A PROCEDURE FOR DETERMINING LOCAL SCOUR DEPTHS IN OVERBANKS AND SHEETFLOW AREAS IN UNINCORPORATED PIMA COUNTY, ARIZONA

PREPARED FOR:

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1. BACKGROUND

The Pima County Regional Flood Control District (District) has promulgated three primary technical policies as aids in constructing safe structures to be placed in floodway fringe areas (hereinafter referred to as "overbanks") and sheetflow areas located within unincorporated Pima County, Arizona. These technical policies are:

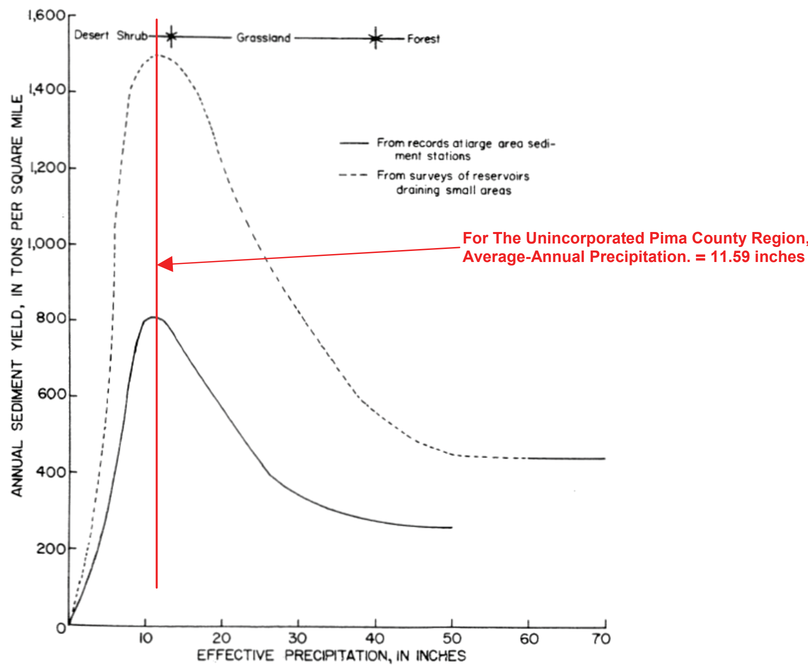
Technical Policy 003, *Minimum Construction Requirements for Manufactured Home Foundations in Floodway Fringe Areas*;

Technical Policy 006, *Erosion Protection of Fill Pads in Floodway Fringe Areas*; and

Technical Policy 014, *Erosion Protection of Stem Wall Foundations in Floodway Fringe Areas*.

A key element contained in each of these three technical policies is the established procedural steps for determining the default local scour depth(s) at and near the footprint(s) of the planned structure(s).

The basis for these three technical policies is rooted in the recognition that flows that occur in the unincorporated Pima County region contain some of the highest annual sediment yields, and thus erosion rates, in the nation. The following figure illustrates the severity of annual sediment yield in the unincorporated Pima County region.



Sediment Yield, as Affected by Climate (Langbein and Schumm, 1958)

As the preceding figures indicates, for the unincorporated Pima County region the predicted annual sediment yield is essentially at the apex of each of the two graphs depicted. This is due to an average-annual precipitation of 11.59 inches (NWS, June 2011)—which is not quite enough precipitation to provide sufficient ground cover to mitigate erosion, yet more than enough precipitation to create significant erosion issues.

Given the severity of erosion issues in the unincorporated Pima County region, the District recognized that there was a need to select appropriate methodologies that would enable the magnitude of erosion to be quantified both properly and adequately. Many methodologies are available in the technical literature for quantifying erosion, including specific procedures for determining scour at a point location (e.g., local scour depths caused by obstructions such as bridge piers and abutments). These latter types of procedures are important for quantifying local scour depths at structures located not only in floodways and channels, but in overbanks and sheetflow areas as well. These are the procedures that the District relied upon during its development of the three primary technical policies it now recommends be used to predict local scour depths—with its preference being to select methodologies, when available, that were developed for semi-arid conditions similar to those of Pima County.

As its basis for predicting local scour depths, the District ultimately selected the methodologies presented in *Chapter 6* of the City of Tucson *Drainage Standards Manual*. In addition, these selected methodologies were supplemented with results from two-dimensional (FLO-2D) model studies that were conducted to assess localized flow patterns at structures. Once the methodologies were selected, the District chose both specific and variable hydraulic parameters for use in quantifying the magnitude of local scour depths. The District relied upon a fixed Manning n-value and a range of longitudinal slopes observed in local overbanks and sheetflow areas in order to create an established envelope of flow depths and flow velocities that would encompass the array of flow conditions anticipated to occur in overbanks and sheetflow areas located within unincorporated Pima County, from which local scour depths could then be predicted in a straightforward manner.

However, subsequent to adoption of the referenced Technical Policies it became clear that the available scour equations for piers and abutments (used for determining local scour at stem walls or fill pads in overbanks and sheetflow areas) are *time independent*, and thus are based on the precept that the local scour depth created by such an obstruction will attain its maximum predicted depth for the specified flow, regardless of flow duration. The District recognized that the problem with applying this precept to overbanks and sheetflow areas in the unincorporated Pima County region is that most, if not all, of the time due to the short duration of such flows there is a significant overestimation of the amount of local scour that would occur when applying *time independent* scour equations to overbank/sheetflow areas.

