PIMA COUNTY
WASTEWATER RECLAMATION

APPENDIX A
SCOUR PROCEDURES AND GUIDELINES FOR SANITARY SEWER CROSSINGS OF ALLUVIAL WATERCOURSES

Engineering Design Standards

SCOUR PROCEDURES AND GUIDELINES FOR SANITARY SEWER CROSSINGS OF ALLUVIAL WATERCOURSES

PREPARED FOR:

PIMA COUNTY REGIONAL WASTEWATER AND RECLAMATION DEPARTMENT

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SCOUR PROCEDURES AND GUIDELINES

I. BACKGROUND

This report presents scour procedures and guidelines for the computation of scour/erosion and lateral migration at sanitary sewer crossings of alluvial watercourses.

Over the past 20 years, advancements in the fields of hydraulic engineering, scour/erosion assessment, and sediment-transport technology have provided improved generalized methods for computing the components of scour. Historically, local design standards available for computing scour/erosion depths and lateral-migration limits of sanitary sewer and other types of pipeline crossings of alluvial watercourses have been found to provide reasonable results when all components of the scour/erosion and lateral-migration processes are considered and when they are properly applied to each individual scenario. The difficulty is that, occasionally, within the environs of central and southern Arizona in particular, hydraulic conditions which prevail in a given watercourse may fall outside of the envelope within the set of standard scour equations that were originally developed, and thus publicized procedures may not be strictly applicable under all ranges of conditions. This document is intended to provide such an approach to the computation of scour/erosion and lateral migration—one that would not only have applicability within the urban/metropolitan areas of Pima County, but would also apply in a more regional fashion to the majority of alluvial watercourses located in rural Pima County, as well as in the semi-arid and arid desert areas of Arizona, in general.

II. CONDITIONS OF USE

This simplified hydraulic design procedure contains technical concepts and guidelines to aid the engineer in the planning and design of pipelines to be placed across and/or parallel to alluvial watercourses. This procedure has been developed for use only with alluvial watercourses that are primarily composed of sands and gravels, and that are subject to erosion with a regulatory (i.e., 100-year) peak discharge of less than 10,000 cfs ($Q_{p100} < 10,000$ cfs).

When used in this type of conditions, the procedure presented herein should produce adequate results. However, site-specific evaluations, which encompass more detailed analyses and consider ALL local factors, are recommended whenever and wherever possible. This procedure has been developed using both sound engineering judgment and the best information currently available. Nonetheless, this procedure should only be accepted and used by the recipient individual or group entity with the express understanding that Pima County and/or the Authors of this procedure make no warranties, either express or implied, concerning the accuracy, reliability, usability, or suitability for any particular purpose of the information contained herein. As such, Pima County and/or the Authors of this procedure assume no liability, whatsoever, to any such individual or group entity by reason of any use made thereof. Furthermore, use of the procedure shall not be presumed to be the best engineering practice for any site-specific situation.

III. FOUR-STEP INITIAL EVALUATION PROCESS

Before beginning any scour/erosion assessment, the four-step initial-evaluation process which is described below should be conducted. This process is intended to assist the user in determining whether he or she can prepare his or her own scour evaluation; or, whether there needs to be a consultant retained to prepare the investigation who is an expert in the field of erosion and sediment transport. Please note that the following section is only a guideline meant to assist the user in determining the risk associated with computing his or her own scour estimates for sanitary sewer crossings of alluvial watercourses.

The Project Risk (R) can be evaluated by the equation, $R = R_1 + R_2 + R_3$, as follows:

Step 1) What is the risk/expectation of vegetal growth (R_1) in areas that are environmentally restricted, particularly with regard to maintenance concerns? (Scale from 1-10). Example:

- Low—shallow weeds/grasses within maintained channels, $R_1 = 1$,
- High—shrubs/trees growing in environmentally protected watercourses, $R_1 = 10$.

Step 2) What is the Social/Economical Risk (R_2) that is associated with failure of a sanitary sewer? (Scale from 1-10). Example:

- Low Contamination/Cost—Improvements within Small Service Area, $R_2 = 1$.
- High Contamination/Cost—Improvements within Regional Service Area, $R_2 = 10$.

Step 3) Local Risk Factors (R_3) within the proximity of the sanitary sewer are equal to the sum of the associated risks as presented in the table below, where, $R_3 = \sum R_i$:

Table 1: Factors That influence Local-Erosion Risk Evaluation, (Scale from 0-10)

Local Risk within proximity of Sanitary Sewer (R_i)	Point Value
Flow Contraction Due to Bridge Abutment Encroachments, (R_{BE})	0 or 1
Bridge Piers, (R_{BP})	0 or 1
Presence of Drop Over a Rigid Grade-Control Upstream of Crossing, (R_D)	0 or 1
Sanitary Sewer Partially or Fully Exposed to Flow, (R_P)	0 or 2
Presence of In-Stream Sand-and-Gravel Mine in Proximity of Sanitary Sewer, (R_M)	0 or 3
Curvature of Watercourse where Sanitary Sewer Crossing Is Located	0, 1 or 2
-If $(r_c/T) \geq 10$	0
-If $0.5 < (r_c/T) < 10$	1
-If $(r_c/T) \leq 0.5$	2

Step 4) Upon completion of the preceding three steps, sum R_1 thru R_3, and compare risk level with the table below:

Table 2: Scour/Erosion-Hazard Risk Evaluation

Computed Risk ($R=R_1+R_2+R_3$)	Risk Potential	Recommendation
$0 < R < 10$	Low	Proceed with Caution
$10 \leq R < 20$	Moderate	Experienced Professional Is Recommended
$20 \leq R \leq 30$	High	Experienced Professional and Detailed Investigation Are Recommended

IV. STANDARD SCOUR PROCEDURE

The following text summarizes the scour procedures and the guidelines to be used for evaluating scour/erosion and lateral-migration impacts at sanitary sewer crossings of alluvial watercourses.

4.0 Total Scour

Total scour potential (including future conditions) is loosely defined as the sum of the individual scour components described in the following subsections of this document. All recommended sanitary sewer cover depths are to be measured from the thalweg (i.e., the lowest elevation in the streambed) when the sanitary sewer is placed at the watercourse crossing. If the crossing is braided, then the burial depth is to be measured from the thalweg of the most deeply entrenched thread. As a general rule-of-thumb, the user should recognize that the upper limit of maximum scour (Z_{MAX}) computed should not exceed five times the maximum flow depth (Y_{MAX}); that is $Z_{MAX} \leq 5Y_{MAX}$.

4.1 Maximum Scour Depth

$$Z_{MAX} = Z_{TSE} + Z_{LTD} \qquad \text{Where, } Z_{MAX} \leq 5Y_{MAX} \qquad \text{(Equation 4.1.1)}$$

$$Z_{MAX} = 5Y_{MAX} \qquad \text{Where, } Z_{MAX} > 5Y_{MAX} \qquad \text{(Equation 4.1.2)}$$

Where:

- Z_{TSE} = Total single-event (100-year) scour depth, in feet;
- Z_{LTD} = Long-term aggradation or degradation depth, in feet;
- Y_{MAX} = Maximum depth of flow in channel, in ft;
- Z_{MAX} = Upper limit of maximum predicted scour depth, in feet;

A general equation for computing the single-event scour depth, Z_{TSE}, for a 100-year peak discharge along either a curved or a straight reach of an alluvial watercourse is:

$$Z_{TSE} = C_U(Z_G + Z_A + Z_T + Z_B + Z_C + Z_L) \qquad \text{(Equation 4.1.3)}$$

Where:

- C_U* = Non-uniform scour coefficient to account for flow irregularities (per factor-of-safety from Table 3), unitless;
- Z_{TSE} = Total single-event (100-year) scour depth, in feet;
- Z_G = General scour depth, in feet;
- Z_A = Anti-Dune scour depth, in feet;
- Z_T = Low-flow thalweg depth, in feet;
- Z_B = Bend scour, in feet;
- Z_C = Confluence scour, in feet; and
- Z_L = Local scour depth, in feet

*The C_U coefficient is meant to account for increased unit discharge due to flow irregularities.

Table 3: Non-Uniform Scour Coefficients[1] (C_U)		
Channel Geometry	**Value for Erodible Bed and Banks**	**Value for Erodible Bed and Protected Banks**
Natural/Irregular	1.55	1.75
Parabolic	1.50	1.70
Trapezoidal	1.33	1.55
Rectangular	1.20	1.33

[1]As derived from Blodgett (1986)

4.2.1 General Scour (Z_G):

It is reasonable to assume that general scour processes would be strongly correlated to the unit discharge of flow within the channel of an alluvial watercourse. Therefore, for purposes of this document General Scour, Z_G, has been quantified in terms of unit discharge, q, as shown below:

$$Z_G = 0.293q^{2/3}\left[q^{1/15} - 1.073\right] \qquad \text{(Equation 4.2.1.1)}$$

(If $Z_G < 0.10$ feet, assume $Z_G = 0.10$ feet)

Where:

Z_G = General scour depth, in feet; and,
q = Average peak discharge per unit width of the channel, in cfs/ft.

4.2.2 Anti-Dune Scour (Z_A):

Anti-dunes are bed forms, similarly shaped to dunes, but which move along the streambed in an upstream direction, rather than downstream as do dunes, and move in phase with the surface waves in the channel. Anti-dunes typically form when transitional flow occurs or when the flow in the channel is at near-critical or critical flow conditions—conditions which, by far, are indicative of the most common flow regimes which are found along alluvial watercourses in metropolitan Pima County during major flow events. As noted, the corresponding surface waves are in phase with the anti-dunes, and tend to break like surf when the waves reach a height approximately equal to 0.14 times the length of the wave. From this knowledge, a relationship has been established that relates the average channel flow velocity to the predicted anti-dune scour depth along the streambed. This relationship, excerpted from the *City of Tucson Standards Manual for Drainage Design* (1989, and revised in 1998) is shown below (see Figure 1, Page 5):

$$Z_A = 0.0137V^2 \qquad \text{(Equation 4.2.2.1)}$$

Where:

Z_A = Anti-Dune scour depth, in feet; and
V = Average channel flow velocity, in ft/s.

