US-93 INTERIM WICKENBURG BYPASS BRIDGE HYDRAULICS AND BANK PROTECTION REPORT

Wickenburg, Arizona

FINAL

WEST CONSULTANTS, INC.

Water • Environmental • Sedimentation • Technology

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

US-93 INTERIM WICKENBURG BYPASS

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Wickenburg, Arizona

FINAL

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Arizona Department of Transportation

Phoenix, AZ

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1. INTRODUCTION

1.1. Authorization

This report was prepared by WEST Consultants, Inc. (WEST) for the Arizona Department of Transportation (ADOT) under subcontract to Jacobs Engineering Group, Inc. (Jacobs Engineering). The report contains results of the detailed hydraulic analyses for the proposed bridge crossing at the Hassayampa River, the proposed bridge crossing at Sols Wash, and the proposed US-93 interim road embankment. This report was authorized for use in conjunction with the design of the two bridge crossings developed by ADOT and Jacobs Engineering and the road embankment designed by Jacobs Engineering.

WEST provided technical assistance on the following aspects of the project: (a) hydrologic analyses, (b) hydraulic design of the new bridges, (c) the impact of the project on the floodplain, (d) determination of freeboard for the new road embankment along the Hassayampa River, (e) sediment transport and scour analyses, and (f) determination of required top of bank, toe down, and general design requirements for bank protection along the new road embankment.

1.2. Purpose

The main purpose of this report is to provide a detailed description of the hydrologic, hydraulic, and scour analyses performed as part of the design of the new bridges and the embankment. The report also presents recommendations for the low chord elevation for the new bridges as well as the toe down elevations for the proposed bank protection near the bridges and along the proposed embankment.

1.3. Bridge Hydraulic Design Criteria

The design criteria for the bridge hydraulics consist of the following: (a) sufficient conveyance for the 100-year design flood, (b) sufficient conveyance for the 50-year Design flood with 3 feet of freeboard, (c) the bridge piers, abutments, and other structures should not be impacted by scour subject to adverse conditions resulting from floods ranging from the 50-year to the superflood event, (d) adequate bank protection measures to protect against flows ranging up to the 100-year design flood.

1.4. Embankment Hydraulic Design Criteria

Most, but not all, of the new US-93 road embankment is designed to be a levee which would protect the town of Wickenburg, AZ on the west side from flooding in the Hassayampa River. The primary implication for the embankments to serve as levees is that they must have at least 3 feet of freeboard above the 100-year flood (CFR Title 44, Section 65.10(b)(1)(i)), with an additional foot required in certain circumstances. Furthermore, Federal regulations require that the banks be protected:

> *"Engineering analyses must be submitted that demonstrate that no appreciable erosion of the levee embankment can be expected during the base flood, as a*

result of either currents or waves, and that anticipated erosion will not result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability. The factors to be addressed in such analyses include, but are not limited to: Expected flow velocities (especially in constricted areas); expected wind and wave action; ice loading; impact of debris; slope protection techniques; duration of flooding at various stages and velocities; embankment and foundation materials; levee alignment, bends, and transitions; and levee side slopes." (CFR Title 44, Section 65.10(b)(3))

Within 100 feet of structures (such as bridges) riverward of the levee, 4 feet of freeboard is required (CFR Title 44, Section 65.10(b)(1)(i)).

The bank protection selected for this project is soil cement.

1.5. Project Location and Description

The project reaches are located along the Hassayampa River and Sols Wash in Wickenburg, Arizona. The project focuses on the proposed US-93 Wickenburg Bypass at the Hassayampa River, the proposed US-60 Bidge over the Hassayampa River (which is part of the bypass), the proposed US-93 Bridge over Sols Wash, and the road embankment which will function as a levee system. The Hassayampa River flows north to south in the project reach. The hydraulic study limits along the Hassayampa River extend from about one-quarter (1/4) mile downstream to approximately one and one-half (1-1/2) miles upstream of the current US-60 bridge crossings. Sols Wash flows west to east in the project reach. The study limits along Sols Wash extend from the confluence with the Hassayampa River to upstream of North Tegner Street. The project is within the Hassayampa River / Sols Wash 100-year regulatory floodplain with nearly 40 percent of the roadway length encroaching into the FEMA defined floodway. A vicinity map showing the location of the existing and proposed bridges, as well as the route of the proposed embankment, is shown in Figure 1.1.

The upper reaches of the Hassayampa River are characterized by steep, high-energy channels that have gravel beds. Farther downstream, the Hassayampa River enters the Sonoran Desert where the channel is wider, has more sand, and is less steep (Capesius and Lehman, 2001). This sandy and wider condition is the condition of the channel in the project reach.

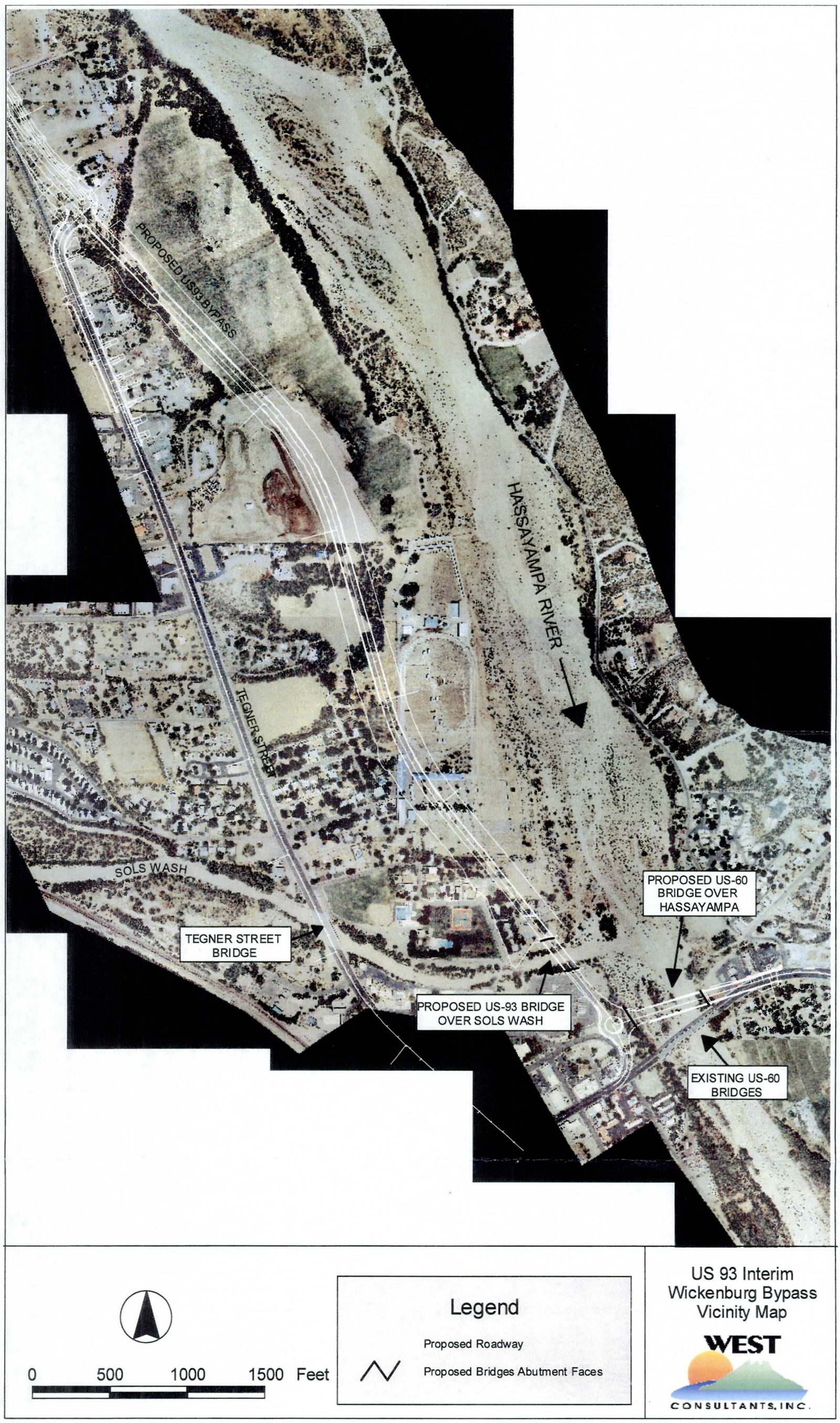

Figure 1.1. Vicinity map

2. HYDROLOGY

2.1. Flood Frequency Analysis

Hassayampa River and Sols Wash flood discharges for different event frequencies were needed for the following purposes:

- To calculate the required levee height for the new US-93 road embankment (using 100-year flood elevation of the Hassayampa River plus 3 feet).
- To calculate the 50-year water surface elevation to meet the Arizona Department of Transportation requirement that the freeboard at bridges exceeds the 50-year water surface by at least 3 feet. (ADOT Roadway Design Guidelines, 610.2C).
- For use in the sediment transport model, to simulate the effect of long-term changes in the river.

2.2. Hassayampa River

2.2.1. Hassayampa River: Gage History

The drainage area of the Hassayampa River at the US-60 Bridge is 711 square miles (FEMA, 2001). The nearest upstream tributaries with basins of significant size are Sols Wash, located about 750 feet upstream of the US-60 crossing over the Hassayampa River, and Martinez Wash, located about 2 ½ miles upstream of the US-60 crossing. Both of these washes are on the west side of the Hassayampa River. Sols Wash at the confluence with the Hassayampa River has a drainage area of 147.2 square miles (FEMA, 2001), while Martinez Wash has a drainage area of 103 square miles (FEMA Yavapai County FIS, Table 7).

An old USGS gaging station was previously present at Box Canyon (also know as Box Damsite) which is 8 miles upstream of the US-60 bridge near Wickenburg (09515500). The drainage area at this old Box Canyon gage is 417 square miles. In addition to the contributing area from Martinez Wash and Sols Wash, minor washes and local drainage contribute 44 square miles between the Box Canyon gage and the US-60. The peak flows at Box Canyon were recorded in 1925, 1927, 1937-38, and 1946-82. The flow of record for this gage is 58,000 cfs on 9/5/1970. Another nearby gage is the Morristown USGS gage (09516500) which is about 8 miles downstream of the existing US-60 bridge. Peaks were recorded there from 1939-47, 1964-81, and 1983-2003. The peak of record for the Morristown gage is 47,500 cfs, on 9/5/1970.

In recent years, the Flood Control District of Maricopa County (FCDMC) has maintained pressure transducer gages on the Hassayampa River at Box Canyon, at the US-60, and near Morristown. These gages record stage levels every few minutes during flood events; these stages are converted to flows using a rating curve and are available for download at the FCDMC's website. The Box Canyon gage has been in place since October, 1991. The flow of record for that gage is 15,451 cfs on 2/20/1993. The US-60 gage has been in place since March,

1994. The flow of record for that gage is 15,376 cfs on 9/26/1997. The Morristown gage has been in place since May, 1996. The flow of record for that gage is 9,095 cfs on 9/26/1997. A large difference in the peak flow for the 9/26/1997 event can be observed between the three gages: US-60 (15,376 cfs), Morristown gage (9,095 cfs), Box Canyon (3,549 cfs).

The largest flood for which a detailed hydrograph could be obtained (Mr. Burt Duet, USGS Field Office, Tempe, Arizona, personal communication, April 26, 2004) was the event of January, 1993 at the Morristown gage. This hydrograph was available in 15-minute increments, and this flood had a peak of 26,300 cfs.

2.2.2. Hassayampa River: Flood Frequencies

The current regulatory (FEMA) 100-year flood frequency for the Hassayampa River at Wickenburg is 71,000 cfs, with a drainage area of 711 square miles (FEMA July 2001). The 10-year, 50-year, and 500-year floods are not published in the current flood insurance study (FIS). However, these floods appear in a previous FIS (FEMA 1977, revised 1983). The published values in that study are provided in Table 2-1.

Table 2-1. Hassayampa River floods from July 1977 FEMA Wickenburg FIS (revised 1983)

Location	Drainage Area (sq. miles)	10-year flood (cfs)	50-year flood (cfs)	100-year flood (cfs)	500-year flood (cfs)
0.32 miles (approximately) upstream from corporate limits	564	14,500	47,000	71,000	185,000
0.14 miles (approximately) downstream from corporate limits	671	14,000	46,000	70,000	184,000

There is a discrepancy in the drainage areas shown in the July 1977 FIS (revised 1983) study versus what is shown on the current FIS. The current FIS reports the drainage area of the Hassayampa River at Wickenburg (downstream of Sols Wash) to be 711 square miles, which was verified to be approximately correct (within 1 square mile) by WEST based on a hydrologic unit code map and a stream network map. The 671 square miles shown in the 1977 FIS, and reproduced in Table 2-1 appears to be an error. Sols Wash is shown as having a drainage area of 145 square miles in the same study (it has 147 square miles according to the latest study).

Cella-Barr Associates, Inc.'s (Cella-Barr) 1988 analysis of the 100-year flood for the Hassayampa River, which is largely based on flood frequency analysis, resulted in 100-year discharges somewhat lower than the 71,000 cfs reported in the FIS (Cella-Barr, 1988).

In the current flood insurance study, 71,000 cfs is used as the 100-year flood both upstream and downstream of Sols Wash. In the 1977 study (revised in 1983), 71,000 cfs is reduced to 70,000 cfs at some point downstream of Sols Wash. This flow reduction does not appear in the most recent (July 2001) FIS, and it was not adopted for this report. In the 1977 study, the 50-year flood in the reach upstream of Sols Wash is 47,000 cfs.

A flood frequency analysis was performed for the Morristown gage, located about 7 miles downstream of the US-60 bridge and the Box Canyon gage upstream. The software program HEC-FFA was used (U.S. Army Corps of Engineers, 1992) and the results are summarized in Table 2-2.

Table 2-2. HEC-FFA analysis

Gage	Number of events	Drainage Area (square miles)	10-year flood (cfs)	50-year flood (cfs)	100-year flood (cfs)	500-year flood (cfs)	Comments
Hassayampa River at Morristown	47	796	19,400	46,200	60,800	141,000	Expected probability. Using regional skew of –0.1.
Hassayampa River at Box Canyon	38	417	10,000	47,100	67,200	100,000	Expected probability. Using regional skew of –0.1. Three events are historic (1925, 1927, 1937).

The flood frequency analysis for the bounding gages confirms that the 50-year flood of 47,000 cfs from the 1977 FIS is reasonable. The 47,000 cfs value was adopted for the 50-year flood, which established the freeboard requirement for the new US-60 bridge over the Hassayampa River. The 100-year flood of 71,000 cfs from the FEMA (2001) study is slightly higher than our HEC-FFA analysis. Note that the 10-year flood from the 1977 FIS, at 14,500 cfs, is about halfway between the HEC-FFA values calculated for the 10-year flood at Morristown and at Box Canyon.

2.2.3. Hassayampa River: Superflood

For the superflood used by ADOT, the regulations specify that "*bridge foundations shall be checked to ensure that they will not fail due to scour resulting from the occurrence of flood in magnitude between the design event and a superflood on the order of a 500-year flood. A flood 1.7 times the magnitude of the 100-year flood may be used if the 500-year flood is not known.*" (ADOT Roadway Design Guidelines, p. 600-23, 1996). Although a 500-year flood of 185,000 cfs is presented in the July 1977 FEMA Study (revised 1983), that flood event is not present in FEMA studies published more recently. Furthermore, the 185,000 cfs value is considerably greater than the 500-year floods from the flood frequency analysis for Box Canyon (100,000 cfs) and Morristown (141,000 cfs), as summarized in Table 2-2.

In light of the absence of the 500-year flood from recent FEMA studies, and the discrepancy between the gage analysis and the 185,000 cfs quoted in the 1977 FEMA study, the 500-year flood was assumed to be "not known." Therefore, 1.7 times the magnitude of the 100-year flood was used for the superflood. This works out to 1.7 X 71,000 cfs = 120,700 cfs, which was adopted as the superflood.

2.2.4. Hassayampa River: Adopted Flood Discharges

The following table summarizes the adopted discharges, their origin, and the relevance of the flood to this project:

Table 2-3. Adopted flood discharges for Hassayampa River

	10-year	50-year	100-year	Superflood
Discharge (cfs)	14,500	47,000	71,000	120,700
Source	1977 FIS	1977 FIS	2001 FEMA FIS, from Cella-Barr Study (Cella-Barr, 1988)	1.7 times 100-year; see section 2.2.3
Relevance	Establishing backwater conditions for Sols Wash.	ADOT freeboard criterion and bridge scour analysis, backwater condition for Sols Wash.	FEMA floodplain elevations, bank protection freeboard requirements, bank protection toe scour analysis, backwater condition for Sols Wash.	Bridge scour analysis.

2.3. Sols Wash

2.3.1. Sols Wash: Gage History

The nearest gage on Sols Wash is at Sols Wash near Matthie, which has a drainage area of 121.4 square miles (compared to 147 square miles at the confluence with the Hassayampa River). The FCDMC maintains that gage, which records flood events. The gage has been operating since August 4, 1995. The highest flow measured is 10,792 cfs which was recorded on 10/21/2001.

2.3.2. Sols Wash: Flood Frequencies

The FEMA published flood values for Sols Wash are: 10-year: 7,019 cfs, 50-year: 12,453 cfs, 100-year: 15,045 cfs, and 500-year: 20,836 cfs (FEMA, 2001). These values come from a TR-20 hydrologic model of the Sols Wash watershed by Cella-Barr Associates (Cella-Barr Associates, November 1985).

An internal memo obtained from the Corps of Engineers (Corps of Engineers, personal communication May 1992), gives discharges calculated by the Corps for the Flood Insurance Study of 1975 as 10-year: 4,000 cfs, 50-year: 16,500 cfs, 100-year: 24,000 cfs and 500-year: 59,000 cfs. These discharges are also referenced in a table in the Cella-Barr report (Cella-Barr Associates, November 1985).

The Cella-Barr report also refers to discharges from two other previous studies: a 1981 study by PRC Toups ("Hydrology Report, Sols Wash") and a 1974 study by the Soil Conservation Service ("Watershed Work Plan – Wickenburg Watershed, Maricopa and Yavapai Counties, Arizona"). The discharges from both of these studies are similar to those obtained by the 1985 Cella-Barr study.

For the purpose of this report, the current FEMA Flood Insurance Study Discharges (originally derived in the 1985 Cella-Barr study) were adopted.

2.3.3. Sols Wash: Summary of Adopted Discharges

The following table summarizes the adopted flood discharges, their origin, and their relevance for this project:

Table 2-4. Adopted flood discharges for Sols Wash

	10-year	50-year	100-year	Superflood
Discharge, (cfs)	7,019	12,453	15,045	20,836
Source	FEMA (2001), from Cella-Barr Study (Cella-Barr, 1985)	FEMA (2001), from Cella-Barr Study (Cella-Barr, 1985)	FEMA (2001), from Cella-Barr Study (Cella-Barr, 1985)	FEMA (2001), from Cella-Barr Study (Cella-Barr, 1985)
Relevance	ADOT freeboard requirements for Sols Wash Bridge (with 50-year flood in Hassayampa as backwater).	ADOT freeboard requirement for Sols Wash Bridge (with 10-year flood in Hassayampa as backwater), scour analysis of Sols Wash Bridge.	FEMA floodplain elevations and delineation, scour hole analysis downstream of Sols Wash Bridge.	Scour analysis of Sols Wash Bridge.

3. QUALITATIVE GEOMORPHIC ANALYSIS OF PROJECT REACHES

Aerial photographs, topographic maps, and ADOT bridge inspection reports were used to analyze historical trends in the Hassayampa River and Sols Wash. The most useful of the historical data sources was a set of topographic workmaps based on 1971 topography (FCDMC, 1971).

3.1. Analysis of Topographic Maps and Elevation Data

The aerial topographic survey for this project was flown on June 24, 2003. The limits of the topography are from about 2,000 feet south of the existing US-60 bridges over the Hassayampa River, extending north to almost 2 miles upstream of the US-60 bridges. The 2003 aerial topography also covers Sols Wash from the confluence with the Hassayampa River until just upstream of the Tegner Street bridge. Additional topography for an area farther upstream on Sols Wash used for the 2-D modeling discussed in Appendix B was obtained from JE Fuller; that topography was used for their Goldmine Village CLOMR (JE Fuller, 2002). The FCDMC also supplied Digital Terrain Models (DTM) which were used to supplement the 2003 topography (a) for the existing conditions model's right overbank, (b) in the main channel model outside of cross-section 51.208, and to determine the flooding extent near the right limits of 51.208, (c) in the left overbank models, to determine the flooding extents in a backwater area of the left limits of a few of the cross-sections.

WEST obtained flood insurance study workmaps of the Hassayampa River from the early 1970's (based on aerial photography taken on 3/2/1971) from the warehoused archives of the FCDMC. This is the primary source of historical topography. These workmaps (hereafter referred to as the 1971 workmaps) have cross hairs showing grid coordinates, which made it possible to geo-reference scans of these maps. The workmaps have a contour interval of 4 feet, and call out frequent spot elevations to the nearest 0.5 feet.

WEST also obtained workmaps from the Cella-Barr flood insurance studies (the effective flood insurance studies) for the Hassayampa River and Sols Wash. Cella-Barr's Hassayampa River workmaps are based on aerial photography taken on March 8, 1988, and Cella-Barr's Sols Wash workmaps are based on aerial photography taken on March 13, 1986. The Cella-Barr drawings, however, are not a good source of topographic information because spot elevations are shown infrequently. Furthermore, the Cella-Barr Hassayampa workmaps have very low contrast, which makes the contours difficult to interpret. The vertical datum of all topographic maps, including the new mapping, is NGVD 1929.

In the part of the Hassayampa River and Sols Wash where the new topography is available, there does not appear to be a significant change in the bank lines since 1971. However, a comparison of 72 spot elevations from the 1971 workmap with the current topography shows that:

- The Hassayampa River has, on average, degraded slightly since 1971. The degradation is slightly more pronounced in the right overbank area of the channel (the area east and southeast of the racetrack). The degradation has been about 1 foot on average, with significant variations, for example the occasional disappearance or appearance of sandbars, and some shifts in the thalweg.

- There are six spot elevations on the 1971 workmap in Sols Wash downstream of Tegner Street. Five of the six points are higher in the recent topography, and one is lower, than shown on the 1971 map. The median elevation increase of these points is 1 foot, although one point is 2.5 feet higher. Therefore, Sols Wash appears to have aggraded slightly since 1971.

- A small region of the Hassayampa River, south of the Sols Wash outlet into the Hassayampa River, has aggraded since 1971. This is possibly due to sediment from Sols Wash settling out as the flow carrying it expands into the Hassayampa River.

3.2. Cross-Sections from ADOT Field Inspections

ADOT conducts periodic field inspections of its bridges. Part of the field inspection sometimes involves a series of measurements from the bridge low chord to the ground. WEST obtained the field inspection records for both the eastbound and westbound US-60 bridges over the Hassayampa River. The eastbound bridge had a more complete set of measurements, extending back to 1968. The ADOT measured distances from the low chord were converted to elevations by using the surveyed 2004 elevations of the bridge low chord. The calculated elevations based on the ADOT field inspections are shown in Figure 3.1. The long-term trend has been degradation, mostly on the east side of the channel which has degraded about 2 feet. The average degradation across the width of the channel at the bridge is about 1 foot from 1969 to

2001. The west side had continued to degrade until 1995, but between 1995 and 2001 aggraded back approximately to the 1969 level.

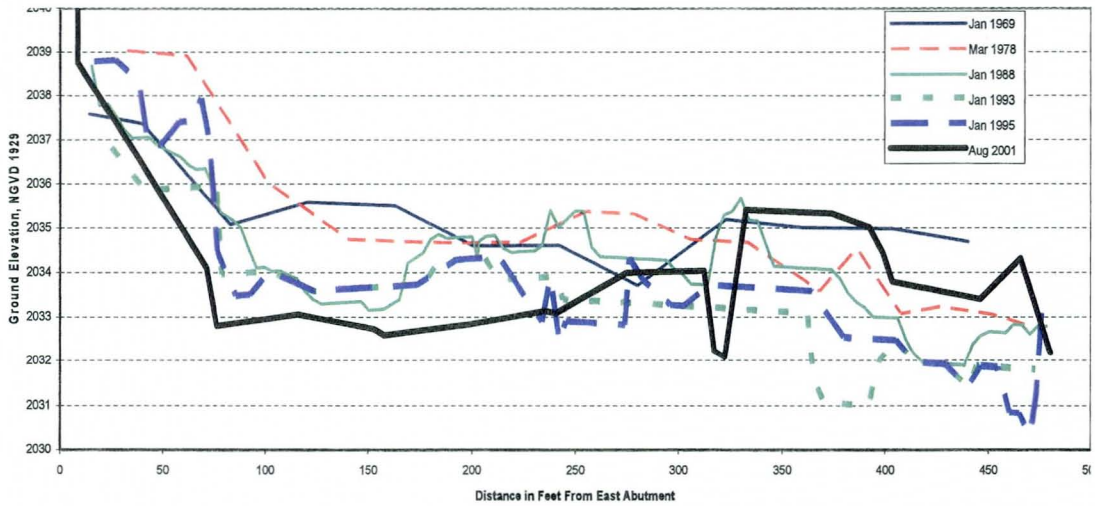

Figure 3.1. Historical cross-sections based on ADOT measurements – eastbound US-60 Bridge

3.3. Hassayampa Low Flow Channels based on 1971 and 2003 Topography

The thalweg for the Hassayampa River was delineated using the 1971 topography and the 2003 topography. The comparison region was limited by the 2003 topography, which extends about 6500 feet along the channel. In some places, the thalweg has shifted as much as 200 to 300 feet, but in other areas it is nearly unchanged. No trend could be discerned on the basis of the two topographic maps. Under the existing US-60 bridges, the thalweg has shifted to the east which is also confirmed by the ADOT bridge inspection reports. The thalweg is shown in Figure 3.2

A plot of the thalweg profile by river station along the Hassayampa River shows degradation at many river stations, little change at others, and slight aggradation in a few places. The thalweg profile is shown in Figure 3.3. The irregularities in the 1971 thalweg profile may be due to inaccuracies from the 1971 topography, attributable to the large 4-foot contour interval. The one-foot contours from the 2003 topography yield a thalweg profile with a fairly uniform slope.

Figure 3.2. Hassayampa River thalweg profiles 1971 versus 2003 (flow is north to south)

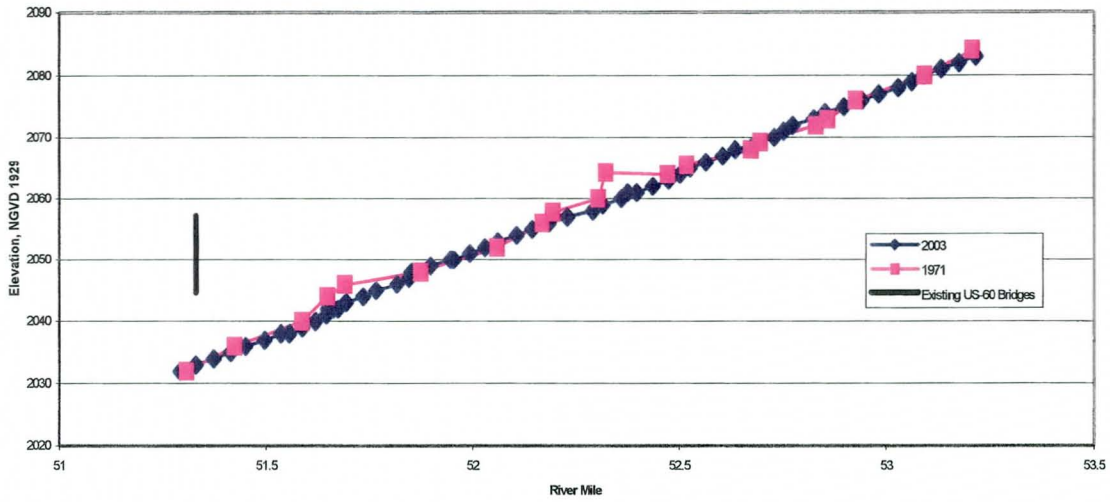

Figure 3.3 Historical thalweg profiles

3.4. Aerial Photography

Aerial photographs were used primarily to examine changes in vegetation. The photographs were also used to examine changes in the banklines in areas where the recent topography was not available.

WEST obtained historical aerial photographs of the project reach for the years 1953, 1984, and 1991. The photographs were used to delineate the horizontal bank locations in ArcView GIS and study lateral migration of the reach. The bank locations in the older photographs were compared to the corresponding bank locations in the 2003 photography. A USGS digital orthographic quadrangle (DOQQ) from 5/24/1997 was also available, which was used as the modern-day reference for regions beyond the limits of the 2003 aerial photography.

3.4.1. Changes since 1953

The 1953 aerial photograph was taken from a high altitude, and has fairly poor resolution. However, some major conclusions can be drawn. There are places where the vegetation line has moved. In an area near the upstream limit of the project, the channel appears to have straightened somewhat; however, most of the channel does not appear to have moved significantly. A region about 4,000 feet upstream of the US-60 bridges on the west bank, the vegetation line has moved about 100 feet, making the unvegetated channel wider now compared to that in 1953.

3.4.2. Changes since 1984 and 1991

The most recent aerial photographs (2003) indicate there is significantly less vegetation in the channel than in the 1991 and 1984 images. The 1953 image is not very sharp and is more difficult to interpret. However, the 1953 image shows vegetation in the channel, especially in the first 2500 feet downstream of the US-60 bridge.

The entire area appears to be less vegetated now than in the past. Even areas on the overbank, between the houses for example, show significantly less vegetation than in the 1991 photograph. The 1984 photograph appears to be slightly more vegetated than the 1991 photograph. The 1997 DOQQ shows more vegetation than the recent photograph, but less than in 1991. Thus, there seems to be a trend toward less vegetation from 1984 to 1991, and especially from 1993 to 2003.

Comparison of the banklines on the basis of vegetation was hampered by the reduction in vegetation over the years. Areas where it may appear that the bank has receded may in fact be due to the loss of vegetation which has occurred in the entire region.

In a 1000-foot reach downstream (south) of the racetrack (and upstream of the US-60 bridge), the vegetation line on the left (east) bank has moved since 1991 by about 40-80 foot to the east, moving it closer to Jack Burden Road. A small amount of shift in the same vegetation line appears to have occurred between 1984 and 1991 as well. Although the topographic information in this area from 1971 workmap is sparse, it appears that the channel has actually degraded in this area, and the main channel has worked its way east.

The biggest change between 1953 and 1984 was the appearance of the racetrack, and a patch of graded land next to it about 300 feet wide and 2000 feet long (the longer dimension parallel to the channel). By 1991, a road and what seems to be a number of corrals (or their remnants) appear on this patch of land. By 1997, and in the recent aerials, evidence of the corrals is gone, apparently obliterated by sediment from the channel. The elevation of most of this area is within a couple of feet of the channel thalweg.

3.5. Conclusion: Qualitative Geomorphic Analysis

The floodplain and the channel appear to be reasonably stable laterally within the last 30-50 years or so. For example, the thalweg of the Hassayampa River in the project reach has scarcely moved in over 30 years. The region does appear to be less vegetated than in the past.

The river does meander somewhat, which creates the potential for bend scour. This is discussed in 7.3.2, where bend scour is calculated for a portion of the proposed embankment. Bend scour is also applied to the west abutment of the proposed Hassayampa River Bridge; this is discussed in 7.10.

Vertically, the Hassayampa River appears to have degraded slightly since 1971, while Sols Wash appears to have aggraded. The degradation in the Hassayampa River means the long-term degradation will need to be considered for all the toe down depths (bank protection and bridge scour).

4. HASSAYAMPA RIVER HYDRAULICS

There are two existing parallel bridges over the Hassayampa River. The westbound (upstream) bridge was built in 1936. The eastbound (downstream) bridge was built in 1962.

For the Hassayampa River, there are five models mentioned in this report. The first model is the HEC-2 model from Cella-Barr, the remaining models were developed by WEST using HEC-RAS version 3.1.2 (U.S. Army Corps of Engineers, 2002). A summary of the models follows:

1. Effective Model. This the HEC-2 model from Cella-Barr upon which the existing flood insurance study is based.

2. Duplicate Effective Model. A version of the existing flood insurance study HEC-2 model, but in HEC-RAS. Modifications were made to the HEC-RAS to most faithfully duplicate the hydraulics and water surface elevations of the HEC-2 model. An encroached version of this model was also created.

3. Corrected Effective Model. The duplicate effective model with corrections made to some parameters and to the elevation of the bridges, when those parameters appeared to be incorrect in the effective model. An encroached version of this model was created.

4. Existing Conditions Model. A model of the hydraulics using the 2003 aerial topography.

5. Proposed Conditions Model. A model showing the hydraulics with the proposed embankment and bridge in place. This was constructed using the most recent topography, plus the plans of the new bridge and new road embankment.

The first three models were created to comply with FEMA requirements for a future Conditional Letter of Map Revision. The fourth, existing conditions, model was used to establish the current hydraulics. The final model, the proposed conditions model, establishes the future hydraulics. The most relevant comparisons are between the proposed conditions and the existing conditions models, since the differences show the impacts of the project.

4.1. Vertical and Horizontal Datum

The vertical datum of the FIS (Cella Barr) models is NGVD 1929 according to the published flood insurance profiles for the Hassayampa River and Sols Wash (FEMA, 2001). The new topography and all other elevations were also in the same vertical datum.

The horizontal datum of the mapping was performed using a "ground" coordinate system established for this project. Ground coordinates can be obtained by converting from the Arizona State Plane Central NAD83 grid, International Feet. State plane coordinates, international feet, multiplied by a grid-to-ground scale factor of 1.00013938 yields the ground coordinates for this project (memo from Chuck Gardner of Project Engineering Consultants, Ltd. to Dan Stough of

Jacobs Engineering, January 30, 2004, and personal communication with Chuck Gardner on November 12, 2004).

4.2. Hassayampa River Effective Model

The effective flood insurance model of Hassayampa River in Maricopa County is based on nine HEC-2 models, each consisting of one reach of the river. The region with the US-60 bridge is in the ninth, and most upstream, reach of the model. The original HEC-2 files have the names HASS.R9 (unencroached profile) and HASS.F9 (encroached profile). Since the Hassayampa River also extends upstream into Yavapai County, separate models which were used to delineate the floodplain in Yavapai County are available. However, those reaches were not needed for this study.

4.3. Hassayampa River Duplicate Effective Model

A duplicate effective model was created for this reach first importing the HEC-2 model's geometry into HEC-RAS. The import process copied the cross-section geometries, bridge geometries, n-values, and bank stations. In addition to the imported geometry, the following steps were taken so that the results from the HEC-RAS model would match the results from the HEC-2 model:

- The downstream boundary condition at cross-section 49.04, the downstream limit of reach 9, was transferred from HEC-2 to the HEC-RAS model. For the unencroached profile, the water surface elevation was 1982.65 feet, while for the encroached profile it was 1983.05 feet.

- The discharges were set per the HEC-2 model: 71,000 cfs at cross-sections 50.46 and upstream, and 67,635 cfs at cross-section 50.4 and downstream. The 67,635 cfs in the downstream reach is not mentioned in the published Flood Insurance Study, even though it appears in the model (the 67,635 cfs discharge begins almost a mile downstream of the existing conditions or proposed conditions models and is not used in either of them).

- The encroachment stations were copied from the HEC-2 model to the HEC-RAS model.

- The conveyance calculation option in HEC-RAS was changed to the HEC-2 method, where conveyance is calculated separately between each cross-section point on the overbanks.

- The internal bridge cross-sections for the US-60 bridge between cross-sections 51.33 and 51.34 (HEC-RAS assigned the bridge to section 51.335 on importation from HEC-2) were modified. HEC-RAS uses the bounding cross-sections to establish the geometry under the bridge. In the HEC-2 model, the bridge was coded as a special bridge (SB), and the area under the bridge appears in Field 7 of the SB card. It was coded as 4650 square feet. Because the bounding cross-sections for the bridge resulted in an area lower than this, the internal bridge cross-sections in the duplicate

effective model needed to be modified. The internal cross-sections were modified to be rectangular sections such that the area under the bridge low chord, excluding piers, was 4650 square feet.

- The contraction and expansion coefficients at cross-sections 50.4, 50.46, 50.56, and 50.65 were set a 0.2 and 0.4, respectively. These were the coefficients used in the HEC-2 model for these sections. At all other cross-sections the contraction and expansion coefficients were set at 0.1 and 0.3 as in the HEC-2 model.

With these modifications, the water surface profile for the HEC-RAS unencroached duplicate effective model's elevations are within 0.01 feet of the HEC-2 model's elevations at every cross-section.

For the encroached HEC-RAS duplicate effective model, the water surface profile is within 0.01 feet at most cross-sections compared to the HEC-2 model. However, for the cross-sections upstream of the existing bridges, and the next few cross-sections, the duplicate effective model has more than 1.0 feet of surcharge. The reason for this large discrepancy is that the encroached HEC-2 model neglects to consider the encroachment at the bridge section, and as a result, erroneously allows for conveyance in the right overbank at the bridge section of the encroached model. In HEC-RAS, the encroachment stations at the upstream and downstream section automatically cause the bridge to be encroached as well.

4.4. Hassayampa River Corrected Effective Model

The following modifications were made to the duplicate effective model to create the corrected effective model:

The bridge low chord elevations were modified. In the effective model, the bridge is shown as having a uniform low chord of 2043.3 feet. A field survey was conducted to check the low chord elevations, which are summarized in

- Table 4-1. For the corrected effective model, the elevations from the 1936 bridge, which is the lower of the two bridges, were used.

- The bridge was changed from having a single 40-foot wide pier (as it is represented in the HEC-2 model) to having 5 piers, each 4 foot wide at the top, widening at a ratio of 1:24 on each side. This is the pier configuration of the 1936 bridge, which has wider piers than the 1963 bridge, and provides the larger obstruction to flow.

- An ineffective flow area was added at cross-section 51.4. This cross-section is 300 feet upstream of the face of the bridge. A large portion of this cross-section is in the "hydraulic shadow" of a plateau on the left side of the channel. This portion of the cross-section would not convey flow.

- An ineffective flow area was added at cross-section 51.29, which is downstream of the bridge. The leftmost 580 feet of the cross-section inside the floodway is in the "hydraulic shadow" of the bridge and should not be considered effective flow.

- Expansion and contraction coefficients upstream and downstream of the bridges were changed to 0.5 and 0.3, respectively.

Table 4-1. Existing Hassayampa River Bridges low chord elevations as surveyed in March 2004

	At East Abutment (ft)	Center (ft)	At West Abutment (ft)
1936 bridge (westbound)	2044.62	2045.47	2044.62
1962 bridge (eastbound)	2046.79	2047.42	2046.75

4.5. Hassayampa River Existing and Proposed Conditions Manning's n-Values

For the region that was modeled in the existing conditions and proposed conditions models, the n-values are based on the n-values found in the Flood Insurance Study model. The channel n-values are 0.035 throughout the reach. The overbank n-values in the effective model are generally 0.04 at cross-section 51.64 and higher (upstream), and the overbank n-values are generally 0.045 at cross-section 51.55 and downstream. These same channel and overbank n-values were applied to the existing and proposed conditions model, except that some areas in the effective (Flood Insurance Study) model where the n-values were set at 0.13 or 1.0. Each exception was examined, and addressed in the existing and proposed conditions models. The width of the channel to which the 0.035 n-value applied for each cross-section was based on visual determination of the limits of the sand bed channel based on overlaid aerial photographs and topography. These widths approximately correspond to the channel widths in the HEC-2 model.

