Scour Evaluation for Pipeline Crossing of the Agua Fria River Downstream of Bell Road

Maricopa County, Arizona

WEST

CONSULTANTS, INC.

Water • **E**nvironmental • **S**edimentation • **T**echnology

DRAFT

Hydraulics • Hydrology • Sedimentation • Water Quality • Erosion Control • Environmental Services

5/11/05 comments by. Bing Zhao of FCDMC
(The report does not address the comments)

1. After the removal of three most upstream cross-sections, the location of the Bell Road bridge matches the aerial photo. As explained by WEST, the problem is due to the addition of the three non-georeferenced cross sections. It is verified that the results do not change. Although the velocity and flow depth results do not change after the removal of three cross-sections, the final HEC-RAS model should be revised by removing those three cross-sections and adding the flow rate at the most upstream cross-section.
2. Zeller's general scour: The hydraulic depth should be 4.77 feet instead of 4 feet at RS 19.066 based on HEC-RAS model. The resulting general scour based on Zeller's equation should be 0.01 feet instead of 0.52 feet.
3. The flow depth used in computing critical velocity for contraction scour should be the average flow depth upstream of the bridge instead of hydraulic depth. Please make correction on page 12 in the report. However, the result should not change regarding whether it is live-bed or clear water.
4. On page 13, the contraction scour equation is shown. However, no calculations are presented. It only indicated on page 14 that the contraction scour evaluation did not result in a significant factor for a peak flow of 37500 cfs. How do you know it without calculations?
5. Abutment scour: The final abutment scour is 13.8 feet as presented in the report. More justification is necessary to justify that no scour will happen at the downstream guidebank. Local scour for spur dike and guidebank can be computed. The literature is available from the District.
6. On page 29, RS 18.987 (the sixth line) should be RS 18.978
7. Bedform scour: WEST used the average flow depth. When using the maximum flow depth, the scour depth due to dunes is 4.97 feet instead of 3.1 feet.
8. Bend scour depth: The angle should be around 40 degrees instead of 20 degrees. By using 40 degrees, the bend scour depth should be 3.3 feet instead of 0.4 feet. With 1.3 safety factor, the bend scour should be 4.3 feet instead of 0.5 feet. Here the net increase is 4.3-0.5=3.8 feet.
9. The time that requires to fill the pit should be around 4.75 hours instead of 3.5 hours (page 31).
10. The equation of dimensionless width on page 32 is incorrect (should be Wp/Wc instead of Wc/Wp)
11. The values on Figure 11 on page 32 do not match Q's in Table 9 on page 34.
12. There are some errors in Table 9. For example, T* for hour 2 should be 0.57 instead of 0.00. However, the largest ls should not change significantly. Please make corrections to Table 9.
13. Sand Gravel Mining Pit headcutting: The distance from the upstream pit edge to the pipeline location near the downstream guide bank is about 810 feet. By interpolation, the headcutting should be 9.6 feet instead of 0.7 feet. In addition, they used the 2000 aerial photo which is very different from 2005 photo. Based on 2005 aerial photo, the distance from

WEST never finalized the report based on above comments because SW gasline decided to use directional drilling 50 ft below the bed elevation //

the pit's upstream edge to the pipeline near the downstream guide bank is about 370 feet. By interpolation, the headcutting should be 15 feet.

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INTRODUCTION

WEST Consultants, Inc. (WEST) was retained by Southwest Gas Corporation to conduct a scour analysis for the proposed 16-inch natural gas line crossing of the Agua Fria River at Bell Road. The pipeline crossing is proposed to be located approximately 50 feet downstream from the piers of the Bell Road Bridge. The Bell Road Bridge at the Agua Fria River lies on the section line between T3N R1W Section 1 and T4N R1W Section 36, coincident with the boundary line between the towns of Sun City West and Surprise, Arizona. The project location is shown in Figure 1.

DATA COLLECTION

The Bell Road Bridge at the Agua Fria River has nine (9) spans with an overall length of 1,105 feet from center-to-center of the abutment bearings. It has two 36-foot wide roadways separated by a concrete median barrier and a sidewalk on the downstream side. The overall width of the bridge is 84 feet. The bridge was designed for a stream flow rate of 83,000 cfs, which corresponded to the 50-year flood before the New Waddell Dam was constructed. The bridge was designed by Benson and Gerdin in 1981, and it was built in 1982 as Maricopa County Department of Transportation (MCDOT) Project Number 68067. In 1995, the bridge was annexed by Surprise, Arizona. A photograph of the downstream face of the Bell Road Bridge can be seen in Figure 2.

The piers consist of a concrete cap beam supported on three 5-foot diameter formed columns (see Figure 3). The formed columns extend approximately 11 feet below the existing river bed and are supported on three 6-foot diameter drilled shafts that are founded approximately 68 feet below the existing river bed. The piers are oriented normal to the roadway.

According to the as-built plans, the berm slopes at the abutments are protected by wire-tied gabions that extend below the finished grade and out into the channel. The gabions on the right bank downstream from the Bell Road Bridge can be seen in Figure 4. The piers are also protected by below-grade gabions.

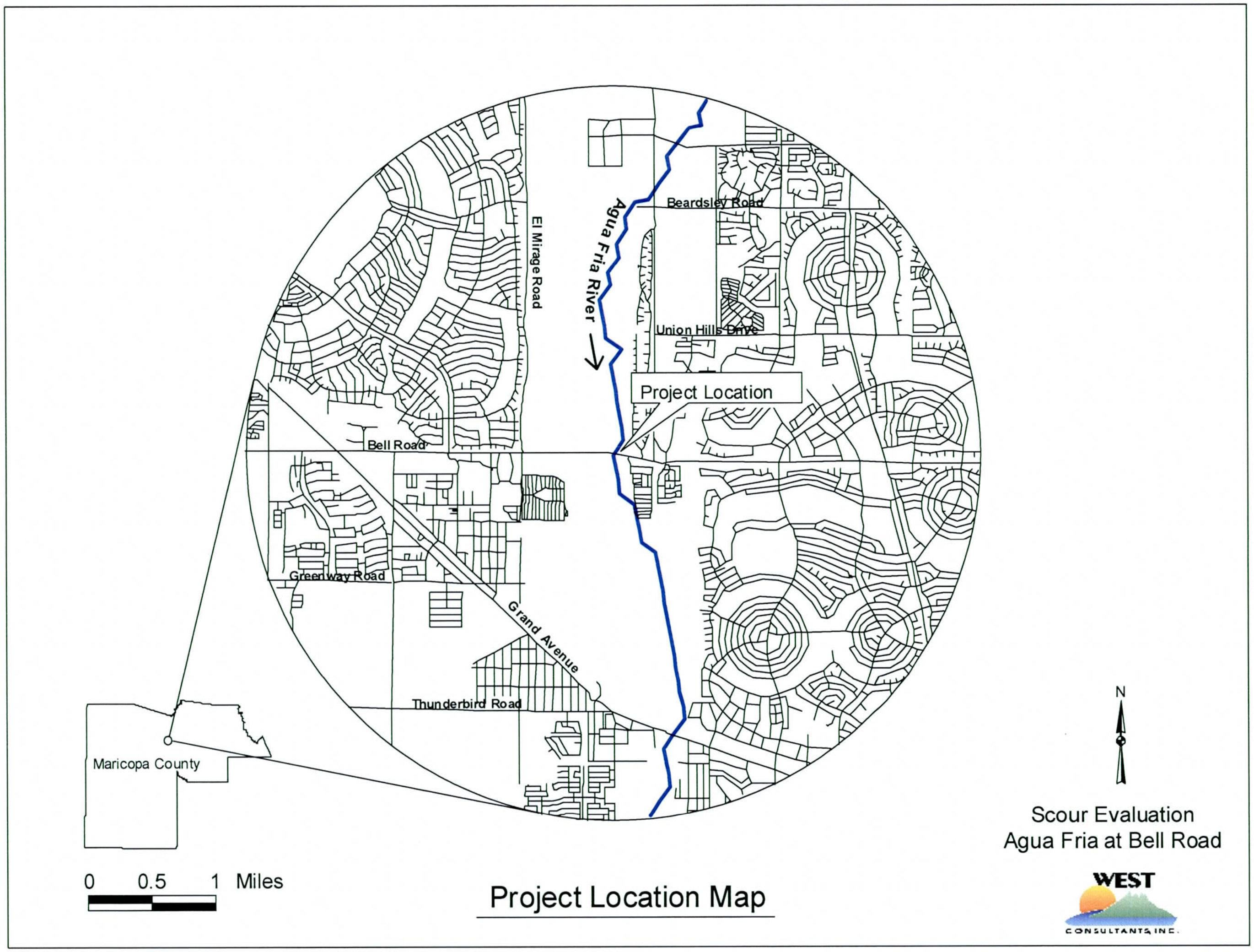

Figure 1. Project location map

Figure 2. Downstream face of the Bell Road Bridge (note the orange stakes indicating the proposed pipeline crossing)

Figure 3. Piers supporting the Bell Road Bridge

Figure 4. Gabions on the right bank downstream from the Bell Road Bridge

The Agua Fria River is a sand bed channel. Information regarding the particle size distribution of the bed sediment was obtained from two studies. The first study was performed by Arizona State University (Carriaga et al. 1994). The sample at the Bell Road Bridge was collected at a depth between 5 and 21 feet and was called Sediment Sample 9. The particle size distribution for the ASU study is shown in Table 1 while a plot of the distribution is shown in Figure 5. The median grain size (D_{50}) for ASU sample was 1.48 mm. A second sediment sample was collected by Hoque and Associates, Inc. for a Kimley-Horn and Associates, Inc. sedimentation study on the Agua Fria River (KHA 2001). The sediment sample collected near the Bell Road Bridge was labeled sample KHA7 and was taken within the first 3 feet of depth. The particle size distribution for the KHA study is shown in Table 2 while a plot of the distribution is shown in Figure 6. The median grain size (D_{50}) for KHA sample was 4.69 mm.

