# Low Flow Channel Design Analysis for Rio Salado (Salt River), Arizona

## FINAL REPORT

Prepared for

U.S. Army Corps of Engineers, Los Angeles District

WEST
CONSULTANTS, INC.

**W**ater · **E**nvironmental · **S**edimentation · **T**echnology

Property of
Flood Control District of MC Library
Please Return to
2801 W. Durango
Phoenix, AZ 85009

**Hydraulics · Hydrology · Sedimentation · Water Quality · Erosion Control · Environmental Services**

# FINAL REPORT

## Low Flow Channel Design Analysis for Rio Salado (Salt River), Arizona

### FINAL REPORT

Prepared for

U.S. Army Corps of Engineers, Los Angeles District

Prepared by

WEST CONSULTANTS, INC.

WEST Consultants, Inc.
4500 South Lakeshore Drive, Suite 210
Tempe, AZ 85282

Contract No. DACW09-97-D-0022
Delivery Order No. 0007

January 11, 2000

WEST
CONSULTANTS, INC.

**California**
11848 Bernardo Plaza Court,
Suite 140-B
San Diego, CA 92128-2417

619.487.9378
619.487.9448 Fax

**Washington**
12509 Bel-Red Road
Suite 100
Bellevue, WA 98005-2535

425-646-8806
425.646.0570 Fax

**Arizona**
4500 South Lakeshore Dr.
Suite 210
Tempe, Arizona 85282

602.345.2155
602.345.2156 Fax

**Hydraulics**

**Hydrology**

**Sedimentation**

**Water Quality**

**Erosion Control**

**Environmental Services**

January 11, 2000

Glenn M. Mashburn
Hydraulic Engineer – Group Leader
U.S. Army Corps of Engineers
Los Angeles District
Attn: CESPL-ED-HH
P.O. Box 532711
Los Angeles, CA 90053-2325

Subject: Contract No. DACW09-97-D-0022
Delivery Order No. 0007
Final Low Flow Channel Design Analysis
100% Report for Rio Salado, Salt River, Phoenix, Arizona

Dear Mr. Mashburn:

Enclosed are 8 copies of the Final Low Flow Channel Design Analysis (100 Percent) Report, Rio Salado, Salt River, Phoenix, Arizona, for your review and comment. The report presents results of the low flow channel design and the corresponding hydraulic and sediment transport analyses. Also included are construction cost estimates for the proposed grade control structures, guide dikes, and bank protection measures.

Please call me at (602) 345-2155 if you have any questions or need additional information regarding this report. It has been a pleasure working with you on this project.

Sincerely,

Dennis L Richards by SMF

Dennis L. Richards, P.E.
Vice President

Enclosures

cc: City of Phoenix (2 copies)
Ron Copeland, WES (1 copy)
Don Rerick, Flood Control District of Maricopa County (2 copies)
Jatin Desai, USACE (2 copies)
David Williams, WEST (1 copy)

# EXECUTIVE SUMMARY

## Low Flow Channel Design Analysis Final Report
## for
## Rio Salado, Salt River, Arizona

### EXISTING CONDITIONS ANALYSIS

- **Hydraulic Analysis.** In the project reach, the existing conditions HEC-RAS 100-year water surface elevations are generally lower than those in the proposed FEMA Flood Insurance Study (FIS) model prepared by Michael Baker Jr., Inc. The difference in the water surface elevations can be attributed to differences in the cross section geometry. The FIS model geometry is understood to be based upon surveys made in 1992 and 1993. In WEST's existing conditions model, the cross section geometry between 211.51 and 216.53 is based on one-foot contour interval mapping flown in November 1998 and completed for the Corps in February 1999. The inverts for the updated cross sections are generally lower than those in the Baker model by three to four feet.

- **Sediment Transport Analysis**. The combination Toffaleti, Meyer-Peter and Muller sediment transport method is used in the sediment transport model. The long-term sediment transport simulations (25 years) for the existing conditions indicate that the bed achieves a quasi-stable configuration with maximum scour and deposition results generally less than two feet. The upper reach is most likely in a quasi-equilibrium condition because of the use of an equilibrium inflowing sediment load and an armored condition initial bed material size distribution. The deposition in the downstream reach in the area of recent channelization and milder slope is supported by field observations at the grade control structure below the 19th Avenue Bridge, where both the upstream and downstream bed elevations are flush with the grade control sill. Sensitivity analyses show that the average bed elevations are slightly affected by the inflowing sediment load and the choice of sediment transport method. The sensitivity to the placement of the 25-, 50- and 100-year peak discharge events in the simulation period results in average bed elevation changes by one to two feet in both degradation and aggradation zones.

## LOW FLOW CHANNEL DESIGN & HYDRAULIC ANALYSIS

➢ **Channel Design**. The goal of this project was to design a low flow channel having "soft" sides and bottom that would convey the design discharge of 12,200 cubic feet per second. This channel would have a minimal footprint, thereby maximizing the area available for overbank park and recreation and habitat development, yet convey the low flow design discharge of 12,200 cubic feet per second without significant scour or deposition. The channel was initially designed using a stable-channel approach and later refined using a sediment transport model. The design slope of the proposed low flow channel is 0.0025 foot/foot. The bottom width of the proposed low flow channel ranges from 205 feet in the lower reach of the project (below Cross Section 215.65) to 160 feet in the upper reach. The low flow channel has 3H:1V vegetated side slope with soil cement only at one location requiring additional erosion protection. Grade control structures and guide dikes are included in the channel design.

➢ **Low Flow Channel Alignment**. The low flow channel alignment is established based on the following guidelines:

1) Avoid and protect major features identified by the Corps, the City of Phoenix and the Flood Control District of Maricopa County (i.e., APS towers and 36" water line near 16th Street Bridge):

2) Avoid locating the top of the low flow channel bank from being too close to the existing outer channel banks or levees at any given location.

3) Prevent locating bridge piers at or near the low flow channel side slopes.

4) Align the flow with the bridge piers.

5) Minimize the difference between the sinuosity of the low flow channel and the sinuosity of the existing channel.

➢ **Water Surface Comparisons**. The water surface elevations computed using HEC-RAS Version 2.2 for the low flow channel are equal to or lower than the WEST's existing conditions model for the 100-year peak discharge of 166,000 cubic feet per second. In this analysis, it is assumed that vegetation damage reduces the overbank Manning $n$ value from 0.085 to 0.04. A sensitivity analysis shows that the overbank $n$ values could be as high as 0.06 upstream of Cross Section 212.84 and still produce computed water surface elevations below those for WEST's existing conditions model and thereby the proposed FIS model.

- **Grade Control Structures**. Four grade control structures are proposed for the project reach. The structures would be located at Cross Sections 216.23 and 215.65 to limit the scour in the upstream reach, at Cross Section 214.65 to reduce scour at the 16th Street bridge and at the buried 36-inch water line located just upstream of the bridge, and at Central Avenue Bridge (Cross Section 213.26) to provide scour protection for the bridge. The grade control structures extend across the full width of the flood control channel. Recommended toe-down depths are 27 feet within the low flow channel and 16 feet in the overbank areas. Three alternatives for downstream scour protection at the grade control structures were initially considered. These alternatives were: 1) riprap protection; 2) soil cement aprons; and 3) cable-stayed block. The project design team agreed that cable-stayed block was not an acceptable alternative and that riprap protection was not economical. Therefore, the designs presented in this report are for downstream protection using soil cement stair-stepped aprons.

- **Public Safety**. The potential formation of hydraulic jumps and significant vertical hydraulic rollers for the proposed grade control design were investigated to judge the potential impact on public safety. If the channel downstream of the grade control structures is protected from developing scour holes, hydraulic jumps and significant hydraulic rollers should not occur when the flow rate is within the low flow channel capacity. When the flow rate exceeds the capacity of the low flow channel, hazardous conditions exist in the flood control channel in general.

- **Guide Dikes**. The low flow channel design includes 34 guide dikes at strategic locations within the overbank area. The guide dikes serve to maintain the alignment of the low flow channel, protect the main channel bank, and minimize formation of secondary channels in the overbank areas. Guide dikes are located at channel bends, areas where the low flow channel is within 50 feet of the existing outer bank, and bridges to help align the flow with the bridge opening. Three guide dike construction alternatives have been evaluated: 1) soil cement guide dikes; 2) slurry trench walls; and 3) gabions for the overbank portion of the guide dike with soil cement for the dike section located along the low flow channel. The selected alternative is a soil cement guide dike with gabions the final 50 feet. If the guide dikes, on an average annual basis with a 50-year life, can prevent 4.5 acres (approximately 2 percent of the total overbank area in the project reach) from having to be completely repaired, the benefit-cost ratio would be greater than one.

- **Bridge Scour**. Bridge scour analysis was conducted for the seven bridges within the project reach using the 100-year peak discharge of 166,000 cubic feet per second. Contraction scour and abutment scour were estimated to be negligible at all seven bridges. Local pier scour was computed using the Colorado State University (CSU) equation in HEC-RAS. Long-term degradation was determined from 100-year HEC-6T simulations using the Laursen-Copeland sediment transport method, which results in maximum scour. The scour elevations must be evaluated against the allowable scour elevations required for structural integrity of each bridge. Based on the results of the scour analysis, it appears that countermeasures will be required at some of the bridges.

- **Bank Protection**. The low flow channel design concept is for a channel with "soft" sides and bottom. However, bank protection is recommended for bends with a ratio of radius of curvature to channel width less than five. The channel bend located downstream of Central Avenue meets this criterion. Therefore, soil-cement bank protection is recommended along the north bank (outside of bend) of the low flow channel for a length of approximately 2,400 feet from Cross Section 213.24 downstream to Cross Section 212.84.

- **Effect of Dam Failure at Tempe Town Lake**. Given the long distance and length of time it will take to reach the project, the flood wave from a dam failure at Tempe Town Lake would be greatly attenuated. Since the flood wave attenuates much faster than that of a normal flood event, it would not have an effect any worse than a normal flood event of the same magnitude. It is estimated that the flood wave from a dam failure at Tempe Town Lake would have a peak discharge of 23,800 cubic feet per second at Central Avenue, which is approximately equivalent to a 6-year peak flood event. Because of the small magnitude of flow, the possible dam failure at Tempe Town Lake will have a negligible impact upon the current project.

## LOW FLOW CHANNEL SEDIMENT TRANSPORT ANALYSIS

- Twenty-two long-term sediment transport simulations were completed for the low flow channel analysis. The transport results were used to 1) evaluate grade control locations, 2) determine overexcavation depth, 3) determine annual maintenance requirements, and 4) estimate the impacts for the 25-, 50- and 100-year peak flood events.

- The sediment transport simulations confirmed the need for the four proposed grade control structures.

- Four overexcavation scenarios (1-foot, 2-foot, 3-foot and 4-foot overexcavation depths) were evaluated. It is recommended that the channel be excavated 2 feet below the design invert downstream of the proposed grade control structure at 214.65. This 2-foot overexcavation scenario results in the least amount of scour at 16th Street Bridge.

- The 25-, 50- and 100-year peak flood events increase scour depths upstream of the proposed grade control structure located at 214.65 by 1 to 3 feet. Deposition increases downstream of 214.65 and additional maintenance is required.

## Cost Estimates

- Preliminary cost estimates are prepared for the construction of the low flow channel. The estimate includes costs for soil cement bank protection, soil cement grade control structures, and soil cement and gabions for guide dikes.

- The unit prices used in the cost estimates are:

  Soil cement bank protection.......................................$35/cy
  Soil cement grade control ..........................................$45/cy
  Soil cement guide dike................................................$40/cy
  Gabions for guide dike...............................................$85/sf

- The cost estimate for soil cement grade control structures, soil cement and gabions for guide dikes, and soil cement bank protection (at one bend) totals **$5,687,810**.

- These cost estimates do not include costs for excavation (other than that incidental to construction of the listed feature), mitigation measures at bridges or utilities, mobilization, operation and general maintenance or any contingencies.

# TABLE OF CONTENTS

# LIST OF TABLES

# LIST OF FIGURES

# 1. INTRODUCTION

## 1.1. PURPOSE

The purpose of this project is to provide a preliminary design for a low flow channel in the Phoenix Reach of the Rio Salado (Salt River) project. This low flow channel will have a minimal footprint, thereby maximizing the area available for overbank park and recreation and habitat development, yet still convey a design discharge of 12,200 cfs without significant scour or deposition.

This report presents the results the hydraulic, sediment transport and scour analyses conducted for the design of the low flow channel. The study reach for the low flow channel design extends from the Interstate 10 (I-10) grade control structure downstream approximately five miles to the 19$^{th}$ Avenue grade control structure.

## 1.2. SCOPE

### 1.2.1. Services Completed

The services performed and documented within this report include:

- **Low Flow Channel Design.** A proposed low flow channel design, including channel geometry, alignment, and location of grade control structures, is presented. In addition, design criteria, preliminary design and economic justification for overbank guide dikes are included. Additional bank protection measures and other considerations for the low flow channel design are also described.

- **Hydraulic Analysis.** Hydraulic analyses of existing and low flow channel conditions are presented and the results are compared to the proposed FEMA Flood Insurance Study (FIS) hydraulic model results (prepared by Michael Baker Jr., Inc.). In addition, the potential effect of a dam failure at the Tempe Town Lake is addressed in a qualitative discussion.

- **Sediment Transport Analysis.** Sediment transport analyses of existing and low flow channel conditions on the project reach are presented. These include HEC-6T models for 25-year long-term simulations for both existing and low flow channel conditions, with additional models simulating the 25-, 50- and 100-year events before and after the 25-year long-term hydrograph. Additionally, there are sensitivity analyses (inflowing sediment load and sediment transport method) for both the existing and low flow channel conditions. A short analysis of gravel mining is also included.

- **Bridge Scour Analysis.** Bridge scour analyses for existing and low flow channel conditions for the five roadway and two conveyor bridge crossings in the project reach are presented. These analyses include both pier scour and long-term degradation components from the sediment transport analyses.

- **Cost Estimate.** A preliminary cost estimate for the proposed low flow channel design is presented.

#### 1.2.2. Cross Section Stationing

Cross section stationing used in the HEC-RAS and HEC-6T models corresponds to river miles as presented in the Feasibility Report (U.S. Army Corps of Engineers, 1998). This cross section stationing is also used in the HEC-RAS models provided by the U.S. Army Corps of Engineers (the Corps).

Project stationing was not available at the beginning of this study. The relationship between the cross section stationing and the project stationing can be determined by comparing the project plans created by the Corps to the maps included in this report.

#### 1.2.3. Construction Material

Soil cement was the primary construction material considered during the analysis and design of the low flow channel. The unit costs presented in this report were reviewed and accepted by the design review team, which included representatives from the Corps, the Flood Control District of Maricopa County, and the City of Phoenix.

Since the completion of this analysis, it has been decided that roller compacted concrete rather than soil cement should be used. The construction of roller compacted concrete structures is the same as for soil cement structures. Roller compacted concrete provides higher strengths than soil cement.

### 1.3. PREVIOUS REPORTS

Previous phases of this study investigated various alternative designs for the proposed low flow channel. These alternatives included low flow channel geometry for the design discharge of 12,200 cfs as well as for discharges of 9,000 cfs and 6,500 cfs. Various grade control alternatives (including drop structures with and without stilling basins and sloped rock stabilizers) were evaluated. The discussion of these alternatives can be found in the following reports:

WEST Consultants, Inc. (1999a) "Low Flow Channel Design Analysis (50 Percent) Progress Report for Rio Salado, Salt River, Arizona," April 5, 1999.

WEST Consultants, Inc. (1999b) "Draft Low Flow Channel Design Analysis Summary (75 Percent) Report for Rio Salado (Salt River) Arizona," May 10, 1999.

WEST Consultants, Inc. (1999c) "Draft Low Flow Channel Design Analysis Summary (90 Percent) Report for Rio Salado (Salt River) Arizona," June 21, 1999.