In the discussion below, the term "inside" an encroachment indicates the side of the encroachment toward the center of the channel, while the term "outside" of an encroachment indicates the side of the encroachment toward the overbank of the channel. All encroachments referenced in the discussion of n-values are the effective FIS encroachments.

- Cross-Section 51.64: From the left limit to 196 feet outside of the existing left encroachment, n = 1.0 in the effective model. This area is in the hydraulic shadow of a hill, water in this region would be ineffective flow. This was addressed in the existing conditions and proposed conditions model by modeling an ineffective area in this region. In the new model, the ineffective region extends from the left limit to about 180 feet from the left encroachment.

- Cross-Section 51.55: From 457 to 25 feet outside of the existing left encroachment, n = 0.13 in the effective model. This area is suburban housing. In the existing conditions and proposed model, this was taken into account by designating a polygon in HEC-GeoRAS to have n = 0.13. For this cross-section in the existing conditions and proposed conditions models, the n = 0.13 region extends from the left limit of the cross-section to about 25 feet outside of the existing left encroachment.

- Cross-Section 51.45: The entire left overbank is shown at n = 0.045 in the effective model. However, aerial photographs show that there is suburban housing from the left limit of the cross-section to about 20 feet outside of the existing left encroachment.

- Cross-Section 51.45: The entire right overbank is shown at n = 0.045 in the effective model. However, aerial photographs indicate buildings on the right overbank. Approximately the right most 500 feet of the cross-section was assigned n = 0.13 in the existing conditions and proposed conditions model.

- Cross-Section 51.4: From the left limit to 125 feet inside of the left encroachment, n = 0.13 in the effective model. There is heavy brush inside the left encroachment, and suburban housing on the outside (the left overbank). In the existing conditions and proposed conditions models, the n = 0.13 region goes from the left limit of the cross-section to about 170 feet inside of the left encroachment. From the right limit to 399 feet outside of the right encroachment, Manning's n is 0.13 in the effective model. There are some buildings in this region. The buildings were taken into account in the existing and proposed conditions model, although the region was smaller based on the aerial photographs.

- Cross-Section 51.29: In the effective model, the region from the left limit to about 20 feet inside of the left encroachment has n = 1.0. This region was apparently assigned the high n-value because it is in the hydraulic shadow of the US-60's left embankment, which is not overtopped, and because there are houses in the area. In the existing and proposed conditions model, these regions were taken into account using ineffective flow areas. In addition, parts of the right overbank were assigned n = 0.13 because there are buildings in the area.

For most cross-sections, the bank stations were set at the boundaries of the 0.035 n-values. Exceptions were made in the interior cross-sections of the existing bridge, and the first few cross-sections downstream of the existing bridge. For these cross-sections, there is some vegetation in the channel, for example underneath the existing bridge on the left side of the cross-section, and on both the left and right side of the cross-section in some of the sections downstream of the existing bridge. Rather than move the bank stations for these cross-sections, an n-value of 0.045 was used for those parts of the channel that contained the vegetation.

4.6. Hassayampa River Existing and Proposed Conditions Models

In the existing conditions, there are two US 60 bridges across the Hassayampa River: the 1962 bridge on the downstream side and the 1936 bridge on the upstream side. Because the two bridges are adjacent, they are modeled as one bridge in the existing conditions model.

In the proposed conditions model, the 1936 bridge will be removed, and the 1962 bridge will be left in place. Furthermore, the proposed bridge over the Hassayampa River will be constructed as well as the new road embankment. The purpose in leaving the 1962 bridge in place is so that it can be used to run gas lines across the Hassayampa River. Because the 1962 bridge has a

higher low chord and narrower piers than the 1936 bridge, the removal of the 1936 bridge allows for more conveyance in the proposed conditions than in the existing conditions model. In the proposed condition, it is assumed that the 1962 bridge will no longer be the responsibility of ADOT, and no ADOT freeboard criterion was applied to it.

It has also been proposed that a borrow pit, to provide soil to build the road embankment, be excavated in left overbank of the river. That borrow pit, though considered in the sediment model, was not considered for the proposed conditions HEC-RAS model. It is not known at this time whether that borrow pit location will be used or not.

Figure 4.1 shows the layout of cross-sections which was used in the existing and proposed conditions models, and also shows the location of the proposed borrow pit. Figure 4.2 shows a zoomed in view of proposed bridge locations. The most downstream cross-section, 51.13, is from the corrected effective model. On the upstream side, cross-section 53.25 and higher are surveyed cross-sections that were not used in the FIS model but were used in the sediment model. All other cross-sections are taken from the DTM. One section shown in the layout, cross-section 51.338, is in the proposed conditions model only, and is not present in the existing conditions model because it is the upstream bounding section of the 1962 bridge. In the existing conditions model the 1936 and 1962 bridges are modeled as one bridge.

The hydraulic ineffective flow areas caused by the existing abutments were accounted for in HEC-RAS with a 1:1 contraction rate upstream of the existing bridges, and an 2:1 expansion rate downstream of the existing bridges.

Contraction and expansion coefficients were set at 0.3 and 0.5 respectively, in those cross-sections upstream and downstream of the bridges where flow was contracting before entering the bridge or expanding after exiting the bridge. Expansion and contraction coefficients were set at 0.1 and 0.3 respectively at all other cross-sections.

Cross-section spacing of 90 to 130 feet between was typical for the first few thousand feet upstream of the existing and proposed bridges over the Hassayampa River. The close spacing is justified by the detailed topography that was available for the channel.

Details regarding the configuration of the proposed bridge over the Hassayampa River are presented in 7.5.1.

Water surface elevations from the existing conditions model are presented together with the proposed conditions water surface elevations in Table 4-2.

4.6.1. Overbank Flow at the Hassayampa River Bridges

Separate hydraulic models were developed in HEC-RAS 3.1.2 to model the overbank flow around the bridges over the Hassayampa River for both the proposed conditions and existing conditions models. In the proposed conditions, this overbank flow bypasses the proposed bridge and the (raised) 1962 bridge openings. In the existing conditions model, the overbank flow bypasses the 1936 and 1962 bridge openings. Three separate hydraulic models were developed: one for the left overbank under the existing conditions, one for the right overbank under existing

conditions, and one for the left overbank under proposed conditions. It is the assumed that the future improvement / levees at Sols Wash will be high enough to restrain the 100-year Hassayampa River flood. Therefore, it was assumed that the 100-year Hassayampa River flood will be contained on the right overbank by the proposed Hassayampa River levees as well as the future Sols Wash levees, and no flooding on the right overbank at the Hassayampa River Bridge will occur.

The cross-sections from each model were cut from either the existing topography or the proposed topography using HEC-GeoRAS. The overbank areas consist primarily of vegetation, streets, buildings, and parking lots. It is also possible that during flood events, cars and semi trailers may also be encountered along the streets and parking lots. For example, a group of six semi-trailers appeared to be parked in parking lot in the aerial photographs for the right overbank. The wide variety of terrain leads to some uncertainty in the roughness value. In order not to underestimate the Manning n, it was deemed appropriate to use a uniform Manning n-value of 0.05 for the overbank areas in both the existing and the proposed condition.

Using the aerial photographs, ineffective flow areas were drawn around the structures to simulate the fact that flow would have to go around the structures rather than through them. Other areas that appear to convey little flow were also identified as ineffective areas using the aerial photographs. These structures obstructed a considerable amount of the flow area, especially in the right overbank.

For the right overbank existing condition model, the downstream boundary condition was set equal to the water surface elevation at River Station (RS) 51.13 in the main model (2037.45 feet). For both the existing and proposed condition left overbank models, the downstream boundary condition was set equal to the water surface elevation at RS 51.303 in the main Hassayampa River model (2042.85 feet). Because all the overbank models went through critical depth near their downstream limits, the downstream boundary condition becomes hydraulically isolated from the rest of the model. Within reasonable variations of the downstream boundary condition, that boundary condition does not impact the results of the overbank models.

At the upstream end of each overbank model, the flow rate was adjusted until the energy grade line elevation at the most upstream cross-section matched the energy grade line at the nearest cross-section of the main Hassayampa River model. For both of the left overbank models (both the proposed and existing conditions models), the flow rate entering the system was adjusted until the energy grade line matched the energy grade from RS 51.418 in the main Hassayampa River model line (within a tolerance of 0.04 feet). The existing condition right overbank model tied into cross-section 51.398 on the upstream side.

Because the flow rates in the overbanks were large for the existing conditions models, flow that entered the overbanks was subtracted from the Hassayampa River model. Thus, the three models (the main Hassayampa River model, the right overbank model, and the left overbank model) were simultaneously adjusted to match the energy grade lines and to balance the discharges in each model.

In the proposed conditions, the discharge modeled as going over the left overbank was not very large. Considering that this flow is somewhat uncertain, it was considered prudent not to subtract the overbank discharge from the main river model.

A summary of the flow rates in the overbanks is shown in Table 4-2.

Table 4-2. Overbank discharges at Hassayampa River Bridges

	Existing Conditions (cfs)	Proposed Conditions (cfs)
Left Overbank	3,900	900
Right Overbank	1,760	0

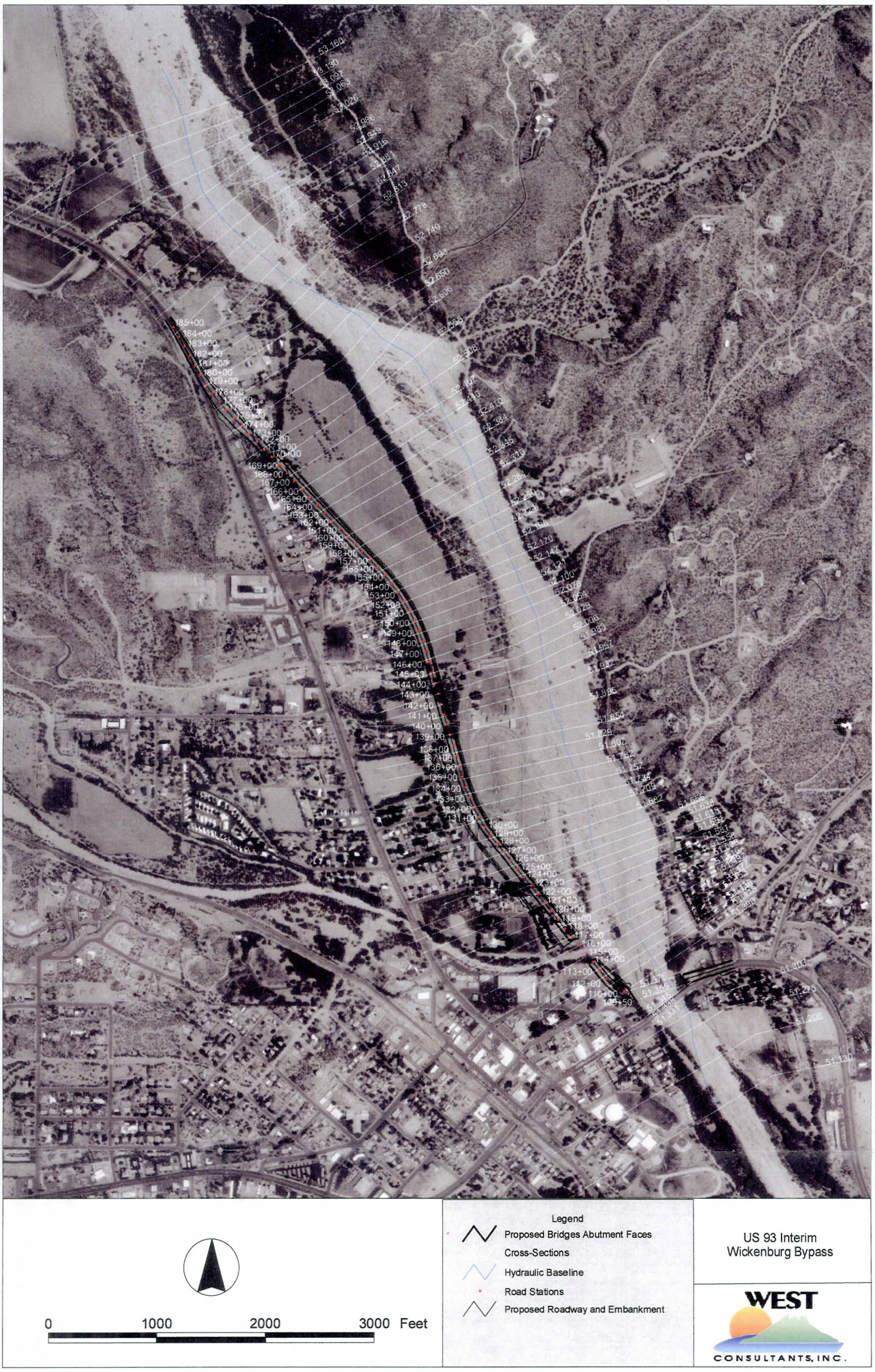

Figure 4.1. Study area with Hassayampa River cross-sections (Hassayampa River flows north to south)

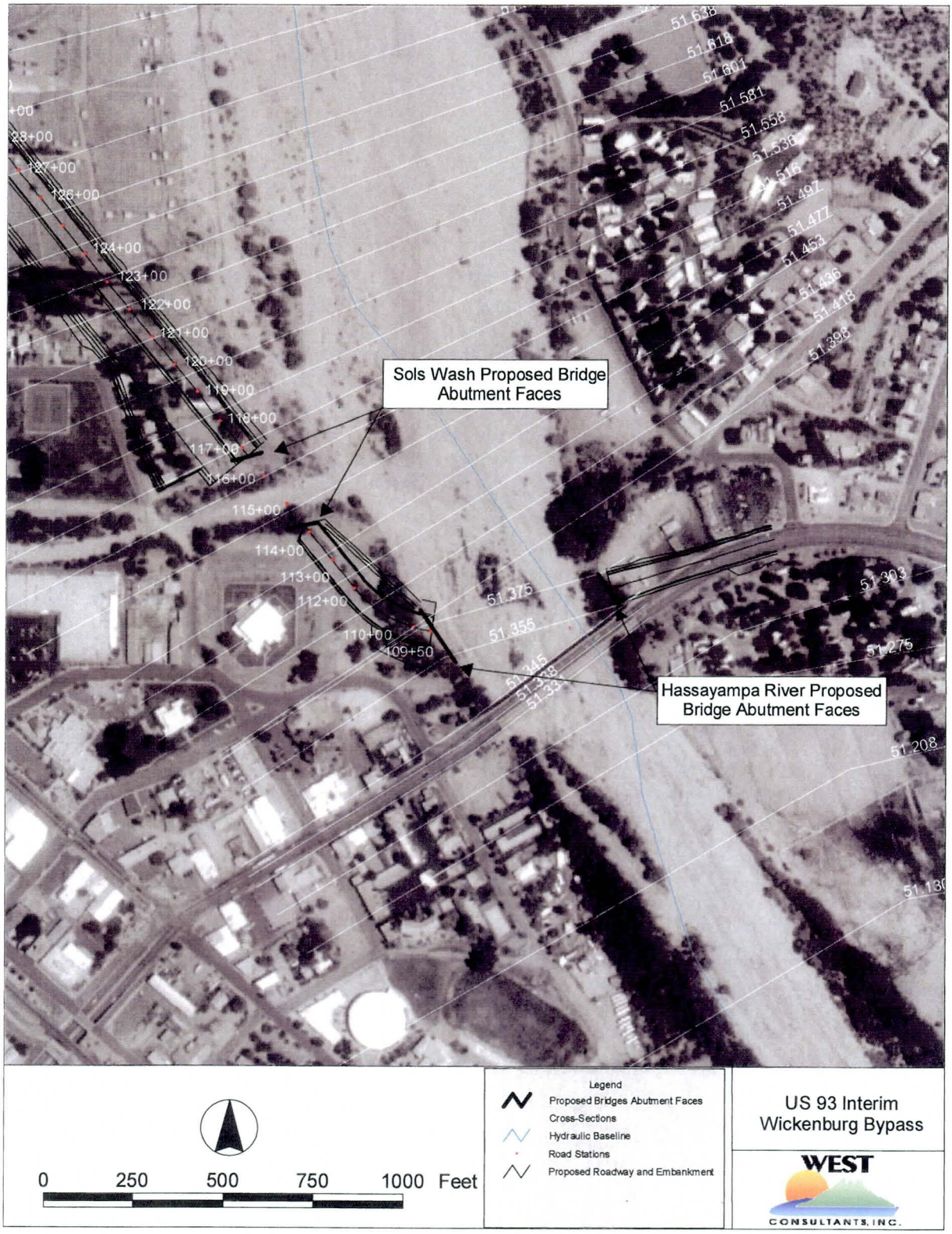

Figure 4.2. Detail showing location of proposed bridge abutments

The cross-sections for the proposed conditions model incorporate the proposed embankment and bridge. All the cross-sections cutlines from the existing conditions model are in the proposed conditions model. There is one additional cross-section in the proposed conditions model, cross-section 51.338. Cross-section 51.338 is just upstream of the 1962 (eastbound) bridge. In the existing conditions model, the cross-section is not present because the eastbound and westbound bridges are taken together as one equivalent bridge, and the bounding cross-section (51.345) for that equivalent bridge is farther upstream.

The hydraulic ineffective flow areas caused by the new abutments were considered, with a 1:1 contraction rate upstream of the proposed bridge, and a 2:1 expansion rate downstream of the 1962 bridge. These ineffective flow limits on the downstream side are unchanged from the existing conditions model since the existing embankments will remain in place.

The existing bridge has railbank protection on the left abutment which obstructs some of the flow. This was modeled in HEC-RAS by placing a blocked obstruction which was offset by 15 feet from the left abutment.

The proposed Hassayampa River Bridge abutments are skewed 17 degrees to the portion of the bridge that is straight. The alignment of the abutments was set to be parallel to the estimated flow direction from a 100-year flood in the Hassayampa River.

Table 4-3 shows the 100-year water surface elevations between the FIS model, the corrected effective model, the existing conditions model, and the proposed conditions model. It also shows, in the last column, the difference between the proposed condition and the existing condition models. Encroachments are discussed further in 4.8. Figure 4.3 shows the profiles graphically. Cross-sections from the FIS model are only included in the table when they are coincident or very close to the cross-sections of HEC-RAS model. Table 4-4 shows the proposed floodway elevations compared to existing and proposed conditions water surface elevations.

Table 4-3. Hassayampa River 100-year water surface profiles, NGVD 1929 feet

Cross-Section	Equivalent FIS Section	Effective Model (ft)	Corrected Effective Model (ft)	Existing Conditions Model (ft)	Proposed Conditions Model (ft)	Proposed Minus Existing (ft)
53.160	53.16	2088.39	2088.4	2088.32	2088.32	0.00
53.130				2087.53	2087.53	0.00
53.097				2086.77	2086.77	0.00
53.063	53.06	2085.3	2085.3	2085.86	2085.86	0.00
53.026				2084.90	2084.9	0.00
52.986				2084.15	2084.15	0.00
52.949				2083.58	2083.58	0.00
52.916				2083.17	2083.17	0.00
52.881	52.87	2081.92	2081.93	2082.72	2082.72	0.00
52.847				2081.61	2081.61	0.00
52.813				2080.24	2080.24	0.00
52.778				2079.22	2079.22	0.00
52.740				2077.87	2077.87	0.00
52.693				2076.28	2076.28	0.00
52.650				2074.31	2074.31	0.00
52.606				2072.81	2072.9	0.09
52.558				2071.68	2071.97	0.29

Cross-Section	Equivalent FIS Section	Effective Model (ft)	Corrected Effective Model (ft)	Existing Conditions Model (ft)	Proposed Conditions Model (ft)	Proposed Minus Existing (ft)
52.508				2070.48	2071.09	0.61
52.467				2069.67	2070.29	0.62
52.440				2069.10	2069.8	0.70
52.410	52.4	2069.87	2069.87	2068.64	2069.34	0.70
52.384				2068.23	2068.89	0.66
52.345				2067.58	2068.22	0.64
52.316	52.3	2068.09	2068.09	2066.92	2067.60	0.68
52.284				2066.41	2066.88	0.47
52.250				2065.80	2066.1	0.30
52.224	52.21	2065.22	2065.22	2064.87	2065.48	0.61
52.196				2064.20	2064.8	0.60
52.170				2063.26	2063.7	0.44
52.147				2062.62	2063.08	0.46
52.121				2061.70	2062.02	0.32
52.100				2060.82	2061.11	0.29
52.078				2060.05	2060.5	0.45
52.054				2059.72	2059.91	0.19
52.029				2059.06	2059.18	0.12
52.006				2058.70	2058.79	0.09
51.983				2058.10	2058.18	0.08
51.957				2057.69	2057.75	0.06
51.932				2057.26	2057.33	0.07
51.895				2056.30	2056.53	0.23
51.854	51.83	2056.04	2056.01	2055.62	2055.51	-0.11
51.829				2055.31	2055.03	-0.28
51.807				2055.10	2054.42	-0.68
51.782				2054.87	2054	-0.87
51.757	51.74	2052.45	2052.51	2054.63	2053.61	-1.02
51.734				2054.57	2053.4	-1.17
51.709				2054.51	2053.17	-1.34
51.682				2054.46	2053.01	-1.45
51.656	51.64	2051.68	2051.8	2054.42	2052.94	-1.48
51.638				2054.38	2052.87	-1.51
51.618				2054.36	2052.83	-1.53
51.601				2054.26	2052.64	-1.62
51.581				2054.20	2052.49	-1.71
51.558	51.55	2051.06	2051.2	2054.12	2052.39	-1.73
51.536				2054.09	2052.28	-1.81
51.516				2054.09	2052.19	-1.90
51.497				2054.01	2051.96	-2.05
51.477				2053.93	2051.9	-2.03
51.453	51.45	2050.73	2050.89	2053.69	2051.43	-2.26
51.436				2053.60	2051.11	-2.49
51.418				2053.58	2051.01	-2.57
51.398	51.4	2050.41	2050.37	2053.42	2050.36	-3.06
51.375				2053.21	2048.84	-4.37
51.355				2053.15	2046.62	-6.53
51.345	51.34	2049.01	2048.85	2052.57	2046.59	-5.98
51.338					2046.46	
51.331	51.33	2044.95	2045.75	2045.22	2044.17	-1.05
51.303				2042.82	2042.97	0.15
51.275				2041.36	2041.4	0.04
51.208	51.2	2039.38	2039.4	2039.39	2039.39	0.00
51.130	51.13	2037.45	2037.47	2037.45	2037.45	0.00

Table 4-4. Summary of floodway (encroached) water surface profiles

Cross-Section	Existing Conditions Model (ft)	Proposed Conditions Model (ft)	Effective (FIS) Encroached Profile (ft)	Encroached Existing Conditions (with existing floodway) (ft)	Encroached Existing Conditions (with proposed floodway) (ft)	Encroached Proposed Conditions (with proposed floodway) (ft)	Encroached Proposed Minus Base Flood Proposed (ft)
53.160	2088.32	2088.32	2089.13	2089.11	2089.11	2089.11	0.79
53.130	2087.53	2087.53		2088.45	2088.45	2088.45	0.92
53.097	2086.77	2086.77		2087.64	2087.65	2087.65	0.88
53.063	2085.86	2085.86	2086.18	2086.79	2086.8	2086.8	0.94
53.026	2084.90	2084.9		2085.55	2085.63	2085.63	0.73
52.986	2084.15	2084.15		2084.56	2084.67	2084.67	0.52
52.949	2083.58	2083.58		2083.79	2083.92	2083.92	0.34
52.916	2083.17	2083.17		2083.16	2083.33	2083.33	0.16
52.881	2082.72	2082.72	2081.95	2082.66	2082.83	2082.83	0.11
52.847	2081.61	2081.61		2081.55	2081.68	2081.68	0.07
52.813	2080.24	2080.24		2080.3	2080.41	2080.41	0.17
52.778	2079.22	2079.22		2079.37	2079.28	2079.28	0.06
52.740	2077.87	2077.87		2078.02	2078.04	2078.04	0.17
52.693	2076.28	2076.28		2076.22	2076.33	2076.33	0.05
52.650	2074.31	2074.3		2074.3	2074.32	2074.32	0.01
52.606	2072.81	2072.9		2073.03	2072.94	2072.97	0.07
52.558	2071.68	2071.97		2072.26	2072.09	2072.16	0.19
52.508	2070.49	2071.1		2071.47	2071.13	2071.27	0.18
52.467	2069.67	2070.29		2070.75	2070.37	2070.41	0.23
52.440	2069.09	2069.8		2070.25	2069.91	2070.03	0.23
52.410	2068.63	2069.34	2070.75	2069.79	2069.52	2069.56	0.22
52.384	2068.21	2068.87		2069.19	2069.03	2068.98	0.09
52.345	2067.59	2068.22		2068.53	2068.27	2068.32	0.10
52.316	2066.92	2067.59	2068.78	2067.95	2067.79	2067.72	0.12
52.284	2066.41	2066.88		2067.08	2066.9	2066.87	-0.01
52.250	2065.80	2066.1		2066.42	2066.16	2066.14	0.04
52.224	2064.87	2065.48	2065.78	2065.87	2065.55	2065.56	0.08
52.196	2064.20	2064.8		2065.18	2064.88	2064.87	0.07
52.170	2063.26	2063.7		2063.94	2063.76	2063.77	0.07
52.147	2062.62	2063.08		2063.37	2063.16	2063.16	0.08
52.121	2061.70	2062.02		2062.2	2062.23	2062.22	0.20
52.100	2060.82	2061.11		2061.46	2061.5	2061.48	0.37
52.078	2060.05	2060.5		2060.99	2061.08	2061.04	0.54
52.054	2059.72	2059.91		2060.51	2060.64	2060.57	0.66
52.029	2059.06	2059.18		2059.86	2059.98	2059.86	0.68
52.006	2058.70	2058.79		2059.4	2059.57	2059.4	0.61
51.983	2058.10	2058.18		2058.62	2058.99	2058.62	0.44
51.957	2057.69	2057.75		2058.12	2058.64	2058.12	0.37
51.932	2057.26	2057.33		2057.66	2058.33	2057.66	0.33
51.895	2056.30	2056.53		2056.6	2057.82	2056.64	0.11

Cross-Section	Existing Conditions Model (ft)	Proposed Conditions Model (ft)	Effective (FIS) Encroached Profile (ft)	Encroached Existing Conditions (with existing floodway) (ft)	Encroached Existing Conditions (with proposed floodway) (ft)	Encroached Proposed Conditions (with proposed floodway) (ft)	Encroached Proposed Minus Base Flood Proposed (ft)
51.854	2055.62	2055.51	2056.05	2055.47	2057.44	2055.56	0.05
51.829	2055.31	2055.02		2054.91	2057.3	2055.02	-0.01
51.807	2055.10	2054.42		2054.42	2057.16	2054.49	0.07
51.782	2054.87	2054		2054.09	2057.1	2054.12	0.12
51.757	2054.63	2053.61	2052.77	2053.8	2057.04	2053.8	0.19
51.734	2054.57	2053.4		2053.66	2057	2053.58	0.18
51.709	2054.51	2053.17		2053.56	2056.95	2053.38	0.21
51.682	2054.46	2053.01		2053.47	2056.91	2053.22	0.21
51.656	2054.42	2052.94	2052.2	2053.42	2056.88	2053.12	0.18
51.638	2054.38	2052.87		2053.34	2056.86	2053.04	0.17
51.618	2054.36	2052.83		2053.29	2056.83	2052.97	0.14
51.601	2054.26	2052.64		2053.15	2056.77	2052.8	0.16
51.581	2054.20	2052.49		2053.08	2056.7	2052.64	0.15
51.558	2054.12	2052.39	2051.64	2052.93	2056.63	2052.49	0.10
51.536	2054.09	2052.28		2052.86	2056.58	2052.36	0.08
51.516	2054.09	2052.19		2052.82	2056.54	2052.25	0.06
51.497	2054.01	2051.96		2052.79	2056.45	2052.02	0.06
51.477	2053.93	2051.9		2052.66	2056.43	2051.97	0.07
51.453	2053.69	2051.43	2051.23	2052.13	2056.24	2051.43	0.00
51.436	2053.60	2051.11		2052.02	2056.14	2051.18	0.07
51.418	2053.58	2051.01		2051.93	2056.07	2051.11	0.10
51.398	2053.42	2050.36	2050.82	2051.59	2055.71	2050.46	0.10
51.375	2053.21	2048.84		2050.74	2054.98	2048.88	0.04
51.355	2053.15	2046.62		2050.52	2054.91	2046.57	-0.05
51.345	2052.57	2046.59	2049.75	2050.41	2055	2046.59	0.00
51.338		2046.46				2046.46	0.00
51.331	2045.22	2044.17	2045.72	2045.58	2044.97	2044.97	0.80
51.303	2042.82	2042.97		2043.18	2043.1	2043.1	0.13
51.275	2041.36	2041.4		2041.98	2041.98	2041.98	0.58
51.208	2039.39	2039.39	2040.17	2040.17	2040.17	2040.17	0.78
51.130	2037.45	2037.45	2037.62	2037.62	2037.62	2037.62	0.17

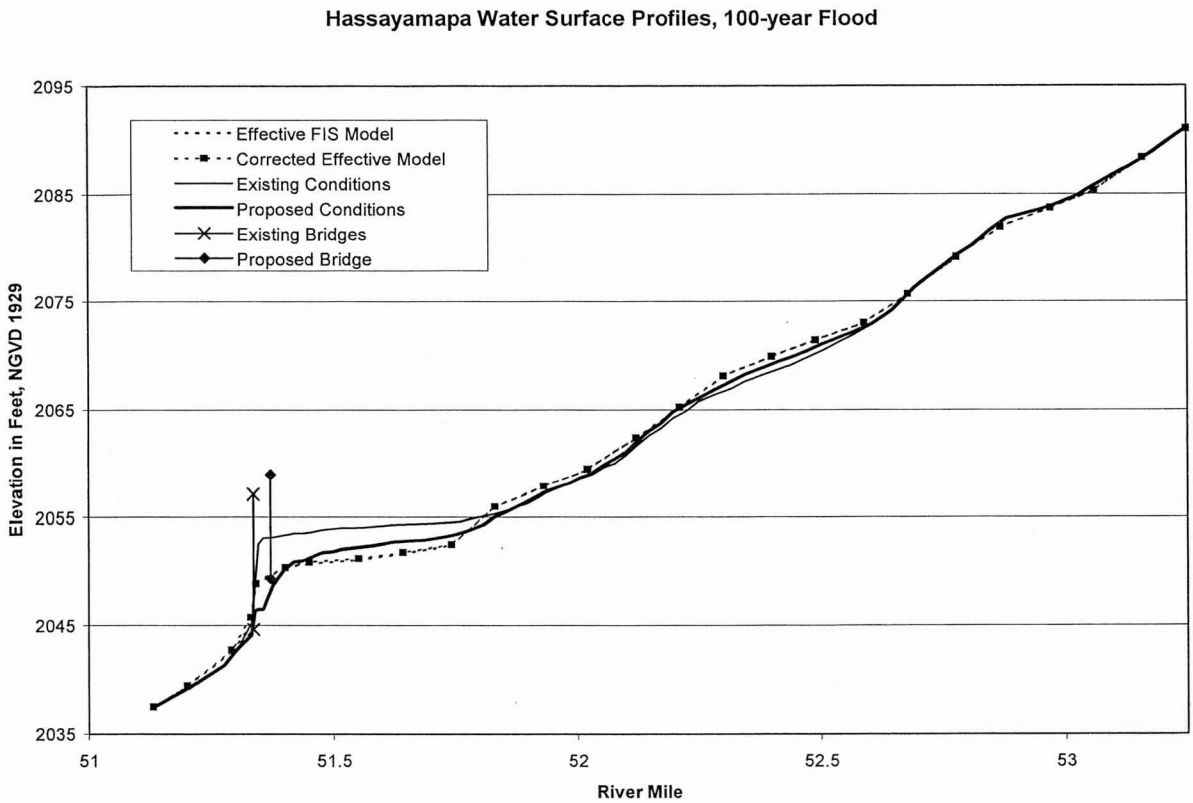

Figure 4.3. Hassayampa River 100-year water surface profiles

For the 50-year water surface elevation, a comparison was made between the existing conditions model and the proposed conditions. The results are shown in tabular form in Table 4-5 and in graphical form in Figure 4.4.

Table 4-5. Hassayampa River 50-year water surface profiles, existing and proposed conditions

Cross-Section	Existing Conditions (ft)	Proposed Conditions (ft)	Increase (Decrease shown as negative) (ft)
53.16	2087.36	2087.36	0
53.13	2086.54	2086.54	0
53.097	2085.7	2085.7	0
53.063	2084.84	2084.84	0
53.026	2083.77	2083.77	0
52.986	2082.87	2082.87	0
52.949	2082.22	2082.22	0
52.916	2081.73	2081.73	0
52.881	2081.29	2081.29	0
52.847	2080.28	2080.28	0
52.813	2078.95	2078.95	0
52.778	2077.89	2077.89	0
52.74	2076.73	2076.73	0

Cross-Section	Existing Conditions (ft)	Proposed Conditions (ft)	Increase (Decrease shown as negative) (ft)
52.693	2075.18	2075.18	0
52.65	2073.34	2073.34	0
52.606	2071.77	2071.79	0.02
52.558	2070.57	2070.75	0.18
52.508	2069.31	2069.79	0.48
52.467	2068.44	2068.94	0.5
52.44	2067.83	2068.4	0.57
52.41	2067.33	2067.91	0.58
52.384	2066.86	2067.4	0.54
52.345	2066.23	2066.77	0.54
52.316	2065.58	2066.19	0.61
52.284	2064.98	2065.36	0.38
52.25	2064.4	2064.65	0.25
52.224	2063.55	2064.05	0.5
52.196	2062.87	2063.36	0.49
52.17	2062.04	2062.4	0.36
52.147	2061.43	2061.8	0.37
52.121	2060.52	2060.77	0.25
52.1	2059.72	2059.93	0.21
52.078	2059.02	2059.29	0.27
52.054	2058.6	2058.71	0.11
52.029	2057.92	2057.94	0.02
52.006	2057.54	2057.52	-0.02
51.983	2056.85	2056.84	-0.01
51.957	2056.36	2056.36	0
51.932	2055.82	2055.91	0.09
51.895	2054.71	2055.08	0.37
51.854	2053.71	2054.02	0.31
51.829	2053.15	2053.49	0.34
51.807	2052.66	2052.75	0.09
51.782	2052.17	2052.18	0.01
51.757	2051.27	2051.45	0.18
51.734	2050.87	2050.93	0.06
51.709	2050.64	2050.5	-0.14
51.682	2050.43	2050.17	-0.26
51.656	2050.3	2049.99	-0.31
51.638	2050.19	2049.86	-0.33
51.618	2050.13	2049.77	-0.36
51.601	2049.93	2049.54	-0.39
51.581	2049.85	2049.38	-0.47
51.558	2049.73	2049.23	-0.5
51.536	2049.64	2049.1	-0.54
51.516	2049.61	2048.98	-0.63
51.497	2049.52	2048.7	-0.82
51.477	2049.39	2048.64	-0.75
51.453	2048.97	2048.12	-0.85
51.436	2048.85	2047.8	-1.05
51.418	2048.81	2047.72	-1.09
51.398	2048.67	2047.22	-1.45
51.375	2048.35	2046.12	-2.23
51.355	2048.28	2043.98	-4.3
51.345	2047.75	2043.96	-3.79
51.338	n/a	2043.81	
51.331	2042.83	2042.83	0
51.303	2040.89	2040.89	0
51.275	2039.87	2039.87	0

Cross-Section	Existing Conditions (ft)	Proposed Conditions (ft)	Increase (Decrease shown as negative) (ft)
51.208	2038.09	2038.09	0
51.13	2036.03	2036.03	0
53.16	2087.36	2087.36	0
53.13	2086.54	2086.54	0
53.097	2085.7	2085.7	0

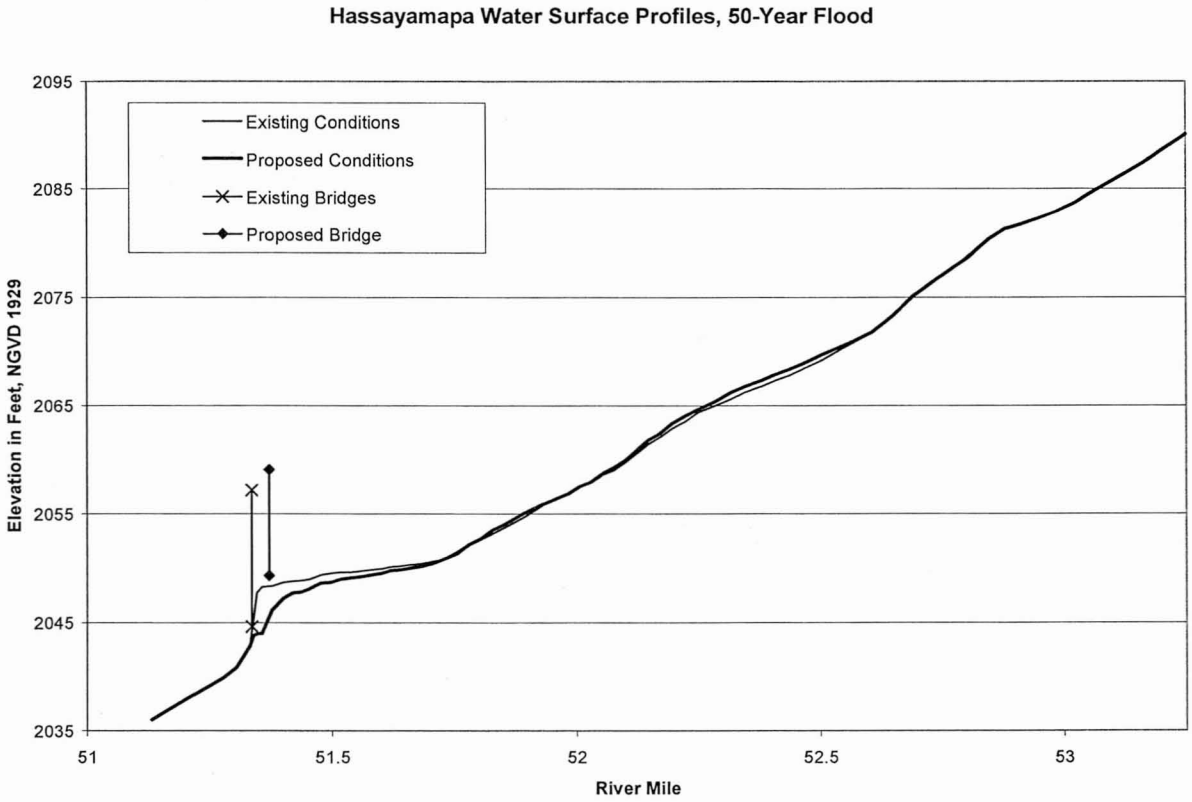

Figure 4.4. Hassayampa River 50-year water surface profiles

4.7. Hassayampa River Bridge Hydraulics

In the effective FIS model (the Cella-Barr HEC-2 model), the existing bridges over the Hassayampa River are not sufficiently high enough for the 100-year flood to pass underneath with the freeboard required by ADOT. However, the low chord elevation for the bridge given in the effective FIS model, 2043.3 feet, is incorrect. Recent field surveys verify that the actual bridges are higher than this. Nevertheless, even when the bridges are raised to the correct elevation, the 100-year flood does not pass under the bridge with any freeboard in either the corrected effective or in the existing conditions model, and a pressure flow situation develops.