It appears that the median grain size in the bed of the Agua Fria River has become coarser in the years between the ASU study and the KHA study. This may be a result of the sampling strategy. However, it appears that the D_{15} has remained about the same between the two studies while the D_{50} and D_{85} have significantly increased. It is unlikely that the natural flow events on the Agua

Fria River between the two studies could significantly increase the median grain size. A field visit was conducted on April 18, 2005. During the field visit, it was observed that the D_{50} around the Bell Road Bridge was closer in size to the 1.48 mm reported by Carriaga et al. (1994) than the 4.69 mm reported by KHA (2001). For the scour analysis, both the 1.48 mm and the 4.69 mm values of D_{50} were used to calculate the scour depths. The median grain size that yielded the most severe predicted scour was then used as the representative median grain size.

The Manning *n*-value used for the main channel in the hydraulic model of the Agua Fria River (WEST 2002) is 0.035. This *n*-value would appear to be appropriate based on the site visit.

Table 1. Particle size distribution on the Agua Fria River near the Bell Road Bridge from the ASU study (Carriaga et al. 1994)

Sediment Size (mm)	Percent Finer by Weight (%)
0.062	2.279
0.125	4.1
0.25	12.376
0.5	27.492
1	39.64
2	61.44
4	67.724
8	76.484
16	91.103
32	100
64	100

D_{15} (mm) = 0.29
D_{50} (mm) = 1.48
D_{85} (mm) = 12.66

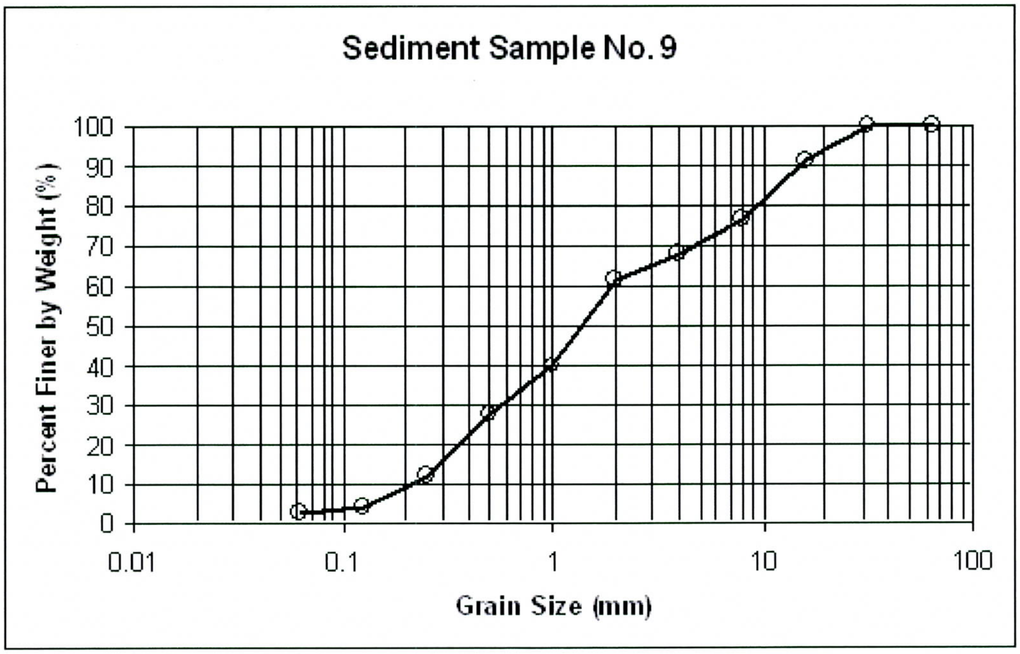

Figure 5. Plot of particle size distribution for the Agua Fria River near the Bell Road Bridge from the ASU study (Carriaga et al. 1994)

Table 2. Particle size distribution on the Agua Fria River near the Bell Road Bridge from the KHA study (KHA 2001)

Sediment Size (mm)	Percent Finer by Weight (%)
0.075	0.3
0.15	1.0
0.3	5.1
0.425	10.8
0.6	19.2
1.18	33.9
2	42.8
2.36	44.9
4.75	50.1
6.3	53.7
9.5	58.8
12.5	62.5
19	69.5
25	74.9
37.5	87.1
50	91.3
62.5	100.0
75	100.0

D_{15} (mm) = 0.51
D_{50} (mm) = 4.69
D_{85} (mm) = 35.33

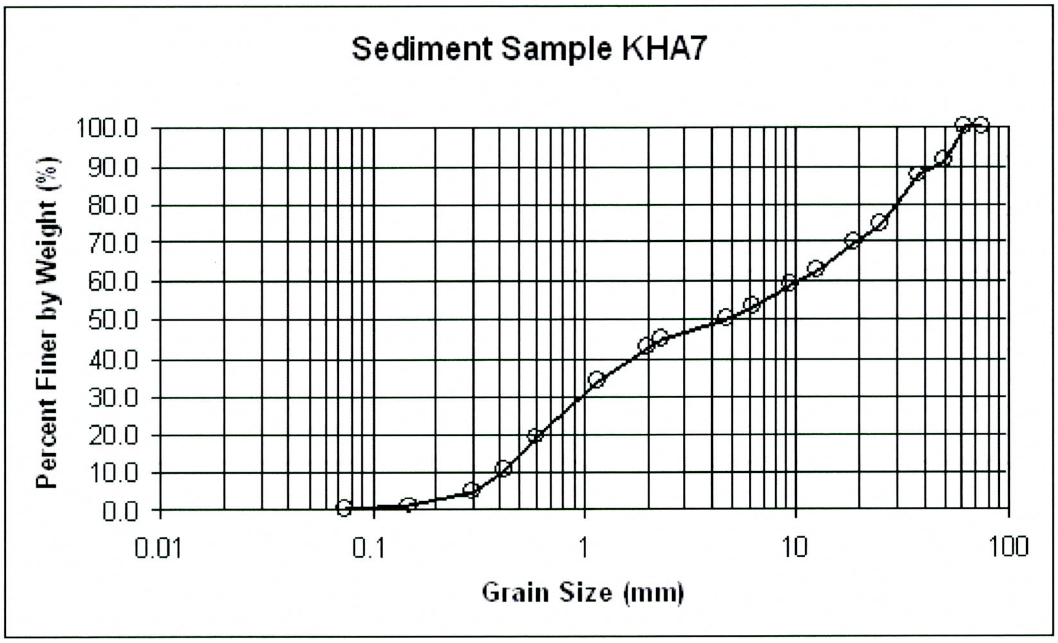

Figure 6. Plot of particle size distribution for the Agua Fria River near the Bell Road Bridge from the KHA study (KHA 2001)

HYDROLOGY

Prior to 1992, the hydrology of the Agua Fria River was based on a US Army Corps of Engineers hydrologic evaluation. Details on this hydrologic evaluation can be found in the flood insurance study performed by Jerry R. Jones and Associates, Inc. in 1989 (Jones 1989). The pre-1992 hydrology for the Agua Fria River at Bell Road can be seen in Table 3.

In 1992, construction was completed on the New Waddell Dam on the Agua Fria River. This dam, which was built to replace the smaller Waddell Dam, was designed to provide water supply, regulatory storage of Central Arizona Project (CAP) water, and recreation. In 1995, the USACE completed a new hydrologic evaluation of the Agua Fria River based on the New Waddell Dam (USACE 1995). This is the same hydrology used in the most recent flood insurance study performed by Coe and Van Loo Consultants, Inc. (CVL 1996). The USACE (1995) report presents the peak discharges for return periods from 2 to 500 years and these values are shown in Table 4. Note that the construction of the New Waddell Dam significantly lowered the peak flows in the Agua Fria River. For example, the 10-year peak discharge before the construction of the New Waddell Dam was 37,000 cfs. Following construction of the New Waddell Dam, the 10-year peak discharge dropped to 11,000 cfs and the 100-year peak discharge dropped to 37,500 cfs.

Table 3. Pre-1992 Frequency-discharge relationship on the Agua Fria River at Bell Road (Carriaga et al. 1994)

Return Period (years)	Peak Discharge (cfs)
10	37,000
25	60,000
50	87,000
100	115,000
500	182,000

Table 4. Post-1992 Frequency-discharge relationship on the Agua Fria River at Bell Road (UASCE 1995)

Return Period (years)	Peak Discharge (cfs)
10	11,000
25	20,500
50	29,000
100	37,500
200	46,500
500	59,000

HYDRAULICS

The US Army Corps of Engineers River Analysis System standard-step backwater computer program (HEC-RAS, Version 3.1.2) was used to compute channel hydraulics (USACE 2004). The model was originally developed by CVL (1996) as part of their flood insurance study. This hydraulic model was later refined by WEST Consultants, Inc. using more recent topography (WEST 2002a). The spatial limits of this HEC-RAS hydraulic model are from Bell Road downstream to approximately Cactus Road. The downstream boundary conditions for the updated model were determined from the 1996 CVL model. The downstream study limit on the Agua Fria River was fixed by copying the geometry of cross-section 15.564 from the 1996 CVL model. The water surface elevation was then fixed to reflect that of the 1996 CVL model at this cross-section. Cross-sections were also added to the upstream end of the model in order to tie into the effective flood mapping developed by CVL (1996). The WEST (2002a) hydraulic model is included on the CD attached to the report.

