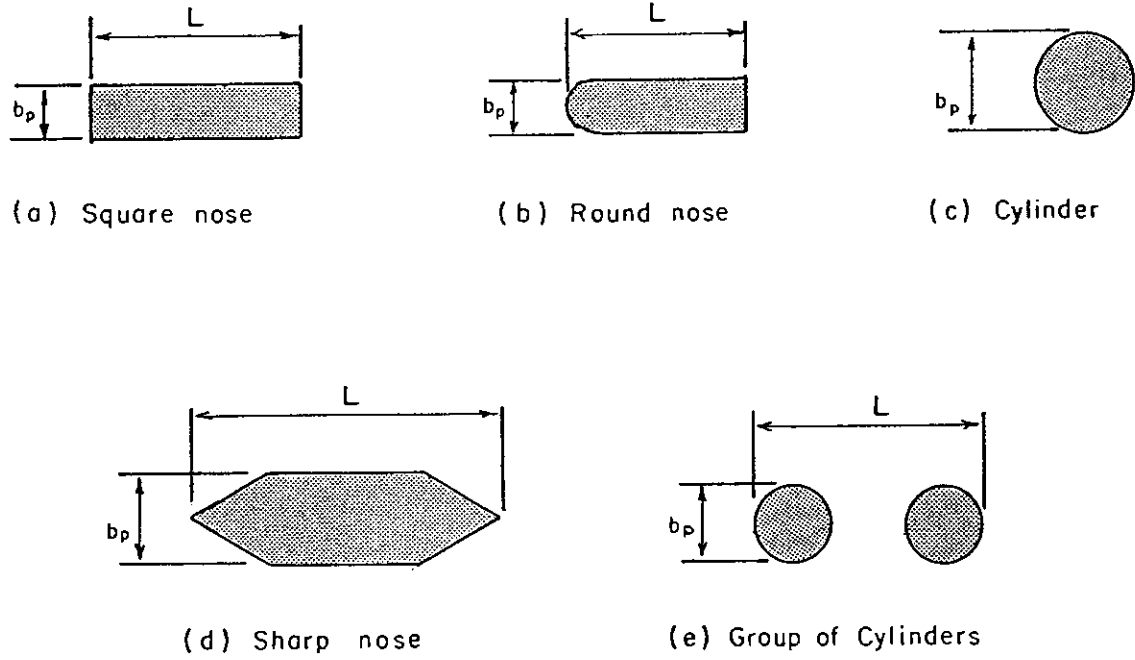


FIGURE 6.4  
COMMON PIER SHAPES

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transport is anticipated (e.g., within regional watercourses),  $b_p$  should be assumed equal to a width of five feet or  $1.5 b_p$ , whichever value is greater. Additionally, pier scour will increase significantly as the direction of flow at the pier becomes more and more skewed in relationship to the pier wall. In such instances, an effective pier width,  $b_{pe}$ , can be calculated from the following equation and substituted into Equation 6.9 in lieu of  $b_p$ .

$$b_{pe} = L \sin \phi_p + b_p \cos \phi_p \quad (6.10)$$

Where:

- $b_{pe}$  = Effective pier width, in feet;
- $L$  = Length of pier wall, in feet;
- $\phi_p$  = Angle of approach flow in relationship to pier wall, in degrees ( $\phi_p = 0^\circ$  for cylindrical piers); and,
- $b_p$  = As defined in Equation 6.9.

In Equation 6.10,  $b_p$  should incorporate any width increase due to debris, where applicable.

Local scour caused by embankments projecting into the flow, such as at bridge abutments, fill projections, and overbank levees, can be computed from the following equation:

$$Z_{lse} = 2.15 \sin(\theta_a) Y \left[ \frac{a_e}{Y} \right]^{0.4} F_u^{0.33} \quad (6.11)$$

Where:

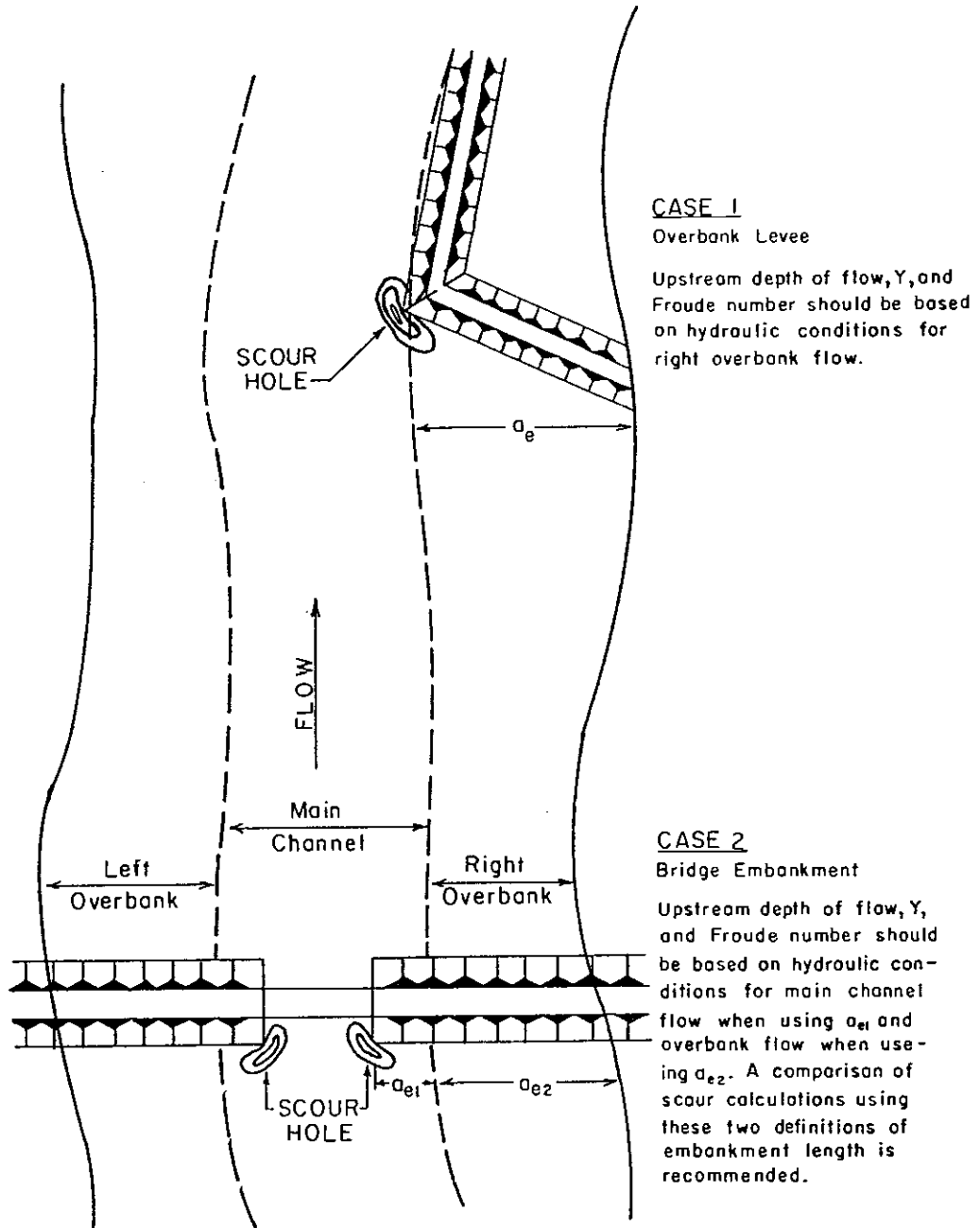
- $Z_{lse}$  = Local scour depth due to embankment, in feet;
- $\theta_a$  = Slope angle of abutment face, measured from the horizontal, in degrees;
- $Y$  = Upstream normal flow depth, in feet;
- $a_e$  = Embankment or encroachment length, measured normal to the edge of the floodplain or channel bank, in feet (see Figure 6.5); and,
- $F_u$  = Upstream Froude number.