There are several reasons for this overprediction of local scour. First, overbanks contiguous to riverine flows only experience flooding when the discharge is high enough to cause flows to be “out of bank.” The typical time period that a peak discharge occurs on a majority of the watercourses within the unincorporated Pima County region is less than 30 minutes—and in many cases the duration is less than 10 minutes. Consequently, enough time simply does not exist for local scour at a pier or abutment (for stem walls or fill pads) located in an overbank area to attain its maximum predicted depth calculated using the *time independent* engineering formulas found in the technical literature. Furthermore, the character of soils in overbank areas often differ from the character of soils in the active streambed of a watercourse. The presence of clays or caliche lenses close to the ground surface, as well as desert pavement at the ground surface, can retard the normal process of local scour that would otherwise occur if the flow were in the streambed of a watercourse comprised of active sands and gravels.

For the most part, the same reasoning that applies to overbanks contiguous to riverine flows can be applied to sheetflow areas located in the unincorporated Pima County region. That is, flow durations for sheetflow areas are typically short—thus the flow durations of peak discharges are even shorter. Furthermore, as with riverine flows in overbank areas the character of surface soils in sheetflow areas often can differ from the character of soils in the streambed of a watercourse comprised of active sands and gravels. Thus, the District recognized that *time independent* scour-prediction equations for piers and abutments (for stem walls or fill pads) are likely to overpredict scour depths for flows within overbanks and sheetflow areas of unincorporated Pima County. Accordingly, the procedure contained within this document was developed to provide a means for predicting the amount of *time dependent* local scour that will occur at piers and abutments (for stem walls or fill pads) that are located in overbanks and sheetflow areas in the unincorporated Pima County region.

II. LITERATURE SEARCH

Prior to development of the procedure for determining local scour in overbanks and sheetflow areas, an extensive literature search was conducted. After obtaining and reviewing the literature, the contents of many documents were found to be inapplicable, given hydraulic conditions in overbanks and sheetflow areas in the Pima County region. With this in mind, following is a listing of all the literature obtained and reviewed for data and information in anticipation that such literature *might* provide technical guidance and/or insight into predicting local pier or abutment scour in overbanks and sheetflow areas in the Pima County region.

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III. FLOW CHARACTERIZATION

Characterization of flow in overbanks and sheetflow areas is most efficiently accomplished, with accuracy, by using a one-dimensional backwater model, such as HEC-RAS; though for complex flow conditions, it may be more appropriate to use a two-dimensional flow model, such as FLO-2D. Typically, model selection (i.e., 1D or 2D) is made on a case-by-case basis. One major difficulty in characterizing flow in overbanks and sheetflow areas is determining the top-width of the active flow conveyance portion of a flood plain. The problem becomes more acute in sheetflow areas, where there often is a lack of channel definition for which the area of primary flow conveyance can be identified. In very broad sheetflow areas, where the floodplain width can be thousands of feet, or more, a flow top-width/widths, for the active conveyance area/areas becomes even more difficult to determine. Accordingly, in broad overbanks and sheetflow areas it is important to be able to predict, in an accurate manner, the unit discharge occurring at any particular point in a flood plain during a design flood. Unit discharge is simply the product of the depth of flow times the velocity of flow ($= yV$).

Absent computer-modeling results that can definitively characterize flow in overbanks and sheetflow areas, "rule-of-thumb" techniques must be applied. These techniques are described below. However, whenever the bed/energy slope is steep, such that supercritical conditions would otherwise be calculated, it is presumed that, instead, critical depth is always maintained along the alluvial flow path.

When only limited data (e.g., discharge and generalized topographic data, such as USGS quad maps) are available, unit discharge, depth, and velocity of flow in overbank/sheetflow areas encompassing the primary flow conveyance area can be approximated using the following "rule-of-thumb" relationships (Note that *these relationships should only be used when a backwater analysis cannot be conducted in a reasonably efficient, cost effective manner*).

$q = 0.083Q^{0.6}$ when flow is near critical ($F_r \geq 0.86$).

$q = 0.055Q^{0.6}$, when flow is subcritical ($F_r < 0.86$).

$y = 0.177(S_c/n^2)^{-0.3}Q^{0.36}$ and $V = 0.469(S_c/n^2)^{0.3}Q^{0.24}$, when flow is near critical ($F_r \geq 0.86$).

$y = 0.138(S_c/n^2)^{-0.3}Q^{0.36}$ and $V = 0.398(S_c/n^2)^{0.3}Q^{0.24}$, when flow is subcritical ($F_r < 0.86$).

The preceding equations were derived based upon a presumed range in hydraulic geometry for wide, shallow flow. First for flow at or near critical conditions (when minimum energy dissipation occurs), with a typical width/depth ratio of 200 when $F \approx 1.0$ (high erodibility); and second for flow well below critical conditions, also with a typical width/depth ratio of 200, but when $F \approx 0.35$ (low erodibility).