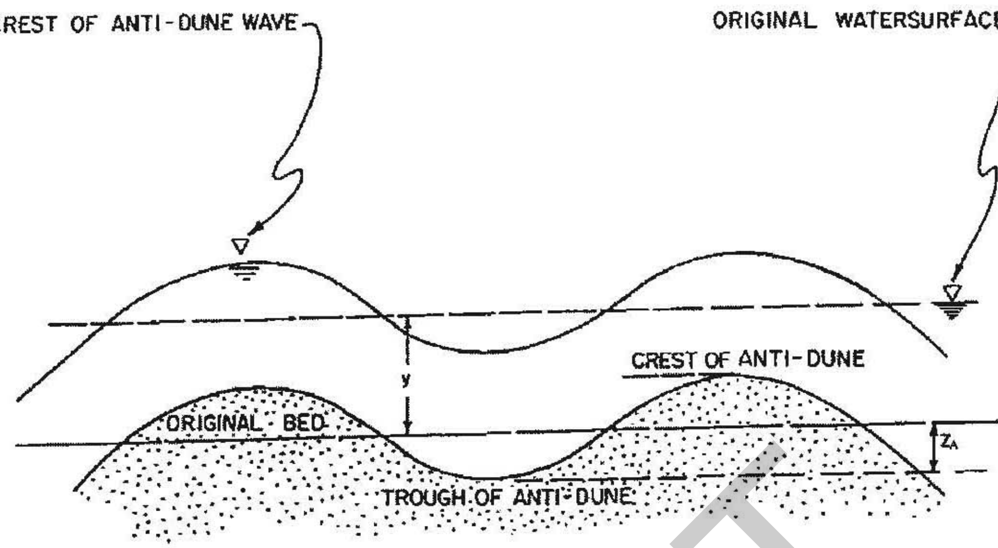

FIGURE 1

When computing Z_A, it is important for the user to keep in mind that the height of an anti-dune can never exceed one-half the maximum depth of flow ($1/2Y_{MAX}$). Accordingly, if Z_A should exceed $1/2Y_{MAX}$, then Z_A should be set equal to $1/2Y_{MAX}$ (i.e., $Z_A = 1/2Y_{MAX}$

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4.2.3 *Low-Flow Thalweg (Z_T):*

The low-flow thalweg is a small "inner channel" which forms in the streambed of the main channel. A low-flow thalweg typically develops when the width-to-depth ratio of the main channel of an alluvial watercourse is large in comparison to the ordinary flows which occur on an annual basis. Depending upon the relationship between flow velocity, V, flow depth, Y, and flow width, W, during a 100-year event, the scour depth to account for a low-flow thalweg (Z_T) follows the relationships adopted from the *City of Tucson Standards Manual for Drainage Design* (1989, and revised in 1998), as shown below:

$Z_T = 0$ $\quad W/Y \leq 1.15V$ (Equation 4.2.3.1)

$Z_T = 1$ $\quad W/Y > 1.15V$ $\quad (DA < 30\ mi^2)$ (Equation 4.2.3.2)

$Z_T = 2$ $\quad W/Y > 1.15V$ $\quad (DA \geq 30\ mi^2)$ (Equation 4.2.3.3)

Where:

Z_T = Low-flow thalweg scour depth, in feet;
W = Flow width, in feet
Y = Average flow depth of channel, in ft; and
V = Average velocity of the channel, in ft/s

Note, however, that if a low-flow thalweg is present at the site of a sanitary sewer crossing, the observed thalweg depth should be used in lieu of results generated by the preceding equations.

4.2.4 Bend Scour (Z_B):

Bend scour occurs along the outside (concave side) of a bend. It is created by spiral, transverse currents which form within the flow as the water moves through the curved alignment in the bend. The following relationships for bend scour, Z_B, are expansions of Equation 4.2.1.1, found in Section 4.2.1 of this document, upon consideration of curvature effects. These relationships are designated for use based upon different magnitudes of the ratio r_c/T, and are an additive component of the total scour, Z_{TSE}.

$$Z_B = 0.243q^{0.733}$$

(use for nearly direct impingement [e.g., right-angle bends)

If $\alpha > 60°$ $[(r_c/T) \le 0.5]$ (Equation 4.2.4.1)

$$Z_B = 0.293q^{0.733}\left(2.1\left[\frac{\sin^2\left(\frac{\alpha}{2}\right)}{\cos\alpha}\right]^{0.2} - 1\right)$$

If $17.75° < \alpha < 60°$ $[0.5 < (r_c/T) < 10]$ (Equation 4.2.4.2)

$$Z_B = 0$$

If $\alpha \le 17.75°$ $[(r_c/T) \ge 10]$ (Equation 4.2.4.3)

The relationship between α and r_c/T is mathematically described below (also, see Figure 2):

$$\frac{r_c}{T} = \frac{\cos\alpha}{4\sin^2\left(\frac{\alpha}{2}\right)}$$

(Equation 4.2.4.4)

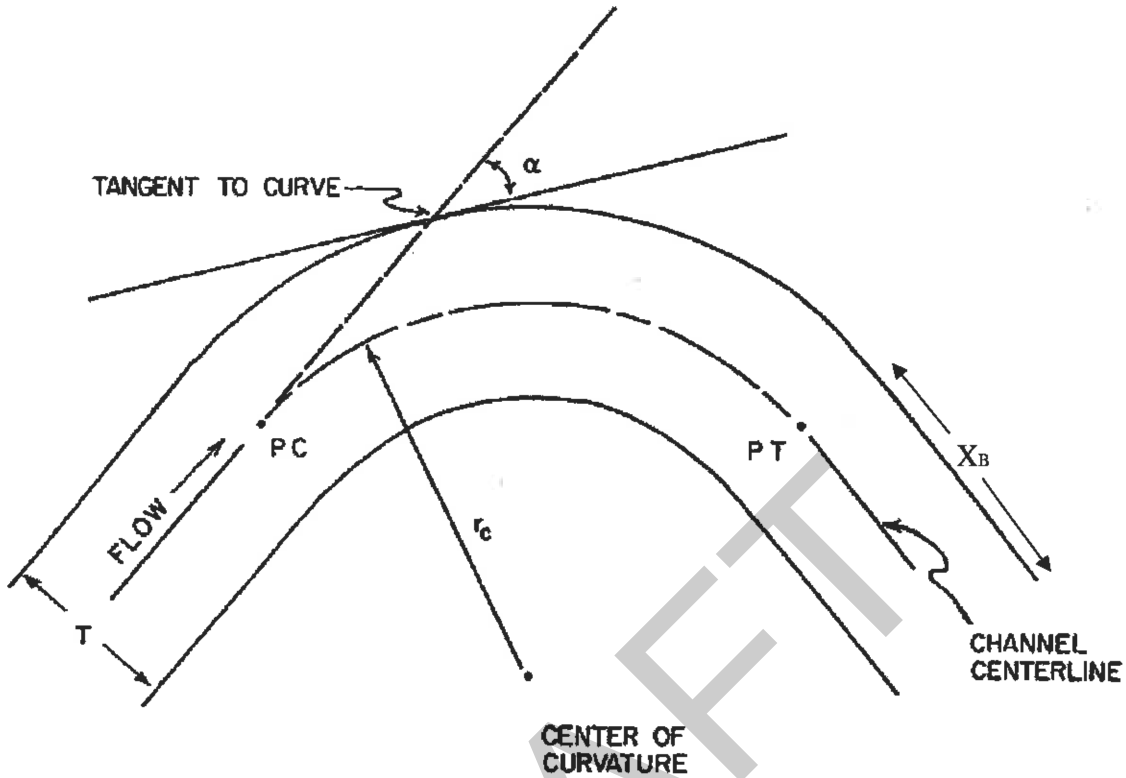
Note: Bend scour should not be applied beyond X_B, which is the downstream distance where scour is no longer influenced by flow curvature, and is mathematically defined as:

$$X_B = \left(\frac{0.6}{n}\right)Y_{max}^{1.17}$$

(Equation. 4.2.4.5)

Where:

Z_B	=	Bend scour depth, in feet;
q	=	Unit discharge of flow, Q/T, approaching bend, in cfs/ft;
r_c	=	Radius of curvature along centerline of channel, in ft;
T	=	Channel top-width immediately upstream of the bend, in ft;
α	=	Angle formed by the projection of the channel centerline from the point of curvature to a point that meets a line tangent to the outer bank of the channel, in degrees (see Figure 2);
X_B	=	Distance from bend to where scour is no longer influenced by flow curvature (see Figure 2);
n	=	Manning's "n" value; and
Y_{max}	=	Maximum depth of flow immediately upstream of the bend, in ft.



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

FIGURE 2

4.2.5 Scour at Confluence of Two or More Watercourses:

Confluence scour for two or more watercourses is determined by the following relationship:

$$Z_C = Y_{MC} - Y \qquad \text{(Equation 4.2.5.1)}$$

Maximum flow depth in a confluence, Y_{MC}, can be calculated for different sediment classes using the following relationship (Ashmore and Parker 1983; Klaassen and Vermeer 1988):

$$\frac{Y_{MC}}{\overline{Y}_{MS}} = 2.24 + (0.031)\alpha_c \text{ (non-cohesive sands/gravels } [30° < \alpha_c < 90°]) \qquad \text{(Equation 4.2.5.2)}$$

Where:

Z_C	=	Scour due to a confluence of two or more watercourses, in ft;
Y_{MC}	=	Maximum flow depth in the confluence scour hole, in ft;
Y_{MS}	=	Average flow depth, from water surface to mean scoured depth, in ft;
Y	=	Average flow depth, in ft.
α_c	=	Angle of confluence of two watercourses, in degrees (Note: This is not the same α depicted in Figure 2).

