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US Army Corps
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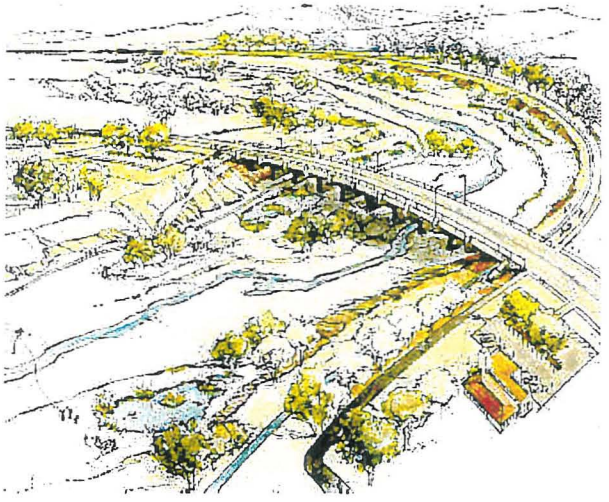
RioSalado

SALT RIVER, ARIZONA

DESIGN DOCUMENTATION REPORT



Low Flow Channel, Phoenix Reach

February 2000

INTRODUCTION

Together with the City of Phoenix and the City of Tempe, Arizona, the Corps of Engineers, Los Angeles District completed the "Rio Salado, Salt River, Arizona, Feasibility Report and Environmental Impact Statement" in April 1998. The Report proposes to restore the ecosystem function of the Salt River as it weaves through Phoenix and Tempe. Besides establishing a variety of habitats, the Report imparts a multitude of recreational opportunities.

The targeted areas of improvement includes five miles of the Salt River in Phoenix and 1.3 miles of the tributary Indian Bend Wash with another mile of the Salt River in Tempe. Biological improvements include mesquite terraces, pockets of willow and cottonwood, wetland marshes, aquatic strands, and open edges. The recreational plan recommends trails, parking lots, rest rooms, educational signage, shelters, and associated features. The completed project will harmoniously combine differing habitats with diverse recreational opportunities resulting in an enriching experience for all visitors.

Within the Phoenix Reach, a low flow channel is proposed to maintain the existing level of flood control impacted by the restoration plans. This Design Documentation Report (DDR) specifically focuses on detailing the design of this Low Flow Channel (LFC), Phoenix Reach. The LFC will carry 12,200 cubic feet per second of flow along five miles of the Salt River and pass below five bridges. Within the LFC will be a perennial stream connecting lakes and meandering around islands inhabited by wildlife.

This DDR explains the basis of design for the LFC's construction plans and specifications

I. PROJECT DESCRIPTION

A. Project Authorization

The 1994 Senate Energy and Water Development Appropriations Bill initiated the Rio Salado, Salt River, AZ Reconnaissance Report, U.S. Army Corps of Engineers, Los Angeles District, South Pacific Division, March 1995 which eventually led to the Rio Salado, Salt River, Arizona, Feasibility Report and Environmental Impact Statement, April 1998. On August 20, 1998, the Chief of Engineers (COE), U.S. Army Corps of Engineers recommended that the Secretary of the Army approve the Feasibility Study's selected plan and allow the project to proceed into the Preconstruction Engineering and Design (PED) phase per ER 1110-2. The Rio Salado Project is divided into the Phoenix Reach and Tempe Reach. PED for the Phoenix Reach was authorized by 31 July 1998. This report serves as the Design Documentation Report, as required by Engineering Regulation 1110-2-1150 (April 1998 Draft), for the Low Flow Channel, Rio Salado Project, Phoenix Reach, Arizona. The project sponsor is the City of Phoenix (COP), who is partnered with the Flood Control District of Maricopa County (FCDMC). The FCDMC's contribution will be to award and construct the LFC.

B. Overview of the Feasibility Study's Recommended Plan

1. Introduction

The Feasibility Study recommends environmental restoration and recreation along approximately five miles of the Salt River as it flows westward through Phoenix, Arizona from the Interstate10 (I-10) Highway bridge to the 19th Avenue bridge. Along this reach are five bridges namely 24th Street, 16th Street, 7th Street, Central Avenue, and 7th Avenue. Environmental restoration is composed of establishing a variety of indigenous habitats. In order to maintain the existing 100-year flood capacity, the Feasibility Study suggests entrenching a low flow channel (LFC) within the river's bottom to mitigate the capacity reduction induced by the restorative features. This report presents the design decisions, criteria, and process used to prepare the construction plans and specifications for the LFC.

2. Low Flow Channel

Design discharge for the LFC is set at 12,200 cubic feet per second (cfs) based on 5-year frequency flood event and anticipated 30-day release duration from Salt River Project. The LFC starts from the Arizona Department of Transportation's grade control structure downstream of the I-10 bridge and daylights with existing grade almost one mile upstream of 19th Avenue bridge. The piers of the five bridges mentioned previously will be protected based on recommendations by a Structural Consultant. LFC will be an entrenched, soft-bottom trapezoidal channel with side slopes varying from 3H: 1V to 4H: 1V with a transition of 10:1 to go from one side slope to the other. Base width of the LFC will vary between 160 and 205 feet and reach down to 20 feet below the existing river bottom. Integrity of the LFC will be maintained by Roller Compacted Concrete (RCC) side slopes where necessary with 5-foot toedown.

3. Perennial Stream

Environmental restoration within the LFC hinges on a system providing water required for propagation of natural vegetation and wildlife habitat. The system includes a perennial stream connecting four lakes. The stream would be about 20 feet in width, less than a foot deep, meander over a length of 7 miles, and carry 5 to 10 cfs. Each lake will average three feet deep over two to three acres and be underlain by a clay liner or geomembrane (to be determined by the A/E for the Environmental Restoration project) to prevent excessive infiltration. The lakes will be located in the vicinities of 16th Street, 7th Street, Central Avenue, and 7th Avenue. The perennial system will not be constructed by FCDMC.

C. Value Engineering Study

A Value Engineering (VE) conference was held in Phoenix during September 1998. A draft report followed and a conference was held between COP and COE regarding the VE recommendations on October 20, 1998. There was a consensus on implementing or investigating some of the recommendations. A reoccurring suggestion was to use more grade control structures (GCS) to eliminate or at least minimize the need for side slope protection. This change will allow the LFC to meander thereby creating a more natural looking river populated by valuable habitat. WEST Consultants, Inc., was contracted to prepare a hydraulic study investigating the viability

of the VE suggestions. Four GCS were included in the design along with 36 guide dike structures (GDS) to minimize bank erosion and keep the LFC within the reach.

D. Low Flow Channel Design & Hydraulic Analysis

Channel Design - A low flow channel design alternative was selected for detailed evaluation. The design slope of the proposed low flow channel is 0.0025 ft/ft. 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 . A portion of the low flow channel was widened to 205 feet in order to reduce the amount of scour occurring at the 16th Street Bridge (214.79) and at the 36" water line crossing (214.80) just upstream of the bridge. The low flow channel has 3H:IV vegetated side slope with RCC 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 following guidelines:

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

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

3) Align low flow channel to avoid bridge piers.

4) Align the flow with the bridge piers.

5) Minimize the change in the channel sinuosity.

Water Surface Comparisons - The water surface elevations computed using Hydrologic Engineering Center's River Analysis System (HEC-RAS) for the low flow channel design alternatives are equal to or lower than the WEST's existing conditions model for the

100-year peak discharge of 166,000 cfs. In this analysis, it is assumed that vegetation damage reduces the overbank Manning n value from 0.08 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 Baker proposed FIS model.

Grade Control Structures - Four grade control structures, constructed from RCC, 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 cross section 213.24 to protect bridge piers at Central Avenue. The grade control structures extend across the full width of the flood control channel. Recommended toe-down depths are 30 feet within the low flow channel and 16 feet in the overbank areas. Riprap protection in the low flow channel was computed for the 10-year discharge upstream and downstream of the structures using a gradation with maximum stone diameter of 3.5 feet and a mean stone diameter of 2 feet.

Guide Dikes - The selected low flow channel alternative includes 36 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.

Bridge Scour - Bridge scour analysis was conducted for the seven bridges within the project reach using the 100-year discharge of 166,000 cubic feet per second. Contraction scour and abutment scour was 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 degradati6n was determined from the 25-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. Structural analysis of the bridges

is being performed at present and design of recommended fix will be included in the 100% plans and specs.

Bank Protection - The low flow channel design concept is for a channel with "soft" sides and bottom. However, RCC 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, RCC 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. The height of this protection (low flow channel depth + toe down) is 25 feet

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 of the Western Dam at Tempe Town Lake would be greatly attenuated. And since the flood wave attenuates much faster than that of a normal flood event, its downstream effect would be 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 cfs at Central Avenue. This is approximately equivalent to a 6-year 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.

E. Low Flow Channel Sediment Transport Analysis

A twenty-five year long-term historical flood hydrograph sediment transport simulation was used 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 flood frequency events. The simulations reflect the increase in the low flow channel bottom width between cross sections 214.65 and 215.65.

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 and has a moderate number of annual maintenance events.

The 25-, 50- and 100-year flood events increase scour depths upstream of the proposed grade control structure located at 214.65 by 1 to 2 feet. Deposition increases downstream of 214.65 and additional maintenance is required.

RIO SALADO PHOENIX

LOW FLOW CHANNEL - PHASE 1

HYDRAULIC ANALYSIS

FINAL SUBMITTAL

February 2000

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 19th 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).

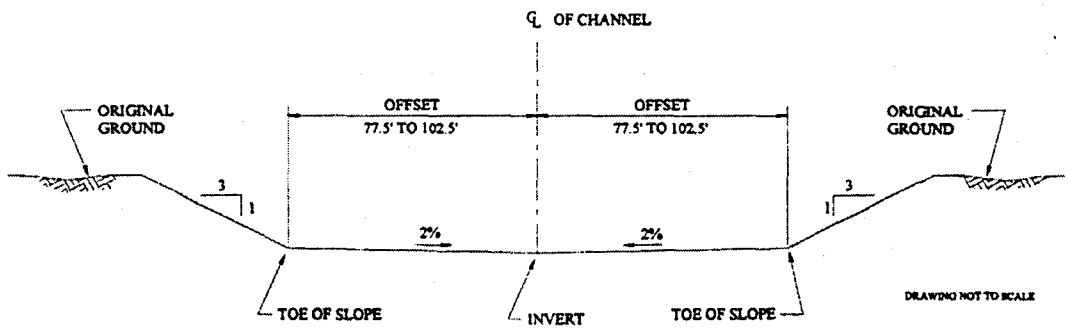


Figure 2-1. Typical cross section of the low flow channel.

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 7th 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)

ΔH = 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.

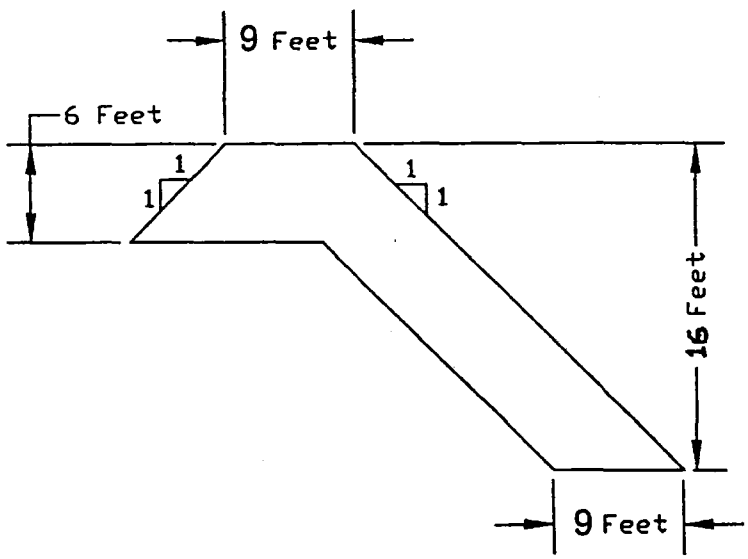


Figure 2-2. Typical cross section of soil cement grade control sill in overbank area, with a downstream toe-down depth of 16 feet. Within low flow channel, the toe-down depth should be 27 feet.

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.

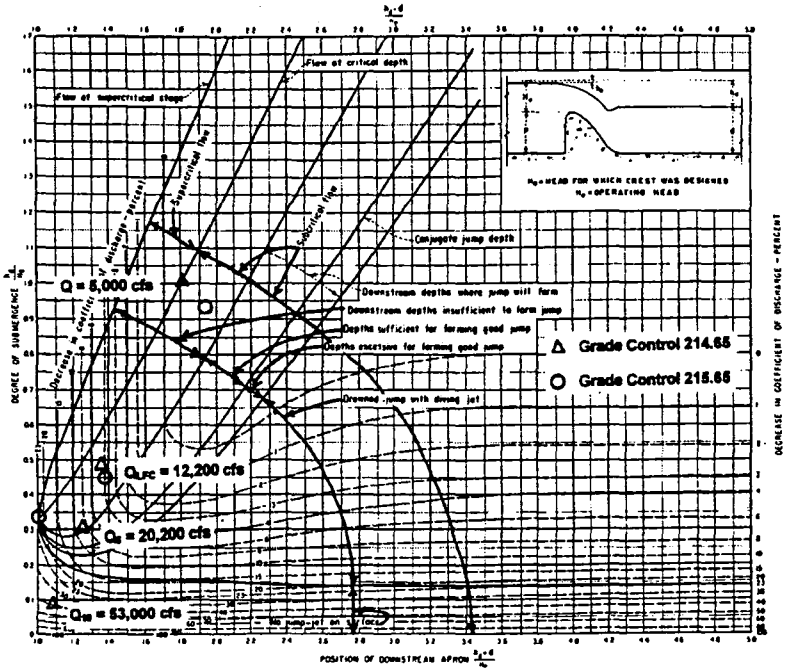


Figure 2-3. Plot of hydraulic drop-specific head ratio versus total head-specific head ratio used for evaluating grade control structure hydraulics (flow classification chart from USBR, 1977).

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.

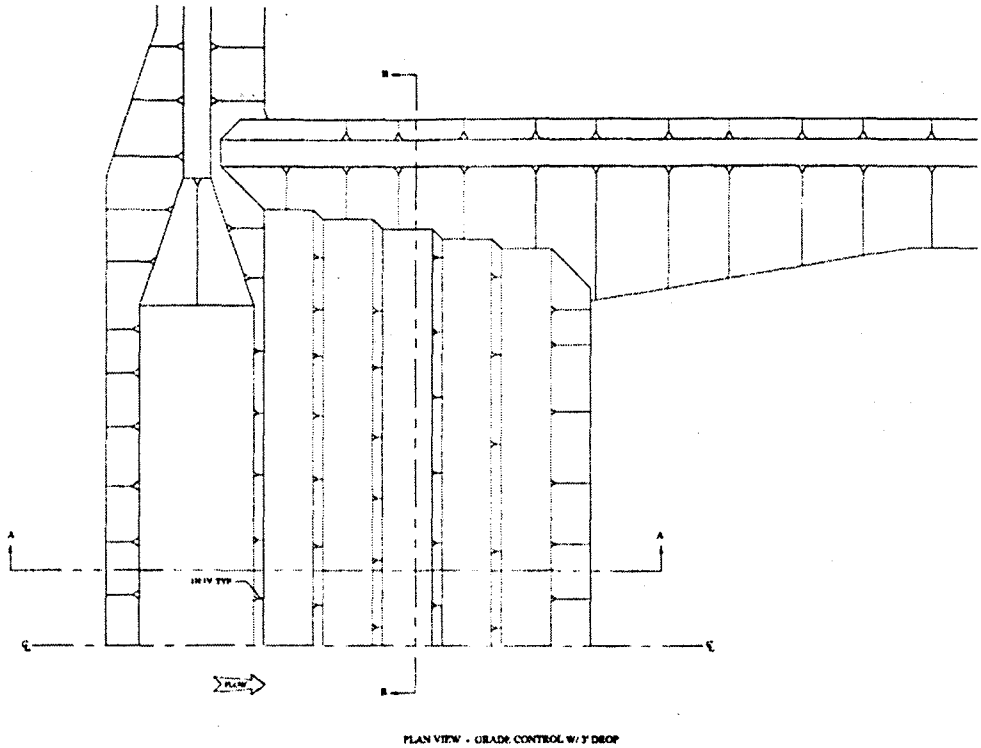


Figure 2-4. Typical plan view of grade control with 3 foot drop. Centerline of channel is at bottom of figure.

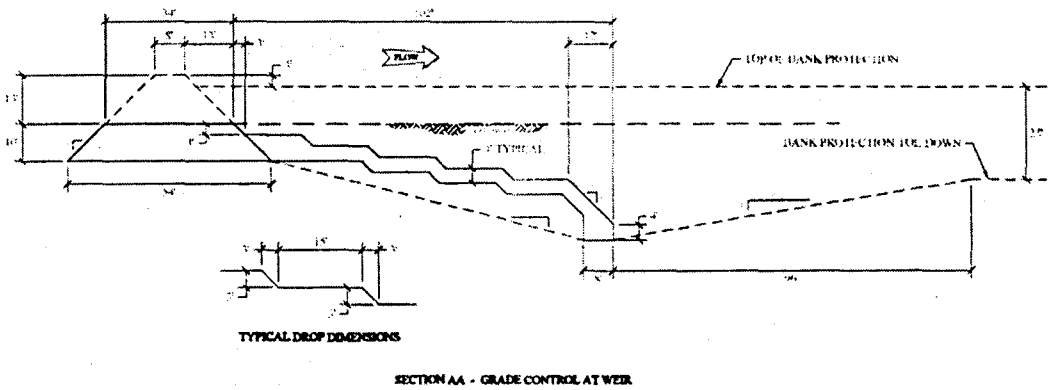


Figure 2-5. Cross section along low flow channel centerline of grade control structure with 3 foot drop. Soil cement apron is stepped to 10-year scour depth and then sloped at 1H:1V to the 100-year scour depth.