For the proposed conditions, the 1936 bridge, which is the lower of the two existing bridges, will be removed. The 1962 bridge will be left in place. However, the 1962 bridge causes the water

surface profile to pressurize in HEC-RAS because the energy grade line is higher than the highest low chord of the bridge.

The 1962 bridge low chords, as surveyed in March of 2004, are 2046.79 ft. on the west side, 2047.42 ft. in the center, and 2046.74 ft. on the east side. These were all measured on the south side of the bridge. To prevent pressurization in the HEC-RAS model, the bridge will be physically raised by 2.0 feet. The existing and proposed low chords are shown in Table 4-6.

Table 4-6. Existing and proposed low chord elevations for 1962 bridge

	Current Low Chord (ft)	Proposed Low Chord (ft)
East Abutment (Phoenix Side)	2046.74	2048.74
Center Pier (high point)	2047.42	2049.42
West Abutment (Wickenburg Side)	2046.79	2048.79

The proposed low chord elevations shown in the rightmost column of Table 4-6 are to be the minimum heights for the low chord after the bridge is raised. After the bridge has been raised, the center of the bridge will be 0.43 feet higher than the energy grade line which is at 2048.99 feet. The bridge will also have 2048.74 – 2043.36 = 2.38 feet of freeboard from the 100-year water surface elevation.

The upstream cross-section from HEC-RAS of the proposed bridge is shown in Figure 4.5.

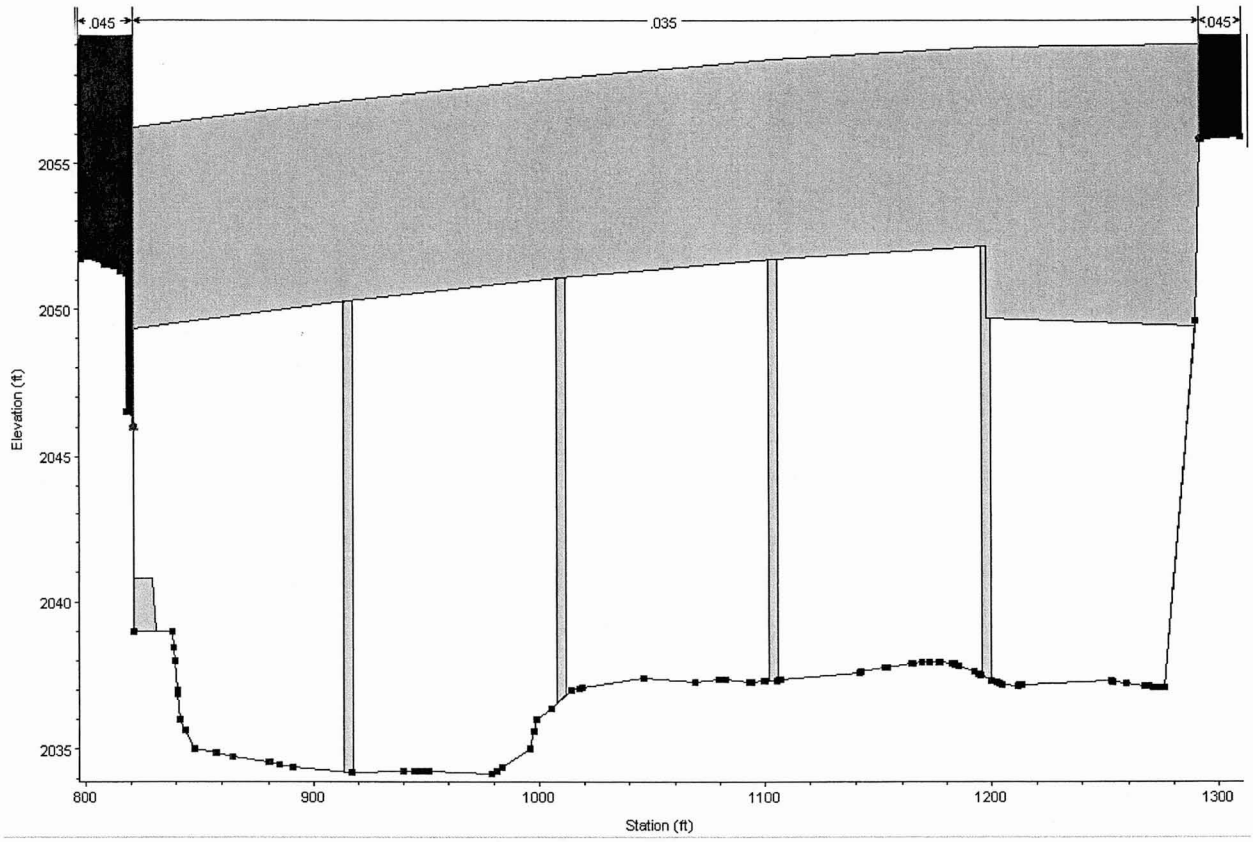

Figure 4.5. Proposed Bridge upstream cross-section from HEC-RAS

Table 4-7 shows the freeboard for the 100-year water surface profile at the proposed and 1962 bridge for the proposed condition.

Table 4-8 shows the freeboard for the 50-year water surface profile at the proposed bridge for the proposed condition. The 50-year freeboard criterion is not relevant to the 1962 bridge because that bridge will no longer be an ADOT bridge at the completion of the project.

Table 4-7. Freeboard at bridges: Hassayampa River 100-year profile, proposed condition

Cross-Section	Description	Water Surface Elevation (ft)	Low Chord Bridge Elevation (ft)	Freeboard, (Feet)
51.375	Proposed bridge, upstream face	2048.75	2049.30 (lowest low chord elevation)	0.55
			2052.18 (highest low chord elevation)	3.43
51.338	1962 bridge, upstream face	2046.36	2048.74 (future lowest low chord elevation)	2.38
			2049.42 (future highest low chord elevation)	3.06

Table 4-8. Freeboard at proposed bridge: Hassayampa River 50-year profile, proposed condition

Cross-Section	Description	Water Surface Elevation (ft)	Low Chord Bridge Elevation (ft)	Freeboard, (Feet)
51.375	Proposed bridge, upstream face	2046.12	2049.30 (lowest low chord elevation – ADOT freeboard criterion)	3.18
			2052.18 (highest low- chord elevation)	6.06

The existing conditions floodplain is shown in Figure 4.6. The proposed conditions 100-year floodplain is shown in Figure 4.7; however, the backwater flooding into Sols Wash is not shown in the proposed conditions floodplain because the proposed levee improvement to Sols Wash were not incorporated into the model, and the exact extent the future floodplain there is not known. It is expected, however, that there will be some flooding of Sols Wash, but that it will be prevented from spilling out the channel by levees that will be constructed there.

Figure 4.6. Existing conditions 100-year floodplain

Figure 4.7. Proposed conditions 100-year floodplain

4.8. Hassayampa River Floodway Encroachments

The floodway is that portion of the floodplain upon which no development is permitted which could cause a rise in the water surface elevations. Typically, this means that no buildings can be constructed in the floodway. The original floodway was laid out by Cella-Barr so that the encroached elevations (blocking of conveyance outside the floodway) would not be more than one foot higher than the floodplain elevations, per FEMA regulations.

GIS shapefiles for the existing floodplain and floodway were obtained from the FCDMC.

4.8.1. Floodway Surcharge

Because the hydraulic model uses newer (and higher resolution) topography than the effective model and because the road embankment encroaches on the floodway, it is not entirely clear what criterion should be used to check the encroached floodway water surface elevations. In a telephone communication with Cathy Regester of FCDMC on November 4, 2004, Ms. Regester indicated that what seemed to be the best criterion to evaluate the floodway was to compare the encroached proposed conditions elevations with the existing conditions water surface elevations. The increase in the encroached proposed conditions compared to the existing conditions should be less than or equal to one foot everywhere in the project reach.

However, a comparison of encroached proposed conditions versus existing conditions was also performed to ensure that the surcharge in this case would also not exceed one foot. Both surcharges are shown in Table 4-4.

For the area downstream of the 1962 bridge, an expansion of the existing floodway is required for the floodway profile to pass the 1962 bridge without pressurizing. An additional floodway expansion is proposed from about river mile 52.1 to river mile 52.5. Minor adjustments to the floodway on the west side (river left) were made so that the floodway limits would be at or outside the bank stations, and to avoid negative surcharges. The entire existing and proposed floodways are shown in Figure 4.8.

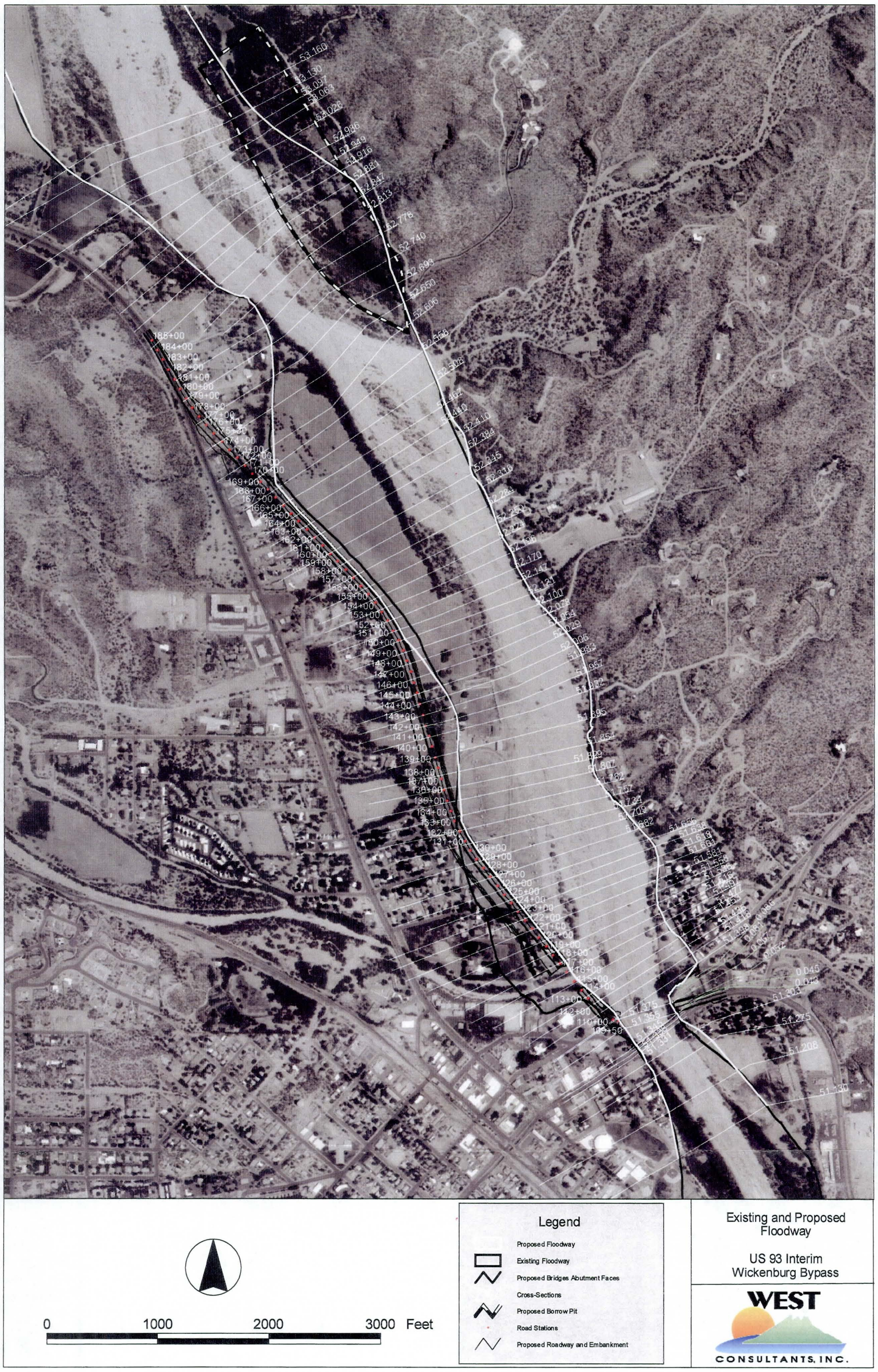

Figure 4.8. Existing and proposed floodway

5. Sols Wash Hydraulics

5.1. Freeboard for Proposed Bridge

A HEC-RAS model was created for the purposes of establishing freeboard values for the proposed Sols Wash Bridge. Cross-sections were taken at the upstream and downstream bridge faces, and several cross-sections were taken downstream of the Sols Wash Bridge, extending 139 feet downstream of the downstream bridge face. Although the downstream cross-sections are essentially in the Hassayampa River, it is conservative to include them because it makes the water surface elevation at the Sols Wash Bridge higher. Ineffective flow areas were set on the downstream cross-sections assuming a 2:1 expansion ratio of flow leaving the proposed Sols Wash Bridge.

The proposed bridge is skewed 25 degrees to the apparent flow direction of Sols Wash. The bounding bridge cross-sections had this skew applied to them in HEC-RAS, and the cross-sections downstream of the bridge also had appropriate skews applied to them, because the first few cross-sections are not perpendicular to the Sols Wash flow direction. The proposed Sols Wash Bridge opening is shown in Figure 5.1 along with the HEC-RAS cross-section near the bridge.

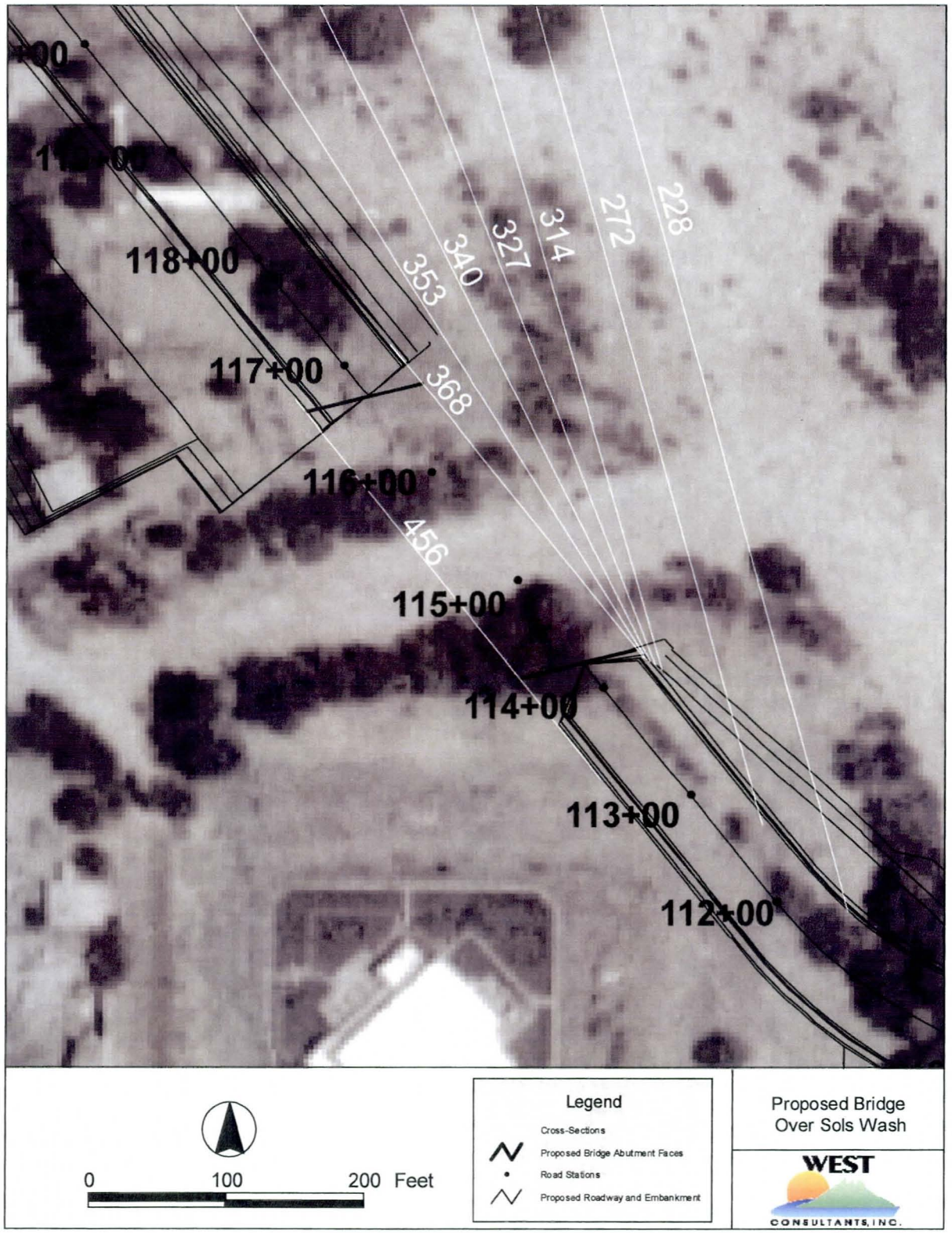

Figure 5.1. Cross-sections at proposed bridge over Sols Wash

Sols Wash encounters a backwater condition from the Hassayampa River if the Hassayampa River is flowing at the same time. That backwater condition was taken from cross-section 51.497 of the Hassayampa River model, which is the most upstream cross-section that crosses the opening of the proposed Sols Wash Bridge. The backwater condition was applied to the most downstream cross-section in the Sols Wash HEC-RAS model.

In order to establish the backwater condition, it was necessary to assume a certain flood level in the Hassayampa River. Because of the large difference in drainage areas (147 square miles for Sols Wash versus about 564 square miles drainage area for the Hassayampa River just upstream of Sols Wash), it was assumed that the large flood (50-year or 100-year) will not occur simultaneously on Sols Wash and the Hassayampa River. Various combinations of floods were simulated in order to determine the condition that causes the highest water surface elevation at the upstream face of the Sols Wash Bridge. These methods involved either combining a large flood on Sols Wash (50-year or 100-year) with a 10-year flood in the Hassayampa River, or combining a smaller flood on Sols Wash (10-year) with a larger flood on the Hassayampa River (50-year or 100-year). The 50-year / 10-year combinations were used to establish freeboard for the ADOT criterion of 50-year water surface elevation plus 3 feet. The 100-year / 10-year combinations were used to establish freeboard for criterion that the 100-year flood should not contact the minimum low chord of the bridge.

In all cases, it was the combination of the large flood on the Hassayampa River coupled with a smaller flood in Sols Wash which caused the higher water surface elevations at the upstream proposed Sol Wash Bridge face. In these situations, the backwater condition from large Hassayampa River events flooded much of the Sols Wash Bridge opening.

Table 5-1. Freeboard at proposed Sols Wash Bridge

Flood	Configuration	Discharge, Sols Wash (cfs)	Hassayampa River model discharge downstream of Sols Wash (to obtain backwater condition at Sols Wash) (cfs)	**Upstream water surface at proposed Sols Wash Bridge (ft)**	**Freeboard, in feet, with bridge low chord at 2053.31 (ft)**
50-year floods	Sols Wash 10-Year with Hassayampa River 50-year backwater	7019 (Sols 10-year)	54,019 = 7,019 (Sols 10-year) + 47,000 (Hassayampa 50-year)	2049.79	3.52
	Sols Wash 50-Year with Hassayampa River 10-year backwater	12,453 (Sols 50-year)	26,953 = 12,453 (Sols 50-year) + 14,500 (Hassayampa 10-year)	2048.32	4.99
100-year floods	Sols Wash 100-Year w/Hassayampa River 10-year backwater	15,045 (Sols 100-year)	29,545 = 15,045 (Sols 100-year) + 14,500 (Hassayampa 10-year)	2049.21	4.10
	Sols Wash 10-year w/Hassayampa River 100-year backwater	7019 (Sols 10-year)	71,000 (Hassayampa 100-year flood)	2051.85	1.46

Note that to obtain the backwater condition from the Hassayampa River, the discharge used in the Hassayampa River model was the sum of the discharges of the Hassayampa River flood and the Sols Wash flood, with one exception. The exception was that the for 10-year Sols Wash event combined with 100-year Hassayampa River event, the 100-year Hassayampa River flood was used to establish the backwater condition in the Hassayampa River model. It was judged that adding the 10-year Sols Wash flood to this 100-year Hassayampa River flood was excessively conservative, since it would have led to taking a backwater condition from a flood that had greater than a 100-year recurrence interval.

6. SEDIMENT TRANSPORT ANALYSIS

For the Hassayampa River sediment model, the main function is to predict long term changes in the profile of the bed. Factors considered include the borrow pit, the presence of the new embankment, and the construction of the new abutments for the proposed bridge.

6.1. Determination of Runoff Hydrographs for Sediment Models

6.1.1. Hassayampa River

To run the sediment transport model (HEC-6T) simulations, a series of hydrographs needed to be developed. Simulated future hydrographs are required to determine whether the project is likely to have a significant impact on the deposition or degradation in the reach. The objective is to correctly simulate both the peak and general hydrograph shape which would correspond to actual hydrographs that the reach might experience.

The historical record was used to generate the simulated future hydrographs. Ideally, the actual hydrographs of historical events should be used for the sediment model. Although there are years of data of hydrograph peaks, data on the shape and duration are difficult to obtain. There is no known hydrologic model for this basin. Detailed hydrographs are available only since the 1990's, when numerous gages were installed by both the USGS and FCDMC.

FCDMC pressure transducer gages have been in operation since March 1994 at the US-60, and since May 1996 at Morristown. Another FCDMC gage at Box Canyon has been in operation since 1991.

There are significant differences in the recorded hydrographs for a particular event between gages. The January 1993 event exemplifies this: in January 1993 the Box Canyon gage (USGS) registered almost no flow, while the Morristown gage recorded a peak of 26,300 cfs. For the September 1997 event, the Box Canyon gage peak was 3,421 cfs, the US-60 peak was 15,367 cfs, and the Morristown peak was 9,095 cfs.

The flow volumes calculated from the hydrographs are also significantly different, even from the US-60 gage to the Morristown gage downstream. For the September 1997 event, integrating the hydrograph shows that about 185 million cubic feet passed the US-60 gage. For the Morristown gage downstream, for the same event, only 51 million cubic feet passed the gage. Either there was some error in one of the two rating curves, significant measurement error, or most of the discharge infiltrated between the two locations.

The largest event for which a hydrograph is available is the January 1993 event at the Morristown gage. This event had a peak flow of 26,300 cfs. Based on the USGS hydrograph, the total volume of flow for this event is in just over 1 billion cubic feet. This volume is considerably larger than that for the September 1997 event (185 million cubic feet). The ratio of volumes between the two events is about 5:1 while the ratio of peaks is about 3:1. The difference in the ratios is a result of the longer duration for the January 1993 flood.

The following data were used to develop the hydrographs for the Hassayampa River HEC-6T sediment transport model:

- 48 annual peak flow records for the USGS Morristown gage near the Hassayampa River. These peak flow records are from 1939 to 1947 and from 1964 through 2003 (with 1 year missing). This gage is about 8 miles downstream of the US-60.

- Detailed hydrographs for all flood events, from the FCDMC, for all dates since 3/14/1994 at the US-60 gage.

- A 15-minute interval hydrograph obtained from the USGS, at the Morristown gage, for the event of 1/6/93-1/10/93. This event had a peak of 26,300 cfs, the third largest peak on record for this gage.

- The flow-frequency curve based on the FEMA floods (both from the current and 1977 study).

For dates after 3/14/1994, the FCDMC detailed real-time gage record was used to model the historical record. For 1993 and prior, when a peak flow was available from the Morristown gage, a simulated hydrograph was created using the following procedure:

- Using the flow-frequency relationship from the FFA analysis of Morristown, the exceedance interval of each flood was established. For example, the October 1984 flood of 26,700 cfs at Morristown is equivalent to the 5.3% (or 18.8 year) exceedance event.

- That exceedance interval was used to "look up" the corresponding flood peak at the US-60, using the flow-frequency curve from the FIS-specified floods. For example, the 18.8 year event at the US-60 is 24,070 cfs.

- The detailed hydrograph with the closest peak was identified (either one of the hydrograph since 1994 or the 1993 Morristown hydrograph). For example, for a peak of 24,070 cfs, the detailed hydrograph with the nearest peak is the January 1993 hydrograph from Morristown.

- The detailed hydrograph ordinates (flows) were all scaled up or down to make the peak of the hydrograph match the peak estimated from (b). For example, to obtain a peak flood of 24,070 cfs, the discharges on the 1993 Morristown hydrograph were all multiplied by 24,070 / 26, 300 = 0.915, resulting in the simulated 1984 hydrograph at the US-60.

- The end result for each year's peak flow was a simulated hydrograph with the same peak flow.

- Combining these simulated hydrographs with the 1994 and more recent gage records from the FCD gage led to a series of hydrographs that simulates about 48 years of gage record. It should be noted that the highest simulated peak flow from this

combined series of events is 54,984 cfs, which comes from the simulated hydrograph of 9/5/1970 for Morristown. The Morristown peak on this date was 47,500 cfs, which interpolates as the 64.6-year flood at Morristown. Looking up the 64.6 year flood on the Hassayampa River Flood Frequency curve yields a peak of 54,984 cfs.

- In addition to a 48-year simulation, another hydrograph using the 100-year peak flow of 71,000 cfs was added to the end of the simulated flow record. This series of hydrographs was labeled as the "long-term" flow simulation that was run through HEC-6T.

6.2. Sediment Model Parameters

6.2.1. Boundary Condition

Using normal flow considerations, an elevation-discharge rating curve based on the HEC-2 models were developed to establish the downstream boundary conditions.

6.2.2. Manning's Roughness

To arrive at conservative estimate for the sediment transport analysis the Manning's n-values were lowered to a value of 0.025 in the channel and 0.035 for the overbanks. The lowered Manning's n-values will result in increased erosion resulting in lower values of average bed and thalweg elevations. This, in turn, will provide a higher value for the scour depth building in conservatism in the estimates.

6.3. Sediment Data

The decisive factor in selecting the proper sediment transport function was based on available bed gradation and maximum grain size. Bed material in the project location is comprised of sand and gravel, making it necessary to use an appropriate transport function. The transport function used in the study is based on Yang Stream Power (Thomas, W.A., 2002). This function accounts for sand and gravel, giving a higher, realistic measure of total sediment concentration (and thus total sediment load).

6.3.1. Bed Sediment Characteristics

The sediment gradation curves for various samples are presented in Figure 6.1. The sediment samples are discussed further in Appendix C.

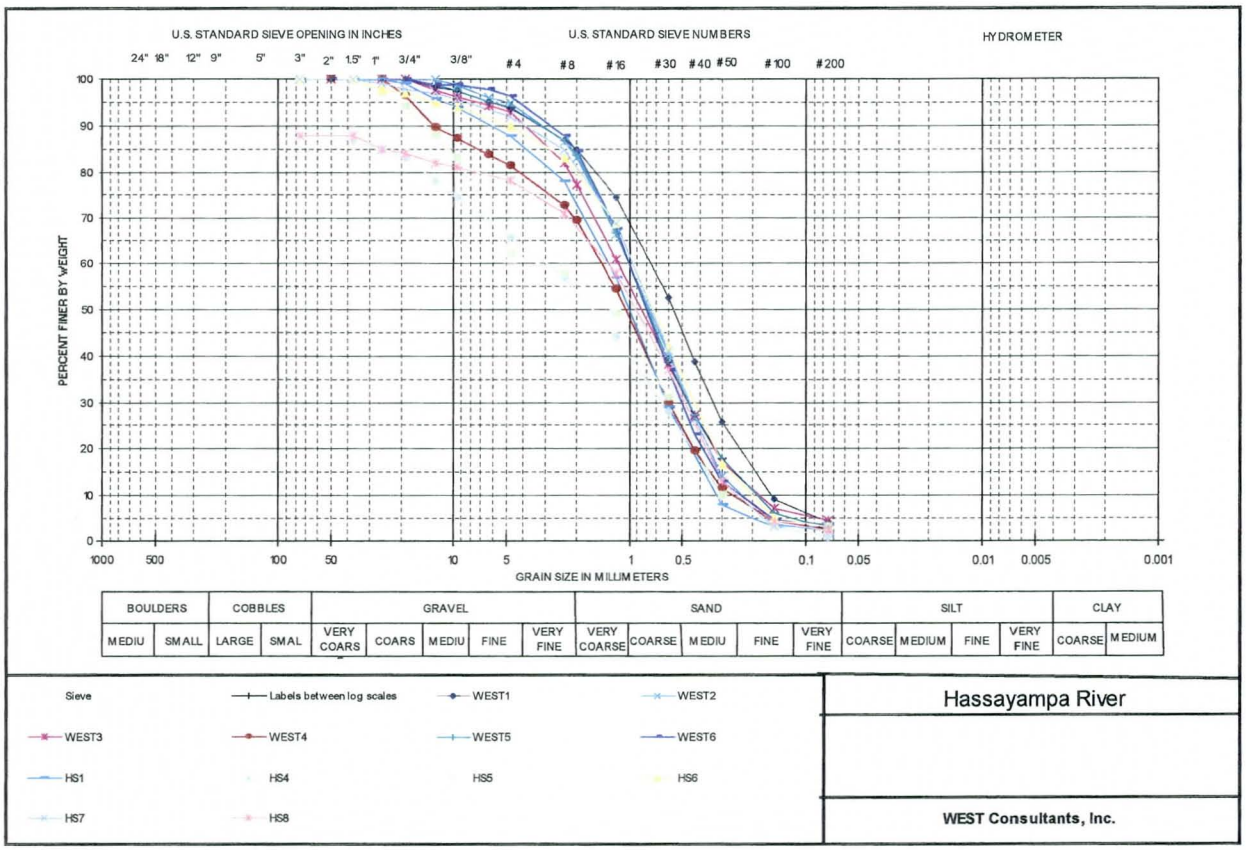

Figure 6.1. Sediment gradations at various locations along the Hassayampa River

Different gradations were used in the sediment model, assigned by their location to the appropriate cross-sections in the sediment model. As can be seen in Figure 6.1, the D_{50} ranges from just over 0.5 mm to just over 1 mm. The median D_{50} of the different samples is about 0.8 mm.

6.3.2. Inflowing Sediment Load

The sediment transport model cannot be directly calibrated to historical conditions because detailed historical bed elevation data are not readily available. The HEC-6T model requires input of the bed material load at the upstream limit of the project reach for the entire range of discharges. For this purpose, the HEC6-T model developed was run with the recirculation option ($RE, see Thomas, W.A. 2002). This model considers the reach in the upstream part of the study reach where the recirculation of sediment load is performed until steady sediment inflow is obtained. The values obtained in this manner are input into the present HEC-6T model at the upstream most cross-section located at 53.72.

6.3.3. Movable Bed and Erosion Limits

In general, sediment dynamics tend to be more significant within the active channel, where the bed can either degrade or aggrade in response to erosion or deposition. The overbank areas tend to be more stable and normally are free of erosion, but can experience deposition. HD records

were used to specify a bed sediment depth of 25 feet for all cross-sections. Movable bed limits were identified at the boundary of the main channel in HD records. The movable bed limits extend beyond the defined bank stations. During high flows, significant deposition and scour was expected to occur within the movable bed limits but not expected to extend to the overbank areas.

Average bed elevations were generated using movable bed limit bank limits. This allowed wetting of the movable bed cross-sections and provides an average elevation across the cross-section.

6.4. Sediment Transport Results

The HEC-6T model simulations were performed to predict the long-term degradation. Simulations were performed for the following scenarios: a) existing geometry b) proposed geometry without borrow pit, and c) proposed geometry with borrow pit. The purpose of modeling the borrow pit was to determine the possible tailcut in the downstream direction, which could impact the scour conditions for the soil cement protection of the road embankment and to estimate the depth and extent of the headcut in the upstream direction.

The proposed conditions HEC-6T model was developed using the existing conditions model as the base and by incorporating the new bank alignment as encroachments. The rest of the model is identical to the existing conditions model. The proposed-with-pit conditions model was also based on the corresponding HEC-RAS model with the flow being diverted into the pit in the upstream end of the model.

The results are presented in Figure 6.2 and Figure 6.3 which show the changes in the average bed elevations and the thalweg respectively. The results indicate that the average bed elevations drop by about 3 to 4 feet while the thalweg drops by 4 to 5 feet. These results indicate a conservative estimate due to lowered Manning's n-values used in the computations.

The results indicate that the average bed is lowered by 1 to 2 feet near the bridge location while the thalweg is lowered by an additional foot. The influence of the proposed conditions and proposed with pit conditions is not very significant in the reach downstream of the bridge. This can be seen by comparing the results of the existing conditions case with the proposed conditions and proposed-with-pit conditions cases. A difference of less than a foot is observed which is probably close to the order of numerical accuracy of the computations performed.

The presence of the pit in the upstream end of the model results in the following: a) deposition within the pit, b) a head-cut at upstream of the pit, and c) a tail-cut of about half a mile in the downstream side of the pit. The influence of the pit is mostly restricted to a region about half a mile in the downstream direction. At the upstream limit of the model, the average bed-change from headcut (versus existing conditions) is about 2 feet due to the proposed borrow pit. The model was not extended further upstream because it would extend the model beyond the confluence with Martinez Wash, and the distribution of flows between the Martinez Wash and the Hassayampa River are not known and cannot be estimated reasonably without a hydrologic model.

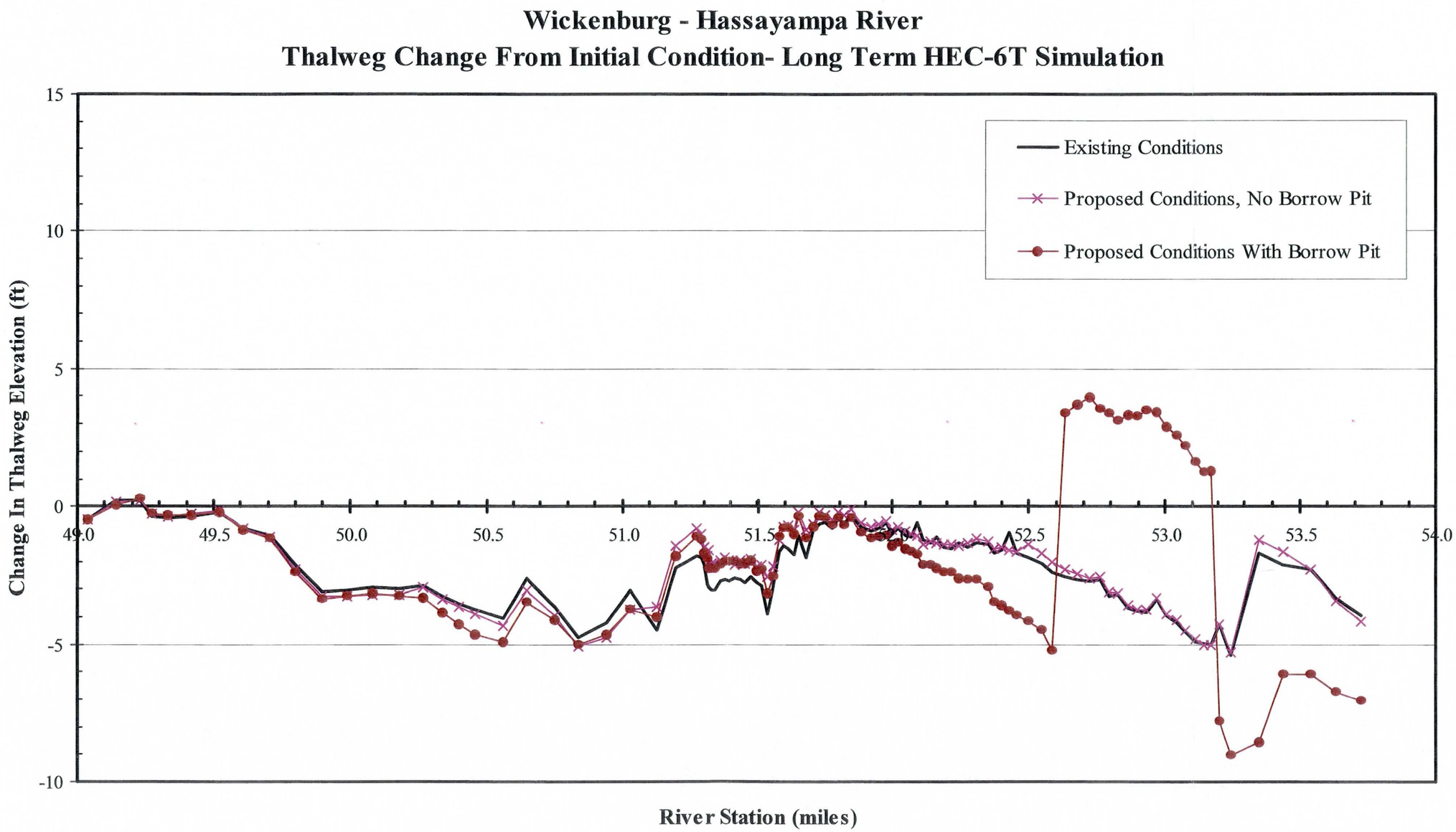

Figure 6.2. Hassayampa River sediment models thalweg change from initial conditions

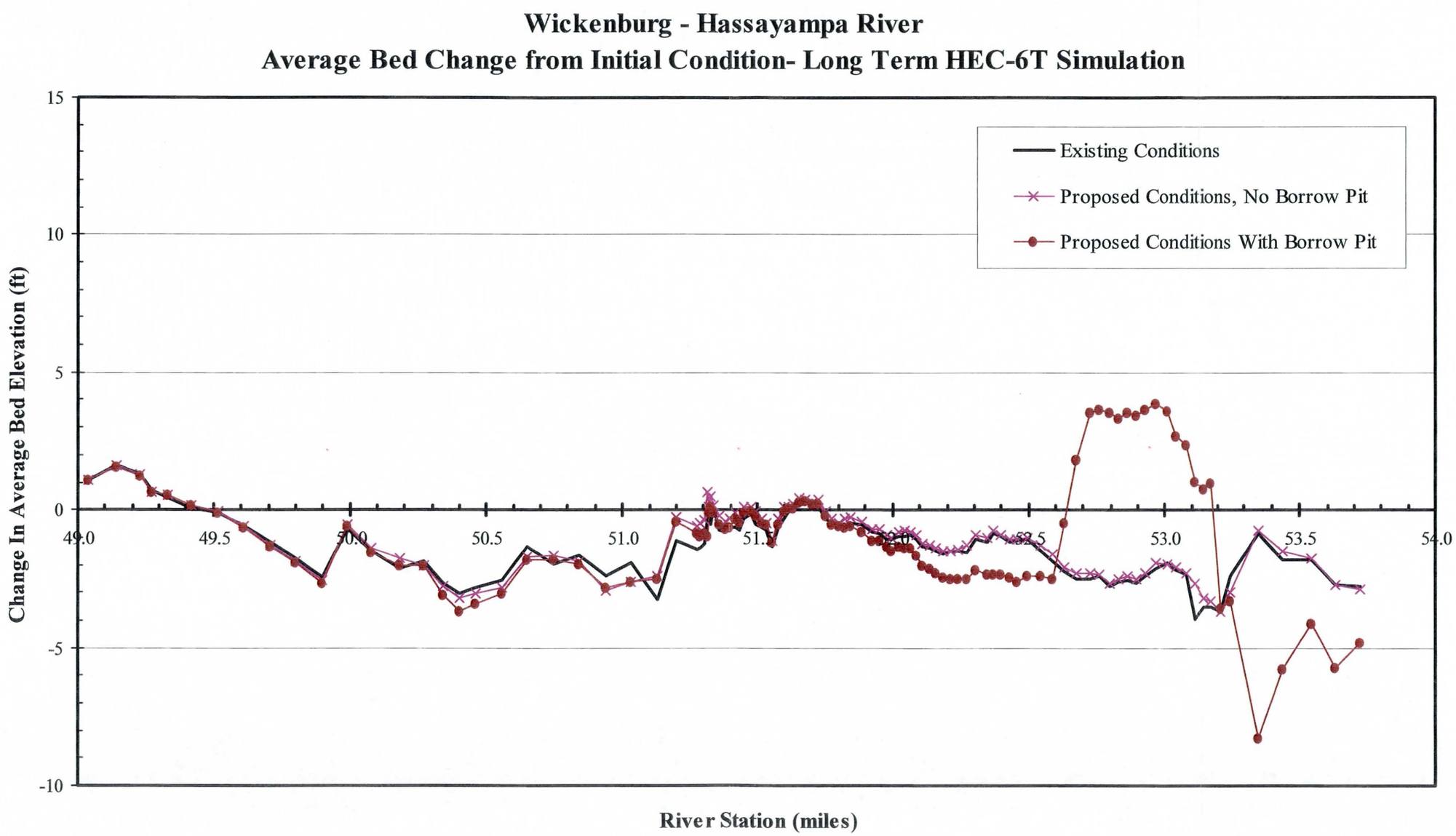

Figure 6.3. Hassayampa River sediment models average bed change from initial conditions

7. ABUTMENT/BANK PROTECTION MEASURES

The bank protection is to be soil cement. On the west side of the river, the soil cement will have its downstream limit just upstream of the existing bridge abutments and its upstream limit where the proposed embankment ties into a bluff. The toe downs on these embankments were designed to withstand the 100-year event. Furthermore, because the embankments are expected to be certified as levees in the future, the Flood Control District of Maricopa County's Guidelines for the factor of safety were used. Section 10.4.2 of the FCDMC consultant guidelines (Third Edition, December 1 2003, Revision 1) reads:

> *10.4.2 Toe-down elevations for bank protection shall be based on the sum of all scour components determined to occur for the 100-year peak flow, times a factor of safety of 1.3, and shall be measured from the channel thalweg elevation. A factor of safety of 1.5 shall be used in cases where only one component of scour is present (usually local scour).*

Since all the scour calculations involved multiple components of scour, the 1.3 factor of safety was adopted.

