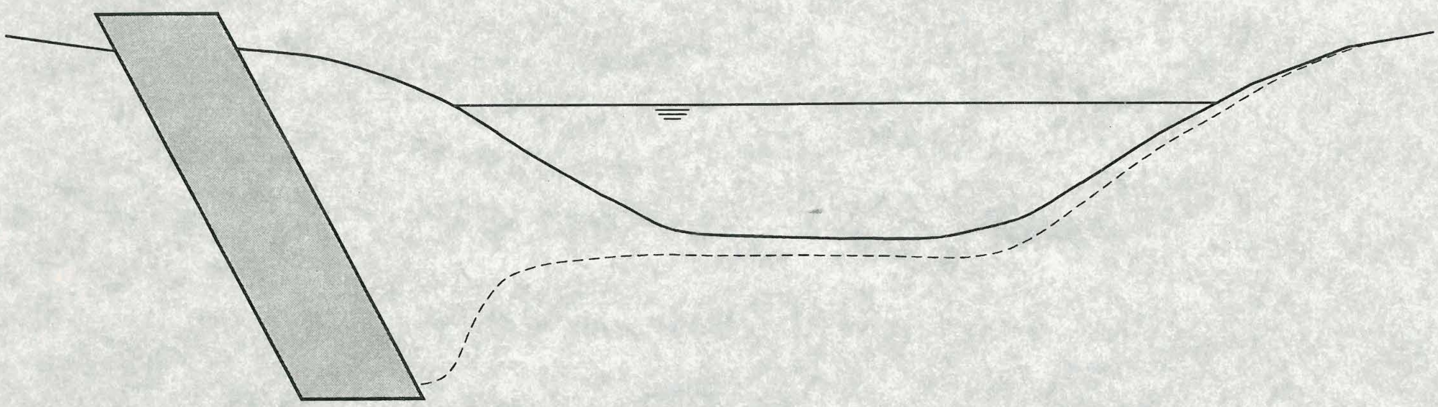
COMPARISON OF METHODS USED TO ESTIMATE SCOUR AT FLOOD WALLS ALONG ARROYOS



By
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To Stephanie,

for her constant encouragement, support, and patience while I lived at the office
for giving me the time to complete my graduate studies

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ABSTRACT

Government agencies in the southwest region of the United States employ varying techniques to estimate scour along flood walls and the banks of sand-bed channels. A total of eight methodologies used by these agencies for estimating scour at flood walls are evaluated and compared. The eight government agencies include:

- Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA),
- Maricopa County Flood Control District,
- City of Tucson Department of Transportation, Engineering Division,
- Clark County Flood Control,
- Denver Urban Drainage and Flood Control District,
- United States Bureau of Reclamation,
- Federal Highway Administration, and
- United States Army Corps of Engineers.

This paper applies these methods to a particular project, the Mirehaven Arroyo Scour Wall at Las Lomitas Unit IIB Subdivision located in Albuquerque, and compares the results. Scour resulting from a single storm event is addressed. A new approach to predicting scour depth along flood walls, based on the AMAFCA, Maricopa County, and City of Tucson methods, is presented and evaluated.

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I. INTRODUCTION

Following the July 1988 flood at the base of the Sandia Mountains, emphasis has been placed on the affects of sediment on the hydraulics of natural channels and flood control structures in the Albuquerque metropolitan area. An important component of sedimentation in erodible channels is scour of the channel bed during flood flows. This paper specifically addresses methods for estimating scour at flood walls or banks along natural channels.

Government agencies in the southwestern region of the United States have attempted to predict the extent of erosion and scour to arroyos or natural sand-bed channels caused by the passage of flood flows. Arroyos in the Southwest, which are ephemeral streams, are prone to erosion and scour because they are characterized by wide sandy beds, shallow vertical banks, and steep grades. Various government agencies have published guidelines for the design engineer to follow to estimate the magnitude of scour during flood flows in order to determine the protection required, if any, to stabilize the channel bank.

Scour is defined as the erosion that occurs due to flowing water from a single storm event. Total scour depth is usually comprised of numerous component scour depths. These component scour depths can include long-term aggradation/degradation, general scour depth, antidune scour depth, local scour depth, bend scour depth, shear stress differential scour, and impingement scour. Long-term aggradation and degradation, a result of the channel response to several storms over a period of time, is not caused by a single event, and therefore it is not considered in this study. Scour is a function of the channel hydraulics (flow depth and flow velocity) and bed material characteristics (material gradation).

In this study, the methodologies of eight government agencies to estimate scour depth for a certain design storm are described. The eight government agencies include:

- Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA),
- Maricopa County Flood Control District,
- City of Tucson Department of Transportation, Engineering Division,
- Clark County Flood Control,
- Denver Urban Drainage and Flood Control District,
- United States Bureau of Reclamation,
- Federal Highway Administration, and
- United States Army Corps of Engineers.

Each technique is applied to a particular project, the Mirehaven Arroyo Scour wall at Las Lomitas Unit IIB Subdivision located in Albuquerque, and the results are compared. The components of each methodology are discussed to determine which method to predict scour gives the most reasonable results. A new approach to predicting scour depth along flood walls, based on the AMAFCA, Maricopa County, and City of Tucson methods, is presented and evaluated. Finally, conclusions and recommendations for further research are made.

II. BACKGROUND

Flood walls or channel bank revetments are constructed to contain or eliminate the lateral migration or bank retreat of arroyos. Lateral migration or bank retreat is a result of the passage of flood flows and sediment in an unstable arroyo causing the bed and banks to erode. An arroyo is stable or in equilibrium when the sediment supplied to a certain channel reach is equal to the sediment being transported out of that reach. If the supply of sediment is less than the sediment transport capacity of a particular reach, the bed and/or banks of the arroyo will erode resulting in channel instability.

Arroyos are ephemeral streams with steep gradients that are generally characterized by steeply sloping or vertical banks and wide, sandy beds. Arroyos are shaped by infrequent flood flows. An arroyo is a type of incised channel that ranges in size from rills that are a few inches deep to major entrenched streams that may be up to 50 feet deep (Resource Consultants & Engineers, 1994). Rivers or streams can be classified broadly in terms of channel patterns including straight, meandering, and braided (Simons and Senturk, 1977). Figure 1 shows the three channel patterns.

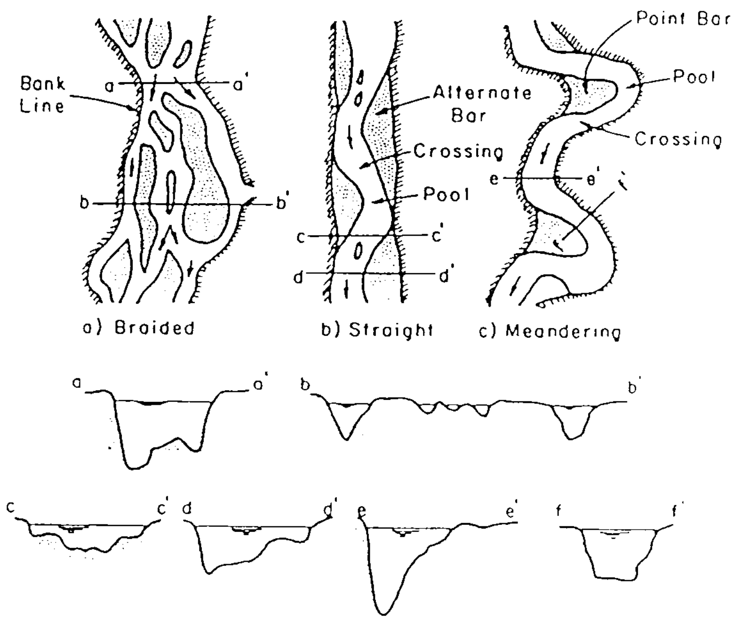


Figure 1 Stream Channel Patterns (SOURCE: Simons and Senturk, 1977)

A straight channel is a stream that does not follow a sinuous course. Although a straight channel may have relatively straight banks, the low flow thalweg is usually sinuous. Straight channels have a sinuosity (ratio of thalweg length to reach length) of 1.5 or less (Leopold et al., 1964). As the thalweg wanders in a sinuous path back and forth from one bank to the other, bars or accumulations of sediment form on the opposite side of the thalweg. At low flow stages, the thalweg in a straight reach tends to meander within the high-water channel, forming short pools and long shallow channels. At flood flow stages, the water flows across the entire channel section, ignoring the meandering thalweg.

A meandering channel consists of alternating bends resulting in an S-shaped pattern. Deep pools form in the bends and shallow, short, straight reaches connect the bends resulting in the sinuous meandering planform shape (Simons and Senturk, 1977). The pools tend to be triangular in section with point bars located on the inside of the bend. At low flows, the slope is steeper and velocities are higher in the straight reaches than in the pool. At low stages the thalweg is located close to the outside of the bend, whereas, at higher stages the thalweg tends to straighten or move away from the outside of the bend encroaching on the point bar. The shifting of the current during high stages can cause chute channels to develop across the point bar.

Braided streams are generally wide channels with shallow, unstable banks on a steep gradient with multiple channel divisions separated by alluvial islands (Simons and Senturk, 1977). Braided streams are caused by sediment overloading and steep slopes (Lane, 1957). Sediment overloading occurs when the sediment supply to a reach is greater than the sediment transport capacity of that reach, which results in deposition of part of the sediment load. The steep slopes produce a wide shallow channel where bars and islands form readily. The braided stream is unstable, changes its alignment quickly, and conveys large volumes of sediment.

It is usually assumed that during high stages, arroyos with steep slopes convey supercritical flows. Recent studies have shown that flows in natural channels may actually be in the subcritical flow regime (Trieste, 1994). There are two schools of thought on this matter: one is that supercritical flows do not occur except for short channel reaches, whereas the other is that both supercritical and subcritical flows occur in long channel reaches depending on flow resistance and slope. The main premise for non-occurrence of supercritical flows in natural channels is that flow resistance increases to the level required for predominantly subcritical flow to occur. Richardson (Mussetter et al., 1994) states that average Froude numbers in stable sand-bed streams usually range from 0.7 to 1.0 at high discharges.

Generally, there are two forms of channel-bed scour: live-bed scour and clear-water scour (Richardson et al., 1991). Live-bed scour occurs when the bed material is moving during the conveyance of storm flows. Clear-water scour occurs when there is no movement of the bed material. Live-bed scour, where scour depth increases during the rising limb of the storm and refills with sediment during the recession of the hydrograph, is the predominant type of scour found in the Southwest. Equilibrium scour depth and live-bed scour fluctuates with respect to bed material transport.

Equilibrium scour depth occurs when the sediment transported into the channel reach equals the sediment transported out of the reach.

According to Mussetter et al. (1994), the mechanism causing scour along a flood wall, when the flow is parallel to the wall, is an increased boundary shear stress produced by locally increased velocity gradients that result from the reduced roughness of the flood wall as compared to the arroyo. The scour along a flood wall will continue until the local flow area has increased enough to reduce the local velocity and boundary shear stress to those of the natural channel. This is valid for flood walls with smooth surfaces such as concrete, but may not be true for bank protection with rough surfaces such as riprap. Local scour from flow impinging on the flood wall occurs where the flow is accelerated due to the obstruction in the flow. The principal erosion mechanism is the creation of vortices or eddies by the obstruction and resultant acceleration of flow. Pemberton and Lara (1984) state that scour of the bed or banks caused by bankline structures is that created by higher local velocities or excessive turbulence at the structure.

To date, a wide variety of empirical equations based on a limited range of laboratory data have been developed to estimate scour depths. The majority of the equations were derived for subcritical flows in perennial streams. The local scour equation used by several government agencies was developed from data collected at rock dikes on the Mississippi River under subcritical flow conditions. Although these equations may or may not be applicable to ephemeral stream conditions flowing supercritically, they are the only equations available at this time. Using an equation developed for subcritical flows for streams flowing supercritically may introduce errors in the scour depths computed.

III. DESCRIPTION OF METHODS

This section describes each of the eight methods to predict scour depth along flood walls next to arroyos that were compared using the example project. Some of the methods have completely different approaches, while others are similar to each other. The following methods are summarized herein:

- Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA),
- Maricopa County Flood Control District,
- City of Tucson Department of Transportation, Engineering Division,
- Clark County Flood Control,
- Denver Urban Drainage and Flood Control District,
- United States Bureau of Reclamation,
- Federal Highway Administration, and
- United States Army Corps of Engineers.

A. *ALBUQUERQUE METROPOLITAN ARROYO FLOOD CONTROL AUTHORITY*

Following the flood of 1988, the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA) recognized the importance of including sediment transport effects of flood flows in channel hydraulics. AMAFCA contracted with Resource Consultants & Engineers, Inc. (RCE) to develop the "Sediment and Erosion Design Guide" (Mussetter et al., 1994) for the Albuquerque Metropolitan area. The "Sediment and Erosion Design Guide," completed in November 1994, includes a section on the methodology of determining scour of the channel bed along flood walls. The procedure discussed in the 1994 AMAFCA Design Guide evolved from the first draft published in March 1992 (Resource Consultants & Engineers, 1992). Prior to 1988, AMAFCA endorsed the use of "Design Guidelines and Criteria, Channels and Hydraulic Structures on Sandy Soils" (Simons, Li & Associates, 1981) to calculate scour.

The 1994 AMAFCA Design Guide describes methods to compute bed scour along a flood wall for two cases: flow parallel to the wall and flow impinging on the wall at an angle. The first case, flow parallel to the wall, can only be used where parallel flow can be assured such as when flood walls are located along both arroyo banks. The second case, flow impinging on the wall at an angle, is used for most analyses.

1. Flow Parallel to a Flood Wall

For flow parallel to a flood wall, scour results from the increased boundary shear stress produced by an increase in flow velocity due to the reduced roughness of the flood wall as compared to the natural channel. The maximum boundary shear stress on the side of a channel is assumed to be 0.76 times the average boundary shear stress, which is defined as γRS. A shear stress multiplier of 3 is applied to the average boundary shear stress to obtain the locally increased shear stress adjacent to a flood wall. This multiplier is based on the assumption that the roughness factor (n) for the channel is twice that of a flood wall. The reduction of n at the flood wall doubles the discharge and velocity next to the wall. Since boundary shear stress is proportional to velocity squared, doubling of the velocity increases the boundary shear stress by a factor of 4. Therefore, the average boundary shear stress multiplier results from multiplying the maximum boundary shear stress on the side (0.76) times the factor of 4.

The reduction in velocity required to reduce the shear stress is equal to the inverse of the square root of the shear stress multiplier (3) which gives 0.577. Due to continuity (Q=VA) the flow area must be increased by the inverse of 0.577 or 1.73. Therefore, the scour depth along a vertical flood wall is 0.73 times the flow depth (Y).

$$\frac{Y_s}{Y} = 0.73 \tag{1}$$

where: Y_s = Scour depth (ft)
Y = Hydraulic depth of flow (ft)

Another component of scour along a flood wall where flow is parallel to the flood wall is antidune scour. Antidunes that form in steep sand-bed channels with the passage of high flows can increase the magnitude of scour. Antidune scour is estimated to be equal to one-half of the antidune height. The following equations estimate the antidune height (Kennedy, 1963):

$$h_a = 0.14\frac{2\pi\ V^2}{g} = 0.28\pi\ YF_r^2 \tag{2}$$

$$Y_s = \frac{1}{2}h_a = 0.14\pi\ YF_r^2 \tag{3}$$

where: h_a = Antidune height (ft)
V = Flow velocity (ft/s)
g = Acceleration of gravity (32.2 ft/s^2)
Y = Hydraulic depth of flow (ft)
F_r = Froude Number
Y_s = Antidune scour depth (ft)

The total scour along a flood wall, where flow is parallel to the wall, is the scour resulting in a shear stress differential plus the antidune scour. A factor of safety of 1 foot is added to the equation.

$$\frac{Y_s}{Y} = 0.73 + 0.14\pi\, F_r^2 + 1 \qquad (4)$$

2. Flow Impinging on a Flood Wall at an Angle

The second case is scour from flow impinging on the flood wall at an angle. There are three components that contribute to the total scour estimated at the flood wall. These components are local scour from flow impingement, scour resulting from shear stress differential from parallel flows, and antidune scour. Usually the largest part of total scour is local scour from flow impingement.

The equation used to compute local scour at flood walls is derived from an equation developed for bridge embankments based on laboratory studies (Liu et al. 1961):

$$\frac{Y_s}{Y_1} = K\left(\frac{a}{Y_1}\right)^{0.40} Fr_1^{\;0.33} \qquad (5)$$

where: Y_s	=	Equilibrium scour depth (ft)
Y_1	=	Upstream flow depth (ft)
a	=	Embankment length normal to bank (ft)
Fr_1	=	Upstream Froude Number
K	=	1.1 for embankments, or
		2.15 for embankments terminating at vertical walls

Field data for scour at embankments and similar structures, such as flood walls, are scarce. But, data collected at rock dikes on the Mississippi River indicate that scour depth is independent of a/Y_1 (where $a/Y_1 > 25$) and depends only on the upstream Froude number and depth of flow.

$$\frac{Y_S}{Y_1} = 4 Fr_1^{\;0.33} \qquad (6)$$

This equation was discussed in "Design Guidelines and Criteria, Channels and Hydraulic Structures on Sandy Soils" (Simons, Li & Associates, 1981). In the first draft of the AMAFCA Design Guide (Resource Consultants & Engineers, 1992), equation (6) was the only equation used to calculate local scour along a flood wall. In later versions of the AMAFCA Design Guide (Resource Consultants & Engineers, 1993 and Mussetter et al., 1994), the angle of the flow impinging on the wall was included in the local scour equation. Figure 2 shows the angle of impingement of the flow to the flood wall. The following equation (7) allows for the inclusion of the angle of impingement (θ) in the calculation. A factor of safety of 1 foot is added to the equation. Equation 7 is based on an assumption that two scour mechanisms, local scour due to an obstruction to the flow and local scour due to parallel flow, are related to the change in momentum caused by the change in flow direction from some angle to the wall (impingement angle) to a direction parallel to the wall.

$$\frac{Y_s}{Y} = \left(0.73 + 0.14\pi\, F_r^2\right)\cos\theta + 4F_r^{0.33}\sin\theta + 1 \qquad (7)$$

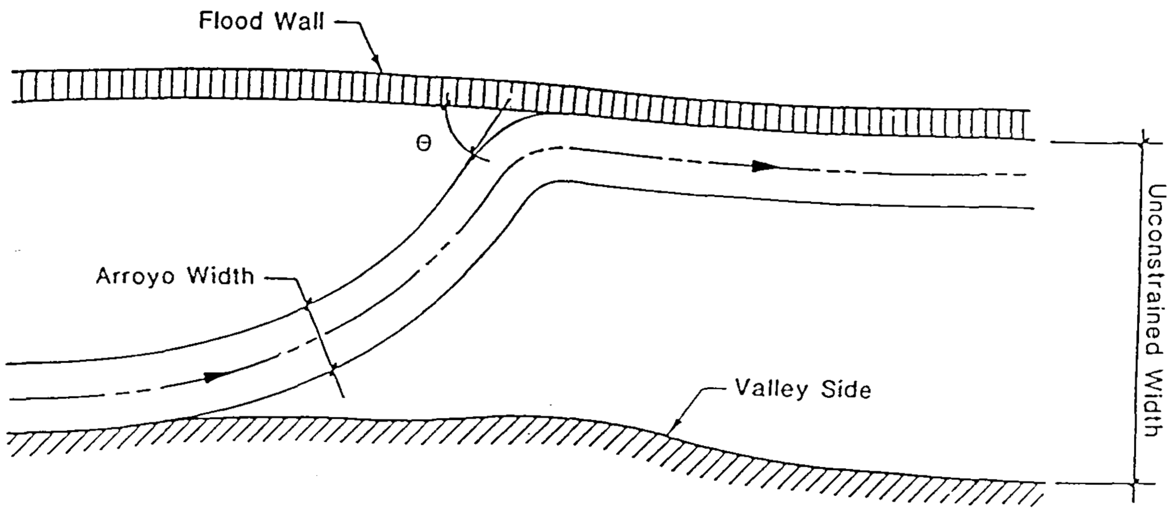


Figure 2 Schematic of channel alignment associated with a flood wall. (SOURCE: Mussetter et al., 1994)

The angle of impingement of the flow is derived from the radius of curvature for a meandering natural channel. The discussion for determining the radius of curvature for a meandering arroyo is based on an optimal bend shape for the channel beyond which significant lateral erosion will not occur. The maximum lateral erosion rate for a meander bend occurs when the radius of curvature (R_c) to channel width (W_D) is between 2 and 4 (Nanson and Hickin, 1983). River meanders move toward a constant R_c/W_D of 2 to 3 (Leopold and Wolman, 1960). River meanders generally follow the shape of a sine-generated curve (Langbein and Leopold, 1966). The length of a typical meander for perennial streams as well as ephemeral streams is approximately 10 to 14 times the channel width (Leopold et al., 1966). From this relation it can be shown that the maximum deviation of a channel from a straight line or maximum lateral erosion (Δ_{max}) will be 2.5 to 3.5 times the channel width (W_D). Figure 3 illustrates an idealized meander bend. In this case the channel width (W_D) is that associated with the dominant discharge. Dominant discharge is the peak discharge of the storm event that delivers the average annual sediment load. In the Albuquerque area the dominant discharge (Q_D) is estimated to be a 5-year to 10-year peak discharge (Mussetter et al., 1994).

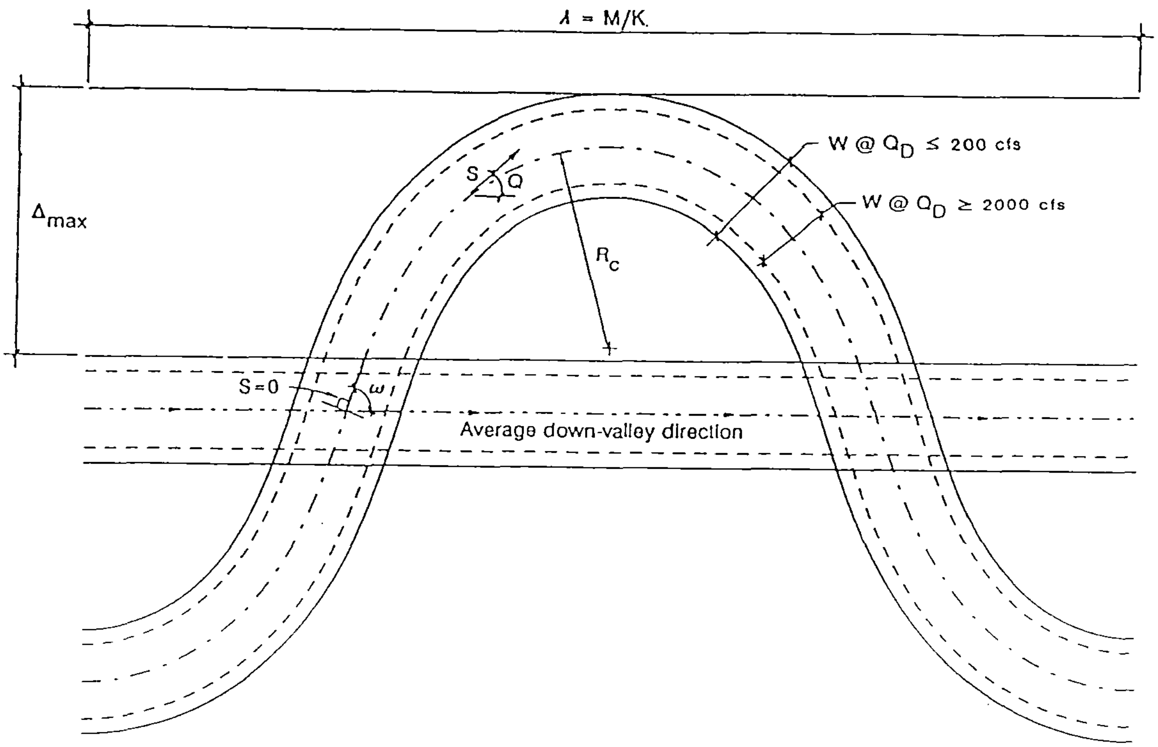


Figure 3 Schematic of an idealized meander bend
(SOURCE: Mussetter et al., 1994)

Using the relationships discussed above, the impingement angle will vary from 0° to 71°. At 0° the flow is parallel to the wall and at 71° the "unconstrained valley width" is 3.5 times the width of the arroyo. The "unconstrained valley width" can be defined as the floodway or floodplain width. Figure 4 gives the scour depth as a function of "unconstrained valley width".