## 1.4. FIELD RECONNAISSANCE TRIPS

Two field reconnaissance trips were conducted. The first field trip took place on Friday, February 19, 1999. Those participating included staff from the City of Phoenix (City), the Flood Control District of Maricopa County (FCDMC), and WEST Consultants, Inc. (WEST). A second field trip occurred on Monday, March 1, 1999 with representatives from the U.S. Army Corps of Engineers (Corps), the City, the FCDMC, and WEST. Summaries of field notes, observations and action items are included in Appendix 1. Don Rerick of the FCDMC and Marc Schulte of WEST prepared the field trip summaries.

## 1.5. AUTHORIZATION AND ACKNOWLEDGEMENTS

This study was authorized under Contract DACW09-97-D-0022, Delivery Order No. 0007 for Hydraulic Engineering Services between the U.S. Army Corps of Engineers, Los Angeles District and WEST Consultants, Inc. (WEST).

Dr. David Williams was the principal in charge of this project for WEST. Mr. Dennis Richards was the project manager for this study. He was assisted by Dr. Selena Forman, Ms. Adrienne Tober and Messrs. Brian Doeing, Marc Schulte, Carlos Mendoza, Thomas Grace, Krishna Poudyal, and Ramesh Chintala. Ms. Mary Dahlke provided clerical support in assembling the report.

## 1.6. REFERENCES

U.S. Army Corps of Engineers, Los Angeles District, South Pacific Division (1998), *Rio Salado Salt River, Arizona Feasibility Report and Environmental Impact Statement,* (April 1998).

# 2. LOW FLOW CHANNEL DESIGN

## 2.1. Channel Design

The goal of this project was to design a low flow channel having "soft" sides and bottom that would convey the design discharge of 12,200 cubic feet per second. The term "soft" implies an earthen channel, possibly vegetated, in contrast to a "hard" channel constructed with concrete or soil cement. This channel would have a minimal footprint, thereby maximizing the area available for overbank park and recreation and habitat development, yet convey the low flow design discharge of 12,200 cubic feet per second without significant scour or deposition. The channel was initially designed using a stable-channel approach and later refined using a sediment transport model. Several methods were used to estimate an appropriate stable slope for the low flow channel. In addition, a low flow channel alignment was proposed and grade control structures were located to minimize scour.

### 2.1.1. Stable Channel Analysis

#### *2.1.1.1. Existing Conditions*

Existing channel slopes within the project reach range from 0.0012 to 0.0028 foot/foot, and the existing low flow channel appears to have a width-depth ratio between 20 and 25. However, these existing conditions might not be considered stable for several reasons. There has been considerable activity along the Salt River system in recent years and consequently, the channel has not established an equilibrium condition. Activity has included channelization and levee work, both upstream and downstream of the project reach, as well as sand and gravel mining throughout the system. In addition, the raising of Roosevelt Dam will reduce peak flows and flow duration and affect the stable or equilibrium slope.

Stable slope is inversely proportional to the channel-forming discharge, so if the channel-forming discharge (usually on the order of a 5-10 year event for ephemeral streams like the Salt River) is decreased, the equilibrium slope will tend to become steeper. On this basis, the current conditions slope would be lower than the ultimate stable slope.

#### *2.1.1.2. Corps EM 1110-2-1418*

The U.S. Army Corps of Engineers Engineering Manual No. 1110-2-1418 (Corps, 1994) offers a tentative design guide for erodible channels. Using nomographs from this manual, stable channel dimensions can be bracketed. The analysis for the Salt River assumed very coarse granular banks with a median grain size diameter of approximately 30 mm. The width-depth ratio for "bank full" design discharge and resulting channel geometry was approximately 18. The predicted "stable" channel slope was approximately 0.0010 foot/foot.

The Corps developed the curves in EM 1110-2-1418 assuming a low bed-material transport rate. The manual warns that if the bed-material transport rate is high, the nomographs will underestimate the stable slope and depth. This is especially true of sand-bed channels and ephemeral channels, where flash floods will carry a great deal of sediment. Since the Salt River is a flashy system, one would expect this to be an underestimate of the ultimate stable slope. This expectation is corroborated by the fact that the stable slope estimate from EM 1110-2-1418 is slightly lower than that found currently on the project reach.

### *2.1.1.3. AMAFCA*

The AMAFCA Sediment and Erosion Design Guide (Resource Consultants, 1994) offers a different estimate of stable channel slopes which does not involve sediment particle size distribution. For this method, the stable slope ($S_s$) is estimated as:

$$S_s = 18.28 n^2 F^{0.133} Fr^{2.133} Q^{-0.133}$$

where:

| | | |
|---|---|---|
| $n$ | = | Manning roughness coefficient |
| $F$ | = | width-depth ratio of water flowing full in arroyo |
| $Fr$ | = | maximum Froude number |
| $Q$ | = | bank-full or channel-forming discharge, in cfs |

The width (in feet) of the resulting channel can be estimated as:

$$W = 0.5 F^{0.60} Fr^{-0.40} Q^{0.40}$$

The width to depth ratio ($F$) is usually on the order of 40, but the results from the Corps method suggest that the ratio could be around 20, perhaps as low as 10 to 15. Using a width to depth ratio of 20, a channel-forming discharge of 12,200 cubic feet per second, a Froude number of 0.70 (estimated from HEC-RAS modeling of the system), and a Manning roughness coefficient $n$ of 0.030, the AMAFCA estimate of stable channel slope ($S_s$) is 0.0033 foot/foot. This is close to the overall slope ($S = 0.0027$ foot/foot) currently found on the study reach. The channel widths are smaller than the estimates obtained by the Corps method. Using a 5-year and a 10-year (post-Roosevelt modification) discharge in the AMAFCA equation, the resulting slopes ($S_s = 0.0031$ foot/foot and $S_s = 0.0027$ foot/foot, respectively) are still slightly steeper than the current over-all slope of the study reach.

### *2.1.1.4. Corps of Engineers Hydraulic Design Package for Channels (SAM)*

The Corps of Engineers Hydraulic Design Package for Channels, or SAM (Copeland et al., 1996), allows the user to calculate a "family" of stable channels based upon hydraulic and sediment data for the channel. The SAM Hydraulic design package utilizes an analytical procedure for calculating stable channel dimensions developed by Copeland. This procedure determines dependent design variables of width, slope and depth from the independent variables of discharge, sediment inflow, and bed material composition. The method uses the sediment transport and resistance equations developed by Brownlie. Williams (1995) reports that the Brownlie relations work well for low flow velocities and depths with medium sands, but not so well for larger streams with large flow velocities and depths with sediment sizes up to coarse sands.

To estimate a value for the concentration of bed-load sediment, the full gradation from the Corps' composite sample downstream of 19th Avenue ($d_{50}$ = 30 mm) was used with the Ackers-White bed transport function. The Ackers-White relation can theoretically be used for sediment ranging from 0.04 mm to 4.94 mm. Williams (1995) recommends that the range be limited to 0.125 mm to 0.50 mm. In spite of the above limitation, it was felt that this would be a useful tool to evaluate the reasonableness of the design.

The resulting family of channel geometries from SAM showed a "minimum stream power" solution at a bottom width ($b$) of 72 feet and a slope ($S$) of 0.0087 foot/foot, with a corresponding depth of 8.8 feet. The width to depth ratio of this minimum stream-power stable channel is nine. In the bank-full sense, 12,200 cubic feet per second is a channel-forming discharge. However, in a frequency sense, its recurrence interval is a little too low, only about four years. For flashy streams in the arid west like the Salt River, the channel-forming discharge is often on the order of less frequent, higher recurrence interval 5- to 10-year event, which is between 20,200 and 53,000 cubic feet per second on the Salt River (post-Roosevelt Dam modification hydrology). It is reasonable to assume that the true stable slope might be associated with a channel-forming discharge above 12,200 cubic feet per second. Therefore, the SAM results could be seen as the upper bound for the stable channel design, on the steep side of a good estimate.

### 2.1.2. Channel Geometry

Using the results of the stable channel analysis, a design slope of 0.0025 foot/foot was selected. This was a compromise between the many estimates of stable slope explained above. A slope of 0.0025 foot/foot resides on the upper range of the current slopes of the project reach, which are probably striving for a steeper slope due to changes in hydrologic regime. The selected slope is also above the stable slope predicted by EM 1110-2-1418, which is likely underestimating the stable slope of a flashy, sediment-laden system like the Salt River. The selected slope is also much flatter than those suggested by the SAM package, which was considered an upper bound for the stable channel design. The selected slope is much closer to that predicted by the AMAFCA methodology, which was designed for use in the arid west. Additionally, the AMAFCA estimate is based on the existing width to depth ratios of the system.

There is a transition to the low flow channel from the existing grade control sill at Cross Section 216.40 to a grade control sill at the upstream end of the low flow channel

(Cross Section 216.23, elevation 1066.85). At Cross Section 216.23, the channel bottom is 165 feet wide, with a sideslope cotangent of three ($z = 3$) and a depth of 10 feet. There is a grade control sill with a three-foot drop at Cross Section 215.65 (crest elevation 1059.12) and another grade control sill with a three-foot drop at Cross Section 214.65 (crest elevation 1042.72). Through this reach, the channel has a sideslope cotangent of three and a depth of at least eight feet. Immediately downstream of the drop at Cross Section 215.65, the channel bottom is 205 feet wide, with a sideslope cotangent of three ($z = 3$). At Cross Section 214.23, the low flow channel changes to a slope of 0.00125 foot/foot to complete the transition to the existing channel. The low flow channel starts to daylight at Cross Section 212.84, where it slopes at a rate of 0.000251 foot/foot and meets the existing invert near 19th Avenue, at Cross Section 211.63. In addition, there is some planned over-excavation between Cross Sections 214.65 and 213.03, which is discussed more fully in the sediment transport section of this report (Section 4).

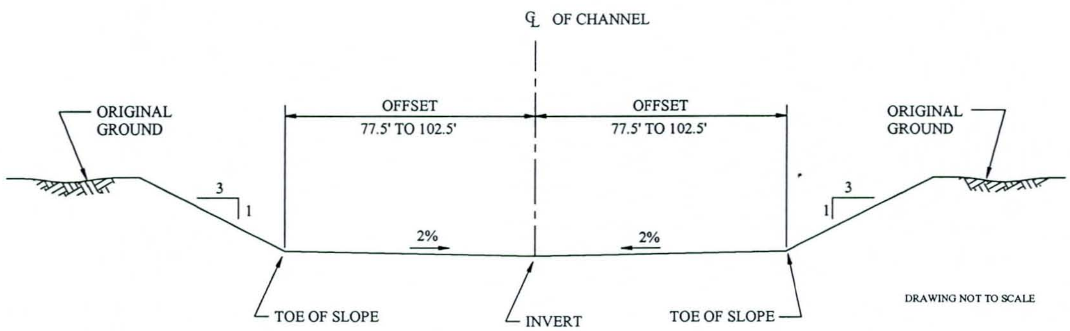



Figure 2-1. Typical cross section of the low flow channel.

## 2.1.3. Channel Alignment

The alignment for the low flow channel was developed by locating the existing channel thalweg from one-foot contour interval mapping. The alignment was adjusted to avoid the top of bank from being too close to the Salt River outer banks or levees at any given location, avoid Arizona Public Service (APS) transmission towers, and minimize the number of bridge piers impacted by the low flow channel. At bridges, the low flow channel location and alignment considered the number of bridge piers within the low flow channel as well as aligning flow with the bridge piers.

The alignment was also adjusted to avoid major features that were identified by the Corps, the City of Phoenix (City), and the Flood Control District of Maricopa County (District), and to be as consistent as possible with the original design concept by the City.

#### *2.1.3.1. Sinuosity*

Channel sinuosity was compared for existing conditions and for the low flow channel design between Cross Sections 216.23 and 211.63. It was computed as the channel length divided by the air distance. The channel length was taken as the sum of the main channel reach lengths in the base condition HEC-RAS model. The low flow channel length was taken as the sum of the channel reach lengths in the proposed low flow channel HEC-RAS model. The air distance was measured as a straight line between the low flow channel centerlines at Cross Sections 216.23 and 211.63 from the AutoCAD section layout file. Total sinuosity is 1.06 for the existing (base) conditions, and is 1.07 for the low flow channel design.

#### *2.1.3.2. Minimum Radius of Curvature of Low Flow Channel*

The most severe bend of the low flow channel in the project reach occurs between Central Avenue and 7$^{th}$ Avenue. The ratio of the radius of curvature to top width at this location is approximately five. According to criteria in the Corps' EM 1110-2-1601 (Corps, 1991), the radius of curvature to top width should be equal to or greater than three to minimize helicoidal flow for tranquil flow. For the final design, the wall height on the outside of the curve should be increased by an amount determined from Equation 2-31 in EM 1110-2-1601.

#### *2.1.3.3. Summary*

The proposed geometry and alignment of the low flow channel is a compromise between stable channel design, constraints imposed by "ground facts" and considerations of non-hydraulic goals of the project. The channel maintains the equilibrium shape (with a width-depth ratio between 20-25) of the existing low flow channel on the Salt River. The channel shape also correlates closely to the shape predicted by EM 1110-2-1418, which suggested that the stable channel would have a width-depth ratio of approximately 18. Channel slopes stay close to stable channel slope estimates, but still daylight out to match channel inverts downstream of the project reach. Channel widths were chosen to ensure that the low flow channel conveyed the design discharge of 12,200 cubic feet per second. At the same time, the channel footprint was minimized, therefore maximizing the area available for habitat and recreational development. The channel alignment is not too close to the Salt River outer banks or levees at any given location, avoids Arizona Public Service (APS) transmission towers, and minimizes the number of bridge piers impacted by the low flow channel. The channel sinuosity matches the current channel sinuosity well.

### 2.2. GRADE CONTROL SILL AND DROP-STRUCTURES

Four grade control structures are proposed for the project reach. The first grade control is at grade and is located at Cross Section 216.23. It does not require a drop below the sill. The next two grade controls are located at Cross Sections 215.65 and 214.65, respectively. These grade controls both involve a 3-foot drop below their sills.

A fourth grade control is located immediately downstream of Central Avenue, at approximately Cross Section 213.26. This grade control is at grade and does not have a drop below its sill.

While the low flow channel is designed to convey 12,200 cubic feet per second (approximately a 4-year peak event), the grade control sills are designed for a 100-year peak event. In addition, protection of the low flow channel is recommended near the grade control structures. Three alternatives for downstream scour protection at the grade control structures were initially considered. These alternatives were: 1) riprap protection; 2) soil cement aprons; and 3) cable-stayed block. The project design team agreed that cable-stayed block was not an acceptable alternative and that riprap protection was not economical. Therefore, the designs presented in this report are for downstream protection using soil cement aprons. The results of the riprap evaluation are included in Appendix 2.

Two criteria form the basis for the designs presented here. The first criterion is structural integrity. The grade controls are designed so that the toe of the structure is at or below the scour depth predicted for the 100-year peak event. The second criterion is public safety. Although the grade control structures are almost entirely below the grade of the low flow channel, they may be exposed during and after larger flood events. The exposed grade control structures might then present a hazard resulting from hydraulic rollers. As a "rule of thumb," which is corroborated by the work of Carreaga and Deschamps (1999), a 3-foot drop was found to be the maximum drop allowable to avoid the development of hydraulic rollers.

### 2.2.1. Toe-Down Depths

Two equations were used to estimate the local scour depth immediately downstream of the proposed grade control structures. The first was the Veronese equation, as presented by Pemberton and Lara (1984):

$$D_s = 1.32 \Delta H^{0.225} q^{0.54} - y_2$$

The second equation was developed by Simons, Li and Associates, (1985):

$$D_s = 0.54 q^{0.67} \left[ \frac{h}{y_2} \right]^{0.158} \left[ 1 - \left( \frac{h}{y_2} \right) \right]^{-0.134}$$

In these equations,

| | | |
|---|---|---|
| $D_s$ | = | depth of scour (ft) |
| $q$ | = | unit discharge (cfs/ft) |
| $y_2$ | = | downstream (tailwater) depth (ft) |

$\Delta H$ = difference in head from upstream reservoir to tailwater (ft)

$h$ = drop height (ft)

The Simons, Li and Associates method has been used in the design of other structures on the Salt River (Richards and Morrison, 1995).