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.

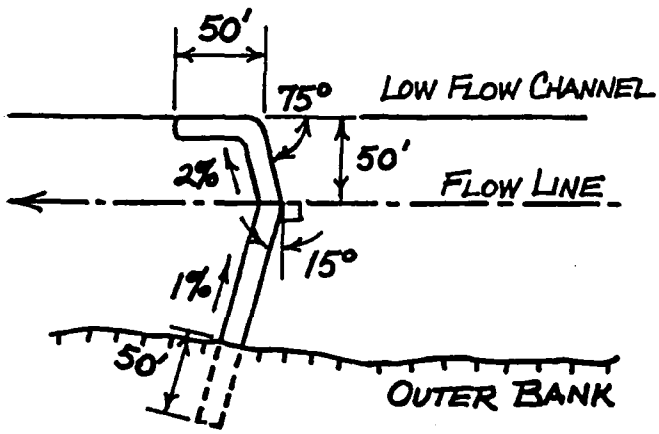


Figure 2-6. Typical plan view of originally proposed guide dike installation.

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.

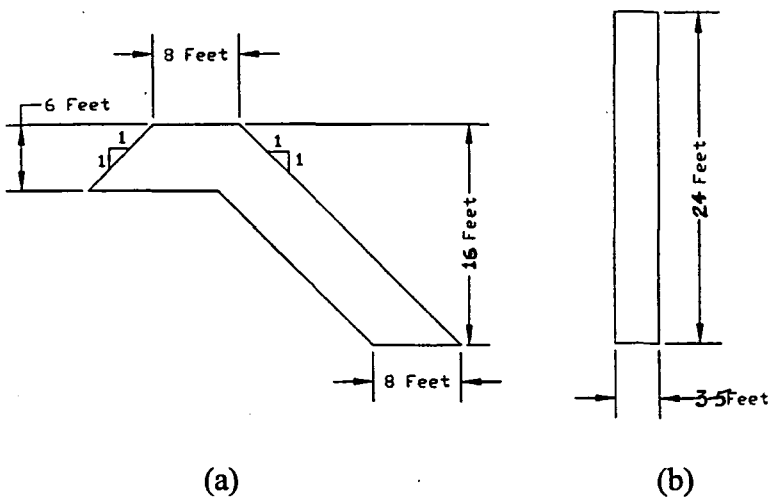


Figure 2-7. Typical guide dike cross sections for (a) soil cement construction and (b) slurry-wall construction.

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.

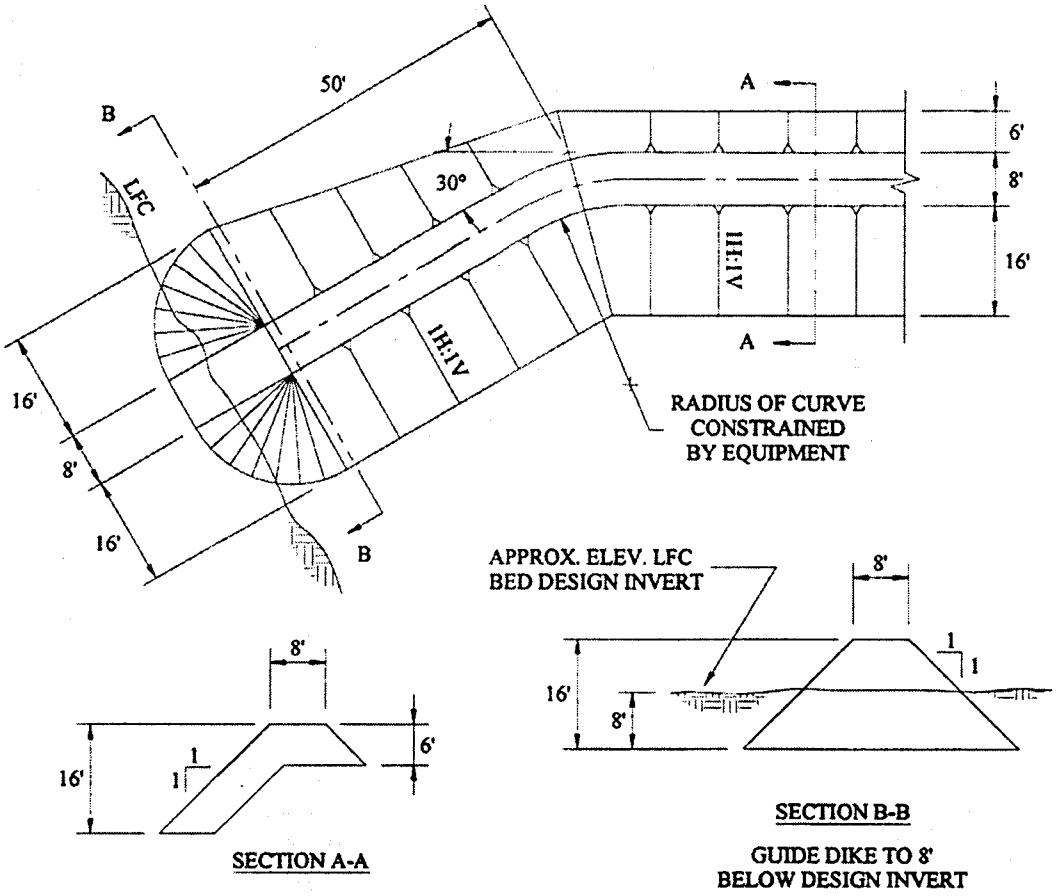


Figure 2-8. Plan view of revised guide dike design, end detail for low flow channel side.

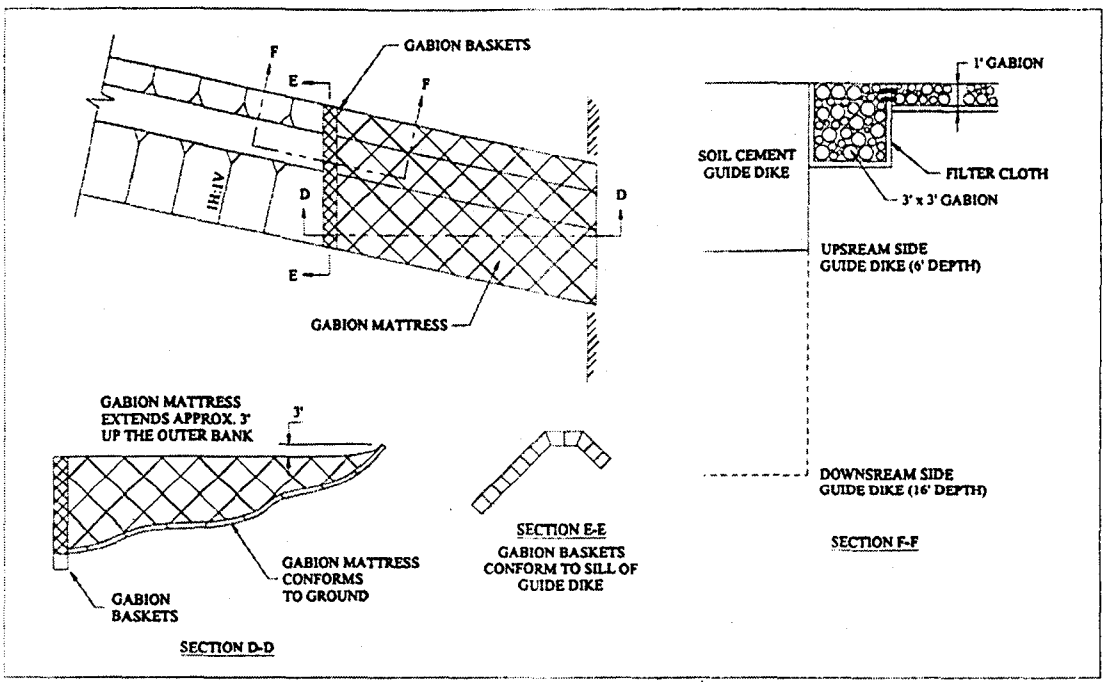


Figure 2-9. Plan view of revised guide dike design, end detail for transition from soil cemebt to gabions near outer bank.

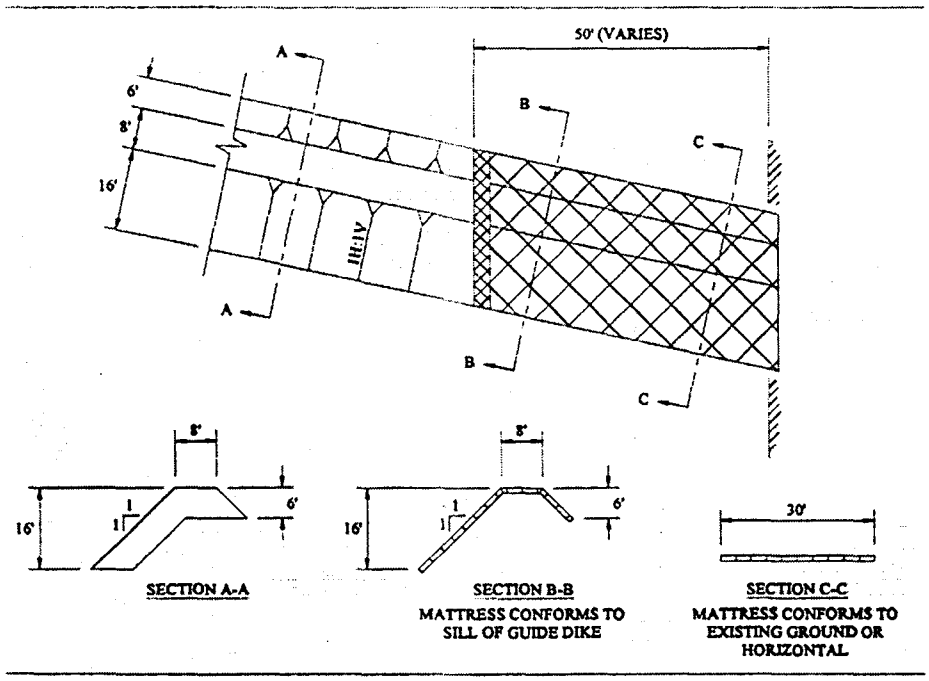


Figure 2-10. Plan view of revised guide dike design, end detail for layout of gabion matress near outer bank.

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 24th Street to the 3rd 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

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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 19th 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

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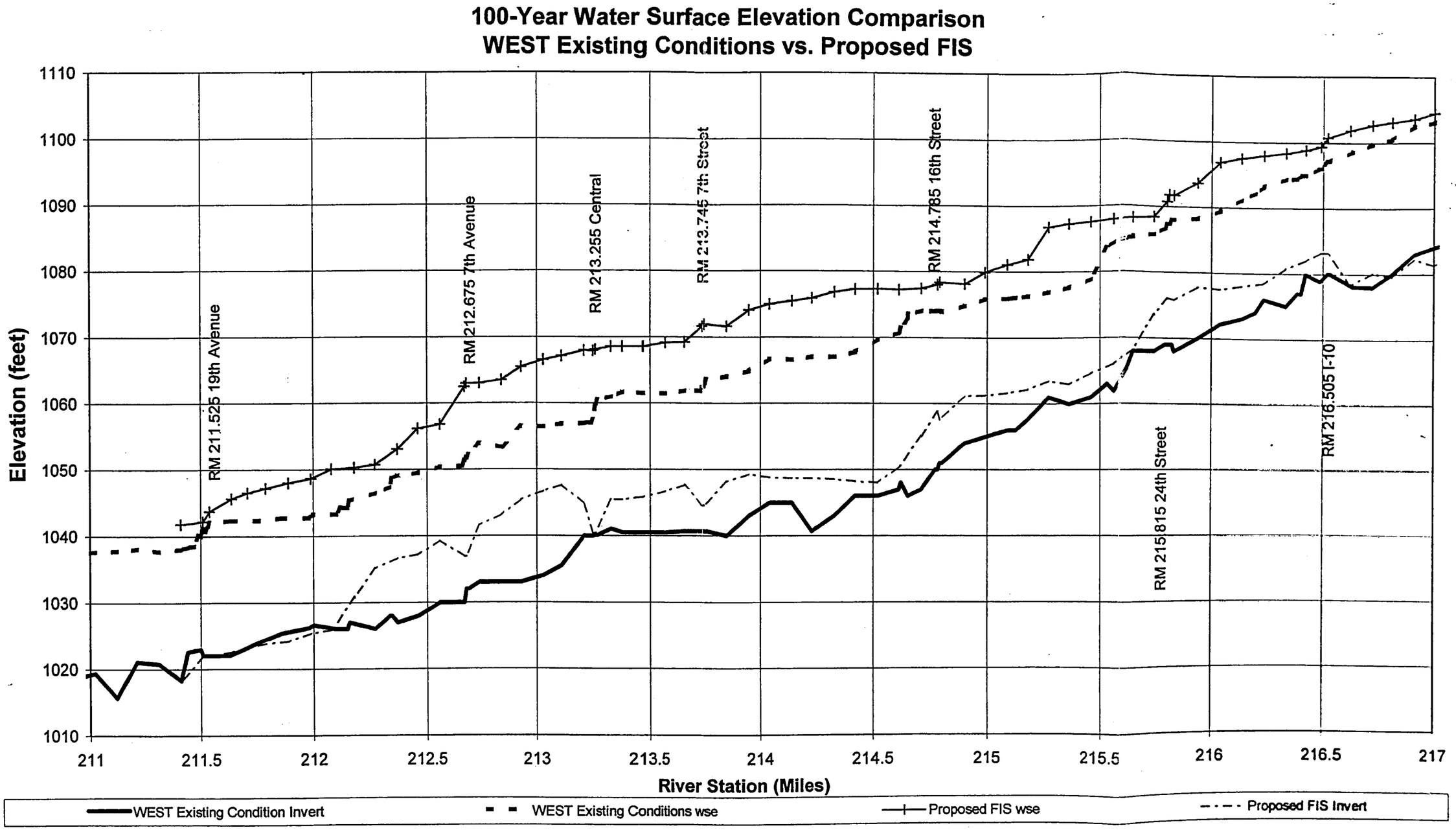

Figure 3-1. Comparison of WEST's existing condition HEC-RAS water surface profile to the proposed FIS water surface profile.

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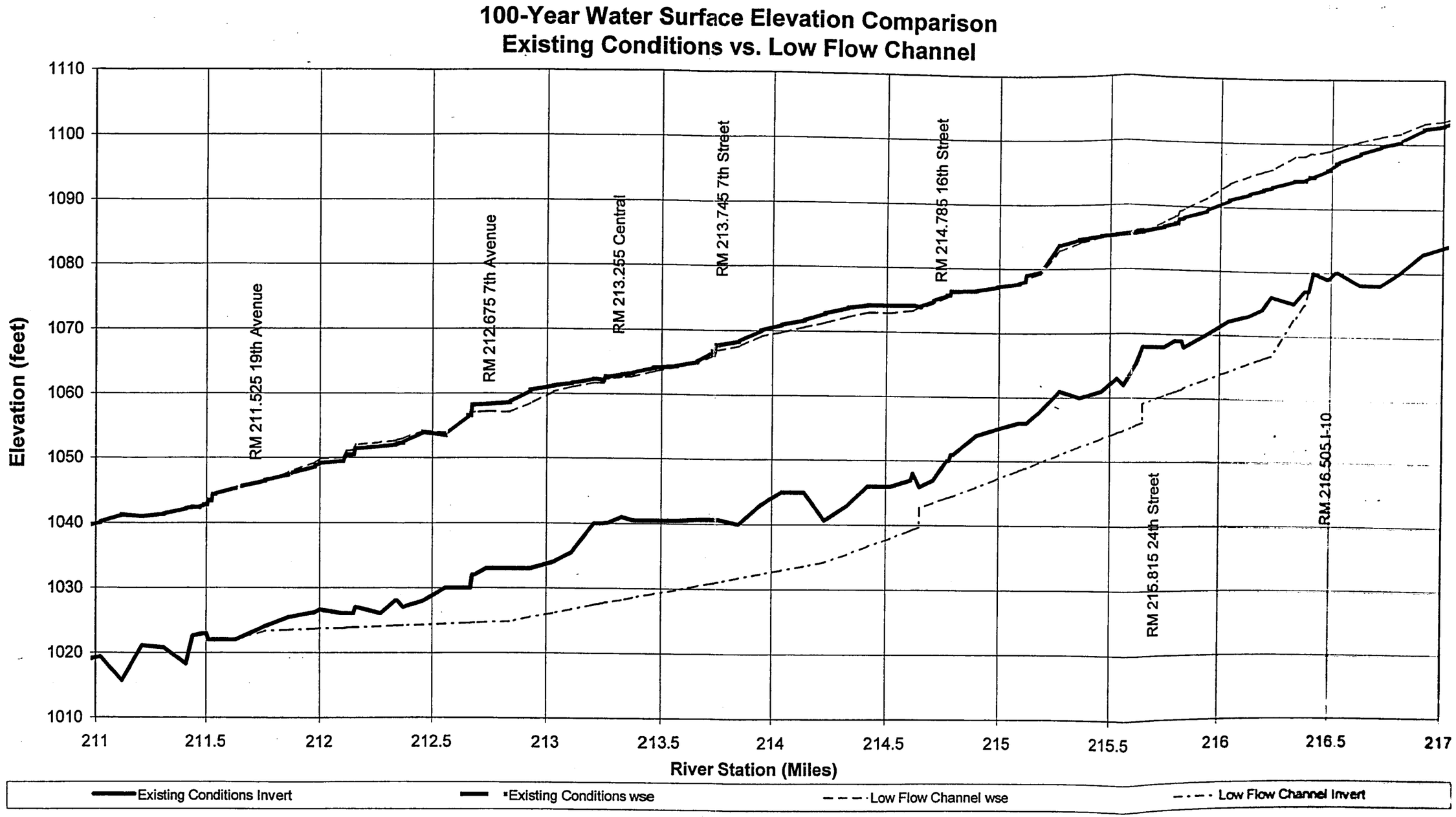

Figure 3-2. Comparison of WEST's existing condition and low flow channel water surface profiles.