Figure 4 is based on ideal meander geometry and approximate scour relationships. The AMAFCA Design Guide states that it is possible for flow to impinge perpendicularly to the flood wall. In all cases, one foot of depth should be added to the scour depth as a factor of safety.

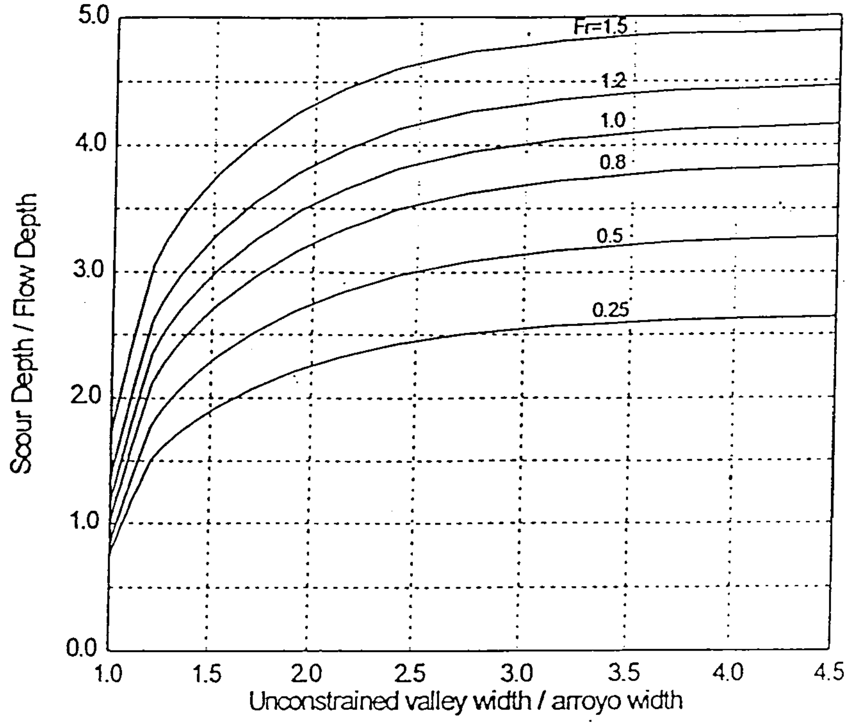


Figure 4 Scour along a flood wall as a function of unconstrained valley width (SOURCE: Mussetter et al., 1994)

B. MARICOPA COUNTY FLOOD CONTROL DISTRICT

The Drainage Design Manual for Maricopa County, Volume II (NBS Lowry Engineers, et al., 1991), addresses hydraulics of streets, culverts, open channels, bridges, and detention ponds. Additional criteria for the design of major watercourses were recently developed by the Maricopa County Flood Control District. These criteria specifically addressed required toe depths for bank protection. Maricopa County defines major watercourses as those watercourses conveying greater than 2500 CFS during a 100-year storm event. This criteria does not affect minor watercourses because all minor watercourses must be hard lined.

The total scour at banks of channels is comprised of several types of scour including contraction scour, bed-form scour, long-term aggradation or degradation, bend scour, and local scour multiplied by a factor of safety of 1.3 or 1.5 (Shah, 1995). Long-term aggradation or degradation is not considered in this study.

$$d_{ts} = F.S.(d_{cs} + d_a + d_{lt} + d_{bs} + d_{ls}) \tag{8}$$

where: d_{ts}	=	Depth of total scour (ft)
F.S.	=	1.3 if more than one scour component is acting on the bank, or 1.5 if only one scour component is acting on the bank
d_{cs}	=	Depth of contraction scour including general scour (ft)
d_a	=	Depth of bed-form scour due to the passage of dunes or antidunes (ft)
d_{lt}	=	Depth of long-term aggradation/degradation (ft)
d_{bs}	=	Depth of scour due to a river bend (ft)
d_{ls}	=	Depth of scour due to any local obstruction (ft)

Contraction scour in the vicinity of bridge crossings and river sections that have been constricted due to landfill or any other type of encroachment are computed by methods described in Hydraulic Engineering Circular (HEC) No. 18 (Lagasse et al., 1991) and Hydraulic Engineering Circular No. 20 (Richardson et al., 1991). Contraction scour in a natural channel affects the entire width of the channel. Contraction scour occurs when the flow area of the channel is decreased. There are two forms of contraction scour: live-bed scour and clear-water scour. Live-bed scour occurs when there is sediment being transported into the scour hole. Clear-water scour is when the sediment transport in the upstream uncontracted channel reach is zero. In the arid Southwest the majority of scour is live-bed scour.

Contraction scour equations are based on the principle of conservation of sediment transport. There are two flow conditions that must be addressed for contraction scour at channel banks: 1) overbank flow forced back into the channel, and 2) flow confined to the channel. An equation developed by Laursen (1960) can be used to predict the contraction scour for the condition of flow forced back into the channel.

$$\frac{y_2}{y_1} = \left(\frac{Q_{mc2}}{Q_{mc1}}\right)^{6/7} \left(\frac{W_{c1}}{W_{c2}}\right)^{K_1} \left(\frac{n_2}{n_1}\right)^{K_2} \tag{9}$$

where: y_1	=	Average depth in the main channel (ft)
y_2	=	Average depth in the contracted section (ft)
Q_{mc1}	=	Flow in the approach channel that is transporting sediment (cfs)
Q_{mc2}	=	Flow in the contracted channel, which is often Q_{total} (cfs)
W_{c1}	=	Bottom width of the main channel (ft)
W_{c2}	=	Bottom width of the contracted section (ft)
n_1	=	Manning's roughness factor for the main channel
n_2	=	Manning's roughness factor for the contracted section
K_1	=	Exponent determined below in Table 1
K_2	=	Exponent determined below in Table 1

Table 1 Method to determine K_1 and K_2 exponents
(SOURCE: NBS Lowry Engineers et al., 1991)

V_{*c}/w	K_1	K_2	Mode of Bed Material Transport
< 0.50	0.59	0.066	mostly contact bed material discharge
0.50 to 2.0	0.64	0.21	some suspended bed material discharge
> 2.0	0.69	0.37	mostly suspended bed material discharge
where: V_{*c} =		$(gy_1S_1)^{0.5}$, shear velocity (ft/sec)	
w =		Fall velocity of D_{50} of bed material (ft/sec)	
g =		Acceleration due to gravity, 32.2 (ft/sec^2)	
S_1 =		Slope of energy grade line of main channel (ft/ft)	

For the second flow condition where the flow is confined to the channel, the Laursen equation can be simplified:

$$\frac{y_2}{y_1} = \left(\frac{W_{c1}}{W_{c2}}\right)^{K_1} \tag{10}$$

Bed-form scour due to the passage of dunes or antidunes can be estimated by the equation developed by Kennedy (1963) as given in Section III.A:

$$h_a = 0.14\frac{2\pi\ V^2}{g} = 0.28\pi\ YF_r^2 \tag{2}$$

$$Y_s = \frac{1}{2}h_a = 0.14\pi\ \ YF_r^2 \tag{3}$$

where: h_a = Antidune height (ft)
V = Flow velocity (ft/s)
g = Acceleration of gravity (32.2 ft/s^2)
Y = Hydraulic depth of flow (ft)
F_r = Froude number
Y_s = Antidune scour depth (ft)

The scour due to river bends occurs along the outside of bends and is caused by spiral, transverse currents in the flow. The following relationship (Zeller, 1981) is used to determine bend scour.

$$d_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left[2.1 \left(\frac{\sin^2(\alpha/2)}{\cos\alpha} \right)^{0.2} - 1 \right] \tag{11}$$

where: d_{bs} = Bend-scour component of total scour depth (ft)
= 0 when $r_c/T \geq 10.0$, or $\alpha \leq 17.8°$
= computed value when $0.5 < r_c/T < 10.0$, or $17.8° < \alpha < 60°$
= computed value at $\alpha = 60°$ when $r_c/T \leq 0.5$, or $\alpha \geq 60°$

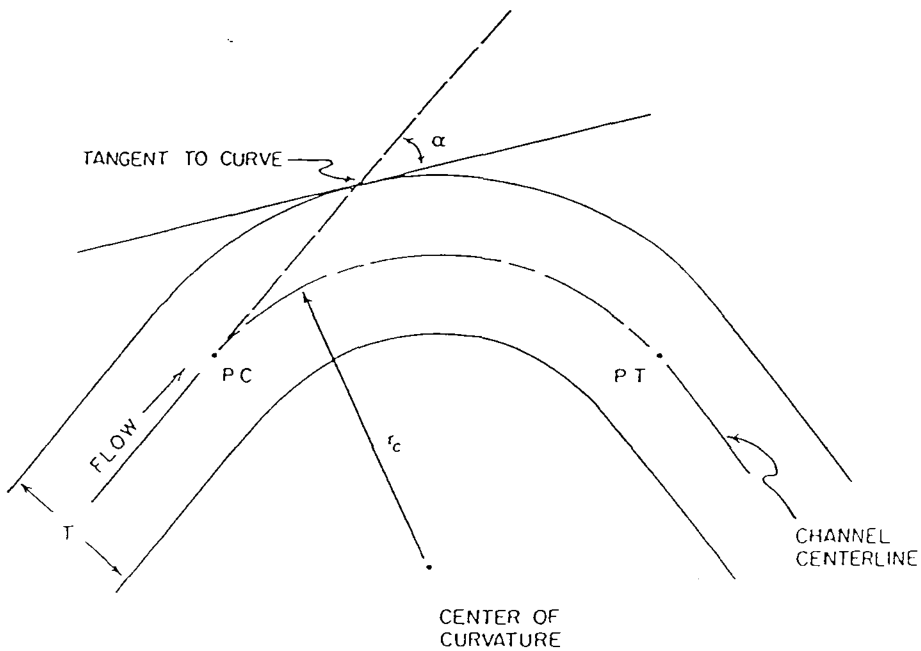
V_m = Average velocity of flow immediately upstream of bend (ft/sec)

Y_{max} = Maximum depth of flow immediately upstream of bend (ft)

Y_h = Hydraulic depth of flow immediately upstream of bend (ft)

S_e = Energy slope immediately upstream of bend or bed slope for uniform flow conditions (ft/ft)

α = 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 (°) (see Figure 5)



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

Figure 5 Schematic of channel bend illustrating α in equation 11
(SOURCE: Simons, Li & Associates, 1989)

Local scour due to obstructions in the flow are also considered. Local scour could be caused by bridge piers or bridge embankments within the channel section. As discussed in a previous section (Section IIIA), the equation to determine local scour at bridge embankments is based on laboratory studies by Liu et al. (1961).

$$\frac{Y_s}{Y_1} = K\left(\frac{a}{Y_1}\right)^{0.40} Fr_1^{\ 0.33} \tag{5}$$

where: Y_s = Equilibrium scour depth (ft)
Y_1 = Upstream flow depth (ft)
a = Embankment length normal to bank (ft)
Fr_1 = Upstream Froude number
K = 1.1 for embankments, or
2.15 for embankments terminating at vertical walls

C. CITY OF TUCSON DEPARTMENT OF TRANSPORTATION, ENGINEERING DIVISION

In 1989 the City of Tucson published a design manual addressing its technique to estimate scour at structures (Simons, Li & Associates, 1989). Several parameters contribute to the scour or lowering of a channel bed as expressed by the following equation:

$$Z_t = 1.3\left(Z_{gs} + \frac{1}{2}Z_a + Z_{ls} + Z_{bs} + Z_{lft}\right) \tag{12}$$

where: Z_t = Design scour depth, excluding long-term aggradation/degradation (ft)
Z_{gs} = General scour depth (ft)
Z_a = Antidune trough depth (ft)
Z_{ls} = Local scour depth (ft)
Z_{bs} = Bend scour depth (ft)
Z_{lft} = Low-flow thalweg depth (ft)
1.3 = Factor of safety to account for non-uniform flow distribution

The equations for depth of scour that follow apply only to sand-bed channels.

The depth of general scour can be 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 of bed levels throughout the event, and the sediment supply into the reach. An alternative approach for smaller channels involves using the following equation (Zeller, 1981) to predict general scour.

$$Z_{gs} = Y_{\max}\left(\frac{0.0685V_m^{0.8}}{Y_h^{0.4}S_e^{0.3}} - 1\right) \tag{13}$$

where: Z_{gs} = General scour depth (ft)
V_m = Average velocity of flow (ft/sec)
Y_{max} = Maximum depth of flow (ft)
Y_h = Hydraulic depth of flow (ft)
S_e = Energy slope or bed slope for uniform flow (ft/ft)

Should Z_{gs} become negative, the general scour component is assumed equal to zero.

Antidunes, which are bedforms in the shape of dunes, can form during transitional flow between subcritical and supercritical flow or during supercritical flow. The antidune trough depth can be calculated using the following equation (Simons, Li & Associates, 1982), which is similar to equation 3:

$$Z_a = \frac{1}{2}(0.14)\frac{2\pi V_m^2}{g} = 0.0137V_m^2 \tag{14}$$

where: Z_a = Antidune trough depth (ft)
V_m = Average velocity of flow (ft/sec)
g = Acceleration due to gravity (32.2 ft/sec^2)

It is important to note that the antidune trough depth can not be greater than one-half the depth of flow.

Low-flow thalwegs or channels form when the width-to-depth ratio of the main channel is large. When the ratio of the flow width to the flow depth of a channel is greater than 1.15 times the velocity of flow for the 100-year discharge, a low-flow thalweg is included in scour calculations. Although there is no known methodology to estimate low-flow thalweg depth, a depth of 2 feet for regional watercourses and 1 foot for minor channels is assumed for the Tucson area.

Bend scour occurs along the outside of bends and is caused by spiral, transverse currents in the flow. The following relationship (Zeller, 1981), which is the same equation used by Maricopa County, is used to determine bend scour.

$$Z_{bs} = \frac{0.0685 Y_{max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left[2.1 \left(\frac{\sin^2(\alpha/2)}{\cos\alpha} \right)^{0.2} - 1 \right] \tag{11}$$

where: Z_{bs} = Bend-scour component of total scour depth (ft)
= 0 when $r_c/T \geq 10.0$, or $\alpha \leq 17.8°$
= computed value when $0.5 < r_c/T < 10.0$, or $17.8° < \alpha < 60°$
= computed value at $\alpha = 60°$ when $r_c/T \leq 0.5$, or $\alpha \geq 60°$

V_m = Average velocity of flow immediately upstream of bend (ft/sec)

Y_{max} = Maximum depth of flow immediately upstream of bend (ft)

Y_h = Hydraulic depth of flow immediately upstream of bend (ft)

S_e = Energy slope immediately upstream of bend or bed slope for uniform flow conditions (ft/ft)

α = 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 (°) (see Figure 5)

For a simple circular curve approximating the bend, the following relationship exists.

$$\frac{r_c}{T} = \frac{\cos\alpha}{4\sin^2(\alpha/2)} \tag{15}$$

where: r_c = Centerline radius of curvature (ft)

T = Channel top width (ft)

α = 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 (°)

The distance to extend the bend scour component from the point of tangency of the bend was developed by Rozovskii (1961).

$$x = \frac{0.6}{n} Y^{1.17} \tag{16}$$

where: x = Distance from the end of channel curvature (point of tangency, PT) to the downstream point at which secondary currents have dissipated (ft)

n = Manning's roughness coefficient

Y = Depth of flow (ft)

In determining the extent of the bend scour, the bend scour should begin at the upstream point of curvature (PC) and extend a distance x (from the above equation) beyond the downstream point of tangency (PT).

Local scour occurs whenever there is an abrupt change in the direction of flow caused by obstructions to the flow. 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:

$$Z_{lse} = 2.15\sin(\theta_a)Y\left(\frac{a_e}{Y}\right)^{0.4} F_u^{0.33} \tag{17}$$

where: Z_{lse} = Local scour depth due to embankment (ft)
θ_a = Slope angle of embankment face, measured from the horizontal (°)
Y = Upstream normal flow depth (ft)
a_e = Embankment or encroachment length, measured normal to the edge of the floodplain or channel bank (ft)
F_u = Upstream Froude number

The location of the scour hole is at the upstream face of the embankment where the constriction of flows occurs. Defining the embankment length is difficult due to large differences between channel and overbank hydraulics. In the case where an overbank levee or flood wall is constructed adjacent to the main channel, the upstream Froude number should be based only on the hydraulic conditions for the overbank flow.

D. CLARK COUNTY FLOOD CONTROL

Clark County Regional Flood Control District, located in southern Nevada, published the "Hydrologic Criteria and Drainage Design Manual" (WRC Engineers, 1990), which presents a procedure to estimate scour at flood walls. The total scour that can occur at a particular location is equal to the sum of degradation, general scour, and local scour. Long-term aggradation or degradation is not included in this study.

General scour occurs when the flow area is contracted, causing an increase in velocity and bed shear stress. This results in more bed material being transported through the contracted reach. The depth of scour is obtained by subtracting the depth of flow in the upstream reach from the depth of flow in the contracted reach. The depth of scour in the contracted reach is computed by using continuity relationships and sediment transport equations.

Local scour is caused by vortices resulting from obstructions in the flow. The depth of scour at embankments follows the relationship developed by Liu et al. (1961) for subcritical flow conditions. For subcritical flow, the equilibrium scour depth for local scour at embankments can be expressed by the following equation:

$$\frac{Y_s}{Y_1} = 1.1\left(\frac{a}{Y_1}\right)^{0.40} Fr_1^{\ 0.33} \tag{5}$$

where: Y_s = The equilibrium scour depth measured from the mean bed level to the bottom of the scour hole (ft)

Y_1 = The upstream flow depth (ft)

a = The embankment length measured normal to the bank (ft)

Fr_1 = The upstream Froude number

The scour depth for long embankments, where a/Y_1 is large, can be calculated from the following equation.

$$\frac{Y_s}{Y_1} = 4 Fr_1^{\ 0.33} \tag{6}$$

Based on the previous discussion, the scour at flood walls along a channel that does not constrict the flow width is only due to long-term degradation. The Design Manual discusses scour of riprap-lined channel banks. For toe protection, it states that the riprap blanket shall extend a minimum of 3 feet below the channel bed (Suckow,1995). The total scour analysis, discussed above, may result in extending the toe protection deeper below the channel bed. Therefore, if the long-term degradation is greater than 3 feet, then the maximum depth of riprap is equal to the degradation depth.

E. DENVER URBAN DRAINAGE AND FLOOD CONTROL DISTRICT

The Denver Urban Drainage and Flood Control District published a set of documents that addresses analysis and design of drainage facilities. These documents are the "Urban Storm Drainage Criteria Manual," Volumes 1 and 2 (Wright-McLaughlin Engineers, 1969). The only discussion about protection against scour appears in the section for riprap linings (Section 5.4). It states that where only the channel sides are to be lined, the riprap blanket should be extended a minimum of 3 feet below the channel bottom. In fact, for bank protection requirements in sandy soils the Manual refers the reader to specific criteria found in the Design Guidelines for channels on sandy soils (Simons, Li & Associates, 1981).

The Design Guidelines for channels on sandy soils considers total scour at structures to be equal to the sum of degradation, general scour, local scour, and one-half of the antidune height. Degradation is not considered in this analysis. It suggests that bank protection should extend to a depth below the channel bed equal to the total scour depth.

General scour occurs when the flow area is contracted by embankments, channelization, and accumulation of debris. For flow confined to the channel, scour due to the contraction of flows is estimated from the following equations.

$$q_{s2} = \frac{W_1}{W_2} q_{s1} \tag{18}$$

where: q_{s1} = Sediment transport rate in the upstream channel (cfs/ft)
q_{s2} = Sediment transport rate in the contracted reach (cfs/ft)
W_1 = Width of upstream channel (ft)
W_2 = Width of contracted channel (ft)

Knowing that $q_1 = Y_1V_1$, Y_2 and V_2 can be determined using the equation $q_2 = Y_2V_2$. Then, the depth of scour due to a contraction can be determined as:

$$Y_s = Y_2 - Y_1 \tag{19}$$

Local scour at embankments is caused by vortex systems induced by the obstruction of flow. As discussed in previous sections, for subcritical flow the equilibrium scour depth for local scour at embankments can be expressed by the following equation (Liu et al., 1961):

$$\frac{Y_s}{Y_1} = 1.1\left(\frac{a}{Y_1}\right)^{0.40} Fr_1^{\,0.33} \tag{5}$$

where: Y_s = The equilibrium scour depth measured from the mean bed level to the bottom of the scour hole (ft)
Y_1 = The upstream flow depth (ft)
a = The embankment length measured normal to the bank (ft)
Fr_1 = The upstream Froude number

For large a/Y_1 values, the equation reduces to the following.

$$\frac{Y_s}{Y_1} = 4 Fr_1^{\,0.33} \tag{6}$$

An illustration of the embankment length for a natural channel with riprap protection is shown in Figure 6.

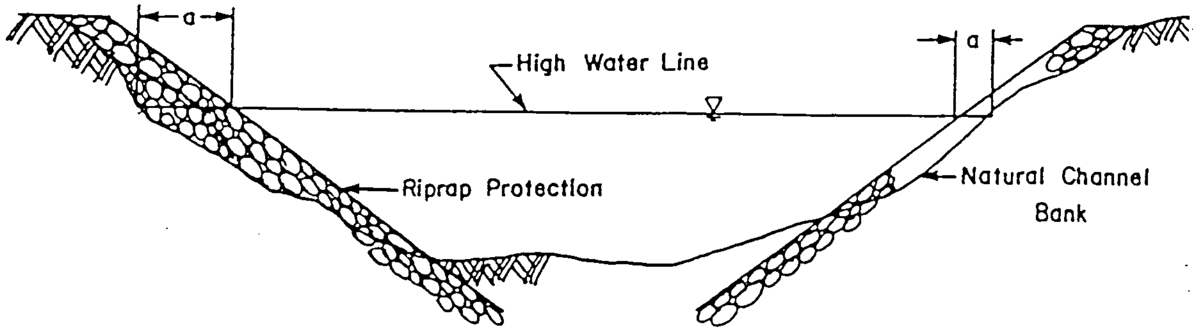


Figure 6 The embankment length (a) measured normal to the flow
(SOURCE: Simons, Li & Associates, 1981)

F. UNITED STATES BUREAU OF RECLAMATION

The United States Bureau of Reclamation (USBR) has developed methodologies for computing scour depth that are followed for all USBR facilities. These techniques are entirely different than the other seven methods discussed in this study. Pemberton and Lara (1984) authored a Technical Guideline for the USBR that describes these methods to estimate scour depth. This document is entitled "Guide for Computing Degradation and Local Scour." The guideline describes techniques for estimating general degradation, degradation limited by armoring, degradation limited by a stable slope, and channel scour during peak floodflows. This discussion focuses on the methods used to calculate channel scour during peak flows.