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4.2.6 Local Scour (Z_L):

Local scour occurs whenever there is a hydraulic structure or an obstruction which causes an abrupt change in the flow direction. Causes of abrupt changes include, but are not limited to, culverts, bridge piers and bridge abutments, fill encroachments such as directional dike and levees, and grade-control structures. Local scour is caused mainly by abrupt changes in both the direction and velocity of flow, which often sets up eddy currents that create localized scour. Local scour can best be accounted by a direct summation of the contribution of scour depths as shown in the equation below:

$$Z_L = Z_{CUL} + Z_{LB} + Z_{LE} + Z_{LGC} + Z_{SS} \qquad \text{(Equation 4.2.6.1)}$$

Where:

- Z_{CUL} = Local scour due to the presence of a culvert, in ft;
- Z_{LB} = Local scour due to the presence of bridge piers/abutments, in ft;
- Z_{LE} = Local scour due to encroachments, in ft;
- Z_{LGC} = Local scour due to the presence of a grade-control structure, in ft;
- Z_{SS} = Local scour due to presence of a sanitary sewer in scour zone, in ft

4.2.6.1 Local Scour due to Culvert (Z_{CUL})

If a sanitary sewer crossing is located too close to the downstream side of a culvert—which shape may be circular, box, or other variations—the sanitary sewer may be impacted by local scour created by the jet of flow issuing from the culvert outlet. Accordingly, local scour due to culverts should be determined from the following equations for cohesionless soils, which were adapted and refined from procedures presented within the *City of Tucson Standards Manual for Drainage Design* (1989, and revised in 1998).

For a circular culvert flowing full:

$$Z_{CUL} = 0.5312\left(\frac{Q_{100}^{0.50}}{D^{0.25}}\right) \quad \text{Applies when } D_{50} < 8 \text{ mm} \quad (0.315") \qquad \text{(Equation 4.2.6.1.1)}$$

Where, D equals the diameter of the culvert, in feet.

For a non-circular or partially-full culvert:

$$Z_{CUL} = 0.3897\left(\frac{Q_{100}^{0.50}}{A^{0.125}}\right) \quad \text{Applies when } D_{50} < 8 \text{ mm} \qquad \text{(Equation 4.2.6.1.2)}$$

Where, A equals the cross-sectional area of flow, in square feet.

Likewise, the length, $L_{Z_{CUL}}$, and width, $W_{Z_{CUL}}$, of the scour hole created at the outlet of the culvert can be computed from the following equations.

For a circular culvert flowing full:

$$L_{Z_{CUL}} = 3.3667\left(\frac{Q_{100}^{0.62}}{D^{0.55}}\right) \quad \text{Applies when } D_{50} < 8 \text{ mm} \qquad \text{(Equation 4.2.6.1.3)}$$

$$W_{Z_{CUL}} = 1.2487\left(\frac{Q_{100}^{0.89}}{D^{1.225}}\right) \quad \text{Applies when } D_{50} < 8 \text{ mm} \qquad \text{(Equation 4.2.6.1.4)}$$

For a non-circular or partially-full culvert:

$$L_{Z_{CUL}} = 2.2884\left(\frac{Q_{100}^{0.62}}{A^{0.275}}\right) \quad \text{Applies when } D_{50} < 8 \text{ mm} \qquad \text{(Equation 4.2.6.1.5)}$$

$$W_{Z_{CUL}} = 0.6820\left(\frac{Q_{100}^{0.89}}{A^{0.6125}}\right) \quad \text{Applies when } D_{50} < 8 \text{ mm} \qquad \text{(Equation 4.2.6.1.6)}$$

Where, both length and width are in feet.

Figure 4 of this document is intended to be applicable to local scour at a drop, discussed under Section 4.2.6.4 of this document. However, for purposes of local scour at a culvert it can also be assumed that the longitudinal profile of the scour hole at a culvert outlet is identical to the longitudinal profile of the scour hole depicted in Figure 4. Parameters Z_{CUL} and $L_{Z_{CUL}}$ are therefore substituted for parameters Z_{lss} and L_s, shown in Figure 4, to determine the point Xsce where maximum scour terminates, which is located downstream one-half the length of the calculated culvert outlet scour, L_s. In addition, for design purposes it should be assumed that maximum scour, Z_{CUL}, occurs everywhere along the streambed between the brink of the culvert outlet and the point X_{sce}.

4.2.6.2 Local Scour due to Bridge Piers (Z_{LB}):

The local scour due to bridge piers is dependent upon the shape of the bridge pier. Due to the likelihood of debris on piers during flood events the following equation, originally derived for square-nosed shaped piers, should be used for local scour due to the bridge pier:

$$Z_{LB} = \left[2.2Y\left(\frac{b_{pe}}{Y}\right)^{0.65} F^{0.43}\right] \qquad \text{(Equation. 4.2.6.2.1)}$$

$$b_{pe} = L\sin(\phi_p) + 1.5b_p\cos(\phi_p) \quad \text{for } 1.5b_p > 5 \qquad \text{(Equation 4.2.6.2.2)}$$

$$b_{pe} = L\sin(\phi_p) + 5\cos(\phi_p) \quad \text{for } 1.5b_p \le 5 \qquad \text{(Equation 4.2.6.2.3)}$$

Where:

Z_{LB}	=	Local scour contribution due to bridge piers with a pier shape reduction factor of 1.0 included, in ft;
Y	=	Depth of flow, in ft;
F	=	Upstream approach Froude number; and
b_{pe}	=	Effective pier width, from Equation 4.2.6.1.2 or 4.2.6.1.3, in ft.
b_p	=	Physical pier width, in ft;
ϕ_p	=	Angle of approach flow to pier wall ($\Phi_p = 0°$ for cylindrical piers), in ft; and
L	=	Length of pier wall, in ft;

4.2.6.3 Local Scour due to Encroachments:

Local scour due encroachments projecting into the flow of a channel (see Figure 3, Page 11), such as, but not limited to, bridge abutments and fill projections, such as overbank levees, can be computed from the following equations. Note that the equation to be utilized is dependent upon the quantity L_e/Y. For large values of L_e/Y Equation 4.2.6.3.2 should be used.

$$Z_{LE} = 2.15\sin(\theta_a)Y\left(\frac{L_e}{Y}\right)^{0.4}F^{0.33} \qquad \text{If } \frac{Z_{LE}}{YF^{0.33}} < 4.0 \qquad \text{(Equation 4.2.6.3.1)}$$

$$Z_{LE} = 4YF^{0.33} \qquad \text{If } \frac{Z_{LE}}{YF^{0.33}} \geq 4.0 \qquad \text{(Equation 4.2.6.3.2)}$$

Where:

Z_{LE}	=	Local scour contribution from encroachments, in ft;
θ_a	=	Slope angle of encroachment face (measured from horizontal), in degrees;
L_e	=	Encroachment length (use caution determining embankment length), in ft;
F	=	Upstream Froude number; and
Y	=	Upstream depth of flow, in ft.

4.2.6.4 Local Scour at Drops (Z_{LD}):

Local scour immediately below channel drops can occur under two conditions. The first condition, say at a high-head grade-control structure, is a special case where the drop is a free, unsubmerged overfall. The second condition is where the drop is submerged, as will be the circumstance for most low-head drops comprised of grade-control structures placed across alluvial watercourses. Equation 4.2.6.4.1 should be used for the first condition, while the Equation 4.2.6.4.2 should be used for the second condition. Both equations are shown below.

$$Z_{LD} = 1.32q^{0.54}H_T^{0.225} - TW \qquad \text{If } \frac{h}{Y_1} \geq 1.0 \qquad \text{(Equation 4.2.6.4.1)}$$

$$Z_{LD} = 0.581q^{0.667}\left(\frac{h}{Y_1}\right)^{0.411}\left[1-\left(\frac{h}{Y_1}\right)\right]^{-0.118} \qquad \text{If } \frac{h}{Y_1} < 1.0 \qquad \text{(Equation 4 2.6.4.2)}$$