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.

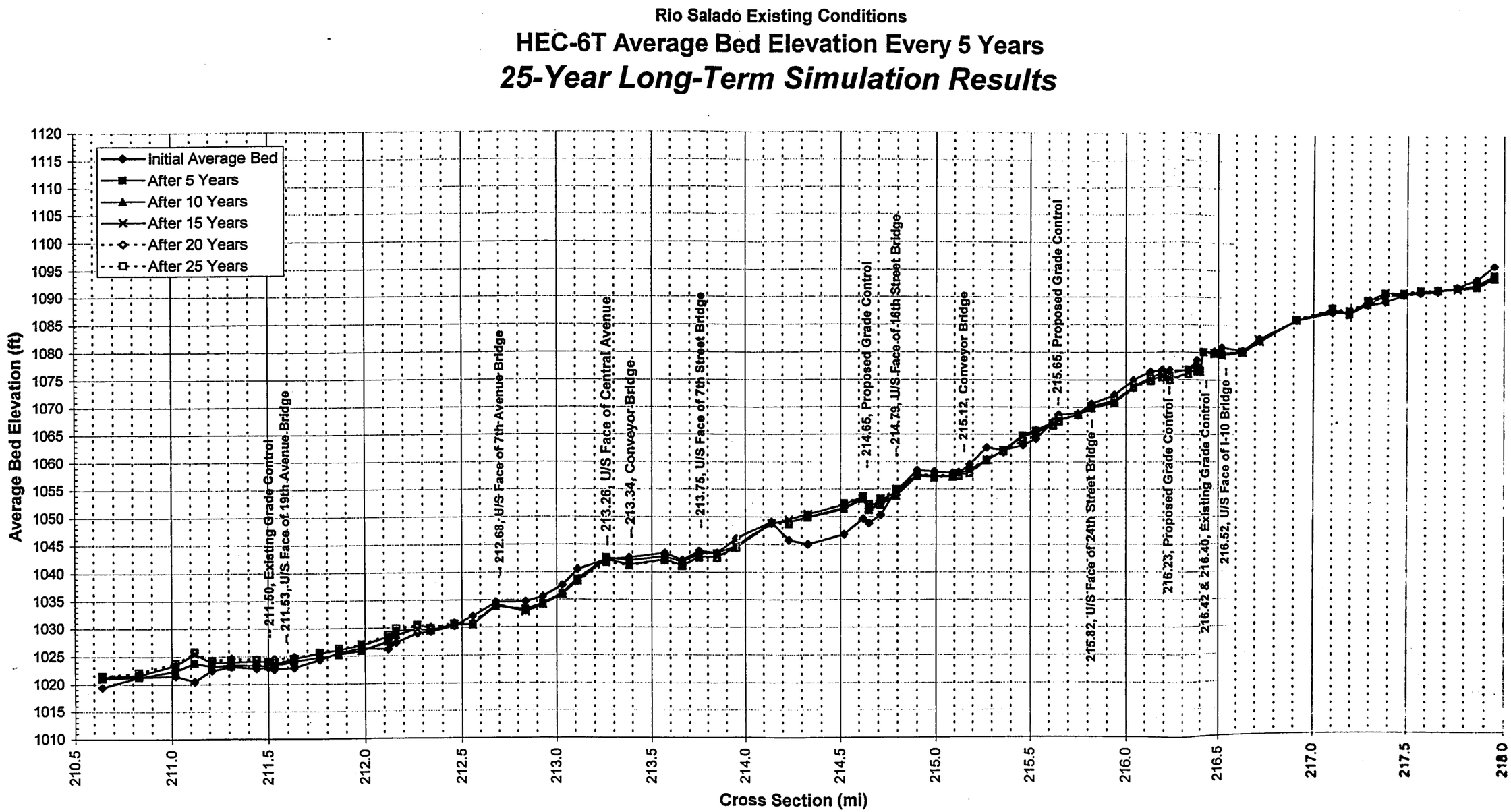

Figure 4-1. Average bed elevation at 5-year intervals for the existing conditions 25-year long-term simulation.

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).

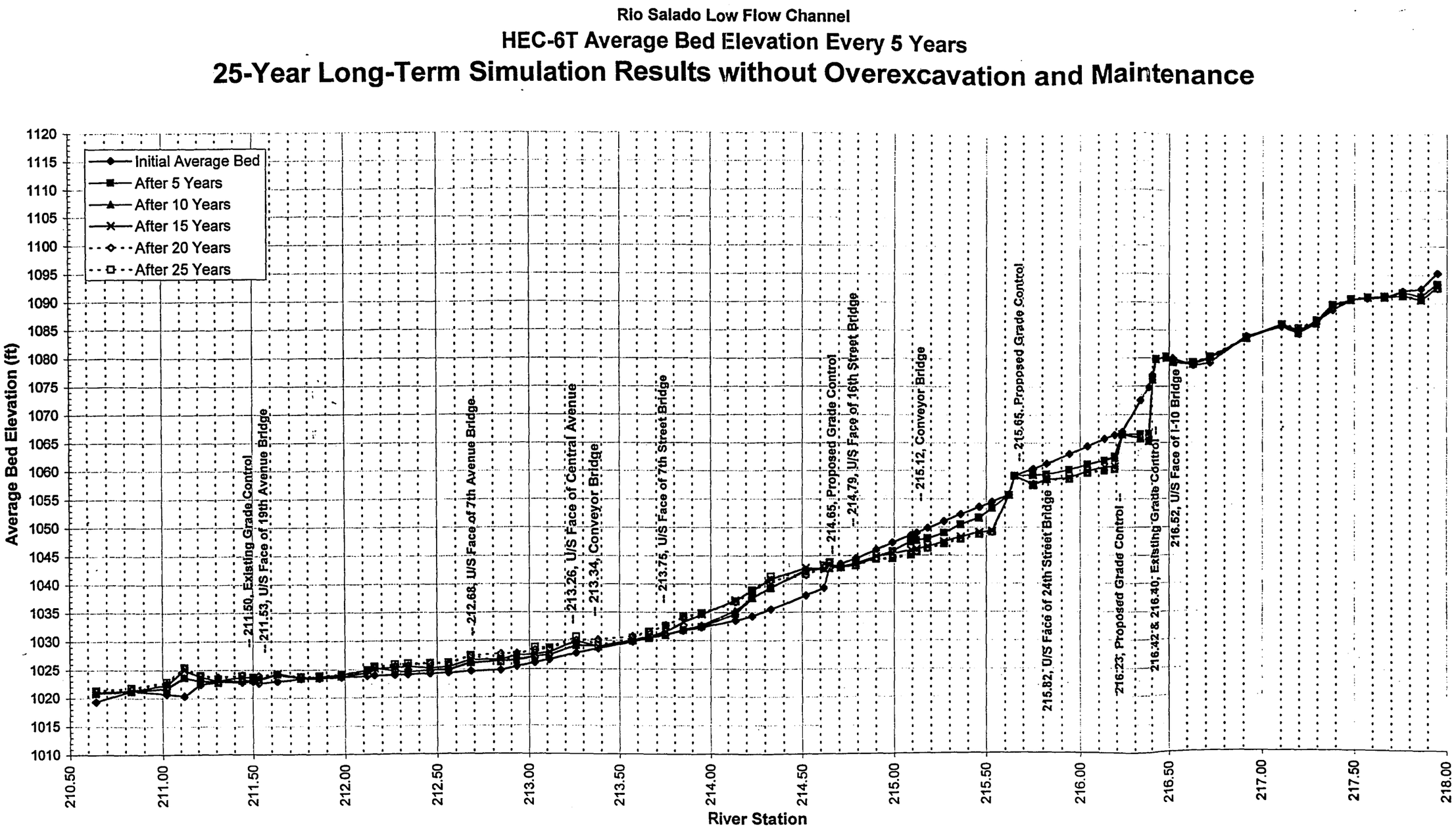

Figure 4-2. Average bed elevation at 5-year intervals for the low flow channel with the equilibrium inflowing sediment load and the four proposed grade control structures. Overexcavation and annual maintenance is not modeled.

Rio Salado Low Flow Channel

HEC-6T Average Bed Elevation Every 5 Years

25-Year Long-Term Simulation Results with 2-foot Overexcavation and Maintenance

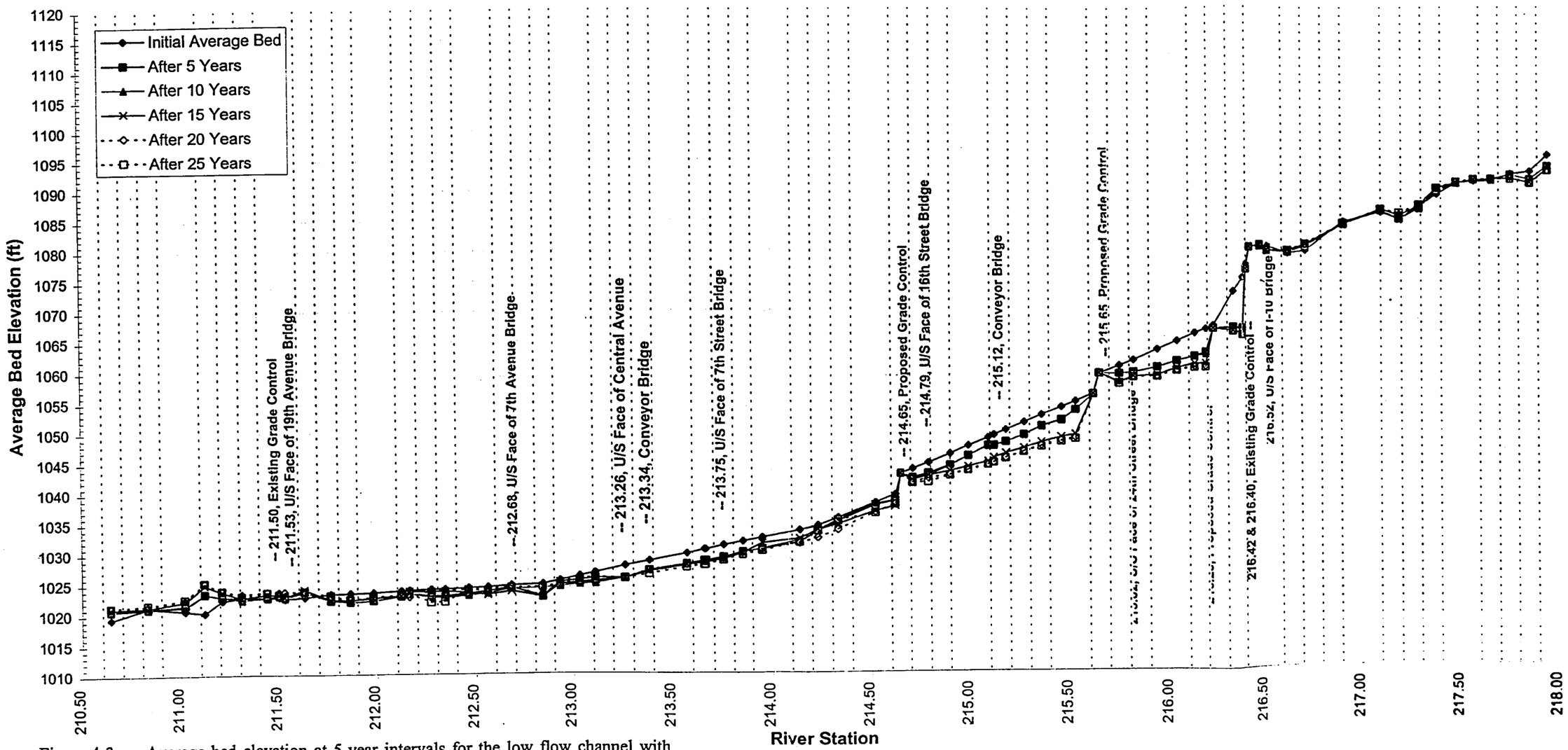

Figure 4-3. Average bed elevation at 5-year intervals for the low flow channel with the equilibrium inflowing sediment load, the four proposed grade control structures, and 2-foot overexcavation and annual maintenance.

Rio Salado Low Flow Channel

HEC-6T 25-Year Long-Term Simulation - Final Average Bed Elevations

Sensitivity to Inflowing Load with 2-Foot Overexcavation and Maintenance

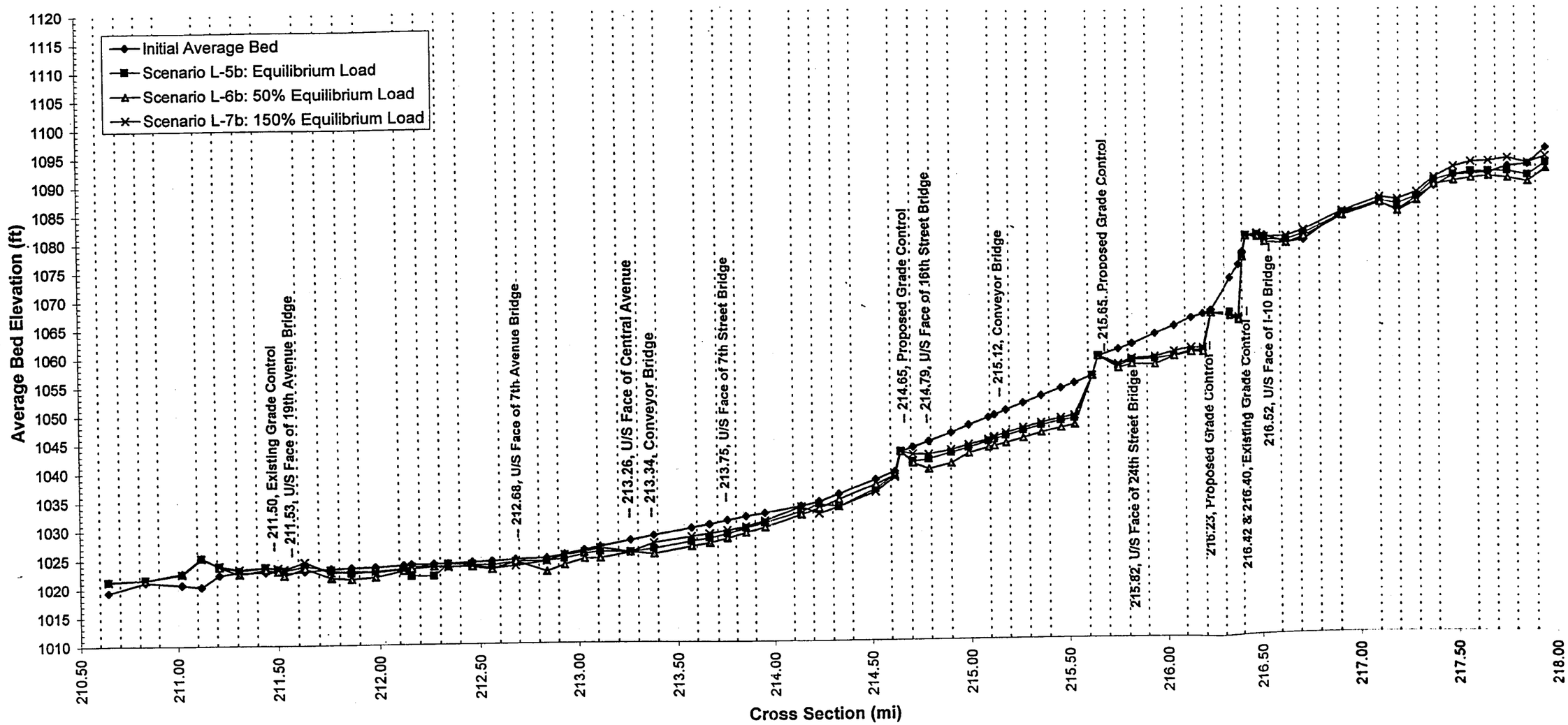

Figure 4-4. Sensitivity of the sediment transport results to the inflowing sediment load for the low flow channel with the four proposed grade control structures and 2-foot overexcavation and maintenance. A comparison of the average bed elevations at the end of the 25-year simulation period for 50%, 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 24th Street, the pile tips are approximately 35 feet below the scour elevation while at 16th Street, 7th Street, and 7th 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 24th 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)
24th 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
16th Street (214.785)	1051.00	0.00	19.03	**1031.97**	1055.00	Caisson	950.00	1057 (scaled)
7th 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
7th 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 19th 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.