On the east side of the river, the soil cement serves the function of protecting the bridge embankment from scour, and has its downstream limit just downstream of the proposed abutment. This embankment was designed to ADOT criterion. Its toe down was designed to withstand the 50-year flood event, and a 1.1 factor of safety was applied.

7.1. Soil Cement and Embankment

Most, but not all, of the road embankment is designed to function as a levee. The portion of the road embankment from the upstream limit to approximately road station 154+50 will not function as a levee. For that reach, the top of the bank protection elevation was set at the 100-year flood elevation. There is a roadside dike as shown in Figure 7.1 on the west side of the embankment at road station 154+27 at elevation 2070 feet which prevents flow of any backwater from north of the dike to the south. The dike elevation of 2070 feet is more than 4 feet higher than the 100-year water surface elevation of the Hassayampa River at road station 154+27.

The remainder of the road embankment, from downstream of road station 154+50 until the proposed Hassayampa River Bridge, is designed to function as a levee (except for the opening at the Sols Wash tributary). The freeboard to top of the road embankment is in excess of 3 feet along the entire embankment. At the upstream limit of the embankment, the freeboard to the top of the embankment is in excess of 15 feet. The soil cement bank protection is designed with a minimum of 3 feet of freeboard south (downstream) of road station 147+20, and to be at or above the 100-year water surface elevation for road stations north (upstream) of 147+20.

On the west bank, for the first 100 feet upstream of the proposed bridge of the Hassayampa River, and until the downstream limit of the soil cement, freeboard to the top of soil cement (or other concrete bank protection in the transition regions near the bridge) was set at a minimum of 4 feet. In the area under the proposed bridge over the Hassayampa River, the soil cement height

was set per ADOT criterion on the east side of the bridge which is that the top of soil cement should be at least as high as the 50-year flood.

For all of the bank protection along the road embankment, and through the proposed Hassayampa River Bridge on the right (west) abutment, the toe down elevation of the bank protection was designed to withstand scour from the 100-year event.

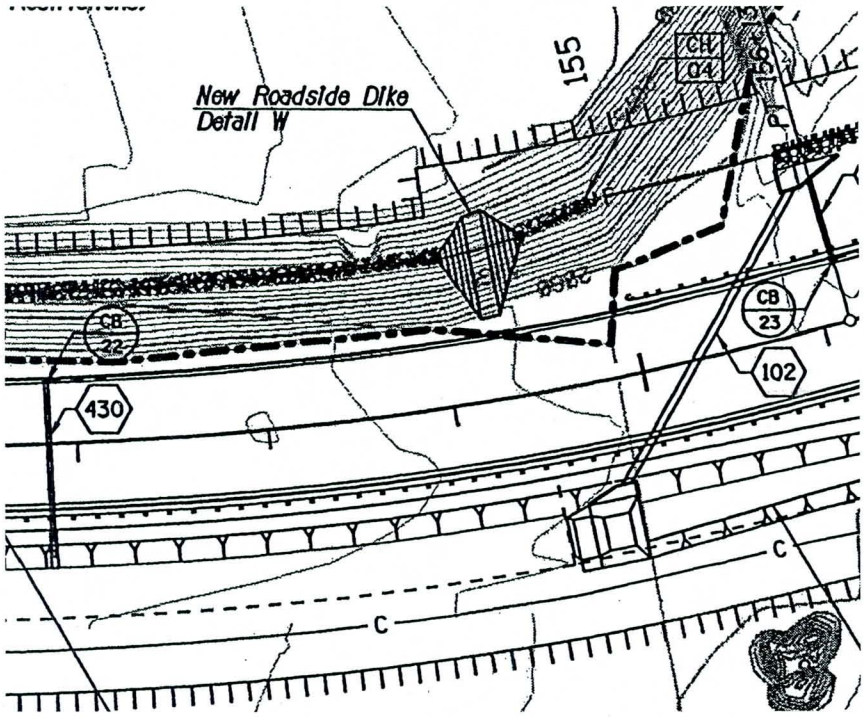

Figure 7.1. Sketch of the new roadside dike

There are three culverts along the road embankment/levee system. Culvert 101 is located just downstream of the proposed bridge at cross-section 51.350. The upstream invert elevation of this culvert is 2041.01 feet. A flap gate is going to be installed on this culvert because without one, everything to the west of the culvert would flood to an elevation of 2046.6 feet.

Culvert 401/402 is located just upstream of the proposed bridge at cross-section 51.465. This culvert is designed to take runoff from the roadway and dump it into the Hassayampa River. This single culvert is not a traditional culvert in that it has two drop inlets (pipe 401 and pipe 402) from the roadway. The lowest elevation of the grate for pipe 401 is 2055.60 feet while the lowest elevation for the grate for pipe 402 is 2055.76 feet. A flap gate will not be installed on this culvert.

Culvert 430 is located at the upstream limit of the levee at cross-section 52.168. This culvert is designed to take runoff from the roadway and dump it into the Hassayampa River. The lowest elevation of the grate for the drop inlet of culvert 430 is 2067.84 feet. A flap gate will not be installed on this culvert.

7.2. Sols Wash: Toe Down Upstream of Proposed Sols Wash Bridge

Scour that occurs on the upstream side of Sols Wash Bridge is relevant for the upstream cutoff wall on the bridge. It is also relevant for bank protection which will be placed on the south side of Sols Wash, just upstream of the bridge, and for the bank protection just off the left side of bridge which wraps around the embankment. Details for the various scour calculations can be seen in Appendix A.

Because the bridge is a superbox, with a concrete bottom, the abutment and pier scour equations do not apply. Zeller general scour was calculated as 4.1 feet for the 100-year event. In the proposed condition, it assumed that the planned improvements to Sols Wash, which will essentially prevent overbank flow upstream of the bridge, will result in little contraction scour. Therefore, the calculated Zeller general scour of 4.1 feet was used as the general scour. At the time of the design of the Sols Wash improvements, both the general scour and contraction scour should be recalculated based on the new configuration to determine if there has been any increase in general scour.

Antidune scour was also considered, based on the velocities at the bridge opening, and was found to be 1.9 feet.

No long-term scour was considered as Sols Wash is an aggrading reach.

The formula therefore used was:

*Toe down depth = (General scour + Bedform scour) * Factor of safety*

where the factor of safety is equal to 1.3. The various scour components are shown in Table 7-1 and the desired toe down depth is calculated to be 7.8 feet.

Table 7-1. Toe down upstream of Sols Wash (for cutoff wall and bank protection near the bridge)

	Depth of Scour (ft)	Comment
General Scour (Zeller)	4.1	For 100-year event
+ Bed Form Scour (Antidunes)	1.9	For 100-year event
Total =	6.0	
Plus 30% Factor of Safety	1.8	
Desired Toe Below Thalweg	**7.8**	

The current thalweg at the upstream side of the proposed Sols Wash Bridge is 2040.0 feet. Therefore, it is recommended that any bank protection on the right bank of Sols Wash near the bridge, and bank protection on the east side of proposed embankment, just left of the proposed Sols Wash Bridge, have a toe down elevation of 2040.0 – 7.8 = 2032.2 feet. However, the bank protection should be recalculated once the future improvements to Sols Wash have been designed.

The upstream cutoff wall for the Sols Wash Bridge will be 6.0 feet deep, which starts at the bottom of the superbox elevation of 2036.0 feet. Therefore the upstream cutoff wall will toe down to 2036.0 – 6.0 = 2030.0 feet. This is lower than the calculated toe down elevation by 2.2 feet.

7.3. Toe Down for Bank Protection, West Side of the Hassayampa River

On the right bank of the river (the road embankment), the toe down of the soil cement was calculated separately for three separate reaches. The reaches used were:

(1) From the upstream limit of the soil cement to station 147+30

(2) From station 147+30 to Sols Wash

(3) From downstream of Sols Wash to the proposed bridge over the Hassayampa River

The general formula used to calculate toe down was:

Toe down = (General scour +

Bend scour +

*Bedform scour) * Factor of safety +*

Long-term degradation

where the factor of safety is equal to 1.3

No term was included for thalweg formation because the cross-sections are natural channel cross-sections taken from detailed topography, so the thalweg of the channel is already present in the cross-section. Although the thalweg is currently well away from where the road embankment will be, all scour was measured below the existing thalweg of each cross-section, under the assumption that in the worst case the thalweg could migrate over to the new embankment.

Most bank protection scour parameters were taken from a version of the hydraulic model that had the channel n-value reduced from n = 0.035 to n = 0.025, and the channel velocities (not average velocities) from HEC-RAS were used to calculate the scour. This was done to obtain higher velocities for the scour calculation, which will lead to more conservative scour values. The only exception is that contraction scour (a form of general scour) was calculated for the bank protection and bridge scour at and near the bridge on the west side, that contraction scour was calculated using the base model with n = 0.035 in the channel. The contraction scour calculation depends on water depths and not on velocities so the lower n-value model would not be conservative in this case.

The toe down was calculated by cross-section, then translated to road-stations when required using the intersections of the cross-sections with the embankment.

The toe downs were then smoothed to provide a constantly sloped line based on road station, with the toe down at each location equal to or greater than the toe downs calculated on a cross-section by cross-section basis.

7.3.1. Hassayampa River General Scour

The Zeller equation for general scour (Simons, Li & Associates, 1985) was used:

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

where:

y_{gs} = general scour depth (ft)

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

V_m = average velocity of flow (ft/s)

y_h = hydraulic depth of flow (ft)

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

(Should y_{gs} become negative, assume the general scour component is equal to zero).

The Zeller general scour equation was evaluated at every cross-section where bank protection will be used. For the reach upstream of Sols Wash, the results were reasonably uniform across the sections, with an average value of 3.4 feet and a maximum value of 4.4 feet. The average value of 3.4 feet was used for general scour for the Hassayampa River above Sols Wash.

7.3.2. Hassayampa River Bend Scour

Bend scour should be considered in areas where the embankment may abut against an outer bend in the channel or a bend in the entire floodplain. This can occur due to the natural meanders of the channel moving, or due to bends already built into the proposed conditions geometry.

Zeller's bend scour equation (Simons, Li & Associates, 1985) for bend scour was used:

$$\Delta Z_{bs} = \frac{0.0685 Y V^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left[2.1 \left(\frac{\sin^2 \frac{\alpha}{2}}{\cos \alpha} \right)^{0.2} - 1 \right]$$

where:

ΔZ_{bs} = bend scour component of total scour depth (ft)

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

Y = maximum depth of upstream flow (ft)

Y_h = hydraulic depth of upstream flow (ft)

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

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

The relationship (Simons, Li & Associates, 1985):

$$r_c / W = (\cos \alpha) / (4 (\sin^2(\alpha/2))$$

where:

r_c = radius of curvature to centerline of channel (ft)

W = channel top width (ft)

was used determine the ratio $\cos \alpha / \sin^2(\alpha/2)$ on the basis of r and the channel top width.

In the first significant event, the proposed borrow pit is expected to capture the channel, that is, the main channel of the river will flow through the proposed borrow pit instead of on its current course. This will happen because the proposed borrow pit was estimated to be 10 feet deeper than the current channel and almost adjacent to it; any significant event will cause the channel to re-route through the borrow pit.

Moving downstream of the borrow pit, it appears possible that the channel could migrate over to the opposite, or west, side of the floodplain. If the channel should migrate over, that channel would then need to curve back to the west in order to go through the proposed bridge. This means an outer bend of the channel could be abutting against the bank protection. The position of the borrow pit, which is on the east side of the Hassayampa River's floodplain, means that the lateral shift in the channel to the west side of the floodplain will require some distance along the channel before it can reach the other side. It was estimated that the most upstream point at which the main channel could reach the opposite side and begin its bend back to the east would be at road station 147+00.

Examination of the natural bend radius of long arcs of the river in the project reach as well as in areas upstream and downstream indicate that the natural radius of curvature in the meanders is about 7,000 feet. The top width in the area where the bend scour could be an issue is relatively constant, and averages about 1,500 feet. Although the natural meander radius is about 7,000 feet, this was lowered slightly for use in the Zeller bend scour equation to be more conservative.

It was decided that a bend radius of 5,000 feet should be used in conjunction with a total flow width of 1,500 feet.

To get parameters required for Zeller's bend scour equation, the velocity and energy slope were taken from cross-section 52.147, using the model which has the Manning's n-value in the channel equal to 0.025. This cross-section is just upstream of the reach where it is believed that bend scour could occur, and it also has a higher velocity than any of the sections nearby. The hydraulic depth of 5.31 feet and maximum channel depth of 8.44 feet was also taken from this cross-section. Using Zeller's bend scour equation with average velocity of 10.78 ft/s yields a bend scour of 2.4 feet. Although Zeller's equation specifies that mean velocity be used, the scour was also checked using the channel velocity of 14.96 ft/s, which yielded a bend scour of 3.21 feet, though this may be excessively conservative since the channel velocity is being used.

Another potential for bend scour comes not from the natural meander radius of the channel but a bend in the entire floodplain which would be taking place at about road station 132+00, which is at about cross-section 51.757. In this area both the road embankment on the right side of the channel and the bluff on the left side of the channel are both turning left. Circumscribing circles onto the channel, it appears that this bend has a radius of about 3,700 feet. When applying this bend radius and the other parameters needed for the Zeller bend scour equation at the approach cross-section of 51.895, the Zeller bend scour is 2.8 feet.

Therefore, the bend scour from the bend near road station 132+00 appeared to cause more severe bend scour than the natural bends in the river might. It was determined that the reach from downstream of road station 147+30 until the confluence with Sols Wash should have a bend scour of 2.8 feet applied.

Another place where a possible bend may be taking place is through the bridge opening, which lies toward the right side of the floodplain. On the upstream side of the proposed conditions model, the new embankment constrains the right floodplain. On the downstream side there is a bluff on the right side of the channel. Meanwhile, on the left bank, the road embankment (from the US-60 approach from the east) juts into the floodplain; on the both the upstream on the downstream side there is a considerable portion of floodplain that is obstructed by that embankment.

In this configuration, there seems to exist a potential that the flow will need to turn to get through the opening, and that the outer radius of this turn could abut the soil cement. It is difficult to tell if this effect will actually occur, but the offset of the bridge opening with respect to the floodplain opens the possibility.

It is prudent to account for this potential on the bank protection on the west side of the embankment. The region where this potential turning could take place would seem to be in the region downstream of Sols Wash. Shortly upstream of Sols Wash, it appears the effect might be possible as well, but in this reach, a bend scour of 2.8 feet has already been applied for reasons explained previously. In the reach downstream of Sols Wash, a bend radius of 5,000 feet was applied to account for possible bending of the flow. Calculating the Zeller bend scour using different approach cross-sections ranging from 51.656 through 51.516 resulted in a maximum

bend scour of 4.7 feet. The determination was made to apply a bend scour of 4.7 feet to the reach downstream of Sols Wash and through the proposed bridge opening.

7.3.3. Antidunes

Based on the flow regime, it appeared that antidunes are possible.

The formula for antidune height is:

$$h_a = 0.14\frac{2\pi V^2}{g} = 0.027V^2$$

where:

h_a = antidune height from crest to trough (ft)

V = channel velocity (ft/s)

g = gravitational constant, 32.2 ft/sec^2

The scour depth due to antidunes is half the antidune height, or

Antidune scour = $h_a / 2 = (0.027/V^2) / 2$

The antidune scour averaged 1.9 feet in the reach upstream of Sols Wash, with a maximum height of 3.0. The average antidune scour of 1.9 feet was applied for the reach upstream of Sols Wash.

In the reach downstream of Sols Wash, the antidune scour was less than 1 foot, except at the upstream bridge cross-section, where it was 2.2 feet. In the cross-section downstream of the proposed bridge, the antidune scour was 3.0 feet. An antidune scour value of 2.2 was applied in the region from Sols Wash until the downstream limit of the soil cement.

7.3.4. Hassayampa East Embankment Summary of Toe Down Scour Calculations

Table 7-2. Toe down from upstream limit of soil cement to road station 147+30

	Depth of Scour (ft)	Comment
General Scour (Zeller)	3.6	Average upstream of Sols Wash
+ Bed Form Scour (Antidunes)	1.9	Average upstream of Sols Wash
Total =	5.5	
Plus 30% Factor of Safety	1.7	
+ Long Term Degradation	3.0	
Desired Toe Below Current Thalweg	**10.2**	

Table 7-3. Toe down from road station 147+30 (at upstream end) to Sols Wash (at downstream end)

	Depth of Scour (ft)	Comment
General Scour (Zeller)	3.6	Average upstream of Sols Wash
+ Bend Scour	2.8	Zeller Bend Scour using 3700 foot radius
+ Bed Form Scour (Antidunes)	1.9	Average upstream of Sols Wash
Total =	8.3	
Plus 30% Factor of Safety	2.5	
+ Long Term Degradation	3.0	
Desired Toe Below Current Thalweg	**13.8**	

Table 7-4. Toe down from the southern abutment of proposed Sols Wash Bridge to downstream limit of bank protection

	Depth of Scour (ft)	Comment
Contraction Scour	8.7	For the 100-year flood, upstream cross-section 51.477
+ Bend Scour	4.7	Zeller Bend Scour using 5,000 foot radius
Total	13.4	
Plus 30% Factor of Safety	4.0	
+ Long Term Degradation	3.0	
Desired Toe Below current thalweg	**20.4**	

At approximately road station 170+00, the embankment encounters a bluff to the north which constitutes the right bank of the river. The upstream limit of the soil cement should be tied into this bluff such that the soil cement is completely buried, top to bottom, for a distance of at least 25 feet.

Note that for the toe downs shown in Table 7-4, from the southern abutment of the proposed Sols Wash Bridge to the downstream limit of bank protection, antidune scour was not included as one of the scour components. Antidune scour was not included in this calculation because it is believed that antidune scour cannot occur at the same time as the bend scour and the large contraction scour which have already been applied. In the pier scour guidelines from the FHWA, for example, only very large dunes are considered for increasing pier scour. Antidune scour for piers is considered via a change in the K_3 factor. That K_3 is not increased due to dunes unless the dune height exceeds 3 meters. The antidunes in this reach are well below that threshold.

7.3.4.1. Potential Scour Hole Due to Outflow from Sols Wash

The region downstream of Sols Wash needs to be protected from scour due to Sols Wash floods, as the floods exit Sols Wash Bridge. These floods could cause a scour hole to develop on the downstream side of the bridge, which could undermine the bridge.

An analysis was done to check for the possible depth of a scour hole due to water exiting from Sols Wash. Simons, Li & Associates (1986) published a report with equations to determine the scour depths downstream of grade control structures. Similar equations, with corrections, appear in Bormann (1986). The situation is approximately equivalent to a drop from a grade control structure with a 1:1 downstream face slope as shown in Figure 7.2. The Simons, Li & Associates report (1986) provides an equation for the scour hole depth for this situation:

$$y_S = 0.54\, q^{0.67}\, (h / y_t)^{0.158}\, (S_r)^{-0.134}$$

where:

y_s = depth of scour (ft)

q = discharge per unit width (cfs/ft)

h = drop height (ft)

y_t = downstream flow depth (ft)

S_r = relative submergence = $(y_t - h) / y_1$

y_1 = upstream flow depth (ft)

Equation applies for $S_r < 0.99$.

Maximum scour depth occurs at a distance downstream of approximately 6 times the depth of scour.

Scour hole extends a distance downstream of approximately 12 times the depth of scour.

Also note that the factor in the Simons, Li & Associates (1986) document is 0.483 rather than 0.54; the 0.54 value is from Bormann (1986) and is an updated value. The 0.54 value used is more conservatively than the 0.483 value.

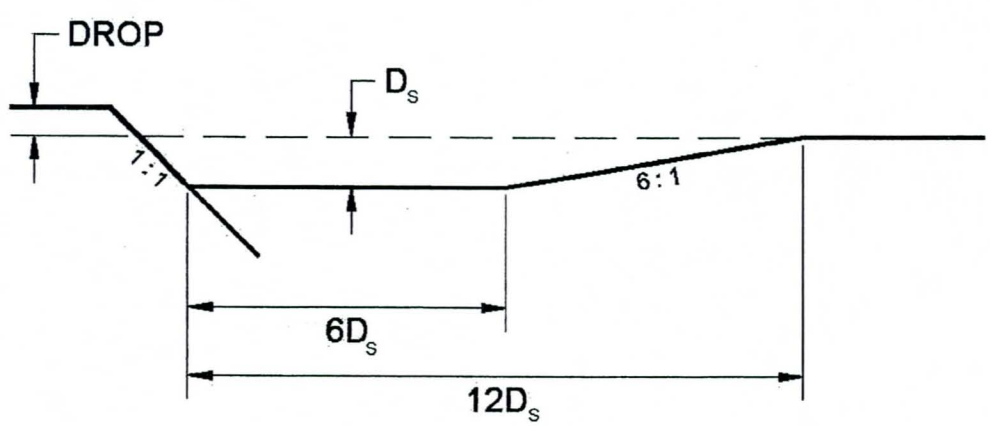

Figure 7.2. Sketch of scour hole downstream of drop

The following assumptions were made in the scour hole analysis:

- That the Hassayampa River thalweg had migrated over to be adjacent to the Sols Wash Bridge outlet. The elevation of the thalweg at cross-section 51.477, which is the more downstream Hassayampa River cross-section that crosses through the Sols Wash Bridge opening, is 2036.39 feet.

- That 3 feet of long-term scour has occurred on the Hassayampa River, and that the thalweg is therefore 2036.39 – 3.0 feet = 2033.39 feet.

- That the Sols Wash Bridge, during the peak flow of a large event, would scour out to the bottom of the superbox. The bottom of the superbox elevation is 2036.0 feet.

Therefore, the flow out of the Sols Wash Bridge would be facing a drop of 2036 = 2033.39 feet, or 2.61 feet. For the 100-year discharge of 15,045 cfs, it was found this would cause a scour hole 10.79 feet deep. Including a safety factor of 30%, the scour elevation would be 2033.39 – 10.79 * 1.3 = 2019.36 feet. This was rounded to the nearest half foot or 2019.5 feet for design scour elevation.

7.3.5. Soil Cement Bank Protection Toe Downs by Reach, at West Embankment and Under West Side of Proposed Bridge

The toe downs were calculated separately for four reaches. Going upstream, those reaches are: (a) The toe down from the downstream limit of the bank protection until the southern limit of the bridge opening at Sols Wash (about road station 114+23); (b) the region across from bridge opening at Sols Wash from about road station 114+23 to 116+71; (c) from the northern limit of the bridge opening at Sols Wash about road station 116+71 until road station 147+30; and (c) from road station 147+30 until the upstream limit of the project. For the reaches upstream of Sols Wash the toe downs are shown graphically in Figure 7.3.

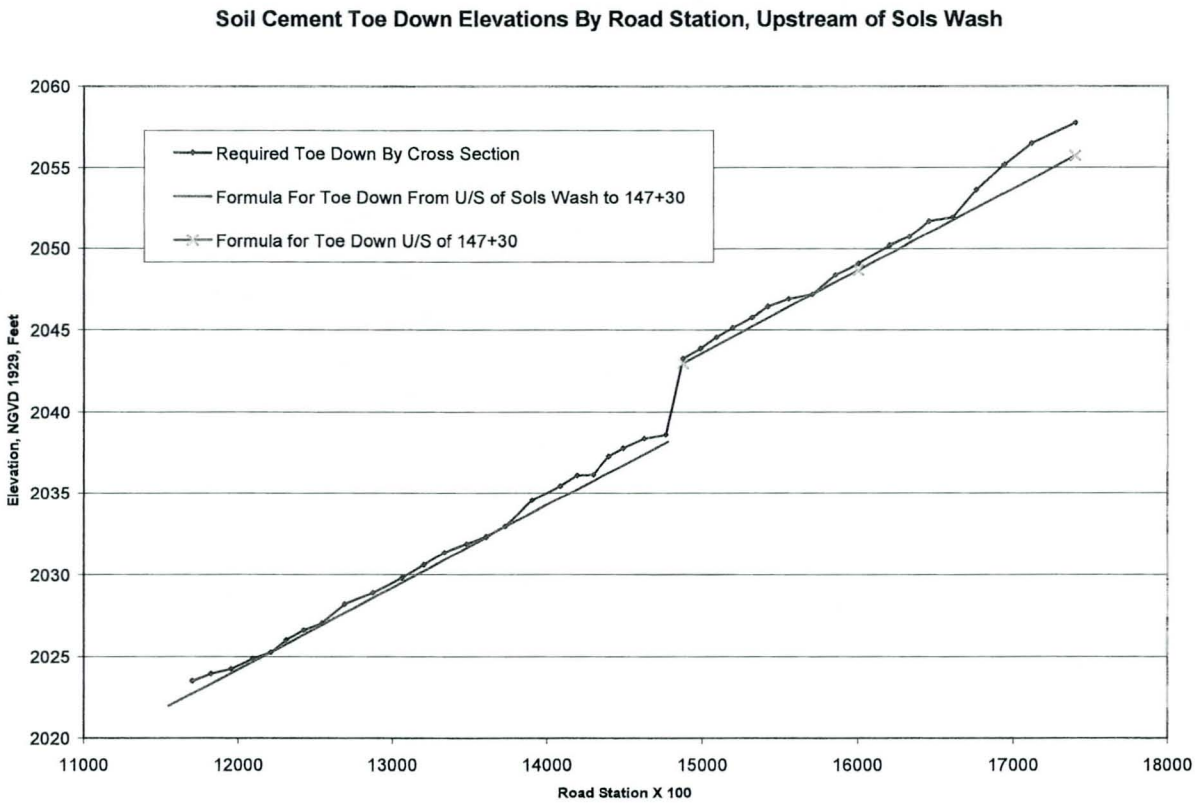

Figure 7.3. Toe downs upstream of Sols Wash

The formulas to calculate the toe downs are:

For Reach #1 (downstream of Sols Wash):

For the portion of the embankment downstream of Sols Wash, until the downstream limit of the soil cement, set the toe down as follows: The toe down at road station 114+23, the upstream limit of this reach, should be 2015.6 feet. The toe down at the downstream limit, which is at the northern limit of the existing bridge abutments, is 2011.2 feet. At all points in between, interpolate along the bank protection to obtain the toe down.

For Reach #2 (across Sols Wash Bridge opening)

For the toe down across the Sols Wash Bridge opening, from the northern limit of the Sols Wash Bridge opening (at road station 116+71) to the southern limit of the bridge opening (at road station 114+23), use a constant toe down of 2019.5 feet.

For Reach #3 (from northern limit of Sols Wash Bridge opening (at about road station 116+71) to road station 147+30):

Toe down elevation = 0.50183 * (Road station) + 1963.99 feet

For Reach #4 (from road station 147+30 to northern limit of bank protection):

Toe down elevation = 0.50541 * (Road station) + 1967.80 feet

7.4. Guidebank and Abutment Protection at East Side of Proposed Hassayampa River Bridge

7.4.1. Toe Down and Top of Bank

The soil cement protection for the east abutment of the proposed Hassayampa River Bridge is designed for the 50-year flood.

Two feet of freeboard to the top of bank is recommended per FWHA HEC-23, Section 10 (FHWA, 2001b). Therefore a minimum top of bank elevation of 2048.2 feet is recommended under the bridge and downstream of the bridge, which is just over two feet higher than the 50-year water surface elevation at cross-section 51.375, the upstream bounding bridge cross-section (WSE = 2046.12 feet). Adjacent to the abutment, a pedestrian walkway will be in place; for the portion under the abutment, the top of bank elevation will be lower, at the level of the pedestrian walkway.

Upstream of the bridge, a transition to a higher top of bank is recommended. The next upstream cross-section (51.398) has a 50-year water surface elevation of 2047.22 feet. Although the guide bank will not reach this next cross-section (51.398), it is recommended to design the top of the guide bank per the water surface elevation at that section. The top of soil cement on the guide bank upstream of the bridge should therefore be at least 2049.3 feet. A short transition between top of bank elevations of not more than 25 feet, which takes places just upstream of the abutment, is acceptable.

The total scour at the east abutment, calculated for the 50-year flood, is 26.0 feet (see Table 7-5). The lower of the thalwegs of the bridge's two bounding cross-sections is 2033.5 feet at the downstream cross-section. Therefore the toe down elevation for the abutment soil cement protection should be 2033.5 – 26.3 = 2007.2 feet. This toe down should be applied to the upstream guidebank as well.

7.4.2. Guidebank Shape

The shape of the guide bank was designed by considering the current embankment shape upstream of the bridge, and the guidelines presented in HEC-23 circular Design Guideline 10 (FHWA, 2001b).

Designing the guidebank to follow the procedures in FHWA Hydraulic Engineer Circular No. 23, Design Guideline 10 (Guidebanks), results in a recommended guidebank with a projected length of 65 feet and an offset of 0.4 * 65 feet = 26 feet. Because there is an existing embankment in place upstream of the bridge opening, the soil cement guide bank was designed to wrap around the existing embankment. The proposed guidebank is shown in Figure 7.4. This results in a guide bank with a projected length of about 84 feet. The offset of the guidebank is 48 feet. The general shape is approximately the quarter ellipse recommended in HEC No. 23. At

the upstream end of the guide bank, it turns back into the embankment, until it makes approximately a 45 degree angle with approaching flow.

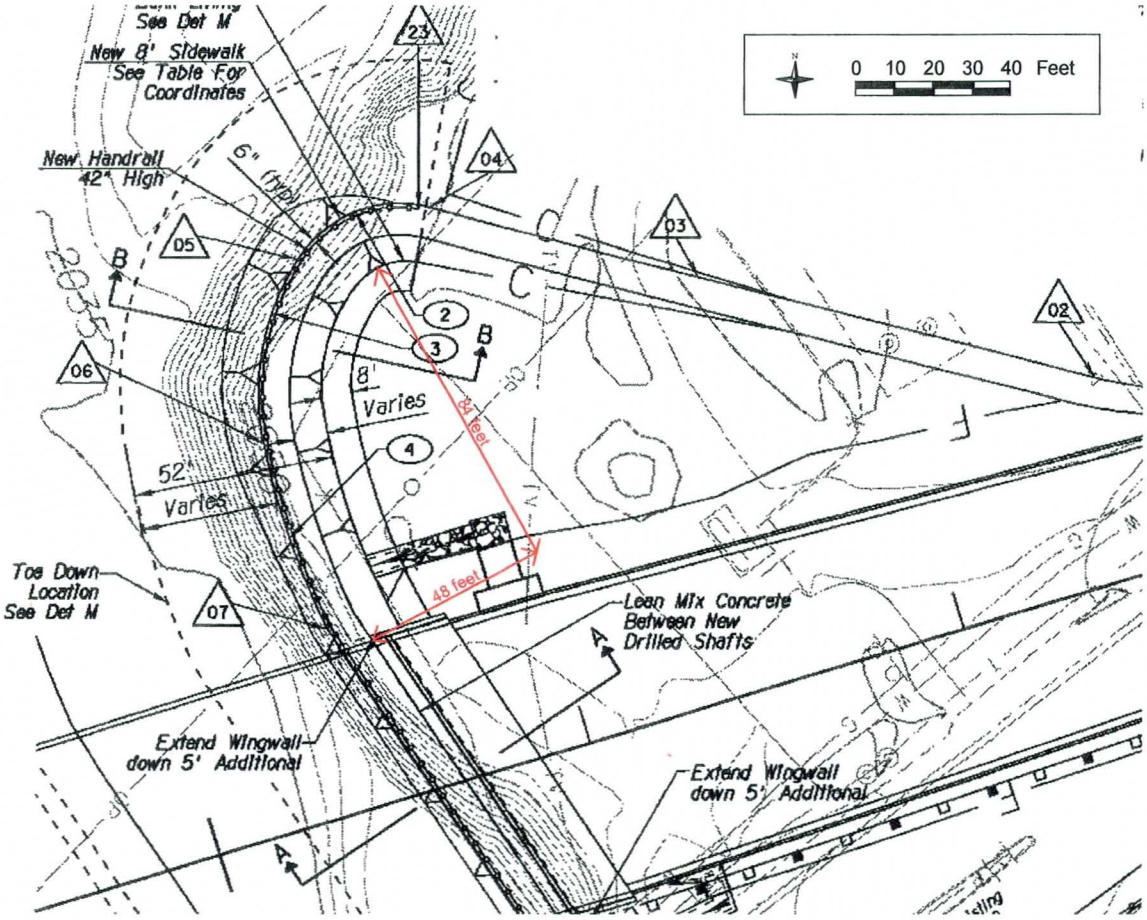

Figure 7.4. Proposed guidebank on east side of proposed Hassayampa River Bridge

7.5. Bridge Scour Analysis: Proposed Hassayampa River Bridge

The scour analysis was performed for the proposed bridge crossing at the Hassayampa River. The various pertinent scour components considered as part of the scour analysis are presented in the following sections. The results from the hydraulic (HEC-RAS) model used to calculate floodplain elevations were used to perform the scour calculations. The calculations performed were based on the procedures presented in FHWA HEC-18 (FHWA, 2001a) and are presented in Appendix A.

Scour analysis was performed for the 50-year flood (47,000 cfs) and for the superflood (120,700 cfs, see 2.2.3). For the superflood condition, some of the discharge went on the overbanks, and the remainder passed underneath the bridges. An analysis using the overbank flow models for the proposed conditions showed that 94,200 cfs went under the bridge, 17,500 cfs went over the

left overbank, and 9,000 cfs went over the right overbank. The right overbank flow took into consideration that Sols Wash may have a levee on its right side when the improvements to Sols Wash are completed in the future. Because of the uncertainty in the overbank flows, however, the superflood was modeled with a higher discharge of 99,000 cfs going under the bridge. This corresponds to the water surface elevation at the upstream internal bridge cross-section being about 0.1 feet lower than the lowest high chord of the bridge (which is on the east side).

7.5.1. Configuration and Layout of Bridge

The shafts supporting the abutments and piers are show in Figure 7.5. Note that North is facing down in that figure, so the flow is approximately from bottom to top of the figure. The piers and abutments, designed to be parallel to the assumed flow, are skewed at 17 degrees from the main centerline of the bridge. There are 4 piers which are approximately equally spaced between the abutments. The pier groups were designed so that pier groups 1, 2 and 3 have at least 16 feet clear between the shafts for all piers. Group 4 has two sets of 3 piers; the two sets are separated by 16 feet clear, however in each of the sets of 3 the clear spacing is less than 16 feet.

Above the ground, the piers are each 4.5 feet in diameter. The shafts themselves, underground, are 5 feet in diameter, which is the dimension shown in Figure 7.5. The pier diameter used for the scour calculations is 4.5 feet, the diameter at the ground surface.

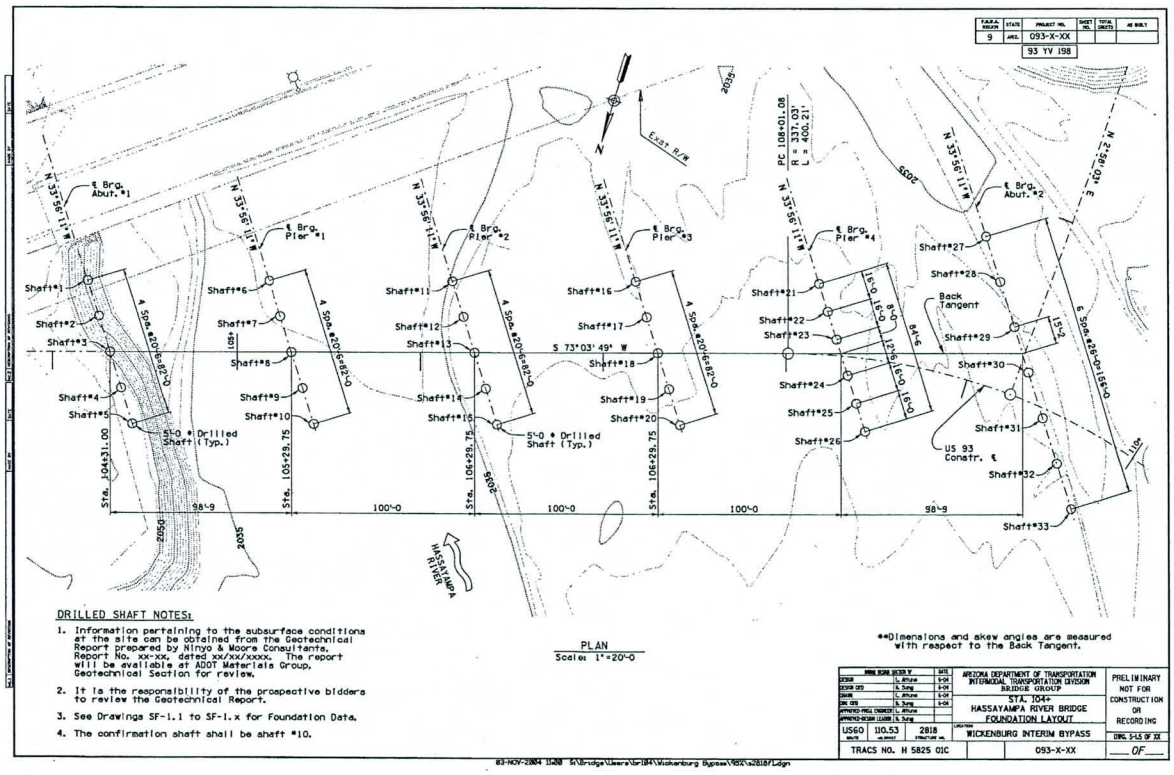

Figure 7.5. Shafts supporting abutments and piers at proposed bridge over the Hassayampa River

The abutments are 492.27 feet apart, along a line which is skewed 17 degrees to the pier groups and abutments. Most of that line is parallel with the centerline of the bridge, but as the bridge approaches the traffic circle on the west side, the centerline turns. Since the bridge is assumed to be 17 degrees skewed to the river, the skewed width (including the piers and soil cement along the abutments) as the Hassayampa River sees the bridge is 492.27 feet * cosine (17 degrees) = 470.76 feet.