Examples

In a sheetflow area, the 100-year peak discharge is predicted to be 1,000 cfs, the topographic slope is 0.013 ft/ft, and the estimated Manning n-value is 0.030. Flow is near critical conditions, so predicted depth of flow is 0.96 feet, and the predicted velocity is 5.48 ft/sec.

In a sheetflow area, the 100-year peak discharge is predicted to be 1,000 cfs, the topographic slope is 0.008 ft/ft, and the estimated Manning n-value is 0.045. For this subcritical flow condition, predicted depth of flow is 1.10 feet, and the predicted velocity is 3.15 ft/sec.

IV. APPLICABLE TIME INDEPENDENT EQUATIONS

The Literature Search provided guidance regarding use of the appropriate *time independent* equations for predicting maximum pier scour and abutment scour. Accordingly, the following equations have been determined to be the most appropriate *time independent* abutment scour and pier scour equations to use for the unincorporated Pima County regions, and come from *HEC-18 (Fourth Edition)* published by the FHWA in 2001. The equations are:

$$\frac{y_{sa}}{y_a} = 2.27 K_{1a} K_{2a} \left(\frac{L'}{y_a} \right)^{0.43} F_{ra}^{0.61} + 1 \quad \text{for Abutment Scour} \quad \text{(Equation No. 1a)}$$

$$y_{sp} = 2.0K_1K_2K_3K_4b^{0.65}y_p^{0.35}F_{rp}^{0.43}$$ for Pier Scour (Equation No. 1b)

<u>Where, for Abutments,</u>

F_{ra} = Froude Number of the approach flow, upstream of the abutment = $F_{ra} = \frac{V}{\sqrt{gy_a}}$, from uniform flow or backwater model

K_{1a} = Coefficient for abutment shape = 0.55 for spill-through abutments

K_{2a} = Coefficient for angle of embankment to flow = $(\theta/90^{\circ})$ ($\theta < 90^{\circ}$ when abutment embankment points downstream); typically = 1.0 (assumes abutment normal to flow)

L' = Length of active flow obstructed by abutment embankment, in feet (value of parameter should not exceed one-half of the width of the building foundation or fill pad)

V = Approach velocity of flow, in feet/sec

y_a = Depth of flow on the floodplain directly upstream of the abutment, in feet, from uniform flow or backwater model

y_{sa} = Scour depth at abutment, in feet (except that the maximum $y_{sa} \leq 2.24y_a^{0.5}q_a^{0.33} + y_a$)

+1 = Safety factor to envelope 98 percent of the measured data

<u>Where, for Piers,</u>

b = Pier width, in feet ("b" should include added width to account for debris pile-up)

F_{rp} = Froude number directly upstream of the pier, from uniform flow or backwater model

K_1 = Correction factor for pier nose shape

K_2 = Correction factor for angle of attack of flow

K_3 = Correction factor for bed condition

K_4 = Correction factor for armoring of bed material

y_p = Flow depth on the floodplain directly upstream of the pier, in feet, from uniform flow or backwater model

y_{sp} = Depth of scour at pier, in feet

Note: For round nose piers aligned with flow, maximum scour depth is limited as follows:

$y_{sp} \leq$ 2.4 times the pier width (a) for $F_{rp} \leq 0.8$

$y_{sp} \leq$ 3.0 times the pier width (a) for $F_{rp} > 0.8$

For ease of use, the Equation No. 1a and Equation No. 1b were transformed to eliminate the Froude number as a required variable. The transformation is based upon:

$$F_{ra/p} = \frac{q_{a/p}}{\sqrt{gy_{a/p}^3}}$$ (Equation No. 2)

Where,

$F_{ra/p}$ = Froude number of approach flow upstream of abutment (a) or at the face of a pier (p)

$q_{a/p}$ = Unit discharge of approach flow upstream of abutment (a) or at the face of a pier (p)

g = Acceleration due to gravity

$y_{a/p}$ = Depth of approach flow upstream of abutment (a) or at the face of a pier (p)

Time Independent Abutment Scour Equation

Typically, for the unincorporated Pima County region it can be assumed in Equation No. 1a that K_{1a} = 0.55 (spill-through), that K_{2a} = 1.0 (abutment is normal to flow), and that L' = 0.5L_P (where L_P = length of elevated building pad).

Given that K_{1a} = 0.55; K_{2a} = 1.0; and L' = 0.5L_p, and substituting Equation No. 2 into Equation No. 1a, the following transformed equation for abutment scour results:

$$y_{sa(ti)} = 0.3214 L_p^{0.43} y_a^{-0.345} q_a^{0.61} + y_a \qquad \text{(Equation No. 3)}$$

Time Independent Pier Scour Equation

Typically, for overbanks and sheetflow conditions within unincorporated Pima County, K_1 through K_4 in Equation No. 1b are assigned the following values:

K_1 = 1.1 (this factor assumes a square-nosed shape, which also accounts for the possibility of debris pile-up).

K_2 = 1.0 (this factor assumes a cylindrical pier shape, which is invariant to flow direction).

K_3 = 1.1 (assumes the occurrence of small dunes or anti-dune flow).