For embankments where the quantity  $a_e/Y$  is exceedingly large, such that  $Z_{lse}/YF_u^{0.33} \geq 4.0$ , the following equation (Richardson et al., 1975) should be used in lieu of Equation 6.11:

$$Z_{lse} = 4YF_u^{0.33} \quad (6.12)$$

Equations 6.11 and 6.12 are based upon relationships developed from both empirical observations and experiments in laboratory flumes. As can be seen from the formulas, the scour depth can be significantly affected by embankment length. In

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**FIGURE 6.5**

DEFINITION SKETCH OF EMBANKMENT LENGTH " $a_e$ "

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practical situations, the embankment may span a wide floodplain overbank and extend partially into the main channel itself. Due to the normally large differences which exist between channel and overbank hydraulics, caution must be exercised in defining the embankment length. Figure 6.5 shows a recommended embankment length definition for different cases that might be encountered. In the situation where the embankment crosses the entire overbank and extends into the main channel, it is recommended that the scour be computed by utilizing the overbank hydraulics in combination with the embankment length  $a_{e2}$ , and that this depth of scour then be compared to the scour depth computed by utilizing the main-channel hydraulics in combination with the embankment length  $a_{e1}$ . The larger of the two values should then be used for design purposes.

### 6.6.6 Scour Below Channel Drops

Scour below channel drops, such as grade-control structures, is a special case of local scour. Where the drop consists of a free, unsubmerged overfall, the depth of scour below the drop (U.S. Bureau of Reclamation, 1977) shall be computed from:

$$Z_{1sf} = 1.32 q^{0.54} H_t^{0.225} - TW \quad (6.13)$$

Where:

- $Z_{1sf}$  = Depth of local scour due to a free-overfall drop, in feet, measured below the streambed surface downstream of the drop;
- $q$  = Discharge per unit width of the channel bottom, in cubic feet per second per foot;
- $H_t$  = Total drop in head, measured from the upstream energy grade line to the downstream energy grade line, in feet; and
- $TW$  = Tailwater elevation (downstream water-surface elevation), in feet.

Figure 6.6 shows the relationship of the parameters in Equation 6.13.

Where the drop is submerged, as will be the case for most instances involving grade-control structures placed along watercourses located within the City of Tucson, the depth of scour below the drop (Simons, Li & Associates, 1986) shall be computed from:

$$Z_{1ss} = 0.581q^{0.667}(h/Y)^{0.411}[1-(h/Y)]^{-0.118} \quad (6.14)$$

Where:

- $h/Y \leq 0.99$ ; and,
- $Z_{1ss}$  = Depth of local scour due to a submerged drop, in feet, measured below the streambed surface downstream of the drop;
- $q$  = Discharge per unit width of the channel bottom, in cubic feet per second per foot;
- $h$  = Drop height, in feet; and,
- $Y$  = Downstream depth of flow, in feet.