SCOUR CALCULATIONS

The proper consideration of scour at a site requires a determination of the total scour. Total scour refers to the total depth of scour at a given location and is the sum of all scour components that apply to the site of interest. These scour components can included:

- General scour or contraction scour
- Local scour
- Long-term degradation
- Bend scour
- Bed form scour
- Low-flow channel incisement

A factor of safety may be applied to account for uncertainty of the data, degree of variability of the channel conditions, level of risk, etc. The factor of safety may be applied to some or all of the scour components. The total scour at a given location is the sum of the individual components that are applicable at that location.

General Scour

General scour is the lowering of the streambed across the channel or stream over relatively short time periods (e.g., the general scour in a given reach after the passage of a single flood event). The lowering may be uniform across the bed or non-uniform (i.e., the depth of scour may be deeper in some parts of the cross-section).

General scour may result from concentration of the flow when the flow area of a stream is decreased from the normal either by a natural constriction or a manmade constriction (i.e., local encroachment, bridge, etc.). With the decrease in flow area there is an increase in average velocity and bed shear stress.

The following equations were used to estimate the general scour for the 100-year recurrence interval flood which is equal to 37,500 cfs (Zeller 1981):

$$y_{gs} = y_{\max}\left[\frac{0.0685V_m^{0.8}}{\left(y_h^{0.4}S_e^{0.3}\right)} - 1\right]$$

where: y_{gs} = general scour depth (ft),

y_{max} = maximum depth of flow at design discharge (ft),

V_m = average velocity of flow at design discharge (ft/s),

y_h = hydraulic depth of flow at design discharge (ft), and

S_e = energy slope or bed slope for uniform flow (ft/ft).

The general scour was evaluated at RS 19.066, which is about 500 feet upstream from the Bell Road Bridge. Using WEST's HEC-RAS model of the Agua Fria River (WEST 2002a), the following hydraulic variables were determined:

- $y_{max} = 7.1$ ft,
- $V_m = 8.7$ ft/s
- $y_h = 4.0$ ft, and
- $S_e = 0.005227$.

Note that the hydraulic depth that was used in this calculation was the hydraulic depth for the entire cross-section that was conveying flow. The hydraulic depth for the main channel could have been used; however, doing so leads to almost zero general scour. To be on the conservative side, it was decided that the hydraulic depth for the entire cross-section that was conveying flow would be used. Using this information, the general scour can be calculated as:

$$y_{gs} = (7.1)\left[\frac{(0.0685)(8.7)^{0.8}}{(4.0)^{0.4}(0.005227)^{0.3}} - 1\right] = 0.52 \text{ ft}$$

Using a factor of safety of 1.3 the general scour, y_{gs}, was estimated to be 0.7 ft.

Contraction Scour

Contraction scour is a form of general scour that is based on the principle of conservation of sediment continuity. In the case of live-bed scour, the fully developed scour in the contraction reaches equilibrium when sediment transported into the contracted section equals sediment transported out. For clear-water scour, the transport into the contracted section is essentially zero and the maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section.

To determine whether to use live-bed or clear-water contraction scour, the critical velocity above which the bed material of size D_{50} and smaller will be transported (V_c) needs to be calculated. If V_c is greater than the average velocity, then clear-water contraction scour will occur. If V_c is less than the average velocity, then live-bed contraction scour will occur. Richardson and Davis (2001) give the following equation for calculating the critical velocity, V_c:

$$V_c = K_u y_h^{1/6} D_{50}^{1/3}$$

where: V_c = critical velocity (ft/s),

K_u = 11.17 for English units,

y_h = average depth of flow upstream of the bridge (ft), and

D_{50} = particle size in a mixture of which 50% are smaller (ft).

In the HEC-RAS model of the Agua Fria River (WEST 2002a), the average velocity upstream of the bridge is 5.31 ft/s for the 100-year discharge of 37,500 cfs while the average depth is 6.55 ft. Using a D_{50} of 4.69 mm (or 0.0154 ft), the critical velocity can be calculated as:

$$V_c = 11.17(6.55)^{1/6}(0.0154)^{1/3} = 3.80 \text{ ft/s.}$$

Since, V_c is less than the average velocity of 5.31 ft/s, the contraction scour at the bridge is consider live-bed contraction scour. Using a D_{50} of 1.48 mm (or 0.004856 ft), the critical velocity can be calculated as:

$$V_c = 11.17(6.55)^{1/6}(0.004856)^{1/3} = 2.59 \text{ ft/s.}$$

Again, V_c is less than the average velocity of 5.31 ft/s and the contraction scour can be considered live-bed contraction scour. Since both values of D_{50} reported for the Agua Fria at Bell Road indicate that live-bed contraction scour occurs at the bridge, only the live-bed contraction scour equation was evaluated. The live-bed contraction scour was evaluated at the Bell Road Bridge using a modified version of Laursen's (1960) equation for live-bed contraction scour at a long contraction (Richardson and Davis 2001). This live-bed contraction scour equation assumes that the bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7}\left(\frac{W_1}{W_2}\right)^{k_1}$$

$$y_s = y_2 - y_0$$

where: y_s = average live-bed contraction scour depth (ft),

y_1 = average depth in the upstream channel (ft),

y_2 = average depth in the contracted section (ft),

y_0 = existing depth in the contracted section before scour (ft),

Q_1 = flow in the upstream channel transporting sediment (cfs),

Q_2 = flow in the contracted channel (cfs),

W_1 = bottom width of the upstream main channel that is transporting bed material (ft),

W_2 = bottom width of the main channel in the contracted section less pier width(s) (ft),

k_1 = exponent determined from Table 5.

Note that the top width can be used as an approximation of the bottom width in the above equation (Richardson and Davis 2001).

Table 5. Value of k_1 as a function of V_* / ω (from Richardson and Davis 2001)

$\frac{V_*}{\omega}$	k_1	Mode of Bed Material Transport
< 0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
> 2.0	0.69	Mostly suspended bed material discharge

where: V_* = shear velocity in the upstream section (ft/s) = $\sqrt{\frac{\tau_0}{\rho}} = \sqrt{g y_1 S_1}$,

ω = fall velocity of bed material based on the D_{50} (ft/s) (see Figure 7),

g = acceleration of gravity (32.2 ft/s^2),

S_1 = slope of the energy grade line of the main channel (ft/ft),

τ_0 = shear stress on the bed (lb/ft^2), and

ρ = density of water (1.94 slugs/ft^3).

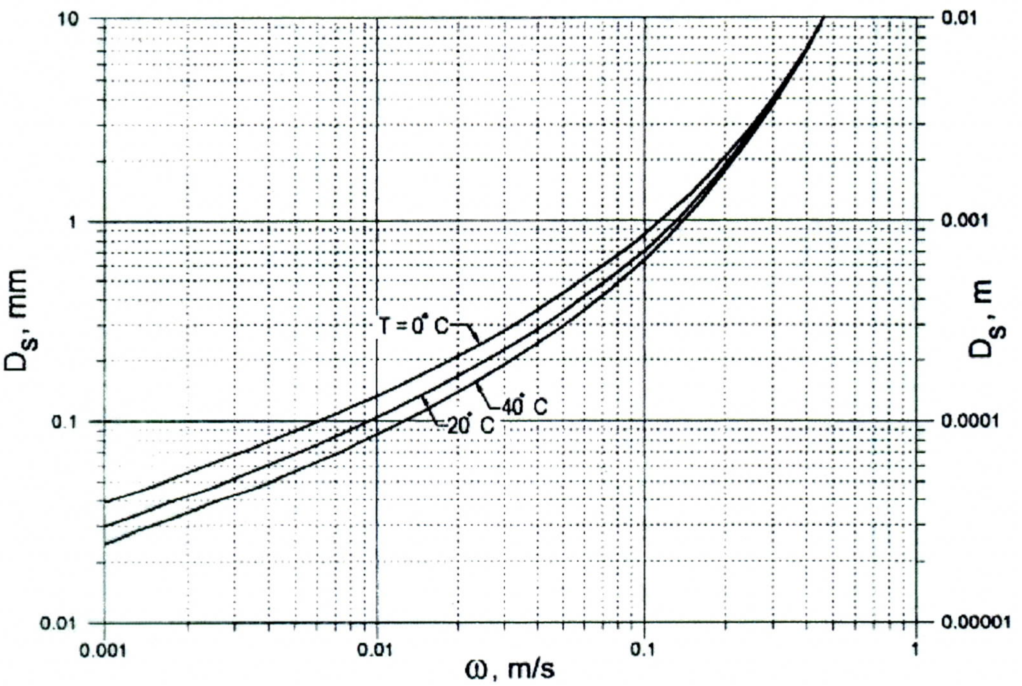

Figure 7. Fall velocity of sand-sized particles with specific gravity of 2.65 in metric units (from Richardson and Davis 2001)

Note that the fall velocity obtained from Figure 7 is in metric units; it must be multiplied by 3.28 to convert it to English units.