K_4 = 1.0* (assumes that, for overbanks and sheetflow conditions, D_{50} is smaller than 2.0 mm and D_{95} is smaller than 20 mm).

*Note: Should D_{50} be larger than 2.0 mm in size and D_{95} be larger than 20 mm in size, then *HEC-18 (Fourth Edition)* should be consulted for the procedure to use to determine the appropriate value for K_4.

Thus, adopting the preceding values and incorporating them into Equation No. 1b, and also substituting Equation No. 2 for F_{rp}, the *time independent* equation for computing maximum pier scour for overflow and sheetflow conditions can be simplified as follows:

$$y_{sp(ti)} = 1.15(b)^{0.65}(y_p)^{-0.295}(q_p)^{0.43} \qquad \text{(Equation No. 4)}$$

The boundary condition for Equation No. 4 is that the maximum scour depth is limited as follows:

$y_s \leq$ 2.4 times the pier width, b, for $F_{rp} \leq 0.8$

$y_s \leq$ 3.0 times the pier width, b, for $F_{rp} > 0.8$

(In the preceding two equations, "*b*" should include added width to account for debris pile-up)

For most overbanks and sheetflow conditions within unincorporated Pima County, $F_{rp} \leq 0.80$, so the *time independent* scour, $y_{s(ti)}$, should not exceed 2.4 times the pier width, *b*, which includes accounting for debris pile-up, as applicable.

Thus:

$$q_p \leq 5.5(y_p)^{0.686}(b)^{0.814}. \qquad \text{(Equation No. 5)}$$

Should $F_{rp} > 0.80$, the *time independent* scour, $y_{s(ti)}$, should not exceed 3.0 times the pier width, b, once again noting that pier width should account for debris pile-up, as applicable.

Thus:

$q_p \leq 9.3(y_p)^{0.686}(b)^{0.814}$. (Equation No. 6)

Consequently, for a specific pier width, b, per Equation No. 5 and Equation No. 6 the unit discharge, q_p, used to compute maximum *time independent* pier scour depth in Equation No. 4 must not exceed $5.5(y_p)^{0.686}(b)^{0.814}$ when $F_{rp} \leq 0.80$, and must not exceed $9.3(y_p)^{0.686}(b)^{0.814}$ when $F_{rp} > 0.80$. Otherwise, set $y_s = 2.4$ times the pier width, b, for $F_{rp} \leq 0.8$; and set $y_s = 3.0$ times the pier width, b, when $F_{rp} > 0.8$.

Note that when applying scour procedures for overbanks and sheetflow areas, it is intended that Equation No. 3 should replace Equation No. 6.9, and that Equation No. 4 should replace Equation No. 6.11, found in the City of Tucson *Drainage Standards Manual*.

Examples

For an abutment

In a sheetflow area where slope = 0.0085 ft/ft and the Manning n-value = 0.060, a depth of flow = 1.5 feet, with a velocity = 3 ft/sec, approaches a building pad 50 feet in width. Note that the effective width of the abutment is 25 feet ($0.5L_p$); since this value equals, but does not exceed, one-half of the width of the building pad. The unit discharge, q_a, = 4.5 cfs/ft.

Applying Equation No. 3 yields:

$y_{sa(ti)} = 0.3214(50)^{0.43}(1.5)^{-0.345}(4.5)^{0.61} + 1.5 \approx \underline{\underline{5.3 \text{ feet}}}$ $[2.24(1.5)^{0.5}(4.5)^{0.33} + 1.5 \approx 6.0 \text{ ft}]$

$y_{sa(ti)} = 5.3$ feet is the *time independent* abutment scour depth for this example, since the *time independent* abutment scour is less than the maximum allowable abutment scour depth of $2.24y_a^{0.5}q_a^{0.33} + y_a = 6.0$ feet.

For a pier

In a sheetflow area where slope = 0.0085 ft/ft and the Manning n-value = 0.060, a depth of flow = 1.5 feet, with a velocity = 3 ft/sec, approaches a pier with an effective width of 2 feet (i.e., a 1-foot-diameter pier with 0.5 feet of debris extending outward on either side, since effective pier width = 2 feet). The unit discharge, q_p, = 4.5 cfs/ft.

For this example, $F_{rp} = 0.43$, < 0.80, and Equation No. 5 yields $q_p = 5.5(1.5)^{0.686}(3)^{0.814} = 17.76$ cfs/ft. Thus, Equation No. 4 is applicable, and:

$y_{sp(ti)} = 1.15(b)^{0.65}(y_p)^{-0.295}(q_p)^{0.43} = 1.15(2)^{0.65}(1.5)^{-0.295}(4.5)^{0.43} \approx \underline{\underline{3.1 \text{ feet}}}$.

$y_{sp(ti)} = 3.1$ feet is the *time independent* pier scour depth for this example.

When checked with Equation No. 5 or Equation No. 6 in order to establish an upper limit for pier scour, Equation No. 4 is a straightforward relationship that can be used to compute *time independent* maximum pier scour depth for overbanks and sheetflow conditions.