Soil cement with a 1:1 side slope is to be constructed at each abutment. Under the bridge, the soil cement has 2-foot wide top shelf on the west side. For the purpose of the hydraulic model and the parameters obtained for the bridge scour calculations, the soil cement was assumed to begin just below the low-chord on the west side of the bridge. On the east side of the bridge, the top of soil cement was set at the elevation of the pedestrian underpass, at 2040.8 feet.

7.5.2. Long Term Degradation

The long-term degradation was determined using the HEC-6T sediment transport model. The long-term simulation was performed using a hydrograph for a period of 48 years plus a hydrograph for the 100-year event. The results indicate an overall degradation of 3 feet within the project reach.

7.6. General Scour

General scour represents the scour that occurs in short time durations such as the scour due to passage of a single flood event. General scour is estimated using the Zeller equation which is valid for sand bed streams (Simons, Li and Associates, 1985). The general scour using Zeller equation for the 50 year and the superflood events are estimated as 0.4 feet and 7.5 feet respectively.

At places such as bridge crossings, the flow area reduces resulting in an increase in the average velocity as well as bed shear. This results in scour which is known as contraction scour, which is a special case of general scour. The contraction scour was calculated assuming that the entire flow in the right "overbank," upstream of the bridge, was in fact sediment-transporting channel flow. The reasoning behind this is that this right overbank area is in fact at almost the same elevation as the channel, and during a large event could transport sediment much as the channel does. This assumption leads to higher contraction scour values than just using the main channel as defined in HEC-RAS, and is therefore conservative.

The contraction scour for the 50-year and the superflood events for the Hassayampa River Bridge crossing are estimated as 6.1 feet and 13.3 feet respectively. Because the contraction scour values are greater than the general scour calculated using the Zeller equation, the contraction scour values govern.

7.7. Local Pier Scour

Local scour calculations at bridge piers were performed for the 50-year and superflood events using the CSU method presented in HEC-18 (FHWA, 2001a). The effective pier width was increased by 4 feet to accommodate the presence of debris accumulation. Due to the presence of

the debris, a square nose is assumed for the purpose of scour calculations. The piers were assumed to be parallel to the flow. The influence of antidunes on the local pier scour is also considered by selecting an appropriate correction factor for plane and antidunes.

Each column in all the piers has a clear pier spacing of 16 feet from the nearest column, therefore the columns can be considered individually for scour estimates. The effective column width is 8.5 feet which is the column width of 4.5 plus 4 feet of debris. The local pier scour for piers 1, 2 and 3 is estimated as 14.8 feet for the 50-year and 15.3 feet for the superflood events. For the superflood event, because the flow depth at the pier is greater than 12 feet at piers 1, 2 and 3, the scour with debris is measured from the low chord of the bridge minus 12 feet rather than from the ground (per ADOT memo 1/10/1991). This reduces the superflood pier scour depths, these reduced depths range from 11.3 to 12.9 feet. These pier scour depths were greater than scour depths calculated using no debris, so they govern.

7.8. Local Abutment Scour

Local scour at abutments is caused by a local change in the flow direction as well as velocity. An increase in turbulence results from the presence of the abutment embankment in the path of the overbank flow. HEC-18 presents Froehlich's equation and the HIRE equation which provide estimates of live-bed abutment scour. The HIRE equation is applicable when the ratio of projected abutment length to the flow depth is greater than 25 with Froehlich's equation valid elsewhere. The east embankment is aligned perpendicular to the flow. The hydraulic parameters needed for the abutment-scour estimates were obtained from the left overbank flow in the HEC-RAS models. Preliminary local scour at the east abutment for the 50-year and superflood events were calculated at 12.9 feet and 31.1 feet, respectively. However, the 31.1 feet, which was calculated using the HIRE equation, was believed to be excessively conservative because the abutment scour equations are known to be extremely conservative, and furthermore:

(1) This abutment will have soil cement which toes down to the 50-year scour depth of 26.3 feet.

(2) The abutment will have a guidewall which should reduce the scour potential.

In consideration of these factors, the local scour at the east abutment for the superflood was reduced to the highest of the local scour depths for the piers, which was 15.3 feet for Pier 4. It was determined also that the maximum local pier scour of 14.8 feet for the 50-year event would also be used for the east abutment.

No local abutment scour was applied at the west abutment. The proposed road embankment upstream functions as a long guidewall which transitions smoothly into the bridge opening. Therefore the west abutment does not obstruct overbank flow in a way that makes it relevant to use the HEC-18 abutment scour procedures. Note that both general (contraction) scour and bend scour are applied to the west abutment, however.

7.9. Low-Flow Incisement

Low-flow incisement was not added to the final scour number because the detailed topography should already capture the existing low-flow channel.

7.10. Bend Scour

Because the proposed bridge lies on the right side of floodplain, and because of the possibility that bend scour could occur against the right bank (as discussed in 7.3.2 in relation to bend scour as it applies to the bank protection), a bend scour component was added to the west abutment. A bend radius of 5000 feet, the same radius used to compute bend scour for the soil cement bank protection, was also applied to the west abutment.

The Zeller bend scour for the 50-year and superflood event were calculated using various cross-sections as the approach section. For the superflood condition, cross-section 51.418 did not contain the flood; an additional 700 feet of top width was added to the HEC-RAS top width (to arrive at an estimated true top width) for the purpose of bend scour calculation. The total top width, including the 700 feet, was 2998 feet. Using a bend radius of 5,000 feet yielded a bend scour of 8.1 feet for the superflood. Calculating the Zeller bend scour using different approach cross-sections ranging from 51.686 through 51.516, resulted in bend scours ranging from 2.4 to 3.1 feet. It was determined that a bend scour of 3.1 feet should be approved.

7.11. Total Bridge Scour

The total bridge scour is estimated by calculating the sum of all the individual scour components and then applying a factor of safety to the sum of all the components. The presence of debris etc. has built a certain amount of conservatism into the estimates. Therefore, a factor of safety of 1.1 was used to arrive at the estimate of total scour. The results are summarized in Table 7-5. The reference thalweg elevation for all the scour depths shown in Table 7-5 is 2033.5 feet.

Table 7-5. Summary of computed scour depths for the proposed bridge over the Hassayampa River

	Component	50-year event, scour in feet	Superflood event, scour in feet
East Abutment	Local Scour	14.8	15.3
	General (contraction) scour	5.8	14.8
	Long-term Scour	3.0	3.0
	10% factor of safety	2.4	3.3
	Total Scour	**26.0**	**36.4**
West Abutment	Local Scour	-	-
	General (contraction) scour	5.8	14.8
	Bend Scour	3.1	8.1
	Long-term Scour	3.0	3.0
	10% factor of safety	1.2	2.6
	Total Scour	**13.1**	**28.5**
Pier 1 / 2 / 3	Local Scour	14.8 (all same)	11.3 / 13.2 / 12.9
	General (contraction) scour	5.8	14.8
	Long-term scour	3.0	3.0
	10% factor of safety	2.4	2.9 / 3.1 / 3.1
	Total Scour	**26.0**	**32.0 / 34.1 / 33.8**
Pier 4	Local Scour	14.8	15.3
	General (contraction) scour	5.8	14.8
	Long-term scour	3.0	3.0
	10% factor of safety	2.4	3.3
	Total Scour	**26.0**	**36.4**

8. REFERENCES

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APPENDIX A

Calculations

WEST CONSULTANTS, INC.

Client: Jacobs/AZ Dept. of Transportation Sheet: ___ of ___
Project: US93 Interim Wickenburg Bypass Computed By: LRK Date: 7/25/06
Project No: ___ Checked By: DSG Date: 1/23/06
Computations For: ___

General Scour for 50-year Flood Event

Zeller Equation:

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

where

y_{gs} = general scour depth (ft.)

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

V_m = Average velocity of flow (ft./s)

y_h = hydraulic depth of flow (ft.)

S_e = energy slope (ft/ft)

Input Parameters (from HEC-RAS XS # 51.378)

$y_{max} = 12.0$ ft. $V_m = 11.2$ ft/s $y_h = 9.1$ ft. $S_e = 0.004133$

Calculations

$$y_{gs} = 12.0 \times \left[(0.0685 \times 11.2^{0.8})/(9.1^{0.4} \times 0.004133^{0.3}) - 1\right]$$

$$= 0.2 \text{ ft.}$$

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: ___ of ___
Project: US93 Interim Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No: ___ Checked By: DFR Date: 1/23/06
Computations For: ___

Local Pier Scour for 50-year flood event

$$y_s = 2.0 K_1 K_2 K_3 K_4 \left(\frac{a}{y_1}\right)^{0.65} F_r^{0.43} y_1$$

where

K_1 = correction factor for nose shape
K_2 = correction factor for angle of attack, θ

$$= \left(\cos\theta + \frac{L}{a}\sin\theta\right)^{0.65}$$

where L – length of piers
a – pier width

K_3 = Bed form factor
K_4 = correction for armoring
F_r = Froude number upstream of piers
y_1 = maximum flowdepth upstream of piers

Input Parameters (Hydraulics from HEC-RAS XS # 51.418)

Debris width = 4 ft.
A square nose shape is assumed due to presence of debris
Therefore, K_1 = 1.1
Angle of attack, θ = 0 degrees
Therefore, K_2 = 1.0

Average Velocity upstream of bridge, V = 6.77 ft/s
Maximum depth of flow upstream of bridge, y_1 = 12.66 ft
Froude Number immediately upstream of bridge, F_r = 0.335

D_{50} – Median grain size = 1 mm
D_{90} = 6 mm
D_{95} = 9.5 mm
Bed condition = Plane bed + Antidunes
$\Rightarrow K_3 = 1.1$

Pier type = Group of cylinders

Since $D_{50} < 2$ mm and $D_{95} < 20$ mm, $K_4 = 1.0$

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transporation Sheet: ___ of ___

Project: US 93 Interim Wickenburg Bypass Computed By: LRK Date: 7/25/05

Project No: Checked By: DSR Date: 1/23/06

Computations For:

Since pier scouring $\geq$ 16 ft

Pier width, a = pier diameter + debris width

$= 4.5 + 4$

$a = 8.5$ ft

Calculations

$$y_s = 2K_1K_2K_3K_4\left(\frac{a}{y_1}\right)^{0.65} F_r^{0.43}\, y_1$$

$$= 2 \times 1.1 \times 1.0 \times 1.1 \times 1.0 \times \left(\frac{8.5}{12.66}\right)^{0.65} \times 0.335^{0.43} \times 12.66$$

$= 14.8$ ft

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: ___ of ___

Project: US93 Interim Wickenburg Bypass Computed By: LRK Date: 7/25/05

Project No: ___ Checked By: DJR Date: 1/23/06

Computations For: ___

Total Scour at Piers for 50-year Flood Event

Long term degradation	= 3.0 ft.
General contraction scour	= 5.8 ft.
Local Scour at Piers	= 14.8 ft.
	23.9 ft.
Factor of Safety	X 1.1
Total Scour at Piers = 1.1 X 23.9	= 26.0 ft.

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: ____ of ____

Project: U.S. 93 INTERIM WICKENBURG BYPASS Computed By: LRK Date: 7/25/05

Project No.: ____ Checked By: DJG Date: 1/23/06

Computations For: ____

TOTAL SCOUR AT EAST ABUTMENT FOR 50-YEAR FLOOD

Long-term degradation	= 3.0 ft
Local Scour at abutments	= 14.8 ft
Contraction Scour	= 5.8 ft
	23.6 ft
Factor of safety	= × 1.1
Total Scour at East abutment	= 25.6 ft

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: ____ of ____
Project: US 93 INTERIM WICKENBURG BYPASS Computed By: LRK Date: 7/25/05
Project No.: ____ Checked By: DSR Date: 1/23/06
Computations For: ____

GENERAL SCOUR FOR SUPER FLOOD

ZELLER EQUATION:

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

where

y_{gs} = general scour depth (ft)

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

V_m = Average Velocity of flow (ft/s)

y_h = Hydraulic depth of flow (ft)

S_e = Energy Slope (ft/ft)

INPUT PARAMETERS (From HEC-RAS XS # 51.378)

$y_{max} = 22.9$ ft $V_m = 9.37$ ft/s $y_h = 21.17$ ft $S_e = 0.001$

CALCULATIONS

$$y_{gs} = 22.9 \times \left[(0.0685 \times 9.37^{0.8})/(21.17^{0.4} \times 0.001^{0.3}) - 1\right]$$

$$= < 0$$

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Project: US 93 Interim Wickenburg Bypass Computed By: LRK Date: 7/25/05
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Computations For: ___

Local Pier Scour for Super Flood

$$y_s = 2.0 K_1 K_2 K_3 K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr^{0.43} y_1$$

Where

K_1 = correction factor for nose shape

K_2 = correction factor for angle of attack, θ

$= \left(\cos\theta + \frac{L}{a}\sin\theta\right)^{0.65}$

where L - length of pier

a - pier width

K_3 = Bed form factor

K_4 = correction for armoring

Input Parameters [~~Hydraulics from HEC-RAS XS #51.417~~]

Debris width

A square nose is assumed due to presence of debris

Therefore, K_1 = 1.1

Angle of attack, θ = 0 degrees

Therefore K_2 = 1.0

Average velocity upstream of bridge, V = 6.10 ft/s

Maximum depth of flow upstream of bridge, y_1 = 23.66 ft

Fronde number immediately upstream of bridge, Fr = 0.22

D_{50} - Median grain size = 1 mm

D_{90} = 6 mm

D_{95} = 9.5 mm

Bed Condition = Plane Bed and Antidunes

Therefore, K_3 = 1.1

Pier Type = Group of Piers

Since $D_{50} < 2$ mm and $D_{95} < 20$ mm, K_4 = 1.0

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Computations For: ___

Pier Width, a = Pier diameter + debris width

= 4.5 + 4

a = 8.5 ft

Calculations

$$y_s = 2 K_1 K_2 K_3 K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr^{0.43} y_1$$

$$= 2 \times 1.1 \times 1.0 \times 1.1 \times 1.0 \times \left(\frac{8.5}{23.66}\right)^{0.65} \times 0.22^{0.43} \times 23.66$$

$$= 15.3 \text{ ft.}$$

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Computations For: ___

Local Pier Scour for Super Flood

Assuming no debris (see George Lopez Cepero memo of Jan. 10, 1991)

all parameters are the same except

$a = 4.5$ ft

and also, do not need to assume a square nose pier since no debris is present.

so $K_1 = 1.0$

Calculations

$$y_s = 2 K_1 K_2 K_3 K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr^{0.43} y_1$$

$$= 2 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \left(\frac{4.5}{23.66}\right)^{0.65} \times 0.22^{0.43} \times 23.66$$

$$= 9.2 \text{ ft}$$

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Computations For: ___

Superflood Pier Scour Continued:

Applying the scour with debris from 12 ft. below the low chord yields the following scour depths

	Pier 1	Pier 2	Pier 3
Top of Pier Elevation	2050.2	2051.1	2051.7
Scour using 4' debris	16.3	15.3	15.3
Ground elevation (higher of upstream and downstream)	2034.2	2037.0	2037.3
Pier height exposed	16 ft.	14.1 ft.	14.4 ft.
Reduction in scour depth (pier height - 12 ft.)	4.0 ft.	2.1 ft.	2.4 ft.
Adjusted Scour depth	11.3 ft.	13.2 ft.	12.9 ft.

These are all higher than the scour depth with no debris (9.2 ft.), so they govern.

Pier 4 is about 12 ft. from the ground so no reduction in scour is possible

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Computations For: /

LOCAL ABUTMENT SCOUR AT EAST ABUTMENT FOR SUPER-FLOOD

INPUTS (From HEC-RAS XS # 51.418)

Abutment Shape = Spillthrough

Length of active flow obstructed by embankment, $L' = 763$ ft

Average depth of flow in floodplain, $y_a = 10.8$ ft

Flow obstructed by abutment and approach embankment, $Q_e = 25234$ cfs

Average Velocity near embankment, $V_e = 6.9$ ft/s

Embankment angle, $\theta = 90$ degrees

CALCULATIONS

Froude Number, $F_r = \frac{V_e}{\sqrt{g y_a}} = \frac{6.9}{\sqrt{32.2 \times 10.8}} = 0.37$

$K_1 = 0.55$ for spill-through abutments

$K_2 = 1.0$ for $\theta = 90°$

Since $L'/y_a = 70.6$ is greater than 25, HIRE Equation should be used. Therefore,

$$y_s = y_a \times 4 \times F_r^{0.33} \times \frac{K_1}{0.55} \times K_2$$

$$= 10.8 \times 4 \times 0.37^{0.33} \times \frac{0.55}{0.55} \times 1.0$$

$$= 31.1 \text{ ft.}$$

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Client: Jacobs/Az Dept. of Transportation Sheet: ___ of ___

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Computations For: ___

Total Scour at Piers for Super Flood Event

Long-term degradation	= 3.0 ft.
General/contraction scour	= 14.8 ft.
Local Scour at Piers	= 15.3 ft.
	33.1 ft.
Factor of Safety	× 1.1
Total Scour at Piers = 32.3 × 1.1	36.4 ft.

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Computations For: ____

TOTAL SCOUR AT EAST ABUTMENT FOR SUPERFLOOD EVENT

Long-term degradation	= 3.0 ft
Local Scour at East abutment	= 31.1 ft
Contraction Scour	= 14.8 ft
	48.9+ ft
Factor of Safety	= x 1.1
Total Scour at East Abutment	= 53.8 ft

(not used, pier scour was applied instead of abutment scour)

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Client: Jacobs / AZ Dept. of Transportation Sheet: 1 of 3
Project: Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No.: 512A01 Checked By: DSG Date: 1/23/06
Computations For: Proposed Bridge over Hassayampa River
Contraction Scour Calculation, 50-year Flood

Proposed Bridge over Hassayampa River Contraction Scour Calculation, 50-year Flood

The same model was used to determine the proposed conditions flood elevations (the base model) was used to calculate the contraction scour. The 50-year flood is 47,000 cfs.

The maximum contraction scour was found to occur using cross-section 51.477 as the upstream cross-section.

The contracted cross-section was the cross-section at the upstream face of the bridge, 51.375.

Because the "right bank" region in the area near the bridge is very close to the channel elevation, the entire flow on the right bank was considered to be "channel flow" for the purpose of the upstream cross-section used for live-bed contraction scour. This gave a larger top width at the upstream section, and also a larger upstream flow. This resulted in slightly higher contraction scour results than using the bank stations in HEC-RAS to define the channel flow for the contraction scour equation.

Upstream Section

For cross-section 51.477 (the upstream section), the parameters for the contraction scour equation are:

Flow in upstream channel:

$$Q_1 = 27326.2 + 16271.4 = 43,597.6 \text{ cfs}$$

Top width in upstream channel:

W_1 = 422.9 feet (RAS Channel active top width) + 370.5 (RAS Channel Right Bank Top Width) = 793.4 feet

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Client: Jacobs/AZ Dept. of Transportation Sheet: 2 of 3
Project: Wickenburg Bypass Computed By: LRK Date: 1/25/05
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Computations For: Proposed Bridge Over Hassayampa River
Contraction Scour Calculation, 50-year Flood

Contracted Section

W_2 = Average width of bridge opening − width of piers
= 470.26 (width of bridge opening) − 12 feet (approx. average width of soil cement on both sides) − 4 piers * 4.5 feet (average width per pier) = 440.26

Flow in contracted section:
Q_2 = 47,000 (all flow goes through contracted section)

Other parameters

For a median grain diameter of 1.0 mm = 0.003281 feet (sample at bridge)
Fall velocity from Figure 5.8 of HEC-18 (for D_{50} = 1 mm at 20 degrees C):

Fall velocity (ω) = 0.13 m/sec. = 0.4265 ft./sec

Critical Velocity

$V_c = K_u y_a^{1/6} D_{50}^{1/3}$ (HEC-18 Equation 5.1)

Where:
K_u = 11.17 for English Units
y_a = Average depth of flow upstream of bridge = 9.52 feet
D_{50} = 0.0315 feet

And
$V_c = 11.17 (9.52)^{1/6} (0.00262)^{1/3}$
= 2.24 ft/sec

Since the velocity is larger than the critical velocity, live bed scour will occur.

Shear velocity

$V_* = (g \cdot y_a \cdot S_a)^{1/2}$

Where
g = gravitational constant, ft/s² = 32.2, ft./s²
y_a = average depth of flow upstream of bridge = 9.46 feet (from HEC-RAS)
S_a = energy slope = 0.0013

Top width in upstream channel:

$W_1 = 794.01$

Contracted Section

W_2 = Average width of bridge opening - width of piers
= 410.26 (width of bridge opening) - 4 piers * 4.5 (average pier width) = 452.26

Flow in contracted section

$Q_2 = 99{,}000$ cfs goes through bridge

Other Parameters

For a median grain diameter of 1mm = 0.003281 ft
Fall velocity from Figure 5.8 of HEC-18 (for D_{50} = 1mm at 20 degrees C):

Fall velocity (ω) = 0.13 m/sec = 0.4265 ft/sec

Critical Velocity

$V_c = K_u y_a^{1/6} D_{50}^{1/3}$ (HEC-18 Equation 5.1)

Where
K_u = 11.17 for English units
y_a = Average depth of flow upstream of bridge
= 22.92 ft (hydraulic depth in channel at cross-section 51.378)
D_{50} = 0.0315 ft.

And
$V_c = 11.17(22.92)^{1/6}(0.00262)^{1/3}$
$= 2.59$ ft/sec.

Since the velocity is larger than the critical velocity live bed scour will occur.

Shear velocity

$V_* = (g \cdot y_a \cdot S_a)^{1/2}$

Where
g = gravitational constant, $ft/s^2 = 32.2\ ft./s^2$
y_0 = average depth of flow upstream of bridge = 22.94 ft
S_2 = energy slope = 0.0008 (low because of backwater condition created by bridge)
$V_* = (32.2 * 22.94 * 0.0008)^{1/2} = 0.769\ ft./s$

So, $V_*/\omega = 0.769\ ft./s / 0.4265\ ft./s = 1.802$

Live Bed Scour

$$y_2/y_1 = (Q_2/Q_1)^{6/7}(W_1/W_2)^{K_1}$$

And

$$y_s = y_2 - y_0$$

Where
y_1 = average depth in the upstream channel (ft.)
y_2 = average depth in the contracted section (ft.)
y_0 = existing depth in contracted section before scour (ft.)
y_s = y_1 for sand bed channels
Q_1 = flow in upstream channel transporting sediment (cubic ft. per second)
Q_2 = flow in contracted channel (cubic ft per second)
W_1 = width in main channel that is transporting bed material
W_2 = width in main channel in contracted section less pier widths (ft.)
K_1 = 0.64 since $0.5 < V_*/\omega < 2.0$

For the upstream section y_1 = 22.21 (hydraulic depth in upstream channel from HEC-RAS at 51.477)

For the contracted section y_2 = 22.94 ft.

$$y_2/y_1 = (99{,}000/81{,}331.9)^{6/7}(794.01/452.26)^{0.64}$$

or $y_2 = 1.697 * 22.21 = 37.69$

and $y_s = y_2 - y_1 = 37.69 - 22.94 = 14.8$ ft.

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Client: Jacobs / AZ Dept. of Transportation Sheet: 1 of 1
Project: Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No.: 512A01 Checked By: DRR Date: 1/23/06
Computations For: Example General Scour Calculation

Example General Scour Calculation

The Zeller General Scour Equation is (Simons, Li & Associates, 1985)

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

Where

y_{gs} = general scour depth (ft)
y_{max} = maximum depth of flow (ft)
V_m = average velocity of flow (ft/s)
y_h = hydraulic depth of flow (ft)
S_e = energy slope (or bed slope for uniform flow) (ft)
(Should y_{gs} become negative, assume general scour component is equal to zero)

At cross-section 52.054:

The following were gathered from HEC-RAS:

V_m = 14.19 feet/second (channel rather than average velocity used)
y_{max} = 7.09 feet
y_h = 4.93 feet
S_e = 0.00517

$$y_{gs} = y_{max}\left[(0.0685 V_m^{0.8})/(y_h^{0.4} S_e^{0.3}) - 1\right]$$
$$= 7.09\left[(0.0685 * 13.85^{0.8})/(4.93^{0.4}\, 0.00517^{0.3}) - 1\right]$$
$$= 3.1 \text{ feet}$$

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: 1 of 1
Project: Wickenburg Bypass Computed By: LRK Date: 7/25/05
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Computations For: Example Antidune Scour Calculation

Example Antidune Scour Calculation

The formula for antidune height is:

$$h_a = 0.14 \frac{2\pi V^2}{g} = 0.027 V^2$$

Where
h_a = antidune height from crest to trough (ft)
V = channel velocity (ft/s)
g = gravitational constant, 32.2 ft./sec^2

The scour depth due to antidunes is half the dune height, or

Antidune scour = $h_a/2 = (0.027 * V^2)/2$

At cross-section 52.054

V = channel velocity (ft/s) = 13.85 ft/s

Antidune scour = $(0.027 * 13.85^2)/2$
= 2.59 feet

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Client: Jacobs/AZ Dept. of Transportation Sheet: 1 of 2
Project: Wickenburg Bypass Computed By: LRK Date: 7/25/05
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Computations For: Example Zeller Bend Scour Calculation

Example Zeller Bend Scour Calculation

Zeller's (1981) Formula (Simons, Li & Associates, 1985) for bend scour was used:

$$\Delta Z_{bs} = \frac{0.0685 Y V^{0.8}}{Y_h^{0.4} S_e^{0.3}} \left[2.1 \left(\frac{\sin^2 \frac{a}{2}}{\cos a} \right)^{0.2} - 1 \right]$$

Where
- ΔZ_{bs} = bend scour component of total scour depth (ft)
- V = mean velocity of upstream flow (ft/s)
- Y = maximum depth of upstream flow (ft)
- Y_h = hydraulic depth of upstream flow (ft)
- S_e = approach energy slope or bed slope for uniform flow (ft/ft)
- a = angle formed by the projection of the channel centerline from the point of curvature to a point which meets a line tangent to the outer bank of the channel (degrees)

The relationship (Simons, Li & Associates, 1985):

$$r_c/W = (\cos a)/4(\sin^2(a/2))$$

Where
- r_c = radius of curvature to centerline of channel (ft)
- W = channel topwidth (ft)

was used to determine the ratio $\cos a/\sin^2(a/2)$ on the basis of r and the channel top width.

To get the parameters required for Zeller's bend scour equation, the velocity and energy slope were taken from cross-section 52.121, using the model which has the Manning's n in the channel equal to 0.025.

- Y_h = hydraulic depth of upstream flow (ft) = 5.30 feet
- V = mean velocity of upstream flow (ft/s) = 10.64 ft/s
- Y = maximum depth of upstream flow (ft) = 8.14 feet
- S_e = approach energy slope = 0.004632

To calculate $(\cos a)/(4(\sin^2(a/2)))$, need radius and

top width. Approach section 52.121 has an active top width of 1300.7 feet. Assume r_c = 5,000 feet.

Therefore

$$\sin^2(a/2)/\cos(a) = w/(4r_c)$$
$$= 1300.7\text{ feet}/(4*5000)\text{ feet}$$
$$= 0.06503$$

and

$$\Delta Z_{bs} = \frac{0.0685 * 8.14 * 10.68^{0.8}}{5.30^{0.4} * 0.004632^{0.3}} \left[2.1 * 0.06503^{0.2} - 1 \right]$$

$$= 2.06 \text{ feet}$$

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Project: US93 Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No: ___ Checked By: DJR Date: 1/23/06
Computations For: Eastern Guide Bank for proposed bridge over Hassayampa

Using HEC-23 2nd Edition
Design Guideline 10 "Guidebanks"

Variables

Q = Total Discharge, cfs

Q_f = Lateral or floodplain discharge intercepted by the embankment, cfs

Q_A = Discharge in 100 ft of stream adjacent to abutment

b = length of bridge opening

A_{n_2} = cross-sectional area at bridge opening at normal stage, ft²

V_{n_2} = Average velocity through bridge opening

$\frac{Q_f}{Q_A}$ = Guide bank discharge ratio

L_s = Projected Length of Guide Bank

Using 50 year flood

Q = 47,000 cfs
Q_f = 4497 cfs (left bank flow from XS 51.398)
b = 470 ft approx. perpendicular to flow
Q_a = 100/470 × 47,000 cfs
= 10,000 cfs
A_{n_2} = 4535 sq. ft. (per HEC-RAS at XS 51.375)
V_{n_2} = 10.4 ft/sec. (per HEC-RAS at upstream bridge section)

$$\frac{Q_f}{Q_a} = \frac{4497 \text{ cfs}}{10{,}000 \text{ cfs}} = 0.45$$

L = 65 ft. (looked up on Figure 10.3 nomograph using 11 fps an $\frac{Q_f}{Q_A} = 0.45$)

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Client: Jacobs / AZ Dept. of Transportation Sheet: 1 of 2
Project: U.S. 93 Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No.: Checked By: DFR Date: 1/23/06
Computations For: General Scour at Sols Wash Bridge

Zeller equation for General Scour

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

y_{gs} = general scour depth, ft = ?

y_{max} = maximum depth of flow, ft.

V_m = average velocity of flow, ft/s.

y_h = hydraulic depth of flow, ft

S_e = energy slope (ft/ft)

HEC-RAS x-section # 455.517

50-yr event

$y_h = 6.93$ ft

$V_m = 11.10$ ft/sec (use channel velocity)

$y_{max} = 9.04$ ft

$S_e = 0.002$ ft/ft

$$y_{gs} = 9.04\left[(0.0685\ (11.10)^{0.8}) / (6.93^{0.4} \times 0.002^{0.3}) - 1 \right]$$

$= \underline{3.59 \text{ ft.}}$

100-yr event

$y_h = 7.02$ ft.

$V_m = 11.96$ ft/sec

$y_{max} = 9.93$ ft.

$S_e = 0.0022$ ft/ft.

$$y_{gs} = 9.92\left[(0.0685\ (11.96)^{0.8}) / (7.82^{0.4} \times 0.002^{0.3}) - 1 \right]$$

$= \underline{4.1 \text{ ft.}}$

WEST CONSULTANTS, INC.

Client: Jacobs/AZ Dept. of Transportation Sheet: 2 of 2
Project: US 93 Wickenburg Bypass Computed By: LRK Date: 7/25/05
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Computations For: Antidune Scour

Antidune Scour (Based on RAS Model)

Verify Dune Type ["Sediment Transport Technology," Simons & Senturk]

Figure 5.22 → Stream power $= \tau_0 V = (\gamma y_0 S) V$

y_0 = contracted section depth = 7.8 ft.

$$\tau_0 V = (\gamma y_0 S) V$$
$$= (62.4 \times 7.8 \times .0022)\ 11.96$$
$$= 12.8\ (\text{ft-lb/s})/\text{ft}^2$$

(Antidune – O.K. per Simons and Seaturk figure 5.22)

$$\text{Antidune Height} = 0.027 V^2$$
$$= 0.027\ (11.96)^2$$
$$= 3.86 \text{ ft.}$$

$$\text{Antidune Scour} = \frac{3.86}{2} = 1.9 \text{ ft.}$$

Long term scour = Ø (system is aggrading)

Total Scour = General Scour + Antidune Scour
= 4.1 + 1.9
= 6.0 ft.

Including 30% safety factor, Total Scour = 7.8 ft.

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Client: Jacobs / AZ Dept. of Transportation Sheet: 1 of 6
Project: US93 Interim Wickenburg Bypass Computed By: LRK Date: 11/25/05
Project No: Checked By: DFR Date: 1/23/06
Computations For: Scour Hole downstream of Sols Wash

Bottom of Sols Wash Bridge superbox is at 2036.0 elevation.

Assume a free-fall situation with the top of the water surface downstream of the bridge equal to Hassayampa thalweg minus 3 feet long-term scour.

Channel thalweg = 2036.39 (from Hassayampa model section 51.477)

Long Term Scour = 3.0 feet

Assumed top of water surface elevation at

2036.39 - 3.0 = 2033.39

This implies a drop from bottom of superbox of

2036.0 - 2033.39 = 2.61 feet

At sufficiently high flows the unsubmerged free-fall condition becomes unreasonably conservative.

Assume the when the critical depth of the upstream bridge section exceeds the 2.61 foot threshold, the bridge outlet becomes partially submerged.

This occurs at about 5,500 cfs

Assuming a bridge width (perpendicular to flow) of 224.07 feet,

q = unit discharge = 5,500/224.07

= 24.54 cfs/foot

and D_c = critical depth

$$= \left(\frac{q^2}{g}\right)^{1/3}$$

$$= \left(\frac{24.54^2}{32.2}\right)^{1/3}$$

$$= 2.65 \text{ feet}$$

at discharges > 5,500 cfs, the critical depth is even higher, so we assume it will be partially submerged.

At 5,500 cfs, the upstream E.G.L is

E.G.L upstream = 2036 + critical depth × 1.5

$= 2036 + 2.65 \times 1.5$

$= 2039.97$

Assume the downstream E.G.L. is

E.G.L downstream = 2033.43 (Hassayampa thalweg - 3 ft long term scour)

$\underbrace{+ \tfrac{1}{2} \text{ upstream critical depth}}_{\text{an assumption of downstream energy}}$

$= 2033.39 + 2.65/2$

$= 2034.72$

∴ Drop in head = 2039.97 - 2034.72 = 5.25 ft.

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Client: Jacobs / AZ Dept. of Transportation Sheet: 3 of 6
Project: US93 Interim Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No: Checked By: DJQ Date: 1/23/06
Computations For: Scour hole downstream of Sols Wash

using the equation:

$$Z_{isf} = 1.32\, q^{0.54} H_t^{0.225} - TW$$

to calculate scour (Standards Manual For Drainage Design and Floodplain Management in Tucson Arizona, by Simons, Li and Assoc., 1989, p 6.18)

where Z_{isf} = Depth of local scour in feet measured below streambed surface downstream of the drop

q = Discharge per unit width of channel bottom, in cfs/foot

H_t = Total drop in head from upstream E.G.L. to downstream E.G.L., in feet

TW = Tailwater depth in feet

then $q = 24.54$ cfs/ft

$H_t = 5.25$ feet

assume $TW = 0$ (conservative)

and

$$Z_{isf} = 1.32 \times (24.54)^{0.54} (5.25)^{0.225}$$

$$= 10.79 \text{ feet}$$

Including a 30% factor of Safety, the scour depth is

$$\text{Scour Depth} = 2033.39 - 10.79 \times 1.3$$

$$= 2019.35 \text{ feet}$$

Check also for the situation of submerged flow

Using the equation in "Physical Model of Local Scour at Grade Control Structures" by Bormann:

$$D_s = C_1 \frac{q^{0.67}}{g^{0.33}} \left(\frac{\Delta H}{y_t}\right)^{P_1} (S_r)^{P_2}$$

where
D_s = Depth of scour
q = flow in cfs/ft
g = 32.2 ft/s²
ΔH = drop in head
y_t = downstream water depth
P_1 = 0.158 for 1:1 face slope (soil cement)
P_2 = −0.134 for 1:1 face slope (soil cement)
C_1 = 1.70 for 1:1 face slope

assume y_t = critical depth for the upstream section (this is conservative)

at Q = 15,045 cfs (100-year flood)

q = 15,045 / 224.07
= 67.14 cfs/ft.

$y_t = D_c$ (assumed critical depth)

$$= \left(\frac{q^2}{g}\right)^{1/3} = \left(\frac{67.14^2}{32.2}\right)^{1/3} = 5.19 \text{ feet}$$

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Client: Jacobs / AZ Dept. of Transportation Sheet: 5 of 6
Project: US93 Interim Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No: Checked By: DJR Date: 1/23/06
Computations For: Scour hole downstream of Sols Wash

Upstream EGL

$= 2036 + (\text{critical depth}) \times 1.5$

$= 2036 + 5.19 \times 1.5$

$= 2043.78$

assume D/S water surface elevation equal to 2033.39 + critical depth in the upstream section

$= 2033.39 + 5.19$

$= 2038.58$

S_r = relative submergence

$= \dfrac{2038.58 - 2036}{5.19}$

$= 0.497$

assume Downstream EGL = DSWSE = 2038.58

$$D_s = 1.70 \times \frac{67.14^{0.67}}{32.2^{0.33}} \left(\frac{2043.78 - 2038.58}{5.19}\right)^{0.158} \times (0.497)^{-.134}$$

$= 9.95$ feet

which is measured from assumed bed elevation of 2033.39

Scour Elev. $= 2033.39 - 9.95 \times \underbrace{1.3}_{\text{factor of safety}}$

$= 2020.45$

WEST CONSULTANTS, INC.

Client: Jacobs / Az Dept. of Transportation Sheet: 6 of 6

Project: US93 Interim Wickenburg Bypass Computed By: LRK Date: 7/25/05

Project No: Checked By: DPR Date: 1/23/06

Computations For: Scour hole downstream of Sole Wash

lower of the two scour elevations is 2019.35,

use 2019.5 as the scour depth

(round to nearest 0.5 foot elevation)

Physical Model of Local Scour At Grade Control Structures

Noel E. Bormann,[1] M.ASCE

Abstract: A physical model of local scour occurring at grade-control structures was operated at an undistorted Froude number prototype-model scale ratio of 1:4. The maximum unit discharge in the 3 feet wide model flume was 25 cfs/ft, this coincides with the minimum prototype discharge modeled. Of 88 tests completed, 7 cases occurred with similar hydraulic conditions at both 3 cfs/ft and 25 cfs/ft in the flume. This allows a comparison of model-prototype behavior. Results of this comparison indicate that model based predictions of local scour are within 25% of the observed scour depths using an empirical equation developed from the model data.

Introduction

Grade-control structures are employed in alluvial channels to limit or control channel degradation upstream of the structure. With a grade-control structure in place, other hydraulic structures may be constructed predicated upon the controlled channel bed elevation. Therefore, should a grade-control structure fail, subsequent failures of bank protection, bridge foundations, flood protection and diversion structures could result. A common cause of failure for grade-control structures is foundation undermining due to local scour. Because the local scour processes are imperfectly understood, predictions of local scour are typically made based on empirical equations developed from physical model data. However, very little prototype data exists for confirmation of predictions. In addition, many models have used quite small flow rates, with scale ratios of 1:20, or smaller, being common.

This paper presents model-prototype comparisons of local scour observed in an undistorted 1:4 Froude scaled physical model. The model represents grade-control structures with two different face slopes which experience partially submerged flow conditions. The comparisons are made using both the nominal scale of 1:4 and the apparent scale of the test data. The apparent scale results when the

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measured unit discharges (q) and tailwater levels could not be maintained at the required values during model operation.

Experimental Program and Data

The physical model was located in an outdoor flume at the Engineering Research Center of Colorado State University. The flume was constructed of mild steel plate with a total depth of 11.5 feet, an overall length of 90 feet and a width of 3 feet. Water was supplied to the flume by a 36-inch vertical mixed flow pump powered by a 450 hp diesel engine, and conveyed through 36-inch steel pipe. This arrangement was capable of providing approximately 75 cfs, or a maximum of 25 cfs/ft to the flume. The grade-control structure was modeled by welding steel plate to the flume walls at vertical and 1:1 face slopes. The crest of the structure was 7 feet above the flume bottom. Figure 1 is a schematic diagram of the flume. For each test the scour depth was measured by a heavy-duty point gage read to 0.01 feet, the water depths were measured by staff gages with divisions of 0.01 foot and velocities were measured by a Marsh-McBirney Model 201 electromagnetic current meter.