HEC-RAS output for the 100-year event ($Q$ = 166,000 cfs) was used to estimate flow conditions both in the low flow channel and in the overbank areas. For flows contained within the low flow channel, a drop height of 3 feet was assumed. This assumption is a worst-case scenario without riprap protection to protect against scour downstream of the sill. For overbank areas, a drop of 3 feet was also assumed. This is also a worst-case scenario, since the grade control sill should be approximately at-grade in the overbank areas. Additionally, the Manning roughness coefficient ($n$) on the channel sideslopes and overbank areas was reduced to 0.040 to obtain maximum unit discharge when computing scour in the overbank areas. In general, the Simons, Li and Associates method yielded the most conservative estimate of scour depths. The maximum estimate of scour depth was multiplied by a factor of safety of 1.3 to obtain the ultimate toe-down depth. The recommended toe-down depths are 27 feet within the low flow channel and 16 feet in the overbank areas.

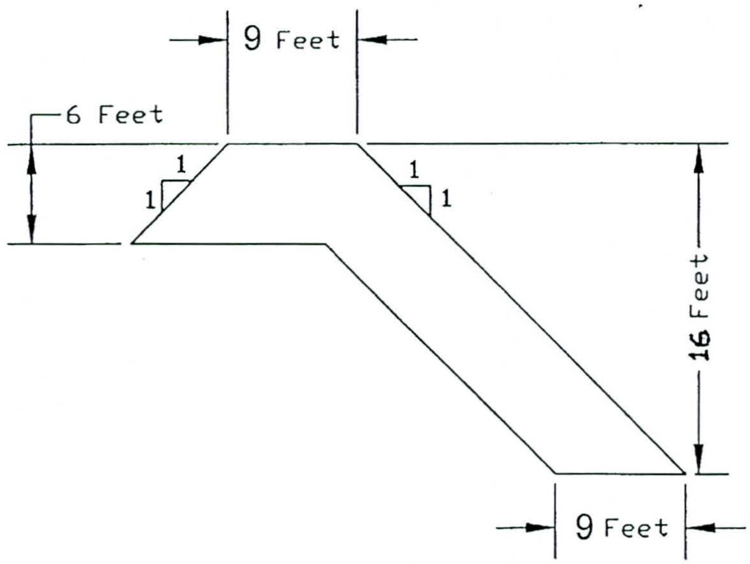



Figure 2-2. Typical cross section of soil cement grade control sill in overbank area, with a downstream toe-down depth of 16 feet. Within low flow channel, the toe-down depth should be 27 feet.

### 2.2.2. Safety

Outside of hydraulic and sediment transport considerations, one of the primary concerns in the design of a grade control structure is safety. Grade control structures with a significant drop in invert elevation can form a hydraulic jump and possibly a dangerous hydraulic roller. Hydraulic rollers are caused by flow re-circulation around a hydraulic jump. These rollers can be particularly menacing if the hydraulic jump is submerged, lulling the public into thinking the waters are safe with quiescent surface conditions.

However, conditions at the proposed grade control structures might not be conducive to the formation of hydraulic rollers. According to Wright et al. (1995), stilling basins handling Froude numbers over 1.7 are prone to the hydraulic roller phenomenon. HEC-RAS modeling of the system does not show that the flow is supercritical (Froude numbers are less than one). Therefore, it appears that a three-foot drop may not be enough to create a hydraulic jump, much less one powerful enough to create a substantial roller.

To check this assumption, the performance of the grade control structures was evaluated for the presence of reverse hydraulic rollers following a procedure proposed by Carriaga and Deschamps (1999). In this methodology, the ratio of the hydraulic drop and specific head is plotted against the ratio of total head and specific head. The hydraulic drop is defined as the difference between the headwater and tailwater elevations. The total head is the sum of the tailwater depth of flow and the hydraulic drop, and the specific head is the specific energy at the headwater section. A plot of these points on a flow classification chart by the U.S. Bureau of Reclamation (USBR, 1977) can be helpful in evaluating the safety of the grade control structure.

The hydraulic drop-specific head and total head-specific head ratios were calculated using output from HEC-RAS for a range of flow conditions. Plotting the ratios on the USBR flow classification chart (Figure 2-3) showed that generally, downstream depths were insufficient to form a hydraulic jump. It is possible that the grade control structure at Cross Section 214.65 will develop dangerous hydraulic conditions briefly between the 5-year and 10-year events (20,200 cfs and 53,000 cfs, respectively). However, these flows exceed the capacity of the low flow channel and flood events of this magnitude create a hazardous condition in general within the flood control channel.

The possibility of hydraulic rollers in the overbank areas was not examined for the reason that the grade control structure is flush with or below the ground surface in these areas. If scour were to occur, it would occur during events exceeding the low flow channel capacity. In the same way, hydraulic rollers could not develop until after the low flow channel capacity had been exceeded. At such large flows, general conditions are hazardous within the flood control channel. Additionally, as flows become larger, small differences in invert elevation have less likelihood of causing a hydraulic jump. Any scour damage in both the low flow channel and overbank should be repaired promptly to ensure the grade control structures perform as designed.

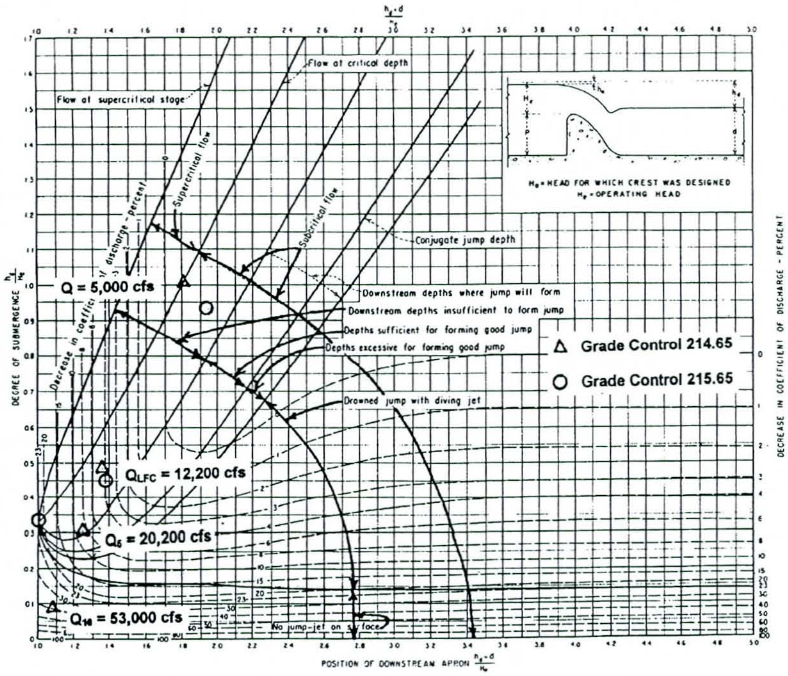



Figure 2-3. Plot of hydraulic drop-specific head ratio versus total head-specific head ratio used for evaluating grade control structure hydraulics (flow classification chart from USBR, 1977).

Four alternatives for the downstream apron were developed. Table 2-1 summarizes these alternatives, and Table 2-2 gives more detail on their dimensions and method of scour calculation. The first two alternatives specify that the soil cement apron be stepped down to the 100-year scour depth. The steps extend to the distance of maximum scour (six times the scour depth). The remaining two alternatives specify that the soil cement apron be stepped down to the 10-year scour depth, and then slope down to the 100-year scour depth at a slope of 1V:1H. For both these alternatives, this 100-year scour depth was taken as the average scour depth multiplied by the 1.30 factor of safety.

Alternative D (steps to the 10-year maximum scour depth and a 1V:1H slope to the 100-year average scour depth multiplied by a 1.3 safety factor) was chosen as the preferred alternative. The primary criterion for selection was that Alternative D accomplished adequate scour protection within the shortest length. Figure 2-4 shows typical plan view of the soil cement apron for the Alternative D grade control design, and Figure 2-5 shows a cross section along the low flow channel centerline.

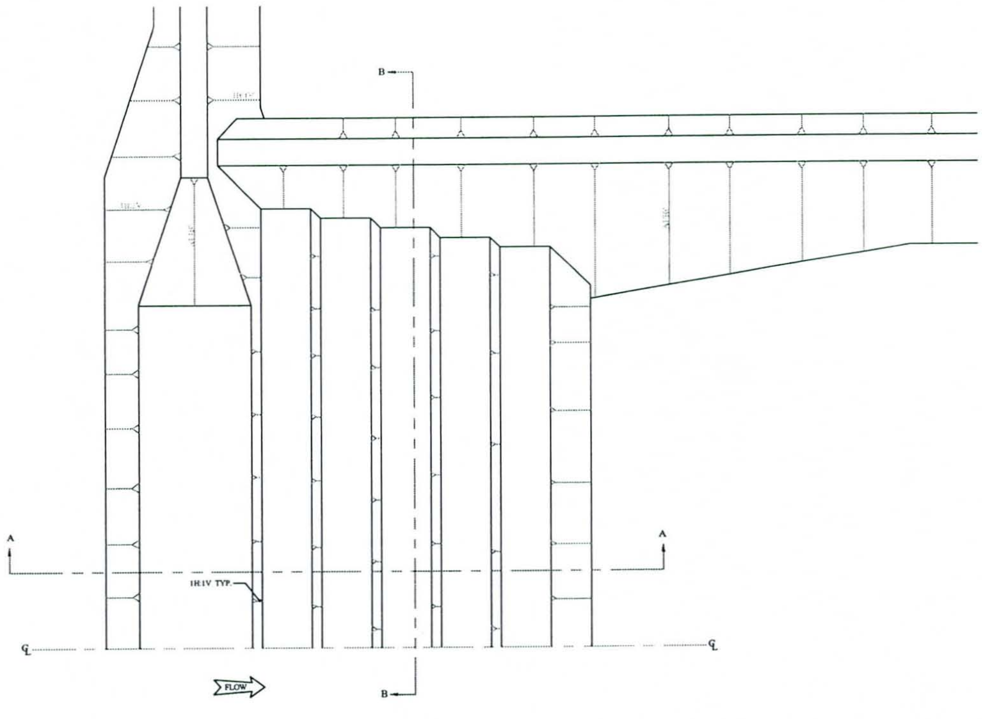
Table 2-1. Alternative grade control designs.

| Alternative | Description |
|---|---|
| A | Steps to 100-year average scour depth x 1.30 |
| B | Steps to 100-year maximum scour depth |
| C | Steps to 10-year average scour depth x 1.30, 1V:1H slope to 100-year average scour depth x 1.30 |
| D | Steps to 10-year maximum scour depth, 1V:1H slope to 100-year average scour depth x 1.30 |

Table 2-2. Alternative grade control designs, dimensions and design criteria.

| Alternative | Total Length (ft)* | Number of Steps* | 10-year Scour Depth | | | 100-year Scour Depth | | |
|---|---|---|---|---|---|---|---|---|
| | | | Depth (ft) | max or avg | SF | Depth (ft) | max or avg | SF |
| A | 165 | 9 | n/a | n/a | n/a | 27 | avg | 1.30 |
| B | 147 | 8 | n/a | n/a | n/a | 24 | max | 1.00 |
| C | 135 | 7 | 18 | avg | 1.30 | 27 | avg | 1.30 |
| D | 120 | 6 | 15 | max | 1.00 | 27 | avg | 1.30 |

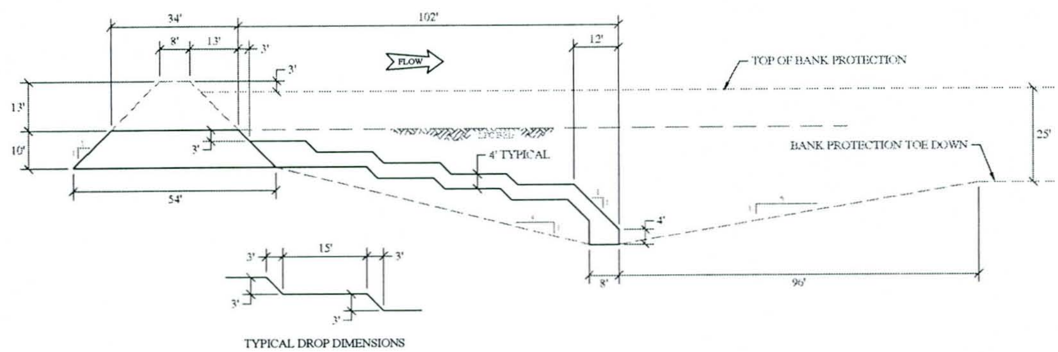
* Numbers in table are for grade controls with 3-foot drop (Cross Sections 215.66 and 214.65). Grade controls at Cross Sections 216.23 and 213.26 (without a drop) require one less step and will be 18 feet shorter.





PLAN VIEW - GRADE CONTROL W/ 3' DROP

Figure 2-4. Typical plan view of grade control with 3 foot drop. Centerline of channel is at bottom of figure.





SECTION AA - GRADE CONTROL AT WEIR

Figure 2-5. Cross section along low flow channel centerline of grade control structure with 3 foot drop. Soil cement apron is stepped to 10-year scour depth and then sloped at 1H:1V to the 100-year scour depth.

## 2.3. Guide Dikes

Guide dikes will be located at strategic locations between the grade control structures. These multi-purpose guide dikes will help to maintain the alignment of the low flow channel, protect the main channel bank (outer bank) from erosion, and reduce damage in the overbank areas. The dikes will prevent the development of secondary low flow channels in the overbanks. During the period of receding flood flows, the guide dikes will direct flow toward the low flow channel, which will help preserve the location of the original meander geometry and location of the low flow channel.

### 2.3.1. Design

Figure 2-6 illustrates the plan view of a guide dike. The guide dikes were initially designed to extend from the outer bank to the low flow channel top-of-bank. The section of the dike located adjacent to the low flow channel directs overbank flow toward the low flow channel during the lower flow events. The outer section of the dike directs overbank flow during larger flow events toward the low flow channel. This outer section also directs the flow away from the outer banks, thereby reducing bank erosion.

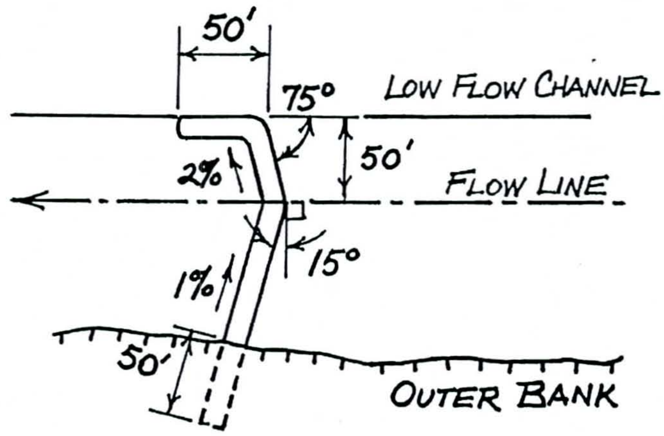


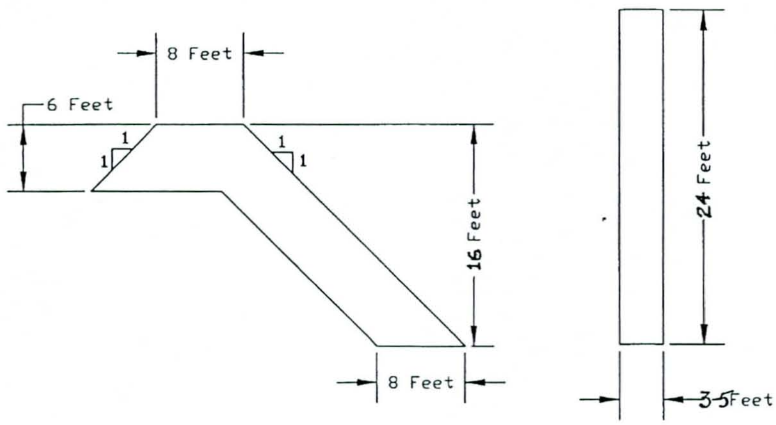

Figure 2-6. Typical plan view of originally proposed guide dike installation.