The Technical Guideline provides procedures to use in estimating maximum scour depth of channels during peak flood flows for design of a structure. Scour is defined as "the enlargement of a flow section by the removal of boundary material through the action of fluid motion during a single discharge event." Whereas, general degradation is "the long-term process by which streambeds and flood plains are lowered in elevation due to the removal of material from the boundary by flowing water." There are two processes of channel scour: 1) natural channel scour, and 2) scour induced by structures. Scour of the bed or banks of a channel by the introduction of structures is created by higher local velocities or turbulence at the structures.

The Technical Guideline presents several equations for estimating channel scour which have been empirically developed from experimental studies. Scour is calculated using the average channel hydraulics for a certain reach. If a structure restricts the width of flow, then scour is computed at the location of the restriction. Scour should be based on the discharge and hydraulics of the main channel only. Maximum channel scour is a function of channel geometry, obstruction created by a structure, velocity, turbulence, and size of bed and bank material. The collection of field data to define channel hydraulics and bed or bank materials is as important as the methodology selected to estimate scour. Also, experience and judgment are important in determining maximum channel scour.

There are four methods for estimating channel scour at a bankline structure (Neill, 1973). After the maximum channel scour is calculated using each equation, the engineer shall use best judgment to determine which methods are relevant. If more than one method is relevant, the average scour depth from all methods can be used. These four methods, including procedures for application, are given below.

1. Field Measurements of Scour Method

This method consists of observing or measuring the actual scour depth at the channel being studied or a similar channel. An equation was developed from scour data obtained from several streams in the southwestern United States (Abbott, 1963).

$$d_s = K(q)^{0.24} \tag{20}$$

where: d_s = Depth of scour below streambed (ft)
K = 2.45
q = Discharge per foot of channel width (cfs/ft)

This equation should only be used for wide sandbed (d_{50} from 0.5mm to 0.7mm), relatively steep slope channels (0.004 to 0.008 ft/ft) which is flat in the Southwest, or as a check on other methods.

2. Regime Equations Supported by Field Measurements Method

Three equations are used to estimate scour and provide a check on each other. The first method involves obtaining field measurements on an incised reach of the channel where the bankfull discharge and hydraulics can be determined (Neill, 1973).

$$d_f = d_i\left(\frac{q_f}{q_i}\right)^m \tag{21}$$

where: d_f = Scoured depth below design floodwater level
d_i = Average depth at bankfull discharge in incised reach
q_f = Design flood discharge per unit width
q_i = Bankfull discharge in incised reach per unit width
m = Exponent varying from 0.67 for sand to 0.85 for coarse gravel

The second equation involves the empirical regime equation by Lacey (1930):

$$d_m = 0.47\left(\frac{Q}{f}\right)^{0.33} \tag{22}$$

where: d_m = Mean depth at design discharge (ft)
Q = Design discharge (cfs)
f = Lacey's silt factor = $1.76(D_{50})^{0.5}$ - D_{50} is in millimeters

The third equation involves the zero-bed sediment transport by Blench (1969):

$$d_{fo} = \frac{q_f^{\ 0.667}}{F_{bo}^{\ 0.333}} \tag{23}$$

where: d_{fo} = Depth for zero-bed sediment transport (ft)
q_f = Design flood discharge per unit width (cfs/ft)
F_{bo} = Blench's "zero-bed factor" (ft/s^2) from figure 7

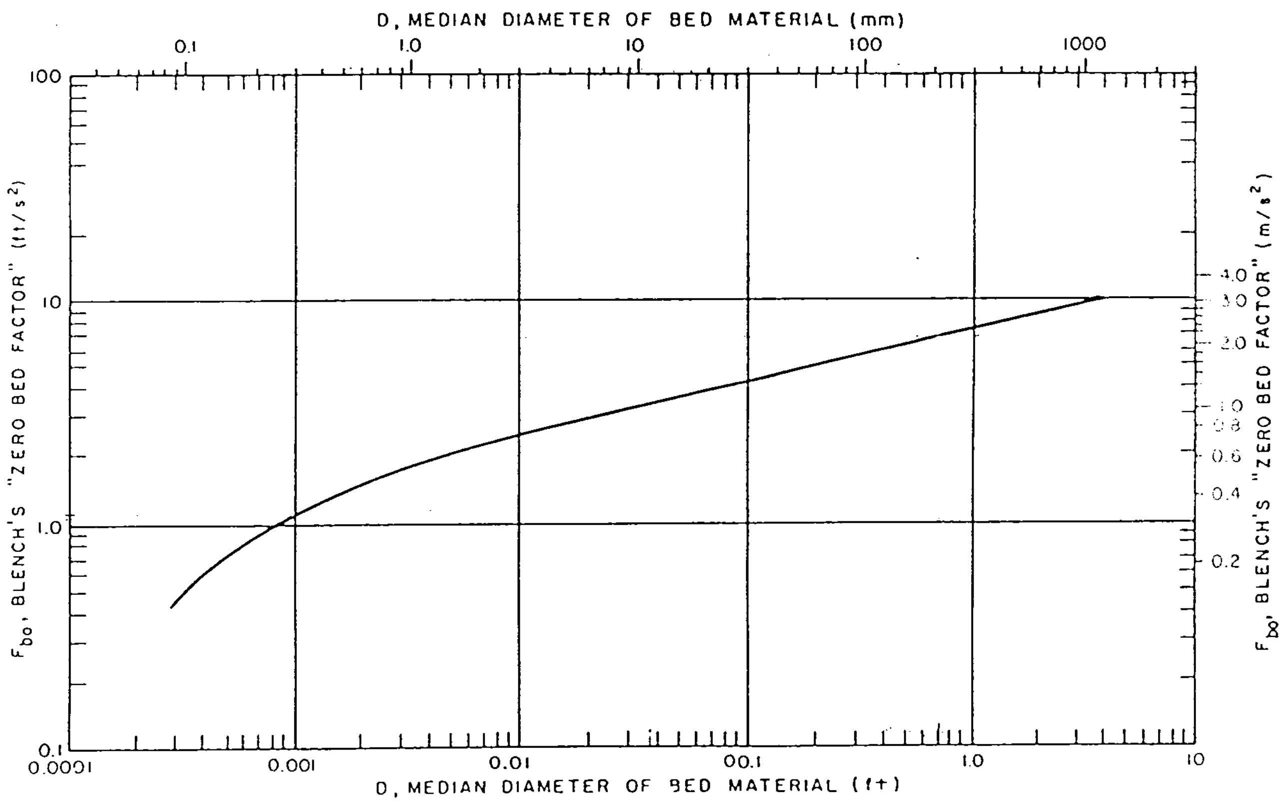


Figure 7 Chart for estimating the zero-bed factor, Fbo
(SOURCE: Pemberton and Lara, 1984)

Each of the above three equations are multiplied by an empirical multiplying factor, Z, shown in Table 2 to account for the probable concentration of flows in some portion of the natural channel.

$$d_s = Zd_f \tag{24}$$

$$d_s = Zd_m \tag{25}$$

$$d_s = Zd_{fo} \tag{26}$$

Table 2 Z factors for Regime equations (SOURCE: Pemberton and Lara,1984)

Condition	Value of Z		
	Neill $d_s = Zd_f$	Lacey $d_s = Zd_m$	Blench $d_s = Zd_{fo}$
Straight Reach	0.5	0.25	0.6
Moderate Bend	0.6	0.5	0.6
Severe Bend	0.7	0.75	0.6
Right Angle Bends		1.0	1.25
Vertical Rock Bank or Wall		1.25	

Figure 8 shows the maximum scour depth of a natural channel using the regime equations.

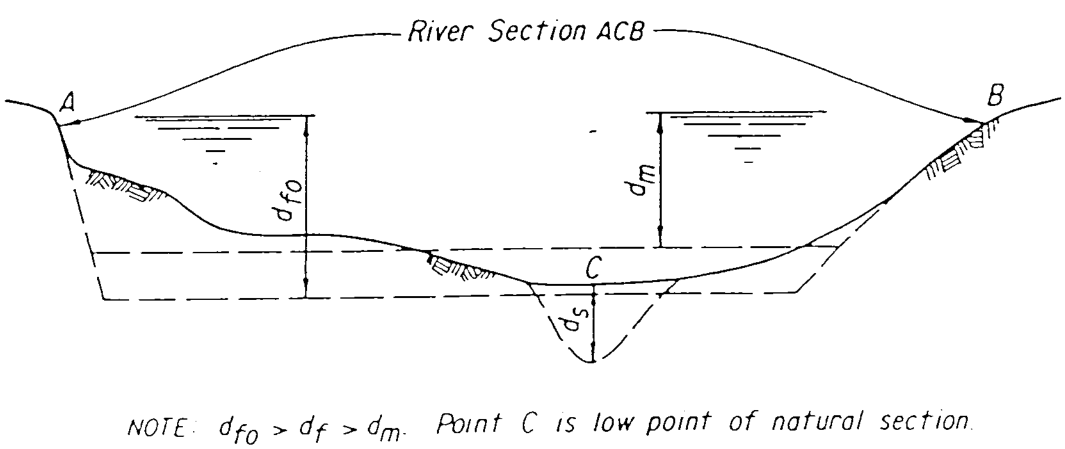


Figure 8 Sketch of natural channel scour by regime method (SOURCE: Pemberton and Lara, 1984)

3. Mean-Velocity from Field Measurements Method

This method makes an adjustment in surveyed channel cross-sections based on an extrapolated design flow velocity. A minimum of four cross-sections are required to perform water surface elevation computations. In order to verify Manning's roughness factor (n), observed water surface elevations at known discharges are also required. The USBR application of this method involves determining the mean channel depth, d_m, from the water surface computations and then adjusting the mean channel depth by the Z values defined by Lacey in Table 2 to obtain the scour depth, d_s.

$$d_s = Zd_m \tag{25}$$

4. Competent or Limiting Velocity Control to Scour Method

This method assumes that scour will occur in the channel cross section until the mean velocity is reduced so that no movement of bed material is occurring. It is similar to the Blench equation for a "zero bed factor." The scour depth or increase in area of scoured channel section is computed from the following equation.

$$d_s = d_m\left(\frac{V_m}{V_c} - 1\right) \tag{27}$$

where: d_s = Scour depth below streambed (ft)

d_m = Mean flow depth (ft)

V_m = Mean velocity (fps)

V_c = Competent mean velocity (fps)

The competent mean velocities for erosion of cohesive materials shown in Figure 9 are recommended by Neill (1973).

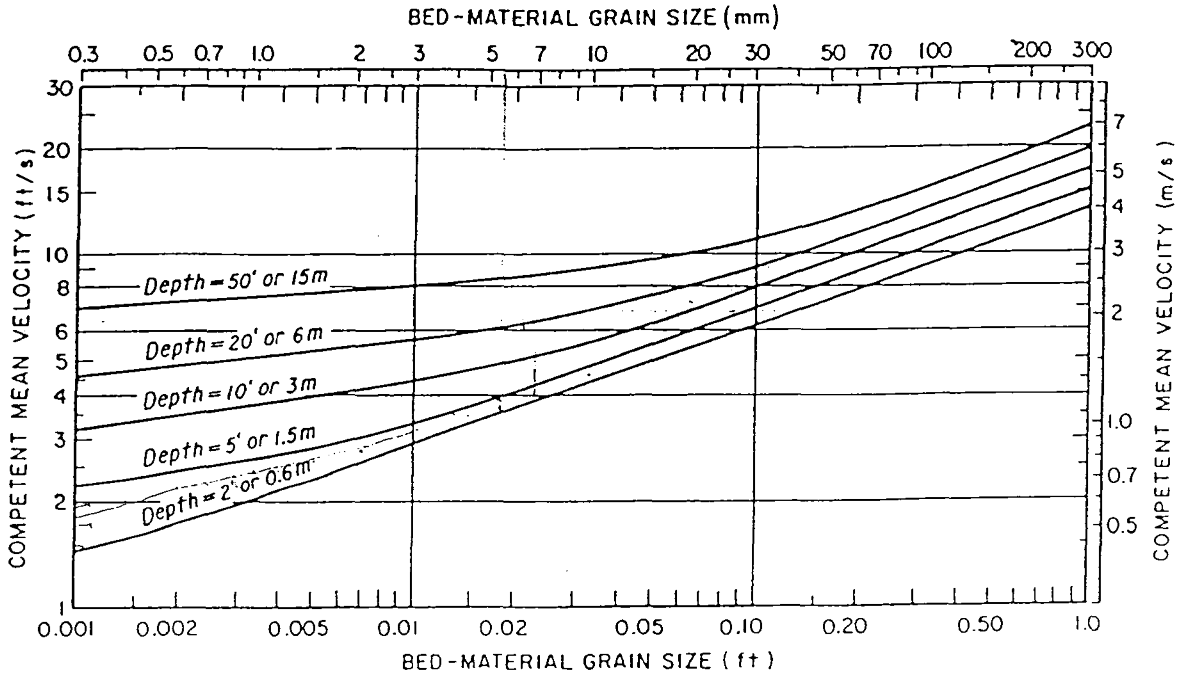


Figure 9 Suggested competent mean velocities for significant bed movement of cohesionless materials (SOURCE: Pemberton and Lara, 1984)

G. *FEDERAL HIGHWAY ADMINISTRATION*

Two recent publications, Hydraulic Engineering Circular (HEC) No. 18 (Lagasse et al., 1991) and Hydraulic Engineering Circular No. 20 (Richardson et al., 1991), have been developed by the Federal Highway Administration (FHWA) to provide guidelines for estimating scour at highway crossings of stream. According to HEC 18, total scour includes the addition of three types of scour: aggradation or degradation scour, contraction or general scour, and local scour. There are two types of local scour: clear-water scour and live-bed scour. The predominant type of local scour in the Southwest is live-bed scour. Live-bed scour is where scour depth increases during the rising limb of the storm and refills with sediment during the recession of a hydrograph.

HEC 18 deals specifically with scour that occurs at bridge crossings. Equations are given to calculate scour at bridge piers and bridge abutments. The Liu et al. (1961) equation (equation 6) for local scour at embankments where a/Y_1 is large ($a/Y_1 > 25$), is given. To estimate the scour along the toe of riprap bank revetments HEC 20 refers to Hydraulic Engineering Circular No. 11 (Brown and Clyde, 1989). In HEC 11, scour at channel banks is discussed in the section concerning vertical extent of bank revetments. The ultimate depth of scour at the toe of a bank lining includes the degradation of the channel bed and the natural scour caused by the bank lining.

Calculation of the long-term degradation of the channel bed is not addressed in HEC 11.

The following equation (Blodgett, 1986) estimates the probable maximum scour depth due to natural scour and fill in straight channels and channels with mild bends.

$$d_s = 12 \text{ ft} \qquad \text{for } D_{50} < 0.005 \text{ ft} \tag{28}$$

$$d_s = 6.5 D_{50}^{-0.11} \qquad \text{for } D_{50} > 0.005 \text{ ft} \tag{29}$$

where: d_s = Estimated probable maximum depth of scour (ft)
D_{50} = Median diameter of bed material (ft)

The depth of scour (d_s) should be measured from the lowest elevation of the channel section. Also, the depth of scour must be added to the depth of long-term degradation and local scour, if any, to obtain the total required toe depth.

H. U.S. ARMY CORPS OF ENGINEERS

The Albuquerque District of the U.S. Army Corps of Engineers (USACE) employs several equations to estimate scour at channel banks, and then, based on experience, a maximum scour depth is determined (D'Antonio and Eidson, 1995). The first method used to obtain a gross estimate of scour depth is discussed in the USACE document "Hydraulic Design of Flood Control Channels," (USACE, 1991a). Secondary methods that are used to check the USACE method are the AMAFCA method described in Section III.A. and the USBR method discussed in Section III.F. Finally, experience with channels of similar hydraulic bed material properties is used to make a final determination of maximum scour.

According to the USACE method, the mechanisms contributing to revetment toe scour are general bed degradation and local scour. General degradation can be determined by numerical modeling methods such as HEC 6 (USACE, 1991b). Approximate local scour depths can be estimated using Figure 10, which was developed from measurements taken at the Mississippi River and Red River at bends in the river alignment. More accurate scour depth information can be obtained from Neill (1973), which was discussed in Section III.F.

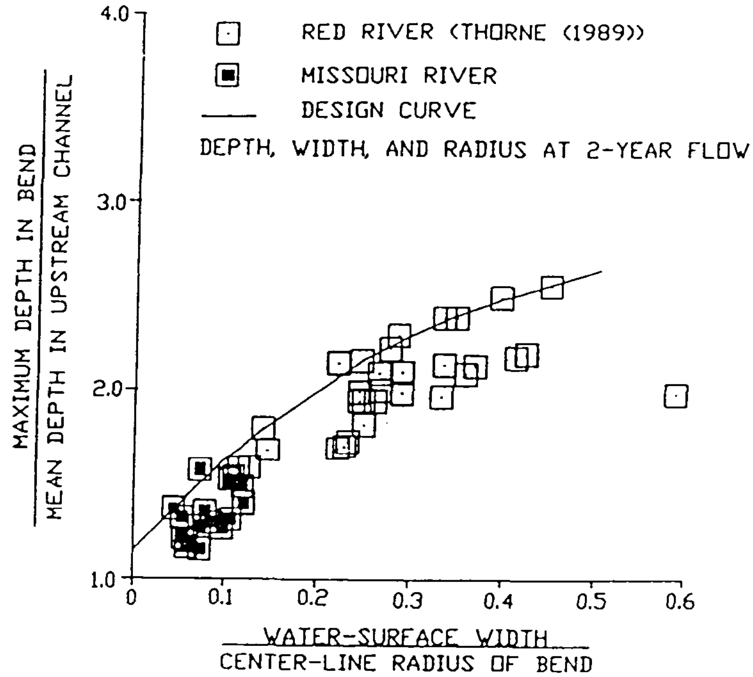


Figure 10 Chart to obtain local scour depth for sand-bed channels with revetted bends (SOURCE: USACE, 1991a)

Once the scour depth is calculated from each method, the Albuquerque District determines a maximum depth of scour based on their experience with channels of similar hydraulic conditions. Both the AMAFCA method and USBR method are evaluated by the Albuquerque District. A common method of bank protection used by the USACE on sand bed streams involves the placement of "launchable riprap" along the banks or bed of the channel. When the launchable riprap fills in the scour hole, it provides a sacrificial layer of riprap which can prevent further scour by the "armoring" principle. Using the launchable riprap technique, the maximum scour depth expected for channels with a high scour potential is 5-8 feet and for channels with a moderate scour potential is 3-5 feet (D'Antonio and Eidson, 1995).

IV. DESCRIPTION OF MIREHAVEN ARROYO PROJECT

In 1993, storm flows in the Mirehaven Arroyo eroded the banks of the channel at the Las Lomitas Unit IIB Subdivision in northwest Albuquerque (Figure 11). As a result of the bank erosion, the arroyo encroached into the property. Therefore, it was decided that a scour wall would be constructed adjacent to the arroyo to protect the property from further damage. In late 1994, a soil cement scour wall was designed and constructed along the north side of the Mirehaven Arroyo at Las Lomitas. As part of this project, the arroyo was regraded to move the channel section away from the property and back within the designated drainage easement.

The Mirehaven Arroyo is a slightly meandering, steep, sand-bed channel. The average slope in the channel reach along the subdivision is 0.0212 ft/ft. There is no vegetation within the channel bottom and sparse vegetation on its side-slopes. In the middle of the channel reach within the project limits, between channel centerline stations 16+00 and 21+75 (see Figures 12 and 13), is a channel bend with a centerline radius of curvature of about 325 feet. The bottom width of the channel ranges from 20 feet to 80 feet with an average of about 40 feet. The side-slopes range from 1:1 on the outside of the bend to 10:1 on the inside of the bend. The channel section geometry through the bend indicates that the outside bank is experiencing scour while deposition of sediment is occurring along the inside bank. The sinuosity of the arroyo reach next to the subdivision is 1.14. The arroyo is classified as a straight channel since its sinuosity is less than 1.5.

A sieve analysis of the bed material was performed to determine the particle size distribution of the bed material. The results indicate that 100% of the bed material passes the 3/4" sieve and the mean diameter (D_{50}) is 0.6 mm. The bed material can generally be classified as a medium sand.