RIO SALADO PHOENIX

LOW FLOW CHANNEL, PHASE 1

GEOTECHNICAL APPENDIX

FINAL SUBMITTAL

February 2000

RIO SALADO HABITAT RESTORATION PROJECT
MARICOPA COUNTY, ARIZONA

GEOTECHNICAL REPORT

US Army Corps of Engineers®

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS

February 2000

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I. Executive Summary

The geotechnical appendix herein covers the general existing geotechnical conditions and concerns of the project prior to construction of the Low Flow Channel. Section A of this appendix discusses the existing regional geology, geology and geotechnical properties of the foundation materials, ground water and hydrogeology, seismicity, large stone borrow sources and HTRW (Hazardous, Toxic and Radioactive Waste) issues for the project. The Low Flow Channel (LFC) for the entire project was explored in 1999 by the Corps of Engineers, Geotechnical Branch, Materials and Investigations Section, for the purpose of defining the types and gradation of native soil materials present in the existing channel in order to formulate a materials design for the LFC. From the exploration analysis, a Roller Compacted Concrete (RCC) mix design was chosen as the construction material of which the LFC will be composed of. The full details of the RCC mix and summary of the exploration are in Section B of this appendix. The Corps of Engineers, Geology and Investigations Section (Geotechnical Branch) installed three ground water monitoring wells along the Salt river banks, at the Central Avenue, 16th, 24th Streets bridge crossings, for the purpose of establishing static ground water levels and determining basic ground water quality along the river. These recent geotechnical investigations and other recent explorations and/or other most recent background information, since the writing of the final feasibility study, dated April 1998, are discussed more fully in detail throughout this appendix.

Section A

1.0 Regional Geology

The project area is in the Phoenix basin of the Salt River Valley. Metropolitan Phoenix is located geomorphically within the Gila Lowland Section of the Sonoran Desert Subprovince, a part of the Southern Basin and Range Physiographic Province. This province is characterized by broad, gently sloping, connected alluvial valleys (basins) bounded by moderately high northwest to southeast trending, rugged mountains (ranges). During late Miocene time (Tertiary period), the mountain ranges were extensively dissected, uplifted and down dropped by northwest to southwest and east to west trending sub-parallel normal faults (Reynolds 1985). An extensive amount of volcanic eruptions and activity accompanied the faulting. After late Miocene time and until the late Tertiary period, the ranges deeply eroded and filled their down dropped areas (basins) with sediments, which were later consolidated into sedimentary rocks. After the late Tertiary and until recent (Holocene) time, the basins, including the Salt River Valley, filled with unconsolidated and occasional semi-consolidated sediment (alluvium) eroded from the ranges. The thickest accumulations of Valley alluvium formed during the early to middle (approximately 1 million years ago) Quaternary period.

Today the alluvium of the Salt River Valley is in the final stages of development as evidenced by the numerous low-lying isolated hills (inselbergs), which project above the valley surfaces. These hills represent peaks of former mountain ranges that are now almost completely buried by alluvial material.

The mountain ranges that border the project area consist mostly of Tertiary age sedimentary and volcanic rocks that lie unconformably upon an ancient Precambrian igneous and metamorphic basement complex (AGS 1986). The complex is composed predominantly of igneous granite and diorite, metamorphosed schist, gneiss and volcanic rock. The Tertiary rocks are made up of volcanic basalt, andesite, rhyolite and sedimentary sandstone, siltstone and conglomerate.

The sediments within the Phoenix area consist primarily of Quaternary to Tertiary sediments that constitute the valley fill. They consist mostly of poorly to well consolidated (cemented) and unconsolidated gravel, sand, silt, and clay, representing several environments and ages of deposition. The total thickness of the alluvial materials range from near zero meters along the mountain fronts to 3,000 meters (9,840 feet) under the valley interior. These Quaternary

sediments as shown on figure 1, geologic map for the project (Arizona Bureau of Mines (ABM), 1957), is the only geologic unit that will be encountered during construction of the Phoenix project portion. The next section, describes in detail, the geotechncial properties of this Quaternary sediment and the appropriate geologic/geotechnical nomenclature that shall be in use for the rest of this appendix

2.0 Geology and Geotechnical Properties of the Salt River Bed Alluvium, Phoenix Project Area-

The Phoenix portion of the Rio Salado Habitat project extends a total of approximately 7.2 km (4.5 miles) along a stretch of the river, from west of the Interstate 10 bridge crossing to just west of the 19th Avenue bridge crossing. The Salt river flows west into the project area from the Superstition and Goldfield mountain ranges. The width of the Salt river bed (channel) ranges from approximately 61 to 243 km (200 to 800 feet) throughout the project area. The habitat project limits extend somewhat beyond the river bed and into the slopes along the channel. The slopes of the channel vary in height from 7.6 to 18.3 km (25 to 60 feet), as measured from the top of the existing river bed.

The predominant natural materials within the river bed are composed of Quaternary age river deposited sediment or alluvium, as previously mentioned, which is a part of the greater Salt River Valley Alluvium, a sequence of alluvial deposition within the entire Phoenix basin (figure 1). For the specific geotechnical purposes of this project and for convenience in nomenclature, the river bed materials, Salt River Valley alluvium, etc., are herein collectively referred to as the **Salt River Bed Alluvium** (figure 1). The upper 12 meters (40 feet) of the Salt River Bed Alluvium is the foundation material upon which the main project hydrologic engineering features LFC, Guide Dike Structures (GDS), etc.) are designed and constructed. The upper 12 meters of foundation material is considered as that measured from the river bed surface to approximately 12 meters (40 feet) depth.

2.0.1 Foundation Materials-

The upper 12 meters (40 feet) of the Salt River Bed Alluvium is the foundation material upon which the main project hydrologic engineering features (LFC, GDS, etc.) are designed and constructed. The upper 12 meters of foundation material is considered as that measured from the river bed surface to approximately 12 meters (40 feet) depth.

Overall, the Salt River Bed Alluvium within the Phoenix portion of the project consists of a general mixture of: approximately 460 meters (1,542 feet) (figure 1) of unconsolidated gravel and boulders, interbedded with irregular silt, sand and gravel lenses that become cemented gradually with depth. On a regional scale, the Salt River Bed Alluvium thickens towards the east and west of Tempe Butte gap, in the city of Tempe as shown in figures 3 and 4. At the gap, the Salt River Bed Alluvium averages less than 18 meters (60 feet) thick and in some places bedrock from Tempe Butte is exposed at the river bed surface. Hydrogeologically, the Salt River Bed Alluvium is divided into three distinct alluvial units named in depositional order, starting with youngest to oldest, as (ADWR, 1993):

A. Upper Alluvial Unit (UAU)- approximately 55 meters (180 feet) thick; the unit extends from river bed surface (0 meters and 0 feet reference) to approximately 55 meters (180 feet) depth; it is primarily a coarse soil (alluvium) which is composed of the following basic Unified Soil Classification (USCS) descriptions of sand (S), gravel (G) and cobbles, with small percentages of fines.

B. Middle Alluvial Unit (MAU) ("Middle Fine Unit" according to Dames & Moore, 1990)- approximately 91 meters (300 feet) thick; the unit extends from approximately 55 meters (180 feet) below river bed surface to approximately 146 meters (480 feet) depth; it consists mostly of fine grained soil (alluvium) which is composed of silts (M) and silty sands (SM), clayey silts (ML) and small amounts of gravel (G, a coarse soil).

C. Lower Alluvial Unit (LAU)- approximately 305 meters (1,000 feet) thick; the unit extends from approximately 146 to 451 meters (480 to 1,480 feet) below the river bed surface; it consists of a mixture of weakly to strongly cemented coarse and fine grained soils (alluvium). The coarse grained soils are composed of gravel (G) and boulders. The fine grained soils are composed of sand (S), sandy clay (SC), silty sand (SM)and interlayered beds of clay (C).

The local geology and general soils description of the Salt River Bed Alluvium is summarized in the stratigraphic profile shown on figure 4. This figure shows the UAU divided into 5 subunits, S, A_1, A_2, B, C (contacts are shown as dotted lines) and in contact with the underlying MAU (MFU), the contact is shown as a dark solid line. The UAU is exposed at the banks of the river and extends from this elevation to approximately 200 feet in depth.

In 1999, trenches were excavated by the Corps Geotechnical Branch, into the upper 5.2 meters (17 feet) of subunit A_1. Logs of the test trenches are shown on figures 9-13). According to the results of the field investigation and laboratory testing, the upper 5.2 meters can be described as a heterogeneous

soil stratigraphys, consisting of:

1.2 to 3 meter (4 to 10 feet) thick layers of light brown, loose, poorly graded gravel with sand (GP) containing approximately 25% cobbles and 5% boulders; 1.2 to 3 meter (4 to 10 feet) thick layers of light brown, loose, well graded gravel with sand (GW) containing approximately 25% cobbles and 5% boulders; 0.3 to.61 meter (1 to 2 feet) thick layers of light brown, very loose, poorly graded sand (SP); 0.3 to 0.61 meter (1 to 2 feet) thick, light brown, loose, poorly graded sand with silt (SP-SM). The general gradation for the river bottom according to the laboratory tests indicates that the percent by weight passing the 3-inch sieve ranges from 85 to 100, the percent by weight passing the No. 4 sieve ranges from 10 to 100, and the percent passing the No. 200 sieve ranges from 1 to 9.

All of the soil within the trenches were dry , except in the six test trenches (TT99-6, 7, 8, 12, 19, 22 and 33), where water was found at variable depths. The water is considered to be perched, except for TT99-19 and 22, for which static water level was encountered.

The trench log soil descriptions are fairly consistent with the composite drill log descriptions of subunit A_1 made by Dames & Moore from previous explorations as shown on the stratigraphic profile in figure 4. One important feature of note on the figure is subunit B, which is a fairly laterally continuous silty sand (SM) that acts as a confining layer within the UAU for most of the phoenix portion of the project area. This layer in turn behaves as a semi-confining layer on a regional hydrogeologic scale.

2.0.2 Excavation-

Analysis from the geotechnical exploration indicates that excavations in the various materials, as mentioned above, would be stable at cut slopes of 1 vertical on 1.5 horizontal for temporary slopes and 1 vertical on 2 horizontal for permanent cut slopes. The excess excavated materials can be disposed of in the various landfills in the project area.

2.0.3 Compacted Fill and Backfill-

The excavated material will be suitable for use as compacted fill and backfill. Materials for compacted fill and backfill will be obtained from suitable materials from channel excavation. A balance factor of approximately 0.9 can be expected for compacted fill when compacting the material to 90 percent of maximum density obtained by ASTM D 698. The compacted fill will be placed in

12-inch loose lifts and compacted to a minimum 90-percent maximum density (ASTM D 698). Backfill placed over the Grade Control Structures, and Guide Dikes will be compacted to 85 percent of maximum density obtained by ASTM D 698. The backfill over the Grade Control Structures, and Guide Dikes will be placed in 24 inch layers and compacted to a minimum 95 percent maximum density (ASTM D 698). The excavation will yield sufficient amounts of suitable materials for the compacted fill and backfill

3.0 Seismicity

3.0.1 Faulting-

Faults in central Arizona are generally short, discontinuous, normal faults, some of which have been interpreted to displace Quaternary formations. Most fall within the Jerome-Wasatch Structural Zone, a 75 km (46.5 mile) wide band which extends from Utah into Mexico. In Utah, the zone is associated with current earthquake activity and displays evidence of abundant Quaternary faulting. In Arizona, the zone includes the Main Street Fault in the northwest corner of the state and the Verde Fault located approximately 90 km north of the Rio Salado. Both faults are considered to be potentially active.

Nearest to the Phoenix portion, a zone (approximately 400 meters (1,312 feet) wide) of exposed, Tertiary age inactive normal faults, exists just north of Tempe Butte gap. The zone trends northwest to southeast and is located approximately 333 meters (1,092 feet) north-northwest of the edge of the Salt River and extends northwestward where it ends at a distance of approximately 4,400 meters (2.75 miles) from here. An east to west trending (approximately 1,760 meter (1.1 mile) long) Tertiary age fault lies concealed below the alluvium, in the middle of the Salt river, at Tempe Butte Gap.

3.0.2 Seismic Conditions-

An evaluation of the geologic and seismic conditions within a 162-km (101 mile) radius of the project area indicates that the proposed project is in an area of low seismicity as referenced in Zone 1 of the Seismic Zone Map of the Contiguous States (Uniform Building Code, UBC 1997). About 30 earthquakes with maximum epicentral intensities between II and VI on the Modified Mercalli Intensity Scale (MM) have occurred within a 162-km radius of the project area from 1870 through 1980. The seismic historical record for the last 124 years indicates that only one major damaging earthquake (1887 Sonora, Mexico) has occurred and was located outside the 162-km radius.

The historical 1887 7.2M Sonora, Mexico earthquake was located more than 411 km from Tempe, AZ, and expressed 50 kilometers (31 miles) of surface

rupture with 3 meters (9.8 feet) of normal displacement, causing rockfalls in the nearby prexposed bedrock hills of the phoenix basin. The most recent (1974) events, located about 24 km (38.6 miles) northeast of the project area, had recorded Richter magnitudes of only 2.5 and 3.0.

3.03 Project Design Earthquake-

The United States Geological Survey (USGS) probabilistic method for determining the peak ground acceleration (PGA) was chosen for this project. The life expectancy for the project was selected as 50 years = T. The PGA for the Operating Base Earthquake (OBE) and Maximum Design Earthquake (MDE) was calculated as directed by Corps of Engineer regulation (ER 1110-2-1806, 1995). The results of the calculations are as follows: For the OBE at 10% probability of excedence in 50 years, the PGA is 0.037% gravity (g). The MDE at 2% probability of excedence in 50 years is 0.077% g.

Definitions:

MDE- the maximum level of ground motion for which a structure is designed or evaluated. Performance requirement is not catastrophic failure. Severe damage or economic loss can be tolerated.

OBE- the earthquake that can reasonably be expected to occur during the service life of the project, with a 50% probability of exceedence during the service life, i.e. a return period of 144 years for a project with a service life of 100 years. The performance requirement is that the project function with little or no damage and without interruption of function.

4.0 Ground Water

The project area overlies portions of the principal aquifer within the Phoenix Basin that consists primarily of Quaternary and late Tertiary alluvium.

The Basin groundwater flow moves generally east to west, from the Salt River toward a major cone of depression near Luke Air Force Base, approximately 24 km (15 miles) west of Phoenix (Schumann, 1974). To a lesser extent, groundwater also flows in a northwestward direction toward a second cone of depression in the Deer Valley area.

Recharge to the groundwater basin is derived from seepage of irrigation waters, Salt river flows, rainfall, and underflow of groundwater. Recharge from streamflow and rainfall is minor, and the amount of recharge from irrigation seepage and underflow has not been high enough to offset progressive lowering of the water table.

Long-term groundwater withdrawal, since the 1940's, has resulted in a general decline in water levels from 67-100 meters (200-300 ft) throughout the Phoenix Basin. However, water-level declines have usually been less than 16.5 meters (50 ft) near the Salt River. The overall trend indicates a progressive decline in water levels westward from the project area toward Luke Air Force Base and northwestward toward Deer Valley.

4.0.1 Ground Water Levels and Ground Water Profile-

A ground water profile for the project was developed from compiling all existing ground water level data found closest to the river bed. This data was obtained from the following references: A. Existing ground water monitoring wells, including Corps installed monitoring wells RSMW-1 through RSMW-3 (the Corps monitoring wells were installed in Fall of 1999 and are in good condition- water level reading data was gathered from wells screened in the upper UAU, and designed to monitor the first encountered water level and it's fluctuations. B. Open gravel pits- water levels observed in open gravel pits as excavated along the river banks by sand and gravel operations. C. Test trenches- water levels observed in the test trenches from the 1999 Corps of Engineers geotechnical explorations for the project.

The ground water depth below the river bed surface varies from 5.5 to 11.5 meters (18 to 38 feet), (figure 5). From the ground water profile, it is anticipated that most of the Low Flow Channel construction, with the exception of the Grade Control Stuctures at Central Avenue bridge crossing and the three Drop Structures (DS) between 16th and 24th Street bridge crossings, will not occur within ground water. The perched ground water table is not continuous across the project, therefore the ground water profile only shows the static water table, as developed from the test trenches and wells in the project area.

4.0.2 Construction Dewatering-

The design drawings of the subgrade elevations for the Low Flow

Channel are shown to be above the elevation of the ground water profile in most

cases for the entire project. Therefore, dewatering is not anticipated during most of the construction of the Low Flow Channel, except within areas of phase 1 of the Phoenix portion, whereby the river bed is constantly saturated from nearby surface water drainage into the river . The LFC dewatering areas, according to engineering stationing, for the phase 1 portions, are as follows: near Central Avenue bridge crossing, approximately between 90+00 to 140+00 and near 7th Avenue, approximately between 50+00 to 75+00. It is also anticipated that sections of the Grade Control Structures and Drop Structures will need dewatering during construction.

The dewatering calculations for the GCS and LFC are given for the phase 1 construction portion of the Phoenix reach of the project only The calculations are based on the following formula (Driscoll 1987) that incorporates depth of the foundations below the water table, the dewatering well radius and the well penetration length into the water table:

$$Q = \frac{K(H^2 - h^2)}{1055 \log(R/r)} + 2\,\frac{x(K)(H^2 - h^2)}{2880(L_o)};$$

x = unit length of trench excavation in feet.
Q = discharge in gallons per minute (gpm).
K = hydraulic conductivity in gallons per day/ft^2.
H = saturated thickness of the aquifer before pumping in feet.
h = depth of water in the dewatering well while pumping in feet.
R = radius of the cone of depression in feet.
r = radius of the dewatering well in feet.
L_o = distance from point of greatest drawdown to point where there is no drawdown in feet.