At the outer bank, the dike will be oriented in an upstream direction to an inflection point located approximately 50 feet from the low flow channel bank. This section of the dike will slope toward the low flow channel on a 1 percent slope. The angle formed by a line perpendicular to the flow line and the upstream side of dike is approximately 15 degrees. At the inflection point, the dike is oriented in a downstream direction to the low flow channel. This section of the dike will slope toward the low flow channel on a 2 percent slope. If the outer bank is within 50 feet of the low flow channel, the guide dike bank will be oriented perpendicular to the outer bank.

As illustrated in Figure 2-6, the initial design recommended the guide dikes abut the low flow channel bank a longitudinal distance of 50 feet in the downstream direction, and the dike extend into the Salt River's outer bank a distance of 50 feet to prevent flanking during larger flow events. These recommendations were based on guidelines included in notes from a Corps Streambank Protection Course (Corps WES, 1992).

Based on the scour analysis, the guide dikes should remain stable for scour depths of 16 feet (on average) below the existing overbank elevation. Three types of construction were investigated for the overbank portion of the guide dikes: 1) soil cement; 2) slurry trench walls; and 3) gabions. Soil cement is recommended for the 50 foot section adjacent to the low flow channel. Gabions are recommended for the area between the toe of the outer bank slope and the end of the guide dike.

For the soil cement guide dike, the recommended top width is 9 feet with side slopes of 1H:1V (see Figure 2-7a). The soil cement dike should extend to a depth of 16 feet below the existing overbank elevation on the downstream side of the dike and to a depth of 5 to 6 feet on the upstream side of the dike. The top of the guide dike should be constructed flush with or just below the overbank grade.




(a) (b)

Figure 2-7. Typical guide dike cross sections for (a) soil cement construction and (b) slurry-wall construction.

For the slurry-trench wall guide dike, the trench walls are vertical with a 3 to 5 foot width (see Figure 2-7b). Since the wall must remain stable with as much as 16 feet of scour on the downstream side, the depth of the wall would need to extend some distance below the scour depth. During excavation, the vertical walls would need to be supported by keeping the trench filled with bentonite slurry. The slurry trench method of excavation can be used to construct walls of soil-bentonite, soil-cement-bentonite, and cement-bentonite. This method of construction would minimize the excavation "footprint" on the terrace area of the Salt River channel when compared to the excavation "footprint" necessary for soil cement construction. However, there are environmental issues associated with construction and the project review team agreed that slurry walls did not appear appropriate for this project.

Due to environmental and constructability issues, the project review team, which was composed of representatives from the Corps, the Flood Control District of Maricopa

County, and the City of Phoenix, agreed to modify the end conditions for the guide dike structures. The 50 feet of embedment into the outer bank was eliminated to minimize impacts to the bank, reduce extensive excavation requirements, minimize the likelihood of encountering buried landfill and regulated materials (see Figure 2-9 and Figure 2-10). It was agreed to move the end of the grade control structure away from the toe of the slope of the outer bank by a distance of 25 to 50 feet. It was also agreed that the 50-foot length of grade control structure along the low flow channel could be eliminated (see Figure 2-8).

Guide dike locations are shown on the plan sheets included in the appendices of this report. The guide dikes are located at channel bends, areas where the low flow channel is within 50 feet of the outer bank, and near bridges to help align the flow with the bridge opening.

### 2.3.2. Economic Justification

The purpose of the guide dikes is to inhibit the lateral movement of the low flow channel. Since the banks of the low flow channel are not constructed of "permanent" material such as soil cement, there will always be slight channel movement even though the low flow channel alignment has a sinuosity, meander belt widths and amplitudes almost equal to the natural condition. Because of this movement, which can occur even during flow events that do not overtop the low flow channel, damage to the channel side slopes and overbank areas would occur. However, there are an infinite number of damaging channel flow paths that can produce the same sinuosity, meander belt widths and amplitudes as the natural condition, especially for those flood events that have significant flow in the overbank areas. The guide dikes were placed at locations designed to fix these paths to the low flow channel alignment. They would also function to funnel the overbank flows to the low flow channel alignment as the flood recedes, thus preventing the formation of large secondary channels. Although some damage would still occur in the overbank areas, major damage would be minimized.

The determination of the number of acres that would be damaged for various flood events, with and without the guide dikes, is not possible. However, a sense of the potential overall benefit of the guide dikes can be obtained from the following analysis.

A traditional method for economic justification of flood control measures is to determine the annualized cost of the measures and the annualized benefit of those measures. The benefit is then compared to the cost to determine if the measures are economically feasible. The annualized cost of the guide dike can be obtained by determining the economic recovery factor. Based upon an interest rate (often called the discount rate) of 6.625 percent (Federally authorized for economic analyses) and an economic life of 50 years, the capital recovery factor is 0.069044. Applying this to an estimated present cost of $2.5 million for the guide dikes, this results in an annualized cost of $172,600. The estimated cost of repairing an acre of overbank area is $38,430 (see City of Phoenix, May 14, 1999). If it can assumed that the present day repair costs increases in proportion to the interest rate, a direct comparison between the annualized guide dikes cost and the annual cost to repair the overbank areas can be made. Dividing the annualized guide dike cost by the repair cost per acre results in approximately 4.5 acres, which is about 2 percent of the total overbank area within the project reach. If the guide dikes, on an average annual basis, can prevent 4.5 acres from having to be completely repaired, the benefit-cost ratio would be greater than 1.

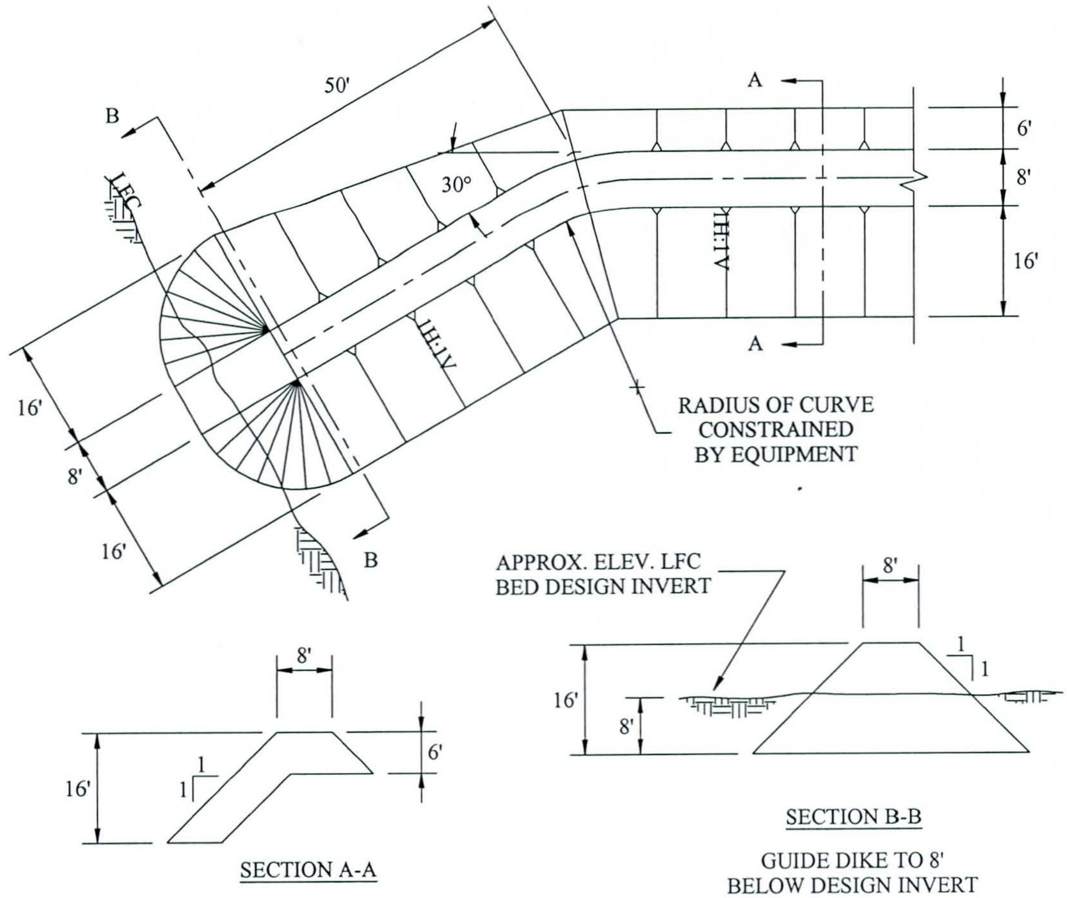



Figure 2-8. Plan view of revised guide dike design, end detail for low flow channel side.

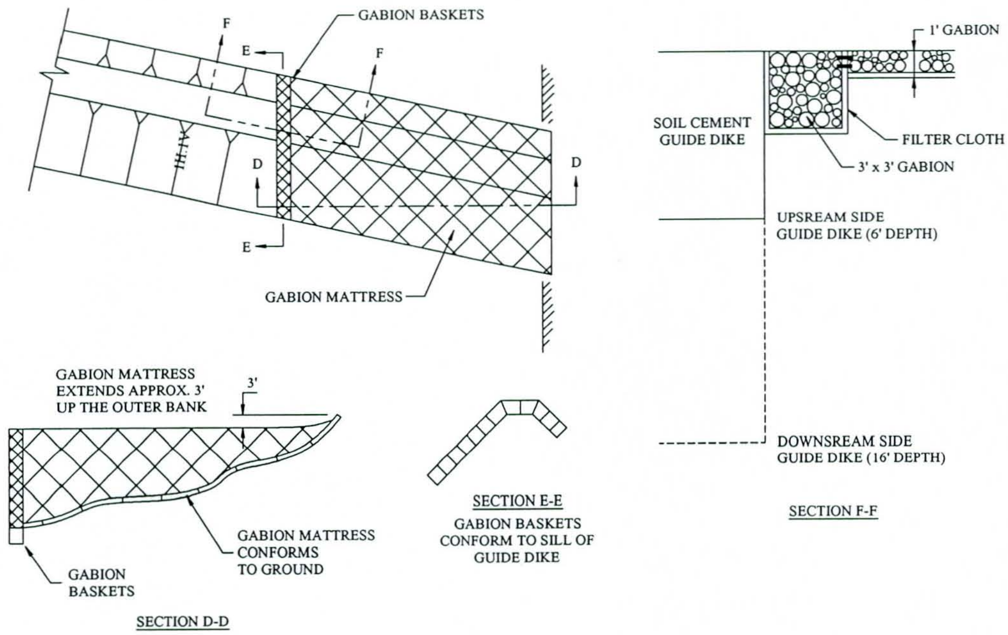



Figure 2-9. Plan view of revised guide dike design, end detail for transition from soil cemebt to gabions near outer bank.

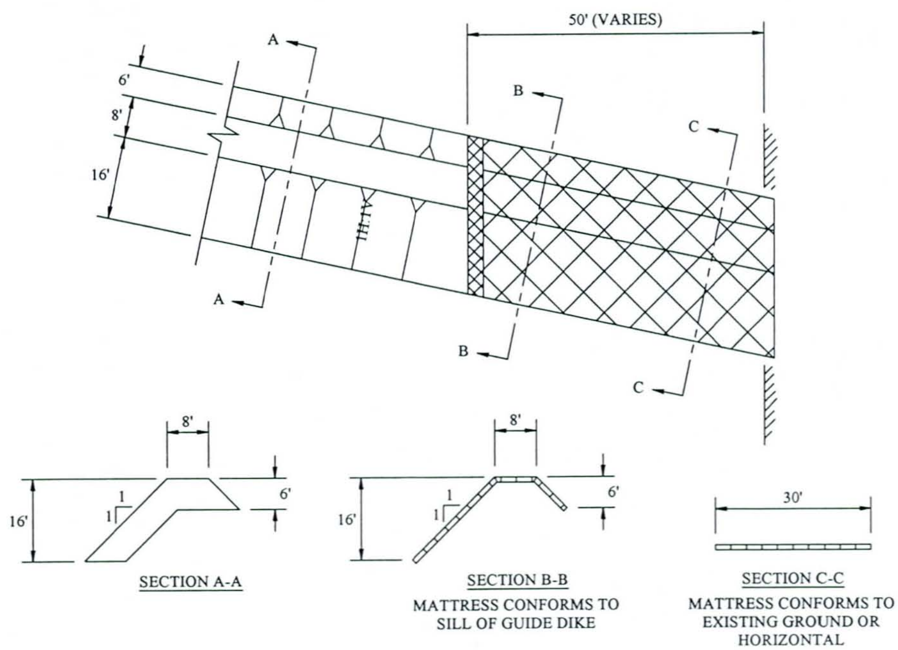



Figure 2-10. Plan view of revised guide dike design, end detail for layout of gabion matress near outer bank.

## 2.4. ADDITIONAL BANK PROTECTION MEASURES

The low flow channel design concept is for a channel with "soft" sides and bottom. However, bank protection is recommended at locations requiring additional erosion protection, such as channel bends with a radius of curvature to channel width ratio of less than five.

The channel bend located downstream of Central Avenue meets this design criterion. Soil-cement bank protection is recommended along the north bank (outside curve of the bend) of the low flow channel for a length of approximately 2,400 feet (Cross Section 213.24 downstream to Cross Section 212.84). It is recommended the top of the soil-cement bank protection be placed above the water surface elevation for the low flow channel design discharge of 12,200 cubic feet per second (approximately 10 feet above the invert), and the toe-down elevation established based on the 100-year scour depth (estimated to be 15 feet). The 100-year scour depth was determined by adding the depths computed for long-term scour, bend scour, and bed-form scour plus a thirty percent factor of safety. The detailed scour calculations are included in Appendix 2.

## 2.5. OTHER CONSIDERATIONS

APS transmission towers are located within the Salt River from west of 24$^{th}$ Street to the 3$^{rd}$ Avenue alignment. APS provided worst case allowable scour elevations around these poles. APS noted the criteria for excavation in the river is that the resultant scour depth should not be lower than the elevation provided.

Four of the APS towers (Pole Numbers 9/2, 9/3, 9/4, and 10/3) are located in the north terrace area of the main channel. The maximum approach velocity for the 100-year discharge is 5 feet per second with local scour depths estimated to be less than 20 feet. Since the minimum allowable scour depth at any of the four poles is 33 feet, the transmission towers are not adversely affected (see Table 2-3).

Table 2-3. APS utility poles and potential scour.

| Pole Location | APS Pole Number | Ground Elev. at Pole (feet) | Velocity (fps) | Max. Scour Depth (feet) | Elev. of Allowable Scour (feet) |
|---|---|---|---|---|---|
| 1st Pole West of 24th Street | 8/2 | 1094 | N/A | N/A | 1027 |
| 2nd Pole West of 24th Street | 8/3 | 1086 | N/A | N/A | 1021 |
| 3rd Pole West of 24th Street | 9/1 | 1085 | N/A | N/A | 1011 |
| 1st Pole West of 16th Street | 9/2 | 1060.8 | 4.97 | 54.8 | 1006 |
| 2nd Pole West of 16th Street | 9/3 | 1046 | 4.43 | 33 | 1013 |
| 3rd Pole West of 16th Street | 9/4 | 1048.5 | 4.50 | 37.5 | 1011 |
| 4th Pole West of 16th Street | 10/1 | 1060 | N/A | N/A | 1002 |
| 1st Pole West of 7th Street | 10/2 | 1065 | N/A | N/A | 996 |
| 2nd Pole West of 7th Street | 10/3 | 1040 | 4.99 | 45 | 995 |
| Lattice Tower at 3rd Avenue Alignment | 10/4 | Unknown | N/A | N/A | 1040 |

## 2.6. References


Carriaga, C.C. and Deschamps, P.Q. (1999) *Safety Assessment of Grade Control Structures*, Proceedings, ASCE 26th Annual Water Resources Planning and Management Conference, Tempe, AZ, June 6-9, 1999.

City of Phoenix (1999) Memorandum: "Rio Salado Terrace Construction Cost Analysis," from Mr. Walt Kinsler, Parks, Recreation and Library Department to Mr. Peter Atonna, Planning Department dated May 14, 1999.

Copeland, R.R., D.N. McComas, N.K. Raphelt and W.A. Thomas (1996) *DRAFT User's Manual for the Hydraulic Design Package for Channels (SAM)*, prepared for U.S. Army Corps of Engineers, Washington, D.C.