The 100-year peak flow rate of 1677 CFS was determined from a previous study of the Mirehaven Arroyo watershed (Easterling & Associates, 1988). Field cross-sections of the arroyo (see Figures 12 and 13) were obtained to be used as input for a water surface profile model using HEC 2 (USACE, 1990). The field cross-sections were modified to be consistent with the proposed channel realignment and flood wall construction. The channel realignment and widening to the width required to pass the dominant discharge, was designed according to the AMAFCA method. The channel modifications are located between stations 16+00 and 23+38 (see Figures 12 and 13). The Manning's n values in the model were not calculated for a flood wall along one bank of the arroyo because the flood wall is completely buried with soil. The Manning's n value was assumed to be 0.035 for the entire channel section. A HEC 2 model was developed and run for the proposed supercritical flow conditions of the arroyo. Table 3 shows the results of the proposed conditions supercritical HEC 2 model. After analyzing the model, it was concluded that two of the cross-sections,

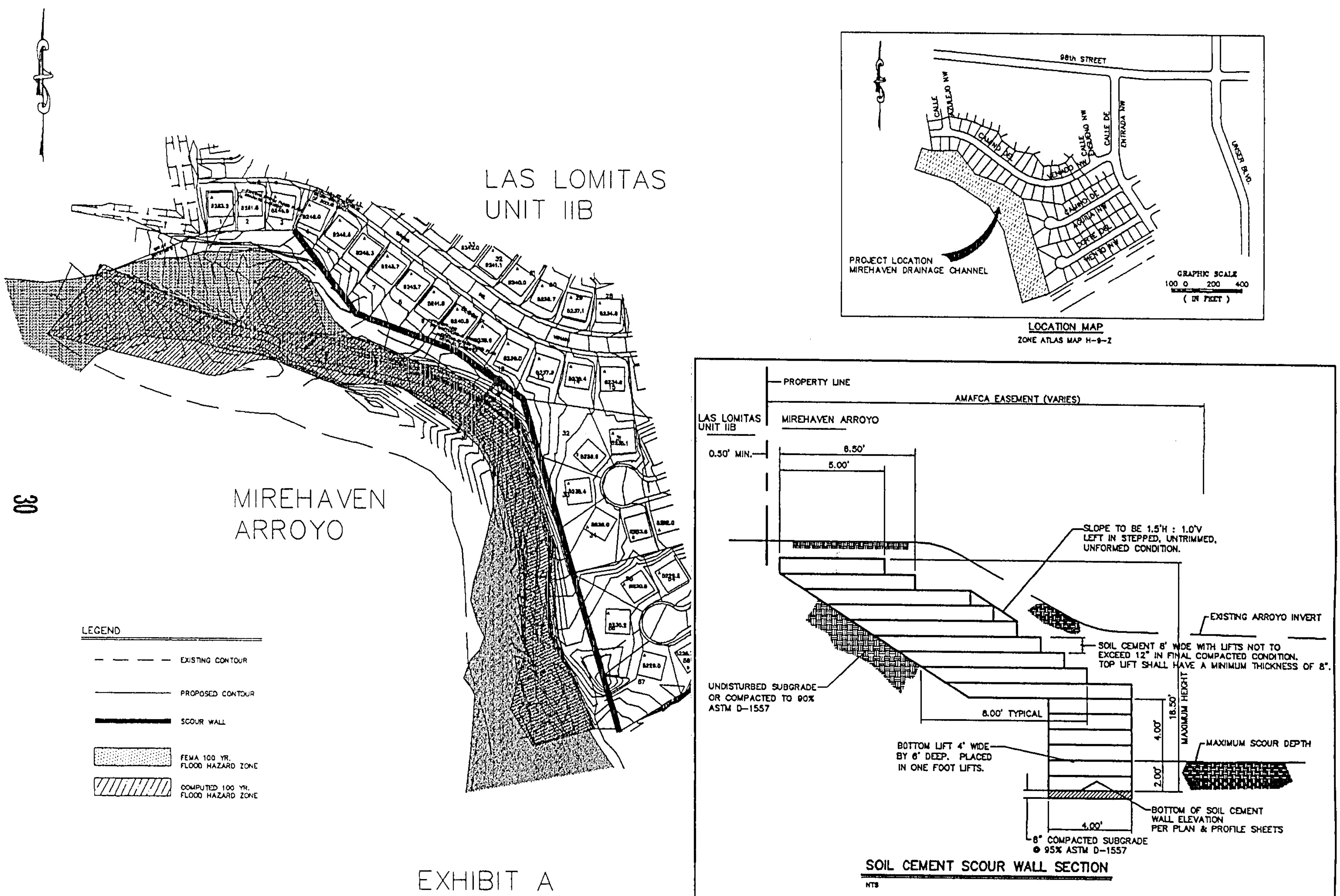


Figure 11 Location map, overall plan view, and soil cement wall detail of Las Lomitas Unit IIB Subdivision (SOURCE: AVID Engineering, Inc., 1994)

AVID ENGINEERING, INC.
Civil • Structural • Transportation
6100 Seagull St. NE – Suite 102
Albuquerque, NM 87109 – (505) 881-5357

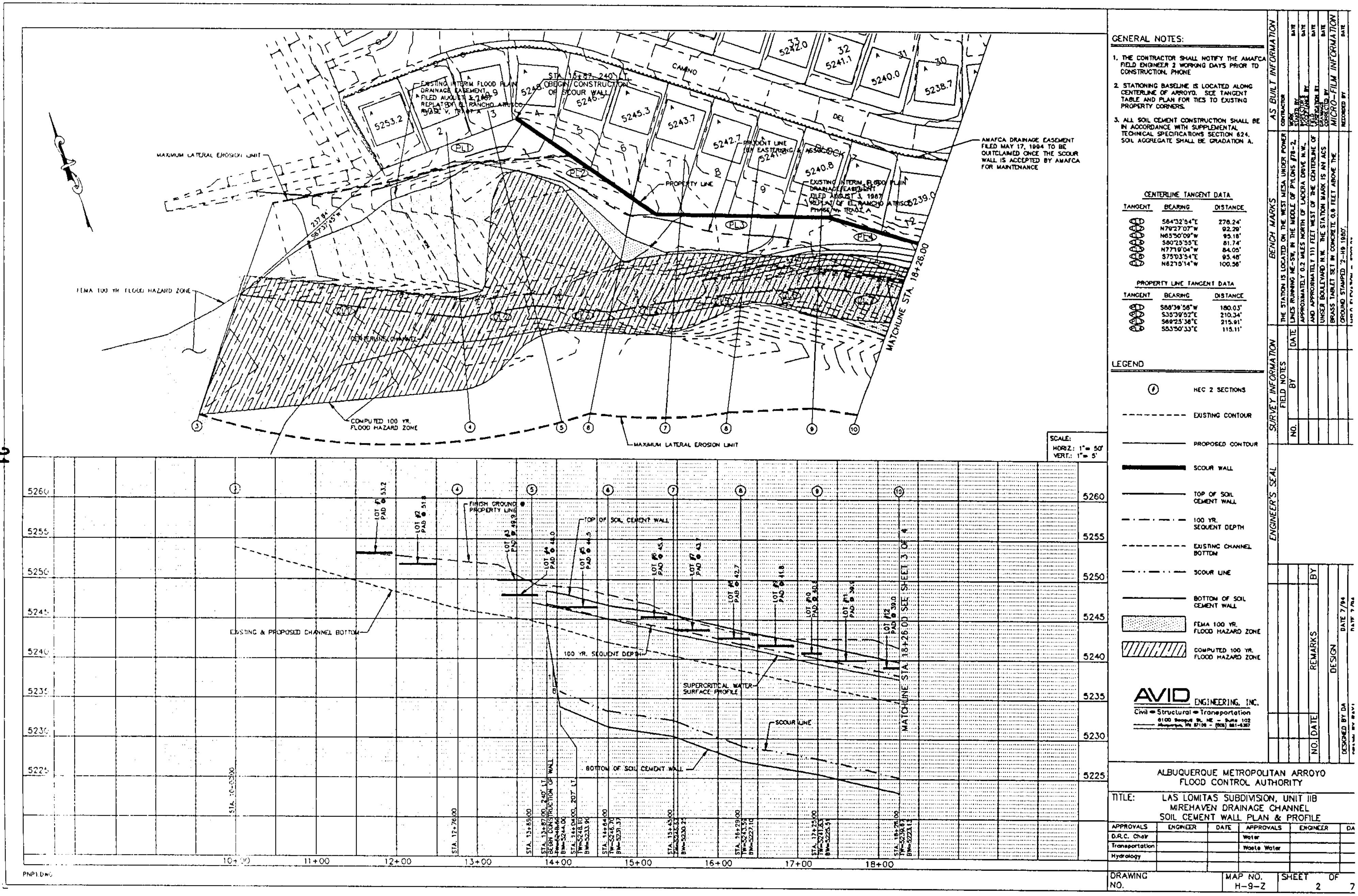


Figure 12 Plan and Profile of the flood wall at Las Lomitas Unit IIB Subdivision, Stations 10+00 to 18+26 (SOURCE: AVID Engineering, Inc., 1994)

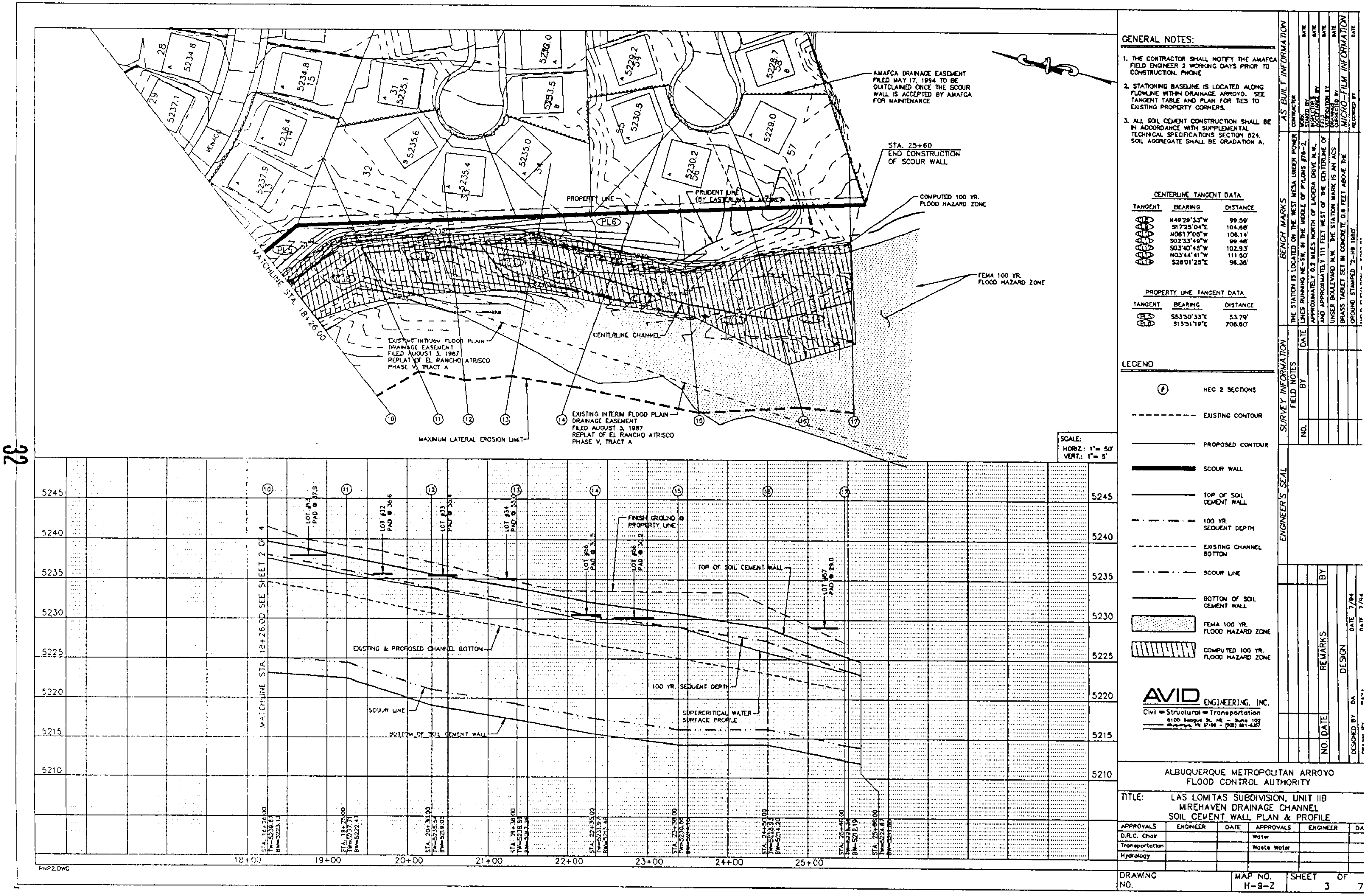


Figure 13 Plan and Profile of the flood wall at Las Lomitas Unit IIB Subdivision, Stations 18+26 to 25+46 (SOURCE: AVID Engineering, Inc., 1994)

which reached critical depth during the supercritical run, are at critical depth and do not jump to subcritical depth.

Table 3 Results of HEC 2 run with supercritical flow conditions

SEC. NO.	STA.	VEL. (FT.)	AREA (FT^2)	TOPWID (FT)	DEPTH (FT)	FROUDE
3	10+00	6.98	248.43	177.44	2.83	0.97
4	12+76	11.32	148.21	92.52	2.36	1.58
5	13+69	7.93	246.48	204.91	2.30	1.01
6	14+64	8.42	202.65	95.22	2.99	0.98
7	15+45	10.11	168.63	94.17	2.78	1.29
8	16+29	9.41	178.26	80.66	2.99	1.12
9	17+25	10.19	164.52	81.43	2.86	1.26
10	18+26	9.78	171.50	78.74	2.92	1.17
11	19+25	9.60	174.68	92.25	2.60	1.23
12	20+30	9.03	185.71	79.83	3.08	1.04
13	21+36	9.73	172.36	79.43	3.07	1.16
14	22+35	10.18	164.70	79.32	2.81	1.25
15	23+38	8.58	204.21	93.51	3.92	0.93
16	24+50	11.43	146.66	104.79	2.64	1.74
17	25+46	8.31	201.80	123.78	2.24	1.15

At the Manning's roughness (n) of 0.035, the flow in the arroyo is mostly in the supercritical regime, although near critical depth. The Froude numbers range from 0.93 to 1.74. The flow velocities average about 9.4 ft/sec with a minimum of 6.98 and maximum of 11.43 ft/sec. The flow depth stays around 2.5 to 3.0 feet while the flow is deeper in the channel bend, as would be expected. Also, the topwidth through the channel bend is narrower than the straight section due to the more uniformly incised channel section in the bend.

V. RESULTS OF SCOUR DEPTH CALCULATIONS USING EACH METHOD

A. *ALBUQUERQUE METROPOLITAN ARROYO FLOOD CONTROL AUTHORITY*

The first step in computing the maximum scour depth using the AMAFCA method is to determine the maximum lateral erosion for the 100-year flow in the arroyo. The maximum lateral erosion is based on determining the optimal bend shape of the channel. The dominant discharge, dominant channel width, and maximum lateral erosion setback from the channel centerline are calculated (Table 4).

Table 4 Results of maximum lateral erosion procedure

COL1	COL2	COL3	COL4	COL5	COL6	COL7	COL8	COL9	COL10	COL11	COL12
SEC NO	STA	Q_{100} (CFS)	Q_D (CFS)	S_C (F/F)	S_{AVG} (F/F)	W_D (FT)	λ	LV (FT)	Δ_{MAX} (FT)	EROS SET(F)	UVW (FT)
3	1000	1677	335.4	0.017	0.028	47.10	513.45	256.73	128.36	151.91	303.82
4	1276	1677	335.4	0.017	0.016	47.33	516.01	258.00	129.00	152.67	305.33
5	1369	1677	335.4	0.017	0.028	47.10	513.45	256.73	128.36	151.91	303.82
6	1464	1677	335.4	0.017	0.021	47.10	513.45	256.73	128.36	151.91	303.82
7	1545	1677	335.4	0.017	0.021	47.10	513.45	256.73	128.36	151.91	303.82
8	1629	1677	335.4	0.017	0.021	47.10	513.45	256.73	128.36	151.91	303.82
9	1725	1677	335.4	0.017	0.020	47.10	513.45	256.73	128.36	151.91	303.82
10	1826	1677	335.4	0.017	0.017	47.10	513.45	256.73	128.36	151.91	303.82
11	1925	1677	335.4	0.017	0.021	47.10	513.45	256.73	128.36	151.91	303.82
12	2030	1677	335.4	0.017	0.019	47.10	513.45	256.73	128.36	151.91	303.82
13	2136	1677	335.4	0.017	0.019	47.10	513.45	256.73	128.36	151.91	303.82
14	2235	1677	335.4	0.017	0.017	47.11	513.63	256.82	128.41	151.96	303.93
15	2338	1677	335.4	0.017	0.017	46.85	510.76	255.38	127.69	151.11	302.23
16	2450	1677	335.4	0.017	0.021	47.10	513.45	256.73	128.36	151.91	303.82
17	2546	1677	335.4	0.017	0.021	47.10	513.45	256.73	128.36	151.91	303.82

Column 1 - Channel cross-section number
Column 2 - Channel centerline station
Column 3 - 100-year peak discharge, Q_{100} (CFS)
Column 4 - Dominant discharge, $Q_D = 0.2Q_{100}$ (CFS)
Column 5 - Critical slope, $S_c = 0.037Q_D^{-0.133}$ (ft/ft)
Column 6 - Average slope of the arroyo, S_{AVG} (ft/ft)
Column 7 - Dominant channel width, $W_D = 4.6Q_D^{0.4}$ (ft)
Column 8 - The meander wavelength, $\lambda/W_D = 0.8 + 4\log(Q_D)$ (ft)
Column 9 - Down-valley length, $LV = \lambda/2$ (ft)
Column 10 - Maximum lateral erosion, $\Delta_{max} = [0.92 + 4.6\log(Q_D)]Q_D^{0.4}$ (ft
Column 11 - Erosion setback $= W_D/2 + \Delta_{max}$ (ft)
Column 12 - Unconstrained valley width, UVW = 2 x erosion setback (ft)

Once the HEC 2 and maximum lateral erosion analyses were complete, the total scour was estimated using the methodology described in Section III.A. This method involves determining the hydraulic depth of the channel sections. After the hydraulic depth is calculated, the Froude numbers are calculated using the hydraulic depth. The unconstrained valley width was reduced to the location of the scour wall. With the Froude number and ratio of unconstrained valley width/dominant channel width, the ratio of scour depth/flow depth is obtained from Figure 4. Table 5 gives the results of the scour calculations. The descriptions of the column headings are in the table.

Table 5 Results of scour depth calculations using AMAFCA method

COL1	COL2	COL3	COL4	COL5	COL6	COL7	COL8	COL9	COL10	COL11	COL12
SEC. NO.	STA.	Q (CFS)	VEL (FPS)	FRDE. NO.	AREA (SF)	TW (FT)	HD (FT)	UVW (FT)	UVW/ WD	YS/Y	YS(FT)
6	1464	1677	8.42	1.02	202.65	95.22	2.13	296.00	6.30	4.10	9.73
7	1545	1677	10.11	1.33	168.63	94.17	1.79	260.00	5.53	4.55	9.15
8	1629	1677	9.41	1.12	178.26	80.66	2.21	250.00	5.32	4.30	10.50
9	1725	1677	10.27	1.27	164.52	81.43	2.02	237.00	5.04	4.50	10.09
10	1826	1677	9.78	1.17	171.50	78.74	2.18	224.00	4.77	4.35	10.47
11	1925	1677	9.60	1.23	174.68	92.25	1.89	192.00	4.09	4.45	9.43
12	2030	1677	9.03	1.04	185.71	79.83	2.33	187.00	3.98	4.15	10.65
13	2136	1677	9.87	1.18	172.36	79.43	2.17	202.00	4.30	4.35	10.44
14	2235	1677	10.18	1.24	164.70	79.32	2.08	237.00	5.04	4.50	10.34
15	2338	1677	8.58	1.02	204.21	93.51	2.18	267.00	5.68	4.10	9.95
16	2450	1677	11.43	1.70	146.66	104.79	1.40	292.00	6.21	5.00	8.00
17	2546	1677	8.31	1.15	201.80	123.78	1.63	272.00	5.79	4.30	8.01

Column 1	-	Channel cross-section number
Column 2	-	Channel centerline station
Column 3	-	100-year peak discharge, Q (CFS)
Column 4	-	Channel velocity (VEL) from HEC 2 (ft/sec)
Column 5	-	Froude number calculated with hydraulic depth
Column 6	-	Cross-sectional area (AREA) from HEC 2 (ft^2)
Column 7	-	Top width (TW) from HEC 2 (ft)
Column 8	-	Hydraulic depth, HD = A/TW (ft)
Column 9	-	Unconstrained valley width, UVW (ft)
Column 10	-	Ratio of unconstrained valley width/dominant channel width, UVW/WD
Column 11	-	Ratio scour depth/flow depth, YS/Y
Column 12	-	Estimated scour (YS) including a factor of safety of one foot (ft)

The maximum scour depth ranges from 8.00 feet at station 24+50 to 10.65 feet at station 20+30. As a factor of safety, one foot is added to the scour depth. For the example project, two feet were added to the scour depth due to the 4-foot wide by 6-foot deep "plug" located at the bottom of the soil cement scour wall (Figure 11).

B. *MARICOPA COUNTY FLOOD CONTROL DISTRICT*

Although Mirehaven Arroyo in the example project would be considered a minor watercourse by Maricopa County standards and would be hard-lined, this study will use that county's scour equations for major watercourses to compute the maximum scour. The total scour acting on the scour wall of the example project includes bed-form scour due to the passage of dunes and antidunes and bend scour. The channel section is not constricted and there are no obstructions in the flow; therefore, contraction scour and local scour are not included in the calculations. The hydraulic depth of flow and Froude number for each channel section were used to determine bed-form scour from antidunes.

To calculate the bend scour, an estimation of α, which is the 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, is required (see Figure 5). The centerline radius of curvature of the channel bend and the channel top width at station 16+29, which is near the beginning of the bend, were input to equation (15) to compute the angle α by trial and error. For the example problem $\alpha = 27.2^\circ$ or 0.4747 radians. These parameters plus the maximum depth of flow at station 16+29 of 2.99 feet were used to compute the bend scour of 0.58 feet. The bend scour acts on the channel bank through the bend from station 16+29 to station 21+36.

To obtain the total scour, the component scour depths were added and then multiplied by the appropriate factor of safety. The total scour within the project reach ranges from 1.41 feet to 2.68 feet. This is less than the minimum toe depth of 3 feet to protect against scour. Therefore, in this case 3 feet should be used for bank protection. Table 6 illustrates the scour calculations.

Table 6 Results of scour depth calculations using Maricopa County method

STA.	Q (CFS)	VEL (FPS)	FRDE	AREA (SF)	TOP WID(FT)	HYD DEP(FT)	ADUNE (FT)	BEND SCOUR	FS	SCOUR (FT)
1464	1677	8.42	1.02	202.65	95.22	2.13	0.97		1.50	1.45
1545	1677	10.11	1.33	168.63	94.17	1.79	1.40		1.50	2.09
1629	1677	9.41	1.12	178.26	80.66	2.21	1.21	0.58	1.30	2.33
1725	1677	10.27	1.27	164.52	81.43	2.02	1.44	0.58	1.30	2.63
1826	1677	9.78	1.17	171.50	78.74	2.18	1.31	0.58	1.30	2.46
1925	1677	9.60	1.23	174.68	92.25	1.89	1.26	0.58	1.30	2.39
2030	1677	9.03	1.04	185.71	79.83	2.33	1.11	0.58	1.30	2.21
2136	1677	9.87	1.18	172.36	79.43	2.17	1.33	0.58	1.30	2.49
2235	1677	10.18	1.24	164.70	79.32	2.08	1.42		1.50	2.12
2338	1677	8.58	1.02	204.21	93.51	2.18	1.01		1.50	1.51
2450	1677	11.43	1.70	146.66	104.79	1.40	1.78		1.50	2.68
2546	1677	8.31	1.15	201.80	123.78	1.63	0.94		1.50	1.41

C. CITY OF TUCSON DEPARTMENT OF TRANSPORTATION, ENGINEERING DIVISION

The City of Tucson method for computing total scour is similar to the Maricopa County method. Local scour was not considered for the example project. The antidune trough depth and bend scour depth are the same as those described for Maricopa County. In addition, general scour and a low-flow thalweg depth of 1 foot are added to obtain total scour. At certain channel sections, the general scour equation gives results that are less than zero. At these sections, the general scour is assumed to be zero. Once all the component scour depths are computed, the scour depths are added and multiplied by a factor of safety of 1.3. Table 7 shows that the total scour depth ranges from 2.56 feet to 5.00 feet.

Table 7 Results of scour depth calculations using the City of Tucson method

STA.	Q (CFS)	VEL (FPS)	FRDE	AREA (SF)	TOP WID(FT)	HYD DEP(FT)	GEN SC (FT)	ADUNE (FT)	BEND SCR	FS	SCOUR (FT)
1464	1677	8.42	1.02	202.65	95.22	2.13	0.00	0.97		1.30	2.56
1545	1677	10.11	1.33	168.63	94.17	1.79	0.27	1.40		1.30	3.47
1629	1677	9.41	1.12	178.26	80.66	2.21	0.00	1.21	0.58	1.30	3.64
1725	1677	10.27	1.27	164.52	81.43	2.02	0.17	1.44	0.58	1.30	4.16
1826	1677	9.78	1.17	171.50	78.74	2.18	0.00	1.31	0.58	1.30	3.76
1925	1677	9.60	1.23	174.68	92.25	1.89	0.27	1.26	0.58	1.30	4.05
2030	1677	9.03	1.04	185.71	79.83	2.33	0.00	1.12	0.58	1.30	3.51
2136	1677	9.87	1.18	172.36	79.43	2.17	0.12	1.33	0.58	1.30	3.95
2235	1677	10.18	1.24	164.70	79.32	2.08	0.21	1.42		1.30	3.42
2338	1677	8.58	1.02	204.21	93.51	2.18	0.00	1.01		1.30	2.61
2450	1677	11.43	1.70	146.66	104.79	1.40	1.06	1.79		1.30	5.00
2546	1677	8.31	1.15	201.80	123.78	1.63	0.01	0.95		1.30	2.54

D. CLARK COUNTY FLOOD CONTROL

Since in the example problem, there is no contraction of the flow area and there are no flow obstructions within the scour wall reach, lowering of the channel bed is caused by long-term degradation. This paper does not address long-term degradation. Therefore it is assumed that the scour depth equals the minimum extension of a riprap blanket below the bed for toe protection, which is 3 feet.