In calculating the contraction sour, the Bell Road Bridge was used as the first cross-section and RS 19.352 was used as the second cross-section. RS 19.352 is about 2,000 feet upstream from the Bell Road Bridge. From the HEC-RAS model (WEST 2002a), the following hydraulic data were collected:

- y_1 = 3.02 ft,
- y_0 = 6.55 ft,
- Q_1 = 37,500 cfs,
- Q_2 = 37,500 cfs,
- W_1 = 2,682 ft (top width),
- W_2 = 996 ft (top width minus the width of the 8 piers),
- D_{50} = 1.48 mm,

- $g = 32.2$ ft/s^2, and
- $S_1 = 0.002318$.

Note again that the average depth used in this calculation was the average depth for the entire cross-section that was conveying flow and not just the average depth in the main channel. The shear velocity is calculated as:

$$V^* = \sqrt{(32.2)(3.02)(0.002318)} = 0.49 \text{ ft/s.}$$

Using Figure 7, the fall velocity, ω, was found to be 0.40 ft/s. Thus, the ratio of the shear velocity to the fall velocity is:

$$\frac{V^*}{\omega} = \frac{0.49}{0.40} = 1.225$$

From Table 5, the k_1 exponent is 0.64. Plugging all of this information into the contraction scour equation yields:

$$y_2 = (3.02)\left(\frac{37{,}500}{37{,}500}\right)^{6/7}\left(\frac{2{,}682}{996}\right)^{0.64} = 5.69 \text{ ft}$$

$$y_s = 5.69 - 6.55 = -0.86 = 0 \text{ ft}$$

Since the contraction scour was calculated to be a negative number, it can be assumed that there is no contraction scour at the Bell Road Bridge. Note that using a D_{50} of 4.69 mm would yield the same k_1 exponent and, therefore, would not change the contraction scour calculations. In addition, the design of the Bell Road Bridge was based on a larger peak flow (83,000 cfs) than currently is anticipated in the Agua Fria River. Therefore, the recommended general scour depth is based on Zeller's Equation. The recommended depth, including factor of safety, is 0.7 feet.

Local Scour

Local scour is the scour that results from an obstruction and abrupt change in the direction of flow. Local scour is caused by an acceleration of flow and resulting vortices induced by the obstruction. It occurs at bridge piers, abutments, embankments, and other structures obstructing the flow.

Pier Scour

Richardson and Davis (2001) recommend the following equation for both live-bed and clear-water local pier scour:

$$y_s = y_1\left[2K_1K_2K_3K_4\left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}\right]$$

where: y_s = local scour depth (ft),

y_1 = flow depth directly upstream of pier (ft),

K_1 = correction factor for pier nose shape from Table 6,

K_2 = correction factor for angle of attack of flow from Table 7,

K_3 = correction factor for bed condition,

K_4 = correction factor for armoring by bed material size,

a = pier width (ft),

Fr_1 = Froude number,

L = length of pier (ft) in Table 7, and

H = dune height (ft) in Table 8.

Table 6. Correction factor, K_1, for pier nose shape (from Richardson and Davis 2001)

Shape of Pier Nose	K_1
Square nose	1.1
Round nose	1.0
Circular cylinder	1.0
Group of cylinders	1.0
Sharp nose	0.9

Table 7. Correction factor, K_2, for angle of attack of flow (from Richardson and Davis 2001)

Skew Angle of Flow	$L/a = 4$	$L/a = 8$	$L/a = 12$
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Table 8. Increase in equilibrium pier scour depths, K_3, for bed condition (from Richardson and Davis 2001)

Bed Condition	Dune Height (ft)	K_3
Clear-water scour	N/A	1.1
Plane bed and antidune flow	N/A	1.1
Small dunes	$2 < H < 10$	1.1
Medium dunes	$10 < H < 30$	1.1 to 1.2
Large dunes	$H > 30$	1.3

The correction factor, K_4, decreases scour depths for armoring of the scour hole for bed materials that have a D_{50} greater than or equal to 2.0 mm and a D_{95} greater than or equal to 20 mm. If the D_{50} is not greater than or equal to 2.0 mm and the D_{95} is not greater than or equal to 20 mm, then K_4 equals 1.0. Otherwise, K_4 can be calculated by the procedure outlined by (Richardson and Davis 2001). To be on the conservative side, it was assumed that the K_4 factor was equal to 1.0 regardless of the median grain size.

Local pier scour was evaluated for two cases for the Bell Road Bridge. The first case assumes no debris build-up on the face of the piers. For this case, a factor of safety is applied to the scour calculated scour depth. The second case assumes debris build-up by using that the effective pier width is equal to the actual pier width plus 4 feet and that the pier has a square or blunt face (K_1 = 1.1). No factor of safety was applied to the calculated scour depth for the second case since these assumptions are considered to provide a conservative scour estimate.

Case 1 – No Debris Build-Up

Using the HEC-RAS model (WEST 2002a), the following parameters were determined:

- $y_1 = 7.1$ ft,
- $a = 5.0$ ft,
- $Fr_1 = 0.7$,
- $K_1 = 1.0$ (circular cylinder),
- $K_2 = 1.0$ (angle of attack = 0°),
- $K_3 = 1.1$ (small dunes), and
- $K_4 = 1.0$ (assumed).

Using this information, the local scour can be calculated as:

$$y_s = (7.1)\left[(2)(1.0)(1.0)(1.1)(1.0)\left(\frac{5.0}{7.1}\right)^{0.65}(0.7)^{0.43}\right] = 10.7 \text{ ft}$$

Using a factor of safety of 1.3, the local scour, y_s, is estimated to be 13.9 ft.

Case 2 – With Debris Build-Up

Now assume that there is 4 feet of debris on the piers. In this case, the parameters needed to calculate the local scour are:

- $y_1 = 7.1$ ft,
- $a = 9.0$ ft,
- $Fr_1 = 0.7$,

- K_1 = 1.1 (square nose),
- K_2 = 1.0 (angle of attack = 0°),
- K_3 = 1.1 (small dunes), and
- K_4 = 1.0 (assumed).

The local scour can then be calculated as:

$$y_s = (7.1)\left[(2)(1.1)(1.0)(1.1)(1.0)\left(\frac{9.0}{7.1}\right)^{0.65}(0.7)^{0.43}\right] = 17.2 \text{ ft}$$

To be on the conservative side, it was assumed that the local scour is the worst-case scenario of the above two calculations. Thus, the local pier scour, y_s, was estimated to be 17.2 ft.

Abutment Scour

Scour occurs at abutments when the abutment and embankment obstruct the flow. The west abutment of the Bell Road Bridge is aligned with the west bank of the Agua Fria River and does not obstruct flow. The east abutment of the Bell Road Bridge will obstruct the flow for the larger flow events.

Richardson and Davis (2001) recommend using either the Froehlich equation or the HIRE equation for live-bed abutment scour. If the ratio of the length of the abutment projecting into the flow, L, to the depth of flow at the abutment on the overbank, y_1, is greater than 25, Richardson and Davis (2001) recommend using the HIRE equation. If the ratio is less than 25, then the Froehlich equation is recommended. For the east abutment on the Bell Road Bridge, L is approximately 1,000 feet (measured from the 2000 aerial photographs). The HEC-RAS model of the Agua Fria River (WEST 2002a) indicates that the overbank depth, y_1, upstream of the abutment is 3.23 feet for the 100-year flow rate of 37,500 cfs. Thus, the L/y_1 ratio equals 309 and the HIRE equation should be used to estimate the abutment scour.

According to the HIRE equation (Richardson and Davis 2001), the abutment scour can be calculated from the following:

$$\frac{y_s}{y_1} = 4Fr^{0.33}\frac{K_1}{0.55}K_2$$

where y_s = abutment scour depth (ft),

y_1 = depth of flow at the abutment on the overbank (ft),

Fr = Froude number based on velocity and depth upstream of abutment on overbank,

K_1 = abutment shape coefficient, and

K_2 = coefficient for skew angle of the abutment to the flow.

The K_1 factor is given in Table 9. There is a guide bank at the east abutment which helps align the flow and also moves the major scour to the tip of the guide bank and upstream of the actual bridge abutment. This guide bank will most likely perform better than a spill-through abutment; however, there is no specific guidance provided on the recommended K_1 factor, so K_1 was selected to be 0.55.

Table 9. Abutment shape coefficients (from Richardson and Davis 2001)

Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

The K_2 factor is given by the following equation (Richardson and Davis 2001):

$$K_2 = \left(\frac{\theta}{90}\right)^{0.13}$$

where θ = the orientation of the embankment angle.

For the abutment on the Bell Road Bridge, the embankment angle is equal to 90°, so the K_2 factor is equal to 1.0.