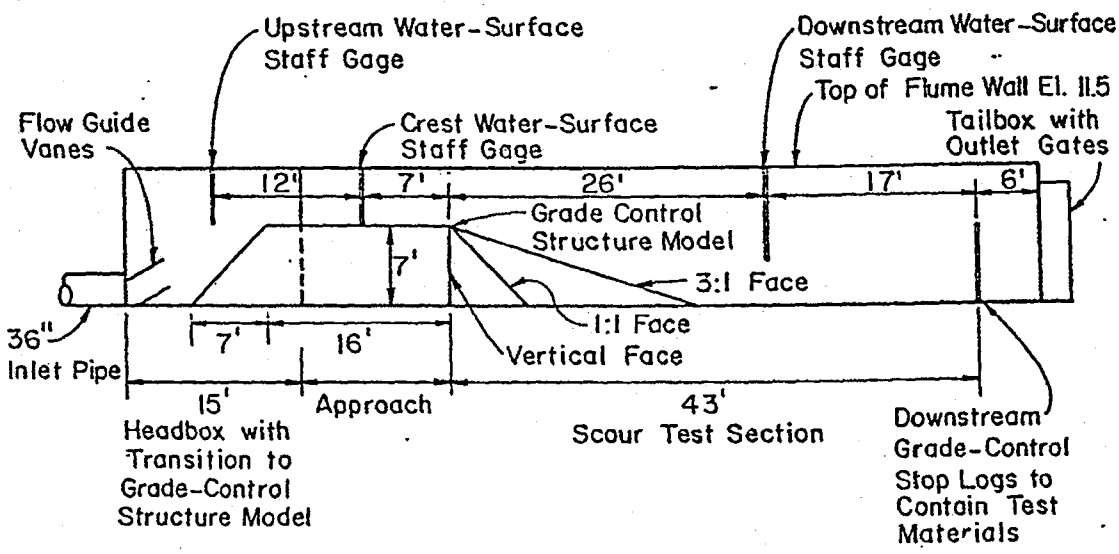


Figure 1. Schematic Diagram of the Experimental Flume.

For each face slope (Sf), a range of q, Sr and Dp values were tested. Since there was no sediment supply to the test section, all tests reflect a clear water scour condition. The sediment used in the model has a d_{90} of 2.3 mm and a d_{50} of 0.43 mm. Each test was conducted for a period of 1 hour. Of the 88 tests completed, 7 pairs have

similar Sf, Sr, ΔH and Dp values and reflect a model-prototype discharge overlap. In each pair, the first test has a q of ≈25 cfs/ft, and the second a q of ≈3 cfs/ft. The test characteristics of these 7 pairs are given in Table 1.

Table 1

Test Data

Pair (1)	Sf (2)	q cfs/ft (3)	Ratio of q (4)	Dp ft (5)	Sr (6)	ΔH ft (7)	Ratio of ΔH obs (8)	Yt ft (9)	Ds obs ft (10)	Ratio of obs Ds (11)	Ds Eq 1 ft (12)	Ratio of calc Ds (13)
1	1:1	25.00		0.75	0.70	1.22		3.55	3.49		4.14	
	1:1	3.61	6.93	0.75	0.72	0.31	3.94	1.56	0.32	10.91	1.03	4.01
2	1:1	25.00		0.75	0.53	1.87		2.90	4.57		4.74	
	1:1	3.61	6.93	0.75	0.48	0.56	3.34	1.27	0.49	9.33	1.24	3.83
3	Vert	25.41		1.25	0.94	0.25		5.42	1.89		1.45	
	Vert	3.09	8.22	1.25	0.91	0.11	2.27	2.32	0.93	2.03	0.36	4.04
4	Vert	24.99		1.25	0.70	1.20		4.00	3.09		3.21	
	Vert	3.21	7.78	1.25	0.70	0.30	4.00	1.96	0.47	6.57	0.61	5.22
5	Vert	23.99		0.25	0.91	0.41		4.25	1.53		1.90	
	Vert	3.15	7.63	0.25	0.95	0.06	6.83	1.46	0.75	2.04	0.34	5.57
6	Vert	23.46		0.25	0.70	1.15		2.95	3.82		3.42	
	Vert	3.12	7.52	0.25	0.70	0.30	3.83	0.96	1.17	3.26	0.81	4.23
7	Vert	26.40		0.75	0.68	1.24		3.43	3.10		3.60	
	Vert	3.65	7.23	0.75	0.71	0.33	3.76	1.57	0.41	7.56	0.76	4.73

Due to the large model, it was difficult to set the discharge and relative submergence levels exactly at the values required to achieve precise Froude number similitude. The ratio of q for a 1:4 model is nominally 8, column 4 of Table 1 shows the observed ratio of q. Similarly the values in columns 8 and 11 should nominally equal 4. Column 13 reflects the apparent scale of the pairs of data considering the measured values of the data.

Predictive Equation

The data from the 88 tests conducted were analyzed using a non-linear regression algorithm (Dixon, 1983) to determine an empirical equation for the prediction of local scour at grade-control structures with partially submerged

flow. This analysis determined a separate equation for each face slope and is written:

$$D_s = C_1 \frac{q^{0.67}}{g^{0.33}} \left[\frac{\Delta H}{y_t}\right]^{P_1} (Sr)^{P_2} \qquad (1)$$

The values of C_1, P_1 and P_2 for each face slope, as well as the mean square error of prediction for equation (1) is shown in Table 2.

Table 2

Parameter Values in Equation (1)

Face Slope	C_1	P_1	P_2	MSE
(1)	(2)	(3)	(4)	(5)
Vertical	1.83	0.411	-0.118	1.81
1:1	1.70	0.158	-0.134	8.20

Results and Discussion

Three methods can be used to compare the predictive ability of the physical model to prototype results. First, a simple examination of the ratio in column 11 of Table 1. This ratio should nominally equal 4 for all pairs. The average value of this ratio is 5.96, this average is not different from 4.0 at even the 20% significance level using the standard t-test.

Second, predictions using equation (1) for the 3 cfs/ft cases are multiplied by the nominal scale of 4, and the results compared to the observed values of Ds for the 25 cfs/ft cases. This is shown in Figure 2. And, lastly, predictions using equation (1) for the lower (model) flow is multiplied by the apparent scale in column 13, and the results again compared to the observed values of the 25 cfs/ft (prototype) flow. This comparison is also given in Figure 2. The average value of the apparent scale (column 13) is 4.52, which is not different from the nominal scale of 4.0 at the much more restrictive 5% significance level.

Examination of Figure 2 reveals that all predicted points are within 25% of the observed values, using both the apparent scale and the nominal scale.

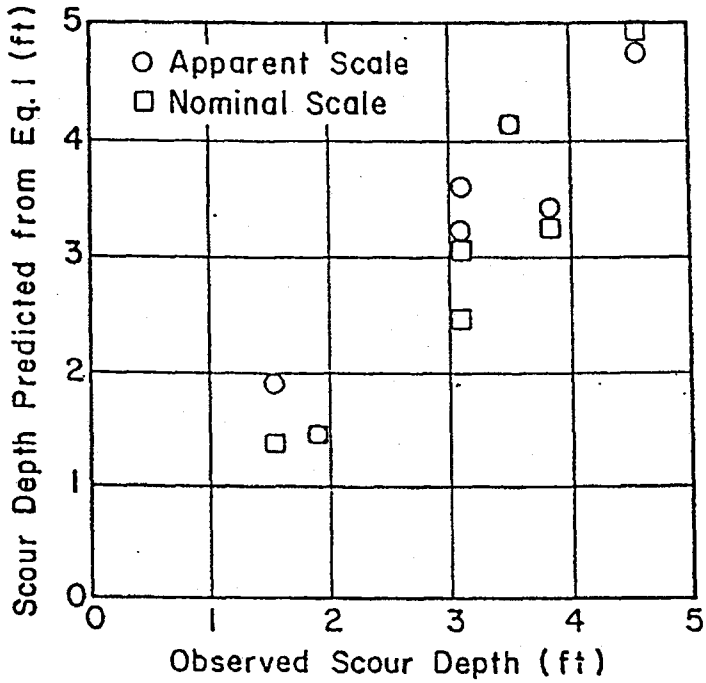


Figure 2. Predicted vs Observed Scour Depths.

Conclusions

Despite variations up to approximately 10% in flow rates and water levels, the physical model results (in the form of equation (1)) can predict observed values of local scour for prototype flows of 25 cfs/ft with an error less than 25%. Results of predictions using an apparent scale to reflect variations of experimental conditions are not statistically different from results using the nominal scale of 4. These comparisons imply that the model results (equation (1)) can predict prototype scour depths for flows up to 200 cfs/ft with an error less than 25%, if prototype flow conditions are determined with an error of less than 10%.

Acknowledgments

The writer is grateful to Mr. Charles H. Huckelberry, Director of the Pima County, Arizona, Department of Transportation and Flood Control District for permission to use data from a project supported by the District. The data were collected by the writer while employed at Simons, Li and Associates, Inc.

Appendix I. - References

1. Dixon, W. J., editor, BMDP Statistical Software, University of California Press, Berkeley, 1983.

Appendix II. - Notation

Dp = drop height of grade-control structure, measured vertically from the crest to the original (unscoured) bed elevation downstream

Ds = equilibrium clear-water scour depth, measured vertically from the original bed elevation to the point of maximum scour

d_x = standard sieve diameter of sediment for which x percent is finer, by weight

g = gravitational constant, 32.2 ft/sec^2

ΔH = difference in water surface elevation upstream and downstream of the grade-control structure

MSE = mean square error of prediction

Sf = slope of the face of the grade control structure given as a ratio of vertical to horizontal distance

Sr = relative submergence calculated as $\left[\frac{y_t - D_p}{y_1} \right]$

y_1 = depth of water upstream of grade-control structure measured vertically from the crest

y_t = depth of tailwater downstream of grade-control structure measured vertically from the original bed elevation

Appendix III. - Conversion Factors

1 cfs = 0.028 m^3/sec

1 cfs/ft = 0.093 m^2/sec

1 ft = 0.305 m

1 hp = 745.7 watts

Keywords

Scour
Local Scour
Grade-Control Structures
Drop Structures
Degradation
Hydraulic Model
Alluvial Hydraulics

Wickenburg Bypass Highway Embankment Antidune and Zeller Scour by Cross-Section
Leo R. Kreymborg, PE
July 25, 2005
U.S. Units
Using n=0.025 in Channel

XS Id Number	Approx Road Station	100-yr Design Q cfs	Thalweg Elevation	Average (Mean) Velocity, ft/s	Channel Velocity, ft/s	Antidune scour = 1/2 * 0.027 * V^2 (using Channel Velocity), feet	Hydraulic Depth, ft	Maximum Depth, ft	Energy Slope	Zeller General Scour, feet	Shear Velocity, ft / s	V/Shear Velocity	Shear lbs / ft2	Stream Power, ft -lb /s/ft2
52.65	17400.2	71000	#N/A	10.09	14.46	**2.82274**	4.6	6.53	0.006	3.0	0.942955	10.7004	2.65	26.7385
52.606	17115	71000	2066.69	9.81	13.17	**2.34156**	4.27	5.8	0.00661	2.1	0.953329	10.29026	1.87	18.3447
52.558	16940	71000	2065.39	9.16	11.99	**1.94076**	4.44	6.19	0.00525	2.0	0.866444	10.57195	1.52	13.9232
52.508	16762.7	71000	2063.83	8.17	10.41	**1.46297**	4.95	6.81	0.00327	2.1	0.722277	11.31146	1.16	9.4772
52.467	16608.5	71000	2062.11	8.74	11.27	**1.71467**	4.87	7.47	0.00372	2.6	0.763567	11.44628	1.29	11.2746
52.44	16457.5	71000	2061.88	8.58	11.17	**1.68438**	4.99	7.21	0.00348	2.6	0.747232	11.48237	1.2	10.296
52.41	16331.2	71000	2060.92	8.2	10.53	**1.49689**	5.37	7.79	0.00271	2.8	0.684541	11.97882	1.12	9.184
52.384	16200.6	71000	2060.4	8.31	10.74	**1.55719**	5.26	7.91	0.00274	3.0	0.680612	12.2096	1.14	9.4734
52.345	16003.3	71000	2059.26	8.07	10.21	**1.4073**	5.68	8.57	0.00239	2.9	0.660875	12.21109	1.11	8.9577
52.316	15853.9	71000	2058.57	8.4	10.91	**1.60688**	5.97	8.69	0.00233	3.5	0.669257	12.55123	1.22	10.248
52.284	15706.1	71000	2057.38	9.11	12.74	**2.19115**	5.22	8.95	0.00323	4.6	0.737053	12.36003	1.37	12.4807
52.25	15555.2	71000	2057.09	9.47	13.61	**2.50063**	5.69	9.1	0.00356	4.5	0.807283	11.7307	1.53	14.4891
52.224	15432.1	71000	2056.62	9.77	14.38	**2.79159**	5.64	8.3	0.0039	4.4	0.841373	11.61198	1.63	15.9251
52.196	15319.6	71000	2055.97	9.58	13.64	**2.51167**	5.57	8.52	0.00352	4.4	0.795012	12.05014	1.64	15.7112
52.17	15192.5	71000	2055.34	10.7	15.03	**3.04966**	5.28	8.84	0.00474	4.7	0.89799	11.9155	2.06	22.042
52.147	15090.6	71000	2054.76	10.78	15.08	**3.06999**	5.31	8.44	0.00496	4.3	0.920443	11.71175	1.92	20.6976
52.121	14988.2	71000	2054.09	10.64	14.7	**2.91722**	5.3	8.14	0.00463	4.2	0.889099	11.96717	2.36	25.1104
52.1	14875.5	71000	2053.48	10.66	14.47	**2.82664**	5.17	8.51	0.00475	4.2	0.888774	11.99405	2.38	25.3708
52.078	14767	71000	2052.41	10.62	14.19	**2.71831**	5.07	7.87	0.00486	3.8	0.890372	11.92761	2.19	23.2578
52.054	14627.9	71000	2052.19	10.6	13.85	**2.5896**	4.93	7.09	0.00517	3.1	0.906109	11.69837	1.84	19.504
52.029	14491.3	71000	2051.58	10.56	13.95	**2.62713**	4.76	7	0.00533	3.2	0.903593	11.68668	1.95	20.592
52.006	14398.8	71000	2051.08	9.14	12.35	**2.05905**	5.31	7.36	0.00388	2.9	0.81429	11.2245	1.62	14.8068
51.983	14296.9	71000	2049.93	10.19	13.79	**2.56722**	4.72	7.51	0.00556	3.2	0.919421	11.08306	1.69	17.2211
51.957	14193.1	71000	2049.9	8.97	12.09	**1.97327**	5.38	7.31	0.0037	2.8	0.800824	11.20096	1.44	12.9168
51.932	14085.4	71000	2049.23	8.5	11.47	**1.77607**	5.68	7.66	0.00305	2.8	0.746392	11.38812	1.34	11.39

XS Id Number	Approx Road Station	Design Q cfs	Thalweg Elevation	Average (Mean) Velocity, ft/s	Channel Velocity, ft/s	Antidune scour = 1/2 * 0.027 * V^2 (using Channel Velocity), feet	Hydraulic Depth, ft	Maximum Depth, ft	Energy Slope	Zeller General Scour, feet	Shear Velocity, ft / s	V/Shear Velocity	Shear lbs / ft2	Stream Power, ft-lb /s/ft2
51.895	13902.2	71000	2048.31	8.74	12.05	**1.96023**	5.69	7.81	0.00306	3.3	0.748152	11.68212	1.39	12.1486
51.854	13730.8	71000	2046.71	9.8	13.7	**2.53382**	5.36	9.01	0.00407	4.3	0.837917	11.69567	1.56	15.288
51.829	13607.9	71000	2046.09	9.32	13.02	**2.28853**	5.81	9.3	0.0035	4.1	0.808611	11.52594	1.56	14.5392
51.807	13481.1	71000	2045.67	10.65	14.72	**2.92516**	5.21	8.83	0.00519	4.2	0.933375	11.41021	1.72	18.318
51.782	13337.6	71000	2045.16	10.08	13.86	**2.59334**	5.41	9.01	0.00438	4.1	0.873601	11.53845	1.45	14.616
51.757	13202.8	71000	2044.41	10.1	13.74	**2.54863**	5.28	9.08	0.00432	4.2	0.856813	11.78787	1.32	13.332
51.734	13059.2	71000	2043.59	8.72	11.87	**1.90211**	5.45	9.61	0.0028	4.5	0.70123	12.4353	0.96	8.3712
51.709	12873.9	71000	2042.7	7.94	10.26	**1.42111**	6.39	9.94	0.00195	3.6	0.633588	12.5318	0.88	6.9872
51.682	12691.3	71000	2042.01	7.07	9.03	**1.1008**	7.05	10.26	0.00137	3.3	0.557473	12.68222	0.71	5.0197
51.656	12544.7	71000	2040.83	6.16	7.9	**0.84254**	7.57	11.29	0.00095	3.2	0.480199	12.82801	0.55	3.388
51.638	12427.1	71000	2040.42	5.85	7.5	**0.75938**	7.91	11.67	0.00082	3.1	0.456171	12.82415	0.5	2.925
51.618	12309.9	71000	2039.82	5.4	6.96	**0.65396**	8.49	12.27	0.00065	3.0	0.420891	12.82994	0.43	2.322
51.601	12211.7	71000	2039.07	5.64	7.53	**0.76546**	8.06	12.8	0.00074	3.8	0.438535	12.86099	0.5	2.82
51.581	12093.5	71000	2038.66	5.4	7.84	**0.82979**	8.57	13.07	0.00077	3.9	0.460661	11.72228	0.56	3.024
51.558	11952.7	71000	2038.03	5.15	7.94	**0.85109**	9.01	13.61	0.00076	3.9	0.468019	11.00382	0.58	2.987
51.536	11822.9	71000	2037.74	5.18	8	**0.864**	9.58	13.82	0.00073	3.8	0.474539	10.91585	0.57	2.9526
51.516	11704.7	71000	2037.31	5.3	7.96	**0.85538**	10.06	14.18	0.00068	3.9	0.469678	11.28432	0.55	2.915
51.497	11597.5	71000	2036.9	5.76	8.77	**1.03832**	9.85	14.29	0.00077	4.8	0.494187	11.6555	0.56	3.2256
51.477	11477.7	71000	2036.39	5.68	8.42	**0.9571**	10.94	14.78	0.00067	4.3	0.486905	11.66552	0.51	2.8968
51.453	11319.7	71000	2036.01	7.09	8.94	**1.07897**	10.06	14.69	0.00075	5.3	0.493555	14.36516	0.68	4.8212
51.436	11232.2	71000	2035.5	7.95	9.59	**1.24157**	10.36	14.89	0.0009	5.2	0.547326	14.52517	0.78	6.201
51.418	11128.5	71000	2035.06	7.99	9.06	**1.10813**	12.06	15.3	0.00077	4.1	0.545045	14.65934	0.81	6.4719
51.398	10995.6	71000	2034.49	9.56	10.56	**1.50543**	12.27	15.2	0.00104	4.6	0.640705	14.92107	1.13	10.8028
51.375	10979.4	71000	2034.15	12.69	12.69	**2.17399**	11.89	14.22	0.00177	4.3	0.823898	15.4024	2.2	27.918

HEC-RAS Plan: July 21 05TS River: Hassayampa Reach: 1 Profile: 100Yr

Radius 5000

Zeller Bend Scour
Using Total Velocity and
all parameters at the current section

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # C	Vel Total (ft/s)	Max Chl Dp (ft)	Hydr Depth (ft)	
1	53.72	100Yr	71000	2098.2	2106.35	2105.07	2107.57	0.00378	9.96	8940.26	1780.55	0.64	7.94	8.15	5.02	
1	53.63	100Yr	71000	2096	2103.28	2102.57	2105.11	0.006309	11.6	6822.89	1243.38	0.81	10.41	7.28	5.49	
1	53.54	100Yr	71000	2092.7	2098.85	2098.85	2101.19	0.009533	12.76	5967.21	1233.28	0.97	11.9	6.15	4.84	
1	53.44	100Yr	71000	2090.6	2095.82	2094.86	2096.82	0.005605	8.56	8943.53	2072.6	0.72	7.94	6.52	4.32	
1	53.35	100Yr	71000	2087.8	2094.5	2094.5	2094.95	0.00225	6.32	16927.58	2856.93	0.47	4.19	9.1	5.93	
1	53.25	100Yr	71000	2085	2091.11	2089.89	2091.85	0.003899	7.92	10691.48	2474.47	0.61	6.64	6.11	4.32	
1	53.16	100Yr	71000	2082.2	2088.38	2087.7	2089.49	0.005996	9.89	9539.62	2514.21	0.76	7.44	6.18	3.79	2.2
1	53.13	100Yr	71000	2080.54	2087.03	2086.84	2088.5	0.005021	11.7	8657.59	2381.8	0.96	8.2	6.49	3.63	2.6
1	53.097	100Yr	71000	2079.31	2086.33	2085.98	2087.63	0.004474	11.11	9117.19	2650.19	0.91	7.79	7.02	3.71	3.0
1	53.063	100Yr	71000	2078.53	2085.36	2085.04	2086.84	0.005255	11.64	8307.15	2409.45	0.97	8.55	6.83	3.79	2.8
1	53.026	100Yr	71000	2077.33	2084.33	2084.03	2085.8	0.00546	11.64	8181.36	2110.2	0.99	8.68	7	3.92	2.5
1	52.986	100Yr	71000	2076.57	2083.79	2082.57	2084.75	0.003119	9.46	9813.22	2332.49	0.76	7.24	8.02	4.77	3.0
1	52.949	100Yr	71000	2075.67	2083.25	2081.95	2084.24	0.002646	9.54	9941.49	2156.6	0.72	7.14	7.58	5.1	2.7
1	52.916	100Yr	71000	2074.98	2082.92	2081.37	2083.85	0.002141	9.26	10511.01	1972.37	0.66	6.75	7.95	5.33	2.6
1	52.881	100Yr	71000	2074.15	2082.62	2080.68	2083.51	0.001702	8.97	10658.63	1744.14	0.6	6.66	8.47	6.11	2.5
1	52.847	100Yr	71000	2073.07	2081.39	2080.67	2083.04	0.00314	11.85	8123.49	1527.78	0.81	8.74	8.32	5.32	2.4
1	52.813	100Yr	71000	2072.18	2079.97	2079.97	2082.31	0.004879	14.27	6892.39	1406.14	1	10.3	7.79	4.9	2.1
1	52.778	100Yr	71000	2071.07	2078.7	2078.7	2081.06	0.004669	14.28	6892.74	1377.93	0.98	10.3	7.63	5	2.0
1	52.74	100Yr	71000	2069.72	2077.54	2077.54	2079.94	0.005149	14.79	6768.27	1333.16	1.02	10.49	7.82	5.08	2.0
1	52.693	100Yr	71000	2068.91	2076.16	2076.16	2078.42	0.005378	14.79	7085.11	1537.16	1.04	10.02	7.25	4.77	2.1
1	52.65	100Yr	71000	2067.95	2074.48	2074.48	2076.63	0.006003	14.46	7036.51	1691.79	1.08	10.09	6.53	4.6	2.0
1	52.606	100Yr	71000	2066.69	2072.38	2072.19	2074.2	0.00661	13.17	7238.1	1887.66	1.09	9.81	5.8	4.27	1.9
1	52.558	100Yr	71000	2065.39	2071.34	2070.87	2072.92	0.005251	11.99	7754.95	1850.15	0.98	9.16	6.19	4.44	2.0
1	52.508	100Yr	71000	2063.83	2070.55	2069.55	2071.82	0.003273	10.41	8693.48	1800.96	0.79	8.17	6.81	4.95	2.2
1	52.467	100Yr	71000	2062.11	2069.58	2068.88	2071.08	0.003718	11.27	8124.45	1731.83	0.85	8.74	7.47	4.87	2.4
1	52.44	100Yr	71000	2061.88	2069.09	2068.36	2070.56	0.003475	11.17	8271.39	1746.26	0.82	8.58	7.21	4.99	2.3
1	52.41	100Yr	71000	2060.92	2068.71	2067.71	2070.06	0.00271	10.53	8662.81	1739.69	0.74	8.2	7.79	5.37	2.5
1	52.384	100Yr	71000	2060.4	2068.25	2067.28	2069.67	0.002735	10.74	8544.08	1814.46	0.75	8.31	7.91	5.26	2.7
1	52.345	100Yr	71000	2059.26	2067.83	2066.44	2069.11	0.002388	10.21	8796.38	1655.92	0.7	8.07	8.57	5.68	2.6
1	52.316	100Yr	71000	2058.57	2067.26	2066.12	2068.74	0.00233	10.91	8453.88	1498.15	0.71	8.4	8.69	5.97	2.5
1	52.284	100Yr	71000	2057.38	2066.33	2065.92	2068.24	0.003232	12.74	7792.97	1794.14	0.83	9.11	8.95	5.22	3.1
1	52.25	100Yr	71000	2057.09	2065.61	2065.18	2067.64	0.003557	13.61	7494.85	1423.5	0.87	9.47	9.1	5.69	2.4
1	52.224	100Yr	71000	2056.62	2064.92	2064.63	2067.12	0.003898	14.38	7269.25	1299.45	0.92	9.77	8.3	5.64	2.0
1	52.196	100Yr	71000	2055.97	2064.49	2064.07	2066.57	0.003524	13.64	7409.58	1339.9	0.87	9.58	8.52	5.57	2.2
1	52.17	100Yr	71000	2055.34	2063.44	2063.44	2065.98	0.004743	15.03	6637.4	1285.19	1	10.7	8.84	5.28	2.2
1	52.147	100Yr	71000	2054.76	2062.6	2062.6	2065.14	0.004955	15.08	6585.75	1277.45	1.02	10.78	8.44	5.31	2.1
1	52.121	100Yr	71000	2054.09	2061.98	2061.98	2064.47	0.004632	14.7	6673.38	1300.72	0.98	10.64	8.14	5.3	2.1
1	52.1	100Yr	71000.00	2053.48	2061.16	2061.16	2063.62	0.004745	14.47	6659.52	1324.67	0.99	10.66	8.51	5.17	2.2
1	52.078	100Yr	71000	2052.41	2060.28	2060.28	2062.67	0.004856	14.19	6686.5	1345.03	0.99	10.62	7.87	5.07	2.1
1	52.054	100Yr	71000	2052.19	2059.28	2059.18	2061.57	0.005172	13.85	6698.34	1601.7	1.01	10.6	7.09	4.93	2.2
1	52.029	100Yr	71000	2051.58	2058.58	2058.58	2060.89	0.005327	13.95	6721.98	1654.79	1.02	10.56	7	4.76	2.2
1	52.006	100Yr	71000	2051.08	2058.44	2057.75	2060.12	0.003878	12.35	7766.74	1879.58	0.88	9.14	7.36	5.31	2.5
1	51.983	100Yr	71000	2049.93	2057.44	2057.35	2059.56	0.005562	13.79	6968.01	1932.1	1.04	10.19	7.51	4.72	2.7
1	51.957	100Yr	71000	2049.9	2057.21	2056.47	2058.83	0.003702	12.09	7913.73	1900.14	0.86	8.97	7.31	5.38	2.5
1	51.932	100Yr	71000	2049.23	2056.89	2055.88	2058.36	0.003046	11.47	8352.48	1805.46	0.79	8.5	7.66	5.68	2.5
1	51.895	100Yr	71000	2048.31	2056.12	2055.3	2057.75	0.003055	12.05	8126.85	1623.24	0.8	8.74	7.81	5.69	2.3
1	51.854	100Yr	71000	2046.71	2054.88	2054.59	2056.98	0.004068	13.7	7243.03	1495.13	0.92	9.8	9.01	5.36	2.6
1	51.829	100Yr	71000	2046.09	2054.56	2053.9	2056.42	0.003495	13.02	7617.12	1423.24	0.86	9.32	9.3	5.81	2.4
1	51.807	100Yr	71000	2045.67	2053.45	2053.45	2055.86	0.005193	14.72	6667.98	1507.99	1.02	10.65	8.83	5.21	2.5
1	51.782	100Yr	71000	2045.16	2052.9	2052.63	2055.06	0.004381	13.86	7042.52	1428.2	0.95	10.08	9.01	5.41	2.4
1	51.757	100Yr	71000	2044.41	2052.29	2052.08	2054.47	0.004318	13.74	7028.1	1419.03	0.94	10.1	9.08	5.28	2.5
1	51.734	100Yr	71000	2043.59	2052.21	2051.35	2053.87	0.002802	11.87	8143.3	1606.3	0.77	8.72	9.61	5.45	3.0
1	51.709	100Yr	71000	2042.7	2052.15	2050.57	2053.44	0.001951	10.26	8946.74	1609.57	0.65	7.94	9.94	6.39	3.0
1	51.682	100Yr	71000	2042.01	2052.1	2049.78	2053.11	0.001369	9.03	10046.71	1866.05	0.55	7.07	10.26	7.05	3.4
1	51.656	100Yr	71000	2040.83	2052.12	2048.99	2052.89	0.000946	7.9	11527.95	1864.11	0.46	6.16	11.29	7.67	3.6
1	51.638	100Yr	71000	2040.42	2052.09	2048.57	2052.78	0.000817	7.5	12129.93	1919.04	0.43	5.85	11.67	7.91	3.8

HEC-RAS Plan: July 21 05TS River: Hassayampa Reach: 1 Profile: 100Yr

Radius 5000

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # C	Vel Total (ft/s)	Max Chl Dp (ft)	Hydr Depth (ft)	Zeller Bend Scour
1	51.618	100Yr	71000	2039.82	2052.09	2047.92	2052.67	0.000648	6.96	13151.27	2117.57	0.39	5.4	12.27	8.49	4.2
1	51.601	100Yr	71000	2039.07	2051.87	2047.82	2052.59	0.000741	7.53	12592.85	2220.52	0.42	5.64	12.8	8.06	4.7
1	51.581	100Yr	71000	2038.66	2051.73	2047.56	2052.51	0.000769	7.84	13148.5	2193.5	0.43	5.4	13.07	8.57	4.4
1	51.558	100Yr	71000	2038.03	2051.64	2047.17	2052.41	0.000755	7.94	13793.84	2032.22	0.43	5.15	13.61	9.01	4.1
1	51.536	100Yr	71000	2037.74	2051.56	2046.72	2052.32	0.00073	8	13702.88	2326.34	0.42	5.18	13.82	9.58	4.6
1	51.516	100Yr	71000	2037.31	2051.49	2046.38	2052.24	0.000681	7.96	13383.66	2088.86	0.41	5.3	14.18	10.06	4.4
1	51.497	100Yr	71000	2036.9	2051.19	2046.83	2052.11	0.00077	8.77	12319.42	1827.18	0.44	5.76	14.29	9.85	4.1
1	51.477	100Yr	71000	2036.39	2051.17	2045.73	2051.97	0.000673	8.42	12506.32	1839.91	0.41	5.68	14.78	10.94	4.2
1	51.453	100Yr	71000	2036.01	2050.7	2046.09	2051.79	0.000752	8.94	10011.15	1756.58	0.44	7.09	14.69	10.06	4.8
1	51.436	100Yr	71000	2035.5	2050.39	2046.15	2051.66	0.000898	9.59	8928.41	1708.44	0.48	7.95	14.89	10.36	4.9
1	51.418	100Yr	71000	2035.06	2050.36	2045.22	2051.53	0.000765	9.06	8888.41	1599.04	0.44	7.99	15.3	12.06	4.7
1	51.398	100Yr	71000	2034.49	2049.69	2045.2	2051.3	0.001039	10.56	7427.94	1435.92	0.52	9.56	15.2	12.27	4.4
1	51.375	100Yr	71000	2034.15	2048.37	2045.42	2050.87	0.001773	12.69	5595.16	470.76	0.65	12.69	14.22	11.89	-0.1
1	51.37		Bridge													#DIV/0!
1	51.355	100Yr	71000	2033.47	2046.12	2044.91	2049.56	0.002984	14.88	4772.58	470.76	0.82	14.88	12.65	10.14	-0.1
1	51.345	100Yr	71000	2033.21	2046.27	2043.92	2049.11	0.002585	13.53	5246.67	459.24	0.71	13.53	13.06	11.42	
1	51.338	100Yr	71000	2033.1	2046.21	2043.67	2048.97	0.00263	13.35	5319.48	462.6	0.69	13.35	13.11	11.5	-0.2
1	51.33		Bridge													#DIV/0!
1	51.331	100Yr	71000	2032.83	2043.91	2043.91	2048.4	0.005919	16.99	4177.91	462.6	1	16.99	11.08	9.03	-0.2
1	51.303	100Yr	71000	2032.04	2043.13	2043.13	2047.37	0.004003	17.12	4700.63	1298.09	0.97	15.1	11.09	8.28	
1	51.275	100Yr	71000	2031.15	2041.6	2041.6	2045.53	0.004093	16.73	4989.62	1381.05	0.97	14.23	10.45	7.86	3.1
1	51.208	100Yr	71000	2029.31	2039.36	2039.36	2042.48	0.003949	16.22	6368.38	1876.35	0.95	11.15	10.05	6.46	3.6
1	51.13	100Yr	71000	2026.3	2037.45	2037.07	2039.48	0.004131	14.57	8181.59	1622.71	0.94	8.68	11.15	5.04	3.2
1	51.2	PF 1	71000	2028.5	2039.47	2039	2042.02	0.006575	14.69	6263.05	1904.12	0.87	11.34	10.97	6.14	3.6
1	51.13	PF 1	71000	2026.3	2037.45	2036.54	2038.94	0.005774	12.31	8181.59	1622.71	0.79	8.68	11.15	5.04	2.9

HEC-RAS Plan: Prop Cond Ju River: Hassayampa Reach: 1 Profile: 100Yr

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # C	Vel Total	Max Chl Dp	Hydr Depth	Radius 3700
HEC-RAS Plan: Prop Cond Ju River: Hassayampa Reach: 1 Profile: 100Yr																
Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Flow Area (sq ft)	Top Width (ft)	Froude # C	Vel Total (ft/s)	Max Chl Dp (ft)	Hydr Depth (ft)	Zeller Bend Scour Using Total Velocity and all parameters at the current section
1	53.72	100Yr	71000	2098.2	2106.35	2105.07	2107.57	0.00378	9.98	8940.26	1780.55	0.64	7.94	8.15	5.02	
1	53.63	100Yr	71000	2096	2103.28	2102.57	2105.11	0.006309	11.6	6822.89	1243.38	0.81	10.41	7.28	5.49	
1	53.54	100Yr	71000	2092.7	2098.85	2098.85	2101.19	0.009533	12.76	5967.21	1233.28	0.97	11.9	6.16	4.84	
1	53.44	100Yr	71000	2090.6	2095.82	2094.86	2096.82	0.005605	8.56	8943.53	2072.6	0.72	7.94	6.52	4.32	
1	53.35	100Yr	71000	2087.8	2094.5	2094.5	2094.95	0.00225	6.32	16927.58	2856.93	0.47	4.19	9.1	5.93	3.1
1	53.25	100Yr	71000	2085	2091.13	2089.89	2091.87	0.003849	7.89	10733.77	2474.62	0.61	6.61	6.13	4.34	2.6
1	53.16	100Yr	71000	2082.2	2088.32	2087.7	2089.46	0.006305	10.05	9370.48	2506.09	0.78	7.58	6.11	3.74	2.7
1	53.13	100Yr	71000	2080.54	2087.53	2086.5	2088.42	0.004907	8.84	9852.04	2428.95	0.69	7.21	6.99	4.06	3.0
1	53.097	100Yr	71000	2079.31	2086.77	2085.74	2087.59	0.004554	8.49	10203.27	2714.9	0.66	6.96	7.46	4.06	3.4
1	53.063	100Yr	71000	2078.53	2085.86	2084.81	2086.81	0.005197	8.88	9409.81	2465.56	0.7	7.55	7.33	4.23	3.2
1	53.026	100Yr	71000	2077.33	2084.9	2083.77	2085.82	0.004955	8.6	9374.58	2122.85	0.69	7.57	7.57	4.48	3.0
1	52.986	100Yr	71000	2076.57	2084.15	2082.38	2084.86	0.003342	7.34	10550.87	2334.09	0.57	6.73	8.38	5.12	3.4
1	52.949	100Yr	71000	2075.67	2083.58	2081.7	2084.3	0.003034	7.58	10574.74	2163.39	0.55	6.71	7.91	5.42	3.1
1	52.916	100Yr	71000	2074.98	2083.17	2081.11	2083.86	0.002646	7.55	11001.61	1975.56	0.52	6.45	8.2	5.57	3.0
1	52.881	100Yr	71000	2074.15	2082.72	2080.36	2083.43	0.002341	7.59	10843.02	1744.42	0.5	6.55	8.57	6.22	2.9
1	52.847	100Yr	71000	2073.07	2081.61	2080.36	2082.84	0.004274	10.08	8451.9	1529.41	0.67	8.4	8.54	5.53	2.8
1	52.813	100Yr	71000	2072.18	2080.24	2079.65	2081.92	0.006306	11.92	7274.12	1413.58	0.81	9.76	8.06	5.15	2.5
1	52.778	100Yr	71000	2071.07	2079.22	2078.36	2080.76	0.005278	11.4	7604.82	1389.28	0.75	9.34	8.15	5.47	2.5
1	52.74	100Yr	71000	2069.72	2077.87	2077.17	2079.55	0.006232	12.02	7210.65	1339.17	0.81	9.85	8.15	5.38	2.4
1	52.693	100Yr	71000	2068.91	2076.28	2075.83	2077.95	0.007074	12.27	7264.72	1540.67	0.86	9.77	7.37	4.88	2.5
1	52.65	100Yr	71000	2067.95	2074.3	2074.14	2076.16	0.009513	12.72	6767.19	1689.71	0.97	10.49	6.35	4.43	2.3
1	52.606	100Yr	71000	2066.69	2072.9	2071.96	2074.12	0.006136	9.75	8123.99	1893.49	0.77	8.74	6.32	4.78	2.4
1	52.558	100Yr	71000	2065.39	2071.97	2070.65	2072.99	0.004754	8.87	8863.42	1857.18	0.68	8.01	6.82	5.05	2.5
1	52.508	100Yr	71000	2063.83	2071.1	2069.33	2071.97	0.003468	8.17	9669.29	1806.62	0.59	7.34	7.36	5.5	2.6
1	52.467	100Yr	71000	2062.11	2070.29	2068.64	2071.24	0.003639	8.64	9311.3	1742.01	0.61	7.63	8.18	5.56	2.9
1	52.44	100Yr	71000	2061.88	2069.8	2068.1	2070.73	0.003442	8.59	9456.05	1781.01	0.6	7.51	7.92	5.61	2.8
1	52.41	100Yr	71000	2060.92	2069.34	2067.47	2070.24	0.00298	8.4	9681.73	1746.84	0.56	7.33	8.42	5.99	2.9
1	52.384	100Yr	71000	2060.4	2068.87	2067.09	2069.81	0.00308	8.66	9556.71	1829.98	0.57	7.43	8.53	5.84	3.1
1	52.345	100Yr	71000	2059.26	2068.22	2066.24	2069.18	0.00312	8.66	9410.8	1686.98	0.58	7.54	8.96	5.98	3.1
1	52.316	100Yr	71000	2058.57	2067.59	2065.86	2068.69	0.003162	9.35	8926.82	1501.65	0.59	7.95	9.02	6.3	2.9
1	52.284	100Yr	71000	2057.38	2066.88	2065.64	2068.12	0.003737	10.26	8604.73	1804.42	0.64	8.25	9.5	5.74	3.5
1	52.25	100Yr	71000	2057.09	2066.1	2064.8	2067.44	0.004067	10.84	8139.36	1429.62	0.67	8.72	9.59	6.17	3.0
1	52.224	100Yr	71000	2056.62	2065.48	2064.22	2066.88	0.004205	11.19	7996.63	1306.59	0.69	8.88	8.86	6.2	2.5
1	52.196	100Yr	71000	2055.97	2064.8	2063.67	2066.28	0.004495	11.29	7816	1353.97	0.71	9.08	8.83	5.83	2.7
1	52.17	100Yr	71000	2055.34	2063.7	2063.05	2065.54	0.006146	12.52	6962.56	1296.02	0.82	10.2	9.1	5.51	2.7
1.00	52.15	100Yr	71000.00	2054.76	2063.08	2062.27	2064.78	0.005634	12.02	7187.79	1297.23	0.78	9.88	8.92	5.72	2.61
1	52.121	100Yr	71000	2054.09	2062.02	2061.63	2064	0.006938	12.89	6719.21	1301.44	0.86	10.57	8.18	5.33	2.4
1	52.1	100Yr	71000	2053.48	2061.11	2060.86	2063.15	0.007552	12.98	6602.64	1324.07	0.89	10.75	8.46	5.13	2.6
1	52.078	100Yr	71000	2052.41	2060.5	2059.99	2062.3	0.006626	12.1	6973.2	1347.88	0.83	10.18	8.09	5.28	2.5
1	52.054	100Yr	71000	2052.19	2059.91	2058.92	2061.39	0.005462	10.88	7553.46	1619.12	0.75	9.4	7.72	5.5	2.7
1	52.029	100Yr	71000	2051.58	2059.18	2058.32	2060.67	0.00567	10.97	7559.13	1665.95	0.77	9.39	7.59	5.33	2.7
1	52.006	100Yr	71000	2051.08	2058.79	2057.44	2060	0.004551	9.92	8282.74	1886.57	0.69	8.57	7.71	5.66	2.9
1	51.983	100Yr	71000	2049.93	2058.18	2057.05	2059.46	0.005082	10.24	8056.6	1961.44	0.72	8.81	8.25	5.44	3.2
1	51.957	100Yr	71000	2049.9	2057.75	2056.14	2058.84	0.003936	9.42	8700.38	1942.23	0.64	8.16	7.85	5.91	3.0
1	51.932	100Yr	71000	2049.23	2057.33	2055.56	2058.36	0.003496	9.16	8992.83	1813.13	0.61	7.9	8.1	6.11	2.9
1	51.895	100Yr	71000	2048.31	2056.52	2054.98	2057.66	0.003664	9.78	8699.09	1640.32	0.63	8.16	8.21	6.09	2.8
1	51.854	100Yr	71000	2046.71	2055.5	2054.24	2056.83	0.00428	10.64	8091.53	1551.94	0.68	8.77	9.63	5.97	3.2
1	51.829	100Yr	71000	2046.09	2055.01	2053.54	2056.28	0.004025	10.39	8209.35	1446.25	0.67	8.65	9.75	6.25	3.0
1	51.807	100Yr	71000	2045.67	2054.39	2053.09	2055.76	0.00449	10.7	7881.39	1547.29	0.69	9.01	9.77	6.15	3.2
1	51.782	100Yr	71000	2045.16	2053.97	2052.28	2055.16	0.003649	9.99	8437.06	1438.67	0.63	8.42	10.08	6.47	3.1
1	51.757	100Yr	71000	2044.41	2053.56	2051.73	2054.69	0.003307	9.66	8722.14	1436.51	0.61	8.14	10.35	6.53	3.2
1	51.734	100Yr	71000	2043.59	2053.34	2051.06	2054.25	0.002437	8.7	9837.67	1622.46	0.53	7.22	10.74	6.54	3.6
1	51.709	100Yr	71000	2042.7	2053.1	2050.33	2053.91	0.001981	7.98	10287.73	1622.52	0.48	6.9	10.89	7.33	3.6
1	51.682	100Yr	71000	2042.01	2052.94	2049.57	2053.61	0.00152	7.25	11241.35	1887.19	0.42	6.32	11.1	7.86	4.0