Resource Consultants and Engineers, Inc. (1994) *AMAFCA Sediment and Erosion Design Guide*, RCE Ref. No. 90-5560.

Richards, D. and Morrison, T. (1996) "Grade Control Structures for Salt River Channelization," ASCE Conference Proceedings, Anaheim, CA.

U.S. Army Corps of Engineers (1994) *Channel Stability Assessment for Flood Control Projects*, EM-1110-2-1418, Washington, D.C.

U.S. Army Corps of Engineers, Waterways Experiment Station (1993) *Streambank Protection Course, Course Notes and Workbook*, Vicksburg, Mississippi.

U.S. Army Corps of Engineers (1991) *Hydraulic Design of Flood Control Channels*, EM 1110-2-1601, Washington, D.C.

U.S. Bureau of Reclamation (1977) *Design of Small Dams*, Second Edition, Denver, Colorado.

Williams, D.T. (1995) *Selection and Predictability of Sand Transport Relations Based Upon a Numerical Index*, Ph.D. Thesis, Colorado State University, Fort Collins, Colorado.

# 3. HYDRAULIC ANALYSIS

The Hydrologic Engineering Center's River Analysis System (HEC-RAS) Version 2.2 (U.S. Army Corps of Engineers, 1998) is used to perform one-dimensional steady flow analyses for both the existing and the proposed low flow channel conditions.

## 3.1. Existing Conditions HEC-RAS Model

### 3.1.1. Model Description

The existing conditions HEC-RAS project geometry was based on a model provided by the U.S. Army Corps of Engineers, Los Angeles District (the Corps). This model was based on 4-foot contour interval topographic mapping. WEST Consultants, Inc. (WEST) updated this model with one-foot contour interval mapping flown on November 23, 1998 and completed for the Corps in February 1999. The updated topography extended from Interstate 10 to 19th Avenue. WEST also modified the model to include a CLOMR completed by Simons, Li and Associates (1995). For model comparison, the Corps also provided WEST with the 1998 proposed FEMA Flood Insurance Study (FIS) of the Salt River developed by Michael Baker, Jr. Inc. In order to better match the downstream boundary condition with the FIS model, WEST included several downstream cross sections (Cross Sections 207.99 to 211.41) from the FIS model.

The values for the channel and overbank Manning roughness coefficients *n* in the channel in the existing conditions model provided by the Corps were consistent with the observed field conditions (see field notes and photos in Appendix 1). The roughness values were not changed in WEST's existing conditions HEC-RAS model.

There are seven bridges included in the HEC-RAS model within the project reach (Table 3-1). Also included in the model are several other bridges outside the project reach, including the Interstate 10 bridge immediately upstream and the 19th Avenue Bridge downstream of the project. The dimensions of these bridges (except the conveyor crossing upstream of 16th Street) were obtained from as-built plans.

Table 3-1. Bridge and conveyance crossings included in the HEC-RAS model within the project reach.

| Crossing | Cross Section |
|---|---|
| 24th Street | 215.82 |
| Conveyor | 215.12 |
| 16th Street | 214.78 |
| 7th Street | 213.75 |
| Central Avenue | 213.26 |
| 7th Avenue | 212.68 |
| Conveyor (11th Avenue) | 212.34 |

The model was run under a subcritical flow regime based on the assumption that an earth-bottom channel cannot sustain supercritical flow. The model begins with a normal-depth boundary condition at Cross Section 207.99. It was determined that no matter what estimate of the energy grade slope is used at Cross Section 207.99, the water surface elevation in HEC-RAS converges to the same elevation at Cross Section 210.64 (1 mile downstream of the project limits) for a given discharge. This was verified over a range of discharges.

### 3.1.2. Comparison of Existing Conditions and Proposed FIS HEC-RAS Models

A comparison of WEST's existing condition 100-year water surface profile and the 1998 proposed FIS 100-year water surface profile is shown in Figure 3-1. The water surface elevations in WEST's model are generally lower than those of the 1998 proposed FIS model, except near 19$^{th}$ Avenue landfill channelization project (Cross Section 212.26). Upon investigation, it was determined that the flood profile crossover in this reach resulted from major differences in the channel invert elevations. These differences were as much as 10 feet in some locations. The higher invert elevation in the FIS model caused the flow to locally accelerate, consequently lowering the water surface elevation at this location, which is consistent with subcritical flow.

The one-foot contour interval cross section data used in WEST's model are generally lower than the 1998 proposed FIS data by three to four feet, and up to ten feet lower at some locations. Based on a review of the cross sections in both models, the 1998 proposed FIS model cross sections show long spans containing few points and even fewer points near the invert. This suggests that the aerial photogrammetry might have erroneously picked the water surface elevations as the ground profile. In addition, the 1998 proposed FIS models were based upon topography flown in 1992 and 1993. It is possible that significant changes in the bed profile had occurred in the intervening years.

## 3.2. LOW FLOW CHANNEL HEC-RAS MODEL

### 3.2.1. Model Description

The existing conditions geometry was modified by cutting a template of the low flow channel into the existing cross sections using the HEC-RAS channel modification feature. These modifications extended from Cross Sections 211.76 to 216.33, following the description of the low flow channel design found in Chapter 2. After cutting the low flow channel, the bank stations were moved to the top edge of the low channel. Ineffective flow areas were added as necessary to keep flow within the low flow channel until overtopping.

The low flow channel cross section is divided into five areas having different roughness coefficients. These areas are: the left overbank, left side-slope of the low flow channel, bottom of the low flow channel, right side-slope of the low flow channel, and the right overbank. These areas correspond to the five roughness regions input to the sediment transport model. The Manning roughness coefficient (*n*) of the low flow channel bottom is 0.032. For the sideslopes of the low flow channel, the Manning roughness coefficient is 0.060, which corresponds to a moderate vegetative condition. Under most flow conditions, overbank areas are expected to be fully vegetated. Therefore, the Manning roughness coefficient in these areas is 0.085. Downstream of Cross Section 212.84, the type and amount of planned vegetation decrease and the Manning roughness coefficients for both the sideslopes and overbanks transition to 0.045. Under high flow conditions, 70 to 90 percent of overbank vegetation is expected to be destroyed (Feasibility Report, the Corps, 1998). Therefore, the Manning roughness coefficient for the sideslopes and overbank areas is 0.040 when analyzing the water surface profile for the 100-year event having a discharge of 166,000 cfs. Outside of the project reach, the Manning roughness coefficients remain the same as in the existing conditions model.

### 3.2.2. Comparison of Existing and Low Flow Channel Conditions

The water surface profiles for the existing conditions and the proposed low flow channel HEC-RAS models are compared in Figure 3-2. The computed water surface elevations for the proposed low flow channel are equal to or lower than those for the existing condition for the 100-year flood event (166,000 cfs). As mentioned previously, the Manning roughness coefficient is assumed to be 0.040 on the low flow channel sideslopes and in the overbank areas during the 100-year event. A cursory sensitivity analysis showed that the Manning roughness coefficient in these areas could be as high as 0.060 upstream of Cross Section 212.84 and still produce a water surface profile below the existing condition.

Downstream of Cross Section 212.84, the low flow channel is designed to transition into the existing grade. This reach will have less vegetation than the upstream area of the project. If this is the case, the overbank roughness values downstream of Cross Section 212.84 could be less than 0.040 for the 100-year event. A smaller Manning roughness coefficient would result in a water surface profile even lower than that shown in this analysis.

## 3.3. FAILURE OF TEMPE TOWN LAKE DAM

Tempe Town Lake will be located on the Tempe reach of the Salt River, approximately 4.5 miles upstream of the current project. The lake is formed by an inflatable rubber dam across the Salt River channel. This qualitative discussion addresses the potential impact that the failure of this dam might have on the current project.

Tempe Town Lake Dam Emergency Action Plan (HDR, 1999) reports that the flood wave peak from a dam break at Tempe Town Lake would take 49 minutes to travel 5 miles downstream of the dam to 24th Street (near the upper end of the project reach). The peak discharge at 24th Street would be 30,960 cfs and the flow would be moving at a velocity of 7 feet/sec. The flood wave will take 1.6 hours to reach 3rd Avenue (near the lower end of the project reach), which is 8 miles downstream of the Tempe Town Lake Dam. The peak flow would be 23,240 cfs and the flow velocity would be 5.5 feet/sec at 3rd Avenue.

Given the long distance and length of time it will take to reach the project reach, the flood wave from a dam failure at Tempe Town Lake would be greatly attenuated. And since the flood wave attenuates much faster than that of a normal flood event, it would not have an effect any worse than a normal flood event of the same magnitude. Interpolating between the figures taken from the Emergency Action Plan, a dam failure at Tempe Town Lake would have a peak discharge of 23,800 cfs at Central Avenue (where flood-frequency flow data are available). This is approximately equivalent to a 6-year peak flood event. Because of the small magnitude of flow, the possible dam failure at Tempe Town Lake will have a negligible impact upon the current project.

## 3.4. REFERENCES

HDR, Inc. (1999) *Emergency Action Plan for Tempe Town Lake Dam*, prepared for City of Tempe, Arizona.

U.S. Army Corps of Engineers, Hydrologic Engineering Center (1998), *HEC-RAS River Analysis System User's Manual,* Version 2.2 (August 1998), Davis, CA.

U.S. Army Corps of Engineers, Los Angeles District, South Pacific Division (1998), *Rio Salado Salt River, Arizona Feasibility Report and Environmental Impact Statement,* (April 1998).

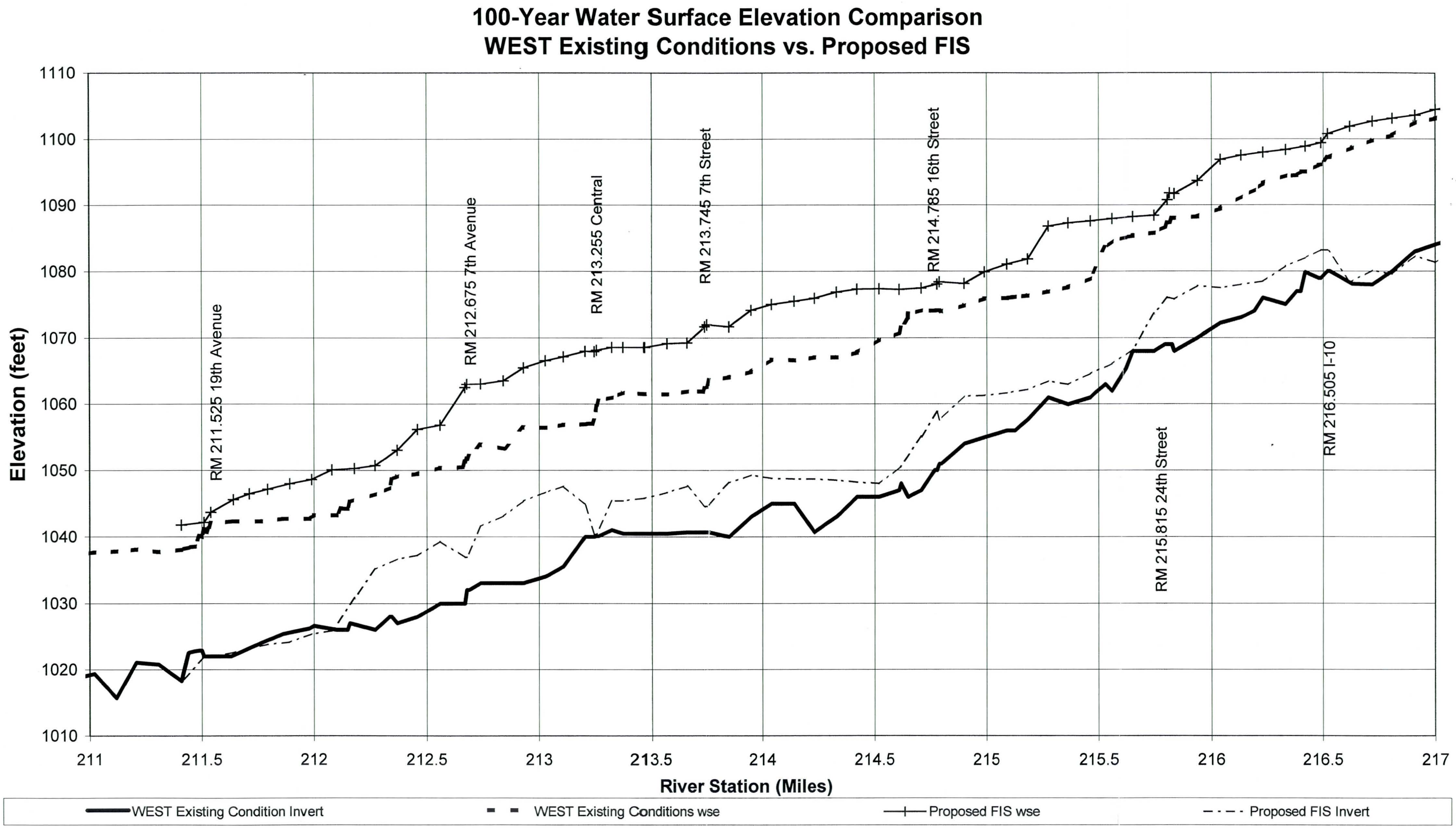



Figure 3-1. Comparison of WEST's existing condition HEC-RAS water surface profile to the proposed FIS water surface profile.



1/11/00, Fig 3-1 and 3-2.xls

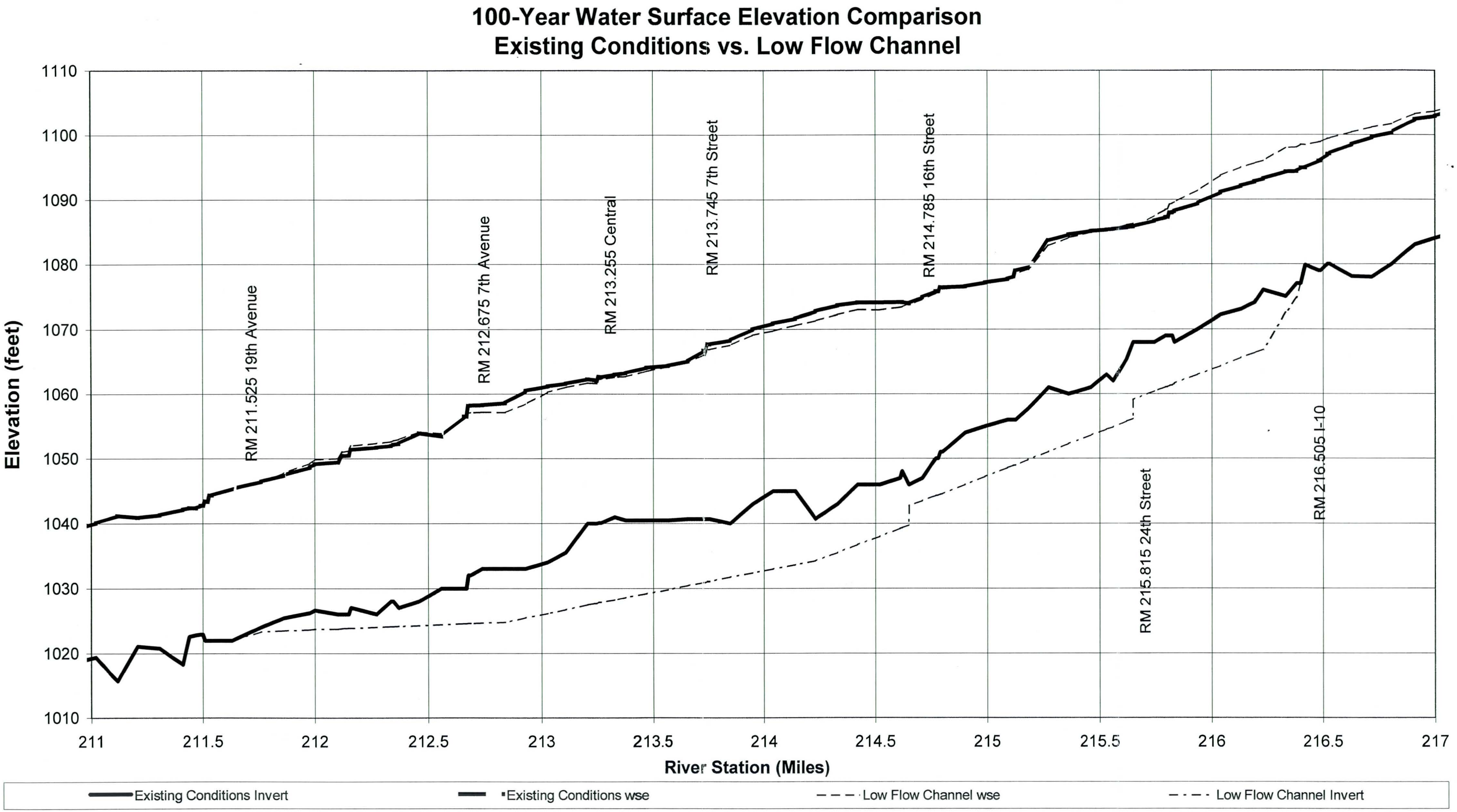



Figure 3-2. Comparison of WEST's existing condition and low flow channel water surface profiles.