Using the HEC-RAS model for the Agua Fria River (WEST 2002a), the following hydraulic parameters were determined at the east abutment for the 100-year discharge of 37,500 cfs:

- $y_1 = 3.23$ ft,
- $Fr = 0.55$

Thus, the abutment scour can be calculated as:

$$y_s = (3.23)(4)(0.55)^{0.33}\frac{0.55}{0.55}(1.0) = 10.6 \text{ ft.}$$

Using a factor of safety of 30%, the local abutment scour was estimated to be 13.8 ft. There is limited guidance provided as to the distance the scour hole will extend in the downstream direction. However, there is information that indicates that for bridges with spill-through abutments and 100-year-flow top widths of greater than 300 feet, the maximum abutment scour occurs near the centerline of the bridge with the general pattern of the scour hole taking the shape of a parabola. As stated earlier, it is expected that the greatest scour depth will be upstream of the bridge near the tip of the guide bank and therefore abutment scour will not extend to the pipeline crossing. Therefore, local scour at the bridge was taken to be the worst-case scenario of the pier scour and abutment scour with the local scour component equal to the pier scour of 17.2 ft.

Long-Term Degradation

Long-term degradation can be evaluated by various methods including: 1) historical evidence and field data; 2) stable slope analysis; 3) sediment routing with a long-term simulation of actual historical flow series; and 4) sediment routing with a long-term simulation of continuous application of the ‘channel forming” discharge. For this study, long-term degradation was evaluated both by reviewing historical data and by sediment routing using a historical flow series. The HEC-6T computer model was used for the sediment transport modeling. The hydraulics, bed material gradations, inflowing sediment load, and historical flow series for the numerical modeling were obtained from a previous study conducted by WEST (2002b) and described later in this section. A stable slope analysis was not utilized since the location of a stable or “pivot” point could not be identified with any degree of certainty downstream of the pipe crossing.

Historical Data

Existing hydraulic models and topographic maps from four (4) different studies were used to map the bed profile of the Agua Fria River near Bell Road. These four (4) studies are summarized below:

1964 Topographic Maps

In 1964, topography of the Agua Fria River near Bell Road was generated as part of the FCDMC’s Agua Fria Contour and Topographic Survey and Floodplain Delineation – Gila River to Pinnacle Peak. The topography was mapped by the Aerial Mapping Company. Although the datum of this data could not be located, the vertical datum appears to be based on 1929 NGVD. This topography was used as the basis of a HEC-2 model developed by the U.S. Army Corps of Engineers (USACE) in 1975. The thalweg elevations from the USACE’s 1975 HEC-2 model were compared to the spot elevations shown in the 1964 topographic mapping. The spot elevations suggest that the HEC-2 model accurately depicts the bed elevations around Bell Road. A plot of the 1964 bed profile of the Agua Fria River around Bell Road is shown in Figure 8. In the 1975 HEC-2 model, Bell Road is located at River Station (RS) 19.59. In the later hydraulic models, the stationing changed slightly and Bell Road was located at RS 18.97. Thus, the stationing for the 1964 data was shifted so that Bell Road corresponded to RS 18.97.

1982/1983 Topographic Data

Over the years of 1982 to 1983, the USACE update the topography of the Agua Fria River from Jomax Road to Cactus Road. This topography was then used in the USACE’s 1983 HEC-2 model of the Agua Fria River. No information or topographic maps could be located for this data set. However, the vertical datum for the topography appears to be based on 1929 NGVD. Using the HEC-2 model, the plot of the 1982/1983 bed profile of the Agua Fria River can be seen in Figure 8. In the 1983 HEC-2 model, Bell Road is located at RS 18.893. The stationing for the 1982/1983 data was shifted so that Bell Road corresponded to RS 18.97 before the profile was plotted in Figure 8.

1989 Topographic Data

In 1989, topographic maps of the Agua Fria River were developed as part of a floodplain study by Jerry R. Jones and Associates (Jones 1989). The topographic mapping was complied by Cooper Aerial of Phoenix, Inc. The mapping scale is 1" = 400' with a 4-foot contour interval. The vertical control was based on 1929 NGVD. Once again, a HEC-2 model was developed for the Agua Fria River using the 1989 topographic data. The spot elevations on the topographic maps agreed well with the elevations reported in the HEC-2 model. A plot of the thalweg elevations of the Agua Fria River using the 1989 topographic data can be seen in Figure 8. Note that in the 1989 HEC-2 model, Bell Road is located at RS 18.92. The stationing for the 1989 data was shifted slightly so that Bell Road corresponded to RS 18.97.

2000 Topographic Data

In 2000, topographic maps of the Agua Fria River were generated as part of a floodplain study by WEST Consultants, Inc. (WEST 2002a). The digital mapping for this analysis was provided by Databased Terrain Mapping, Inc. The topographic work map produced for this study was at a scale of 1" = 200' with a 2-foot vertical contour interval based on the 1929 NGVD. A HEC-RAS model was generated from the 2000 topographic data using HEC-GeoRAS. Bell Road is located at RS 18.97 in this model. A plot of the thalweg elevations of the Agua Fria River near Bell Road for the 2000 topographic data is shown in Figure 8.

Figure 8 suggests that the Agua Fria River near Bell Road degraded by about 4 feet between 1964 and 1982. Since that time, it appears that the Agua Fria River has been very stable and there has not been a significant amount of change in bed elevation.

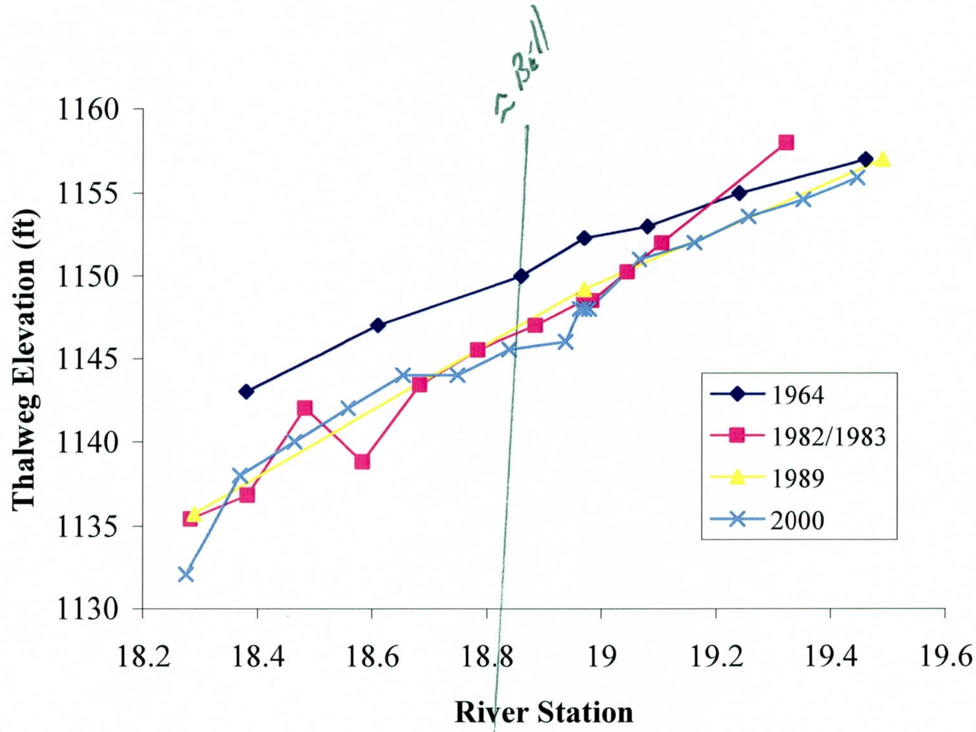

Figure 8. Historic longitudinal profiles of the Agua Fria River near Bell Road (RS 18.97)

Sediment Routing

Sediment transport modeling (HEC-6T) was performed for the purpose of evaluating the long-term degradation potential of the Agua Fria River in the vicinity of Bell Road. The HEC-RAS model used to develop the sediment transport model was obtained from a past study by WEST (2002b) for the FCDMC. Note that the HEC-RAS model used in the sedimentation study (WEST 2002b) is slightly different than the HEC-RAS model discussed earlier (WEST 2002a). The main difference between these two HEC-RAS model is the river stationing. The geometry of the sediment transport model extends from Cactus Road to Happy Valley Road. The average bed elevation changes due to a historical flow series between 1965 and 2000 were calculated. The flow series was developed for the sediment transport study conducted by WEST (2002b). The HEC-6T and HEC-RAS model used in the WEST (2002b) study are included on the attached CD.

The historical hydrologic series was utilized to study the long-term behavior of the channel in the vicinity of the Bell Road Bridge. The New Waddell Dam was completed in 1992. Since the recorded flows before 1992 are not representative of the current flow regime, the historic flow series (1965 – 2000) is considered to be a stricter test than the frequency flows (see Table 4), and most likely represents a longer-term system behavior.

The HEC-6T sediment transport model for this study was developed from the HEC-RAS and the HEC-6T models developed by WEST (2002b). The original study conducted by WEST used a soil gradation having a D_{50} of 4.69 mm, which was obtained from the Kimley-Horn and Associates, Inc. study (KHA 2001). Based on a field reconnaissance conducted by WEST personnel on April 18, 2005, it was felt that the bed material may not be that coarse in the vicinity of Bell Road and that the ASU (Carriaga et al. 1994) gradation data with a D_{50} of 1.48 mm would be appropriate. Yang's Stream Power function was the chosen transport function based on related project experience and because the function is suitable for most sand bed conditions of the Southwest. The movable boundary limits for the Agua Fria River were defined from the top of the left bank to the top of the right bank with exception in areas where the overbank area extends far enough to provide room for deposition. At these locations, the movable bed limits were moved onto the overbank areas. Erosion limits were defined between bank stations. Tailwater rating curves were developed for a range of discharges from 2,000 cfs to 40,000 cfs using the HEC-RAS model (WEST 2002b). The rating curves were used to establish the downstream water surface elevation.