V. TIME DEPENDENT REDUCTION FACTORS

The literature search also provided guidance regarding use of the appropriate *time dependent* reduction factors to apply to the *time independent* equations for predicting maximum pier scour and abutment scour in overbanks and sheetflow areas. Accordingly, using the guidance obtained from the literature search conducted the following table of *time dependent* reduction factors were developed. These *time dependent* reduction factors should be multiplied times the *time independent* abutment and pier scour equations presented in Section II of this document.

TABLE OF REDUCTION FACTORS (R_f's) FOR TIME DEPENDENT SCOUR AT ABUTMENTS AND PIERS*

T_f ($1.27 t_r V_c / y_{a/p}$)‡	R_f ($V_{us}/V_c = 1.25$)†	R_f ($V_{us}/V_c = 1.50$)	R_f ($V_{us}/V_c = 2.00$)	R_f ($V_{us}/V_c = 2.50$)†
1800	0.36	0.35	0.32	0.24
2400	0.40	0.39	0.35	0.27
3000	0.43	0.43	0.38	0.30
3600	0.46	0.45	0.41	0.33
4200	0.48	0.47	0.43	0.35
4800	0.50	0.49	0.45	0.37
5400	0.52	0.51	0.46	0.38
6000	0.53	0.52	0.47	0.39
6600	0.54	0.54	0.49	0.41
7200	0.56	0.55	0.50	0.42
7800	0.57	0.56	0.51	0.43
8400	0.58	0.57	0.52	0.44
9000	0.59	0.58	0.53	0.45
9600	0.60	0.59	0.54	0.46
10,200	0.61	0.60	0.54	0.47
10,800	0.61	0.61	0.55	0.47
21,600	0.71	0.70	0.64	0.56
43,200	0.81	0.80	0.73	0.66
86,400	0.91	0.90	0.83	0.75
172,800	1.00	1.00	0.92	0.84

*All units are dimensionless.

‡The value of "t_r" in the Time factor, T_f, is the rise time of the computed flood hydrograph when using the Pima County Flood Hydrograph Procedure. Note that the methodology for converting from watershed time of concentration to rise time when using the Pima County Flood Hydrograph Procedure can be found in Section 4.5 of the City of Tucson *Drainage Standards Manual*. The resultant value of "$1.27 t_r$" represents the time, at a constant peak discharge, to produce an equivalent volume of runoff as that produced by the actual flood hydrograph. In T_f, V_c = velocity at which incipient motion begins for the D_{50} size of the sediment gradation representative of the project study area (assumes $D_{50} = 1$ mm [i.e., no sediment affect] when $b/D_{50} \geq 50$).

†Should T_f be less than 1800, use $T_f = 1800$ as the minimum default value. In addition, should $V/V_c < 1.25$, use 1.25 as the minimum default value. Finally, should $V/V_c > 2.00$, use 2.00 as the maximum default value.

Examples

Abutment Scour

A flow depth of 1.5 feet with a velocity of flow = 3 ft/sec approaches a building pad 50 feet in width. Note that the effective width of the abutment is 25 feet ($0.5L_p$); since this value equals, but does not exceed, one-half of the width of the building pad. The unit discharge, q_a, = 4.5 cfs/ft. The rise time of the flood hydrograph, t_r, is computed to be 26.5 minutes (1590 seconds). Using these conditions, the *time dependent* reduction factor for abutment scour is computed in the following manner.

From the example on Page 9, $y_{sa(ti)}$ = 5.3 feet for *time independent* abutment scour.

Since no sediment particle size distribution curve is available for the site, and it is highly likely that $b/D_{50} \geq 50$, it can be assumed that the median size of sediments, D_{50}, = 1.0 millimeter.

Then, the velocity at incipient motion can be computed from the following equation:

$$V_c = 1.66(y_{a/p})^{1/6}(D_{50})^{1/3} \qquad \text{(Equation No. 7)}$$

Thus $V_c = 1.66(1.5)^{1/6}(1.0)^{1/3} = 1.78$ feet/sec.

Now, we have:

$V_{us}/V_c = (3.0)/(1.78) = 1.69$, and

$1.27 t_r V_c / y_{a/p} = 1.27(1590)(1.78/1.5) \approx 2400$.

From the preceding table, for $T_f = 2400$, $R_f = 0.39$ for $V_{us}/V_c = 1.50$, and $R_f = 0.35$ for $V_{us}/V_c = 2.00$. Thus, for $V_{us}/V_c = 1.69$, $R_f = 0.375$ (interpolated).

Accordingly, for this example *time dependent* abutment scour is predicted to be:

$y_{sp(td)} = 0.375(5.3) \approx 2.0$ feet.

The predicted *time dependent* abutment scour of 2.0 feet represents more than a 60% reduction in the predicted *time independent* abutment scour of 5.3 feet.

Pier Scour

A flow depth of 1.5 feet with a velocity of flow = 3 ft/sec approaches a pier with an effective width of 2 feet (i.e., assumed to be a 1-foot diameter pier with 0.5 feet of debris extending outward on either side). The unit discharge, q_p, = 4.5 cfs/ft. The rise time of the flood hydrograph, t_r, is computed to be 26.5 minutes (1590 seconds). Using these conditions, the *time dependent* reduction factor for pier scour is computed in the following manner.