HEC-RAS Plan: Prop Cond Ju River: Hassayampa Reach: 1 Profile: 100Yr

Radius 3700

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # C	Vel Total	Max Chl Dp	Hydr Depth	Zeller Bend Scour
HEC-RAS Plan: Prop Cond Ju			River: Hassayampa Reach: 1 Profile: 100Yr													
1	51.656	100Yr	71000	2040.83	2052.87	2048.77	2053.4	0.001106	6.43	12663.68	2008.76	0.36	5.61	12.04	8.31	4.4
1	51.638	100Yr	71000	2040.42	2052.8	2048.36	2053.28	0.000972	6.14	13211.85	2058.08	0.34	5.37	12.38	8.61	4.6
1	51.618	100Yr	71000	2039.82	2052.75	2047.68	2053.17	0.000786	5.72	14181	2234.81	0.31	5.01	12.93	9.15	5.0
1	51.601	100Yr	71000	2039.07	2052.56	2047.66	2053.08	0.000944	6.35	13664.45	2263.03	0.34	5.2	13.49	8.72	5.2
1	51.581	100Yr	71000	2038.66	2052.41	2047.32	2052.97	0.001004	6.67	14188	2274.07	0.35	5	13.75	9.23	4.9
1	51.558	100Yr	71000	2038.03	2052.3	2046.92	2052.85	0.000977	6.71	14794.25	2055.8	0.35	4.8	14.27	9.65	4.6
1	51.536	100Yr	71000	2037.74	2052.19	2046.44	2052.73	0.000942	6.73	14599.16	2333.19	0.35	4.86	14.45	10.2	5.1
1	51.516	100Yr	71000	2037.31	2052.09	2046.08	2052.63	0.000881	6.69	14176.14	2095.47	0.34	5.01	14.78	10.65	4.9
1	51.497	100Yr	71000	2036.9	2051.85	2046.58	2052.51	0.001008	7.43	13151	1848.84	0.36	5.4	14.95	10.52	4.7
1	51.477	100Yr	71000	2036.39	2051.79	2045.4	2052.37	0.00085	6.98	13214.95	1858.32	0.34	5.37	15.4	11.56	4.9
1	51.453	100Yr	71000	2036.01	2051.31	2046.01	2052.16	0.001107	8	10617.29	1801.43	0.38	6.69	15.3	10.66	5.4
1	51.436	100Yr	71000	2035.5	2050.98	2046.1	2052	0.001342	8.64	9441.22	1764.85	0.42	7.52	15.48	10.95	5.5
1	51.418	100Yr	70100	2035.06	2050.9	2045.05	2051.85	0.001167	8.21	9279.65	1635.06	0.39	7.55	15.83	12.58	5.3
1	51.398	100Yr	70100	2034.49	2050.25	2045.05	2051.59	0.001602	9.63	7767.03	1466.71	0.46	9.03	15.76	12.8	5.0
1	51.375	100Yr	70100	2034.15	2048.75	2045.33	2051.03	0.003057	12.14	5773.3	470.76	0.61	12.14	14.59	12.26	0.8
1	51.37		Bridge													
1	51.355	100Yr	70100	2033.47	2046.52	2044.85	2049.62	0.005021	14.13	4961.25	470.76	0.77	14.13	13.05	10.54	0.8
1	51.345	100Yr	70100	2033.21	2046.5	2043.84	2049.16	0.00404	13.09	5353.36	459.31	0.68	13.09	13.29	11.66	0.7
1	51.338	100Yr	70100	2033.1	2046.36	2043.67	2048.99	0.004101	13	5391.43	462.6	0.67	13	13.26	11.65	0.7
1	51.33		Bridge													
1	51.331	100Yr	70100	2032.83	2044.11	2043.83	2048.3	0.008864	16.41	4271.54	462.6	0.95	16.41	11.28	9.23	0.6
1	51.303	100Yr	70100	2032.04	2042.91	2042.91	2046.89	0.009153	16.65	4573.27	1281.5	0.95	15.33	10.87	8.07	3.4
1	51.275	100Yr	70100	2031.15	2041.35	2041.31	2045.01	0.008636	16.2	4832.09	1377.22	0.95	14.51	10.2	7.62	3.4
1	51.208	100Yr	71000	2029.31	2039.39	2038.95	2041.85	0.006212	14.56	6392.45	1878.2	0.85	11.11	10.08	6.48	4.0
1	51.13	100Yr	71000	2026.3	2037.45	2036.61	2038.94	0.005774	12.31	8181.59	1622.71	0.79	8.68	11.15	5.04	3.7

HEC-RAS Plan: SFBalJul25 River: Hassayampa Reach: 1 Profile: PF 1

Radius 5000

Zeller Bend Scour
Using Total Velocity and
all parameters at the current section

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # C	Vel Total	Max Chl Dp	Hydr Depth	
HEC-RAS Plan: SFBalJul25 River: Hassayampa Reach: 1 Profile: PF 1																
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # C	Vel Total	Max Chl Dp	Hydr Depth	
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)		(ft/s)	(ft)	(ft)	
1	53.72	PF 1	120700	2098.2	2108.96		2110.36	0.003049	10.93	13841.57	1902.95	0.61	8.72	10.76	7.27	
1	53.63	PF 1	120700	2096	2105.38	2104.66	2108.09	0.006461	14.19	9450.34	1259.71	0.86	12.77	9.38	7.5	
1	53.54	PF 1	120700	2092.7	2100.84	2100.84	2104.19	0.009054	15.33	8433.68	1246.81	0.99	14.31	8.14	6.76	
1	53.44	PF 1	120700	2090.6	2097.37		2098.93	0.005895	10.72	12156.45	2084.1	0.77	9.93	8.06	5.83	
1	53.35	PF 1	120700	2087.8	2095.06		2096.12	0.004691	9.74	18535.29	2859.39	0.69	6.51	9.66	6.48	3.0
1	53.25	PF 1	120700	2085	2092.86		2093.91	0.003734	9.41	15121.65	2586.5	0.63	7.98	7.86	5.85	3.0
1	53.16	PF 1	120700	2082.2	2090		2091.56	0.006158	12	13764.35	2700.8	0.81	8.77	7.8	5.1	3.0
1	53.13	PF 1	120700	2080.54	2089.28		2090.52	0.004803	10.64	14452.35	2859.53	0.71	8.35	8.74	5.09	3.6
1	53.097	PF 1	120700	2079.31	2088.65		2089.74	0.003894	9.68	14978.38	2764.67	0.65	8.06	9.34	5.89	3.7
1	53.063	PF 1	120700	2078.53	2087.82		2089.07	0.004358	10.15	13794.8	2641.4	0.68	8.75	9.29	6.12	3.6
1	53.026	PF 1	120700	2077.33	2087.06		2088.26	0.003913	9.76	13924.31	2206.53	0.65	8.67	9.73	6.53	3.3
1	52.986	PF 1	120700	2076.57	2086.5		2087.47	0.002749	8.54	15395.03	2344.78	0.55	7.84	10.73	7.47	3.7
1	52.949	PF 1	120700	2075.67	2086.01		2087	0.002571	8.8	15325.59	2206.21	0.54	7.88	10.34	7.81	3.4
1	52.916	PF 1	120700	2074.98	2085.69		2086.61	0.002278	8.73	16023.14	2015.31	0.51	7.53	10.72	7.95	3.3
1	52.881	PF 1	120700	2074.15	2085.18		2086.22	0.002271	9.11	15146.59	1750.6	0.52	7.97	11.03	8.65	3.0
1	52.847	PF 1	120700	2073.07	2083.9		2085.63	0.004112	11.96	11981.25	1563.55	0.69	10.07	10.83	7.66	2.8
1	52.813	PF 1	120700	2072.18	2082.47		2084.75	0.005878	13.95	10525.75	1510.54	0.83	11.47	10.29	6.97	2.7
1	52.778	PF 1	120700	2071.07	2081.51		2083.64	0.005079	13.47	10871.93	1461.25	0.77	11.1	10.44	7.44	2.6
1	52.74	PF 1	120700	2069.72	2079.73		2082.34	0.007069	15.04	9786.8	1461.01	0.9	12.33	10.01	6.97	2.5
1	52.693	PF 1	120700	2068.91	2078.15		2080.58	0.007199	14.7	10063.87	1564.06	0.9	11.99	9.24	6.71	2.5
1	52.65	PF 1	120700	2067.95	2075.95	2075.87	2078.74	0.009847	15.46	9295.37	1711.6	1.03	12.98	8	6.01	2.4
1	52.606	PF 1	120700	2066.69	2074.9		2076.64	0.005575	11.63	11533.1	1915.02	0.77	10.47	8.32	6.76	2.6
1	52.558	PF 1	120700	2065.39	2074.16		2075.59	0.004215	10.52	12742.22	1944.64	0.68	9.47	9.01	6.96	2.8
1	52.508	PF 1	120700	2063.83	2073.42	2070.98	2074.66	0.003165	9.73	13891.62	1966.92	0.6	8.69	9.68	7.3	3.0
1	52.467	PF 1	120700	2062.11	2072.66	2070.32	2074	0.003276	10.17	13309.28	1773.15	0.61	9.07	10.55	7.86	3.0
1	52.44	PF 1	120700	2061.88	2072.24	2069.77	2073.53	0.003074	10.06	13614.54	1838.67	0.6	8.87	10.36	7.94	3.1
1	52.41	PF 1	120700	2060.92	2071.81	2069.15	2073.09	0.002806	9.99	13723.99	1810.06	0.57	8.79	10.89	8.24	3.2
1	52.384	PF 1	120700	2060.4	2071.41	2068.82	2072.7	0.002779	10.1	13753.7	1867.57	0.57	8.78	11.07	8.28	3.3
1	52.345	PF 1	120700	2059.26	2070.8	2067.65	2072.12	0.002835	10.18	13526.75	1743.53	0.58	8.92	11.54	8.42	3.3
1	52.316	PF 1	120700	2058.57	2070.05	2067.73	2071.64	0.003181	11.26	12422.77	1529.34	0.62	9.72	11.48	8.69	2.9
1	52.284	PF 1	120700	2057.38	2069.49	2067.4	2071.11	0.003349	11.76	12627.81	1905.85	0.64	9.56	12.11	7.97	3.8
1	52.25	PF 1	120700	2057.09	2068.54		2070.46	0.004044	12.9	11433.97	1493.05	0.7	10.56	12.03	8.35	3.0
1	52.224	PF 1	120700	2056.62	2067.82	2066.19	2069.88	0.004378	13.49	11034.8	1355.04	0.73	10.94	11.2	8.37	2.6
1	52.196	PF 1	120700	2055.97	2067.23	2065.7	2069.28	0.004354	13.29	11161.02	1445.78	0.73	10.81	11.26	7.92	2.8
1	52.17	PF 1	120700	2055.34	2065.89	2065.1	2068.52	0.006287	15.08	9849.03	1407.13	0.86	12.26	11.29	7.27	2.8
1.00	52.15	PF 1	########	2054.76	2065.24	2064.30	2067.75	0.00586	14.56	9988.71	1389.04	0.83	12.08	11.08	7.52	2.69
1	52.121	PF 1	120700	2054.09	2064.06	2063.71	2066.94	0.007158	15.54	9383.11	1404.27	0.91	12.86	10.22	6.98	2.6
1	52.1	PF 1	120700	2053.48	2063.29	2062.82	2066.09	0.007068	15.18	9414.42	1352.62	0.9	12.82	10.64	7.27	2.5
1	52.078	PF 1	120700	2052.41	2062.86	2061.9	2065.26	0.005846	13.95	10098.3	1379.32	0.82	11.95	10.45	7.59	2.5
1	52.054	PF 1	120700	2052.19	2062.48	2060.84	2064.43	0.004556	12.42	11128.65	1675.2	0.73	10.85	10.29	7.89	2.9
1	52.029	PF 1	120700	2051.58	2062.04	2060.19	2063.81	0.004076	11.92	11650.71	1703.08	0.69	10.36	10.46	8.12	3.0
1	52.006	PF 1	120700	2051.08	2061.86	2059.27	2063.3	0.003181	10.75	12788.32	1959.12	0.61	9.44	10.78	8.68	3.4
1	51.983	PF 1	120700	2049.93	2061.57	2058.82	2062.94	0.002991	10.5	13136.04	2019.17	0.6	9.19	11.64	8.72	3.7
1	51.957	PF 1	120700	2049.9	2061.36	2057.94	2062.55	0.002368	9.75	14039.1	2060.56	0.54	8.6	11.46	9.46	3.6
1	51.932	PF 1	120700	2049.23	2061.15	2057.35	2062.25	0.002053	9.39	14736.61	1944.25	0.5	8.19	11.92	9.57	3.6
1	51.895	PF 1	120700	2048.31	2060.78	2056.79	2061.87	0.001894	9.51	14802.61	1705.21	0.49	8.15	12.47	10.24	3.4
1	51.854	PF 1	120700	2046.71	2060.39	2056.1	2061.5	0.001791	9.61	14757.66	2076.99	0.48	8.18	14.52	10.7	4.6
1	51.829	PF 1	120700	2046.09	2060.21	2055.45	2061.26	0.001617	9.33	15080.32	1968.76	0.46	8	14.95	11.31	4.5
1	51.807	PF 1	120700	2045.67	2060.04	2055	2061.06	0.001552	9.11	15168.32	2016.65	0.45	7.96	15.42	11.63	4.7
1	51.782	PF 1	120700	2045.16	2059.95	2054.19	2060.84	0.001236	8.53	16485.01	1996.39	0.41	7.32	16.06	11.85	4.9
1	51.757	PF 1	120700	2044.41	2059.86		2060.66	0.001049	8.05	17471.23	2189.06	0.38	6.91	16.65	11.99	5.4
1	51.734	PF 1	120700	2043.59	2059.87		2060.49	0.000774	7.18	19816.09	2223.57	0.33	6.09	17.27	12.74	5.5
1	51.709	PF 1	120700	2042.7	2059.76		2060.38	0.000735	7.1	19808.11	2262.38	0.32	6.09	17.55	13.09	5.7
1	51.682	PF 1	120700	2042.01	2059.71		2060.26	0.000603	6.6	21213.6	2444.81	0.29	5.69	17.87	13.67	6.1
1	51.656	PF 1	120700	2040.83	2059.71		2060.16	0.000453	5.87	23178.36	2839.49	0.25	5.21	18.88	15	7.0

HEC-RAS Plan: SFBalJul25 River: Hassayampa Reach: 1 Profile: PF 1

Radius 5000

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # C	Vel Total	Max Chl Dp	Hydr Depth	Zeller Bend Scour
HEC-RAS Plan: SFBalJul25 River: Hassayampa Reach: 1 Profile: PF 1																
1	51.638	PF 1	120700	2040.42	2059.69		2060.11	0.000416	5.69	23869.88	2650.21	0.24	5.06	19.27	15.34	6.7
1	51.618	PF 1	120700	2039.82	2059.68		2060.06	0.000363	5.44	25011	3127.1	0.23	4.83	19.86	15.89	7.7
1	51.601	PF 1	120700	2039.07	2059.55		2060.01	0.000438	6.03	24720.4	3131.67	0.25	4.88	20.48	15.55	7.6
1	51.581	PF 1	120700	2038.66	2059.41		2059.95	0.00051	6.6	25246.24	2960.45	0.27	4.78	20.75	15.73	6.9
1	51.558	PF 1	120700	2038.03	2059.35		2059.89	0.000518	6.74	25978.61	2538.03	0.28	4.65	21.32	16.42	6.1
1	51.536	PF 1	120700	2037.74	2059.29		2059.83	0.000518	6.83	25696.13	2526.97	0.28	4.7	21.55	16.84	6.2
1	51.516	PF 1	120700	2037.31	2059.22	2048.43	2059.77	0.0005	6.84	24645.69	2382.49	0.27	4.9	21.91	17.29	6.2
1	51.497	PF 1	120700	2036.9	2059.07	2048.83	2059.7	0.000571	7.51	24003.3	2460.75	0.29	5.03	22.17	15.33	6.6
1	51.477	PF 1	110900	2036.39	2059.09	2047.41	2059.56	0.000436	6.68	25783.83	2445.29	0.26	4.3	22.7	14.64	6.6
1	51.453	PF 1	110900	2036.01	2058.9	2048.56	2059.47	0.000469	7.01	23425.47	2313.73	0.27	4.73	22.89	13.9	6.9
1	51.436	PF 1	110900	2035.5	2058.85	2048.55	2059.43	0.000484	7.09	22976.9	2294.79	0.27	4.83	23.35	13.79	7.1
1	51.418	PF 1	93400	2035.06	2058.72	2046.48	2059.36	0.000434	6.79	15303.57	2305.44	0.26	6.1	23.66	17.58	8.1
1	51.398	PF 1	93400	2034.49	2058.47	2046.59	2059.27	0.000518	7.52	13620.8	2092.47	0.28	6.86	23.98	18.78	7.7
1	51.375	PF 1	93400	2034.15	2057.65	2047.19	2059.01	0.000923	9.37	9965.44	470.76	0.36	9.37	23.5	21.17	-0.2
1	51.37		Bridge													
1	51.355	PF 1	93400	2033.47	2054.87		2056.58	0.001333	10.5	8892.37	470.76	0.43	10.5	21.4	18.89	-0.2
1	51.345	PF 1	93400	2033.21	2054.86		2056.46	0.001248	10.13	9218.51	462.6	0.4	10.13	21.65	19.93	-0.2
1	51.338	PF 1	93400	2033.1	2054.83	2045.51	2056.39	0.001243	10.03	9308.17	462.6	0.39	10.03	21.73	20.12	-0.2
1	51.33		Bridge													
1	51.331	PF 1	93400	2032.83	2045.68	2045.68	2051.11	0.009418	18.69	4998.25	462.6	1	18.69	12.85	10.8	-0.2
1	51.303	PF 1	93400	2032.04	2044.64	2044.64	2049.4	0.008882	18.3	5561.68	1314.66	0.96	16.79	12.6	9.66	3.0
1	51.275	PF 1	93400	2031.15	2042.92	2042.92	2047.33	0.008523	17.87	5829.69	1425.19	0.97	16.02	11.77	9.14	3.0
1	51.208	PF 1	93400	2029.31	2041.63	2040.13	2043.9	0.004367	14.13	8599.88	2038.97	0.74	10.86	12.31	8.72	4.0
1	51.13	PF 1	120700	2026.3	2039.69	2038.44	2041.71	0.005766	14.65	12002.91	1789.2	0.83	10.06	13.39	6.71	3.8

Wickenburg Bypass Highway Embankment Toe Down Calculations
Leo R. Kreymborg, PE
October 2004
U.S. Units

At Cross Section 52.147

Avg Depth US	5.31	for Zeller (hyd depth)
Max Depth US	8.44	for Zeller
Channel Vel US	15.08	for Zeller
Average Vel US	10.78	for Zeller
Upstream Energy Slope	0.004955	for Zeller

Zeller Bend Scour (using channel velocity) — Width — from Simons, Li (1985) p.5.105-5.106

Radius	Width 500	1297.23	1500	2000
3000	1.43	4.42	4.92	5.97
4000	0.64	3.46	3.93	4.92
5000	0.05	2.75	3.20	4.15
6000	**-0.41**	2.19	2.63	3.54
7000	**-0.78**	1.74	2.16	3.05
8000	**-1.10**	1.35	1.77	2.63

Zeller Bend Scour (using average velocity) — Width — from Simons, Li (1985) p.5.105-5.106

Radius	Width 500	1000	1500	2000
3000	1.10	3.38	3.76	4.56
4000	0.49	2.64	3.01	3.76
5000	0.04	2.10	2.45	3.17
6000	**-0.31**	1.68	2.01	2.71
7000	**-0.60**	1.33	1.65	2.33
8000	**-0.84**	1.04	1.35	2.01

Distance Downstream to apply bend scour 223.8 Rozovskii (1961) from Simons, Li (1985), p. 5.108

Reference: Design Manual For Engineering Analysis of Fluvial Systems
Prepared for Arizona Department of Water Resources
Simon, Li and Associates, March 1985

Calculation of scour at outlet of Sols Wash superbox
Due to flood coming from Sols Wash
To determine sufficiency / required depth of the soil cement which whose top will be flush
With the bottom of the superbox and which will toe down into the Hassayampa

Cross Section 51.477 cuts through Sols Wash near the Southern side of the Wash
The thalweg there, according to the TIN-cut cross-section is 2036.43

The bottom of the superbox will be at 2036.0
Risk of scour in the Hassayampa occurs when the ground elevation in the Hassayampa has
gone below the bottom of the superbox elevation
In which case weir flow will occur off the superbox into the Hassayampa

According to Simons & Li, 1986, scour for a 1:1 downstream face slope grade control
structure can be calculated as long as there is some submergence of the upstream weir
in this case in means the downstream water surface elevation should not drop below
the bottom of the box culvert at 2036.0

Subtracting the long-term degradation of 3.0 from the 2036.43, and assuming the Hassayampa thalweg migrates
over to Sols Wash (it's not there now), this would lead to a thalweg adjacent to
Sols Wash with an elevation of 2036.43 - 3.0 = 2033.43.

The net width perpendicular to flow at the opening of Sols Wash is 224.07 (including pier walls)
The following table shows the critical flow depth for various discharges given this width:

Q, cfs	15045 (100-year)	10000	7500	6500	5500	5000	2000	1000	500
Unit discharge, cfs per foot	67.144196	44.62891	33.47168	29.00879	24.5459	22.31446	8.925782	4.462891	2.231446
Critical Depth	5.1926258	3.954809	3.264623	2.967571	2.654814	2.491374	1.352526	0.852038	0.53675
Velocity	12.930683	11.28472	10.25285	9.775264	9.24581	8.956687	6.599343	5.237902	4.157326

Check using scour hole equation for drop structure with a vertical face:
Assuming assuming a tailwater energy head = 2033.43 + half of the critical depth in the upstream bridge

Upsteam WSE = 2036 + crit depth	2041.1926	2039.955	2039.265	2038.968	2038.655	2038.491	2037.353	2036.852	2036.537
Upstream EGL = 2036 + critical depth * 1.5	2043.7889	2041.932	2040.897	2040.451	2039.982	2039.737	2038.029	2037.278	2036.805
Downstream EGL = 2033.43 + critical depth / 2	2036.0263	2035.407	2035.062	2034.914	2034.757	2034.676	2034.106	2033.856	2033.698
Drop in head	7.7626258	6.524809	5.834623	5.537571	5.224814	5.061374	3.922526	3.422038	3.10675

Using Simons & Li, 1989 Equation
ys = 1.32 * q^0.54 * Ht^0.225 - Tailwater depth

Yscour hole	20.296	15.65469	13.06931	11.95602	10.78272	10.16883	5.854303	3.904638	below the 2033.43 elevation
Yscour, including 30% safety factor	26.3848	20.3511	16.9901	15.54283	14.01754	13.21948	7.610595	5.07603	
Scour Depth	2007.0452	2013.079	2016.44	2017.887	2019.412	2020.211	2025.819	2028.354	

The assumption if water surface downstream at 2033.43 is quite conservative, since in fact there would be some water depth
For flows higher 5500 cfs, these scour depths should not apply since the flow should be partially submerged

For the larger flows, we would expect some submergence
Using the equation from Simons & Li, 1986 for Grade Control with at 1:1 Downstream Face Slope
ys = 0.54 * q^0.67 (h/yt)^0.158 (Sr)^-0.134
Where
ys = depth of scour, ft
q = unit discharge per unit width, cfs/ft
h =drop height, ft
yt = downstream flow depth, ft
Sr = relative submergence = (yt - h) / y1
y1 = upstream flow depth

Assume downstream water surface elevation is the same critical depth going through the bridge
Since it would probably be subcritical, this actual WSE is likely to be higher
Assuming a channel bottom of 2033.43 downstream initially (thalweg - 3 foot long term scour)

Q, cfs	15045	10000	7500	6500	5500	5000	2000	1000	500
Critical Depth = assumed downstream flow depth	5.1926258	3.954809	3.264623	2.967571	2.654814	2.491374	1.352526	0.852038	0.53675
Assumed downstream flow elevation and EGL = 2033.43 + depth	2038.6226	2037.385	2036.695	2036.398	2036.085				
Delta H (upstream EGL - downstream EGL)	5.1663129	4.547405	4.202312	4.053786	3.897407				
Relative Submergence	0.5050674	0.350158	0.212773	0.133972	0.031947	-	-	-	
Yscour	9.9058566	8.09629	7.266285	7.090411	7.77018				
Yscour including 30% safety factor	12.877614	10.52518	9.446171	9.217534	10.10123				
Scour Depth	2020.5524	2022.905	2023.984	2024.212	2023.329				

For Hassayampa scour, using the 13.92 scour depth used upstream leads to 2036.43 - 13.92 = 2022.51

Use a toe-down of 2019.5 feet at the Sols Wash Bridge

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: 3 of 3
Project: Nickenburg Bypass Computed By: LRK Date: 7/25/05
Project No.: 512A01 Checked By: DJR Date: 1/23/06
Computations For: Proposed Bridge over Hassayampa River
Contraction Scour Calculations, 50-year Flood

$V_* = (32.2 * 9.46 * 0.0013)^{1/2} = 0.63 ft/s$

So $V_*/\omega = 0.63 ft/s / 0.4265 ft/sec = 1.478$

Live Bed Scour

$y_2/y_1 = (Q_2/Q_1)^{6/7} (W_1/W_2)^{K1}$

and

$y_s = y_2 - y_0$

Where

y_1 = average depth in the upstream channel (ft)
y_2 = average depth in the contracted section (ft)
y_0 = existing depth in contracted section before scour (ft) = y_1 for sand bed channels
Q_1 = flow in upstream channel transporting sediment (cubic feet per second)
Q_2 = flow in contracted channel (cubic feet per second)
W_1 = width in main channel that is transporting bed material (ft)
W_2 = width in main channel in contracted section less pier width (ft)
K_1 = 0.64 since $0.5 < V_*/\omega < 2.0$

For the upstream section y_1 = 1874.7 (active channel + right bank area) / 193.48 (active channel + right bank top width) = 9.92 ft.
For the contracted section y_2 = 9.63 ft. (avg. depth from HEC-RAS)

$y_2/y_1 = (47000/43598)^{6/7} (793.4/440.26)^{0.64}$
$= 1.555$

or

$y_2 = 1.555 * 9.92 = 15.43 ft.$

and

$y_s = y_2 - y_1 = 15.43 ft. - 9.63 ft. = 5.80 ft.$

Proposed Bridge over Hassayampa River Contraction Scour Calculation, 100-year Flood

The same model used to determine the proposed conditions flood elevations was used to calculate the contraction scour, except that the entire 71,000 was routed through the bridge rather than part of the flow going over the left bank. The 100-year flood is 71,000 cfs at the bridge.

The maximum contraction scour was found to occur using cross-section 51.477 as the upstream cross-section.

The contracted cross-section was the cross-section at the upstream face of the bridge, 51.375

Because the "right bank" region in the area near the bridge is very close to the channel elevation, the entire flow on the right bank was considered to be "channel flow" for the purpose of the upstream cross-section used for live-bed contraction scour. This gave a larger top width at the upstream section and also a larger upstream flow. This resulted in slightly higher contraction scour results than using the bank stations in HEC-RAS to define the channel flow for the contraction scour equation.

Upstream section

For cross-section 51.477 (the upstream section) the parameters for the contraction scour equation are:

Flow in upstream channel:
$Q_1 = 39564.2 + 24135.6 = 63699.8$

Top width in upstream channel:
$W_1 = 422.93$ ft (RAS Channel active top width) +
370.55 (RAS Channel right bank top width)
$= 793.48$ ft

Contracted Section

W_2 = Avg. width of bridge opening − width of piers
= 470.26 (width of bridge opening) + 12 ft. (approx. avg. width of soil cement on both sides)
− 4 piers * 45 ft. (avg. width per pier) = 440.26

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: 2 of 3
Project: Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No.: 512A01 Checked By: DEQ Date: 1/23/06
Computations For: Proposed Bridge over Hassayampa River
Contraction Scour Calculations, 100-year Flood

Flow in contracted section:
$Q_2 = 71{,}000$ (all flow goes through contracted section)

Other parameters

For a median grain diameter of 1.0mm = 0.003281 feet
Fall velocity from Figure 5.8 of HEC-18 (for D_{50} = 1mm at 20 degrees C):
Fall velocity (ω) = 0.13 m/sec = 0.4265 ft/sec

Critical Velocity

$V_c = K_u y_a^{1/6} D_{50}^{1/3}$ (HEC-18 Equation 5.1)

Where
K_u = 11.17 for English Units
y_a = Average depth of flow upstream of bridge = 5702.0 flow area / 467.86 top width = 12.19 feet (at cross-section 51.378)
D_{50} = 0.0315 feet

And
$V_c = 11.17(12.19)^{1/6}(0.00262)^{1/3}$
$= 2.34$ ft/sec

Since the velocity is larger than the critical velocity, live bed scour will occur.

Shear velocity

$V_* = (g \cdot y_a \cdot S_a)^{1/2}$

Where
g = gravitational constant, ft/s² = 32.2, ft/s²
y_a = average depth of flow upstream of bridge = 12.69 feet
S_a = energy slope = 0.0011

$V_* = (32.2 * 12.69 * 0.0011)^{1/2} = 0.670$ ft/s

So $V_*/\omega = 0.670$ ft/s / 0.4265 ft/sec = 1.57

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: 3 of 3
Project: Wickenburg / Bypass Computed By: LRK Date: 7/25/05
Project No.: 512A01 Checked By: DSR Date: 1/23/06
Computations For: Proposed Bridge over Hassayampa River
Contraction Scour Calculations, 100-year Flood

Live Bed Scour

$$y_2/y_1 = (Q_2/Q_1)^{6/7}(W_1/W_2)^{K1}$$

and

$$y_s = y_2 - y_0$$

Where

y_1 = average depth in the upstream channel (ft)
y_2 = average depth in the contracted section (ft)
y_0 = existing depth in contracted section before scour (ft)
= y_1 for sand bed channels
Q_1 = flow in upstream channel transporting sediment (cubic feet per second)
Q_2 = flow in contracted channel (cubic feet per second)
W_1 = width in main channel that is transporting bed material (ft)
W_2 = width in main channel in contracted section less pier widths (ft)
K_1 = 0.64 since $0.5 < V^*/\omega < 2.0$

For the upstream section y_1 = 10459.0 active channel + right bank area / 793.48 active channel + right bank top width = 13.18 feet
For the contracted section $y_0 = y_1$ = 12.36 ft. (avg. depth from HEC-RAS)

$$y_2/y_1 = (71000/63699.8)^{6/7}(793.48/440.26)^{0.64}$$
$$= 1.600$$

or

$$y_2 = 1.600 * 13.18 = 21.09 \text{ feet}$$

and

$$y_s = y_2 - y_1 = 21.09 \text{ feet} - 12.36 \text{ feet} = 8.73 \text{ feet}$$

WEST CONSULTANTS, INC.

Client: Jacobs / AZ Dept. of Transportation Sheet: 1 of 3
Project: US 93 Wickenburg Bypass Computed By: LRK Date: 7/25/05
Project No: Checked By: DRR Date: 1/23/06
Computations For:

Proposed Bridge over Hassayampa River Contraction Scour Calculation, Superflood

A separate model was used to calculate the parameters for the Superflood. A new model was required because (1) the flood overtops the embankments of the proposed bridge which is not allowed in the base model and (2) so that bank stations in the upstream sections could be moved to facilitate the determination of upstream parameters for the non-contracted section.

The discharge used for the superflood was 1.7 times the 100-year flood or 1.7 X 71,000 cfs = 120,700 cfs. However, if the Superflood goes over top the overbanks to the left and right of the bridge, a constant flow of 99,000 cfs, which is the flow where water is about to overtop the bridge deck, was assumed. The downstream boundary condition for the model was obtained by running the Cella Bar HEC-2 model using 120,000 and using the water surface elevation at cross-section 51.13 as the downstream boundary condition.

The maximum contraction scour was found to occur using cross-section 51.477 as the upstream cross-section.

The contracted cross-section was the cross-section at the upstream face of the bridge, 51.378.

Because the "right bank" region from the base model in the area near the bridge is very close to the channel elevation, the entire flow in this area was considered "channel flow" for the purpose of the upstream cross-section used for live-bed contraction scour. The bank stations were moved in the superflood model to encompass this low "right bank" region to facilitate the determination of the channel topwidth and velocities. Manning n-values, however, were left unchanged from the base model.

Upstream Section

For cross-section 51.477 (the upstream section) the parameters for the contraction scour equation are:

Flow in upstream channel:

$Q_1 = 81,331.9$

APPENDIX B

FLO-2D Modeling of Sols Wash

B. Sols Wash Two Dimensional Modeling

This report assumes that the Wickenburg Downtown Flood Hazard Mitigation Project will be completed before or in conjunction with the proposed embankment. In this proposed condition, Sols Wash will be channelized and leveed on both sides. Under this project, flooding in the left overbank area of Sols Wash will essentially be eliminated for the 100-year and smaller events. A Candidate Assessment Report (a preliminary study of these improvements) was completed in 2004 by Aztec Engineering on behalf of the Flood Control District of Maricopa County (Aztec, 2004).

The Wickenburg Downtown Flood Hazard Mitigation Project will prevent the overflow from a 100-year Sols Wash event from flowing into the overbanks. Another future condition, where the Sols Wash levees would not built, was also modeled hydraulically using FLO-2D (2004). The FLO-2D model documented here is *without* the construction of the Sols Wash improvements. Therefore this FLO-2D model is ***not*** a reflection of the anticipated proposed conditions. The model is therefore labeled as the "alternative proposed" conditions model. The actual proposed conditions are not known at this time because the design of the Sols Wash improvements is not complete.

Steady state two-dimensional models were developed for the existing and the "alternative" proposed conditions to study the flow pattern and difference in water surface elevations along the west embankment of the proposed bypass.

B.1. FLO-2D Model Development

The two-dimensional FLO-2D models were developed using FLO-2D version 2004.10. The models were run for the 100-year discharges in the Sols Wash (15,045 cfs). A 10-year discharge of 14,500 cfs was applied in the Hassayampa River to supply the backwater condition. The hydraulic model extends 0.75 miles to the north of the Hassayampa Bridges, and 0.70 miles to the west from the confluence of Sols Wash and Hassayampa River. To the east, the model extends 0.35 miles from the Sols Wash and Hassayampa confluence.

The FLO-2D Grid Developer System (GDS) was used to import DTM points and the computational grid with 40 feet cell size was overlaid. With the model boundary and the computational domain defined, a linear interpolation was done within the GDS to assign elevation to each of the 8,663 grid cells. Because FLO-2D performed an interpolation to arrive at the elevation of each cell, the cell elevations were subsequently reset in the input text files in order to achieve more accurate elevations. This was done externally to FLO-2D, using the digital elevation model (TIN) elevations at the center of each cell. The necessary input files for model control, tolerance, inflow, and outflow nodes were generated within FLO-2D. Seven-hour simulations were completed for the existing and the proposed condition models which was long enough for the depths to stabilize.