For **GCS**:
Given assumptions for Phoenix Reach, Phase 1, for current ground water conditions that include a *static water table and no perched water:*
x = 800 feet (244 meters); K = 1,496 gpd/ft^2 (61.1 m^2/day); r = 0.5 feet (0.15 meters); H = 60 feet (18.3 meters); h = 20 feet (3 meters); R = 40 feet (12.2 meters); L_o = 100 feet (30.5 meters).

Q = 31,695 gallons per minute (120 cubic meters per minute) total.

Thus for a Q of 31,695 gpm across a 800' long X 100' wide (244 meter long X 30.5 meter wide) trench:

The minimum number of wells with a 0.15 meters (0.5 feet) radius needed to be installed to dewater the trench would be approximately 75, spaced a minimum of 3.3 meters (11 feet) apart. Each well would have to pump at least 1.6 cubic meters per minute (423 gallons per minute). In addition, each well would have to penetrate at least 6.1 meters (20 feet) below the bottom elevation of the GCS.

For **LFC**:
Given assumptions for Phoenix Reach, Phase 1, for current ground water conditions that include a *static water table and no perched water:*
x = 6,500 feet (1,982 meters); K = 1,496 gpd/ft^2 (61.1 m^2/day); r = 0.5 feet (0.15 meters); H = 40 feet (12.2 meters); h = 10 feet (3 meters); R = 40 feet (12.2 meters); L_o = 100 feet (30.5 meters).

Q = 101,292 gallons per minute (384 cubic meters per minute) total.

Thus for a Q of 101,292 gpm across a 6,500' long X 200' wide (1,982 meter long X 60.9 meter wide) trench:

The minimum number of wells with a 0.15 meters (0.5 feet) radius needed to be installed to dewater the trench would be approximately 200, spaced a minimum of 9.8 meters (32 feet) apart. Each well would have to pump at least 1.9 cubic meters per minute (506 gallons per minute). In addition, each well would have to penetrate at least 6.1 meters (20 feet) below the bottom elevation of the LFC.

The dewatering wells should be arranged along the perimeter of the total excavation area for the foundation preparation for the GCS or LFC so as not to interfere with the construction activities. As mentioned previously, the dewatering calculations take into account the presence of a static water level only and does not take into account perched water conditions. The dewatering operations should not be affected to a great deal if a perched water is encountered in dewatering during construction activities. It is anticipated that perched water should be withdrawn fairly quickly during dewatering startup activities and should shortly thereafter become a part of the general Q calculated for the dewatering wells.

4.0.3 Production Wells-

Six production wells are planned to be installed during sometime after phase two of the construction of the project, each well will be required to withdrawal a minimum of 1 million gallons per day. The water will ultimately be used to feed the habitat. One of the wells is proposed for installation during phase one of the construction project, once installed this well will provide temporary water for construction activities and will provide long term water for the habitat for the life of the project. The well will be located on the south side of the Salt river at the southwest side of the Central Avenue bridge crossing, very close to the existing Corps monitoring well number RSMW-1 (Rio Salado Monitoring Well-1) site, see figure 6. The well will be named RSPW-1 (Rio Salado Production Well-1). The well shall be carefully drilled to a depth of approximately 190 feet from ground surface (the river bank elevation of approximately 322 meters (1,060 feet) above mean sea level, or to the top of subunit B, such that it does not penetrate below the subunit B layer, a confining layer as previously mentioned. The project goals are to limit the withdrawal of water from all of the production wells to the upper portions [approximately 323 meters (190 feet)] of the UAU.

4.0.4 Hydrogeology-

Ground water at the Rio Salado project site occurs primarily within three major units that are bounded below by impermeable Tertiary and Precambrian basement rocks (USEPA 1991). A north looking conceptual regional hydrogeologic cross section (profile) of the Upper Alluvial Unit (UAU), Middle Alluvial Unit (MAU) and Lower Alluvial Unit (LAU) is seen in figure 3 (ADWR 1993). The amount of storage and flow within the units varies considerably with area and depth (USEPA 1993). The four hydrogeologic units are derived from Phoenix Basin alluvial materials. The UAU is the only unit of concern for this project, since excavation during construction is anticipated to occur at a maximum of approximately 40 feet below the river bed surface. In addition, ground water wells for use during construction and project implementation will only be installed within the UAU (to a maximum of approximately 58 meters (190 feet) depth below the river bank ground surface. The units are described in the following tables (their age of deposition increasing with descending order), (ADWR 1993):

UAU (Upper Alluvial Unit)

The base of this unit occurs atop the bedrock of Tempe Butte at approximately 18 meters (60 feet) below the Salt river bed surface at Tempe and approximately 61 meters (200 feet) below ground surface at Phoenix (figure 3). The unit was formed during the final stages of alluvial development of the Phoenix Basin, approximately late Pleistocene to recent (Holocene, last 10,00 years before present) time. The unit lithology (USCS) consists of unconsolidated sand (S), gravel (G), cobble and boulders with local thin interlayered beds of clay (C) and silt (M). The entire unit is an unconfined aquifer that is both saturated and unsaturated and exhibits the following aquifer characteristics (USEPA 1990):

Hydraulic Conductivity (K) - The K within this unit at Phoenix is approximately 8.20 meters per day (200 gallons per day per foot per foot), (Dames & Moore 1991).

Aquifer Thickness - The saturated aquifer thickness of this unit is approximately 49 meters (160 feet) at Phoenix (Dames & Moore, 1990).

Water Level (water level data as measured from approximately 1990 to 1999, from monitoring wells closest to river bed, including the three 1999 Corps installed monitoring wells RSMW-1 through RSMW-3, test pits from the Corps 1999 geotechnical exploration and from open water surfaces in gravel pits along the river bed) - The current water levels in this unit measure approximately 5.5 to 11.5 meters (18 to 38 feet) below the Salt river bed surface at Phoenix. Ground water levels at Phoenix fluctuate between 7 to 10 meters (23 to 33 feet) during both discharge and recharge events, but rise 0.23 to 0.43 meters (3/4 to 1.5 feet) per day during recharge from flood events (Dames & Moore 1991). The current water levels are declining and represent a discharge event, in direct response from the 1993 flooding at the Salt river.

Aquifer Production - Approximately 25% of the ground water pumpage in the Phoenix basin is directed towards this unit (ADWR 1993). A very large portion of the ground water from the UAU is used for agriculture. Little or none of the water is used for drinking water purposes (Wilson 1991).

MAU (Middle Alluvial Unit)

This unit underlies the UAU and is in contact with the Lower Alluvial Unit (LAU) at approximately 153 meters (500 feet) below river bed surface at Phoenix (figure 3). This unit was formed during the middle stages of alluvial development of the Phoenix Basin, approximately late Tertiary to late Pleistocene time. Unit lithology consists of weakly cemented, interlayered beds of clay (C), silt (M), sand (S) and gravel (G). This unit is a saturated, unconfined aquifer, although it contains layers of aquitards. It exhibits the following aquifer characteristics (USEPA 1993):

Hydraulic Conductivity (K) - The K within this unit is approximately 1 to 10 meters/day (24.5 to 245 gallons per day/ft/ft) within the Phoenix Basin.

Aquifer Thickness - The thickness of this unit is approximately 91 meters (300 feet) at Phoenix.

Semi-Confining Layer - This unit is generally comprised of more than several discontinuous semi-confining layers that consist predominantly of silt and clay.

Aquifer Production - Approximately 50% of the ground water pumpage in the Phoenix basin is directed towards this unit. A large portion of the ground water is used for agriculture. A smaller portion of the ground water is used for drinking water purposes (Wilson 1991).

LAU (Lower Alluvial Unit)

This unit underlies the MAU and is estimated to be at least eight thousand meters (thousands of feet) thick within the Phoenix Basin. This unit was formed during the early stages of alluvial development of the Phoenix Basin, approximately late to middle Tertiary time. The unit lithology consists of weakly to strongly cemented gravel (G), boulders, sand (S), sandy clay (SC), silty sand (SM) and interlayered beds of clay (C). This unit is a saturated, unconfined aquifer that contains layers of aquitards. The LAU exhibits the following aquifer characteristics (USEPA 1993):

Hydraulic Conductivity (K) - The K within this unit is higher than the MAU and averages approximately 1 to 25 meters/day (5 to 60 gallons per day/ft/ft) within the Phoenix Basin.

Aquifer Thickness - The thickness of this unit is unknown.

Semi-Confining Layer - This unit is generally comprised of more than several discontinuous semi-confining layers that consist predominantly of clay and mudstone (a silty clay or clayey silt).

Aquifer Production - Approximately 25% of the ground water pumpage in the Phoenix basin is directed towards this unit. A large portion of the ground water is used for agriculture. A smaller portion of the ground water is used for drinking water purposes (Wilson 1991).

Ground water movement and connection within all three of the upper alluvial units is mostly lateral and somewhat vertical. Vertical ground water flow occurs through a combination of leakage through all three unit geologic contacts and through water wells that extend vertically across more than one unit, but is more prevalent in Tempe, where a steeper vertical ground water gradient exists. Although there are distinct, impermeable layers (perched layers included) in some of the three aquifers, there is a definite natural geologic connection between them at a regional scale, in this regard all three aquifers can be visualized as combined and interconnected hydrogeologically and therefore the Phoenix Basin can be recognized as having one unconfined aquifer.

4.0.5 HTRW (Hazardous, Toxic and Radioactive Waste) Contamination to Ground Water

At present, nearly all of the HTRW contamination to the ground water within or near the project has been attributed to floating and sinking Volatile Organic Carbons (VOCs) leaching into the ground water (ADEQ and EPA 1987-1997). VOC leaching has occurred from either mismanaged storage, pumping into ground water and/or improper dumping of VOC and related chemical compounds at Superfund sites located within or near the project boundaries. VOCs have been detected within the UAU and MAU, but not the LAU. There is no direct evidence that surface water recharge from the Salt River from flooding or normal releases has contaminated the three alluvial aquifers with Hazardous and Toxic Waste (HTW) unless such recharge has been associated with the Superfund sites and/or other recognized HTRW sties.

4.0.6 Ground Water Monitoring Wells-

The feasibility study recommended that a series of twelve monitoring wells be installed and sampled in order to determine the presence, migration and impact of VOC and/or other ground water contaminants to the entire project. In a general sense, three of the wells will be used to determine the immediate HTRW impacts to ground water at the chosen production well locations, the other nine wells will serve as sentry wells to monitor the migration of HTRW contaminants in ground water to the project. Eventually, data from all twelve wells will be used to ultimately determine if wellhead treatment should be designed for the production wells.

As previously mentioned, in the Fall of 1999 the Corps installed three ground water monitoring wells, RSMW-1 through RSMW-3. These wells are in good condition and are strategically located close to the proposed production well locations. Ground water samples collected from these three wells will be analyzed to determine the presence of HTRW contamination at the production well locations and the magnitude, type of contamination, etc., if any, will compared to existing Applicable or Relevant and Appropriate Requirements (ARARs) (ground water cleanup standards set for the project) to determine if wellhead treatment should be designed for the production wells. Ground water test results from these three wells for HTRW constituents were non-detect. The non-detect results indicate that ground water quality is good and may not have any detrimental effects for use during construction activities.

Nine monitoring wells remain to be installed in order to complete the monitoring well program, four of these wells are contracted to be installed in

December 1999 and sampled in January 2000. The last series of wells are scheduled for installation after January 2000. The installation and sampling is being performed for the Corps by the Phoenix offices of Dames & Moore. The final decision on wellhead treatment for the production wells will be made after all the data from all twelve ground water monitoring wells is analyzed.

The installation of all ground water monitoring wells will be limited to the upper 190 feet of the UAU or the top of subunit B, whichever is encountered first during drilling. The wells will be screened at the top 60 feet or so of the water table, separated by blank casing and then screened again at the bottom 60 feet or so of the well. Isolated ground water samples will be withdrawn from the two screened intervals through the use of a downhole inflatable packer, see figure 7. The screen separations are designed so that the differences, if any, in contaminant concentrations at variable depths within the unconfined aquifer (upper UAU) can be determined.

5.0 HTRW Contamination to Soils-

No HTRW contamination to soils was detected in the 1999 Corps geotechnical exploration, except for one of the trenches that contained small amounts of trash. The trash was not characterized, i.e., it was not determined whether the trash composition contained HTRW components, since this trench was abandoned shortly thereafter. Non-HTRW contamination was detected in the project area from the subsequent HTRW explorations by the City of Phoenix (COP) as part of the Phase I and II Environmental Investigations for the project. The contamination was found along the river banks, atop the slopes, primarily in the phase two portion of the project, near the Estes landfill. The contamination consisted of non-hazardous and non-toxic municipal trash, inert construction debris and rubber tires. This type of contamination is man-induced and is a structured or engineered fill type of dumping activity, i.e., dumping that occurs within permitted or engineered landfills, i.e., it is considered a regulated waste. From these findings, it is anticipated that most of the contamination is confined to the either the river bank slopes and/or atop the river bank, however, additional contamination may be present within and throughout the river bed along the entire project. The contamination in the river bed is anticipated to contain scattered piles or areas of municipal trash, construction debri and rubber tires. This type of contamination is man-induced and often sporadic and is considered an unregulated type of dumping activity, i.e., an unregulated waste.

It is anticipated that contamination in soils during construction of the project will be limited to mostly Municipal Solid Waste (MSW). The MSW should

be a solid or semi-solid and can be further described according to the following common criteria as referred to in the waste industry: (the following descriptions/definitions of MSW may differ from those referenced in Arizona statutes.

A. Large percentages of construction waste that consists of wood, metal, cardboard, concrete, brick, dirt, sand, gravel and cobbles. This type of MSW is considered non-HTRW contaminated.

B. Large percentages of commercial and residential waste consisting of rubber tires, i.e., a "special waste" that may occur within landfills and the river bed. This type of MSW is considered non-HTRW contaminated.

D. Small amounts of commercial waste consisting of paper, cardboard, plastics, wood, organic food wastes, glass, metals, fabrics, *special wastes and **hazardous wastes.

E. Small amounts of residential waste consisting of paper, cardboard, plastics, wood, organic food wastes, glass, metals, fabrics, *special wastes and ***household hazardous wastes, furniture, appliances, car bodies and auto parts and yard wastes.

*Special Waste- a waste that is collected separately and recycled, i.e., used oil, batteries, household cleaners and tires, etc.

**Hazardous Waste- a waste that is disposed of at a hazardous waste disposal or recycling facility, it meets the Code of Federal Regulations (CFR) and Environmental Protection Agency (EPA) definitions of hazardous, in that it is either ignitable, flammable, corrosive and/or toxic.

***Household Hazardous Waste- a waste that meets the CFR and EPA definitions as a hazardous waste and is disposed of at a hazardous waste recycling or disposal facility.

As part of construction plans and specifications, the COP will provide a Health and Safety Plan (HSP) that will address all the health and safety issues due to possible soil contamination to the construction workers and visitors to the site during construction. The HSP and also the construction specifications will include provisions for characterizing and removing MSW during construction. In summary, the HSP and construction specifications direct the Contractor to do the following: a. Hire a qualified health and safety specialist (HSP) and/or industrial hygienist (IH) to provide oversight during construction. b. The HSP or IH shall stop all construction activities once MSW is encountered and if obvious, identify

the waste as non-hazardous and then remove and dispose of as non-hazardous MSW. c. If not obvious, the Contractor shall contact the COP hazardous waste Contractor who will then characterize the waste and if hazardous, will remove and dispose of it.

6.0 Subsidence

Subsidence measurements from 1974 suggest that subsidence in the project area has not occurred. Ground failure in the form of subsidence and earth-fissures has occurred in other areas of the Phoenix Basin. The closest ground failure occurrences to the project area are near Luke Air Force Base, approximately 18 miles from the site, where 1/2 to 3 feet of subsidence has been measured and exhibits the shape of a 2 mile diameter "bowl" depression.

Earth-fissures and subsidence are produced by groundwater (pumping) withdrawal, which causes aquifer compaction, whereby ground (soil) compresses (subsides) because it has lost the support of water within its pores. Earth-fissures develop when the soil subsides differentially and pulls apart.

The Phoenix area will continue to be affected by subsidence because of groundwater overdraft, principally where ground water withdrawal is most severe.

7.0 Stone Sources

Two stone borrow sites have been identified as sources of construction material and are available for use, in the event an engineering design is proposed for the Rio Salado project. The two stone quarries are less than 10 miles from the site and have produced stone for previous Corps flood control projects at the Arizona Diversion Canal and Indian Bend Wash areas. Stone from both quarries exhibit a good service record and passed all rock quality compliance tests. The quarries are listed as:

<u>Sunstate Rock and Materials</u> and
-located 20th St. and E. Beardsley Rd, Phx, AZ.
-passed rock 1990 quality tests.
-passed 1994 visual inspection.
-produces granite.

<u>Salt River Sand & Rock</u>
-located at Dobson & McKellips Rds, Phx, AZ.
-passed 1994 rock quality tests.
-passed 1994 visual inspection.
-produces green schist.