E. DENVER URBAN DRAINAGE AND FLOOD CONTROL DISTRICT

For the Denver method, being similar to the Clark County method, the same assumption that the scour depth equals the minimum toe protection depth for riprap of 3 feet was used. Again, there is no general (or contraction) scour or local scour, so only long-term degradation is occurring to lower the channel bed.

As an example of local scour acting on the scour wall, if the scour wall was not covered with soil in the example project, then local scour would occur due to the embankment length projecting into the flow area. If future flood flows remove the soil away from the soil cement scour wall and cause it to project into the flow area, then local scour would occur at the upstream face of the projection. In calculating this local scour, the embankment length (a) could be as much as 8 feet, which is the width of the soil cement layers in the scour wall. It would be difficult to predict where the scour wall could be exposed and thus where the local scour would occur. Therefore, local scour depth was calculated for the entire length of the wall. The local scour depth is given in Table 8. The local scour depths range from 4.29 feet at station 25+46 to 5.78 at station 23+38.

Table 8 Results of local scour depth calculations using the Denver method

STA.	Q (CFS)	VEL (FPS)	FRDE	AREA (SF)	TOP WID(FT)	HYD DEP(FT)	HEC DEP(FT)	SLOPE (F/F)	LOCAL SC
1464	1677	8.42	1.02	202.65	95.22	2.13	2.99	0.0199	4.90
1545	1677	10.11	1.33	168.63	94.17	1.79	2.78	0.0213	5.13
1629	1677	9.41	1.12	178.26	80.66	2.21	2.99	0.0212	5.05
1725	1677	10.27	1.27	164.52	81.43	2.02	2.86	0.0211	5.14
1826	1677	9.78	1.17	171.50	78.74	2.18	2.92	0.0204	5.06
1925	1677	9.60	1.23	174.68	92.25	1.89	2.60	0.0168	4.80
2030	1677	9.03	1.04	185.71	79.83	2.33	3.08	0.0214	5.03
2136	1677	9.87	1.18	172.36	79.43	2.17	3.07	0.0185	5.23
2235	1677	10.18	1.24	164.70	79.32	2.08	2.81	0.0190	5.05
2338	1677	8.58	1.02	204.21	93.51	2.18	3.92	0.0173	5.78
2450	1677	11.43	1.70	146.66	104.79	1.40	2.64	0.0181	5.39
2546	1677	8.31	1.15	201.80	123.78	1.63	2.24	0.0192	4.29

F. UNITED STATES BUREAU OF RECLAMATION

The USBR employs several methods to estimate scour and then suggests that experience and best judgment be used to determine which method is most applicable or average the more applicable methods to ascertain the design scour depth. Five of the six techniques for estimating scour were followed for the example project. The mean-velocity from field measurements method was not included because there were no observed water surface elevations at known discharges to verify Manning's roughness factor (n).

The first equation utilized, the field measurements of scour method (equation 20), was calculated to be used as a check on the other methods. The average width of the channel was calculated by averaging the top width and bottom width at each channel section. Then the average width at each section was divided into the peak discharge to obtain the discharge per unit width of channel. The depth of scour ranges from 4.56 feet at station 13+69 to 5.39 feet at station 18+26.

The Neill bankfull discharge regime equation (equation 21) was used to estimate the scour. In order to determine the depth at bankfull discharge the actual surveyed cross sections were reviewed and a bankfull height measured. To obtain the bankfull discharge, the HEC 2 model was run with varying discharges until the bankfull depths at each cross section were reached. The bankfull discharges and corresponding top widths were then used to compute the bankfull discharge per unit width. Table 9 gives the results of the scour depth using the Neill equation. From station 13+69 to station 15+45 and at station 23+38, the Neill scour depth was multiplied by a Z factor of 0.5 for straight reaches (from Table 2). While a Z factor of 0.6 for moderate bends was used from station 16+29 to station 22+35.

Table 9 Results of scour depth calculations using the USBR Neill bankfull discharge equation (21)

STA.	Q (CFS)	VEL (FPS)	FROUDE	AREA (SF)	TOP WID(FT)	HYD DEP(FT)	Q_f (CFS)	TW_f (FT)	d_i (FT)	NEILL ds (FT)
1464	1677	8.42	1.02	202.65	95.22	2.13	2250	109.70	3.50	1.53
1545	1677	10.11	1.33	168.63	94.17	1.79	6000	165.20	5.00	1.40
1629	1677	9.41	1.12	178.26	80.66	2.21	3350	98.70	4.60	1.90
1725	1677	10.27	1.27	164.52	81.43	2.02	7500	123.50	6.00	1.60
1826	1677	9.78	1.17	171.50	78.74	2.18	6500	114.10	6.00	1.71
1925	1677	9.60	1.23	174.68	92.25	1.89	7500	128.00	5.00	1.28
2030	1677	9.03	1.04	185.71	79.83	2.33	4000	90.00	5.00	1.76
2136	1677	9.87	1.18	172.36	79.43	2.17	7500	106.00	6.00	1.50
2235	1677	10.18	1.24	164.70	79.32	2.08	5500	102.40	5.00	1.51
2338	1677	8.58	1.02	204.21	93.51	2.18	1800	94.20	4.00	1.91
2450	1677	11.43	1.70	146.66	104.79	1.40				
2546	1677	8.31	1.15	201.80	123.78	1.63				

The next regime equation used to estimate scour was the Lacey equation (equation 22). The D_{50} of the bed material measured from samples taken in the field was 0.6 mm. For the entire reach the Lacey equation was multiplied by a Z factor of 1.25 for a vertical wall (Table 2). Also, within the channel bend, from station 16+29 to station 22+35, the Lacey equation was multiplied by a Z factor of 0.25 and added to the scour calculated using the Lacey equation adjusted using the vertical wall Z factor. In the straight channel reaches, from station 14+64 to station 15+45 and from station 23+38 to station 25+46, the Lacey equation was not adjusted by a Z factor. It was assumed that the Z factor for the vertical wall includes the Z factor for the straight reach, while for the channel bend the Z factor of 0.25 is the difference between the Z factor for moderate bends and the Z factor for straight reaches (Baird, 1995).

The third regime equation used was the Blench zero-bed equation (equation 23). The channel bed material D_{50} was used to obtain the "zero-bed factor" from Figure 7. To obtain the scour depth, the depth for zero-bed sediment transport, d_{fo}, is adjusted by the Z factor of 0.6 for a moderate bend.

The last equation used to predict scour was the competent velocity control equation (equation 27). The bed material D_{50} and hydraulic depth were used as input to determine the competent mean velocity from Figure 9. The hydraulic depth and velocity from the HEC 2 results were used in the equation.

Once all of the scour depths were calculated from each equation, the most reasonable results were obtained by the Lacey equation and the competent velocity equation. The scour depths calculated by these two equations were then averaged to determine the scour depth for the USBR method. The scour depths ranged from 5.82 feet at station 25+46 to 8.24 feet at station 21+36. See Table 10 and Figure 14 for the results of this method.

Table 10 Results of scour depth calculations for each equation used in the USBR method supercritical flow conditions

STA.	SCOUR (ABBOTT)	SCOUR (NEILL)	SCOUR (LACEY)	SCOUR (BLENCH)	SCOUR (COMP V)	AVG SCOUR
1464	5.23	1.53	6.15	4.22	7.30	6.73
1545	5.24	1.40	6.15	4.24	7.74	6.94
1629	5.37	1.90	7.38	4.54	8.74	8.06
1725	5.36	1.60	7.38	4.52	8.90	8.14
1826	5.39	1.71	7.38	4.59	9.03	8.21
1925	5.26	1.28	7.38	4.28	7.67	7.53
2030	5.38	1.76	7.38	4.56	8.73	8.05
2136	5.38	1.50	7.38	4.57	9.10	8.24
2235	5.38	1.51	7.38	4.57	9.05	8.21
2338	5.25	1.91	6.15	4.26	7.68	6.91
2450	5.15		6.15	4.04	7.02	6.58
2546	5.01		6.15	3.74	5.50	5.82

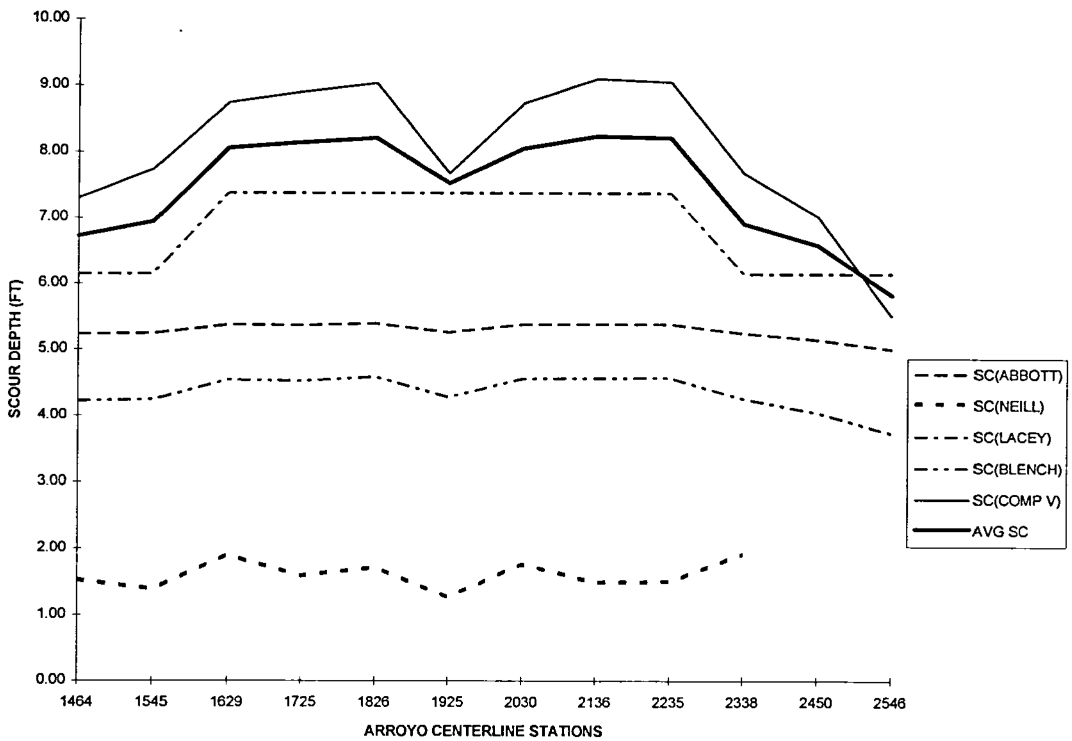


Figure 14 Comparison of scour depths for supercritical flows using USBR equations

G. FEDERAL HIGHWAY ADMINISTRATION

For the example project, the only FHWA scour equation that was employed was the HEC 11 equation for probable maximum scour depth. This is because local scour or contraction scour are not acting on the flood wall. This equation is based solely on the mean diameter of the bed material. The D_{50} of the bed material is 0.6 mm or 0.0019 feet, which is less than 0.005 feet. Therefore, the scour depth is a constant 12 feet along the entire reach of the channel.

H. U. S. ARMY CORPS OF ENGINEERS

Although, the USACE evaluates several methods, including the AMAFCA and the USBR techniques, for this discussion the only method calculated involved using Figure 10. This approach calculates scour at channel bends. Therefore, the scour only occurs along the channel bend between stations 16+29 and 21+36. Inputs to the technique include channel section top width, centerline radius of bend, and mean depth in the upstream channel. Table 11 shows the results of the scour calculations.

Table 11 Results of scour depths calculations using USACE method

STA.	Q (CFS)	VEL (FPS)	FROUDE NO.	AREA (SF)	TOP WID(FT)	HYD DEP(FT)	TW/Rc	SCOUR (FT)
1464	1677	8.42	1.02	202.65	95.22	2.13		
1545	1677	10.11	1.33	168.63	94.17	1.79		
1629	1677	9.41	1.12	178.26	80.66	2.21	0.25	5.74
1725	1677	10.27	1.27	164.52	81.43	2.02	0.25	5.74
1826	1677	9.78	1.17	171.50	78.74	2.18	0.24	5.74
1925	1677	9.60	1.23	174.68	92.25	1.89	0.28	5.74
2030	1677	9.03	1.04	185.71	79.83	2.33	0.25	5.74
2136	1677	9.87	1.18	172.36	79.43	2.17	0.24	5.74
2235	1677	10.18	1.24	164.70	79.32	2.08		
2338	1677	8.58	1.02	204.21	93.51	2.18		
2450	1677	11.43	1.70	146.66	104.79	1.40		
2546	1677	8.31	1.15	201.80	123.78	1.63		

I. COMPARISON OF RESULTS

Once all of the scour calculations for the example project were completed, each method was evaluated and compared (Table 12 and Figure 15). The FHWA approach is the most conservative at a constant 12 feet. The AMAFCA and USBR methods have similar inflections, with the AMAFCA method being more conservative. The depth of scour using the Corps of Engineers technique is extremely close to the average scour of all methods. Whereas, the remaining four methods (Denver, Clark, Maricopa, and Tucson) are the least conservative. The Denver and Clark methods result in a constant 3 feet depth of scour. Maricopa and Tucson techniques scour depth curves are similar with scour depths ranging from 1.41 to 5.00 feet.

Table 12 Comparison of results of scour depth calculations for supercritical flows using all methods

STA.	AMAFCA	MARICOPA	TUCSON	CLARK	DENVER	USBR	FHWA	USACE	AVERAGE
1464	9.73	1.45	2.56	3.00	3.00	6.73	12.00		5.31
1545	9.15	2.09	3.47	3.00	3.00	6.94	12.00		5.48
1629	10.50	2.33	3.64	3.00	3.00	8.06	12.00	5.74	5.87
1725	10.09	2.63	4.16	3.00	3.00	8.14	12.00	5.74	5.93
1826	10.47	2.46	3.76	3.00	3.00	8.21	12.00	5.74	5.92
1925	9.43	2.39	4.05	3.00	3.00	7.53	12.00	5.74	5.73
2030	10.65	2.21	3.51	3.00	3.00	8.05	12.00	5.74	5.86
2136	10.44	2.49	3.95	3.00	3.00	8.24	12.00	5.74	5.95
2235	10.34	2.12	3.42	3.00	3.00	8.21	12.00		5.83
2338	9.95	1.51	2.61	3.00	3.00	6.91	12.00		5.38
2450	8.00	2.68	5.00	3.00	3.00	6.58	12.00		5.57
2546	8.01	1.41	2.54	3.00	3.00	5.82	12.00		4.93

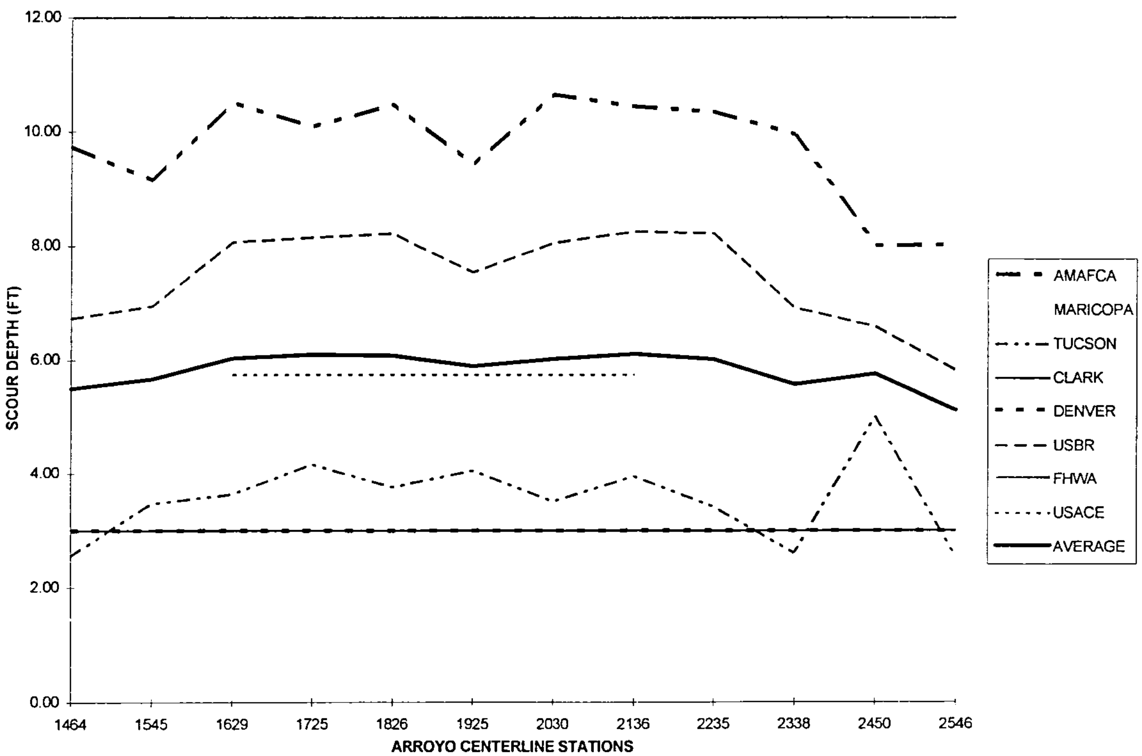


Figure 15 Comparison of scour depths for supercritical flows using all methods

VI. DISCUSSION

A. COMPARISON OF METHODS

In comparing the results of the scour depth calculations shown in Table 12 and Figure 15 for supercritical flows, there is little agreement of the predicted scour depth among the methods. The computed scour depths range from a low of 1.41 feet using the technique prescribed by Maricopa County to a maximum of 12.0 feet following the FHWA approach. Some methods give similar results because they use similar equations. These methods include those of Maricopa County Flood Control and the City of Tucson, and Denver Flood Control and Clark County Flood Control. The AMAFCA method, which is similar in origin to the other local flood control agencies, is the second most conservative approach. The USBR approach is somewhat conservative yet near the average of the eight methods examined, and therefore may be the most reasonable method. Although, the USACE method results in scour depths that are close to the averages of all methods, the USACE states that it should only be used as a first guess at scour depths and should not be relied upon for its accuracy (USACE, 1991a). Finally, the FHWA technique, which relies entirely on the gradation of the bed material, is extremely conservative.

The methods used by Maricopa County and the City of Tucson are alike in that the total scour is made up of the same component scours multiplied by a factor of safety, except that Tucson adds components for general scour and depth of the low-flow thalweg. The other difference is that Maricopa County increases the factor of safety from 1.3 to 1.5 if only one scour component is involved. According to these methods the types of scour acting on the scour wall are bed-form scour, bend scour, and general scour (Tucson method only). Contraction scour and local scour are not included.

The Denver and Clark County methods are similar. Based on the procedures discussed in their guidelines, contraction scour and local scour are not applicable. Thus, the bank protection or scour wall should be buried to a minimum depth of 3 feet below the channel bed.

The AMAFCA method is the only method that tries to predict how the arroyo will meander in the future by computing its maximum lateral erosion. The total scour obtained using the AMAFCA technique is an addition of the antidune scour and local scour due to flow impinging on the scour wall at a maximum angle and local scour due to flow that is parallel to the scour wall. This method is the second most conservative method. Local scour due to the flow impinging on the wall accounts for approximately 90% of the total scour in the AMAFCA method. The maximum impingement angle used in the equation is derived from the minimum radius of curvature for a meandering thalweg within an arroyo. Since the location of the impingement of flows on the wall cannot be predicted due to the uncertainty of the development of the meander pattern of the thalweg, the local scour is assumed to act on the entire length of the flood wall.

Adding component scour depths together, using the same hydraulic parameters, is conservative because the channel bed can only scour so much before it reaches equilibrium and discontinues the scouring action. Since the same hydraulic parameters are used for each component scour equation, the channel bed is assumed to be stable and not scouring. As the bed begins to scour the flow depth and cross-sectional area increases and the velocity decreases, thereby changing the hydraulic parameters for other component scour depth calculations. Therefore, the more component scour equations required, the more conservative the total scour depth.

The USBR approach is entirely different than the other seven methods discussed in this study. There are no component scour depths to add and then multiply by a factor of safety to obtain a total scour. This approach involves using a maximum of six separate equations to calculate total scour and then comparing the results of the equations to make a determination on which equation or equations are valid using engineering judgment. The equations are based on field measurements, regime flow theory, or the incipient motion or velocity of the bed material.

The results indicate that the Neill equation does not apply to the conditions of the example project. The USBR states that the Abbott equation should only be used as a check on the other equations. The results of the three remaining equations suggest that the Blench equation should not be considered because the scour depths are 3 to 4 feet less than the results from the Lacey and competent velocity equations. Therefore, the scour depths obtained by using the Lacey and Competent Velocity equations were averaged and given as the overall results of the USBR method. The Lacey equation is the only equation in the USBR approach that addresses scour along vertical walls or scour walls through the adjustment of the equation by the Z factor.

The FHWA method is the most conservative by a factor of at least 1.25. It may not be valid because it only considers the bed material size. The flow velocity and flow depth have no affect on the scour depth.

Although, the results of the USACE method are near the average of all methods, the USACE warns that it is only an estimate of the gross scour depth and should be used as a comparison of other more accurate methods. Actually, the Albuquerque District uses the AMAFCA and USBR approaches and compares the results to obtain a final design scour depth.

B. PARAMETERS THAT AFFECT SCOUR

Scour is predominantly a function of the channel hydraulics (flow depth and flow velocity); and bed material characteristics (material gradation). The flow depth is proportional to the hydrostatic pressure that acts on the channel bed. An increase in flow depth increases the hydrostatic pressure, which in turn increases the scour of the channel bed. Scour results from an increase in boundary shear stress produced by an increase in flow velocity. The incipient velocity of the bed material depends on the gradation of the material. Smaller bed material results in greater scour depths. Local scour from flow impinging on a flood wall occurs where the flow is accelerated due to the obstruction in the flow. The principal erosion mechanism is the creation of vortices or eddies by the obstruction and resultant acceleration of flow.

A portion of the parameters described above are included in most of the scour prediction methods evaluated in this study. The only method that considers all three variables is the USBR competent velocity method. The methods used by the five local flood control authorities (AMAFCA, Clark County, Denver, Maricopa County, and Tucson), which are mostly based on equations developed by Colorado State University, are dependent on the flow depth and velocity of flow and do not consider the bed material characteristics. The FHWA equation is only a function of the bed material size. The USACE equation considers flow depth and channel geometry.

The five USBR equations used in this study are dependent on different parameters. The Abbott equation is based on the discharge and channel geometry. The Neill equation considers discharge, channel geometry, and bed material particle size. The Lacey approach only involves the discharge and mean bed material size. Parameters that affect the Blench method include discharge, channel geometry, and mean bed material size. Finally, the competent velocity method is a function of flow depth, flow velocity, and bed material size.

For the example project, varying the flow velocity has no affect on the scour depths calculated using the Clark County, Denver, FHWA, USBR Abbott, USBR Neill, USBR Lacey, and USBR Blench methods. Therefore, whether the flow is supercritical or subcritical has no bearing on the calculation to predict scour depth. Also, the depth of flow does not influence the scour depth. Scour depth should be directly related to both of these parameters. For a constant flow rate, the velocity of flow, which is inversely proportional to the flow depth, is directly proportional to the scour depth. But the flow depth is also directly proportional to the scour depth. How the flow velocity and flow depth affect the scour depth is an important consideration. Liu et al. (1961) showed that the scour depth is proportional to the flow depth, whereas the scour depth is proportional to the cube root of the Froude number or flow velocity.