Manning's n-value coverage was developed for the existing and the proposed conditions using ArcView GIS polygons. Manning's n-values were assigned to each polygon based on engineering judgment and in order to match the n-values used in the Cella-Barr FIS Model and

the Goldmine Village CLOMR. The primary difference with previous models is that streets were accounted with a lower n-value of 0.03. Additionally, in some regions of the model, buildings were explicitly accounted for by raising the ground surface elevation sufficiently to prevent flow at the cells where the buildings were located. In the regions of residential housing, houses were accounted for by raising the n-value to 0.1—this is the same approach taken in the Goldmine Village CLOMR.

Table 5-2 lists the Manning's n-values corresponding to land use types. The Manning's n-value coverages were imported into the GDS and values were assigned to each grid cells. Figures 5-2 and 5-3 illustrate the Manning's n-value polygons with the assigned values.

Table B-1. Manning's n-values and corresponding land use type

Land use type	Comment	Manning's n-value
Hassayampa Channel		0.035
Hassayampa Overbank		0.045
Hassayampa Overbank with Trees		0.065
Heavy Urban		0.900
Industrial	Includes buildings	0.100
Overbank	Includes houses	0.065
Overbank with Trees		0.065
Sols Right Overbank, unvegetated parts of the left overbank where buildings where blocked out	Buildings in this region were accounted for by raising the ground elevation. The remaining terrain (except streets) was taken as n = 0.05	0.050
Sols Wash Channel		0.030
Street		0.030
Urban		0.100
Grass	Proposed condition only, on embankment slope and in ditch adjacent to embankment	0.035
Park Grass	In western portion of park	0.035

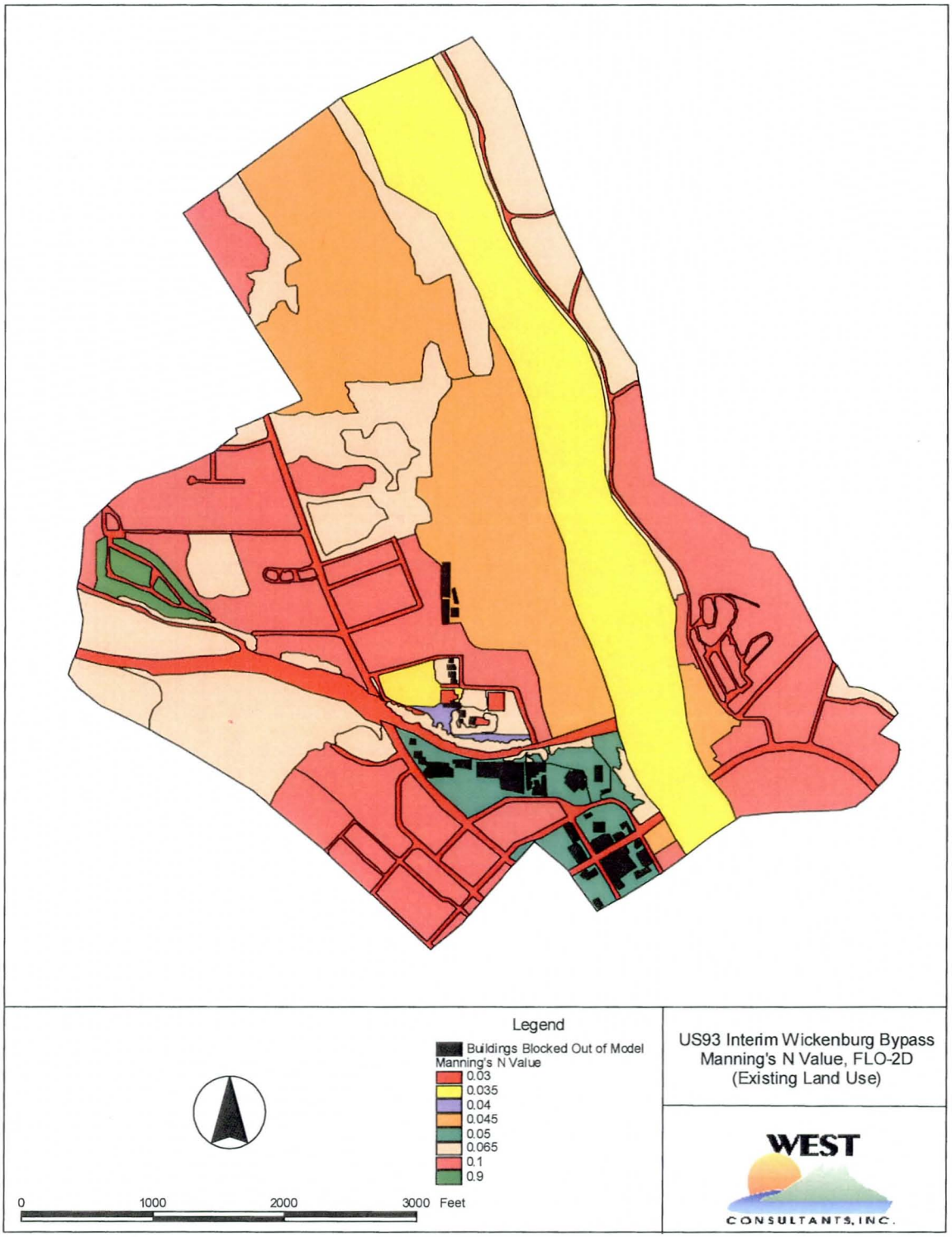


Figure B.1, Existing conditions Manning's n-values

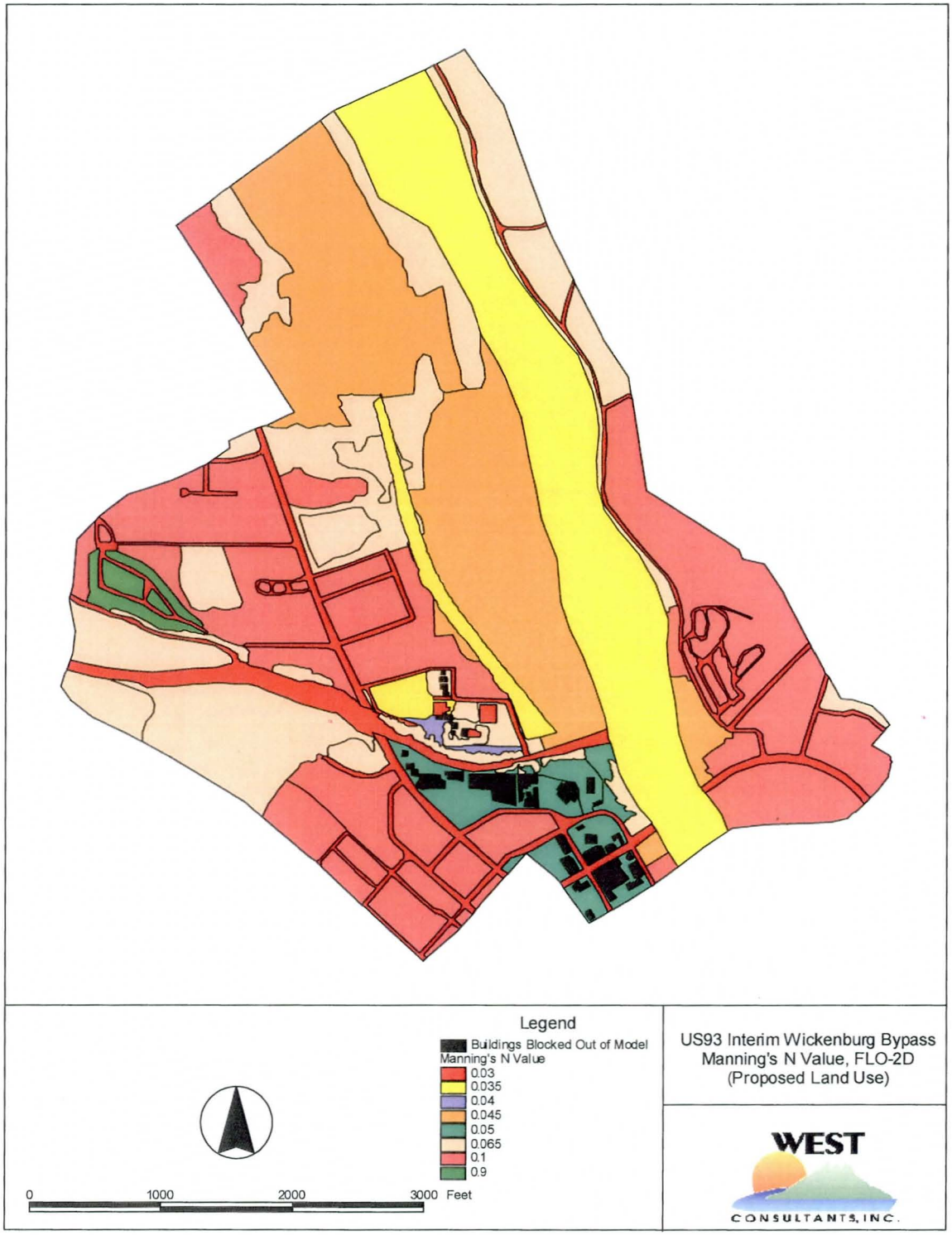


Figure B.2. Alternative proposed conditions Manning's n-values

B.2. Culverts in Alternative Proposed Conditions Model

In order reduce the impact of the alternative proposed embankment on flooding conditions in Sols Wash and the left overbank of Sols Wash, a series of culverts were designed to take water through the embankment. The culverts in the alternative proposed conditions model are shown in Table B-2.

Table B-2. Alternative Proposed Condition Culvert Layout

Station	Number of Barrels	Dimension Each Barrel	Inlet Elevation	Outlet Elevation	Length
118+00	3	6 feet high x 10 feet wide	2042.28	2042.03	124.38
120+00	2	5 feet high x 10 feet wide	2044.22	2043.99	114.42
122+00	2	5 feet high x 10 feet wide	2045.27	2045.05	110.22
123+50	2	5 feet high x 10 feet wide	2045.78	2045.56	109.89
125+00	3	5 feet high x 10 feet wide	2045.79	2045.57	112.97
126+50	2	5 feet high x 10 feet wide	2046.54	2046.32	114.05
128+00	1	5 feet high x 10 feet wide	2046.37	2046.12	122.27

With these culverts in place, there is some increase in the water surface elevation near the embankment, but for the majority of the floodplain, the alternative proposed conditions have no impact on the water surface elevations versus the existing conditions.

B.3. Model Boundary Conditions and Numerical Stability Criteria

FLO-2D requires inflow hydrographs and explicit list of outflow notes. Given this information, FLO-2D uses an explicit finite difference algorithm to route the flows through the computational domain and check for volume conservation at the end of every time step. For steady state flow conditions, this numerical procedure continues until convergence in depth and velocity is achieved.

FLO-2D is sensitive to the computational time step selected. The computational time step is directly proportional to the cell size. Therefore, for a cell size of 40 feet, which is considered fine, a computational time step of 0.01 second is practical. Numerical stability was assisted by several tolerance parameters. A depth tolerance of 5% (percent change in flow depth) was used to control the computational time step. If this tolerance is exceeded, FLO-2D reduces the

computational time step to maintain numerical stability. A minimum and a maximum value for the numerical stability coefficient for full dynamic wave routing were also selected to control volume conservation and reduce surging in the model. The selection of these values was based on the FLO-2D User's Manual.

B.4. FLO-2D Model Analyses

The existing condition water surface elevations matched closely with that in the HEC-RAS model and FEMA FIRM maps. The east boundary of the floodplain limit follows a natural bluff. Convergence was tested by comparing the difference between the maximum and the minimum flow depth in each cell for the last ten outputed time steps (of 0.1 hour each). This difference varied between 0 and 0.2 feet for 94% of the cells.

The proposed condition floodplain boundary to the east follows the proposed roadway embankment. To the south, the floodplain limit extends beyond the existing condition limit by approximately 950 feet given the proposed Sols Wash bridge alignment. To the north the floodplain limit extends approximately 770 feet beyond the existing condition limits along the proposed bypass embankment. Convergence test showed the difference between the maximum and the minimum flow depth in the cells for the last ten outputed time steps in the last hour of simulation to be in the range of 0 – 0.4 feet for the most part.

Velocity convergence was also tested. FLO-2D generates velocity vectors in eight directions: the four cardinal and the diagonal directions within each cell. An absolute velocity magnitude was obtained for all cells and the difference between the minimum and maximum over the last ten outputted time steps was obtained. This difference was found in the range of 0 – 0.5 fps for the most part under existing and the proposed conditions. Lack of velocity convergence was limited to few cells, and was considered to be numerical blips.

There was no notable difference in water surface elevation between the existing conditions and alternative proposed conditions for most of the Sols Wash floodplain, except near the proposed embankment. As expected, significant increases in elevation can be observed along the embankment.

APPENDIX C

Sediment Gradations

C. Sediment Gradations

The sediment gradations used in the HEC-6T models are documented here. There were two sets of samples used. The first set of samples was taken by Ninyo and Moore; there are eight of these samples and they are labeled HS1, HS3, HS4, HS5, HS6, HS7 and HS8. Two additional Ninyo and Moore samples were taken in Sols Wash, HS2 and HS3, but these were not utilized. In addition, WEST Consultants, Inc. took six samples, which are labeled WEST1 through WEST6. All samples were taken at or near the surface.

Table C-1. Sediment sample grid coordinates (Arizona State Plane Central, NAD1983, international feet)

Sample ID	Northing	Easting
WEST1	1091926	449455
WEST2	1092308	448614
WEST3	1092041	449400
WEST4	1102927	449639
WEST5	1093734	449313
WEST6	1088069	451561
HS1	1081862	454509
HS2	1082116	453297
HS3	1082140	452640
HS4	1082976	454220
HS5	1085134	453495
HS6	1087542	452266
HS7	1089044	450288
HS8	1090975	449694

Table C-2. Percent-finer-than gradations for the WEST samples

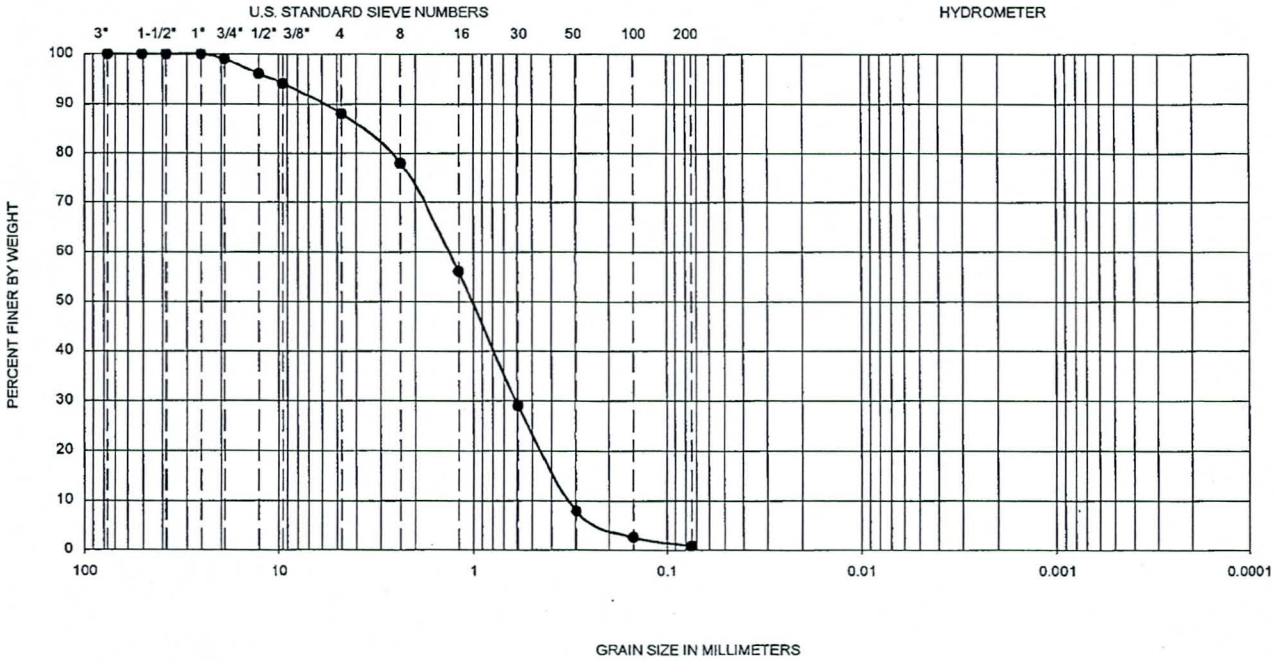
Sieve Number	Sieve Size in mm	WEST1	WEST2	WEST3	WEST4	WEST5	WEST6
3"	75	100	100	100	100	100	100
2"	50	100	100	100	100	100	100
1.5"	37.5	100	100	100	100	100	100
1"	25.4	100	100	100	100	100	100
3/4"	19	100	100	100	96	100	100
1/2"	12.7	98	100	98	90	99	99
3/8"	9.5	98	99	96	87	98	99
1/4"	6.35	95	96	94	84	95	98
No. 4	4.75	94	95	93	81	94	96
No. 8	2.36	87	87	82	73	87	88
No. 10	2	85	83	77	70	84	85
No. 16	1.18	75	67	61	55	67	68
No. 30	0.6	53	40	38	30	39	37
No. 40	0.425	39	27	27	20	27	23
No. 50	0.3	26	14	17	12	18	13
No. 100	0.15	9	4	7	4	6	5

A plot showing the locations of the sediment samples (except for WEST4 and WEST5, which are north of the limits of the figure) appears in Figure C-1.

The gradation charts as provided by Ninyo & Moore follow. The samples are labeled HS1 through HS8.

Figure C-1. Sediment sample locations

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	HS-1	0-3	--	--	--	0.32	0.60	1.40	4.4	0.8	1	SP

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PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-98

Ninyo & Moore

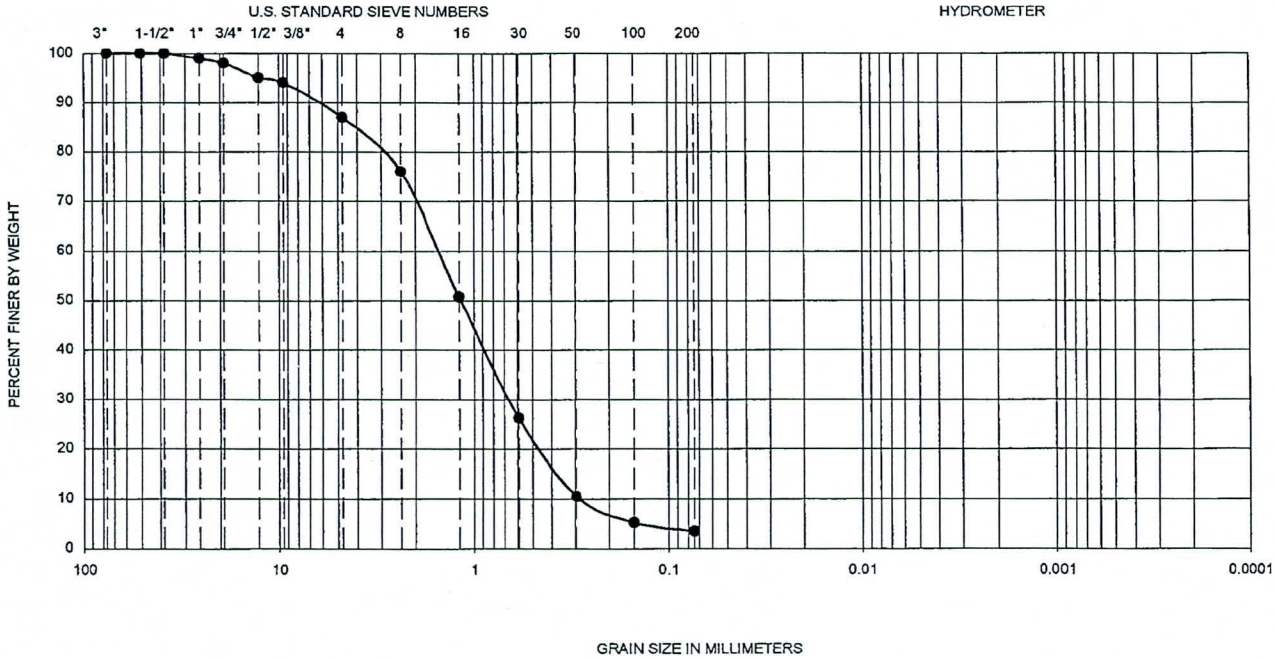
GRADATION TEST RESULTS

WICKENBURG INTERIM BYPASS (US-93)
WICKENBURG, ARIZONA

PROJECT NO.	DATE	FIGURE
600503001	05/04	B-X

SPLITSIEVE #1564 HS-1 0-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	HS-2	0-3	--	--	--	0.29	0.68	1.70	5.9	0.9	3	SP

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PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-98

Ninyo & Moore

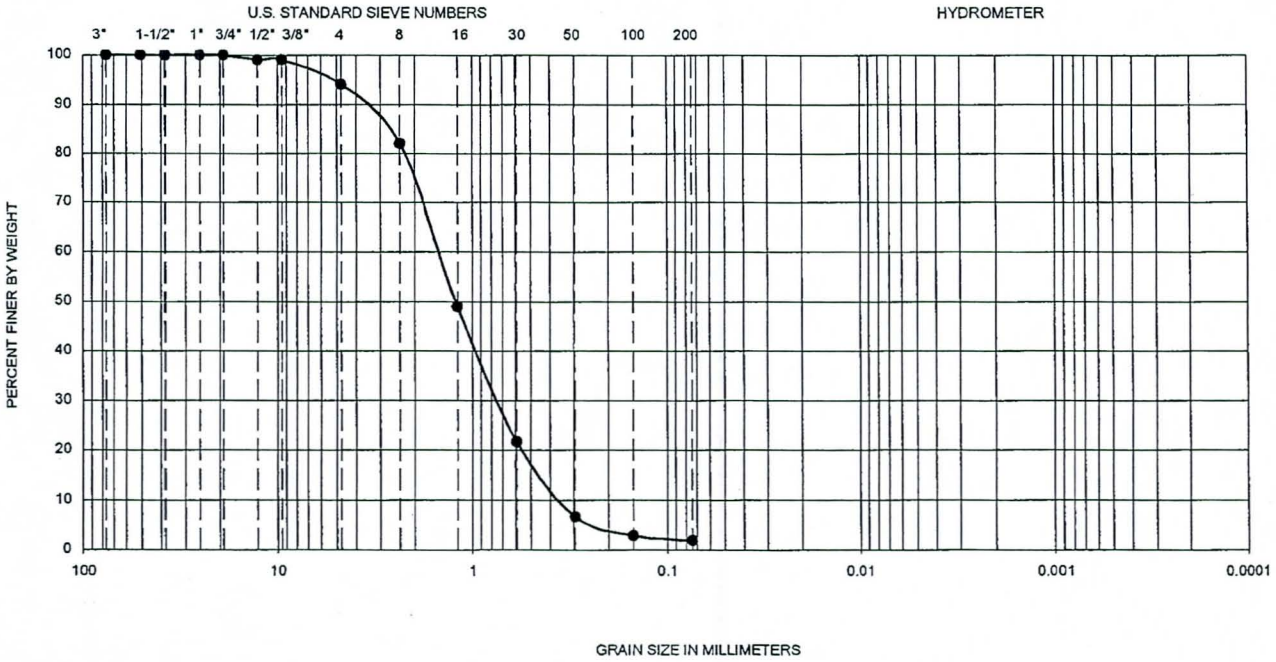
GRADATION TEST RESULTS

WICKENBURG INTERIM BYPASS (US-93)
WICKENBURG, ARIZONA

PROJECT NO.	DATE	FIGURE
600503001	05/04	B-X

SPLITSIEVE #1564 HS-2 0-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	HS-3	0-3	--	--	--	0.37	0.76	1.70	4.6	0.9	2	SP

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PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-98

Ninyo & Moore

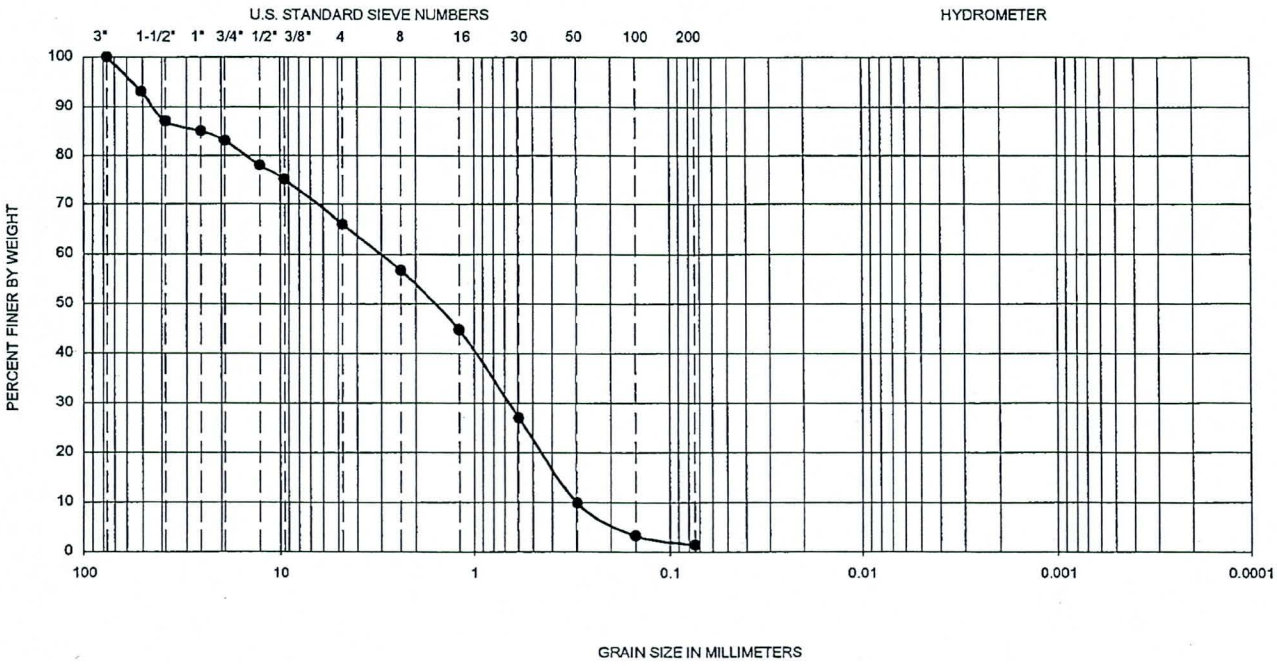
GRADATION TEST RESULTS

WICKENBURG INTERIM BYPASS (US-93)
WICKENBURG, ARIZONA

PROJECT NO.	DATE	FIGURE
600503001	05/04	B-X

SPLITSIEVE #1564 HS-3 0-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	HS-4	0-3	--	--	--	0.30	0.68	3.00	10.0	0.5	1	SP

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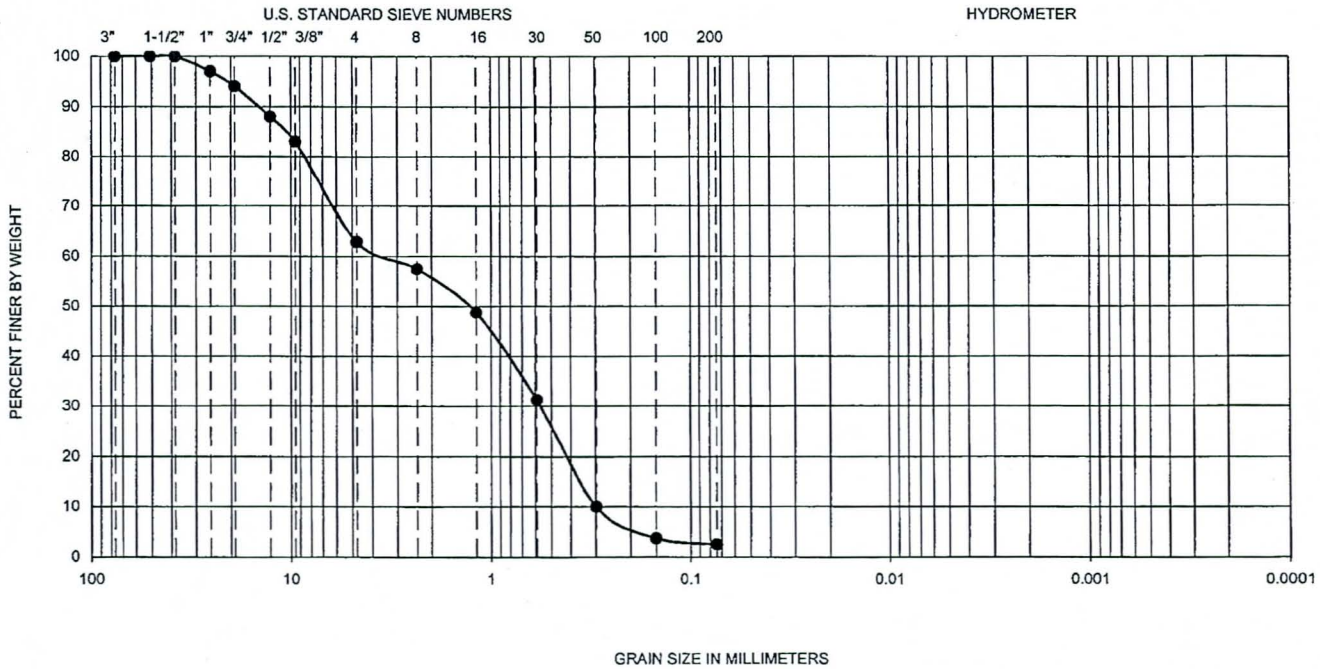
GRADATION TEST RESULTS

WICKENBURG INTERIM BYPASS (US-93)
WICKENBURG, ARIZONA

PROJECT NO.	DATE	FIGURE
600503001	05/04	B-X

SPLITSIEVE #1584 HS-4 0-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	HS-5	0-3	--	--	--	0.30	0.48	3.60	12.0	0.2	3	SP

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Ninyo & Moore

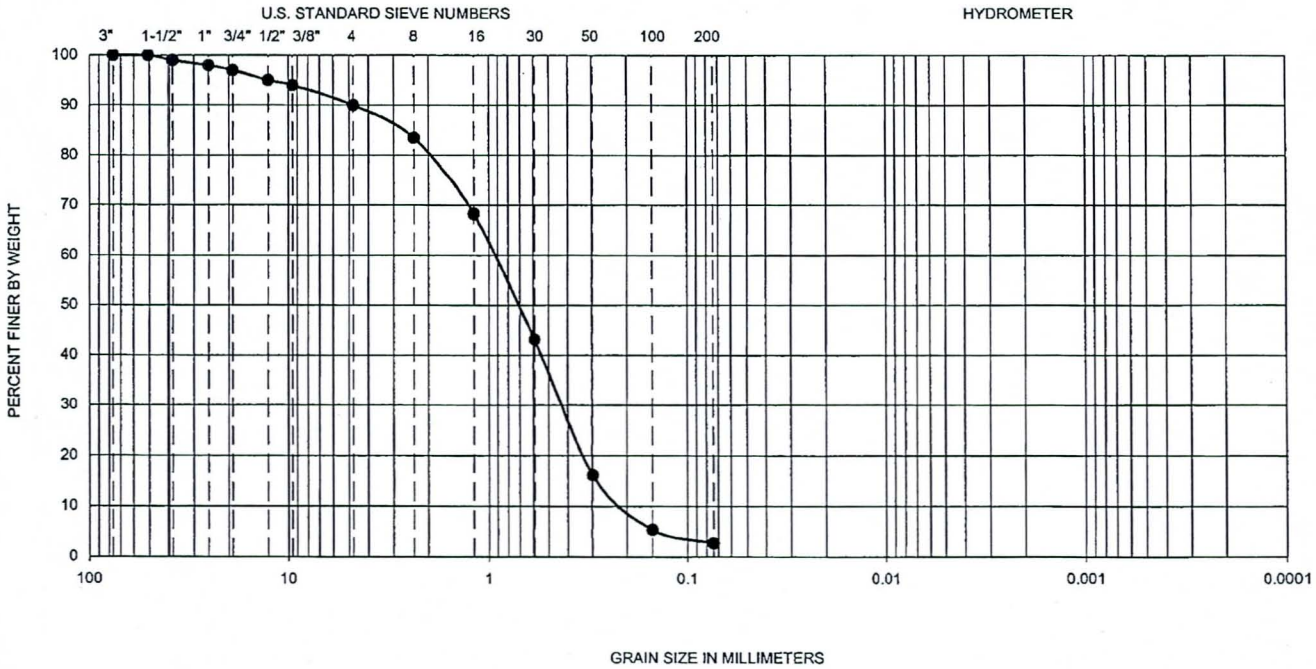
GRADATION TEST RESULTS

WICKENBURG INTERIM BYPASS (US-93)
WICKENBURG, ARIZONA

PROJECT NO.	DATE	FIGURE
600503001	06/04	B-X

SPLITSIEVE #1605 HS-5 0-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	HS-6	0-3	--	--	--	0.22	0.43	0.91	4.1	0.9	3	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-98

Ninyo & Moore

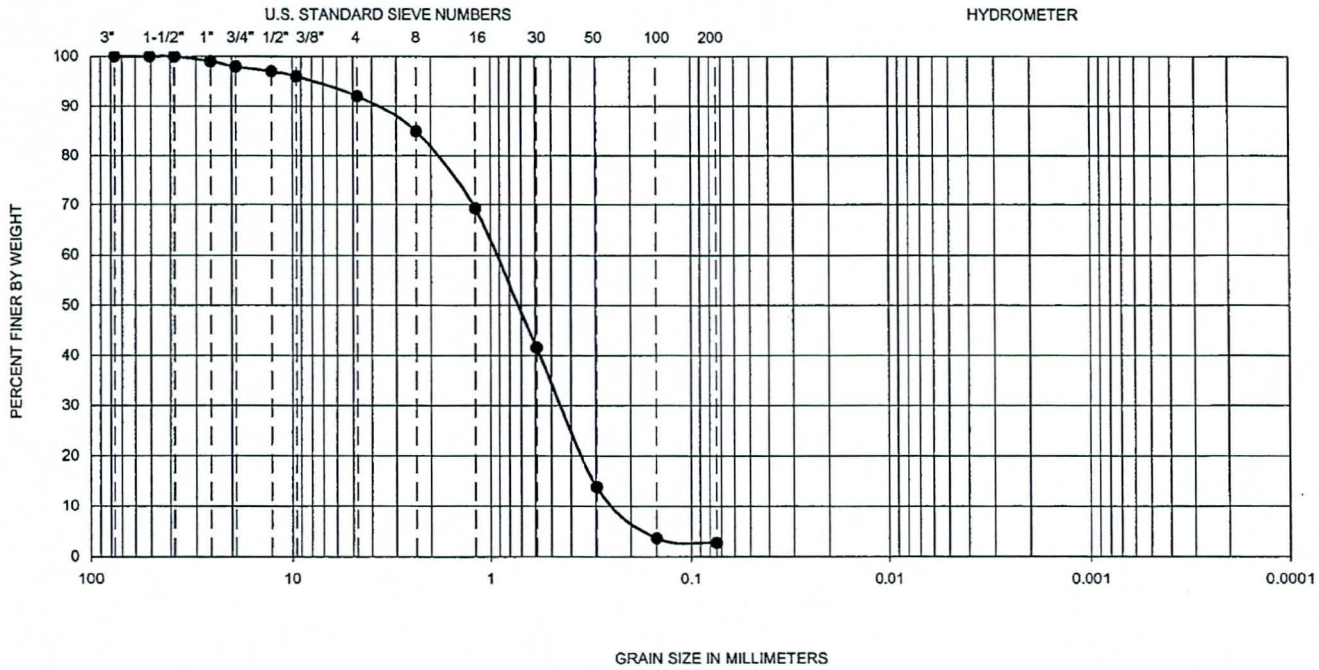
GRADATION TEST RESULTS

WICKENBURG INTERIM BYPASS (US-93)
WICKENBURG, ARIZONA

PROJECT NO.	DATE	FIGURE
600503001	06/04	B-X

SPLITSIEVE #1605 HS-6 0-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	HS-7	0-3	--	--	--	0.26	0.45	0.91	3.5	0.9	3	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-98

Ninyo & Moore

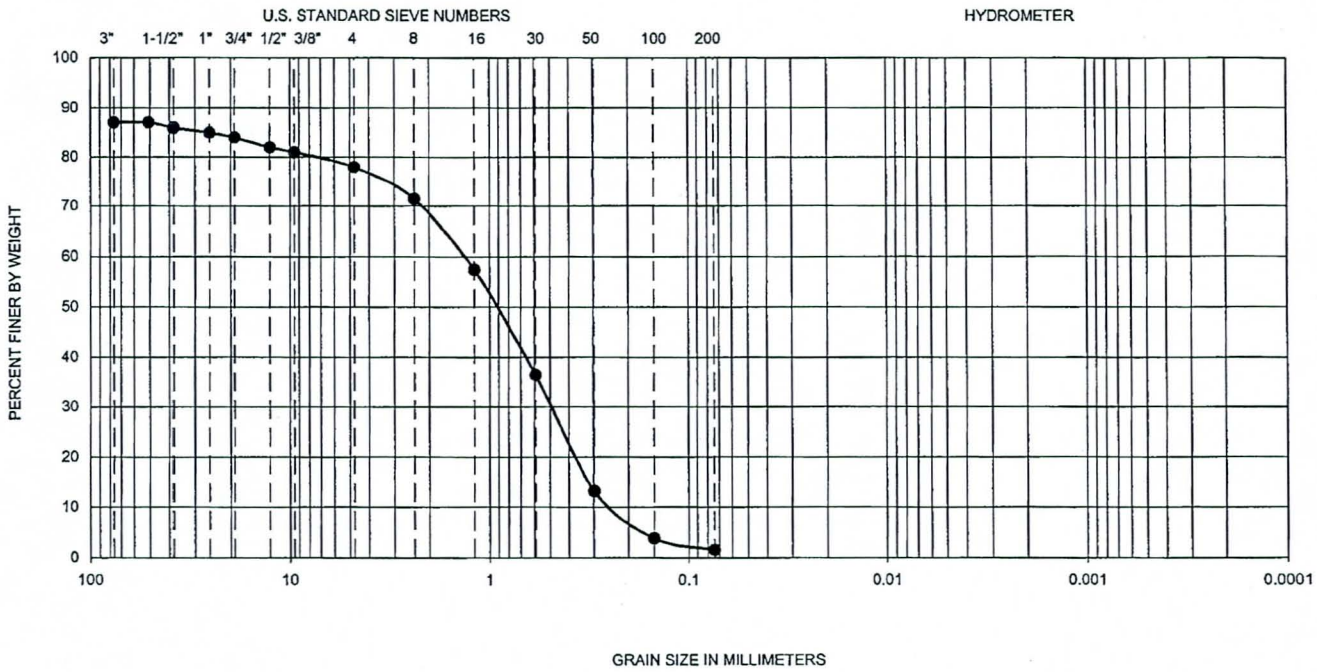
GRADATION TEST RESULTS

WICKENBURG INTERIM BYPASS (US-93)
WICKENBURG, ARIZONA

PROJECT NO.	DATE	FIGURE
600503001	06/04	B-X

SPLITSIEVE #1605 HS-7 0-3

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	U.S.C.S
●	HS-8	0-3	--	--	--	0.26	0.49	1.30	5.0	0.7	2	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-98

Ninyo & Moore

GRADATION TEST RESULTS

WICKENBURG INTERIM BYPASS (US-93)
WICKENBURG, ARIZONA

PROJECT NO.	DATE	FIGURE
600503001	06/04	B-X

SPLITSIEVE #1605 HS-8 0-3