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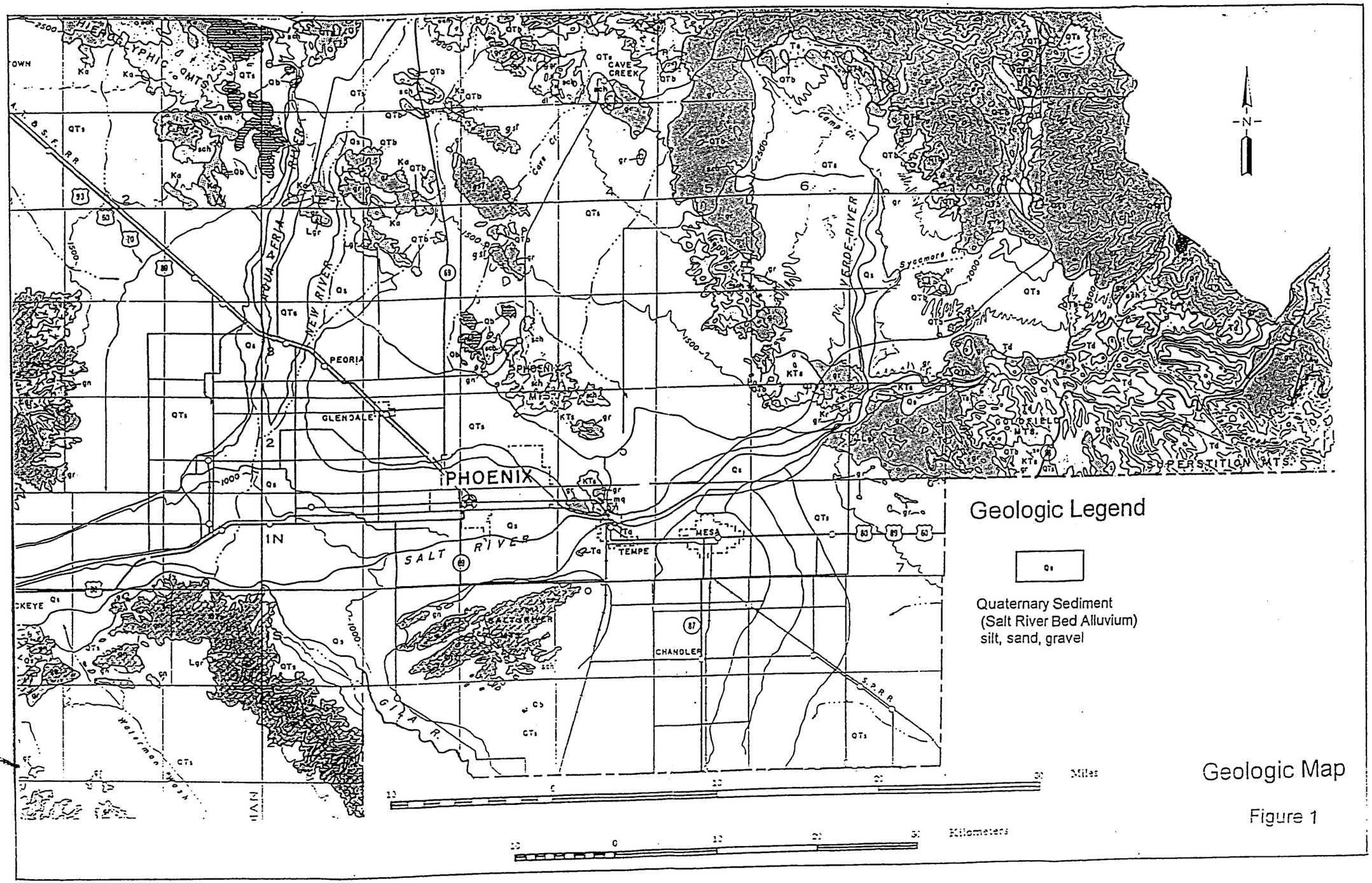


Geologic Map

Figure 1

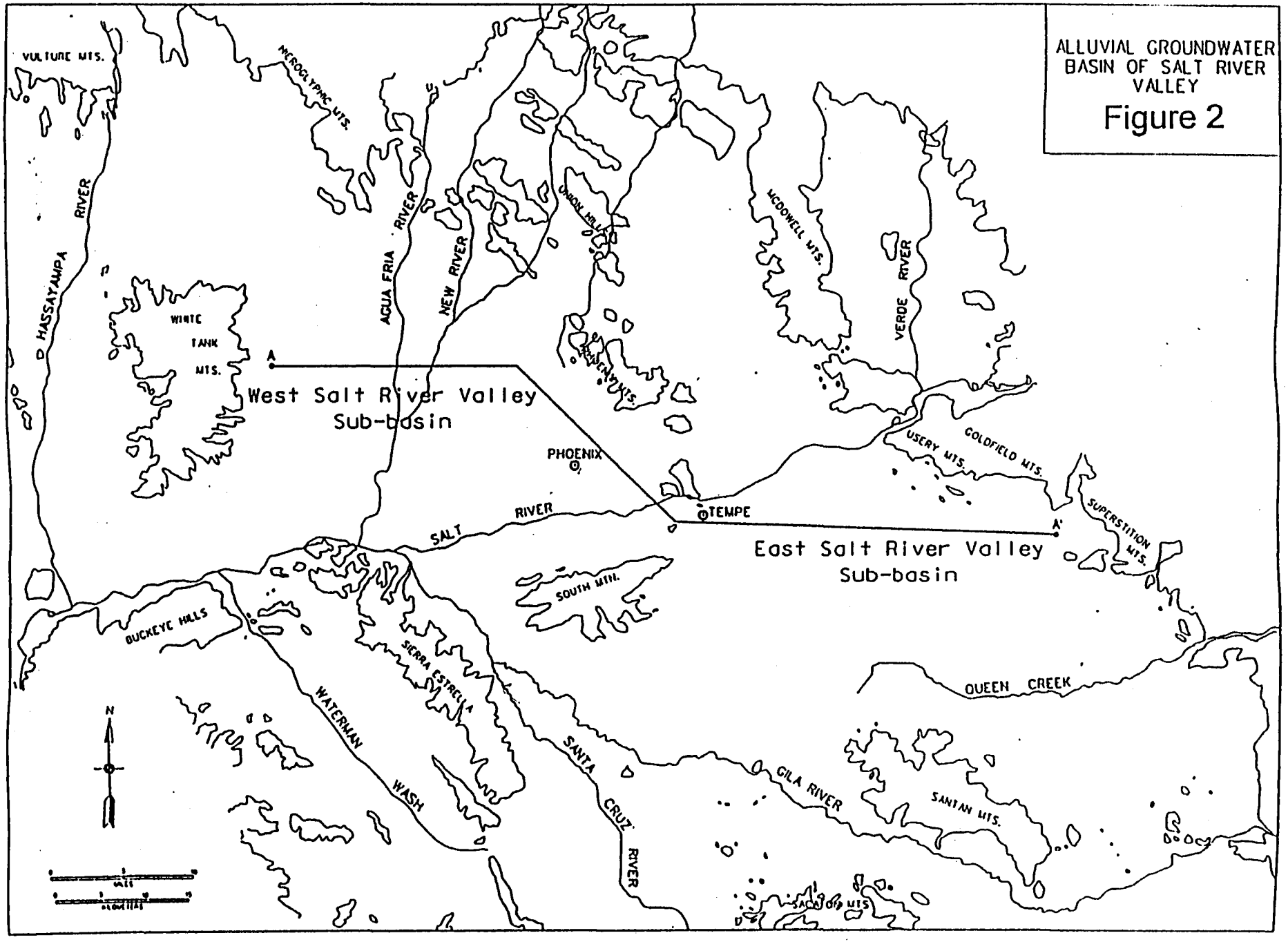
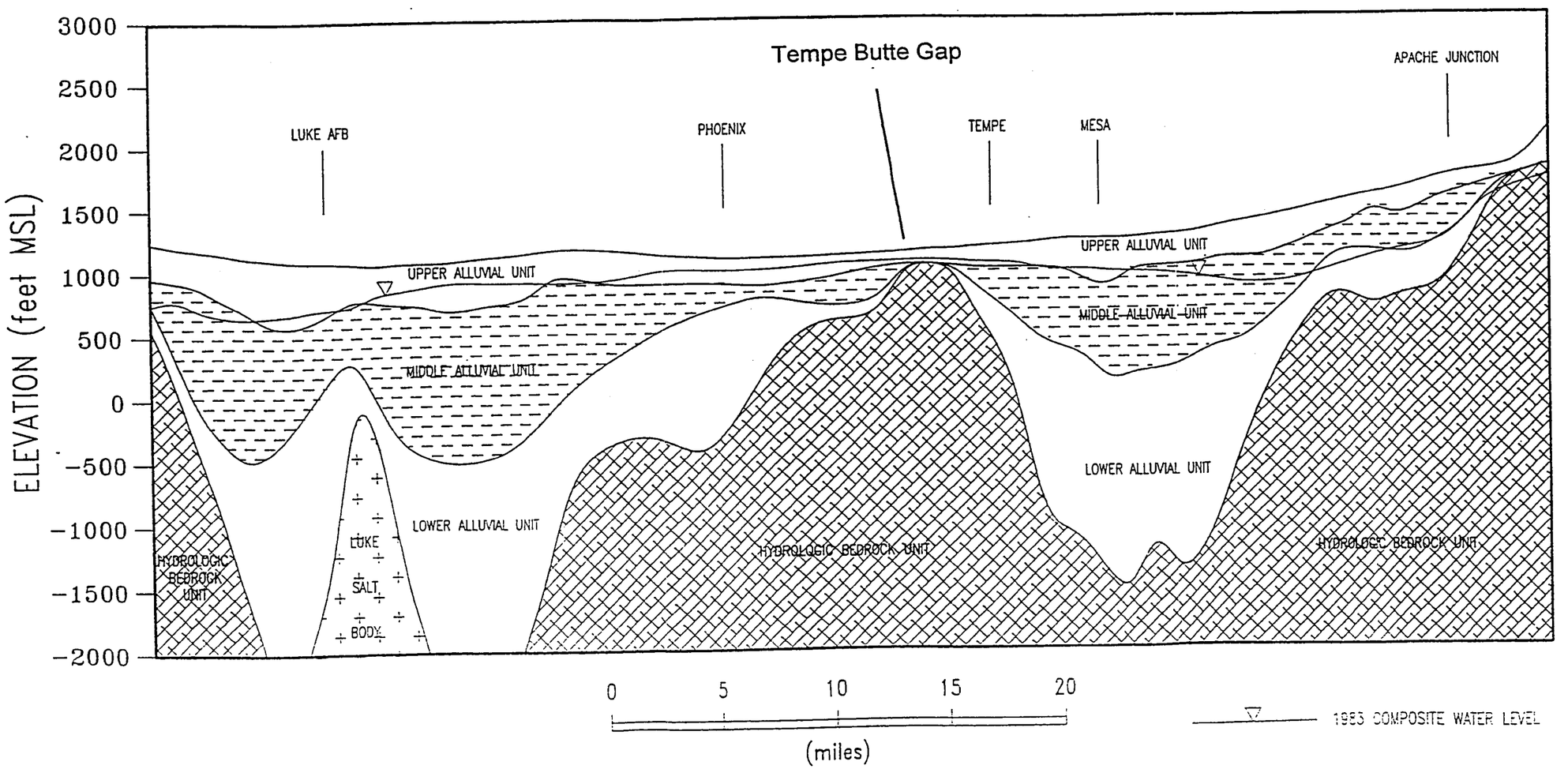

ALLUVIAL GROUNDWATER BASIN OF SALT RIVER VALLEY
Figure 2
VULTURE MTS.
MERCURY MTS.
HASSAYAMPA RIVER
AGUA FRIA RIVER
NEW RIVER
UNION HILLS
MCDOWELL MTS.
VERDE RIVER
WHITE TANK MTS.
A
West Salt River Valley Sub-basin
PHOENIX
TEMPE
SALT RIVER
USERY MTS.
GOLDFIELD MTS.
SUPERSTITION MTS.
A'
East Salt River Valley Sub-basin
SOUTH MTN.
BUCKEYE HILLS
SIERRA ESTRELLA
WATERMAN WASH
SANTA CRUZ RIVER
QUEEN CREEK
GILA RIVER
SANTAN MTS.
SACATON MTS.
N

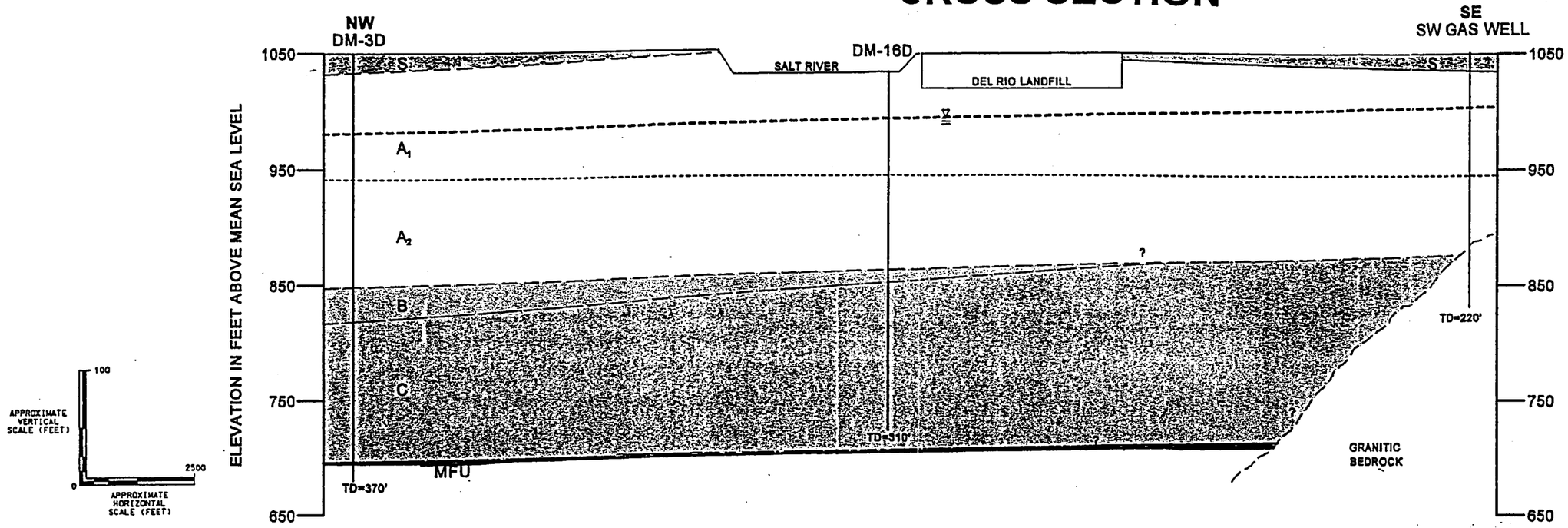
Figure 3

Rio Salado Habitat Project, Phoenix Portion

GENERALIZED HYDROGEOLOGIC CROSS-SECTION



CROSS SECTION



EXPLANATION

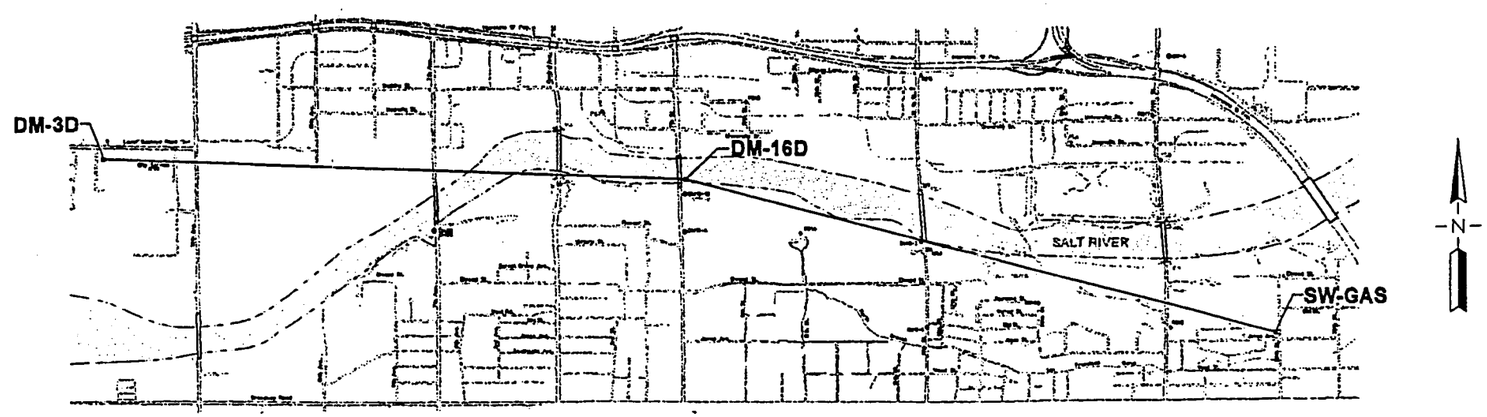
INFERRED UNIT CONTACT

INFERRED SUBUNIT CONTACT (TRANSITIONAL)

APPROXIMATE WATER TABLE DAMES AND MOORE (1991)

	Geologic Subunit	DESCRIPTION
UAU	S	SILTY SAND, BROWN TO LIGHT BROWN, 50-60% FINE TO MEDIUM SAND, 30-40% SILT WITH CLAY, <10% GRAVEL, TRACE CALICHE
UAU	A	SUBUNIT A_1: GRAVEL, 80-90% FINE TO COARSE GRAVEL AND COBBLES, 20-10% MEDIUM TO COARSE SAND, LITTLE TO NO FINES SUBUNIT A_2: SANDY GRAVEL, 60-70% FINE TO COARSE GRAVEL, 30-40% FINE TO COARSE SAND, MINOR SILT, INCREASING FINES (SAND & SILT) WITH DEPTH INTERBEDDED SAND (SP) LENSES
UAU	B	SILTY SAND WITH GRAVEL, REDDISH BROWN, 40-50% FINE TO MEDIUM SAND, 40-50% SILT WITH CLAY, 10-15% FINE GRAVEL
UAU	C	SANDY GRAVEL WITH SILT AND CLAY, BROWN, 40-50% FINE TO MEDIUM GRAVEL, 40-50% FINE TO COARSE SAND, 0-20% SILT WITH CLAY, INTERBEDDED SAND (SP) AND GRAVEL (GP) LENSES, DECREASING FINES (SILT & CLAY) WITH DEPTH
MAU	(MFU)	SILTY SAND WITH GRAVEL, CLAYEY SILTS, BROWN INTERBEDDED SANDY SILT, CLAYEY SILT AND SILTY SAND WITH MINOR FINE TO MEDIUM GRAVEL

NOTE: GEOLOGIC UNIT AND SUBUNIT DESCRIPTIONS ARE FOR UAU AND MAU ONLY, DAMES AND MOORE 1989.