Applying Equation No. 4, on Page 8, $y_{sp(ti)}$ = 3.1 feet for *time independent* pier scour.

Since no sediment particle size distribution curve is available for the site, it will be assumed that the median size of the sediments, D_{50}, = 1.0 millimeter.

Then, the velocity at incipient motion can be computed from Equation No. 7, as follows:

$V_c = 1.66(y_{a/p})^{1/6}(D_{50})^{1/3}$

Thus $V_c = 1.66(1.5)^{1/6}(1.0)^{1/3} = 1.78$ feet/sec.

Now, we have:

$V_{us}/V_c = (3.0)/(1.78) = 1.69$, and

$1.27t_rV_c/y_{a/p} = 1.27(1590)(1.78/1.5) \approx 2400$.

From the preceding table, for $T_f = 2400$, $R_f = 0.39$ for $V_{us}/V_c = 1.50$, and $R_f = 0.35$ for $V_{us}/V_c = 2.00$. Thus, for $V_{us}/V_c = 1.69$, $R_f = 0.375$ (interpolated).

Accordingly, for this example *time dependent* pier scour is predicted to be:

$y_{sp(td)} = 0.375(3.1) = 1.2$ feet.

The predicted *time dependent* pier scour of 1.2 feet represents more than a 60% reduction in the predicted *time independent* abutment scour of 3.1 feet.

The preceding examples reveal that the predicted abutment scour and local pier scour depths will be significantly less for short-duration *time dependent* flows in overbanks and sheetflow areas within unincorporated Pima County than would otherwise be predicted for *time independent* flows.

VI. Additional Scour Component

Due to the character of the flows, the only additional scour component likely to be present in overbanks and sheetflow areas during passage of a flood event is "contraction scour," typically caused by the presence of support piers underneath an elevated structure, such as a mobile home, which causes some contraction of the flow as it passes underneath the elevated structure. For a typical supporting pier configuration, a reasonable contraction of flow would be 20 percent (0.2) of the otherwise wide-open area located underneath an elevated structure. For this typical supporting pier configuration, contraction scour is estimated to be 20 percent of the approach depth of flow (i.e., $y_{cs} = 0.2y_s$).

Accordingly, total scour at a pier located in overbank or sheetflow areas would be the sum of *time dependent* pier scour plus contraction scour, or:

$y_{s(tot)} = y_{sp(td)} + y_{cs}$. Equation No. 8

Thus, in the case of the preceding example $y_{s(tot)} = 0.375(3.1) + 0.2(1.5) = 1.5$ feet. This value represents the predicted total scour at a pier during passage of a flood in an overbank or sheetflow area that has a depth of flow = 1.5 feet, a velocity of flow = 3 ft/sec, and a unit discharge = 4.5 cfs/ft.

Note that when applying scour procedures for overbanks and sheetflow areas, it is intended that Equation No. 8 for total scour should replace Equation No. 6.3 found in the City of Tucson *Drainage Standards Manual*.

VII. Manning N-Value

In overbank and sheetflow areas within unincorporated Pima County, the Manning n-value can vary significantly. This variation, in turn, can affect the localized depth and velocity of flow that are used to compute unit discharge and, subsequently, abutment scour and pier scour. Accordingly, the maximum and minimum default Manning n-values for use in urbanizing areas to compute abutment scour and pier scour in overbanks and sheetflow areas are set at 0.060 and 0.030, respectively. Note that the lower n-value of 0.030 is often typical of areas where significant amounts of vegetation have been cleared near the location of an abutment or pier in order to provide an area within the property that is appropriate for urban land uses.

Given the preceding, from a sensitivity standpoint for an assumed constant bed slope and depth of flow, varying the Manning n-value from 0.060 to 0.030 will result in a doubling of the localized velocity of flow. The implications for scour as a result of this magnitude of localized change in hydraulic parameters, particularly the to the velocity of flow and corresponding unit discharge, are provided in the examples to follow, and are discussed in Section VIII, below.

<u>Examples</u>

Using the previous examples of this report, and assuming an identical flow depth but assigning a Manning n-value of 0.030 instead of 0.060, the following results are obtained.

Abutment Scour—Time Independent

For a sheetflow area where slope = 0.0085 ft/ft and the Manning n-value = 0.030, a depth of flow = 1.5 feet, with a velocity = 6.0 ft/sec, approaches a building pad 50 feet in width. Note that the effective width of the abutment is 25 feet ($0.5L_p$); since this value equals, but does not exceed, one-half of the width of the building pad. The unit discharge, q_a, = 9.0 cfs/ft.

Applying Equation No. 3 yields:

$y_{sa(ti)} = 0.3214(50)^{0.43}(1.5)^{-0.345}(9.0)^{0.61} + 1.5 \approx \underline{\underline{7.2 \text{ feet}}}$ $[(2.24)(1.5)^{0.5}(9.0)^{0.33} + 1.5 \approx 7.2]$

$y_{sa(ti)}$ = 7.2 feet is the *time independent* abutment scour depth for this example, since the *time independent* abutment scour equals the maximum allowable abutment scour depth of $2.24y_a^{0.5}q_a^{0.33} + y_a = 7.2$ feet.