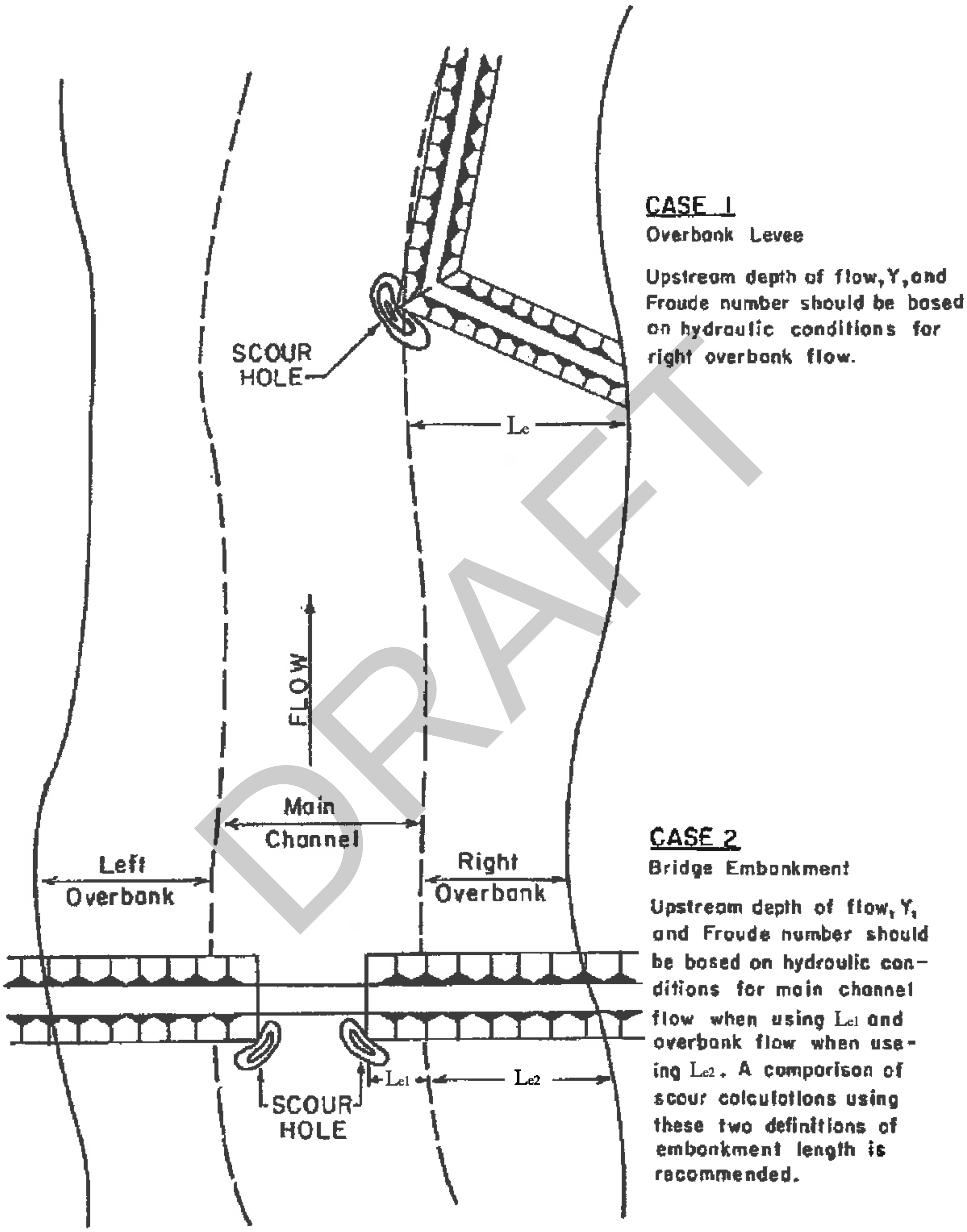

CASE 1
Overbank Levee
Upstream depth of flow, Y, and Froude number should be based on hydraulic conditions for right overbank flow.
SCOUR HOLE
Le
FLOW
Main Channel
Left Overbank
Right Overbank
CASE 2
Bridge Embankment
Upstream depth of flow, Y, and Froude number should be based on hydraulic conditions for main channel flow when using Le1 and overbank flow when using Le2. A comparison of scour calculations using these two definitions of embankment length is recommended.
SCOUR HOLE
Le1
Le2

FIGURE 3

Where:

- Z_{LD} = Local scour contribution from drop (measured from thalweg downstream of control-point), in ft;
- H_T = Total drop in head (measured as the difference between upstream and downstream energy grade lines), in ft (**normally, use the difference in WSELs, Y_1 - Y_2**);
- Y_1 = Upstream depth of flow, in ft;
- Y_2 = Downstream depth of flow = TW (tailwater), in ft;
- F = Upstream Froude number;
- h = Exposed height on downstream side of drop structure, in feet.

Figure 4, below, depicts the longitudinal profile of local scour that occurs immediately below a drop. For design purposes, it should be assumed that $L_s = 12Z_{lss}$, and that $X_{sce} = 6Z_{lss}$. In addition, for design purposes it should also be assumed that maximum scour, Z_{lss}, occurs everywhere along the streambed between the brink of the drop and the point X_{sce}.

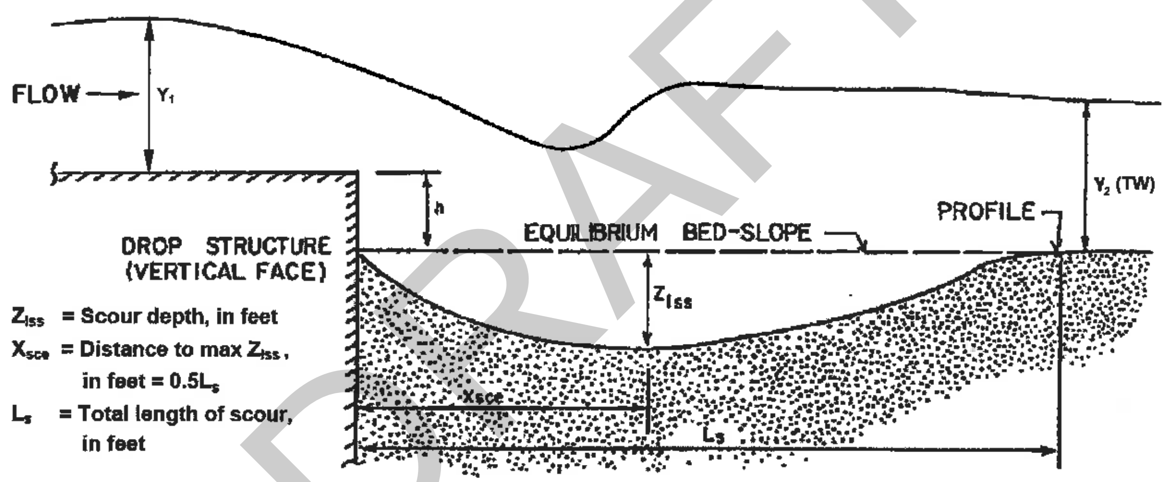


FIGURE 4

Note: If $0.85 < h/Y_1 < 1.0$, the predicted scour below a channel drop should also be computed using Equation 4.2.6.4.1. The smaller of the two values thus computed should then be used for design purposes. Figure 5, on the following page, depicts the longitudinal shape of a scour hole below an overfall.

4.2.6.5 Local Scour due to Sanitary Sewer in the Scour Zone:

The impact upon scour that the position of a sewer pipe has with respect to the streambed level; as well as the Froude number, F, of the approach flow, are two fundamental parameters that are used in the prediction of the final scour depth at a sanitary sewer crossing of an alluvial watercourse. This is because the smaller the clearance between the sewer pipe and the undisturbed streambed, the larger the influence the presence of the sewer pipe has on the scour depth. Variation of the scour depth as a function of the Froude number was presented in an

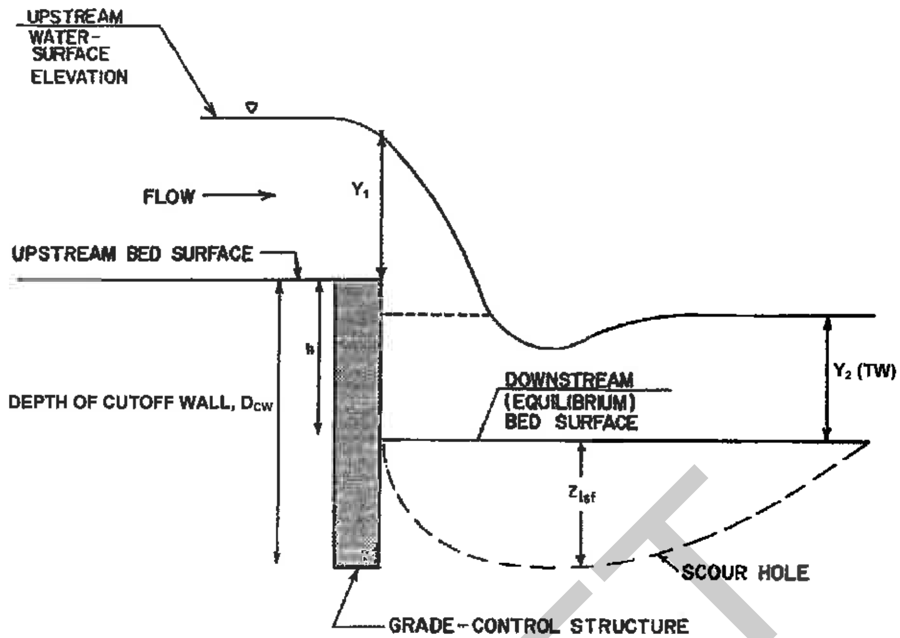


FIGURE 5

article published by Alix Moncada-M and Julian Aguirre-Pe in the *Journal of Hydraulic Engineering* in September of 1999. That published article discusses how the parameter Z_{SS}/D increases with an increase in the Froude number, F. Additionally, it was shown that an increment in the volume of removed material downstream of a sanitary sewer is produced when *e*/D increases, where *"e"* equals the initial gap between the sewer pipe bottom and the undisturbed erodible bed, in ft. Moreover, the maximum scour depth moves downstream of the sewer pipe when *e*/D increases. Equation 4.2.6.5.1 was formulated using a best-fit equation, with mean Froude numbers of 0.4, 0.6, and 0.8.

$$Z_{SS} = 2(DF)\operatorname{sech}\left(1.7\frac{e}{D}\right) \qquad \text{(Equation 4.2.6.5.1)}$$

Where:

Z_{SS} = Local scour due to presence of sanitary sewer within the scour zone, in ft;
D = Effective diameter of sewer pipe, in ft = D_O + 4 (with debris pileup);
D_O = Outer diameter of sewer pipe, in ft;
F = Froude number; and
e = Initial gap between sewer pipe bottom and undisturbed erodible bed, in ft.

Note: If $e = 0$ and $F = 1.0$, $\frac{Z_{SS}}{D} = 2.0$.

4.3 Long-Term Degradation (Z_{LTD}):

Estimating long-term degradation along an alluvial watercourse can be an extremely difficult task to accomplish with reasonable accuracy. This document uses a procedure which is a refinement to the long-term degradation methodology presented in Section 6.9 of the *City of Tucson Drainage Standards Manual*, which was published more than 20 years ago. Since that time, new techniques have been developed for determining long-term degradation—particularly with regard to the time necessary in order to achieve the amount of long-term degradation predicted. Under certain circumstances, application of a time factor can significantly reduce the amount of predicted long-term degradation. Generally, the procedure in this document assesses the long–term changes, based upon a specified dominant discharge within over specified project design life, that are predicted for riverbed slope as the alluvial watercourse approaches either an armoring or an equilibrium-slope (i.e., dynamic-equilibrium) condition.