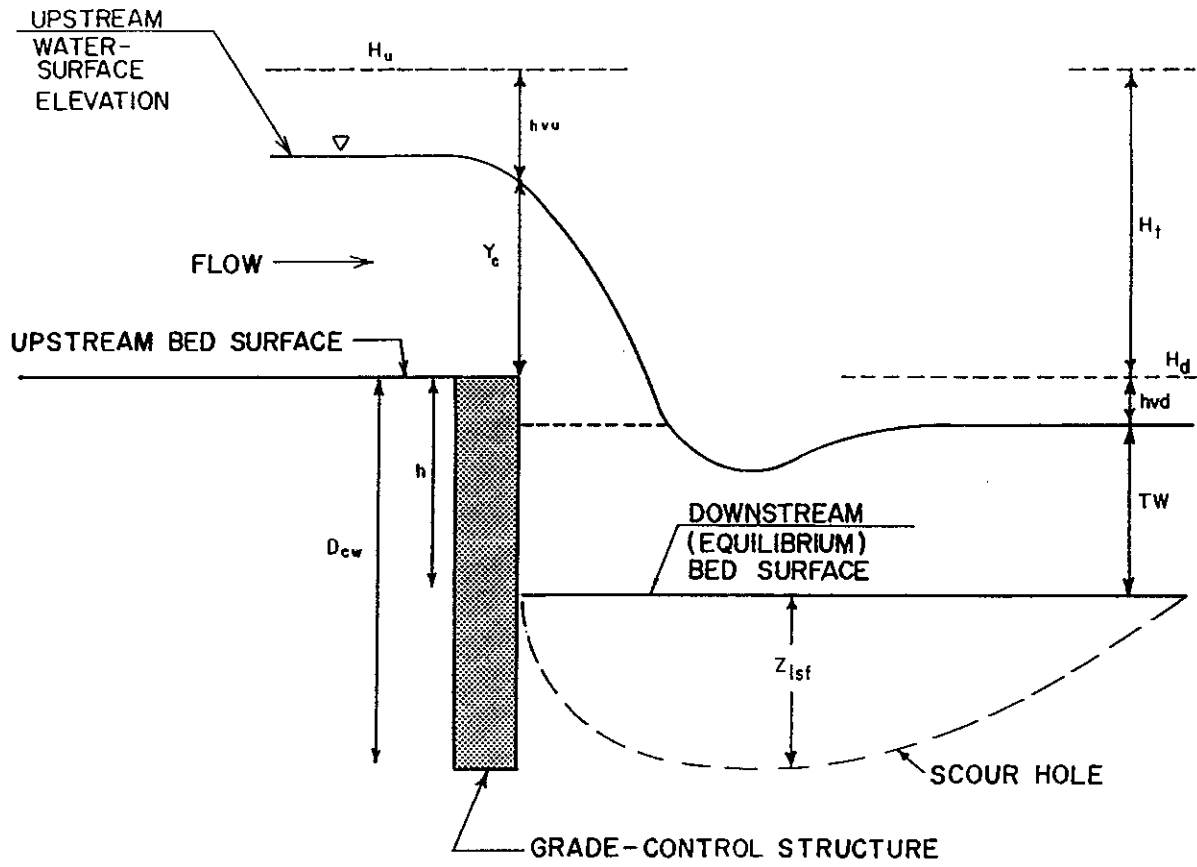


FIGURE 6.6  
DEPTH OF SCOUR BELOW A FREE OVERFALL

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NOTE: If  $h/Y > 0.85$ , the predicted scour below a channel drop should also be computed using Equation 6.13. The smaller of the two values thus computed should then be used for design purposes.

Figure 6.7 gives the relationship of the parameters in Equation 6.14.

The longitudinal extent of a scour hole created by either a free or submerged overfall is represented by the distances  $x_{sce}$  and  $L_s$ , as depicted in Figure 6.7. These dimensions are given by the equations:

$$x_{sce} = 6.0 Z_{lsf}, \text{ or } 6.0 Z_{lsb} \quad (6.15)$$

$$L_s = 12.0 Z_{lsf}, \text{ or } 12.0 Z_{lsb} \quad (6.16)$$

Bank protection toe-downs downstream of a grade-control structure shall extend to the computed depth of scour for a distance equal to  $x_{sce}$  beyond of the grade-control structure, as computed by Equation 6.15. They shall then taper back to the normal toe-down depth within a total distance downstream of the grade-control structure equal to  $L_s$ , as computed by Equation 6.16. Note that  $L_s$  includes  $x_{sce}$ .

In the absence of bridge piers and/or abutments, the depth of scour below grade-control structures is not added to the other scour components. Rather, the depth of scour caused by the grade-control structure is compared to the depth of scour computed by Equation 6.3, and the larger of the two values is then used for toe-down design.

### 6.7 Scour-Hole Geometry at Culvert Outlets

Culverts normally have less cross-sectional area available for the conveyance of flow than do the natural channels they replace. Consequently, flow velocities are increased and a potential for erosion is created at the culvert outlet. Often there is a drop at the culvert outlet, either under design conditions or as a result of outlet scour, and this further increases the possibility of outlet scour. The scour hole created at the outlet of a culvert can become large enough to threaten the culvert, the roadway, adjacent property, or other nearby improvements.

For non-cohesive soils, the dimensions of a scour hole downstream of a culvert outlet *where no drop exists* can be computed by:

$$DSG = \alpha \left( \frac{Q_r}{g^{1/2} D^{5/2}} \right)^\beta (0.09)^\theta \quad (6.17)$$

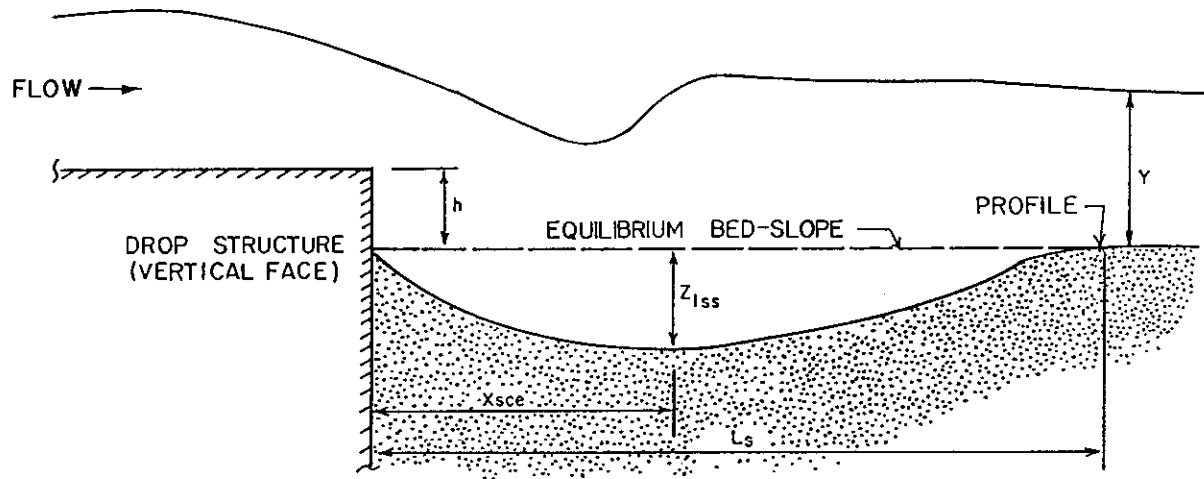


FIGURE 6.7  
 RELATIONSHIP OF VARIABLES IN EQUATION FOR SCOUR BELOW  
 A SUBMERGED GRADE-CONTROL STRUCTURE

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Where:

$$DSG = \text{Dimensionless scour geometry} = \frac{Z_{1sc}}{D}, \frac{W_{sc}}{D}, \frac{L_{sc}}{D} \text{ or } \frac{V_{sc}}{D} ; \text{ and,}$$

$Z_{1sc}$	=	Depth of scour hole below culvert, in feet;
$W_{sc}$	=	Width of scour hole below culvert, in feet;
$L_{sc}$	=	Length of scour hole below culvert, in feet;
$V_{sc}$	=	Volume of scour hole below culvert, in cubic feet;
$D$	=	Culvert diameter, in feet;
$Q_r$	=	Representative discharge, in cubic feet per second;
$g$	=	Acceleration due to gravity (32.2 ft/sec <sup>2</sup> ); and,
$\alpha, \alpha_e, \beta, \theta$	=	Empirically derived coefficients (see Table 6.2).