1/11/00, Fig 3-1 and 3-2.xls

# 4. SEDIMENT TRANSPORT ANALYSIS

## 4.1. INTRODUCTION

Sediment transport simulations for the existing conditions and proposed low flow channel in the Phoenix Reach of the Salt River have been performed using HEC-6T, Version 5.13.05, dated June 29, 1999. HEC-6T is a one-dimensional movable boundary open channel flow numerical code designed to simulate and predict changes in river profiles resulting from scour and/or deposition over long time periods. HEC-6T is an enhanced version of HEC-6 (Corps, 1993) written by William A. (Tony) Thomas, who developed the original HEC-6 code. The inputs for the HEC-6T model include geometric data, sediment data and hydrologic data. The following sediment transport models have been developed for both the existing and proposed conditions:

1. Long-term sediment transport simulations with a 25-year hydrograph.

2. Simulations of the 100-, 50- and 25-year peak flood events before the 25-year hydrograph.

3. Simulations of the 100-, 50- and 25-year peak flood events after the 25-year hydrograph.

The sensitivity of the model results with respect to the inflowing sediment load and sediment transport method has also been evaluated. The effects of gravel mining, flows from Indian Bend Wash and deflation or failure of the Tempe Town Lake Dam upon the sediment transport in the low flow channel is addressed in the sensitivity analysis for the inflowing load in Section 4.5.3.

A detailed discussion of the sediment transport model inputs and results is included in Appendix 4. A summary of the gravel mining and sediment transport analysis is presented in the following sections.

## 4.2. GRAVEL MINING ANALYSIS

In recent years, sand and gravel mining has occurred upstream and downstream, as well as within the project reach of the Salt River. Mining has consisted of both in-stream mining and overbank mining. However, there is currently no in-stream mining taking place within the project reach.

Over the past ten years, Salt River channel improvements have been designed and constructed upstream of the project reach. These channel improvements extend from approximately I-10 upstream to the Loop 101 crossing (Price Road alignment). Because of these channel improvements, sand and gravel mining is not permitted within the channel. In meetings regarding the Phoenix Rio Salado project, the sand and gravel mining companies stated that in-stream mining is not anticipated within the project reach.

The effects of gravel mining on the selected design alternative, should it occur, have been evaluated by adjusting the inflowing sediment load. This is discussed in Section 4.5.3.

## 4.3. HEC-6T MODEL INPUTS

The HEC-6T model inputs include geometric data, sediment data and hydrologic data. The geometric data include cross section geometry, Manning *n* values, deposition and erosion limits, and depth of the bed sediment reservoir. The bed gradations, sediment transport method and inflowing sediment load are part of the sediment data. The hydrologic data is composed of the discharge-elevation rating curve and hydrographs. The inputs and associated modeling assumptions are summarized in the following sections.

### 4.3.1. Geometric Data

The cross section geometry for both the existing conditions and low flow channel were derived from HEC-RAS cross section geometry. The number of cross sections used in the HEC-6T model is less than that used in the HEC-RAS model and was reduced in order to decrease the computation time and improve computational stability related to sediment continuity.

At the bridges, most of the friction losses are the result of pier losses. The bridge pier and deck information is not coded into the HEC-6T geometry since the highest water elevations did not encounter any bridge decks. However, the cross section of the upstream face of each bridge is retained.

For the existing conditions, the flood control channel is divided into three strips: left overbank, channel and right overbank. For the low flow channel, the cross section geometry is divided into five strips representing portions of the cross section with similar Manning *n* values as follows: left overbank, left channel side slope, channel invert, right channel side slope and right overbank. A detailed discussion of the Manning *n* values is included in Appendix 4.

The depth of the bed sediment reservoir was set at 20 feet, except at grade control structures. For the existing conditions, both the deposition and erosion limits were set at the main flood control channel banks to approximately 10 feet above the channel bottom. For the low flow channel, the deposition and erosion limits were set inside the low flow channel bank stations between Cross Sections 216.23 and 212.84 (cross section stations are in relation to river miles) and then gradually transitioned to the main flood control channel banks between Cross Sections 212.84 and 211.76, where the low flow channel daylights to the existing flood control channel.

### 4.3.2. Sediment Data

#### *4.3.2.1. Existing Conditions Bed Gradations*

The armored layer gradations for the existing conditions cannot be directly determined from the Corps' sediment samples. For the sediment transport analysis, the armored layer gradations were calculated using HEC-6T and input into the OF records for the long-term sediment transport simulations. A more detailed discussion of the methodology used to determine the gradation curves is included in Appendix 4.

#### *4.3.2.2. Low Flow Channel Gradations*

The low flow channel will be constructed by excavating the existing channel bed. Since the low flow channel invert is 8 to 10 feet below the existing bed elevation at most locations, the existing armored layer will be removed during excavation. Therefore, composite bed gradations for cross sections within the low flow channel were developed from the Corps' sediment samples taken between 0 to 6 feet below the proposed low flow channel invert. These gradations are included in Appendix 4B. The bed gradations for cross sections beyond the low flow channel correspond to those in the existing conditions model.

### 4.3.3. Sediment Transport Method

In general, the sediment bed is composed of sand (20 percent), gravel (60 percent) and cobbles (20 percent). Since sand transport is the main transport size and there is a high percentage of gravel in the bed, the Toffaleti, Meyer-Peter and Muller combination transport method is used in the HEC-6T sediment transport simulations because the method suitably transports gravel as well as sand. HEC-6T simulations using the Laursen-Copeland method will be used to evaluate the sensitivity of the sediment transport results to the selection of the transport method. Copeland's solution of the Exner equation (EXNER 7 HEC-6T option) is used in all of the sediment transport simulations.

### 4.3.4. Inflowing Sediment Load

The sediment transport model cannot be directly calibrated to historical conditions because detailed historical bed elevation data are not readily available and the bed elevation changes have been influenced by man-made changes to the Salt River (including sand/gravel mining and channelization). Therefore, equilibrium bed material load curves at the upstream reach of the model was calculated for both the Toffaleti, Meyer-Peter and Muller combination and Laursen-Copeland transport methods for a range of discharges up to 166,000 cfs and used as a basis for the inflowing sediment load for the corresponding HEC-6T models. A detailed discussion of the inflowing load calculations is included in Appendix 4.

### 4.3.5. Hydrologic Data

The 25-year long-term hydrograph was taken from a portion of a simulated hydrograph based on hind-cast flows at Granite Reef Diversion Dam with the Modified Roosevelt Dam in place (the Corps, 1996). The hydrographs for the 25-, 50- and 100-year peak discharge events were developed from the 100-year hydrograph provided by the Corps. A more detailed discussion of the hydrograph development is included in Appendix 4.

The rating curve for the downstream boundary condition in HEC-6T was obtained from the HEC-RAS model results for a range of flows at Cross Section 210.64.

## 4.4. Existing Condition Model Results

A long-term sediment transport simulation (25 years) was completed for the existing conditions using the equilibrium inflowing load and the Toffaleti, Meyer-Peter and Muller combination transport method. The average bed elevation profiles at five-year intervals are shown in Figure 4-1. During the 25-year simulation period, the bed appears to achieve a stable configuration with some deposition and scour that fills-in or smooths some bed irregularities. The magnitude of bed scour and deposition is limited to approximately 2 feet except at two locations: 1) between Cross Sections 214.14 and 214.79 and 2) between Cross Sections 211.02 and 211.21.

The deposition that occurs between Cross Sections 214.14 and 214.79 fills a cavity in the channel that is probably related to mining activities. The deposition that occurs downstream of Cross Section 212.46 results from the decrease in the channel slope. The deposition between Cross Sections 211.02 and 211.21, which is far downstream of the project reach, can be attributed to a change in the bed geometry (see cross section plots in the Hydraulic Appendix).

In general, the model results indicate that the reach upstream of Cross Section 214.99 is in a quasi-equilibrium condition, which may be due to the equilibrium inflowing sediment load and armoring. However, downstream of Cross Section 214.99, there is some deposition caused by the recent channelization and a milder channel slope. This result is supported by field observations at the grade control below 19$^{th}$ Avenue bridge where both the upstream and downstream channel elevations are flush with the grade control sill.

### 4.4.1. Comparison of Transport Methods

The 25-year long-term simulations for the existing conditions with the Toffaleti, Meyer-Peter and Muller combination and the Laursen-Copeland transport methods show the same overall scour and deposition trends. Since the Laursen-Copeland method results in more sand and gravel transport, there is more scour in the upstream part of the reach between I-10 and Cross Section 215.75. This increase in scour results in more deposition in the downstream portion of the reach. However, the maximum difference in either scour or deposition between the two transport methods is 1.5 feet. The Laursen-Copeland transport method is used to estimate long-term degradation and aggradation for bridge scour analysis (see Chapter 5) because this method results in more scour (i.e., worst-case scour).

### <u>4.4.2. Sensitivity to Inflowing Sediment Load</u>

The average bed elevations decreased throughout the reach when the inflowing sediment load was decreased by one-half. The elevation differences from the equilibrium inflowing sediment load scenario were generally less than 1 foot. When the equilibrium inflowing sediment load was increased by a factor of 1.5, deposition increased, but the differences in the average bed elevations from those for the equilibrium load were less than 1 foot. In general, the sediment transport results for the existing conditions are not sensitive to the inflowing sediment load because the bed is initially armored, and therefore minimizes scour.

### <u>4.4.3. Impact of Peak Discharge Events</u>

The impacts of the 25-, 50- and 100-year peak discharge events on the sediment transport results before and after the 25-year long-term simulation period have been evaluated. The corresponding average bed elevations are included in Appendix 4.

When the 100-year peak discharge event occurs after the 25-year simulation period, scour increases in the degradational regions by less than 1 foot and deposition increases in the aggradation zones by 1 to 2 feet. The same trends occur when the 25- and 50-year peak discharge events occur after the 25-year simulation period. However, the magnitudes of the average bed elevation changes are reduced.

When the 100-year peak discharge event occurs before the 25-year simulation period, the overall scour and deposition trends are the same as in the stand-alone 25-year simulation depicted in Figure 4-1. After the 100-year event, the average bed elevations are within 1 foot of those in Figure 4-1 at the end of the 25-year simulation. During the 25 years after the 100-year event, the deposition and scour depths increase by 0.5 to 1 foot. The trends are the same when the 25- and 50-year peak discharge events occur before the 25-year simulation period, but the magnitudes of differences in the average bed elevations decrease.

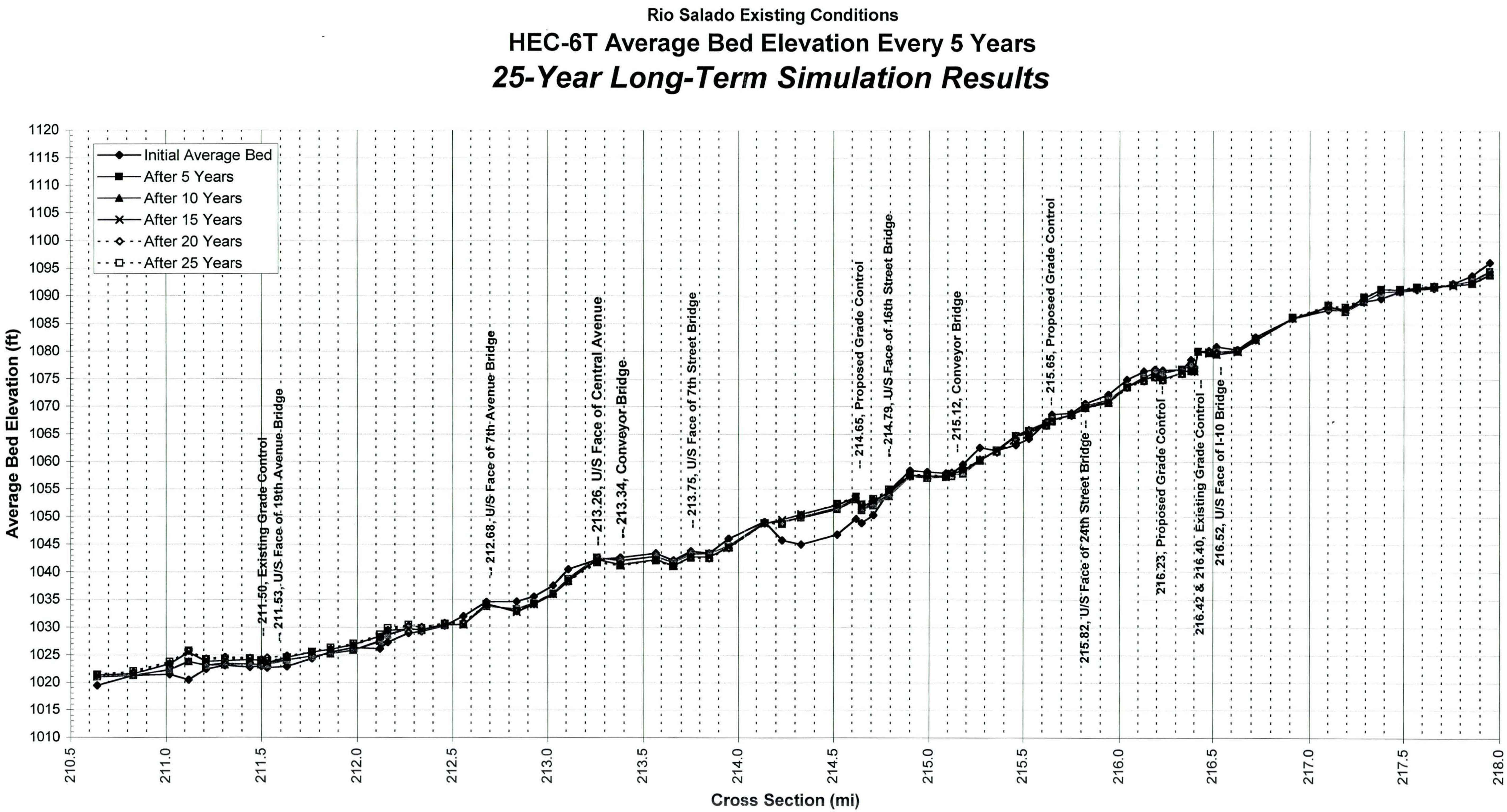



Figure 4-1. Average bed elevation at 5-year intervals for the existing conditions 25-year long-term simulation.

## 4.5. Low Flow Channel Model Results

Twenty-two long-term sediment transport simulations were completed for the low flow channel analysis. The results were used to:

1. Evaluate grade control locations
2. Determine overexcavation depth
3. Determine annual maintenance requirements
4. Estimate the impacts of the 25-, 50- and 100-year discharge events

The sediment transport simulations are discussed in detail in Appendix 4. A summary of the results is presented in the following sections.

### 4.5.1. Grade Control Scenarios

Sediment transport simulations with equilibrium inflowing bed material load were used to evaluate the following five grade control scenarios:

1. Existing grade control structures only.
2. One proposed grade control structure at Cross Section 216.23.
3. Two proposed grade control structures at Cross Sections 216.23 and 215.65.
4. Three proposed grade control structures at Cross Sections 216.23, 215.65 and 214.65.
5. Four proposed grade control structures at Cross Sections 216.23, 215.65, 214.65 and 213.26.