The scour depth is not influenced by the bed material characteristics when using the AMAFCA, Maricopa County, Tucson, USACE, and USBR competent velocity methods. Although all of the methods evaluated are for sand-bed channels, the gradation of the bed material is not considered. The mean diameter of the bed material (D_{50}) is inversely proportional to the scour depth. The larger the particle size, the higher the incipient velocity required to move the bed material, which results in a smaller scour hole.

C. SUBCRITICAL VERSUS SUPERCRITICAL FLOWS

As discussed previously in Section II., recent studies have shown that flows in steep, natural channels may be subcritical, not supercritical (Trieste, 1994). This is primarily due to the resistance of flow increasing to the amount required to produce subcritical flows. The Manning's roughness factor (n) for the example project was adjusted using the equation given in the AMAFCA "Sediment and Erosion Design Guide" (Mussetter et al., 1994):

$$n = (n_b + n_1 + n_2 + n_3 + n_4)m \tag{30}$$

where: n_b = The base value for a straight, uniform channel
n_1 = Value for surface irregularities in the cross section
n_2 = Value for variations in shape and size of the channel
n_3 = Value for obstructions
n_4 = Value for vegetation and flow conditions
m = Correction factor for sinuosity of the channel

Using these relationships for the example project, the Manning's n equals 0.051 instead of the 0.035 used in the HEC 2 model that produced supercritical flows. Running the HEC 2 model with a Manning's n of 0.051 resulted in the channel flowing subcritically (Table 13).

Table 13 Results of HEC 2 run with subcritical flow conditions

SEC. NO.	STATION	VELOCITY (FT)	AREA (FT^2)	TOPWID (FT)	DEPTH (FT)	FROUDE
3	10+00	6.04	293.72	180.07	3.09	0.79
4	12+76	8.33	201.35	93.66	2.93	1.00
5	13+69	4.14	407.83	240.14	3.03	0.55
6	14+64	6.44	260.42	120.65	3.32	0.77
7	15+45	7.12	235.67	118.32	3.25	0.89
8	16+29	6.65	252.24	111.05	3.39	0.78
9	17+25	7.45	225.07	103.11	3.29	0.89
10	18+26	6.61	253.53	101.12	3.60	0.74
11	19+25	7.87	213.00	99.39	2.99	0.95
12	20+30	6.41	261.61	103.00	3.47	0.71
13	21+36	7.51	223.70	106.33	3.26	0.90
14	22+35	6.07	276.40	113.82	3.63	0.69
15	23+38	6.71	252.68	102.77	3.78	0.75
16	24+50	7.66	221.86	127.12	3.28	0.97
17	25+46	7.15	234.64	124.87	2.50	0.92

The hydraulic parameters, shown in Table 13, from the subcritical HEC 2 run were then used to calculate scour depths following each of the eight methods discussed in this study. In studying the results of using the USBR equations to calculate the scour depth for subcritical flow conditions, the only equation not affected is the Lacey equation. The results from the Lacey equation do not change because the flow depth and flow velocity are not considered. The most dramatic difference is in the competent velocity equation. The scour depths decrease on the order of 2 to 3 feet. Although the overall scour depth for the USBR method is still obtained by averaging the results from the Lacey and competent velocity equations, the scour depth decreases from 1 to 1.5 feet (Table 14 and Figure 16).

Table 14 Results of scour depth calculations for each equation used in USBR method for subcritical flow conditions

STA.	SCOUR (ABBOTT)	SCOUR (NEILL)	SCOUR (LACEY)	SCOUR (BLENCH)	SCOUR (COMP V)	AVG SCOUR
1464	5.03	1.66	6.15	3.78	5.16	5.65
1545	5.05	1.13	6.15	3.82	5.47	5.81
1629	5.10	1.18	7.38	3.94	5.68	6.53
1725	5.16	1.06	7.38	4.07	6.38	6.88
1826	5.18	0.75	7.38	4.11	6.22	6.80
1925	5.19	0.95	7.38	4.14	6.73	7.06
2030	5.16	0.91	7.38	4.08	6.03	6.70
2136	5.14	0.76	7.38	4.02	6.21	6.79
2235	5.08	1.05	7.38	3.89	5.33	6.35
2338	5.17	1.46	6.15	4.08	6.22	6.19
2450	4.98		6.15	3.69	5.29	5.72
2546	5.00		6.15	3.72	5.19	5.67

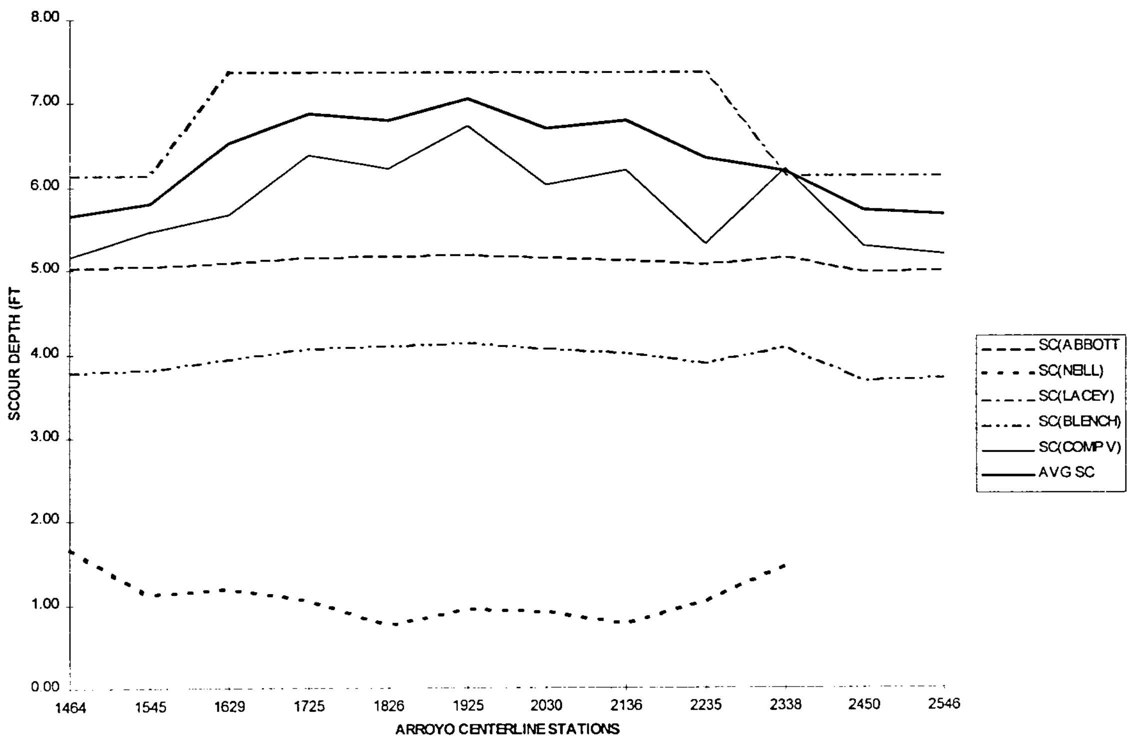


Figure 16 Comparison of scour depths for subcritical flow conditions using USBR equations

Scour depths for all eight methods were calculated using the channel hydraulics for the subcritical HEC 2 model (Table 15 and Figure 17). The Clark County, Denver, and FHWA scour depths are exactly the same as the scour depths for the supercritical flows. The results for subcritical flow conditions using the AMAFCA approach are virtually the same as the results for supercritical flow conditions with scour depths ranging from 8.33 feet to 10.38 feet. Therefore, the increase in depth cancels out the decrease in velocity. Both the Maricopa County and the City of Tucson scour depths decrease slightly, so the decrease in velocity has a greater affect than the increase in depth. The scour depths computed using the USBR method decrease due to the influence of the velocity on the competent velocity equation. Finally, the USACE results increase by about 1.6 feet due to the increase in depth.

The two most conservative approaches (FHWA and AMAFCA) essentially give the same results for subcritical flows as for supercritical flows. Only one technique, the USACE method, increases the scour depth. Since the overall average decreases, the FHWA and AMAFCA methods are even more conservative when comparing them to the average. Once again, the USBR method follows the average, resulting in slightly higher scour depths.

D. COMPARISON OF FEDERAL AGENCIES

Focusing on the results computed for both supercritical and subcritical flows by using the methods described by the three federal agencies (USACE, FHWA, and USBR), the only approach that seems appropriate for estimating scour depths along flood walls or bank revetments is the USBR method. The USBR method is the only method that takes into account the channel hydraulics and bed material characteristics. The USBR method results are closest to, yet higher than, the average scour depths for all eight methods for both supercritical and subcritical flows.

E. COMPARISON OF LOCAL AGENCIES

In comparing the results of the five flood control authorities for both supercritical flows and subcritical flows, the AMAFCA method is more conservative than the other four techniques by a factor of 2.1 to 13.0 (Table 16 and Figure 18 on page 50, and Table 17 and Figure 19 on page 51). The difference in scour depths is mainly due to a local scour component that is included only in the AMAFCA approach. The local scour is a result of the possibility of the flow impinging on the flood wall because the arroyo will tend toward a meander planform over time. Since the possible meander is somewhat random, the location of the local scour is unknown. Therefore, the local scour is assumed to act on the entire length of the flood wall. Local scour accounts for approximately 90% of the total scour in the AMAFCA method. Without the local scour component, the total scour would be similar to the total scour calculated using the approaches of the other four flood control authorities.

Table 15 Comparison of results of scour depth calculations for subcritical flows using all methods

STA.	AMAFCA	MARICOPA	TUCSON	CLARK	DENVER	USBR	FHWA	USACE	AVERAGE
1464	9.21	0.85	2.04	3.00	3.00	5.65	12.00		4.92
1545	8.96	1.04	2.20	3.00	3.00	5.81	12.00		4.96
1629	9.66	1.43	2.66	3.00	3.00	6.53	12.00	7.36	5.54
1725	9.72	1.63	2.86	3.00	3.00	6.88	12.00	7.36	5.64
1826	10.38	1.42	2.65	3.00	3.00	6.80	12.00	7.36	5.66
1925	9.76	1.74	2.97	3.00	3.00	7.06	12.00	7.36	5.70
2030	10.38	1.37	2.60	3.00	3.00	6.70	12.00	7.36	5.64
2136	9.48	1.65	2.87	3.00	3.00	6.79	12.00	7.36	5.61
2235	9.87	0.75	1.96	3.00	3.00	6.35	12.00		5.09
2338	10.28	0.92	2.10	3.00	3.00	6.19	12.00		5.17
2450	8.33	1.20	2.35	3.00	3.00	5.72	12.00		4.90
2546	8.60	1.05	2.21	3.00	3.00	5.67	12.00		4.89

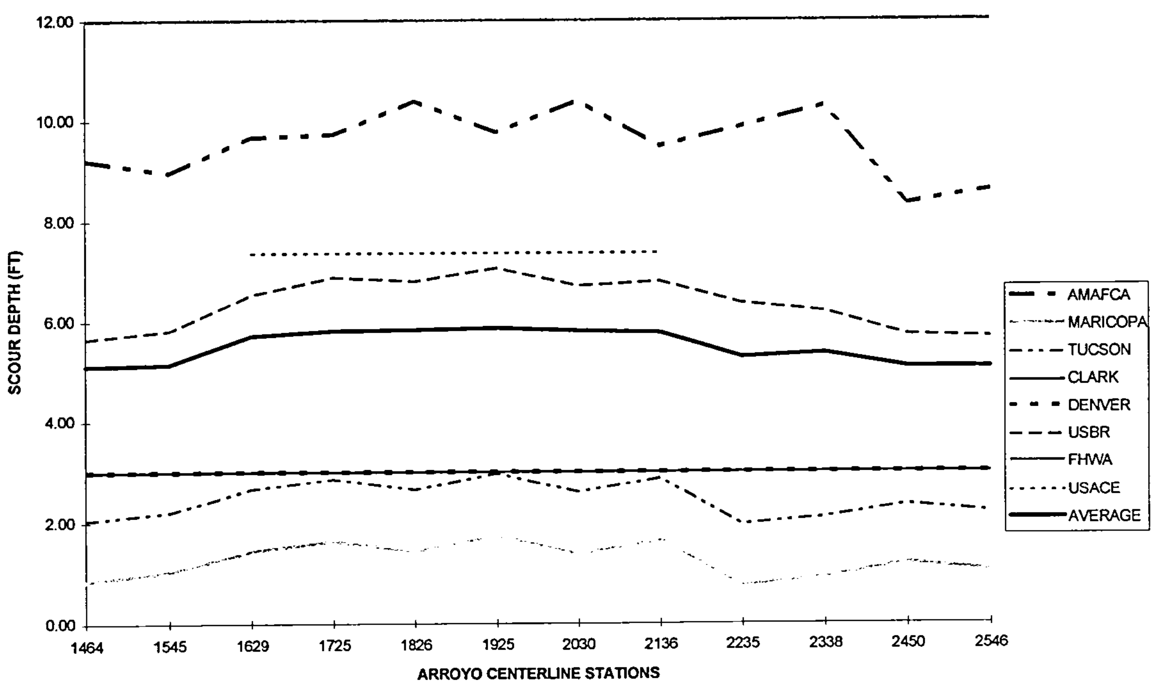


Figure 17 Comparison of scour depths for subcritical flows using all methods

Table 16 Comparison of results of scour depth calculations for supercritical flows using the flood control authorities methods

STA.	AMAFCA	MARICOPA	TUCSON	CLARK	DENVER	AVERAGE
1464	9.73	1.45	2.56	3.00	3.00	3.69
1545	9.15	2.09	3.47	3.00	3.00	3.88
1629	10.50	2.33	3.64	3.00	3.00	4.23
1725	10.09	2.63	4.16	3.00	3.00	4.32
1826	10.47	2.46	3.76	3.00	3.00	4.28
1925	9.43	2.39	4.05	3.00	3.00	4.11
2030	10.65	2.21	3.51	3.00	3.00	4.21
2136	10.44	2.49	3.95	3.00	3.00	4.32
2235	10.34	2.12	3.42	3.00	3.00	4.12
2338	9.95	1.51	2.61	3.00	3.00	3.75
2450	8.00	2.68	5.00	3.00	3.00	4.08
2546	8.01	1.41	2.54	3.00	3.00	3.33

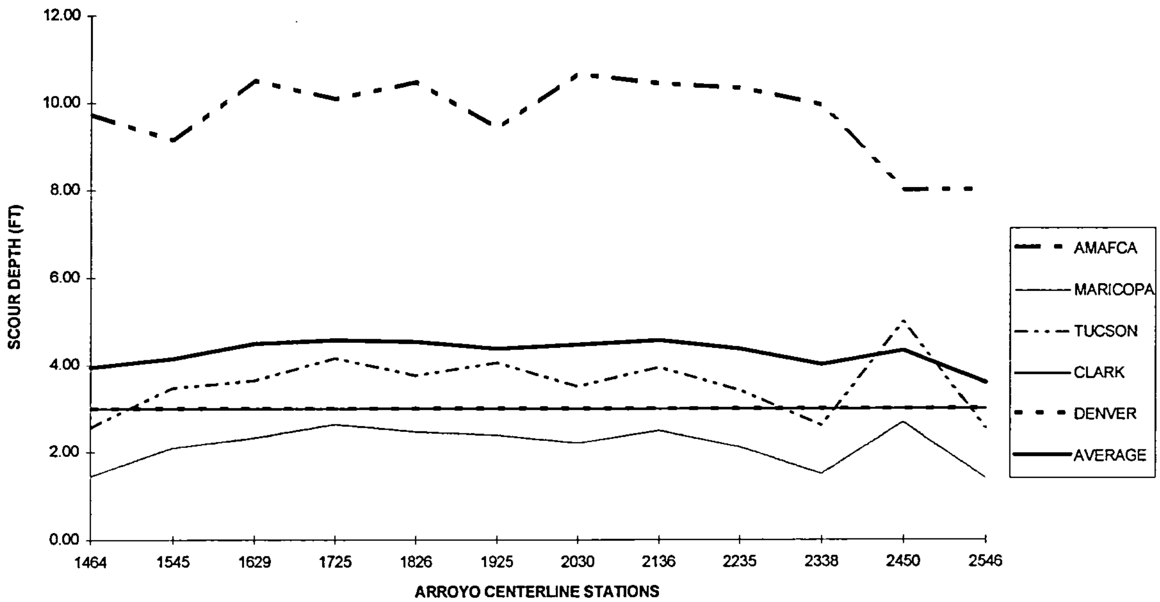


Figure 18 Comparison of results of scour depth calculations for supercritical flows using flood control authorities methods

Table 17 Comparison of results of scour depth calculations for subcritical flows using flood control authorities methods

STA.	AMAFCA	MARICOPA	TUCSON	CLARK	DENVER	AVERAGE
1464	9.21	0.85	2.04	3.00	3.00	3.36
1545	8.96	1.04	2.20	3.00	3.00	3.38
1629	9.66	1.43	2.66	3.00	3.00	3.69
1725	9.72	1.63	2.86	3.00	3.00	3.78
1826	10.38	1.42	2.65	3.00	3.00	3.83
1925	9.76	1.74	2.97	3.00	3.00	3.83
2030	10.38	1.37	2.60	3.00	3.00	3.81
2136	9.48	1.65	2.87	3.00	3.00	3.74
2235	9.87	0.75	1.96	3.00	3.00	3.46
2338	10.28	0.92	2.10	3.00	3.00	3.60
2450	8.33	1.20	2.35	3.00	3.00	3.32
2546	8.60	1.05	2.21	3.00	3.00	3.31

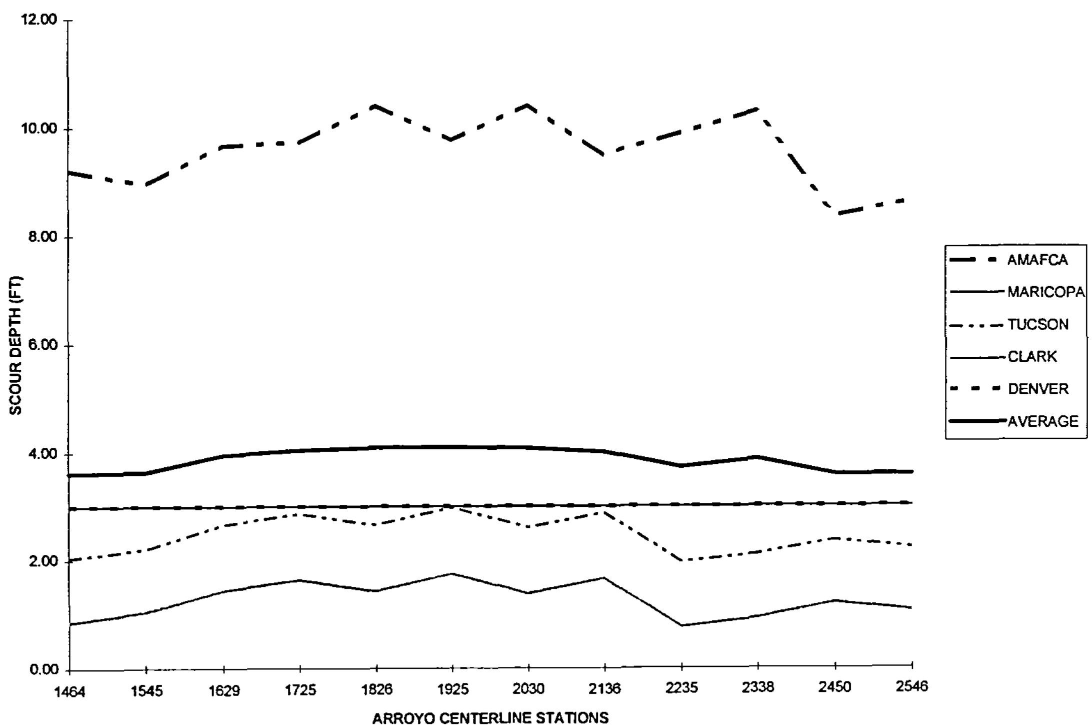


Figure 19 Comparison of results of scour depth calculations for subcritical flows using flood control agencies methods

F. NEW METHOD TO COMPUTE SCOUR DEPTH ALONG FLOOD WALLS

This section presents a new method to estimate scour depth along flood walls that is a modification of the methods used by AMAFCA, Maricopa County, and the City of Tucson. The AMAFCA method, which gives reasonable, yet conservative results, is one of only two methods (USBR Lacey equation) that specifically addresses scour along a flood wall. This new approach is based on how flows are conveyed in each of the three channel classifications: straight, meandering, and braided.

As discussed previously in Section IIIA., according to AMAFCA, the local scour acting on a flood wall is due to the flow impinging on the wall at a certain angle (Figure 2). The impingement angle is determined by estimating the "unconstrained valley width" of an arroyo and then dividing it into the channel width that is formed by the dominant discharge. The unconstrained valley width is equal to 2 times the maximum lateral erosion plus the channel width. The maximum lateral erosion for a meander bend occurs when the centerline radius of curvature for the channel to channel width ratio (R_c/W_D) is between 2 and 4. The channel width to unconstrained valley width ratio (W_D/UVW) is equal to the cosine of the impingement angle. The maximum impingement angle is 71^0 which corresponds to an unconstrained valley width to channel width ratio of 3.5. The new method uses this approach to determine the impingement angle.

Arroyos are generally classified as straight, meandering, or braided channels (see Section II). Straight channels usually have a sinuous low-flow channel within the banks that is formed by the dominant discharge. In straight channels the dominant discharge flows in the sinuous low-flow channel, whereas the flood flows are conveyed within the relatively straight channel banks. In a meandering arroyo, the entire channel follows a sinuous path that is formed by the dominant discharge. Therefore, both the dominant discharge and flood flows are conveyed by the meandering arroyo. Braided streams are wide channels with shallow banks and multiple channels within the stream bed. The low flows are contained within the multiple channels in the stream bed, while the higher discharges flow from bank to bank.

The following discussion relates the scour along a flood wall to the three channel classifications. First, consider a flood wall constructed along the banks of a straight channel. In low flow conditions, where the dominant discharge forms the sinuous thalweg, the flood wall may experience local scour due to the flow impinging on the flood wall. In flood flow conditions, the scour along the flood wall is caused by the flow being parallel to the wall. For a meandering channel with a flood wall located along the outside bank of a curve, the two scouring mechanisms acting on the wall in all flow conditions are bend scour around the outside of the curve and local scour from the flow being parallel to the wall. Finally, for braided streams, a flood wall located along the bank may experience local scour from low flows impinging on the wall from the multiple channels in the channel bed and local scour from flood flows flowing parallel to the wall.

Based on the discussion above, the AMAFCA, Tucson, and Maricopa scour equations can be modified to the following forms. In the first equation (31), which is for straight and braided channels, the antidune scour and local scour from parallel flow are caused by the flow from a 100-year storm, whereas the local scour from impinging flow uses the depth of flow and Froude number from the dominant discharge. Equations 32 and 33 for meandering arroyos includes antidune scour and bend scour.