The representative discharge is the average maximum discharge that can be expected to occur within a thirty-minute time period during the storm runoff event which is selected for design. In the City of Tucson, the design discharge is the 100-year flood. The representative discharge is calculated by:

$$Q_r = \frac{Q_{100}}{2} \left[ 1 + \frac{T_r - 10}{T_r} \right] \quad (6.18)$$

Where:

$Q_r$	=	Representative discharge, in cubic feet per second;
$Q_{100}$	=	100-year peak discharge, in cubic feet per second; and,
$T_r$	=	Hydrograph rise time, in minutes (see Chapter IV, Section 4.5, of this Manual).

For either non-circular or partially-full culverts, the culvert diameter,  $D$ , should be replaced in Equation 6.17 by an equivalent depth,  $Y_e$ , where  $Y_e$  is defined as:

$$Y_e = \left[ \frac{A}{2} \right]^{0.5} \quad (6.19)$$

Where:

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

Equation 6.18 is then modified to the following form:

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TABLE 6.2A: EXPERIMENTAL COEFFICIENTS FOR SCOUR DEPTH,  $Z_{isc}$ , AT CULVERT OUTLETS

MATERIAL	NOMINAL GRAIN SIZE $D_{50}$ (mm)	SCOUR EQUATION	COEFFICIENTS			
			$\alpha$	$\beta$	$\theta$	$\alpha_e$
Uniform Sand	0.20	6.17 or 6.20	2.72	0.375	0.10	2.79
Uniform Sand	2.0	6.17 or 6.20	1.86	0.45	0.09	1.76
Graded Sand	2.0	6.17 or 6.20	1.22	0.85	0.07	0.75
Uniform Gravel	8.0	6.17 or 6.20	1.78	0.45	0.04	1.68
Graded Gravel	8.0	6.17 or 6.20	1.49	0.50	0.03	1.33
<u>Cohesive Sandy Clay:</u>						
60% Sand, PI 15	0.15	6.17 or 6.20	1.86	0.57	0.10	1.53
Clay, PI 5-16	Varies	6.22 or 6.23	0.86	0.18	0.10	1.37

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TABLE 6.2B: EXPERIMENTAL COEFFICIENTS FOR SCOUR WIDTH,  $W_{sc}$ ,  
AT CULVERT OUTLETS

MATERIAL	NOMINAL GRAIN SIZE $D_{50}$ (mm)	SCOUR EQUATION	COEFFICIENTS			
			$\alpha$	$\beta$	$\theta$	$\alpha_e$
Uniform Sand	0.20	6.17 or 6.20	11.73	0.92	0.15	6.44
Uniform Sand	2.0	6.17 or 6.20	8.44	0.57	0.06	6.94
Graded Sand	2.0	6.17 or 6.20	7.25	0.76	0.06	4.78
Uniform Gravel	8.0	6.17 or 6.20	9.13	0.62	0.08	7.08
Graded Gravel	8.0	6.17 or 6.20	8.76	0.89	0.10	4.97
<u>Cohesive Sandy Clay:</u>						
60% Sand, PI 15	0.15	6.17 or 6.20	8.63	0.35	0.07	9.14
Clay, PI 5-16	Varies	6.22 or 6.23	3.55	0.17	0.07	5.63

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TABLE 6.2C: EXPERIMENTAL COEFFICIENTS FOR SCOUR LENGTH,  $L_{sc}$ , AT CULVERT OUTLETS

MATERIAL	NOMINAL GRAIN SIZE $D_{50}$ (mm)	SCOUR EQUATION	COEFFICIENTS			
			$\alpha$	$\beta$	$\theta$	$\alpha_e$
Uniform Sand	0.20	6.17 or 6.20	16.82	0.71	0.125	11.75
Uniform Sand	2.0	6.17 or 6.20	18.28	0.51	0.17	16.10
Graded Sand	2.0	6.17 or 6.20	12.77	0.41	0.04	12.62
Uniform Gravel	8.0	6.17 or 6.20	14.36	0.95	0.12	7.61
Graded Gravel	8.0	6.17 or 6.20	13.09	0.62	0.07	10.15
<u>Cohesive Sandy Clay:</u>						
60% Sand, PI 15	0.15	6.17 or 6.20	15.30	0.43	0.09	14.78
Clay, PI 5-16	Varies	6.22 or 6.23	2.82	0.33	0.09	4.48

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TABLE 6.2D: EXPERIMENTAL COEFFICIENTS FOR SCOUR VOLUME,  $V_{sc}$ , AT CULVERT OUTLETS

MATERIAL	NOMINAL GRAIN SIZE $D_{50}$ (mm)	SCOUR EQUATION	COEFFICIENTS			
			$\alpha$	$\beta$	$\theta$	$\alpha_e$
Uniform Sand	0.20	6.17 or 6.20	203.36	2.0	0.375	80.71
Uniform Sand	2.0	6.17 or 6.20	101.48	1.41	0.34	79.62
Graded Sand	2.0	6.17 or 6.20	36.17	2.09	0.19	12.94
Uniform Gravel	8.0	6.17 or 6.20	65.91	1.86	0.19	12.15
Graded Gravel	8.0	6.17 or 6.20	42.31	2.28	0.17	32.82
<u>Cohesive</u> <u>Sandy Clay:</u>						
60% Sand, PI 15	0.15	6.17 or 6.20	79.73	1.42	0.23	61.84
Clay, PI 5-16	Varies	6.22 or 6.23	0.62	0.93	0.23	2.48

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$$DSG = \alpha_e \left[ \frac{Q_r}{g^{1/2} Y_e^{5/2}} \right]^\beta (0.09)^\theta \quad (6.20)$$

The coefficient  $\alpha_e$  can also be found in Table 6.2.