Pier Scour—Time Independent

In a sheetflow area where slope = 0.0085 ft/ft and the Manning n-value = 0.030, a depth of flow = 1.5 feet, with a velocity = 6 ft/sec, approaches a pier with an effective width of 2 feet (i.e., a 1-foot-diameter pier with 0.5 feet of debris extending outward on either side, since effective pier width = 2 feet). The unit discharge, q_p, = 9.0 cfs/ft.

For this example, F_{rp} = 0.86, > 0.80, and Equation No. 6 yields $q_p = 9.3(1.5)^{0.686}(9.0)^{0.814}$ = 73.45 cfs/ft. Thus, Equation No. 4 is applicable, and:

$y_{sp(ti)} = 1.15(b)^{0.65}(y_p)^{-0.295}(q_p)^{0.43} = 1.15(2)^{0.65}(1.5)^{-0.295}(9.0)^{0.43} \approx \underline{\underline{4.1 \text{ feet}}}$.

$y_{sp(ti)}$ = 4.1 feet is the *time independent* pier scour depth for this example.

Abutment Scour—Time Dependent

From the *time independent* example on Page 13, a flow depth of 1.5 feet with a velocity of flow = 6.0 ft/sec approaches a building pad 50 feet in width. Note that the effective width of the abutment is 25 feet ($0.5L_p$); since this value equals, but does not exceed, one-half of the width of the building pad. The unit discharge, q_a, = 9.0 cfs/ft. The rise time of the flood hydrograph, t_r, is computed to be 26.5 minutes (1590 seconds). Using these conditions, the *time dependent* reduction factor for abutment scour is computed in the following manner.

From the example on Page 13, $y_{sa(ti)}$ =7.2 feet for *time independent* abutment scour.

Since no sediment particle size distribution curve is available for the site, it will be assumed that the median size of the sediments, D_{50}, = 1.0 millimeter.

Then, the velocity at incipient motion can be computed from the following equation:

$$V_c = 1.66(y_{a/p})^{1/6}(D_{50})^{1/3} \qquad \text{(Equation No. 7)}$$

Thus $V_c = 1.66(1.5)^{1/6}(1.0)^{1/3} = 1.78$ feet/sec.

Now, we have:

$V_{us}/V_c = (6.0)/(1.78) = 3.37$ (but since > 2.00, use the default maximum value of 2.00), and

$1.27t_rV_c/y_{a/p} = 1.27(1590)(1.78/1.5) \approx 2400$.

Thus, from the table on Page 10, for $T_f = 2400$, $R_f = 0.35$ for $V_{us}/V_c = 2.00$.

Accordingly, for this example *time dependent* abutment scour is predicted to be:

$y_{sp(td)} = 0.35(7.2) \approx 2.5$ feet (not including contraction scour).

The predicted *time dependent* abutment scour of 2.5 feet represents about a 65% reduction in the predicted *time independent* abutment scour of 7.2 feet. (Note, though, that when there is a high V_{us}/V_c ratio (> 2.00), and higher unit discharge, there are also correspondingly higher amounts of sediment transported into the scour hole from the upstream flow. Accordingly, there are conditions where this circumstance might actually cause the *time dependent* scour depth at an abutment to be slightly less than the *time dependent* scour depth at an abutment subjected to the same depth-of-flow, but where a lower unit discharge occurs.)

Pier Scour

A flow depth of 1.5 feet with a velocity of flow = 6.0 ft/sec approaches a pier with an effective width of 2 feet (i.e., assumed to be a 1-foot diameter pier with 0.5 feet of debris extending outward on either side). The unit discharge, q_p, = 9.0 cfs/ft. The rise time of the flood hydrograph, t_r, is computed to be 26.5 minutes (1590 seconds). Using these conditions, the *time dependent* reduction factor for pier scour is computed in the following manner.

From Page 13, $y_{sp(ti)} = 4.1$ feet for *time independent* pier scour.

Since no sediment particle size distribution curve is available for the site, it will be assumed that the median size of the sediments, D_{50}, = 1.0 millimeter.

Then, the velocity at incipient motion can be computed from Equation No. 7:

$V_c = 1.66(y_{a/p})^{1/6}(D_{50})^{1/3}$

Thus $V_c = 1.66(1.5)^{1/6}(1.0)^{1/3} = 1.78$ feet/sec.

Now, we have:

$V_{us}/V_c = (6.0)/(1.78) = 3.37$ (but since > 2.00, use the default maximum value of 2.00), and

$1.27t_r V_c/y_{a/p} = 1.27(1590)(1.78/1.5) \approx 2400$.

From the preceding table, for $T_f = 2400$, $R_f = 0.35$ for $V_{us}/V_c = 2.00$.

Accordingly, for this example *time dependent* pier scour is predicted to be:

$y_{sp(td)} = 0.35(4.1) = 1.4$ feet (not including contraction scour).