STRAIGHT CHANNELS AND BRAIDED STREAMS

$$Y_s = \left(0.73 + 0.14\pi F_{r100}^2\right)Y_{100} + 4F_{rD}^{0.33}Y_D \sin\theta + 1 \tag{31}$$

where: Y_s = Total scour depth (ft)
- F_{r100} = Froude number due to a 100-year discharge
- F_{rD} = Froude number due to a dominant discharge
- Y_{100} = Flow depth due to a 100-year discharge
- Y_D = Flow depth due to a dominant discharge
- θ = Angle of impingement

MEANDERING CHANNELS

$$Y_s = Y_p + Y_a + Y_{bs} + 1 \tag{32}$$

$$Y_s = \left(0.73 + 0.14\pi F_{r100}^2\right)Y_{100} + \frac{0.0685 Y_{\max} V_m^{0.8}}{Y_h^{0.4} S_e^{0.3}}\left[2.1\left(\frac{\sin^2(\alpha/2)}{\cos\alpha}\right)^{0.2} - 1\right] + 1 \tag{33}$$

where: Y_s = Total scour depth (ft)
- Y_p = Parallel scour depth (ft)
- Y_a = Antidune scour depth (ft)
- F_{r100} = Froude number due to a 100-year discharge
- Y_{100} = Flow depth due to a 100-year discharge
- Y_{bs} = Bend-scour component of total scour depth (ft)
  - = 0 when $r_c/T \geq 10.0$, or $\alpha \leq 17.8°$
  - = computed value when $0.5 < r_c/T < 10.0$, or $17.8° < \alpha < 60°$
  - = computed value at $\alpha = 60°$ when $r_c/T \leq 0.5$, or $\alpha \geq 60°$
- V_m = Average velocity of flow immediately upstream of bend (ft/sec)
- Y_{max} = Maximum depth of flow from a 100-year storm immediately upstream of bend (ft)
- Y_h = Hydraulic depth of flow immediately upstream of bend (ft)
- S_e = Energy slope immediately upstream of bend or bed slope for uniform flow conditions (ft/ft)
- α = 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 (°) (see Figure 5)

These equations are based on the idea that in straight and braided channels, the local scour on the flood wall is caused by: 1) impingement of the dominant discharge at a certain angle to the wall and 2) parallel flows from the 100-year discharge. Also, the antidune scour is caused by the passage of 100-year flows. For meandering channels, where the low flows and flood flows are carried in the same cross section, the scour on the flood wall is due to antidune scour and bend scour from 100-year flows. Since the classification (straight, meandering, or braided) of a channel reach may change due to the change in sediment supply or the random meander process, the scour depth should be calculated using both equations and the most conservative depths used for the design of the flood wall.

In the Albuquerque area, the dominant discharge (335 CFS), which is approximately equal to a 5- to 10-year peak discharge (Mussetter et al., 1994), is estimated by the following equation:

$$Q_D = 0.2Q_{100} \tag{34}$$

where: Q_D = The dominant discharge (CFS)
Q_{100} = The 100-year peak discharge (CFS).

Scour depths for the example project using equations 31, 32, and 33 were computed for supercritical and subcritical flows (Table 18 and Figure 20 and Table 19 and Figure 21). Since the Mirehaven Arroyo is considered to be a straight channel with a sinuosity of 1.14, equation 31 controls the scour depth. The dominant discharge was calculated using equation 34 and then input to the supercritical and subcritical HEC 2 models to obtain the hydraulic parameters for each condition. A factor of safety of 1 foot was added to the total scour depth calculated.

For the supercritical case, the modified equations resulted in the scour depths ranging from 5.81 feet to 7.66 feet (Table 18 and Figure 20). Similar to the supercritical case, the modified equations for the subcritical case resulted in scour depths ranging from 5.87 feet to 7.40 feet (Table 19 and Figure 21). The first modified equation (31) gave the most conservative scour depths for both cases and therefore it was selected. The local scour due to impingement of low flows accounts for about 50% of the total scour depth for both supercritical and subcritical flow conditions.

The scour depths calculated using the Modified method were compared to the other flood control agencies (Table 20 and Figure 22 and Table 21 and Figure 23). While the modified approach decreases the scour depths compared to the AMAFCA method, it is still more conservative than the methods of the other four flood control agencies for both flow conditions.

Table 18 Results of scour depth calculations for supercritical flow conditions using the Modified method

COL 1	COL 2	COL3	COL 4	COL 5	COL 6
STA.	PAR SC(FT)	IMP SC(FT)	EQ31 SC(FT)	BEND SC(FT)	EQ33 SC(FT)
1464	2.52	3.27	6.80	0.00	3.52
1545	2.70	3.46	7.17	0.00	3.70
1629	2.82	3.72	7.55	0.58	4.41
1725	2.92	3.67	7.59	0.58	4.50
1826	2.90	3.73	7.63	0.58	4.48
1925	2.64	3.48	7.12	0.58	4.22
2030	2.81	3.76	7.58	0.58	4.40
2136	2.91	3.74	7.66	0.58	4.50
2235	2.93	3.71	7.64	0.00	4.51
2338	2.60	4.04	7.64	0.00	4.18
2450	2.81	3.57	7.37	0.00	4.39
2546	2.13	2.68	5.81	0.00	3.13

Column 1 - Channel Centerline Station
Column 2 - Scour due to parallel flow and antidune scour (ft)
Column 3 - Scour due to impingement of flow on wall (ft)
Column 4 - Total scour for straight or braided channels (ft)
Column 5 - Scour due to flow around a channel bend (ft)
Column 6 - Total scour for meandering channels (ft)

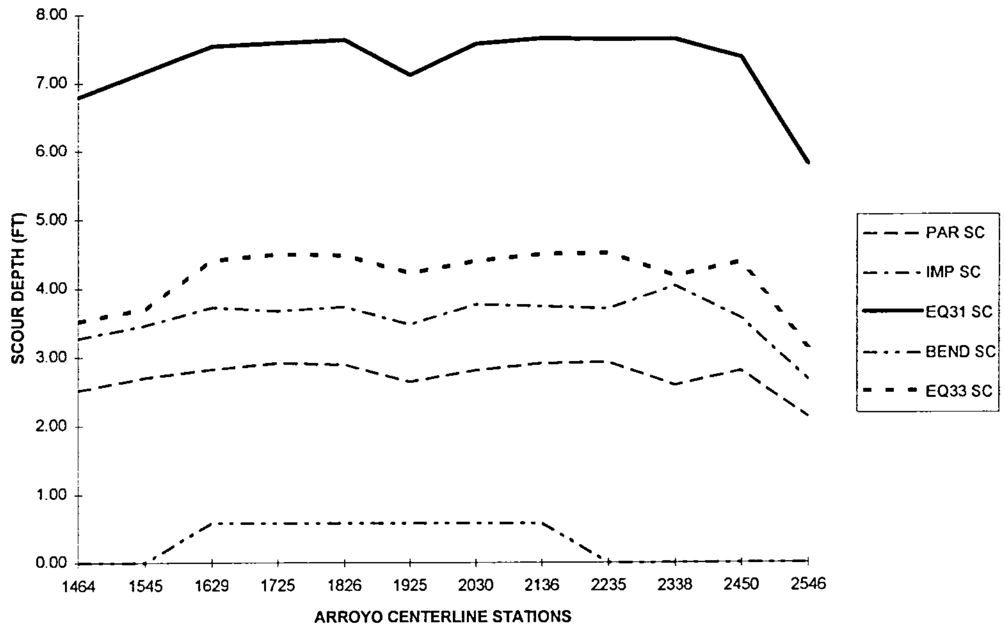


Figure 20 Results of scour depth calculations for supercritical flow conditions using the Modified method

Table 19 Results of scour depth calculations for subcritical flow conditions using the Modified method

COL 1	COL 2	COL 3	COL 4	COL 5	COL 6
STA	PAR SC(FT)	IMP SC(FT)	EQ31 SC(FT)	BEND SC(FT)	EQ33 SC(FT)
1464	2.14	3.47	6.61	0.00	3.14
1545	2.15	3.41	6.55	0.00	3.15
1629	2.26	3.75	7.01	0.44	3.70
1725	2.35	3.54	6.89	0.44	3.79
1826	2.43	3.97	7.40	0.44	3.86
1925	2.41	3.55	6.96	0.44	3.85
2030	2.42	3.82	7.23	0.44	3.85
2136	2.31	3.52	6.82	0.44	3.74
2235	2.28	3.83	7.11	0.00	3.28
2338	2.41	3.65	7.06	0.00	3.41
2450	2.08	3.76	6.83	0.00	3.08
2546	2.07	2.80	5.87	0.00	3.07

Column 1 - Channel Centerline Station
Column 2 - Scour due to parallel flow and antidune scour (ft)
Column 3 - Scour due to impingement of flow on wall (ft)
Column 4 - Total scour for straight or braided channels (ft)
Column 5 - Scour due to flow around a channel bend (ft)
Column 6 - Total scour for meandering channels (ft)

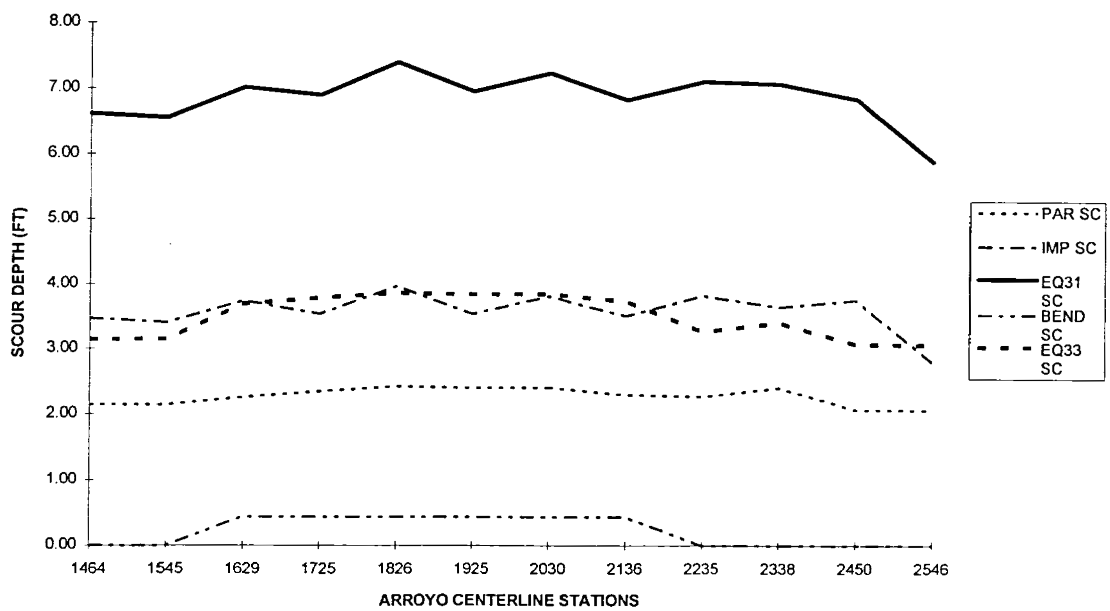


Figure 21 Results of scour depth calculations for subcritical flow conditions using the Modified method

Table 20 Comparison of results of scour depth calculations for supercritical flows using flood control agencies methods and modified method

STA.	MODIFIED	AMAFCA	MARICOPA	TUCSON	CLARK	DENVER	AVERAGE
1464	6.80	9.73	1.45	2.56	3.00	3.00	4.42
1545	7.17	9.15	2.09	3.47	3.00	3.00	4.65
1629	7.55	10.50	2.33	3.64	3.00	3.00	5.00
1725	7.59	10.09	2.63	4.16	3.00	3.00	5.08
1826	7.63	10.47	2.46	3.76	3.00	3.00	5.05
1925	7.12	9.43	2.39	4.05	3.00	3.00	4.83
2030	7.58	10.65	2.21	3.51	3.00	3.00	4.99
2136	7.66	10.44	2.49	3.95	3.00	3.00	5.09
2235	7.64	10.34	2.12	3.42	3.00	3.00	4.92
2338	7.64	9.95	1.51	2.61	3.00	3.00	4.62
2450	7.37	8.00	2.68	5.00	3.00	3.00	4.84
2546	5.81	8.01	1.41	2.54	3.00	3.00	3.96

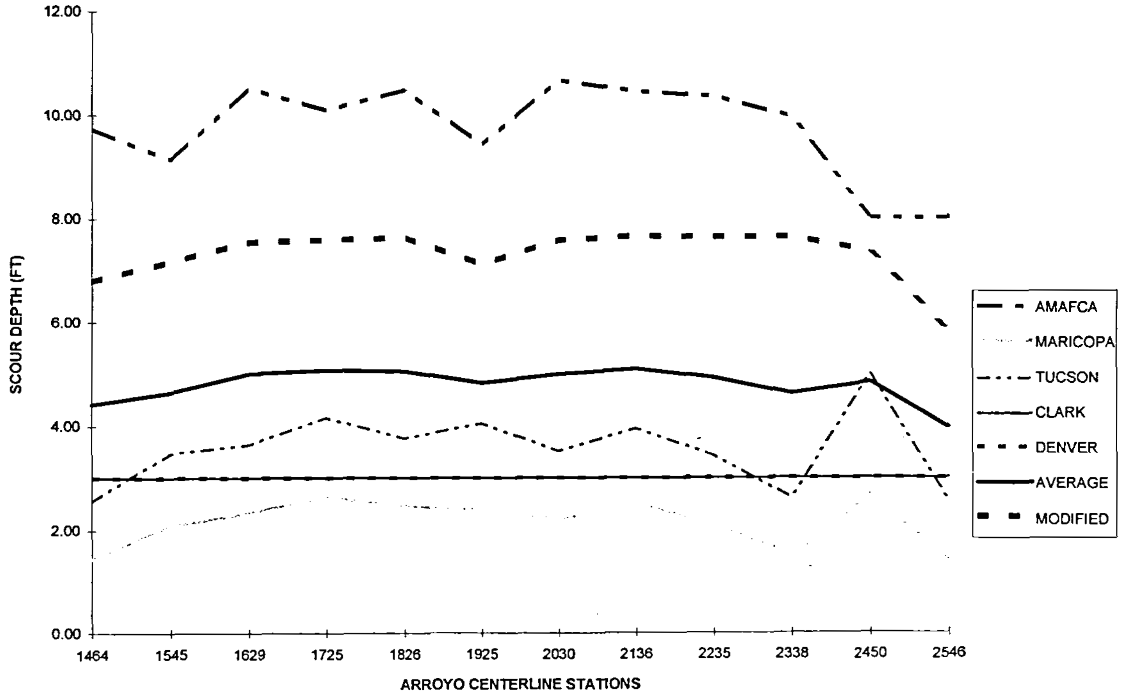


Figure 22 Comparison of results of scour depth calculations for supercritical flows using flood control authorities methods and modified method

Table 21 Comparison of results of scour depth calculations for subcritical flows using flood control agencies methods and modified method

STA.	MODIFIED	AMAFCA	MARICOPA	TUCSON	CLARK	DENVER	AVERAGE
1464	6.61	9.21	0.85	2.04	3.00	3.00	4.12
1545	6.55	8.96	1.04	2.20	3.00	3.00	4.13
1629	7.01	9.66	1.43	2.66	3.00	3.00	4.46
1725	6.89	9.72	1.63	2.86	3.00	3.00	4.52
1826	7.40	10.38	1.42	2.65	3.00	3.00	4.64
1925	6.96	9.76	1.74	2.97	3.00	3.00	4.57
2030	7.23	10.38	1.37	2.60	3.00	3.00	4.60
2136	6.82	9.48	1.65	2.87	3.00	3.00	4.47
2235	7.11	9.87	0.75	1.96	3.00	3.00	4.28
2338	7.06	10.28	0.92	2.10	3.00	3.00	4.39
2450	6.83	8.33	1.20	2.35	3.00	3.00	4.12
2546	5.87	8.60	1.05	2.21	3.00	3.00	3.96

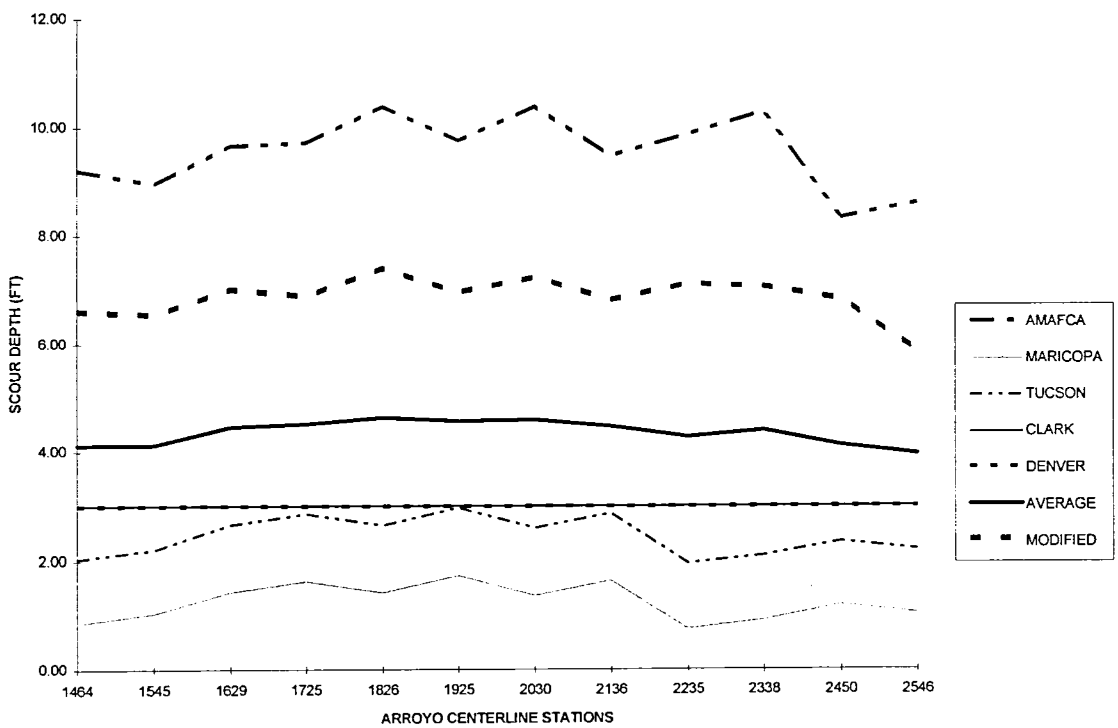


Figure 23 Comparison of results of scour depth calculations for subcritical flows using flood control agencies methods and modified method

F. COMPARISON OF METHODS WITH MODIFIED APPROACH

Now that a modified method has been presented, the final analysis to be performed is to compare it to all of the methods investigated in this study. Table 22 and Figure 24 illustrates the comparison of all methods for supercritical flow conditions. For supercritical flows, the modified method results are virtually the same as the USBR method results. Although these two approaches are not the most conservative, they are close to, yet above the average of all methods.

Table 23 and Figure 25 shows the results for the subcritical flow case. As with the supercritical case, the modified method results are virtually the same as the USBR method results. Although these two approaches and the USACE method are not the most conservative, they are close to, yet above the average of all methods. Since the USACE method is recommended only for a gross estimate of scour depth, it should not be the only method used to determine maximum scour depth.

Table 22 Comparison of results of scour depth calculations for supercritical flows using all methods including the modified method

STA.	MODIFIED	AMAFCA	MCOPA	TUCSON	CLARK	DENVER	USBR	FHWA	USACE	AVG
1464	6.80	9.73	1.45	2.56	3.00	3.00	6.73	12.00		5.66
1545	7.17	9.15	2.09	3.47	3.00	3.00	6.94	12.00		5.81
1629	7.55	10.50	2.33	3.64	3.00	3.00	8.06	12.00	5.74	6.16
1725	7.59	10.09	2.63	4.16	3.00	3.00	8.14	12.00	5.74	6.26
1826	7.63	10.47	2.46	3.76	3.00	3.00	8.21	12.00	5.74	6.25
1925	7.12	9.43	2.39	4.05	3.00	3.00	7.53	12.00	5.74	6.09
2030	7.58	10.65	2.21	3.51	3.00	3.00	8.05	12.00	5.74	6.14
2136	7.66	10.44	2.49	3.95	3.00	3.00	8.24	12.00	5.74	6.27
2235	7.64	10.34	2.12	3.42	3.00	3.00	8.21	12.00		6.22
2338	7.64	9.95	1.51	2.61	3.00	3.00	6.91	12.00		5.83
2450	7.37	8.00	2.68	5.00	3.00	3.00	6.58	12.00		5.99
2546	5.81	8.01	1.41	2.54	3.00	3.00	5.82	12.00		5.20

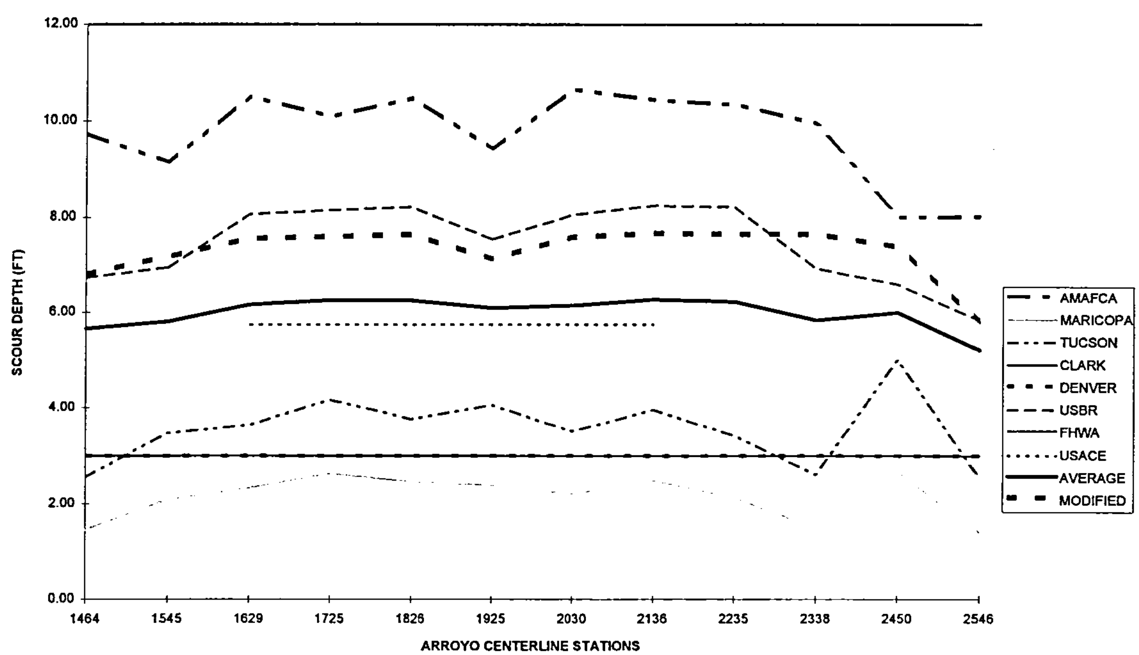


Figure 24 Comparison of results of scour depth calculations for supercritical flows using all methods including the modified method

Table 23 Comparison of results of scour depth calculations for subcritical flows using all methods including the modified method

STA.	MODIFIED	AMAFCA	MCOPA	TUCSON	CLARK	DENVER	USBR	FHWA	USACE	AVG
1464	6.61	9.21	0.85	2.04	3.00	3.00	5.65	12.00		5.30
1545	6.55	8.96	1.04	2.20	3.00	3.00	5.81	12.00		5.32
1629	7.01	9.66	1.43	2.66	3.00	3.00	6.53	12.00	7.36	5.85
1725	6.89	9.72	1.63	2.86	3.00	3.00	6.88	12.00	7.36	5.93
1826	7.40	10.38	1.42	2.65	3.00	3.00	6.80	12.00	7.36	6.00
1925	6.96	9.76	1.74	2.97	3.00	3.00	7.06	12.00	7.36	5.98
2030	7.23	10.38	1.37	2.60	3.00	3.00	6.70	12.00	7.36	5.96
2136	6.82	9.48	1.65	2.87	3.00	3.00	6.79	12.00	7.36	5.89
2235	7.11	9.87	0.75	1.96	3.00	3.00	6.35	12.00		5.51
2338	7.06	10.28	0.92	2.10	3.00	3.00	6.19	12.00		5.57
2450	6.83	8.33	1.20	2.35	3.00	3.00	5.72	12.00		5.30
2546	5.87	8.60	1.05	2.21	3.00	3.00	5.67	12.00		5.18

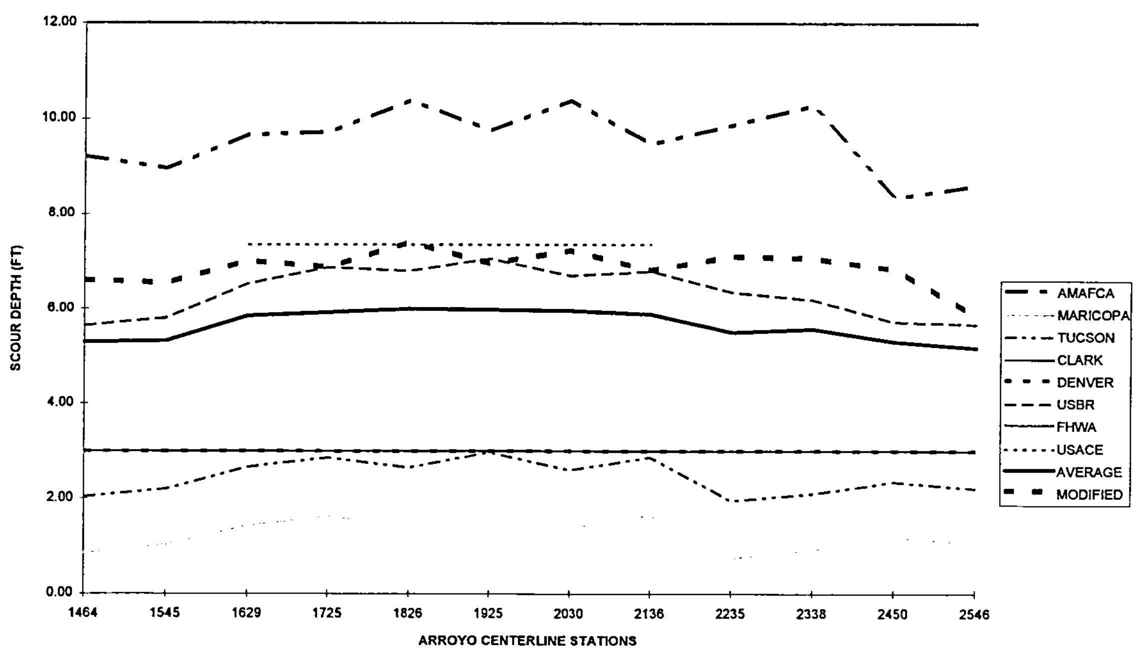


Figure 25 Comparison of results of scour depth calculations for subcritical flows using all methods including the modified method

G. *SENSITIVITY ANALYSIS OF MODIFIED METHOD*

A sensitivity analysis of the Modified method for both supercritical and subcritical flow conditions was completed to determine which parameters have more affect on the scour depths. The parameters that were varied include depth of flow, flow, velocity, Froude number, and α. Normal depths were calculated for a typical channel section and then the results were input to the equations included in the Modified method. The typical channel section used has a 50 foot bottom width, 6:1 sideslopes, and a bed slope of 2%. For supercritical flow conditions the Manning's roughness factor (n) is 0.035 and for subcritical flow conditions n = 0.051.