Bed materials are classified in Table 6.2 as being either uniform or graded. Uniform materials are classified as those for which the standard deviation ( $\sigma$ ) of the grain-size distribution is less than or equal to 1.5. The material is classified as graded if the standard deviation of the grain-size distribution is greater than 1.5. A simple formula often used for computing the standard deviation is:

$$\sigma = \left[ \frac{D_{84}}{D_{16}} \right]^{0.5} \quad (6.21)$$

Where:

- $D_{84}$  = The grain-size diameter for which 84% of the bed material consists of smaller particles; and
- $D_{16}$  = The grain-size diameter for which 16% of the bed material consists of smaller particles.

The grain-size distribution can be determined by a sieve analysis of the bed material. For planning purposes, or in the absence of a sieve analysis, bed material in the City of Tucson should be classified as graded sand, with a median diameter,  $D_{50}$ , equal to one millimeter and  $\sigma = 4.0$ .

If the soil at the culvert outlet is a sandy clay with a mean grain size in the range of 0.10 to 0.20 mm and a plasticity index,  $PI$ , of approximately 15, either Equation 6.17 or 6.20 may be used; where the coefficients for such a soil type are also given in Table 6.2.

Equations 6.17 and 6.20 are not applicable to cohesive soils, which have very different properties than the soil types described above. The potential for scour in cohesive soils is related to the critical shear stress of the soils, and is reflected by Equations 6.22 and 6.23. These equations have a wider range of applicability than do the above expressions. These equations are:

$$DSG = \alpha \left[ \frac{\rho V^2}{\tau_c} \right]^\beta (0.09)^\theta \quad (6.22)$$

For circular culverts, and

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$$DSG = \alpha_e \left( \frac{\rho V^2}{\tau_c} \right)^\beta (0.09)^\theta \quad (6.23)$$

For culverts with other shapes.

Where:

$$\begin{aligned} \frac{\rho V^2}{\tau_c} &= \text{Modified shear number;} \\ V &= \text{Average velocity at outlet, in feet per second;} \\ \tau_c &= \text{Critical tractive shear stress, in} \\ &\text{pounds per square foot; and,} \\ \rho &= \text{Fluid density, in slugs per cubic foot.} \end{aligned}$$

All other terms are as previously defined.

The critical tractive shear stress is defined as:

$$\tau_c = 0.0001 (S_v + 180) \tan (30 + 1.73 PI) \quad (6.24)$$

Where:

$$\begin{aligned} S_v &= \text{Saturated shear strength, in pounds per square inch; and,} \\ PI &= \text{The plasticity index (limits 5-16).} \end{aligned}$$

Equations 6.17 to 6.24 can therefore be used to estimate the dimensions of the scour hole that would form at the outlet of a culvert for varying types of soils. Figures 6.8 and 6.9 should be used to determine the shape of the scour hole. If the scour hole is large enough to threaten nearby improvements, adjacent property, or the culvert itself, outlet protection will be required to contain and/or prevent erosion. The user is referred to a publication by the Federal Highway Administration (1983) for further information regarding the design of culvert outlet protection.

### 6.8 Design of Sediment Basins

On watercourses with a potential for high sediment discharge, sediment basins may be necessary to protect detention basins, culverts, or storm drains from being filled with sediment. If it is felt that sedimentation could pose a problem for a proposed structure, basins should be built to collect and hold sediment for later removal by maintenance personnel. The design of these basins on watercourses where the upstream watershed area is one square mile, or less, shall be in accordance with the guidelines as presented within Section 3.4 of the Pima County and City of Tucson Stormwater Detention/Retention Manual (1987).

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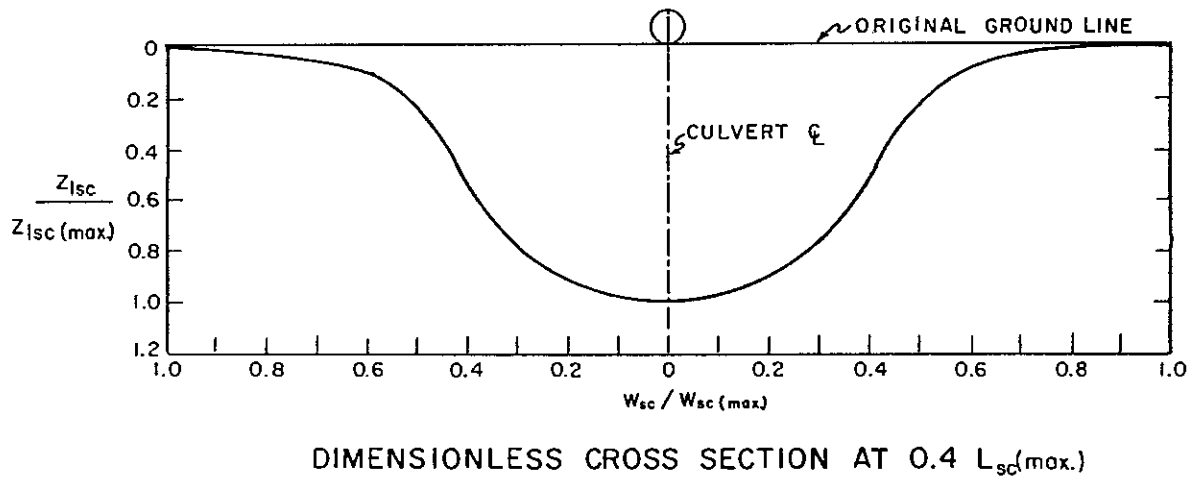
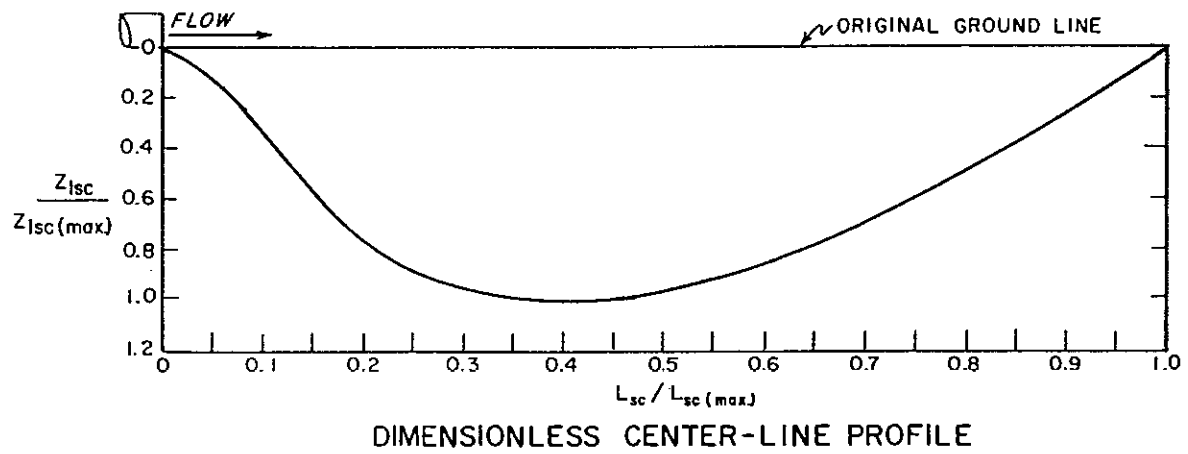