The following three equations are recommended for computing estimates of long-term degradation along an alluvial watercourse. The first two equations, which are based upon the computed estimate of the needed time to achieve a stable slope (T_{SS}), should be used when a downstream control (such as a roadway or grade-control structure) exists, or if $D_a > D_{90}$, the particle size for which only 10 percent of the sediment sizes in the reach are larger, by weight.

$$Z_{LTD} = \frac{8}{13}\left(S_n - S_{eq}\right)L_{dc} \qquad \text{for } T_{SS} < P_O \qquad \text{(Equation 4.3.1)}$$

$$Z_{LTD} = 1502.45\left[KV_{100}^{4.62}\frac{Q_{10}^{0.548}}{Q_{100}^{1.848}}W_{10}^{0.3}AC_{w,10}P_{10(n-hr)}R_sP_o\left(S_n - S_{eq}\right)\right]^{1/2} \qquad \text{(Equation. 4.3.2)}$$

$$\text{for } T_{SS} > P_O$$

However, if D_a is less than the D_{90} particle size of the reach ($D_a < D_{90}$), then the following equation should be used to determine the limit of degradation, given the presence of armoring:

$$Z_{LTD} = \frac{\left(0.6562D_a\right)}{P_C} \qquad \text{(Equation 4.3.3)}$$

Where, in the preceding three (3) equations:

Z_{LTD}	=	Long term degradation, in feet;
A	=	Drainage Area, in square miles;
P_C	=	*Percent* of the material which is coarser that the armoring size;
Q_{10}	=	Discharge for the 10-year event, in cfs;
Q_{100}	=	Discharge for the 100-year event, in cfs;
S_n	=	Natural channel slope, in feet per foot;
S_{eq}	=	Equilibrium channel slope, in feet per foot;
L_{dc}	=	Estimated distance to downstream control; in feet
V_{100}	=	Channel velocity for the 100-year event, in feet per second;
$C_{w,10}$	=	Weighted runoff coefficient for a 10-year rainfall, dimensionless;
$P_{10(n\text{-}hr)}$	=	10-year rainfall over a n-hr storm duration, in inches;
P_o	=	Estimated time period over which streambed degradation will occur (i.e. the design life), in years;

R_s = Sediment reduction factor, in decimal format (i.e., 0.20)
T_{SS} = Estimated time to achieve a stable slope, in years;
W_{10} = Width of channel conveying the 10-year flood, in feet; and,

K = Sediment transport coefficient $= \dfrac{0.0064n^{1.77}G^{0.45}}{D_{50}^{0.61}}$

G = Gradation coefficient of sediment $= 1/2\left[\dfrac{D_{50}}{D_{16}}\right] + 1/2\left[\dfrac{D_{85}}{D_{50}}\right]$

In order to select the proper equation for calculating the long-term degradation for inclusion in Equation 4.3.1, the user must follow the step-by-step procedure described below to estimate the appropriate amount of long-term degradation.

Step 1: Determine the Equilibrium Slope

Compute the equilibrium slope (S_{eq}):

$$S_{eq} = \left(\left[\frac{Q_{u,10}}{Q_{n,10}}\right]^{-1.1}\left[1 - R_S\right]^{0.7}\right)S_n \quad \text{(Equation 4.3.4)}$$

Where:

S_{eq} = Equilibrium channel slope, in feet per foot;
S_n = Natural channel slope, in feet per foot;
$Q_{u,10}$ = 10-year peak discharge for *urbanized* conditions, in cfs;
$Q_{n,10}$ = 10-year peak discharge for *natural* conditions, in cfs; and
R_s = Sediment reduction factor for upstream sediment supply (i.e., the ratio of impervious area to total area [Example: urbanization, sand and gravel mining, detention/retention], varying from 0.0 to 1.0.

Where, typically, R_s = 0.15 for rural/suburban conditions and 0.5 for moderately to highly urban conditions; and,

Where, typically, $\left[\dfrac{Q_{u,10}}{Q_{n,10}}\right]^{-1.1}$ = 1.15 for rural/suburban conditions and = 1.50 for moderately to highly urban conditions.

Step 2: Determine Controlling Factor

Streambed Armoring

First, determine if long-term degradation is controlled by Streambed Armoring. To do this, calculate D_a (size of armoring material, in mm), from Equation 4.3.5:

$$D_a = 0.2659\left[\frac{V_{10}^{3.5}}{q_{10}^{0.5}}\right] \qquad \text{(Equation 4.3.5)}$$

D_a is assumed to be representative of the larger particles observed in the streambed. It should also be assumed that the D_a particle size is consistent with the D_{90} to D_{95} particle sizes in the streambed, and should be designated as the "default" streambed armoring size. In the absence of any sediment data, it should be assumed that D_{50} is 2 mm and that D_{90} is 13 mm (about 1/2 inch). However, a sediment analysis should be conducted in the event that there is evidence of possible D_a particle sizes which would better represent the potential armoring sediments that would be encountered within the study reach. If D_a is consistent with the D_{90} to D_{95} particle sizes in the study reach, then the following equation is to be used to determine the limit of degradation due to armoring:

$$Z_{LTD} = \frac{(0.6562D_a)}{P_C} \qquad \text{(Equation 4.3.3)}$$

Where:

Z_{LTD} = Limit of long term degradation due to armoring, in feet;
D_a = Size of the armoring material, in mm; and,
P_C = *Percent* of the material which is coarser that the armoring size.

If a downstream control exists, or if $D_a > D_{90}$ to D_{95} of the reach, then streambed armoring would not control. Under these circumstances, the following procedure should be applied:

Stable Slope

Determine long-term degradation controlled by Stable Slope using the following:

If the time to achieve the stable slope, T_{SS}, is less than the design life, P_o, of the project, that is if $T_{SS} < P_o$, use:

$$Z_{LTD} = \frac{8}{13}\left(S_n - S_{eq}\right)L_{dc} \qquad \text{(Equation 4.3.1)}$$

If the time to achieve a stable slope is greater than the projected design life of the project, that is if $T_{SS} > P_o$, use the following simplified form of Equation 4.3.1 to determine Z_{LTD}, as appropriate:

$$Z_{LTD} = C_{LTD}\left[\frac{W_{10}^{0.3}}{Q_{10}^{0.376}} AP_o(S_n - S_{eq})\right]^{1/2} \qquad \text{(Equation 4.3.2A)}$$

For primarily *natural* conditions within the upstream contributing watershed (i.e., less than 10% imperviousness cover), use $C_{LTD} = 4.55$.

For primarily *rural* to *suburban* conditions within the upstream contributing watershed (i.e., from 10% to 30% imperviousness cover), use $C_{LTD} = 7.73$.

For essentially *moderately urban* to *highly urban* conditions within the upstream contributing watershed (i.e., more than 30% imperviousness cover), use $C_{LTD} = 13.99$.

The general equation to use for computing the time required to achieve a stable slope (T_{SS}) is:

$$T_{SS} = \left(\frac{1.6776 \times 10^{-7}}{R_S K}\right) \frac{\left(S_n - S_{eq}\right) L_{dc}^{\,2} Q_{100}^{1.848}}{Q_{10}^{0.548} V_{100}^{4.62} W_{10}^{0.3} A C_{w,10} P_{10(n-hr)}} \qquad \text{(Equation 4.3.6)}$$

However, one may use the following simplified form of Equation 4.3.6 to determine T_{SS}, as appropriate:

$$T_{SS} = \frac{C_{TSS}\left(S_n - S_{eq}\right) L_{dc}^{\,2} Q_{10}^{0.376}}{W_{10}^{0.3} A} \qquad \text{(Equation 4.3.6A)}$$

For primarily *natural* conditions within the upstream contributing watershed (i.e., less than 10% imperviousness cover), use $C_{TSS} = 0.0183$.

For primarily *rural* to *suburban* conditions within the upstream contributing watershed (i.e., from 10% to 30% imperviousness cover), use $C_{TSS} = 0.0063$.

For essentially *moderately urban* to *highly urban* conditions within the upstream contributing watershed (i.e., more than 30% imperviousness cover), use $C_{TSS} = 0.0019$.

Where:

Z_{LTD}	=	Long term degradation, in feet;
S_n	=	Natural channel slope, in feet per foot;
S_{eq}	=	Equilibrium channel slope, in feet per foot;
L_{dc}	=	Estimated distance to downstream control; in feet;
V_{100}	=	Flow velocity for the 100-year event, in feet per second;
P_o	=	Estimated time (i.e., the "design life" [typically 100 years]) over which streambed degradation will occur; in years;
T_{SS}	=	Estimated time to achieve a stable slope, in years;
Q_{10}	=	10-year peak discharge, in cfs;
Q_{100}	=	100-year peak discharge, in cfs;
K	=	Sediment property coefficient $= \dfrac{0.0064 n^{1.77} G^{0.45}}{D_{50}^{0.61}}$;
A	=	Drainage Area, in square miles;
$C_{w,10}$	=	Weighted 10-year watershed runoff coefficient;
W_{10}	=	Channel top width, in feet; and
$P_{10(n\text{-}hr)}$	=	10-year, n-hr. rainfall depth, in inches;

Note that long-term degradation can be limited by downstream channel controls, as well as by streambed armoring, and can be influenced by several other factors as well—all of which can be extremely difficult to predict. Nevertheless, if applied properly the preceding relationships should provide a reasonable means of assessing long-term degradation trends which exist along watercourses which traverse less urbanized to highly urbanized watersheds in metropolitan Pima County.

V. Lateral-Migration Setback Requirements

Computation of *default* minimum setback distances along alluvial watercourses in metropolitan Pima County in order to account for lateral channel migration can be estimated from the product of an empirical coefficient times the square root of the 100-yr discharge. The procedure, below, is equivalent to applying a Level-1 analysis, as defined in the *City of Tucson Standards Manual for Drainage Design* (1989, and revised in 1998).

The following setback equations are for use with watercourses which have watershed drainage areas less than 30 square miles in size and (i.e., 100-year) peak discharge of less than 10,000 cfs ($Q_{p100} < 10{,}000$ cfs):

$$SB \geq 1.0(Q_{p100})^{0.5} \quad \text{for } r_c/T \geq 10 \qquad \text{(Equation 5.1)}$$

$$SB \geq 1.7(Q_{p100})^{0.5} \quad \text{for } 5 < r_c/T < 10 \qquad \text{(Equation 5.2)}$$

$$SB \geq 2.5(Q_{p100})^{0.5} \quad \text{for } r_c/T \leq 5 \qquad \text{(Equation 5.3)}$$

Where:

SB = Minimum Setback, in feet, measured from the top edge of the highest channel bank or from the edge of the top of the 100-year water surface elevation, whichever is closer to the channel centerline;

Q_{p100} = Peak discharge of 100-year event, in cubic feet per second.

Note: The numerical value for r_c/T can be calculated by using Equation 4.2.4.4 of this Document.

For watersheds that are more than 30 square miles in size, a more specialized analysis of lateral migration and setback potential should be conducted by a qualified expert in the field of erosion and sedimentation.