CROSS SECTION LOCATION
N.T.S.

Plan View

Phoenix Portion of Rio Salado, From 19th Avenue to I-10

SCHEMATIC STRATIGRAPHIC CROSS SECTION THROUGH PROJECT AREA

U.S. ARMY CORPS OF ENGINEERS
RIO SALADO HABITAT RESTORATION
Phoenix, Arizona

Figure 4

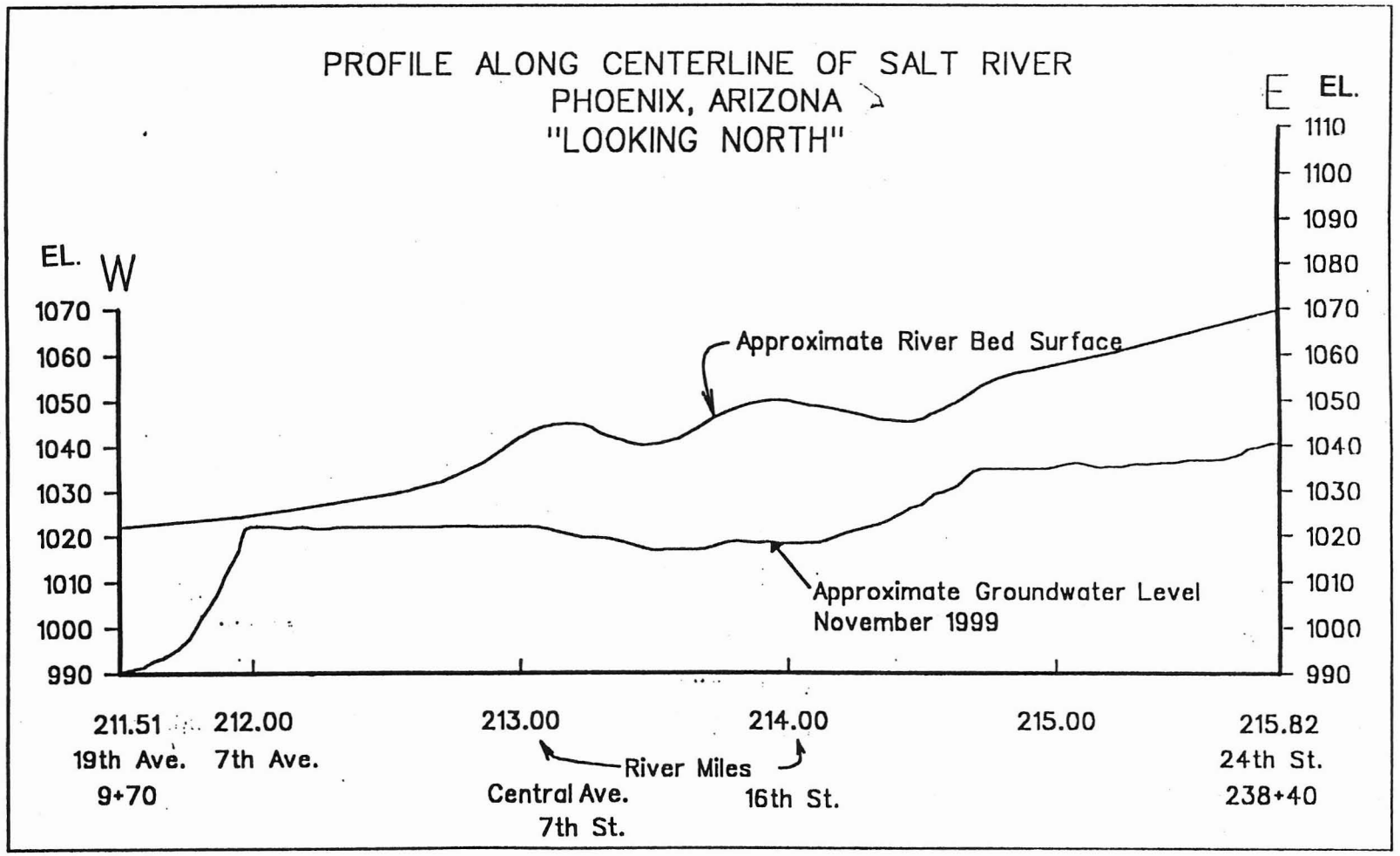
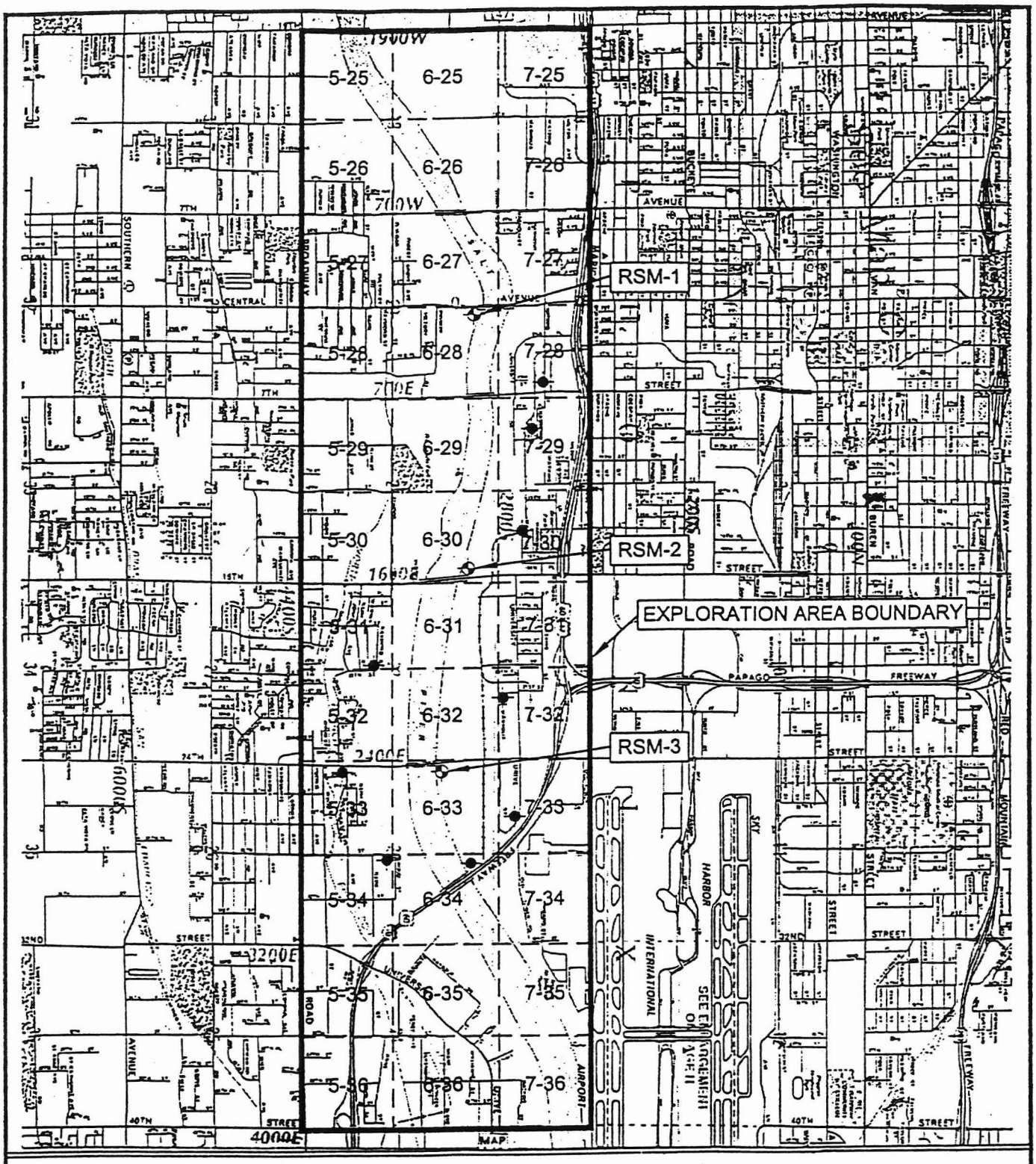

PROFILE ALONG CENTERLINE OF SALT RIVER
PHOENIX, ARIZONA
"LOOKING NORTH"
EL.
W
E
EL.
1070
1060
1050
1040
1030
1020
1010
1000
990
1110
1100
1090
1080
1070
1060
1050
1040
1030
1020
1010
1000
990
Approximate River Bed Surface
Approximate Groundwater Level
November 1999
211.51
19th Ave.
9+70
212.00
7th Ave.
213.00
Central Ave.
7th St.
River Miles
214.00
16th St.
215.00
215.82
24th St.
238+40
1 inch = 2,760 feet

Figure 5



Monitoring Well Locations for RSMW-1 to RSMW-3

EXPLANATION:

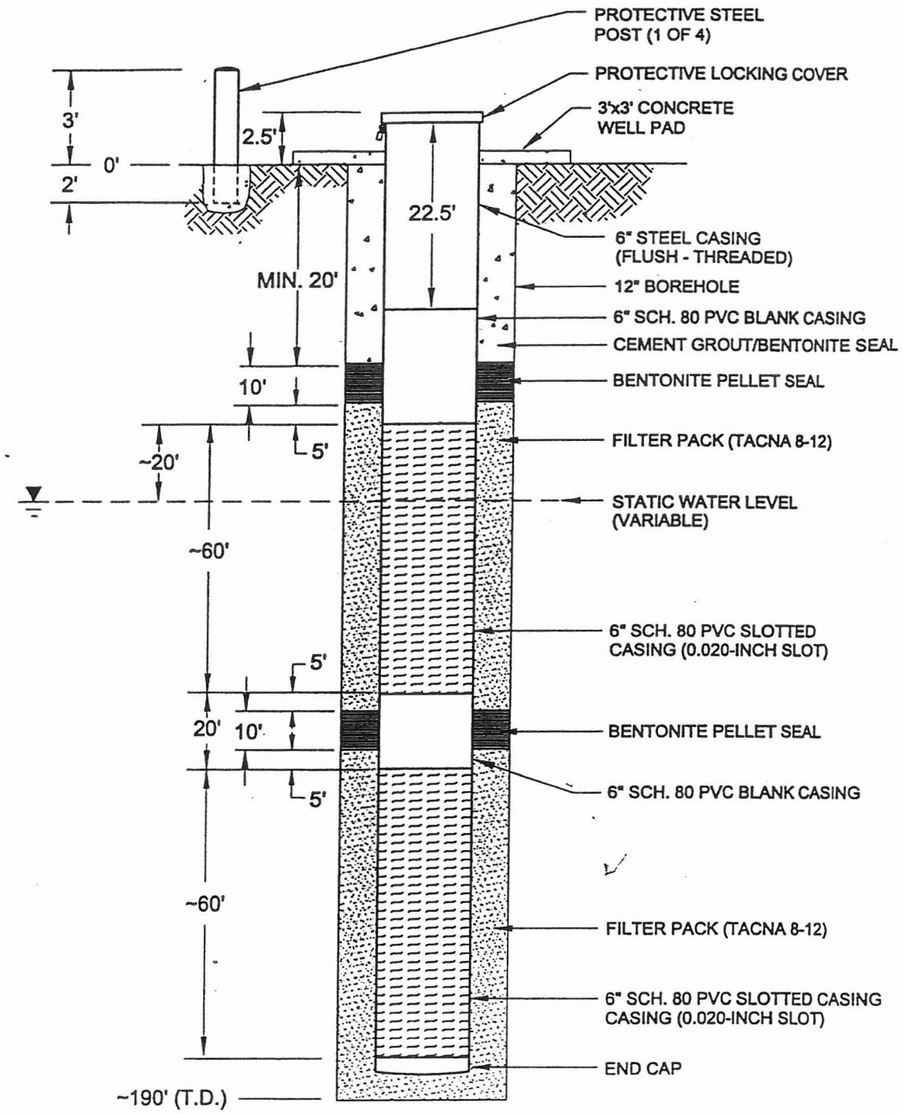
- Currently Proposed Monitor Well Location
- Future Monitor Well Location

N

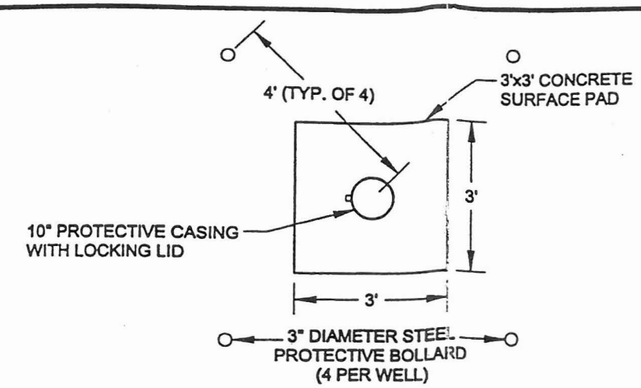
0 1/2 1

Scale in Miles

Figure 6

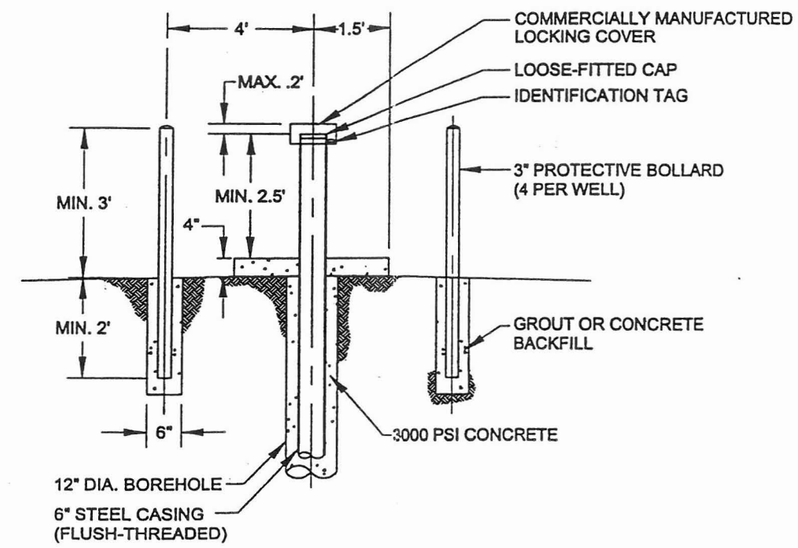


NOT TO SCALE



PLAN VIEW

NOT TO SCALE



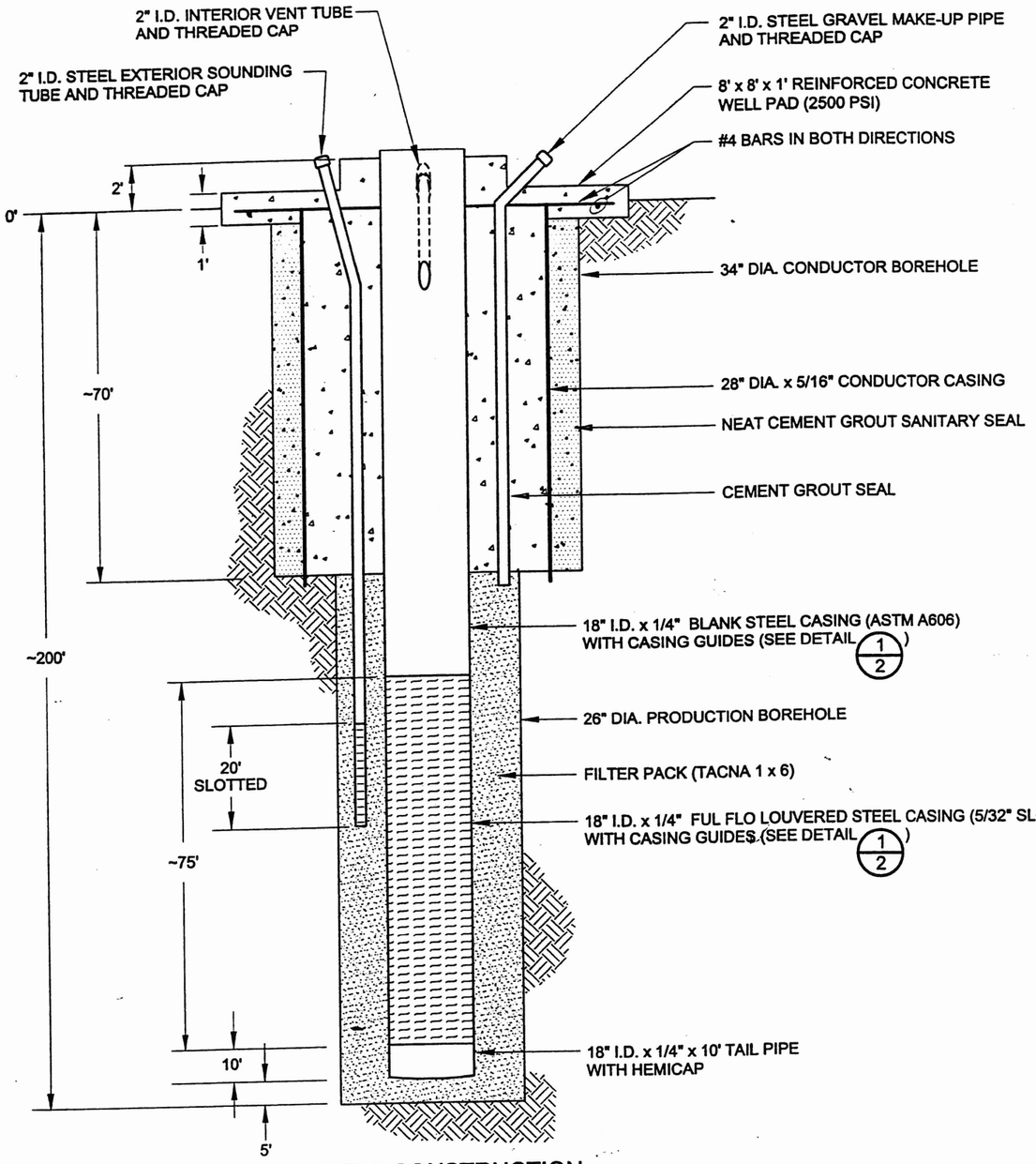
SURFACE COMPLETION DETAILS FOR MONITORING WELLS