FIGURE 6.8  
 DIMENSIONLESS SCOUR-HOLE GEOMETRY  
 FOR MAXIMUM TAILWATER

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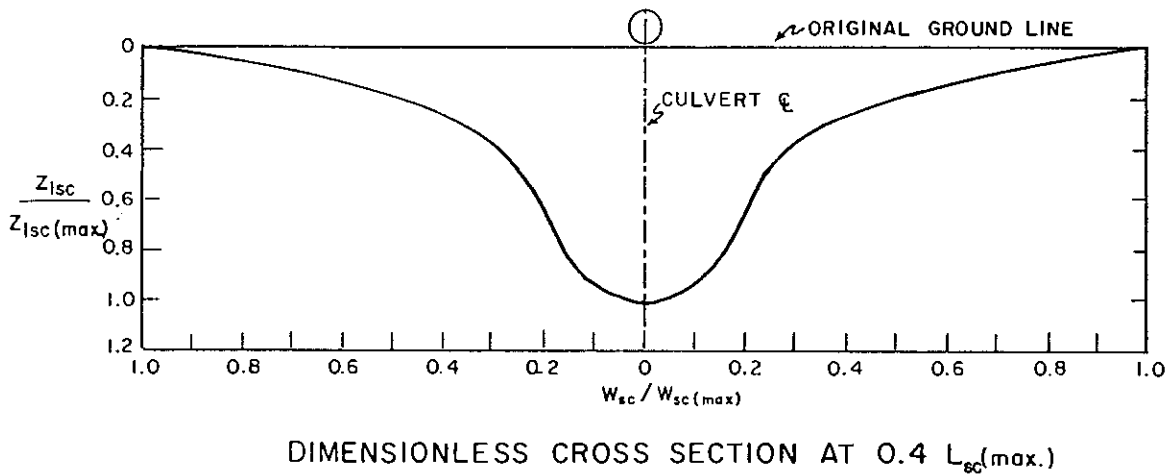
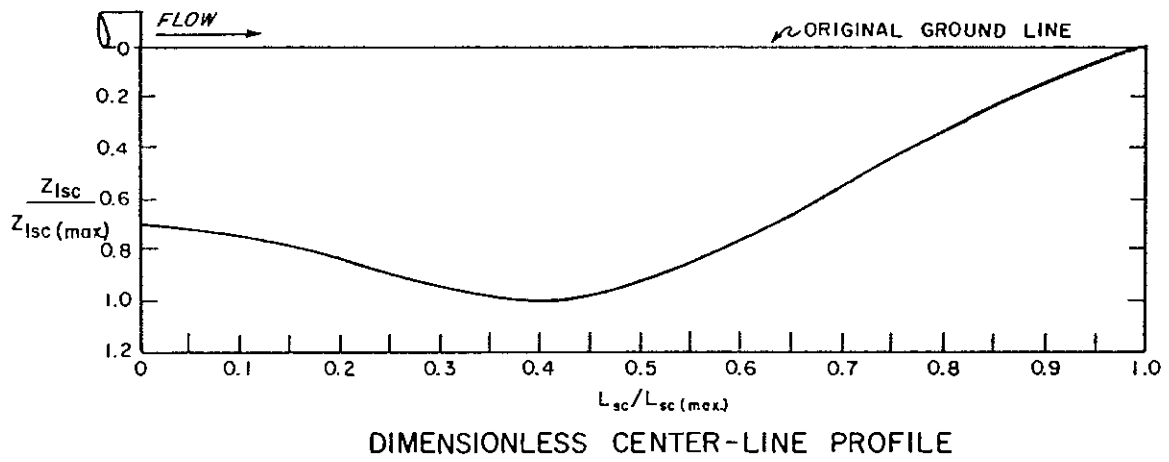


FIGURE 6.9  
 DIMENSIONLESS SCOUR-HOLE GEOMETRY  
 FOR MINIMUM TAILWATER

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On watersheds larger than one square mile, the guidelines cited above may result in overdesign. The design of sediment basins on these watersheds is a more complicated procedure, involving total watershed sediment yield and channel sediment-transport capacity over a range of discharges. Total watershed sediment yield can be estimated by such methods as the Modified Universal Soil Loss Equation (Williams, 1975; and Williams and Berndt, 1977), the Pacific Southwest Inter-Agency Committee (PSIAC) Method (Pacific Southwest Inter-Agency Committee, 1968), the Flaxman Method (Flaxman, 1972), the SCS Method (U.S. Soil Conservation Service, 1971), the Dendy/Bolton Method (Dendy and Bolton, 1976), and the Renard Method (Renard, 1972). A publication by Renard and Stone (1981) contains a detailed discussion and comparison of some of these methods.