The analysis demonstrated the need for and the locations of the recommended four grade control structures. The functions of the proposed structures are listed below:

1. The proposed grade control located at 216.23 limits the degradation below he existing grade control 216.40, which is one-tenth of a mile downstream of I-10 (Cross Section 216.52).
2. The proposed grade control located at 215.65 reduces degradation in the upstream low flow channel reach.
3. The proposed grade control located at 214.65 is added to protect the 16th Street Bridge (214.79) and 36" water line at (214.80) from excessive scour. Without this grade control, the bed scours to the top of the water line's casing using equilibrium inflowing sediment load and downstream channel overexcavation and maintenance.

4. The proposed grade control structure located at Central Avenue (Cross Section 213.26) protects the bridge from pier scour.

### 4.5.2. Overexcavation and Channel Maintenance

The average bed elevations at 5-year intervals for the 25-year long-term simulations with the equilibrium inflowing sediment load and the four proposed grade control structures are shown in Figure 4-2. With the equilibrium inflowing sediment load, deposition occurs in the downstream reach of the low flow channel (4 to 7 feet between Cross Sections 214.13 and 214.62 and 1 to 3 feet of deposition in the remaining downstream cross sections of the low flow channel). The deposition decreases the low flow channel capacity which in turn increases flooding and damage in the overbanks. Therefore, channel excavation is needed periodically to maintain the low flow channel capacity.

Four overexcavation scenarios (1-foot, 2-foot, 3-foot and 4-foot overexcavation depths) were evaluated and the resulting annual maintenance (i.e. channel excavation) examined. The 2-foot overexcavation scenario is recommended because it results in the least amount of scour above the grade control located at 214.65 (i.e. at the 36" water line). For the 2-foot overexcavation scenario, the scour depth at the 36" inch water line is 0.5 foot less than that for the 3-foot and 4-foot excavation scenarios. The average bed elevations at 5-year intervals are shown in Figure 4-3 for the low flow channel with the equilibrium inflowing sediment load and 2-foot overexcavation with maintenance. A more detailed discussion is included in Appendix 4.

### 4.5.3. Sensitivity to Inflowing Sediment Load

A comparison of the average bed elevations at the end of the 25-year simulation period with 50%, 100% and 150% of the equilibrium bed material load and 2-foot overexcavation (Scenarios L-4b, L-3b and L-5b, respectively) is shown in Figure 4-4. The average bed elevations for the upstream reach of the low flow channel between Cross Sections 216.23 and 215.65 are within a 1-foot range for all three inflowing loads. However, for the cross sections downstream of 215.65, the average bed elevations decrease by 1 to 2 feet when the inflowing load is reduced by one-half. Conversely, the average bed elevations increase by 0.5 feet when the inflowing load is increased by a factor of 1.5. In general, the low flow channel is moderately sensitive to changes in the inflowing sediment load with scour increasing as the inflowing sediment load decreases.

Sand and gravel mining upstream of the low flow channel would cause a decrease in the inflowing sediment load. The effects of such mining are bracketed between Scenarios L-6b and L-5b (see Figure 4-4) in which the inflowing sediment loads are 50% and 100% of the equilibrium load, respectively. Upstream sand and gravel mining could cause 1-2 feet of additional scour in the low flow channel. Sediment discharges resulting from flows in Indian Bed Wash and/or the deflation or failure of the Tempe Town Lake Dam would increase the inflowing sediment load to the low flow channel. The effects are bracketed between Scenarios L-5b and L-7b (see Figure 4-4) in which the inflowing sediment loads are 100% and 150% of the equilibrium load, respectively. The failure of the Tempe Town Lake Dam would result in deposition and increase excavation requirements for the low flow channel downstream of Cross Section 214.65.

### 4.5.4. Comparison of Transport Methods

The low flow channel scour upstream of the grade control structure located at 214.65 significantly increases by 4 to 5 feet when the Laursen-Copeland transport method is used. This scenario results in the greatest amount of long-term scour and will be used in the bridge scour analysis. A more detailed discussion and average bed elevation plots are included in Appendix 4.

### 4.5.5. Impact of Peak Discharge Events

The 25-, 50- and 100-year peak discharge events increase degradation upstream of the grade control structure located at 214.65 and increase deposition downstream. When the 100-year peak discharge event occurs after the 25-year simulation period, the scour depths increase by 1 foot between the grade control structures at 214.65 and 215.65 and by 2 to 3 feet between the grade control structures at 215.65 and 216.23. The same trends occur when the 25- and 50-year peak discharge events occur after the 25-year simulation period, but the magnitudes of the scour and deposition are reduced.

When the 100-year peak discharge event occurs before the 25-year simulation period, the scour depths upstream of the grade control structure at 214.65 are 4 to 6 feet. At the end of the simulation period, the average bed elevations upstream of the grade control at 214.65 are generally 1 to 2 feet lower than those without the 100-year event. The trends are the same when the 25- and 50-year peak discharge occur before the 25-year simulation period, but the magnitudes of the scour and deposition are reduced.

In general, the sediment transport results are more sensitive to the sediment transport method than to the 25-, 50- and 100-year peak flood events.

## 4.6. REFERENCES

U.S. Army Corps of Engineers, Hydrologic Engineering Center (1993), *HEC-6 Scour and Deposition in Rivers and Reservoirs. User's Manual,* (August 1993), Davis, CA.

U.S. Army Corps of Engineers, Los Angeles District (1996), *Section 7 Study for Modified Roosevelt Dam, Arizona (Theodore Roosevelt Dam). Hydrologic Evaluation of Water Control Plans, Salt River Project to Gila River at Gillespie Dam,* (March 1996).

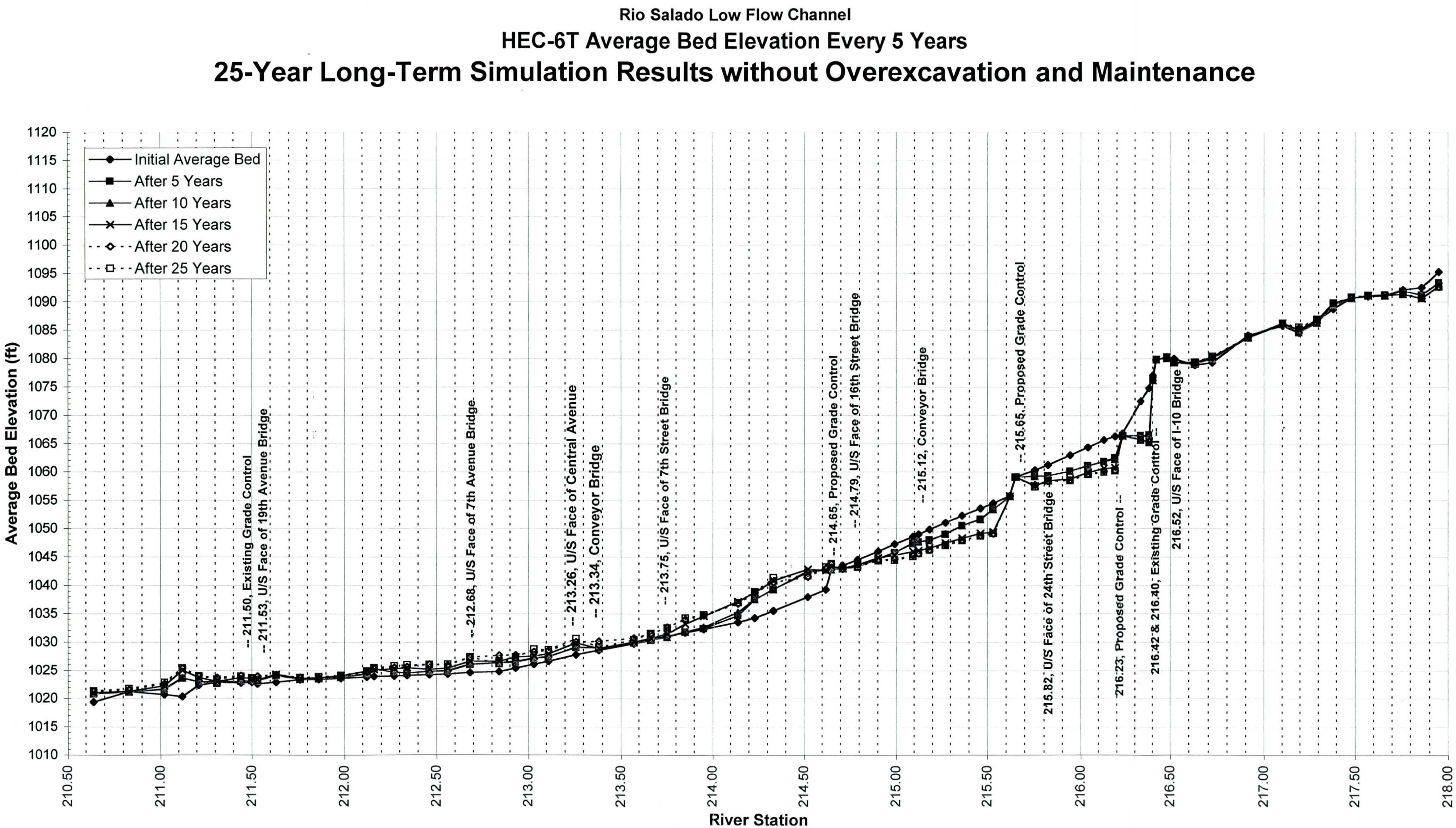



Figure 4-2. Average bed elevation at 5-year intervals for the low flow channel with the equilibrium inflowing sediment load and the four proposed grade control structures. Overexcavation and annual maintenance is not modeled.

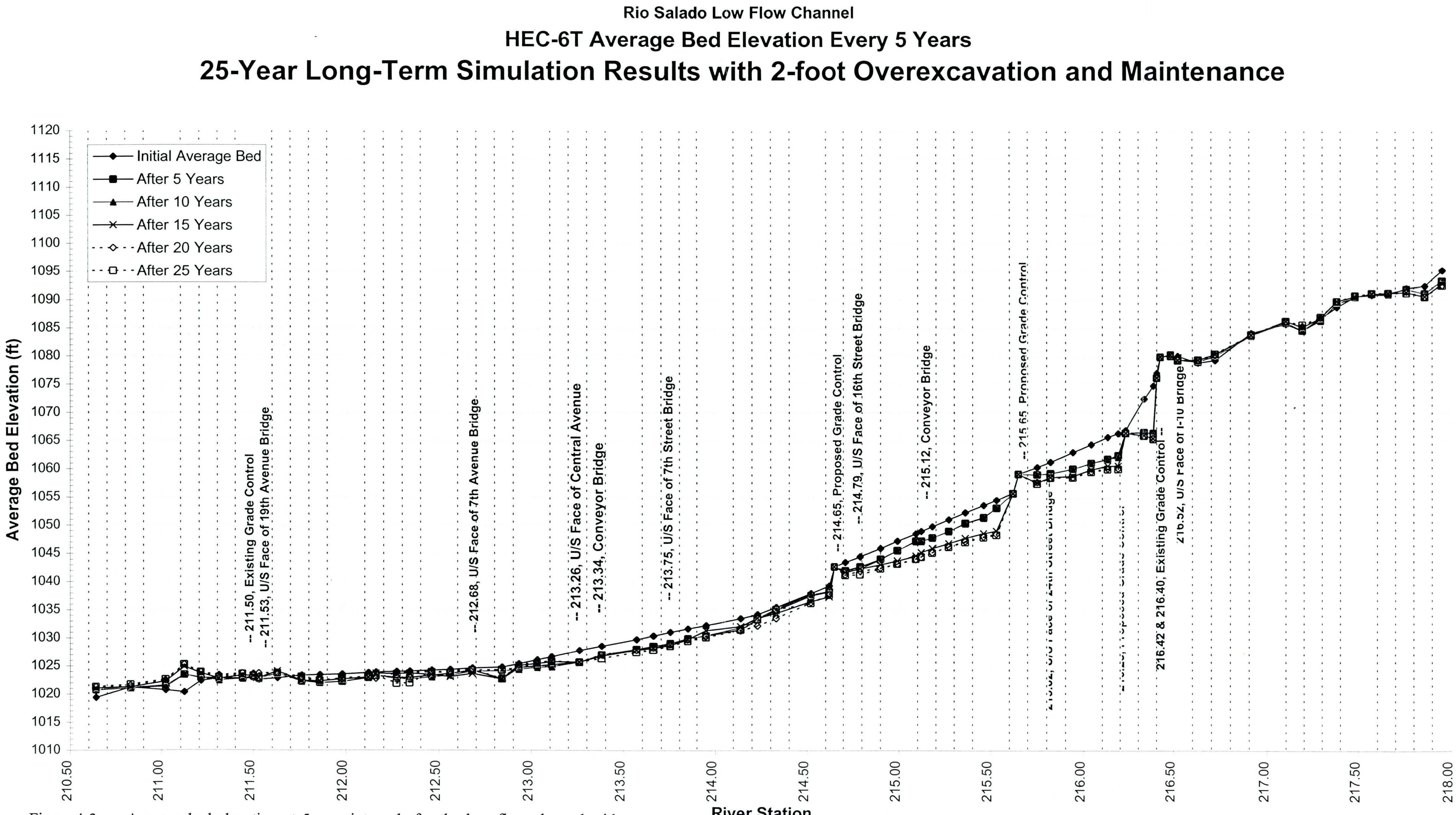



Figure 4-3. Average bed elevation at 5-year intervals for the low flow channel with the equilibrium inflowing sediment load, the four proposed grade control structures, and 2-foot overexcavation and annual maintenance.

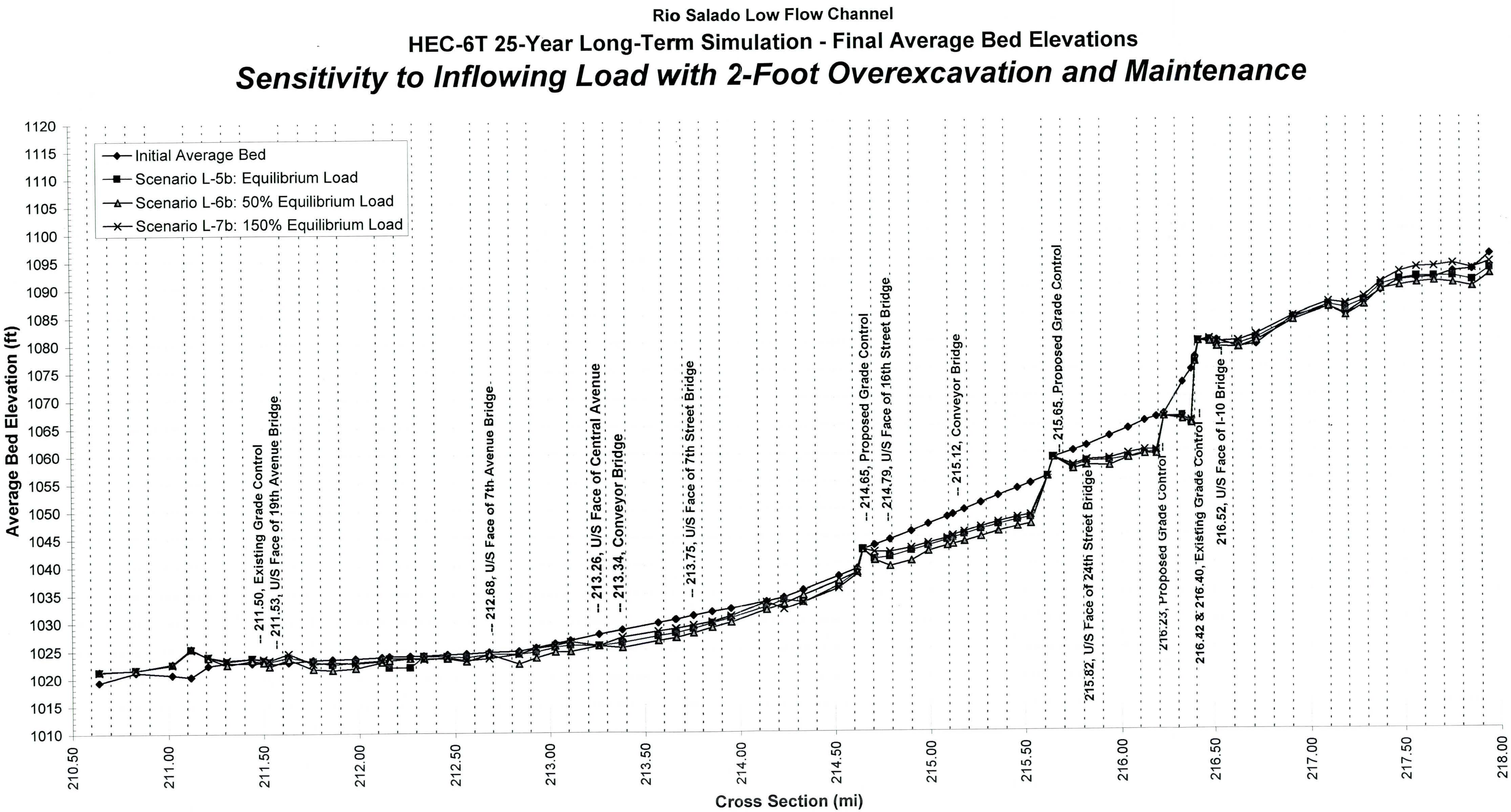



Figure 4-4. Sensitivity of the sediment transport results to the inflowing sediment load for the low flow channel with the four proposed grade control structures and 2-foot overexcavation and maintenance. A comparison of the average bed elevations at the end of the 25-year simulation period for 50%, 100% and 150% of the equilibrium inflowing sediment load.