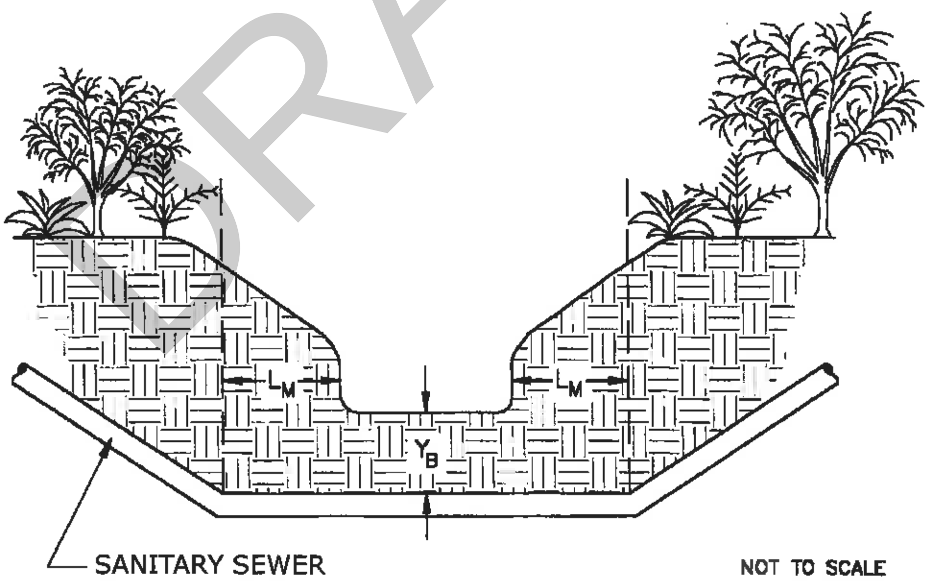
VI. ADDITIONAL DESIGN AIDS FOR SANITARY SEWER CROSSINGS OF ALLUVIAL WATERCOURSES

6.1 Determination of Sediment Particle Size

A precise determination of D_{50} should be accomplished via the collection of streambed sediments and performing a laboratory analysis to determine sediment gradation and plasticity index. However, a typical D_{50} for many washes in the metropolitan Pima County region is about 2 millimeters. It is for this reason that Tetra Tech separated applicable scour procedures into $D_{50} < 8$ mm and $D_{50} > 8$ mm—that is, to provide a simple visual means to recognize which equation was the most appropriate for use. In other words, because a D_{50} of 8 mm is about 1/3 of an inch in diameter, a diameter for which the presence of a preponderance of lesser diameter sediments in the streambed can easily be identified, such a demarcation should easily lead to the selection of the equation for use when $D_{50} < 8$ mm (which, as implied above, applies to the majority of the washes in metropolitan Pima County).

6.2 Vertical Alignment of Sanitary Sewer Crossing of Alluvial Watercourse

Once an estimate of total scour is determined using the procedures in this document, the vertical alignment of the sanitary sewer should be placed as indicated on Figure 6.



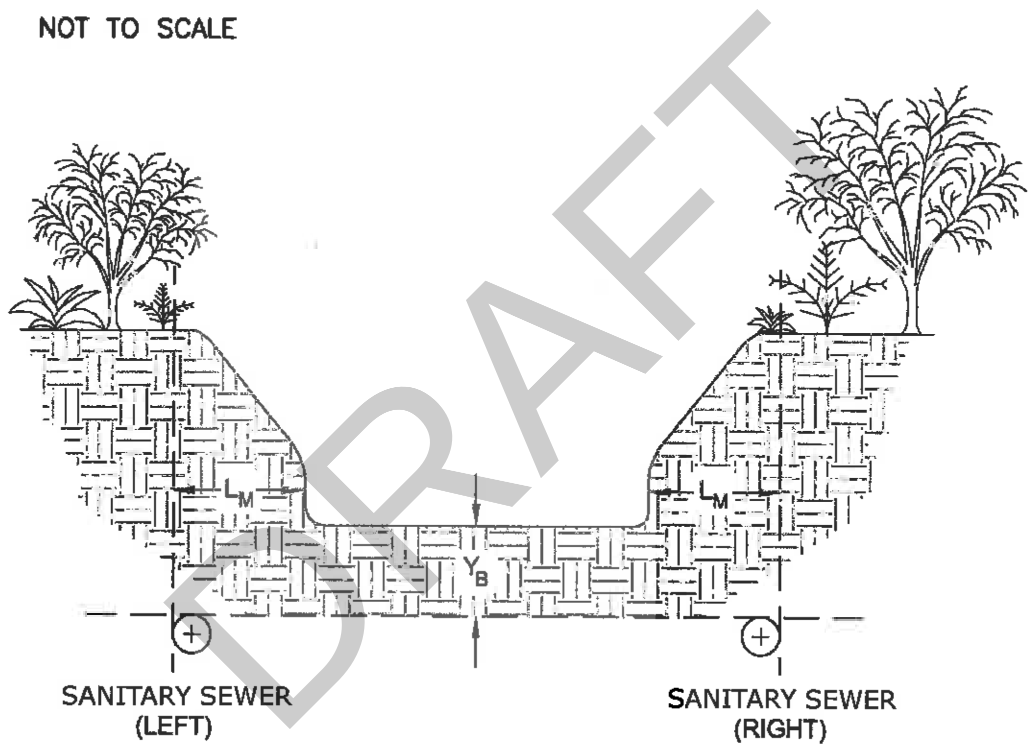
Y_B = BURIAL DEPTH, VARIES = Z_{MAX} + 2 ft
L_M = LATERAL MIGRATION LIMIT, VARIES
= SB (See Equations 5.1 - 5.3)

FIGURE 6

That is, the sanitary sewer should be placed at least 2 feet below the **maximum scour**, Z_{MAX}, (which includes long-term degradation); and, in the absence of permanent bank stabilization, should extend beyond each side of the stream channel a distance which equals or exceeds the predicted lateral-migration limits of the alluvial watercourse at the crossing location.

6.3 Horizontal Alignment of Sanitary Sewer Parallel to Alluvial Watercourse

Where a sanitary sewer is parallel and in close proximity to an alluvial watercourse which does not have permanent bank stabilization, the sanitary sewer should be located as shown in Figure 7.



Y_B = BURIAL DEPTH, VARIES = Z_{MAX} + 2 ft
L_M = LATERAL MIGRATION LIMIT, VARIES
= SB (See Equations 5.1 - 5.3)

Note: IF SANITARY SEWER IS BEYOND L_M, Figure 7 DOES NOT APPLY

FIGURE 7

That is, when a sanitary sewer is parallel and in close proximity to an alluvial watercourse, in the absence of permanent bank stabilization it should be placed at least 2 feet below the **maximum scour**, Z_{MAX}, whenever it is located within predicted lateral-migration limits.

6.4 Confluence Scour Geometry

Confluence scour geometry is characterized by steep slopes dipping downward from the upstream channels into a scour pool, which feathers out along a gently inclined bed slope which leads to a bar with a pronounced foreset slip face. Figure 8 illustrates.

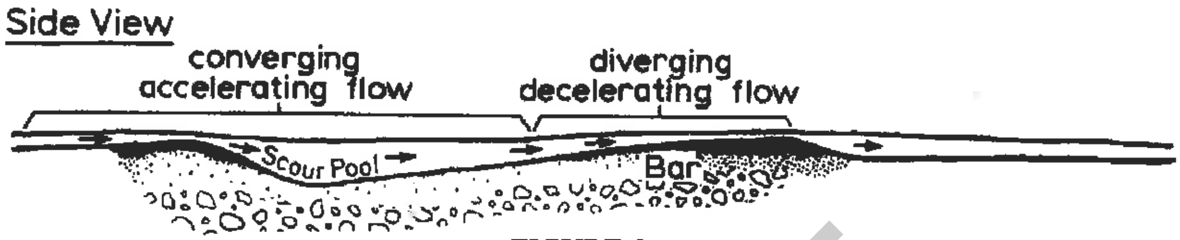


FIGURE 8

The approximate location of the scour pool can be defined as being immediately at and downstream of the confluence. See Figure 9.

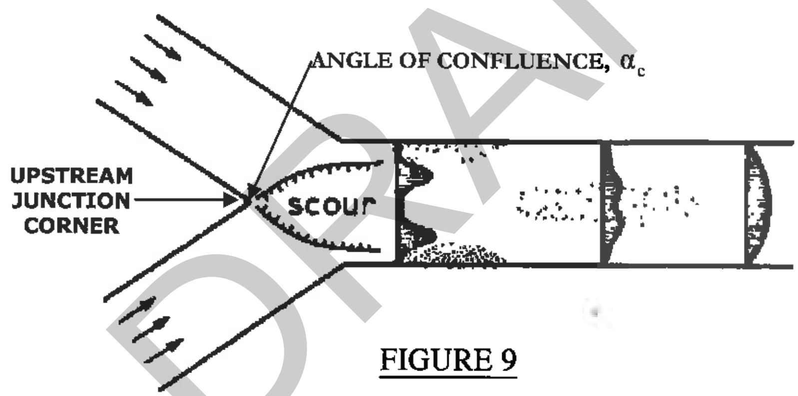


FIGURE 9

For design purposes, it should be assumed that the scour hole extends downstream from the upstream junction corner a distance equal to 2.5 times the bottom width, in feet, of the downstream channel. It is recommended that any sanitary sewer crossing of stay at lest this distance downstream of the confluence of an alluvial watercourse. If this is not possible, then for design purposes assume that the confluence scour component computed using Equation 4.2.5.1 applies everywhere within a region located 2.5 times the bottom width of the downstream channel, as measured from the upstream junction corner of the confluence.

6.5 Local Scour Geometries

Maximum local scour will occur at the locations defined for the applicable scour components presented in this document. In general, the local scour geometries for the various components can be described as follows:

6.5.1 Culvert Scour

The longitudinal profile of the scour hole will be as depicted in Figure 4, from which the contribution due to culvert scour at the point of the sanitary sewer can de determined, dependent upon the distance of the sewer pipe downstream from the brink of the culvert outlet. These criteria for culvert scour should be applied when placing a sanitary sewer across an alluvial watercourse.