The equations for watershed sediment yield which are listed above do not readily distinguish between sediment production that would be classified as wash load and sediment production that would be classified as bed load. Wash load particles are so small that they would generally remain in suspension as the water passes through the detention basin. Therefore, the wash load is not generally to be considered in sediment basin design. An estimate of wash load, as compared to bed load estimated from equations for total watershed sediment yield, can be made by taking samples of the topsoil throughout the watershed.

Total watershed sediment production may not be an entirely accurate estimate of the amount of sediment that would be delivered to a certain point, because there is sediment storage within the watershed system. Sediment-volume estimates must therefore also consider the sediment-transport capacity of the channel. A detailed discussion of this type of analysis will not be presented here. However, the reader is referred to publications by the U.S. Army Corps of Engineers (1977), Simons, Li & Associates (1982, 1985), the American Society of Civil Engineers (1977), Simons and Senturk (1977), and Zeller and Fullerton (1983) for more detailed information about performing such analyses.

### 6.9 Equilibrium Slopes within Constructed Channels

Given a fixed size distribution of sediments, the sediment-transport capacity of a stream is dependent primarily upon flow velocity and depth. Within the City of Tucson, transport of all particle sizes of bed material increases, as flow velocity increases, at a rate proportional to approximately the third to fifth power of the velocity. Correspondingly, transport of sediment particles composed of bed material generally decreases as depth increases, while transport increases with decreased depth. However, flow velocity is by far the more important variable.

For purposes of analysis and design, most natural, undisturbed channels in the Tucson area can be assumed to be at or near a state of dynamic equilibrium with regard to sediment transport. This means that, for a given reach of the channel, the sediment-transport capacity of the channel, over the long term, is more or less equal to the sediment supply. The channel bed slope is therefore "stable."

When channelization occurs, the channel top width is often narrowed, and channel roughness is normally decreased. The result is an increase in velocity and depth, with

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a corresponding increase in sediment-transport capacity. Sediment-transport capacity then exceeds the sediment supply; and, if the bed is composed of sediment that can be transported, the deficiency will be made up from bed material--causing the channel to degrade. Another factor that contributes to this degradation is upstream urbanization. Urbanization increases flood peaks, which also lead to higher flow velocities and depths. Urbanization also reduces the watershed sediment supply, and increases the frequency of runoff. The result of all these occurrences is that channel bed degradation will occur until the channel slope is flat enough to cause the sediment-transport rate to be equal to the incoming sediment supply. This slope then becomes the new, "stable," equilibrium slope. Streambed degradation can threaten underground improvements, bank-protection toe-downs, culverts, and other hydraulic structures that are within and/or that cross the channel. Grade-control structures, or lining of the channel bed, are usually required in order to prevent damage caused by streambed degradation.

The equilibrium slope for a channel which has an upstream sediment supply that is considered to be essentially zero (e.g., a channel located within a highly urbanized watershed) can be computed from:

$$S_{eq} = \left( \frac{1.45n}{q^{0.11}} \right)^2 \quad (6.25)$$

Where:

- $S_{eq}$  = Equilibrium slope after urbanization, in feet per foot;
- $n$  = Manning's roughness coefficient; and,
- $q$  = Channel unit discharge, in cubic feet per second per foot.

For use with Equation 6.25, channel unit discharge is defined as the channel discharge divided by the channel bottom width. Use of this equation will produce the flattest slope that can be reasonably expected to transport sediment within channels located in the Tucson area. The discharge associated with a 10-year flood is normally chosen when computing the unit discharge for use in Equation 6.25.

For lesser degrees of urbanization, the equilibrium slope is computed from Equation 6.26, which is a generalization of the theoretically derived sediment-transport relationships for sandbed channels developed by Zeller and Fullerton (1983):

$$S_{eq} = \left[ \left[ \frac{n_u}{n_n} \right]^2 \left[ \frac{Q_{u,10}}{Q_{n,10}} \right]^{-1.1} \left[ \frac{b_u}{b_n} \right]^{0.4} (1-R_s)^{0.7} \right] S_n \quad (6.26)$$

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Where:

- $n_u$  = Manning's roughness coefficient for an urban channel;
- $n_n$  = Manning's roughness coefficient for a natural or existing channel;
- $Q_{u,10}$  = Ten-year discharge, under urbanized conditions, in cubic feet per second;
- $Q_{n,10}$  = Ten-year or bank-full discharge (whichever is less), under natural conditions, in cubic feet per second;
- $b_u$  = Bottom width of channel, under urbanized conditions, in feet;
- $b_n$  = Bottom width of channel, under natural conditions, in feet;
- $R_s$  = Reduction factor for sediment supply. This factor is usually assumed to be equal to the ratio of the impervious area to the total area of the upstream watershed (i.e.,  $0.0 \leq R_s \leq 1.0$ ); and,
- $S_n$  = Natural or existing channel slope, in feet per foot.

The roughness coefficients for natural and urbanized channel beds are often very nearly the same, so the term in which these coefficients appear in Equation 6.26 can usually be assumed equal to the value 1.0. However, from time to time exceptions to this assumption may occur. For instance, when the existing channel is a wide, flat, sheetflow watercourse; and the proposed channel is a narrow, sand-bed channel,  $n_u$  will ordinarily not be equal to  $n_n$ .

For moderately urbanized to highly urbanized watersheds, the equilibrium slope should be computed by using both Equation 6.25 and Equation 6.26. The steeper of the two computed slopes should then be used for design. The reason for this is that Equation 6.26 can sometimes produce slope values that are too flat to generate reasonable sediment-transport rates for maintenance of channel stability, when impervious cover within a watershed is very high.

Equation 6.26 should be used with caution within the City of Tucson. An underlying assumption of this equation is that the existing or natural channel is itself in equilibrium. This is not always true in the City, because most channels have undergone alteration. If there is any question as to whether or not the existing channel is in equilibrium, it is best to try and determine through old (pre-development) aerial photographs and topography what the channel characteristics were in its original, undisturbed (i.e., natural) state. In the absence of historical information about the original channel, an examination may be made of existing stable channels in the area to help estimate what the channel in question may have looked like before urbanization.