# 5. BRIDGE SCOUR ANALYSIS

## 5.1. INTRODUCTION

Bridge scour analyses were conducted for bridges in the project reach based on the 100-year discharge of 166,000 cubic feet per second. The seven bridges analyzed were the 24th Street Bridge, the upstream conveyor bridge, the 16th Street Bridge, the 7th Street Bridge, the Central Avenue Bridge, the 7th Avenue Bridge, and the downstream conveyor bridge. The analyses were conducted for both existing and low flow channel conditions.

The total scour at a bridge is composed of three components: 1) long-term aggradation and degradation, 2) contraction scour, and 3) local scour at abutments and piers. Aggradation and degradation are long-term streambed elevation changes. Contraction scour is the removal of material from the bed and banks across all or most of the channel width that results from a contraction of the flow area at the bridge. Local scour at the abutments and piers is caused by an acceleration of flow and resulting vortices induced by the flow obstructions.

Contraction scour is considered negligible at all seven bridges because there are no flow contractions between the bridge approach sections and the bridge openings. Since the abutments do not project abruptly in the flow field for all seven bridges, the local scour at the abutments is also insignificant. Therefore, only long-term aggradation and degradation and the local scour at the piers are calculated in this analysis.

Bridge scour was evaluated using the HEC-RAS computer program, Version 2.2, (U.S. Army Corps of Engineers, 1998) per Federal Highway Administration (FHWA) guidelines (FHWA, 1993). The bridge scour results are presented in Table 5-1 and Table 5-2 for the existing and low flow channel conditions, respectively.

Local pier scour was computed using the Colorado State University (CSU) equation. The median grain size ($d_{50}$) and the dominant grain size ($d_{90}$) were estimated from composite gradation curves developed from soil boring data. Two feet of debris loading was added on each side of the piers for the pier scour computations as required by Arizona Department of Transportation guidelines. An angle of attack of zero degrees was used in pier scour computations since the piers of all the bridges are aligned parallel to the flow direction. Pier shape factors in the CSU equation were estimated from field photographs and "as-built" bridge plans. The bed condition was assumed to consist of small dunes (2 to 10 feet). The maximum stream tube velocity and maximum depth within the channel were used to compute pier scour for all piers.

For the existing conditions, local pier scour is computed using the natural channel geometry before long-term aggradation and degradation is added. The total scour is the sum of the local pier scour and long-term degradation. The long-term degradation and aggradation is obtained from a 100-year HEC-6T simulation using the Laursen-Copeland sediment transport method, which results in the worst-case scour (see Chapter 4). The 25-year hydrograph was repeated four times in the 100-year HEC-6T simulation. At locations with long-term aggradation, the total scour equals the local pier scour only.

For the low flow channel, the local pier scour is computed using channel geometry that reflects the 100-year long-term profile computed using HEC-6T. In the depositional zone between Cross Sections 211.76 and 214.65, the overexcavated channel geometry is used except at locations where the channel scoured below the overexcavation depth. The total scour depth is the sum of the local pier scour depth and long-term degradation.

The information available for the two conveyor bridges was not as complete as for the other five bridges. For the upstream conveyor bridge, no plans were available, only a geotechnical report. Aerial photographs and topographic maps were used to determine the location and orientation of these two structures.

## 5.2. EXISTING CONDITIONS

Results for the existing conditions bridge scour analyses are provided in Table 5-1. The table includes the invert elevation at the upstream face of each bridge (obtained from the HEC-RAS model), long-term degradation depth, local pier scour depth, total scour elevation (invert elevation – long-term degradation – local pier scour), top of footing or pile cap elevation, bottom of footing or pile cap elevation, pile or caisson tip elevation, and invert elevation from "as-built" bridge plans. Input data required for the scour analysis along with scour results are included in Appendix 5.

As indicated in Table 5-1, the invert elevation at the upstream face of the Central Avenue Bridge is not significantly different from the invert elevation indicated on the "as-built" bridge plans. Most of the other bridges have an invert elevation five to six feet lower than the "as-built" invert elevation. At 24$^{th}$ Street, the pile tips are approximately 35 feet below the scour elevation while at 16$^{th}$ Street, 7$^{th}$ Street, and 7$^{th}$ Avenue, the piles have more than 60 feet of embedment below the total scour elevation. At Central Avenue, the minimum scour elevation is approximately 12.9 feet above the bottom of the spread footing. The pile tips of the upstream conveyor bridge at Cross Section 215.12 are 6 feet below the total scour elevation. However, at the downstream conveyor bridge at Cross Section 212.34, total scour extends more than 7 feet below the spread footing.

## 5.3. LOW FLOW CHANNEL

Results of the bridge scour analysis with the low flow channel geometry are provided in Table 5-2. The input data used for the scour analysis along with the scour results are included in Appendix 5. The local pier scour is computed using channel geometry that reflects the 100-year long-term profile computed using HEC-6T, therefore the long-term degradation is already accounted for.

With the low flow channel, total scour extends 5 feet below the spread footing elevation at Central Avenue and more than 14.5 feet below the spread footing elevation at the downstream conveyor bridge. The downstream conveyor bridge is to be demolished as part of low flow channel construction project. At the upstream conveyor bridge, the total scour elevation is 8.6 feet below the pile tip elevation for the structure. At 24$^{th}$ Street, the pile tips are approximately 22 feet below the scour elevation while at 16th Street, 7th Street, and 7th Avenue, the piles have more than 50 feet of embedment below the total scour elevation.

Based on the results of the scour analysis, countermeasures may be required at some of the bridges. A structural analysis to determine the structural stability for both existing and low flow channel conditions has been conducted for the 7th Avenue Bridge and the Central Avenue bridges. It was determined that the Central Avenue bridge was not stable with the low flow channel. A grade-control structure located immediately downstream of Central Avenue (Cross Section 213.26) with a soil-cement apron within the low flow channel and extending through the bridge is the recommended scour countermeasure. A structural analysis will be conducted for all structures in the project reach, with an addendum to this report issued, which will include the results of the analysis as well as recommended structural retrofits or scour countermeasures.

## 5.4. REFERENCES


U.S. Army Corps of Engineers, Hydrologic Engineering Center (1995), *Evaluating Scour at Bridges,* Third Edition (November 1995), Washington, D.C.

U.S. Department of Transportation, Federal Highway Administration (1998), *HEC-RAS River Analysis System User's Manual,* Version 2.2 (August 1998), Davis, CA.

Table 5-1. Existing condition bridge scour summary.

| Bridge (Cross Section) | Invert Elevation at U/S Bridge Face (feet) | Long-Term Degradation (feet) | Local Pier Scour (feet) | Total Scour Elevation (feet) | Top of Footing or Pile Cap Elevation (feet) | Bottom of Footing or Pile Cap Elevation (feet) | Pile or Caisson Tip Elevation (feet) | Channel Invert Elevation from 'As-Built' Plans (feet) |
|---|---|---|---|---|---|---|---|---|
| **24$^{th}$ Street (215.815)** | 1069.00 | 3.12 | 15.60 | **1050.28** | 1055.00 | 1051.25 | 1015.25 | 1075.00 |
| **Conveyor (215.12)** | 1056.00 | 0.00 | 20.12 | **1035.88** | 1062.20 | 1059.70 | 1029.7 | Unknown |
| **16$^{th}$ Street (214.785)** | 1051.00 | 0.00 | 19.03 | **1031.97** | 1055.00 | Caisson | 950.00 | 1057 (scaled) |
| **7$^{th}$ Street (213.745)** | 1040.70 | 0.47 | 21.61 | **1018.62** | 1040.00 | Caisson | 945.00 | 1045 |
| **Central Ave. (213.255)** | 1040.10 | 1.84 | 15.34 | **1022.92** | 1015.00 | 1010.00 | Spread Footing | 1040 |
| **7$^{th}$ Avenue (212.675)** | 1032.00 | 0.00 | 20.04 | **1011.96** | 1032.50 | Caisson | 948.00 | 1037 |
| **Conveyor (212.34)** | 1028.00 | 0.00 | 16.90 | **1011.10** | 1021.65 | 1018.65 | Spread Footing | 1033.65 |

Table 5-2. Low flow channel bridge scour summary.

| Bridge (Cross Section) | Invert Elevation of U/S Bridge Face LFC Design (feet) | Long-Term Degradation (feet) | Local Pier Scour (feet) | Total Scour Elevation (feet) | Top of Footing or Pile Cap Elevation (feet) | Bottom of Footing or Pile Cap Elevation (feet) | Pile or Caisson Tip Elevation (feet) | Channel Invert Elevation from 'As-Built' Plans (feet) |
|---|---|---|---|---|---|---|---|---|
| **24th Street (215.815)** | 1061.28 | 7.76 | 15.60 | **1037.92** | 1055.00 | 1051.25 | 1015.25 | 1075.00 |
| **Conveyor (215.12)** | 1048.96 | 5.78 | 22.11 | **1021.07** | 1062.20 | 1059.70 | 1029.7 | Unknown |
| **16th Street (214.785)** | 1044.48 | 4.05 | 22.08 | **1018.35** | 1055.00 | Caisson | 950.00 | 1057 (scaled) |
| **7th Street (213.745)** | 1031.06 | 2.26 | 23.58 | **1005.22** | 1040.00 | Caisson | 945.00 | 1045 |
| **Central Ave. (213.255)** | 1027.84 | 2.00 | 20.81 | **1005.03** | 1015.00 | 1010:00 | Spread Footing | 1040 |
| **7th Avenue (212.675)** | 1024.61 | 2.00 | 21.77 | **1000.84** | 1032.50 | Caisson | 948.00 | 1037 |
| **Conveyor (212.34)** | 1024.11 | 2.00 | 17.95 | **1004.16** | 1021.65 | 1018.65 | Spread Footing | 1033.65 |

# 6. COSTS

## 6.1. DESCRIPTION

Cost estimates for the construction of the selected low flow channel design alternative are provided within this section. The estimate includes costs for soil cement bank protection, soil cement grade control structures, and soil cement and gabions for guide dikes.

Soil cement was the primary construction material considered during the analysis and design of the low flow channel. The unit costs presented in this report were reviewed and accepted by the design review team, which included representatives from the Corps, the Flood Control District of Maricopa County, and the City of Phoenix. Table 6-1 shows the unit prices used for the cost estimates.

Table 6-1. Unit prices.

| Item Description | Unit | Unit Price |
|---|---|---|
| Soil Cement – Bank Protection | CY | $35 |
| Soil Cement – Grade Control | CY | $45 |
| Soil Cement – Guide Dikes | CY | $45 |
| Gabions – Guide Dikes | CY | $85 |

Since the completion of this analysis, it has been decided that roller compacted concrete rather than soil cement should be used. The construction of roller compacted concrete structures is the same as for soil cement structures. Roller compacted concrete provides higher strengths than soil cement.

The soil cement unit price includes the cost for furnishing all equipment, labor, and materials (including cement) necessary to complete the soil cement bank protection or grade control, including dewatering, trench excavation and toe backfill, watering, mixing, placing, and compacting. The soil cement bank protection unit price is based on information received from the Flood Control District of Maricopa County for recently bid or completed projects. The projects include Contract FCD 98-37, Camelback Ranch Levee North and Glendale Airport, and Contract FCD 97-18, Camelback Ranch Levee South. On these two projects, there were seven bids. The soil cement bank protection unit bid prices, including cement, ranged from $29 to $40/cy. The unit price of $35 is typical of soil cement bank protection projects completed along the Salt River during the past ten years. The unit price for soil cement grade control structures is greater due to the increased cement content and additional dewatering requirements.

The gabions unit price includes the cost for furnishing all equipment, labor, and materials, dewatering, trench excavation and toe backfill, and placing of rock riprap.

## 6.2. SELECTED ALTERNATIVE COST ESTIMATES

Preliminary cost estimates for bank protection, grade controls, and guide dikes are provided for the selected low flow channel design alternative are presented in Table 6-2. Quantity calculations for individual items are included in Table 6-2. Bank protection, grade control structures, and guide dikes have been designed with toe-downs based on the 100-year scour depth. The grade control structures are essentially the same type of grade control that has been used along the Salt River upstream of the project reach as well as immediately downstream of 19$^{th}$ Avenue. The primary difference is the addition of a notch for the low flow channel and the stepped apron within the low flow channel.

The cost estimates provided do not include costs for channel excavation (other than those incidental to soil-cement construction), water management, debris disposal, mitigation measures at utilities, mobilization, etc., or any contingencies.

The selected design alternative includes four grade control structures across the full width of the flood control channel with notches for the low flow channel. The low flow channel side slopes are to have vegetated side slopes which vary from 3H:1V to 4H:1V. Soil cement bank protection is recommended for the north bank of the low flow channel downstream of Central Avenue. The length of protection is approximately 2,400 lineal feet. The height of this protection (low flow channel depth + toe down) is 25 feet.

The selected alternative includes guide dike structures at strategic locations along the overbank area. The guide dikes serve to maintain the alignment of the low flow channel, protect the main channel bank, and minimize formation of secondary channels in the overbank areas. Three guide dike construction alternatives have been evaluated: 1) soil cement guide dikes; 2) slurry trench walls; and 3) gabions for the overbank portion of the guide dike with soil cement for the dike section located along the low flow channel. The selected alternative is a soil cement guide dike with gabions the final 50 feet.

Table 6-2. Cost Estimate[1]

| Item | Unit | Quantity | Unit Price | Amount |
|---|---|---|---|---|
| Soil Cement Bank Protection | CY | 23,200 | $35 | $812,000 |
| Grade Control @ 216.23 – Soil Cement | CY | 13,325 | $45 | $599,250 |
| Grade Control @ 215.65 – Soil Cement | CY | 12,324 | $45 | $554,580 |
| Grade Control @ 214.65 – Soil Cement | CY | 12,177 | $45 | $547,965 |
| Grade Control @ 213.26 – Soil Cement | CY | 15,148 | $45 | $681,660 |
| Apron @ 213.26 – Soil Cement | CY | 12,987 | $45 | $584,415 |
| Guide Dikes – Soil Cement | CY | 37,803 | $45 | $1,701,135 |
| Guide Dikes - Gabions | CY | 2,433 | $85 | $206,805 |
| **Total** | | | | **$5,687,810** |

[1] The cost estimates provided do not include costs for channel excavation (other than those incidental to soil-cement construction), water management, debris disposal, mitigation measures at bridges or utilities, mobilization, etc., or any contingencies.