The equilibrium bed material load at the upstream reach of the model was obtained from the WEST (2002b) study. This was calculated for a range of constant discharges between 100 cfs and 60,000 cfs to develop a sediment-water discharge rating curve. The sediment re-circulation option in HEC-6T was used to accomplish this task using the upstream cross sections from RS 22.15 to 24.14 (RS 22.36 to 24.35 in the WEST (2002a) model). The re-circulation model simulation was initiated with zero sediment entering the model. The simulation was repeated for several events representing a single discharge. The calculated sediment concentrations for the first event were used as the sediment inflow for the second event and so forth. This procedure was continued for each constant discharge until the sediment concentration converged. The fractions for the different types of materials from very coarse to very fine sand were collected for each constant discharge as the inflowing sediment load. The sum of the fractions for each discharge totaled one (1).

Based on the channel configuration and associated HEC-RAS model, sediment transport analyses were conducted for a single long-term hydrologic event and a combination of the long-term hydrologic series (1965 – 2000) and the 100-year hydrograph.

The relative stability of the Agua Fria River in the vicinity of Bell Road Bridge is shown in Figure 9. This figure compares the initial average bed elevation with the average bed elevation following the routing of a single long-term hydrologic series and the combination of the long-term series and the 100-year event hydrograph. Figure 10 shows the average bed elevation change. The results of the sediment routing simulation analysis indicate a long-term degradation of 4.2 feet in the vicinity of the Bell Road crossing.

Long-Term Degradation Results

The historical topographic data analysis and the sediment routing analysis provide similar long-term degradation results. The historical topographic analysis indicates a long-term degradation of approximately 4 feet. This is based on data for the period between 1965 and 2000 with the 4 feet change occurring in the period between 1965 and 1982. During this period, it is reported that sand and gravel mining operations were occurring within the channel upstream of Grand Avenue (WEST 2002b). Mining operations reportedly expanded in the channel area at this location until 1977, when the degree of channel alternation reached a maximum. Sand and gravel mining also occurred within the channel just downstream of Grand Avenue during this time period. This extraction from the channel contributed to the lowering of the bed elevation upstream of Grand Avenue. This is the same period during which large flow events occurred in the Agua Fria River, which included flows of 58,400 cfs (12/19/78), 53,280 cfs (2/15/80), and 45,910 cfs (12/20/80). The historical data includes the impact associated with in-stream sand and gravel mining operations. With the current awareness and regulation of such activities, it is not anticipated the long-term impact associated with in-stream mining or other man-induced changes will be a major factor for future conditions.

The sediment routing analysis results in a long-term degradation of 4.2 feet in the vicinity of the Bell Road crossing of the Agua Fria River. This analysis is based on a historical flow series between 1965 and 2000. This historical series includes flows of a magnitude that would not be expected under current conditions. With the completion of the New Waddell Dam in 1992, the 100-year peak flow is 37,500 cfs and the 10-year peak flow is 11,000 cfs. This compares to a 100-year flow of 115,000 cfs and a 10-year flow of 37,000 cfs prior to the completion of New Waddell Dam.

Using the 4.2 feet and applying a factor of safety of 1.3 results in a long-term degradation of 5.5 feet.

Agua Fria River: Cactus Road to Happy Valley Road
Average Bed Elevations for Long Term Hydrologic Series