Tables 24 and 25 show the hydraulic parameters used in the sensitivity analysis for supercritical and subcritical flow conditions. As the depth of flow was increased by 0.5 foot, the flow, velocity, and Froude number increased correspondingly. Since the flow depths are the same for both flow conditions, the cross-sectional area, top width, and hydraulic depths are also the same.

Table 24 Hydraulic parameters for typical channel section for supercritical flow conditions

DEPTH(FT)	Q (CFS)	VEL (FPS)	FROUDE	AREA (SF)	TOP WID(FT)	HYD DEP(FT)
1.00	313.60	5.60	1.04	56.00	62.00	0.90
1.50	631.90	7.14	1.10	88.50	68.00	1.30
2.00	1047.30	8.45	1.15	124.00	74.00	1.68
2.50	1559.60	9.60	1.19	162.50	80.00	2.03
3.00	2170.30	10.64	1.22	204.00	86.00	2.37
3.50	2881.80	11.60	1.24	248.50	92.00	2.70
4.00	3697.00	12.49	1.27	296.00	98.00	3.02
4.50	4619.00	13.33	1.29	346.50	104.00	3.33
5.00	5651.10	14.13	1.31	400.00	110.00	3.64

Table 25 Hydraulic parameters for typical channel section for subcritical flow conditions

DEPTH(FT)	Q (CFS)	VEL (FPS)	FROUDE	AREA (SF)	TOP WID(FT)	HYD DEP(FT)
1.00	215.20	3.84	0.71	56.00	62.00	0.90
1.50	433.70	4.90	0.76	88.50	68.00	1.30
2.00	718.70	5.80	0.79	124.00	74.00	1.68
2.50	1070.30	6.59	0.81	162.50	80.00	2.03
3.00	1489.40	7.30	0.84	204.00	86.00	2.37
3.50	1977.70	7.96	0.85	248.50	92.00	2.70
4.00	2537.20	8.57	0.87	296.00	98.00	3.02
4.50	3169.90	9.15	0.88	346.50	104.00	3.33
5.00	3878.20	9.70	0.90	400.00	110.00	3.64

For supercritical flow conditions, the flow was kept constant at 1000 CFS while the slope of the typical channel was varied which affected the flow depth, velocity, and Froude number. Table 26 shows that while the slope increases from .015 ft/ft to .05 ft/ft, the velocity increases from 7.55 fps to 11.29 fps, and Froude number increases from 1.00 to 1.74, the scour depths only increase from 5.83 feet to 6.08 feet. Therefore, varying the velocity and Froude number have little affect on the scour depth.

Table 26 Results of sensitivity analysis of the Modified method for supercritical flow conditions by increasing the bed slope

DEPTH (FT)	SLOPE (FT/FT)	VEL (FPS)	FROUDE NO.	PAR SC (FT)	IMP SC (FT)	EQ31 SC (FT)	BEND SC (FT)	EQ33 SC (FT)
2.11	0.015	7.55	1.00	2.06	2.77	5.83	0.72	3.78
1.95	0.020	8.32	1.15	2.14	2.68	5.82	0.68	3.82
1.74	0.030	9.53	1.38	2.32	2.55	5.87	0.62	3.94
1.60	0.040	10.49	1.57	2.51	2.45	5.96	0.58	4.09
1.50	0.050	11.29	1.74	2.69	2.39	6.08	0.55	4.25

First considering the supercritical flow conditions, Table 27 and Figure 26 show results of the sensitivity analysis completed by increasing the flow depth in the typical channel section. All three scour components; parallel and antidune scour, impingement scour, and bend scour, increased by similar percentages as the flow depth increased. Since the impingement scour depth starts out higher, the percentage increase has more affect on the scour depths than with the other scour components. It should be noted that the impingement scour depth is calculated using hydraulic parameters for the dominant discharge which is 20% of the 100-year discharge. The greater increase in impingement scour depth causes the straight channel and braided stream scour equation (31) to increase greater than the meandering channel scour equation (33) as the flow depth increases.

Table 28 and Figure 27 show the results of the sensitivity analysis completed by increasing the angle α and the flow depth in the meandering channel scour equation (33). The angle α is increased from 20° to 60°, which is the maximum angle suggested by Zeller in the bend scour equation. As the angle α increases, the bend scour component increases linearly. For the bend scour component, an increase in the angle α increases the scour depth more than an increase in the hydraulic parameters of a channel. Therefore, a sharper curve with a smaller centerline radius of curvature has a greater affect on bend scour than an increase in flow and flow depth.

Similar results occur for subcritical flow conditions as for supercritical flow conditions. The increase in flow depth and corresponding hydraulic parameters increases the impingement scour more than parallel and antidune scour and bend scour because it begins at a higher scour depth. Also, the bend scour depth increases when the angle α and flow depth are increased. The scour depth increases more by increasing the angle α than by increasing the flow depth.

Table 27 Results of sensitivity analysis of the Modified method for supercritical flow conditions by increasing the flow depth

DEPTH(FT)	Q (CFS)	PAR SC(FT)	IMP SC(FT)	EQ31 SC(FT)	BEND SC(FT)	EQ33 SC(FT)
1.00	313.60	1.09	1.35	3.44	0.19	2.28
1.50	631.90	1.65	2.05	4.70	0.30	2.94
2.00	1047.30	2.20	2.75	5.95	0.41	3.61
2.50	1559.60	2.74	3.45	7.19	0.52	4.26
3.00	2170.30	3.28	4.14	8.42	0.64	4.92
3.50	2881.80	3.81	4.84	9.65	0.76	5.57
4.00	3697.00	4.34	5.53	10.87	0.88	6.21
4.50	4619.00	4.86	6.22	12.08	1.00	6.86
5.00	5651.10	5.38	6.90	13.28	1.13	7.51

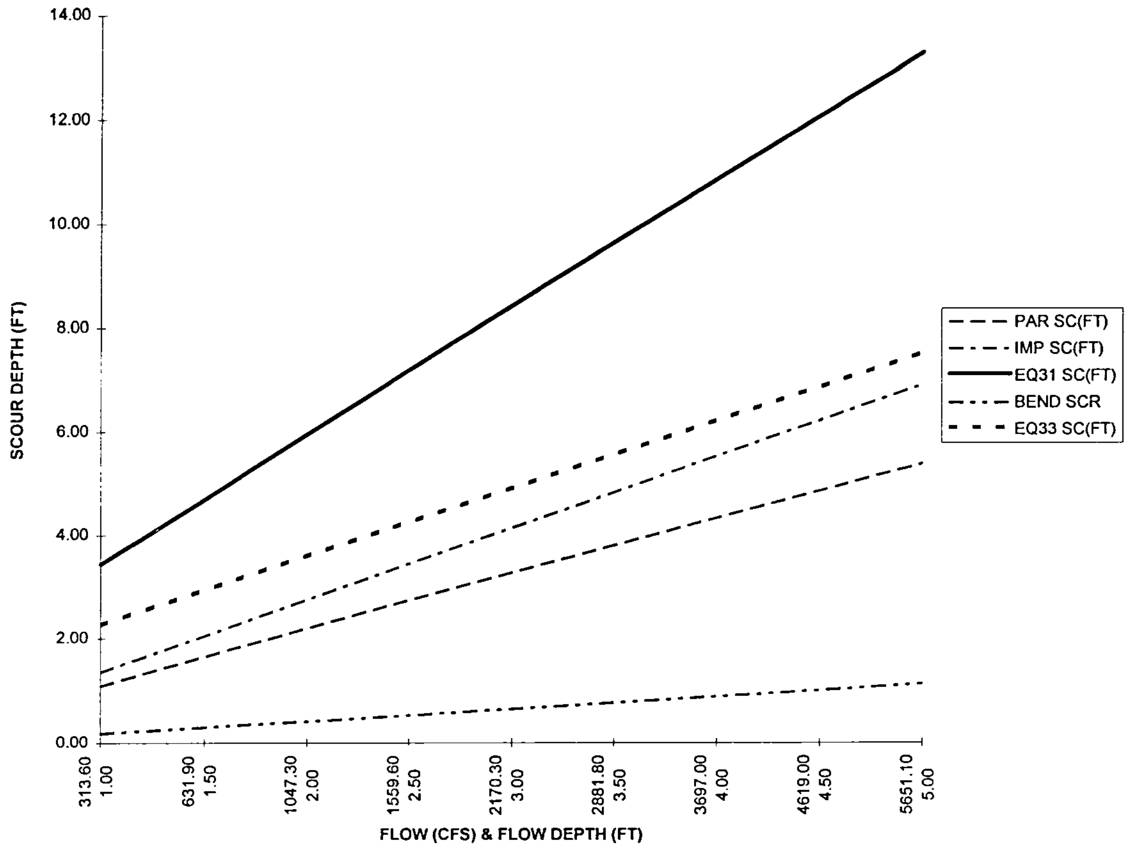


Figure 26 Results of sensitivity analysis of the Modified method for supercritical flow conditions by increasing the flow depth

Table 28 Results of sensitivity analysis of the Modified method for supercritical flow conditions by increasing alpha and flow depth

DEPTH(FT)	Q (CFS)	ALPHA=20	ALPHA=27.2	ALPHA=35	ALPHA=40	ALPHA=50	ALPHA=60
1.00	313.60	0.05	0.19	0.32	0.40	0.57	0.76
1.50	631.90	0.08	0.30	0.51	0.64	0.90	1.19
2.00	1047.30	0.11	0.41	0.70	0.88	1.24	1.65
2.50	1559.60	0.14	0.52	0.90	1.13	1.59	2.11
3.00	2170.30	0.17	0.64	1.10	1.38	1.95	2.58
3.50	2881.80	0.21	0.76	1.30	1.64	2.32	3.07
4.00	3697.00	0.24	0.88	1.51	1.90	2.68	3.55
4.50	4619.00	0.27	1.00	1.72	2.16	3.06	4.05
5.00	5651.10	0.31	1.13	1.93	2.43	3.44	4.55

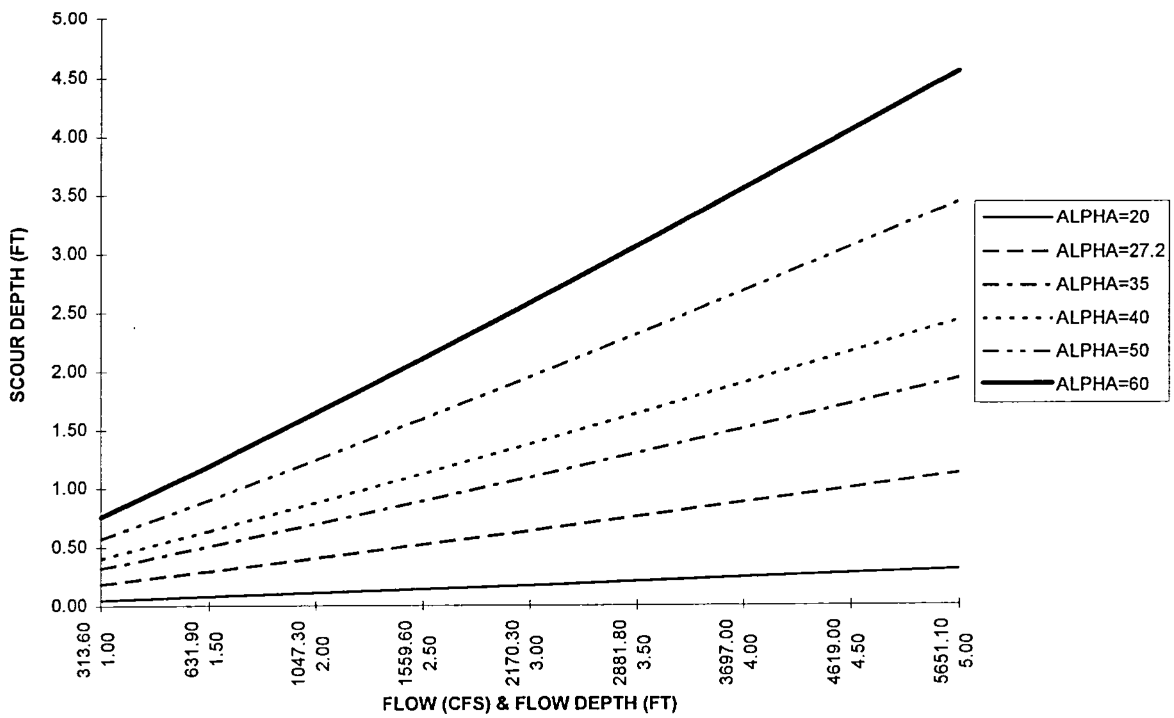


Figure 27 Results of sensitivity analysis of the Modified method for supercritical flow conditions by increasing alpha and flow depth

VII. CONCLUSIONS AND RECOMMENDATIONS

This study evaluated and compared eight methods to estimate scour depth at a flood wall or along a channel bank. An actual project in the Albuquerque area, where a flood wall was designed and constructed to a maximum scour depth following the AMAFCA method, was used for comparing the scour techniques. The Mirehaven Arroyo project has no contraction of the channel section or obstructions within the flow; therefore, contraction scour or local scour due to obstructions in the flow were not included in the analysis. It should be noted that these results are from only one example project and should be verified using other examples with varying hydraulic conditions. Long-term aggradation or degradation were not considered in this study.

Three of the eight methods were developed by federal agencies, while the remaining five approaches are used by local flood control agencies. The three federal agency methods do not have any similarities in their approaches. The FHWA method, which only considers bed material particle size, is not appropriate for the example problem because it is too conservative. The USACE equation is suggested for use only as a gross estimate of scour depth. The most comprehensive method that gives reasonable results is the USBR method. The USBR method employs six separate equations, based on varying theoretical approaches, to estimate scour for particular conditions. The results of each equation are compared and then experience and judgment by the design engineer are used to determine the best answer.

The methods of all five of the local flood control agencies are based on empirical equations developed by Colorado State University with some variation in each approach. The total scour depth is made up of several component scour depths with a factor of safety added. The AMAFCA method considers antidune scour, local scour due to flow impinging on the wall, and local scour due to flow parallel to the wall. The Maricopa County and City of Tucson methods are similar to each other. Each approach resulted in considering antidune scour and bend scour multiplied by a factor of safety. Also include in using the City of Tucson method were general scour and low-flow thalweg depth. The Denver and Clark County approaches are very similar. For the example project, none of the scour components were applicable, therefore the scour depth was assumed to be equal to the minimum depth of bury (3 feet) for bank protection. In comparing all five methods used by the local flood control agencies, the AMAFCA method is the most conservative resulting in a scour depth at least twice that of the average of all flood control agencies methods.

The most conservative result is not necessarily the best result. In this case, conservatism in the estimation of scour depths leads to the design and construction of deeper flood walls which significantly increases the cost of the flood walls. On the other hand, if the estimated scour depths are unconservative, the flood wall could fail due to deeper scour depths occurring during an actual storm event, and therefore not serve its intended purpose of protecting property. The only sure method to determine if estimated scour depths are conservative is to measure actual scour depths during a storm event and compare the estimated scour depths to the actual scour depths.

In comparing the results of the scour depth calculations for supercritical flow conditions (Table 12 and Figure 15) and for subcritical flow conditions (Table 15 and Figure 17), there is little agreement of the predicted scour depths among the eight methods. An evaluation of all eight methods for both supercritical and subcritical flow conditions reveals that the USBR method has the most reasonable results because it is closer to, yet above the average of, all methods. Although it may be too conservative with results that are twice the average of all methods, the next most reasonable approach is the AMAFCA method. The Maricopa and Tucson approaches are the least conservative, and therefore they may not be applicable to the example project conditions.

The results of the USBR method shown in the tables and figures are the average of the two most reasonable techniques; the Lacey regime technique and the competent velocity technique. These two equations resulted in the most conservative scour depths for both supercritical and subcritical flows when compared to the results of the other three USBR equations used. The Lacey regime method is the only USBR method that specifically addresses scour along a flood wall through use of the Z factor. The Lacey regime approach is not affected by the change in flow velocity or flow depth. For the example project, the scour depths using the Lacey method were exactly the same for the supercritical and subcritical flow conditions. The competent velocity approach is the only USBR method that includes the bed material particle size, flow velocity, and flow depth. The competent velocity technique does not specifically consider scour along a flood wall.

The AMAFCA method is one of only two methods that specifically considers scour along a flood wall. The AMAFCA method is the only method prescribed by the local flood control agencies that includes local scour acting on the flood wall. Local scour due to flow impinging on the flood wall accounts for approximately 90% of the total scour depth using the AMAFCA method. The Liu equation (6) for local scour at long bridge embankments in subcritical flow conditions, where $a/Y_1 > 25$, is multiplied by the sine of the impingement angle (θ) of the flow.

$$\frac{Y_S}{Y_1} = 4Fr_1^{0.33} \tag{6}$$

For natural channels, the impingement angle (θ) is usually equal to the maximum angle of 71^0 based on a maximum ratio of unconstrained valley width to channel width of 3.5. Therefore, equation (6) is multiplied by the sine of 71^0 or 0.9455. Since the Froude number for arroyos usually range from 0.7 to 1.0 (Mussetter et al., 1994), the local scour depth (Y_S) with an impingement angle of 71^0 ranges from 3.36 times the flow depth to 3.78 times the flow depth. This part of the method significantly contributes to the scour depths.

The local scour from parallel flows is multiplied by the cosine of the impingement angle (θ). The AMAFCA method, shown in equation (7), is based on an assumption that two scour mechanisms, local scour due to an obstruction to the flow and local scour due to parallel flow, are related to the change in momentum caused by the change in flow direction from some angle to the wall (impingement angle) to a direction parallel to the wall.

$$\frac{Y_s}{Y} = \left(0.73 + 0.14\pi\, F_r^2\right)\cos\theta + 4F_r^{0.33}\sin\theta + 1 \tag{7}$$

The basis of this assumption is not given in the discussion of the AMAFCA method in their Design Guide. The local scour from parallel flows is based on the flood wall having a Manning's roughness factor n 1/2 that of the natural channel. This is valid for concrete flood walls, but not for soil cement walls or riprap lined banks that have n values roughly equal to the channel section.

In the AMAFCA method it is not apparent why the scour due to the passage of antidunes is multiplied by the cosine of the impingement angle (θ). The cosine of the maximum impingement angle of 71^0 is 0.3256. This causes the antidune scour to be reduced by two-thirds of its depth. It seems that the antidune scour should be independent of the impingement angle (θ).

A new approach modifying the AMAFCA, Maricopa County, and City of Tucson methods, is presented which considers the scouring mechanisms of each stream classification. Two separate equations are involved; one for straight and braided streams and the other for meandering streams. Scour depths are calculated using both equations and then the most conservative or the equation resulting in greater scour depths is used for the design of flood walls. The results for the Mirehaven Arroyo project using the modified approach are closer to the USBR method and therefore closer to the average of all nine methods (including the modified method). This modified method may be a reasonable approach to estimate scour depth at flood walls.

A sensitivity analysis of the parameters used in the modified method was completed to determine how each parameter affects the resulting scour depth. Results indicate that varying the flow depth influences the scour depth more than varying the velocity and Froude number. It should be noted that varying the deflection angle, α, dramatically affects the bend scour component of equation (33) for meandering channels.

Scour is predominantly a function of the channel hydraulics (flow depth and flow velocity); and bed material characteristics (material gradation). The only method that considers all three of these parameters is the USBR competent velocity method. The AMAFCA, USBR Lacey, and modified method does not involve all three variables. A comprehensive method that considers all three parameters and specifically addresses scour along flood walls needs to be developed.

Further research to develop a method to accurately predict scour depths along flood walls adjacent to arroyos is required. The next steps in determining the most reasonable approach to estimate scour depth at flood walls adjacent to arroyos are to apply the methods evaluated in this study to other actual projects with varying hydraulic conditions having flood walls and then verify the results of the evaluation performed in this study and the other actual projects with field and laboratory data. This can be accomplished by installing instrumentation in the Mirehaven Arroyo and other arroyos with flood walls to measure flow, flow depth and scour depth during storm events to help validate the results in this study or develop a comprehensive method that estimates scour depth along flood walls. Stream gages can be installed in arroyos at a control section to measure depth of flow and flow rate. Vertical chains, similar to those used by Gerbrandt (1986), can be buried in arroyo beds to measure scour depth. It is important to locate the buried chains carefully and mark their location well so that they can be found later.

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