6.5.2 Bridge Piers and Abutments

The shape of the scour hole created by bridge piers or by abutment scour should be assumed to be more or less consistent with that of a inverted truncated cone, with the base of the "cone" extending away from the pier or the abutment a distance equal to the depth of the computed scour component (i.e., pier scour or abutment scour). Upward from the base, it should be assumed that the sides of the scour hole everywhere slope at an angle of 3H:1V. Therefore the zone of influence of pier or abutment scour should be assumed to extend a distance of $4Z_{LB}$ from the outside diameter of the pier or from the face of the abutment. Maximum scour occurs 0.0 feet to Z_{LB} feet from the face of the pier or abutment, and then tapers off at a rate of 3H:1V until dissipating at a distance of $4Z_{LB}$ feet from the face of the pier or abutment. These criteria for bridge pier and abutment scour should be applied when placing a sanitary sewer across an alluvial watercourse.

6.5.3 Encroachments

Bridge abutments are one form of an encroachment structure for which the local scour geometry has been described in the preceding paragraph. Another type of encroachment, though, is a directional dike or levee. In general, the local scour geometry at the tip of a directional dike or levee is to be treated the same as the local scour geometry at bridge abutments. In such cases, though, Equation 4.2.6.3.2 should be applied to determine Z_{LE} if the length of the levee intercepting flow is such that $Z_{LE}/YF^{0.33} \geq 4.0$. See Figure 3. Along the riverside face of the levee, assume that the local toe scour is $2.2Y_{MAX}$, and that this scour depth extends Y_{MAX} feet from the face of the levee, tapering upward at 3H:1V and dissipating at a distance of $8.8Y_{MAX}$ feet from the face of the levee. These criteria for directional dikes and levees should be applied when placing a sanitary sewer across an alluvial watercourse.

6.5.4 Grade-Control Structure

Local scour geometry immediately downstream of a grade-control structure is generally as depicted in Figure 4 and Figure 5, from which the contribution due to grade-control-induced scour at the point of the sanitary sewer can de determined, dependent upon the distance of the sewer pipe downstream from the brink of the grade-control structure. These criteria for scour immediately downstream of a grade-control structure should be applied when placing a sanitary sewer across an alluvial watercourse.

6.5.5 Sanitary Sewer in Scour Zone

When a sanitary sewer is located within the scour zone of an alluvial watercourse (i.e., becomes exposed to the flow), the local-scour geometry is much like that formed in a bed of cohesionless material when flow issues from underneath a sluice gate. Figure 10 illustrates.

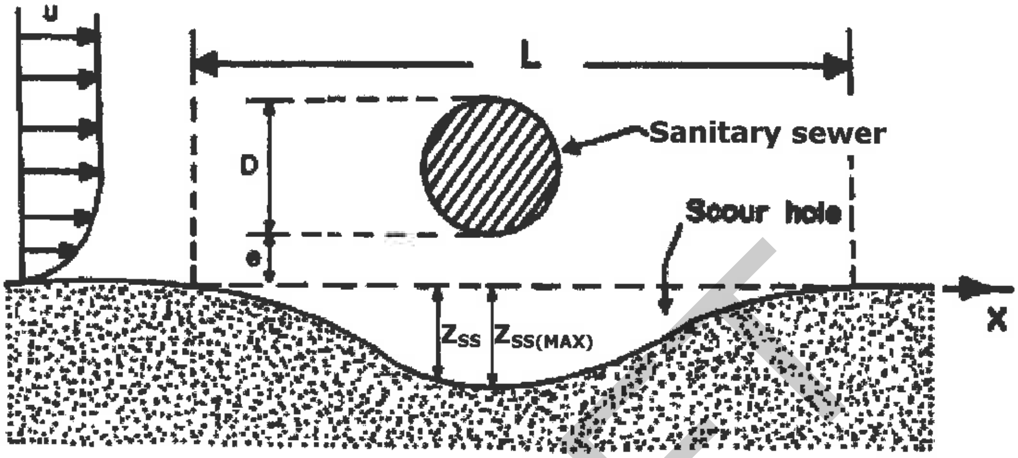


FIGURE 10

The length, L, of the scour hole illustrated in Figure 10 can be determined from the following relationship:

$L = 9.2D$, in feet.

Where:

- L = Length of scour hole under sanitary sewer exposed to flow, in feet; and
- D = Outer diameter of the Sanitary sewer, in feet.

For design purposes, it should be assumed that the maximum scour depth, $Z_{SS(MAX)}$, will occur directly under, and at the longitudinal midpoint of, the sanitary sewer. It should also be assumed that the scour-hole geometry will be symmetrically located upstream and downstream from the longitudinal midpoint of the sanitary sewer.

In addition, in the absence of a detailed HEC-RAS model, or similar backwater modeling analysis, a simple "rule-of-thumb" for selecting a Froude number to use in Equation 4.2.6.5.1 is to assume that $F = 1.0$ for all streambed slopes greater than 0.005 ft/ft, and that $F = 0.7$ for all streambed slopes less than 0.005 ft/ft. Otherwise, a backwater calculation (e.g., HEC-RAS) is required in order to determine a value for F.

6.5.6 Protection Measures

Sections 6.2 and 6.3 of this document provide guidance for both the vertical and horizontal locations of a sanitary sewer that will placed within or adjacent to the "zone of influence" of the various scour components that occur along alluvial watercourses. However, in those circumstances where it is not possible to place the sanitary sewer below or beyond the "zone of influence" of these various scour components, mitigation measures to prevent scour can be designed. Examples of such mitigation measures are provided in Figures 11 and 12, below.

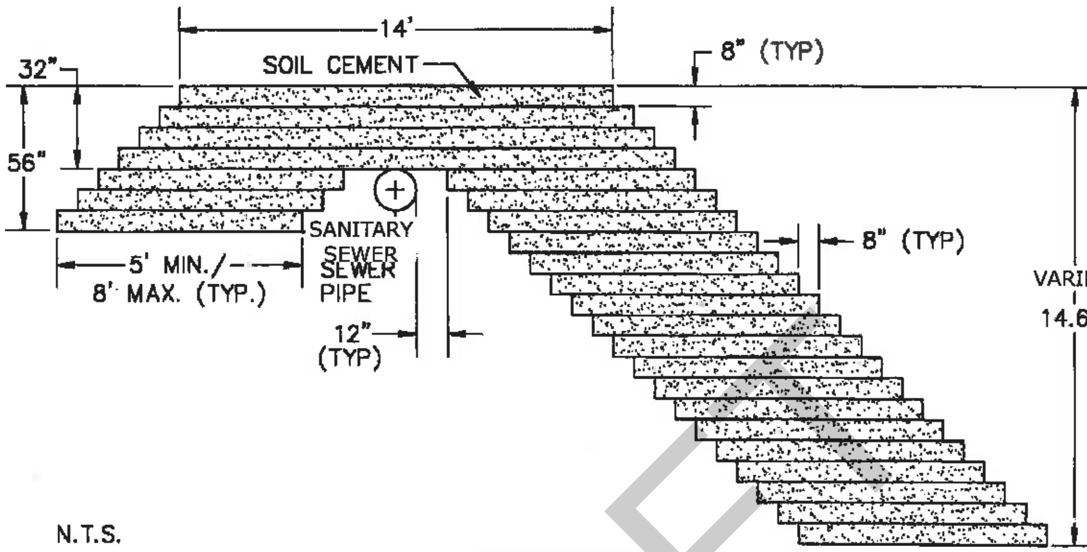


FIGURE 11

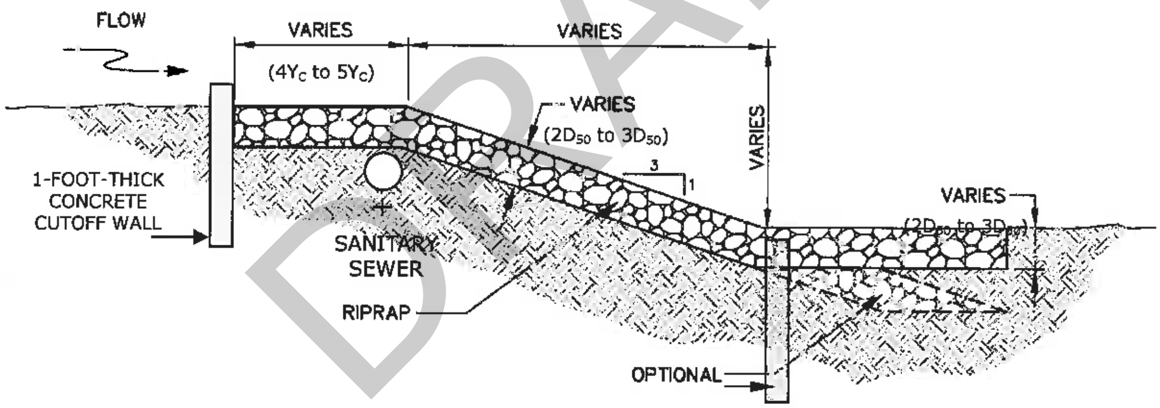


FIGURE 12

Note that Figure 11 depicts the use of a soil-cement grade-control "cap" to protect a sanitary sewer. However, this "cap" can just as easily be fabricated from roller-compacted concrete, from reinforced concrete, or from gabions. Although, based upon the durability of gabions in the metropolitan Pima County area, they should only be used if the sanitary sewer is to be "temporary" in nature (i.e., not expected to be in place more than 20 years). The thickness of materials other than soil cement should be determined when mitigation measures are designed.

Figure 12 depicts a riprap "cap" for protection of a sanitary sewer. However, the "cap" depicted in Figure 12 should only be used if the median diameter (D_{50}) of the riprap is large enough to resist scour along the alluvial watercourse. In general, the median diameter of the riprap, in inches, can be computed using the relationship: $D_{50} = 5.543q^{0.529}S_c^{0.307}$, where q = unit discharge (including C_U, from Table 3 of this document), and S_c = channel slope.