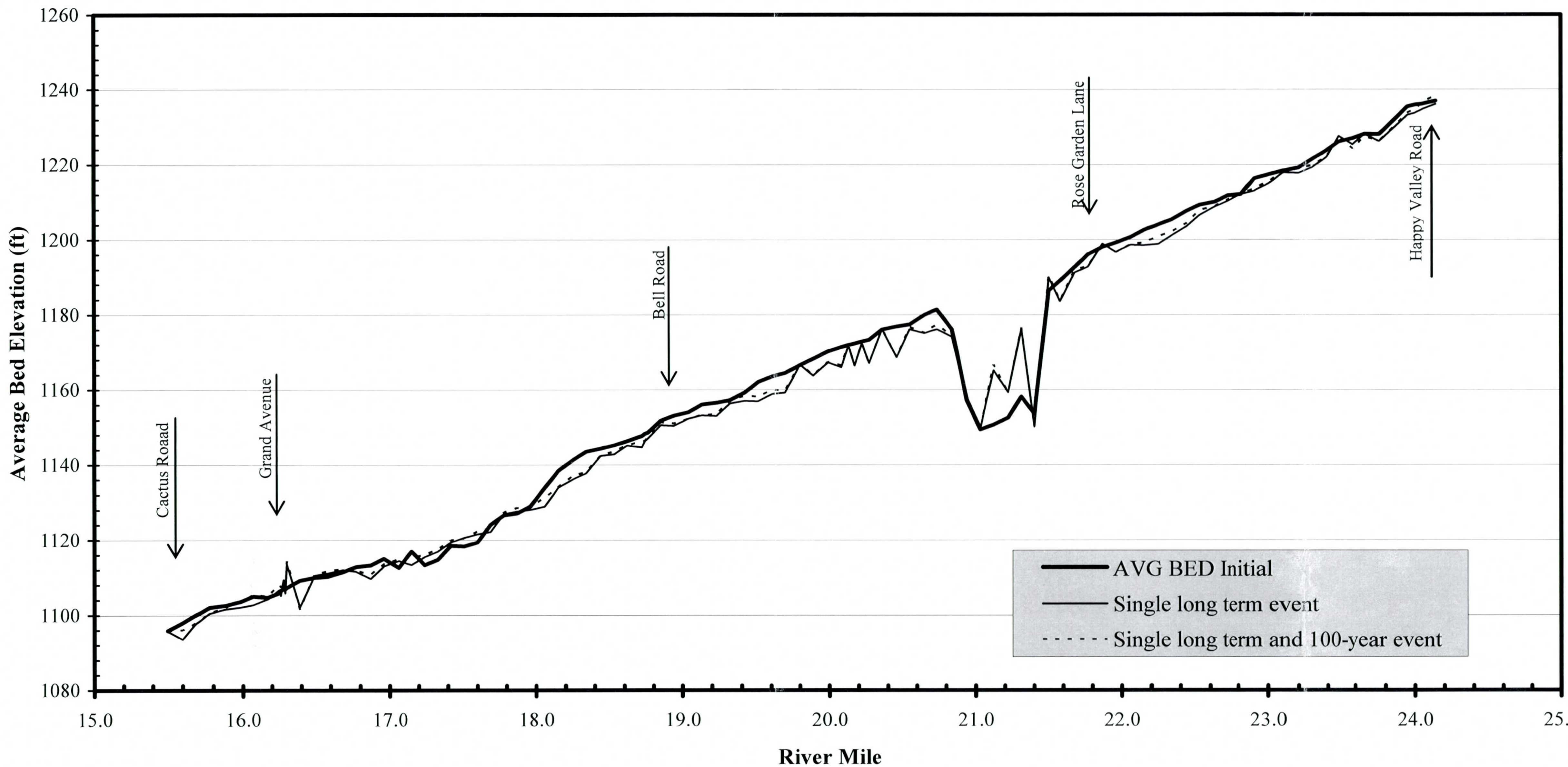

Figure 9. Average bed elevation profile for the Agua Fria River

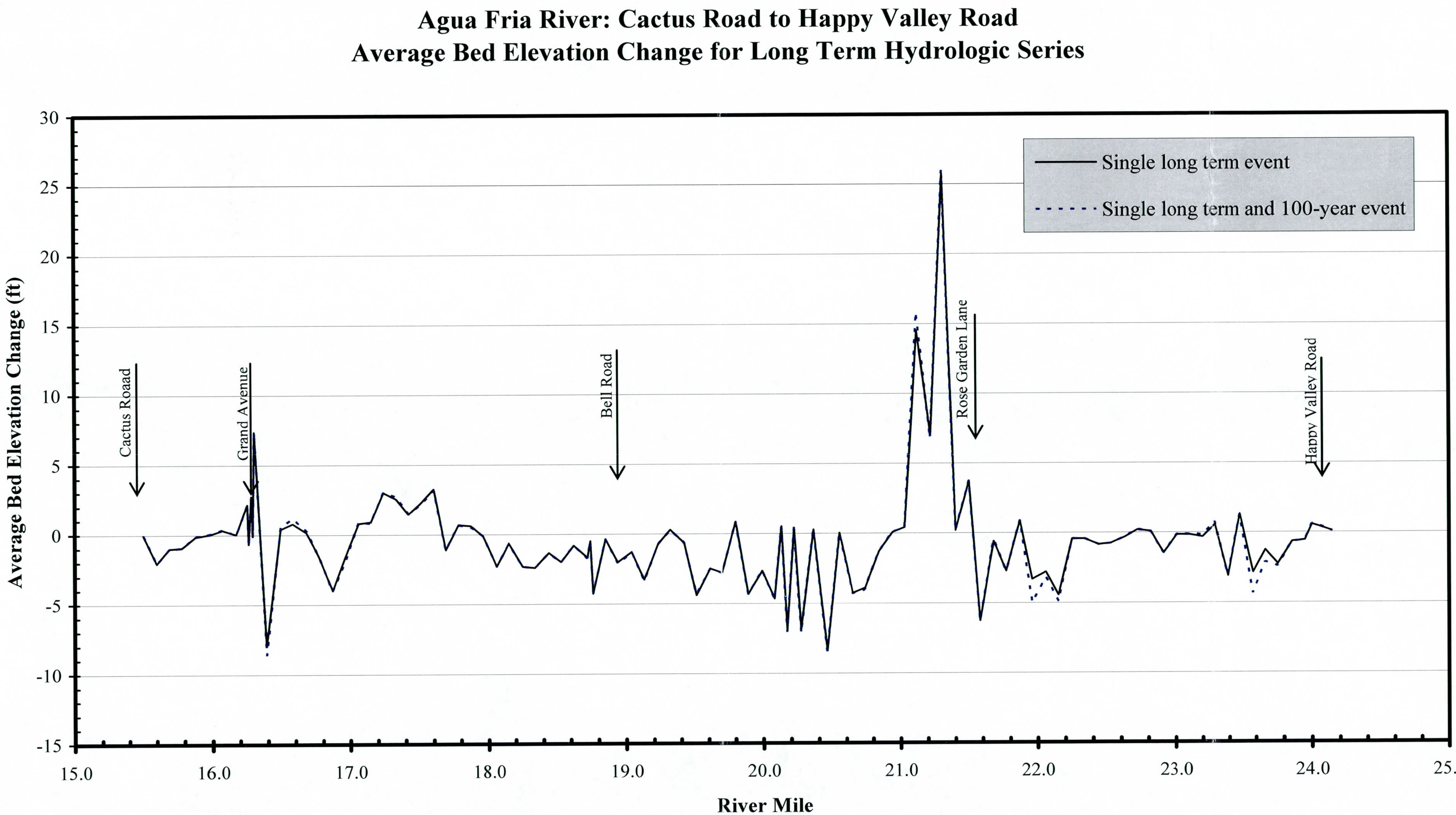

Figure 10. Average bed elevation change for the Agua Fria River

Bedform Scour

For sand bed channels, natural or manmade, it is necessary to estimate the height of the bedforms moving through the channel. Dunes may form in lower regime flow with antidunes forming in transitional or upper regime flow. The flow in the Agua Fria River for a 100-year event would be considered to be in the lower regime. The scour depth due to dunes may range from 0.1 to 0.5 times the maximum depth of flow based on studies conducted by Simons and Richardson (1960). Other studies by Yalin (1964) suggest a scour depth of one-sixth of the average flow depth. From the HEC-RAS model (WEST 2002a), the maximum flow depth downstream from the Bell Road Bridge is 8.1 feet. Therefore, using Simons and Richardson's (1960) method, bed form scour may be in the range of 0.8 feet to 4.1 feet. Taking the average of these values yields a bedform scour depth of 2.4 feet. Applying a factor of safety of 1.3 results in a bedform scour depth of 3.1 feet.

Bend Scour

Bends associated with meandering channels will induce transverse or "secondary" current which will scour sediment from the outside of a bend and cause it to be deposited along the inside of the bend. For the Agua Fria River at Bell Road, the Zeller bend scour equation (SLA 1985) was used to estimate the bend scour:

$$y_{bs} = \left[0.0685\frac{y_{\max}V^{0.8}}{y_h^{0.4}S_e^{0.3}}\right]\left[2.1\left[\frac{(\sin(\alpha/2))^2}{\cos(\alpha)}\right]^{0.2} - 1\right]$$

where: y_{bs} = depth of bend scour (ft),

y_{max} = maximum depth of upstream flow (ft),

V = mean velocity of upstream flow (ft/s),

y_h = hydraulic depth of upstream flow (ft),

S_e = upstream energy slope (ft/ft), and

α = angle formed by projection of channel centerline from point of curvature to a point which meets a line tangent to the outer bank of the channel (degrees).

From aerial photographs taken in 2000, a bend in the Agua Fria River appears to occur right on the Bell Road Bridge. The west end of the bridge is on the outside of the curve and the east end in on the inside. From the aerial photograph, the bend angle, α, was estimated to be approximately 20°. The other hydraulic information needed to calculate the bend scour was determined from the HEC-RAS model (WEST 2002a) at the 100-year discharge (i.e., 37,500 cfs) at RS 18.978, which is located just upstream from the Bell Road Bridge. The following hydraulic variables were determined:

- $y_{max} = 8.29$ ft,
- $V = 5.31$ ft/s
- $y_h = 6.55$ ft, and
- $S_e = 0.001282$.

Substituting these values in the Zeller bend scour equation yields:

$$y_{bs} = \left[0.0685 \frac{(8.29)(5.31)^{0.8}}{(6.55)^{0.4}(0.001282)^{0.3}}\right]\left[2.1\left[\frac{(\sin(10))^2}{\cos(20)}\right]^{0.2} - 1\right] = 0.4 \text{ ft.}$$

Using a factor of safety of 30%, the bend scour, y_{bs}, was estimated to be 0.5 ft.

Sand and Gravel Mining

There is an existing sand and gravel mining operation approximately 1,500 feet downstream of the Bell Road Bridge (see Figure 11). The operation is not located in the main channel; it is located on the left bank of the Agua Fria River and there is a berm separating the mining operation from the main channel of the Agua Fria. It is unlikely that water will breach the top of the berm and fill the pit. However, if this occurs, a headcut may form as water accelerates over the steep face of the pit. A headcut profile will propagate upstream as the river attempts to re-establish an equilibrium slope.

Figure 11. Aerial photograph of sand and gravel operation on the left overbank of the Agua Fria River downstream from the Bell Road Bridge

Cotton and Ottozawa-Chatupron (1990) outline a method to estimate the headcut profile that develops from an in-stream gravel mining operation. The procedure presented by Cotton and Ottozawa-Chatupron (1990) was developed for the Arizona Department of Transportation for the estimation of short-term longitudinal channel response due to in-stream mining. The following parameters are needed to estimate the headcut due to an in-stream gravel mine:

- Shape of the gravel pit (i.e., width, length, depth)
- Discretized design flow hydrograph,
- Channel bed gradient, and
- Excavation pit fill time.

The physical dimensions of the pit were obtained by examining the 2000 aerial photographs and topography. It was estimated from the aerial photographs (see Figure 11) and topography that the pit was 800 feet wide (W_p), 4,000 feet long (L_p), and 40 feet deep (y_p). This implies that the total volume of the pit (V_f) is 128,000,000 ft^3. Using the HEC-RAS model (WEST 2002a), the channel slope (S_0) was estimated to be 0.00254. The USACE (1995) presents a 100-year flood hydrograph for the Agua Fria River at Bell Road. This hydrograph was discretized at 1-hour intervals and is shown in Figure 12. Using the volume (V_f) and the discretized hydrograph, the time it would take to fill the pit (T_f) can be calculated. It was assumed that the fill time (T_f) was equal to the time that it took for the hydrograph to deliver 128,000,000 ft^3 of water (V_f). The 100-year hydrograph shown in Figure 12 would take approximately 3.5 hours (T_f = 12,600 seconds) to deliver 128,000,000 ft^3 of water.

For each discretized block of the hydrograph, the following calculations are performed. First, the inflow channel width (W_c) is calculated based on a regime width equation which was developed using the concept of minimum stream power at the excavation boundaries (Cotton and Ottozawa-Chatupron 1990):

$$W_c = 2.60Q^{0.43}$$

where: Q = flow rate (cfs).

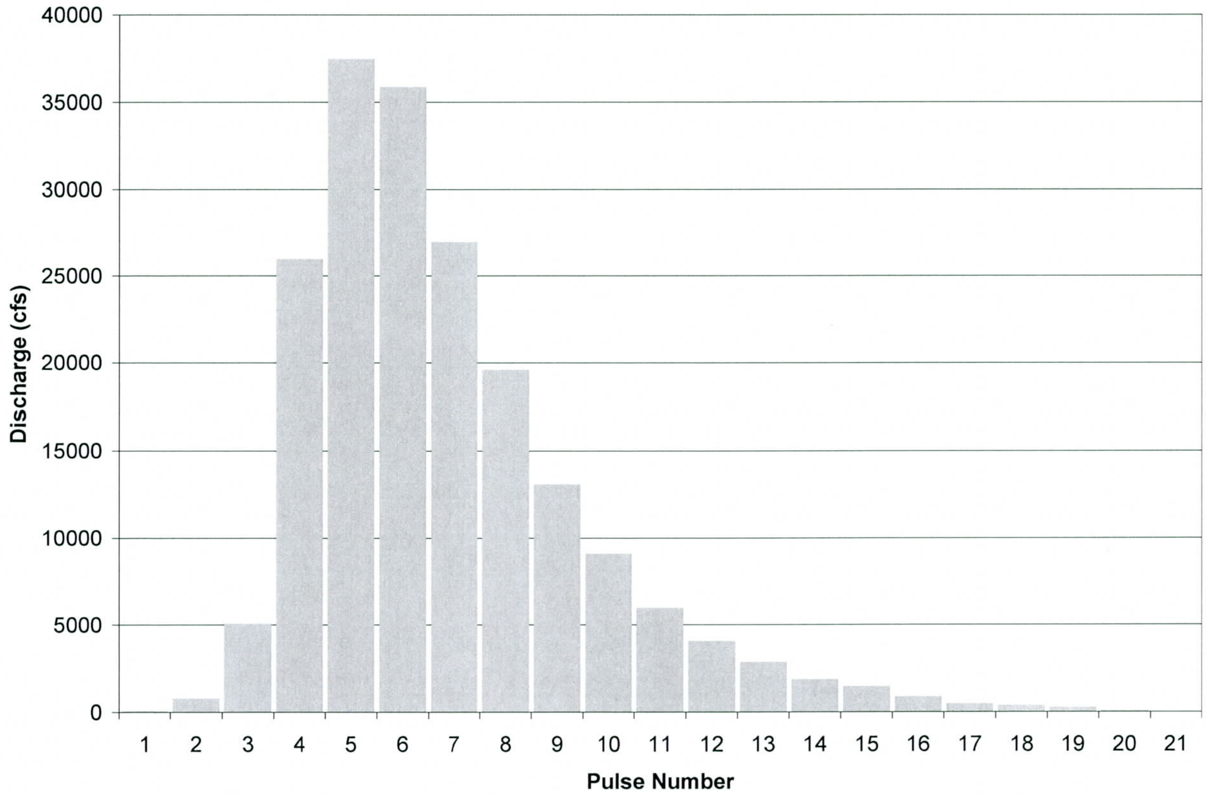

Figure 12. Discretized 100-year flood hydrograph for the Agua Fria River at Bell Road