Equation 6.26 can be used for more than merely the quantification of streambed degradation. It can also be used to determine whether aggradation will occur when a channel is widened beyond existing or natural conditions. Another application would be to use it to design a stable channel cross-section in lieu of installing grade-control structures to otherwise control degradation of the channel bed.

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### 6.10 Spacing and Depth of Grade-Control Structures

If the equilibrium slope of a channel, as determined by use of either Equation 6.25 or Equation 6.26, is flatter than the design slope, grade-control structures may be needed to limit degradation from exceeding a certain depth at any point along the channel. Grade-control structures, sometimes called "cut-off walls" or "check dams," are non-erodible vertical barriers in the channel that prevent the channel bed from degrading at a point located immediately upstream of where they are located. After the channel bed has reached equilibrium, the bed elevation immediately upstream of the grade-control structure is at the design elevation. Downstream of the grade-control structure, the bed is at an "equilibrium" elevation that is lower than the design elevation. For most channels, the design of grade-control structures is an iterative process, involving drop height, reach length, and depth of scour downstream of the drop.

Once a drop height is chosen, the reach length, or spacing, between adjacent structures can be computed from:

$$L_r = \frac{h}{S_{ib} - S_{eq}} \quad (6.27)$$

Where:

- $L_r$  = Reach length, or spacing, between adjacent grade-control structures, in feet;
- $h$  = Drop height downstream of the grade-control structure, in feet;
- $S_{ib}$  = Initial channel bed slope, in feet per foot; and,
- $S_{eq}$  = Channelized equilibrium bed slope, in feet per foot.

If the initial and final bed slopes are approximately the same, the distance between grade-control structures will be very large. Under these circumstances, such structures may not be required.

Normally, the drop height downstream of a grade-control structure which consists of poured concrete without reinforcements shall not exceed two feet; and preferably should be only one foot, where feasible. For economical and technical reasons, grade-control structures should be spaced no closer together than twelve times the local scour depth below the grade-control structures, as computed by the use of either Equation 6.13 or Equation 6.14.

The total height of a cut-off wall or a grade-control structure ( $D_{cw}$ ), from top to toe, shall not be less than the drop height plus the computed depth of scour below the wall or structure (see Figure 6.6). The depth of scour below grade-control structures should be computed according to the guidelines presented in Section 6.6.6 of this Manual. For a one-foot-wide, unreinforced concrete cut-off wall, if structural calculations support same, the maximum allowable height of a cut-off wall, from top to toe, can be six feet. If the depth of scour plus the drop height is greater than six feet, the drop shall be considered to be too great for unreinforced concrete cut-off walls, unless a structural analysis can demonstrate otherwise, and the spacing between

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the cut-off walls must be reduced. The example which follows (i.e., Example 6.1), illustrates the recommended procedure for cut-off wall design.

There will be many design situations, especially when unit discharges are high, where a cut-off wall with a height of six feet, from top to toe, is not sufficient. In such cases, a reinforced concrete cut-off wall that has a height greater than six feet, from top to toe, may be used, provided that a structural analysis is submitted showing that the proposed cut-off wall will be structurally stable. If a structural analysis is submitted and approved, the maximum drop height of two feet will no longer apply.

Grade-control structures for large discharges need not necessarily be vertical on the downstream side. For structural stability, a triangular or wedge-shaped soil-cement grade-control structure is recommended for use on regional watercourses. However, for hydraulic reasons, the use of any grade-control structure with a face flatter than 1:1 on the downstream side shall not be permitted without prior written approval from the City Engineer.

### EXAMPLE 6.1: SPACING AND DEPTH OF GRADE-CONTROL STRUCTURES

A channel in a highly urbanized watershed is to be built to contain the 100-year-flood discharge. The sides of the channel are to be of shotcrete, the bottom of earth.

Channel characteristics are as follows:

Bottom Width	= 20 feet
Design Slope	= 0.006 feet/foot
Side Slopes	= 1:1
Manning's "n"	= 0.022

Hydraulic characteristics are as follows:

$Q_{100} = 700$ cfs	$Q_{10} = 350$ cfs
$Y_{100} = 3.1$ feet	$Y_{10} = 2.1$ feet
$V_{100} = 9.7$ fps	$V_{10} = 7.7$ fps
$q_{100} = 35.0$ cfs/foot	$q_{10} = 17.5$ cfs/foot

Because the watershed is highly urbanized, Equation 6.25 will be used to compute the equilibrium slope. Therefore:

$$S_{eq} = \left[ \frac{1.45 (0.022)}{(17.5)^{0.11}} \right]^2 = 0.0005 \text{ feet/foot.}$$

Assume a two-foot drop height. From Equation 6.27, the spacing between grade-control structures should be:

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$$L_r = \frac{2.0}{(0.006)-(0.0005)} = 364 \text{ feet.}$$

The grade-control structure will be submerged. Using Equation 6.14 yields:

$$Z_{1ss} = 0.581(q_{100})^{0.667}(h/Y_{100})^{0.411}[1-(h/Y_{100})]^{-0.118};$$

$$q_{100} = 35.0 \text{ cfs;}$$

$$Z_{1ss} = 0.581(35.0)^{0.667}(0.645)^{0.411}(0.355)^{-0.118}; \text{ so,}$$

$$Z_{1ss} = 5.9 \text{ feet.}$$

Therefore, the total height of the grade-control structure, from top to toe, should be 5.9 feet *plus* the two-foot drop height; or, 7.9 feet (round to 8.0 feet).

However, it is desirable to keep the total vertical dimension of the grade-control structure, from top to toe, equal to or less than six feet. Therefore, a smaller drop height should be used.

Using a drop height of one foot yields:

$$L_r = \frac{1.0}{(0.006)-(0.0005)} = 182 \text{ feet.}$$

$$Z_{1ss} = 0.581(35.0)^{0.667}(0.323)^{0.411}(0.677)^{-0.118}; \text{ so,}$$

$$Z_{1ss} = 4.10 \text{ feet (round to 4.0 ft).}$$

Since, in this example, the ultimate drop height at the downstream side of a grade-control structure will be set at one foot, cut-off walls with a height of five feet, from top to toe, could be placed at approximately 180-foot intervals along the bottom of the channel to serve as grade-control structures in order to limit long-term bed degradation to a maximum of one foot anywhere along the subject channel.