The predicted *time dependent* pier scour of 1.4 feet represents about a 66% reduction in the predicted *time independent* abutment scour of 4.1 feet. (Note, though, that when there is a high V_{us}/V_c ratio (> 2.00), and higher unit discharge, there are also correspondingly higher amounts of sediment transported into the scour hole from the upstream flow. Accordingly, there are conditions where this circumstance might actually cause the *time dependent* scour depth at a pier to be slightly less than the *time dependent* scour depth for a pier subjected to the same depth-of-flow, but where a lower unit discharge occurs.)

For equivalent slopes and depths of flow, depending upon the differences in Manning n-value it cannot be automatically guaranteed that the predicted abutment scour and local pier scour depths will be greater for short-duration *time dependent* flows in overbanks and sheetflow areas within unincorporated Pima County than would otherwise be predicted for short-duration *time dependent* flows with a higher Manning n-value. In fact, the opposite might actually true. Each circumstance is unique. See Section VIII of this report for additional guidance.

VIII. CONDITIONS OF USE

Nonuniform Flow Distribution

Based upon the results of the examples provided in Section VII, it is important to recognize that within areas where overbank and sheetflow conditions occur, flow is likely to have a highly nonuniform distribution due to localized changes in the slope, the Manning n-value, the depth, and the velocity, each and all resulting in sometimes large variations in unit discharge (i.e., the product of depth and velocity) across the flood plain. This is due to the typically large aspect ratios (top width/depth)—that is, the broad nature of flows associated with such conditions—which occur over topography that can vary significantly when measured along a cross-section oriented normal to the direction of flow. Thus, *it is absolutely necessary when determining maximum abutment or pier scour for overbanks or sheetflow areas that the depth of flow, the velocity of flow, and the unit discharge be properly adjusted to account for nonuniform flow conditions*. Based upon typical topographic conditions for overbank and sheetflow areas in unincorporated Pima County, both the depth of flow and the velocity of flow can vary significantly, thus these hydraulic parameters—and corresponding unit discharges in particular—should be adjusted by an appropriate safety factor to account for such conditions at the locations of abutments or piers where localized scour is to be determined.

For example in many sheetflow areas the average depth may be 1.0 foot and the average velocity may be 1.75 feet/sec; but an erosion rill, or gully (as opposed to a low-flow thalweg), might form over time that could potentially double the local flow depth near the abutment or pier, thus increasing the localized depth of flow to 2.0 feet. Consequently, with all other conditions remaining more or less the same (e.g., no change in the Manning n-value), a doubling of the flow depth would result in a corresponding increase in the localized velocity of the sheetflow near the abutment or pier that would be equal to 1.6 times the average velocity, which would be an increase to 2.8 feet/sec. A change in the unit discharge would likewise occur, increasing from 1.75 cfs/ft to 5.6 cfs/ft (i.e., [2]•[2.8]), which would be 3.2 times the unit discharge for "average" sheetflow conditions. Such a localized increase in unit discharge would in turn cause a localized increase in scour at the abutment or pier, as well as an increase in contraction scour. Accordingly, in order to account for the likely occurrence of nonuniform flow distributions, *an assessment of the potential maximum unit discharge should be conducted each time that abutment or pier scour is determined for overbank and sheetflow areas*. In this regard, historic aerial and topographic information can be very useful for determining nonuniform flow distributions.

With regard to a reduction in abutment scour or pier scour to account for *time dependent* flows of short duration, when using the scour procedure for overbanks and sheetflow areas no safety factor should be applied to the calculated local scour depth, since the safety factor is implicitly included in the *time independent* estimate for abutment or pier scour as a consequence of adjusting the flow depth, the flow velocity, and the resultant unit discharge to accommodate conditions where a nonuniform flow distribution exists.

The term "abutment," as used herein, is intended to represent the *effective* obstruction to flow caused by a building foundation or a fill pad located within the flood plain. The *effective* width of the abutment, or L', is defined under Equation No. 1a as "the length of active flow obstructed by the abutment embankment, in feet." As stated at the top of Page 2 of this document, for overbank flow or sheetflow conditions L' is equivalent to $0.5L_p$, which assumes that that L' does not exceed one-half of the width of the building foundation or fill pad, under the conditions that flow passes around all sides. In this regard, it is noted that Equation No. 3 is derived based upon the presumption that a spill-through (sloped) fill will exist at the edge of the pad. If the flow does not go around both sides of the pad, the *effective* width of the abutment, L', must be adjusted accordingly. In addition, if the edge of the pad is vertical, then the numerical coefficient "0.3124" in Equation No. 3 must be multiplied by the factor 1.8182 to increase the value of the numerical coefficient to "0.5680," which then accounts for the amount of *time independent* abutment scour predicted to occur at the edge of vertical fill.

Debris Pile-Up

For overbank and sheetflow areas located within unincorporated Pima County, the potential for debris pile-up on piers should be accounted for by assuming an effective pier width that incorporates the impact of debris when calculating local scour depths. In this regard, it is recommended that the typical value to use in Equation No. 4 for the maximum effective width of a pier located in overbanks or sheetflow areas be 2 feet, and that the minimum value to use for the effective width of a pier not be less than 1 foot. However, in the event that the physical width of the pier exceeds 2 feet, then the actual width of the pier should be used in Equation No. 4 to compute *time independent* and *time dependent* local pier scour depth.