Next the dimensionless width (W^*) is calculated using the following equation:

$$W^* = \frac{W_p}{W_c}$$

Then, the unit flow (q_c) rate based on W_c is calculated:

$$q_c = \frac{Q}{W_c}$$

A dimensionless time (T^*) also needs to be calculated:

$$T^* = \min\left(\frac{t}{T_f}, 1.0\right)$$

where: t = time (sec). The dimensionless time (T^*) must be less than or equal to 1.0.

All of these parameters can be used to calculate the headcut depth at the lip of the pit (y_s) using the following equation:

$$y_s = 1.24\left(W^*\right)^{-2.46 q_c^{-0.451}} y_p \left(T^*\right)^{0.648}$$

The calculated headcut depth at the lip of the gravel pit must be less than y_{smax}, which is calculated from the following equation:

$$y_{s\max} = \min\left[\left(\frac{y_p}{2}\right), \left(0.120\left(W^*\right)^{0..672} y_p q_c^{0.286\left(W^*\right)^{-0.350}}\right)\right]$$

The length of the headcut (l_s) is estimated from the following equation:

$$l_s = 0.219 L_p q_c^{0.262}\left(W^*\right)^{-0.624}\left(T^*\right)^{0.216 q_c^{0.155}}$$

The calculations used to estimate the headcut depth at the brink of the pit (y_s) as well as the length the headcut travels upstream (l_s) are shown in Table 10. The maximum headcut depth is 20 feet (i.e., the limiting value of one-half of y_p), and the headcut extends upstream approximately 1,554 feet. Using linear extrapolation, it was estimate that the headcut will be approximately 0.7 feet deep at the bridge, which is about 1,500 feet upstream from the pit.

Headcuts generally contribute to the long-term degradation of a system. The headcut depth at the pipeline crossing is less than the estimated depth contributed to long-term degradation. Since it is extremely unlikely that a headcut of this depth would occur in addition to a long-term degradation of 5.5 feet, it is recommended that only long-term degradation be included in predicting the total scour depth.

Table 10. Calculations for headcut profile

Time (hr)	Time (sec)	Q (cfs)	W_c (ft)	q_c (cfs/ft)	W^*	T^*	y_s (ft)	y_{smax} (ft)	l_s (ft)
0	0	100	18.8	5.3	42.5	0.00	0.00	20	0
1	3,600	800	46.1	17.4	17.4	0.29	3.17	20	205
2	7,200	5,100	102.1	49.9	7.8	0.57	14.49	20	541
3	10,800	26,000	205.8	126.3	3.9	0.86	20.00	20	1,243
3.75	13,500	37,500	240.9	155.7	3.3	1.00	20.00	20	1,554
4	14,400	35,900	236.4	151.9	3.4	1.00	20.00	20	1,526
5	18,000	27,000	209.2	129.1	3.8	1.00	20.00	20	1,355
6	21,600	19,600	182.2	107.6	4.4	1.00	20.00	20	1,185
7	25,200	13,100	153.2	85.5	5.2	1.00	20.00	20	1,002
8	28,800	9,100	131.0	69.5	6.1	1.00	20.00	20	860
9	32,400	6,000	109.5	54.8	7.3	1.00	20.00	20	723
10	36,000	4,100	93.0	44.1	8.6	1.00	18.99	20	617
11	39,600	2,900	80.1	36.2	10.0	1.00	16.15	20	534
12	43,200	1,900	66.8	28.4	12.0	1.00	12.87	20	447
13	46,800	1,500	60.4	24.9	13.3	1.00	11.15	20	405
14	50,400	900	48.5	18.6	16.5	1.00	7.82	20	327
15	54,000	500	37.6	13.3	21.3	1.00	4.77	20	256
16	57,600	400	34.2	11.7	23.4	1.00	3.84	20	233
17	61,200	300	30.2	9.9	26.5	1.00	2.83	20	207
18	64,800	100	18.8	5.3	42.5	1.00	0.64	20	131
19	68,400	0	0.0						

RECOMMENDATIONS

The 16-inch gas line is proposed to cross the Agua Fria River approximately 50 feet downstream from the downstream face of the bridge piers. The total scour at this location is the sum of the long-term degradation, general or contraction scour, bedform, and bend scour plus contributions of local pier scour as the result of the zone of influence. The bedform scour would not apply at the bridge piers since the equation used to compute local pier scour also takes into consideration bed form scour. Therefore, the total pier scour is estimated to be 5.5 feet + 0.7 feet + 0.5 feet + 17.2 feet = 23.9 feet.

HEC-18 (Richardson and Davis 2001) indicates the top width of the scour hole from one side of the pier can vary from 1.07 to 2.8 times the scour depth. The long-term degradation, general scour, and bend scour are considered to be uniform throughout the reach being evaluated (upstream of Bell Road Bridge to downstream of gas pipe crossing), and the top width calculation is based on the local pier scour depth only. This would result in a scour hole top width of 18.4 to 48.2 feet at the Bell Road Bridge. Therefore, even with the larger value of 48.2 feet, the local pier scour would not contribute to the total scour at the gas pipe crossing (see Figure 13).

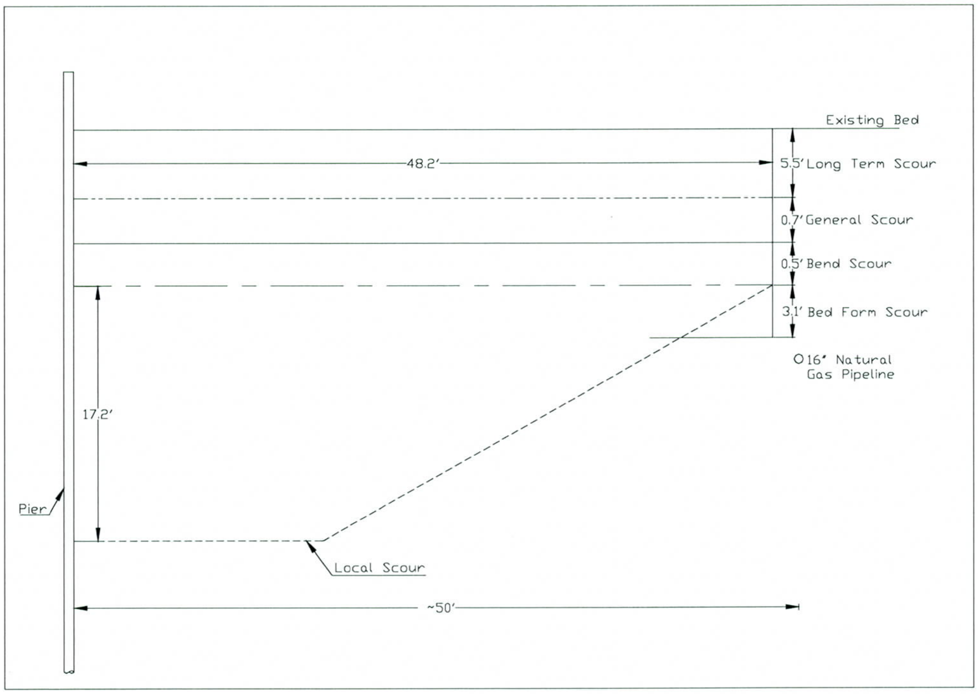

Figure 13. Local scour at piers (note that drawing is not to scale)

The predicted total scour depth (long-term + general scour + bedform scour + bend scour) at the gas pipeline crossing (50 feet downstream of the downstream face of the bridge piers) is 5.5 feet

+ 0.7 feet + 3.1 feet + 0.5 feet = 9.8 feet. This depth should be measured below the thalweg or lowest point in the Agua Fria River at the crossing location to establish the maximum elevation at which to place the pipe. Since the thalweg may move laterally during a flow event, this elevation should be applied across the entire channel cross section.

Based on the 2000 topography, the lowest elevation 50 feet downstream from the Bell Road Bridge is approximately 1,148.0 feet. Thus, the top of the pipeline cross must be placed at an elevation no higher than 1,138.2 feet. It is recommended that the topography of the proposed pipeline crossing be verified to assure the thalweg elevation is not lower than that determined from the 2000 topography.

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