Another method of mitigating scour at sanitary sewer crossings of alluvial watercourses would be to encase the sewer pipe in concrete and fix the line on piling. If such measures were undertaken, though, when computing scour the piles would need to be treated as if they were bridge piers, with debris pileup (see section 4.2.6.2 of this document); and the sewer pipe, also with debris pileup, would need to be considered as located within the scour zone (see Section 4.2.6.5 of this document).

6.5.7 Manholes

Manholes should be treated as if they are independent bridge piers, and the appropriate scour components should be calculated, accordingly. Debris pileup should be included when determining the width of a manhole for purposes of calculating total scour. In addition, if the manhole is located inside the lateral-migration (setback) limits of an alluvial watercourse, then when calculating scour components it should be assumed that the channel can migrate over to the manhole and expose it to the full force of the channel flow.

6.5.7.1 Maximum Scour

Manholes should be treated as if they are independent bridge piers, and the appropriate scour components should be calculated, accordingly. Debris pileup should be included when determining the width of a manhole for purposes of calculating total scour. In addition, if the manhole is located inside the lateral-migration (setback) limits of an alluvial watercourse, then when calculating scour components it should be assumed that the channel can migrate over to the manhole and expose it to the full force of the channel flow.

If manholes are not protected by either revetment along the channel banks or revetment measures within the immediate vicinity of the structure, the maximum total scour to be expected below the channel bed, at the base of the manhole, can be computed from the following relationships:

$$Z_{MAX} = Z_{TSE} + Z_{LTD} \qquad \text{Where, } Z_{MAX} \le 5Y_{MAX} \qquad \text{(Equation 4.1.1)}$$

$$Z_{MAX} = 5Y_{MAX} \qquad \text{Where, } Z_{MAX} > 5Y_{MAX} \qquad \text{(Equation 4.1.2)}$$

Where the variables, above, are as defined in Section 4.1 of this Scour Procedures and Guidelines Document.

6.5.7.2 Forces on Manhole

The fluid forces which comprise the total lateral loading on an individual manhole during a design flood are (1) the hydrostatic force; (2) the hydrodynamic force; and (3) the impact force. These fluid forces, expressed in lbs, can be computed by use of the following equations:

$$F_S = 1/2\gamma B(Y_{mh} + Z_{MAX})^2 = \text{hydrostatic force} \quad \text{(Equation 6.5.7.2.1)}$$

$$F_D = 1/2\rho_s C_D[B_{mh}(Y_{mh} + Z_{MAX})](V_{mh})^2 = \text{hydrodynamic force} \quad \text{(Equation 6.5.7.2.2)}$$

$$F_I = 71.532V\sqrt{W_O} = \text{impact force} \quad \text{(Equation 6.5.7.2.3)}$$

and,

$$F_{TOT} = F_S + F_D + F_I = \text{maximum lateral-loading on manhole} \quad \text{(Equation 6.5.7.2.4)}$$

Where,

B	=	Width of manhole normal to flow, including debris, in feet (assume = a minimum of 2 feet of debris extending beyond each side of manhole)
Y	=	Depth of flow above the streambed at the manhole, in ft
Z_{MAX}	=	Maximum depth of scour below the streambed at the manhole, in ft
V	=	Velocity of flow at the manhole, in ft/sec
C_D	=	Fluid drag coefficient = 1.25
ρ_s	=	Fluid density = γ_{ws}/g = 1.94 slugs
γ	=	Unit weight of water + sediment = 68.6 lbs/ft^3 (assume = $1.1\gamma_w$) and,
γ_w	=	Unit weight of water = 62.4 lbs/ft^3
g	=	Acceleration due to gravity = 32.2 ft/sec/sec
W_O	=	Weight of floating object/debris, in lbs, (assume = 500 lbs)

Accordingly, substituting the assumed vales as indicated above, the total lateral loading on a manhole, in lbs, can be determined from:

$$F_{TOT} = 31.2B(Y_{mh} + Z_{MAX})^2 + 1.2[B_{mh}(Y_{mh} + Z_{MAX})](V_{mh})^2 + 1600(V_{mh}) \quad \text{Equation (6.5.7.2.5)}$$

In Equation 6.5.7.2.5, when applicable it should be assumed that V_{mh}, the velocity of flow at the manhole, is what would be expected in the absence of bank revetment and after lateral migration of the channel exposes the manhole to the full force of the primary flow path. Note that if flow velocities are high enough, impact forces can exceed the sum of the hydrostatic and hydrodynamic forces on manholes directly exposed to primary channel flows.

The preceding equations are all based upon the assumption that the manhole top is, in all cases, elevated above the water-surface elevation of the design flood; or, in other words, always above the depth of flow (Y_{mh}). Should this not be the case, then Z_{max} should be recalculated accordingly. That is, the parameter Y (for flow depth) used in the applicable local-scour equations found in Section 4.2.6 of this document should be adjusted downward to represent the flow depth that comes into contact with the manhole above the streambed prior to any single-event scour, which should include any exposure of the manhole that is predicted to occur due to long-term channel degradation.

6.5.7.2 Nonuniform Flow Distribution

It is important to recognize that, just as is the case when computing maximum scour depth, nonuniform flow distribution is likely to occur at and in the vicinity of a manhole exposed to a primary flow path. Accordingly, when using Equation 6.5.7.2.5 to compute total lateral loading, appropriate safety factors must be incorporated to account for nonuniform flow. When computing the depth of flow at the manhole, the maximum value should be measured from the lowest anticipated (future) thalweg, and must include superelevation, where applicable. When computing the velocity of flow at the manhole, a minimum safety factor of not less than 1.3 ($1.3V_{avg}$) should be used. A value as high as 1.6 ($1.6V_{avg}$) should be considered for use along natural channels, or areas where braided flow conditions exist.

6.5.7.3 Debris Structure or Independent Fender System

When a debris structure or independent fender system is placed upstream of a manhole that is located within a primary flow path (i.e., is located in the middle of the channel or is in an area that has become exposed due to lateral channel migration), it is unnecessary to design for debris forces and, depending upon the design of the structure or fender, may also be unnecessary to design for impact forces. However such a structure, if installed, must be designed to withstand the lateral forces and maximum scour anticipated to occur at its location during passage of the design flood. In this regard, Equation 6.5.7.2.5 is also applicable for determining the lateral loading on a protective debris structure or independent fender system installed upstream of a manhole.

The design of debris structures or independent fender systems is beyond the scope of this Scour Procedures and Guidelines Document; however, an excellent reference for the design of debris structures is:

U.S. Department of Transportation, Federal Highway Administration, Hydraulic Engineering Circular No. 9 (HEC-9), *"Debris Control Structures, Evaluation and Countermeasures, Third Edition,"* Publication No. FHWA-IF-04-016, October 2005, and can be found online at:

http://www.fhwa.dot.gov/engineering/hydraulics/library_arc.cfm?pub_number=9&id=23

6.5.8 Recommendation

The "safest" way to prevent scour impacts at a sanitary sewer and any attendant manholes is to place the sanitary sewer below the predicted maximum total scour (including any predicted long-term degradation) and also place the alignment of the sewer and any attendant manholes beyond the predicted lateral-migration (setback) limits of the alluvial watercourse (see Sections 6.2 and 6.3 of this document).

PIMA COUNTY
WASTEWATER RECLAMATION

APPENDIX B
DESIGN GUIDELINES FOR SIPHONS

Engineering Design Standards

Appendix B

Design Guidelines for Siphons

Special Approval, by the Director or his/her delegate, shall be obtained prior to proceeding with the design of a Public Sewer siphon. The Design Engineer should contact the Department as early as possible to review and discuss the need for a siphon. Siphons will be considered on a case-by-case basis and when no other practical method for avoiding obstacles is feasible. Cost will not be the sole consideration for allowing a siphon. If Special Approval is granted, the design of a siphon shall conform to the requirements of AAC R18-9-E301(D)(6) and the following Department guidelines:

B1. Use a minimum of 3 siphon lines, each with varying diameters (6 inches minimum), to maximize velocities and meet the capacity needs.

B2. Each siphon line shall be uniform in diameter and in horizontal alignment. Vertical bends shall be limited to 45 degrees at the bottom of the siphon lines. Siphon lines shall not be curvilinear and shall allow for the installation of a rigid vacuum hose to the bends at the bottom of the siphon line.

B3. The depth of cover for siphon lines shall conform to Subsection 5.1.11(A). The Department may allow a modification of the elevation difference between the inlet and outlet hydraulic grade lines if excessive depths will have an adverse impact on the downstream sewer.

B4. For flood-prone areas, the design of siphon manholes shall conform to Subsection 5.2.11.

B5. Maintenance vehicle access to the siphon manholes shall conform to Subsections 7.5, 7.6 and 7.7. In addition, maintenance access shall provide for a combo-cleaner truck to be positioned as necessary to reach the bends at the bottom of the siphon line with a rigid vacuum hose from both ends.

B6. Siphon manholes shall be designed to allow for maintenance personnel to enter/exit the manhole, and perform shoveling and debris-clearing activities from within the manhole during hydraulic vacuuming operations.

B7. The interior of the siphon manholes shall be coated or lined in accordance with *Standard Specifications and Details* Subsection 3.3.3(B)(viii) to protect them from corrosion. Refer to the Department's List of Approved Products for the recommended coating and lining manufacturers.

B8. Two double-leaf hatches, secured with an Approved locking system, shall be installed at each siphon manhole and positioned to allow for maintenance access as described in B4 and B5.

B9. Redwood isolation gates, per S.S.D. RWRD-228, shall be provided for each siphon line, at both ends.

B10. A vent assembly per S.D. RWRD-223 or -224 shall be provided at each siphon manhole.

B11. In cases where an air jumper pipe can be constructed in lieu of a vent assembly, the cross sectional area shall not be less than 50 percent of the combined cross section of the siphon pipes. The air jumper pipe shall be located where it will be self-draining under all operating conditions and protected from damage.

B12. Design provisions for odor control facilities will be determined on a case-by-case basis.

B13. A Design Report for the siphon facility shall be sealed by an Arizona Registered P.E. (Civil) and submitted to the Department for review and Approval. Design Report shall include an analysis that shows the siphon will provide for cleansing velocities and will not surcharge the inlet sewer.