NOT TO SCALE

Monitor Well Construction Diagram
and Surface Completion Detail

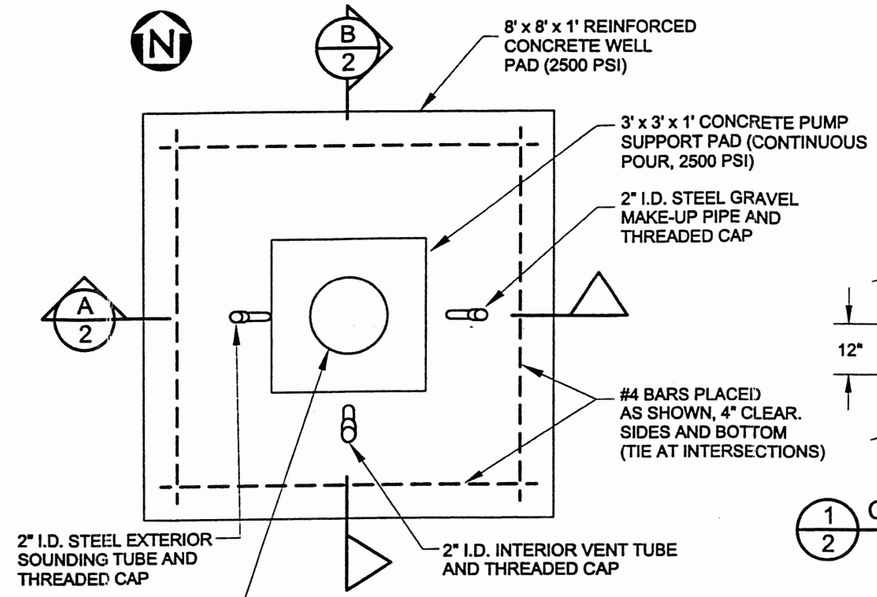
Figure 7

Figure 8



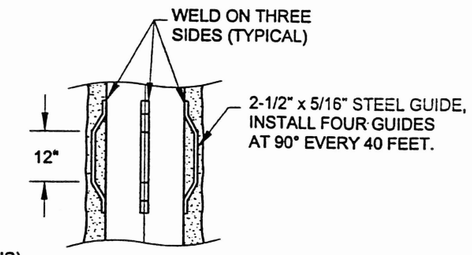
WELL CONSTRUCTION

NOT TO SCALE



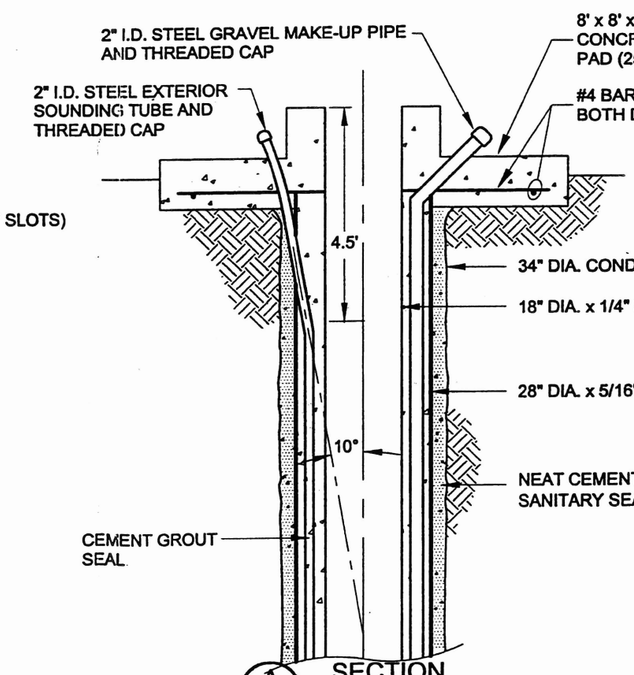
PLAN VIEW

SCALE: 1"=3'-0"



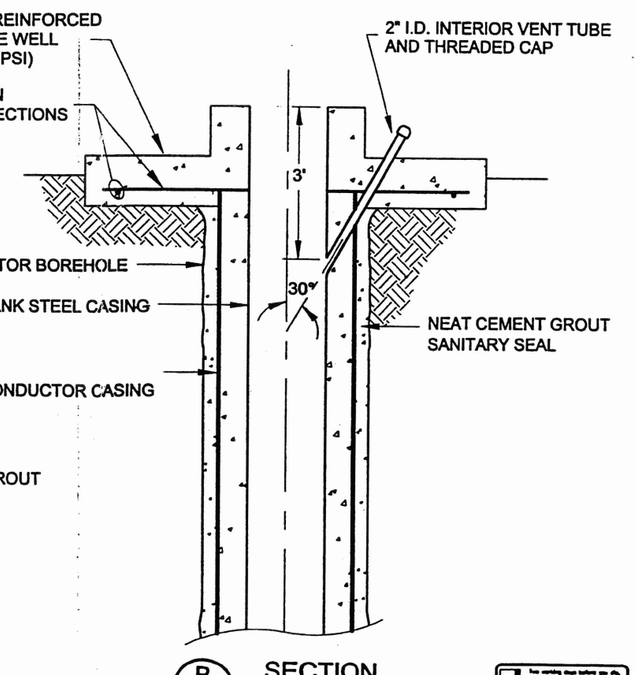
1/2 CASING GUIDE DETAIL

SCALE: 1"=3'-0"



A/2 SECTION

SCALE: 1"=3'-0"



B/2 SECTION

SCALE: 1"=3'-0"

2S3-1100 1-800-STAKE-IT

REFERENCES. TITLE	REFERENCES TITLE	REVISIONS NO.	BY	DATE	DESCRIPTION	REVISIONS NO.	BY	DATE	DESCRIPTION
		1	CB	11/99	PRELIMINARY DESIGN				

SCALE: AS NOTED		DATE
DESIGNED BY:	CCB	11-99
DRAWN BY:	MDH/KLP	11-99
CHECKED BY:	CSW	01-00
APPROVED BY:	XXX	X-00
CLIENT APPROVAL BY:		

DAMES & MOORE

U.S. ARMY CORPS OF ENGINEERS AND CITY OF PHOENIX

PRODUCTION WELL RSPW-1 DESIGN DETAILS

RIO SALADO HABITAT RESTORATION PROJECT

JOB NO. 00109-042-058

DRAWING NO. 2 REV. B

\00109\042\REV-B\A1X15RB 2-11-00

TT99-5

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	90	70	47	37	22	19	15	11	6	4	2		NP	4.5	POORLY GRADED GRAVEL WITH SAND, gray, approximately 20% cobbles and 10% boulders.
5.0		100	63	46	33	27	23	19	14	10	7	4		NP	12.2	
8.0	SP	100	94	91	88	84	81	75	53	23	7	3		NP	16.9	POORLY GRADED SAND WITH GRAVEL, brown, dense.
11.0		100	100	100	100	100	100	98	77	30	8	2		NP	12.3	POORLY GRADED SAND, brown.
16.0	GP	100	78	56	44	39	37	33	22	10	8	3		NP	13.7	POORLY GRADED GRAVEL WITH SAND, brown, gravels, less than 5% cobbles.

TT99-6

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	100	77	59	44	34	28	18	7	2	1	1		NP	4.8	POORLY GRADED GRAVEL WITH SAND, brown, approximately 25% cobbles and 15% boulders.
7.0		100	46	27	18	13	8	5	2	1	1	1		NP	NS	POORLY GRADED GRAVEL, same as above, more cobbles. Stopped trench due to caving and water in the trench.

TT99-7

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	100	79	61	46	36	32	24	12	5	2	1		NP	1.8	POORLY GRADED GRAVEL WITH SAND, brown, approximately 25% cobbles and 10% boulders.
6.0		100	89	38	28	23	21	18	12	6	4	2		NP	25.9	
10.0	GW	100	68	47	36	29	28	26	21	9	4	2		NP	11.4	WELL GRADED GRAVEL WITH SAND, brown, very dense, approximately 20% cobbles.
13.0	GP	100	73	57	44	36	33	29	20	10	6	2		NP	12.4	POORLY GRADED GRAVEL WITH SAND, brown, approximately 10% cobbles.
16.0		100	64	51	39	32	29	26	18	8	4	2		NP	11.2	Water at 14 feet.

TT99-8

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GW	85	77	58	43	33	27	20	12	5	2	1		NP	2.0	WELL GRADED GRAVEL WITH SAND, brown, approximately 20% cobbles and 5% boulders.
6.0	GP	100	62	36	29	23	21	18	13	5	2	1		NP	8.3	POORLY GRADED GRAVEL WITH SAND, brown, approximately 20% cobbles and 5% boulders.
9.0		100	70	43	32	26	25	22	15	8	5	3		NP	NS	Stopped hole due to trench caving, water entered trench beginning at 7 foot depth.

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS More than half of material is larger than no. 200 sieve size.	GRAVELS More than half of coarse fraction is larger than no. 4 sieve size.	Clean Gravels	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.
		Gravels with fines	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS More than half of coarse fraction is smaller than no. 4 sieve size.	Clean Sands	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands, gravelly sands, little or no fines.
		Sands with fines.	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS More than half of materials smaller than no. 200 sieve size.	SILTS AND CLAYS	Low liquid limit.	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts, with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
		High liquid limit.	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
Highly organic soils			Pt	Peat and other highly organic soils.

NOTES:

1. BOUNDRY CLASSIFICATION: SOILS POSSESSING CHARACTERISTICS OF TWO GROUPS ARE DESIGNATED BY COMBINATIONS OF GROUP SYMBOLS. FOR EXAMPLE, GW-GC, WELL GRADED GRAVEL-SAND MIXTURE WITH CLAY BINDER.
2. ALL SIEVE SIZES ON THE CHART ARE U.S. STANDARD.
3. THE TERMS "SILT" AND "CLAY" ARE USED RESPECTIVELY TO DISTINGUISH MATERIALS EXHIBITING LOWER PLASTICITY FROM THOSE WITH HIGHER PLASTICITY. THE MINUS NO. 200 SIEVE MATERIAL IS SILT IF THE LIQUID LIMIT AND PLASTICITY INDEX PLOT BELOW THE "A" LINE ON THE PLASTICITY CHART, AND IS CLAY IF THE LIQUID LIMIT AND PLASTICITY INDEX PLOT ABOVE THE "A" LINE ON THE CHART.
4. THE SOIL CLASSIFICATION SYSTEM IS BASED ON THE AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM).
   A. (ASTM) D2487 STANDARD TEST METHOD FOR CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES.
   B. (ASTM) D2488 STANDARD RECOMMENDED PRACTICE FOR DESCRIPTION OF SOILS (VISUAL MANUAL PROCEDURE).

LEGEND

TT99-5	TEST TRENCH, YEAR AND NUMBER
LL	LIQUID LIMIT.
PI	PLASTICITY INDEX (LIQUID LIMIT - PLASTIC LIMIT).
NP	NONPLASTIC.
NS	NOT SAMPLED
-4	PERCENT OF MATERIAL, BY WEIGHT, PASSING NO. 4 SIEVE.
-200	PERCENT OF MATERIAL, BY WEIGHT, PASSING NO. 200 SIEVE.
N	NUMBER OF BLOWS OF A 140-POUND DROPHAMMER FALLING 30 INCHES REQUIRED TO DRIVE A SAMPLING SPOON ONE FOOT. OUTSIDE DIAMETER IS 2 INCHES: INSIDE DIAMETER IS 1-3/8 INCHES.
PCC	PORTLAND CEMENT CONCRETE (PCC) RUBBLE. PCC RUBBLE FROM VARIOUS DEMOLITION ACTIVITIES.
AC	ASPHALTIC CONCRETE PAVEMENT RUBBLE.

NOTES

1. LOGS OF EXPLORATION INDICATE GEOTECHNICAL CONDITIONS AT THE TIME AND LOCATION OF THE EXPLORATIONS INDICATED. CONDITIONS CAN CHANGE. STRATIFICATION LINES SHOWN ON LOGS REPRESENT APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES.
2. GROUNDWATER WHEN ENCOUNTERED, NOTED ON EACH BORING LOG.
3. TEST TRENCHES WERE EXCAVATED IN JANUARY, 1999, USING RUBBER-TIRED BACKHOE, CASE MODEL 580 SUPER L, WITH A 24-INCH BUCKET.
4. TEST TRENCHES WERE GENERALLY TERMINATED AT 16 FEET DEPTH BY DESIGN.
5. PERCENTAGE OF PLUS 3-INCH MATERIAL BASED ON VISUAL OBSERVATIONS IN THE FIELD. GRADATION INDICATED HEREIN REPRESENTS MINUS 3-INCH SAMPLE RETURNED TO THE LABORATORY FOR DETAILED ANALYSIS.
6. ALL PARTICLES EXAMINED DURING THE INVESTIGATION WERE GENERALLY SUBROUNDED TO ROUNDED, UNLESS NOTED OTHERWISE.
7. TEST TRENCHES 34 TO 39 WERE VISUALLY LOGGED ONLY.
8. SEE SHEETS 3 TO 14 FOR LOCATION OF TEST TRENCHES.

Figure 9

SCALE 5 0 5 10 15 FEET

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

LOGS OF EXPLORATION TT99-5 TO TT99-8

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: W H
DRAWN BY: W H
CHECKED BY: JDD

SUBMITTED BY:
ABBAS T. ROODSARI, P.E.
CHIEF, GEOTECHNICAL BRANCH

CADD FILE NAME: g01.dgn

DISTRICT FILE NO. NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 15 OF 19 SHEETS

TT99-9

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	100	81	61	48	42	39	32	19	7	3	1		NP	4.5	POORLY GRADED GRAVEL WITH SAND, brown, loose, approximately 15% cobbles, and 5% boulders.
6.0		100	80	59	48	21	20	18	14	8	4	1		NP	3.4	
8.0	SP	100	90	77	70	66	60	54	43	25	12	4			17.2	POORLY GRADED SAND WITH GRAVEL, gray, loose, approximately 5% cobbles.
10.0	GP	100	81	65	52	43	40	33	22	10	5	2		NP	9.4	POORLY GRADED GRAVEL WITH SAND, gray, very loose.
13.0		100	45	20	13	10	9	8	6	4	3	2		NP	57.3	POORLY GRADED GRAVEL, primarily brown, some black, green, very moist.
16.0	GW	100	75	54	40	32	28	22	15	8	3	1		NP	20.7	WELL GRADED GRAVEL WITH SAND, gray/brown, approximately 25% cobbles and 15% boulders.

TT99-10

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	100	74	57	44	34	30	26	20	9	5	1		NP	16.7	POORLY GRADED GRAVEL WITH SAND, gray, loose, approximately 20% cobbles and 5% boulders, PCC on surface.
7.0		100	73	54	42	36	33	28	17	7	3	1		NP	5.7	same as above, approximately 25% cobbles.
10.0		100	78	56	43	35	32	28	11	5	4	3		NP	14.6	same as above, reddish brown, approximately, 5% cobbles, loose.
13.0		100	68	41	26	20	18	14	8	4	3	1		NP	21.0	same as above, reddish brown, approximately 5% cobbles.
16.0		100	85	59	41	35	33	30	17	6	3	2		NP	15.1	same as above, reddish brown, approximately 5% cobbles.

TT99-11

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	100	81	63	50	41	38	31	19	9	5	3		NP	1.5	POORLY GRADED GRAVEL WITH SAND, light brown, loose, approximately 10% cobbles, some boulders.
6.0	GW	100	70	45	34	30	29	27	14	3	1	1		NP	2.6	WELL GRADED GRAVEL WITH SAND, light brown, loose, approximately 10% cobbles, less than 5% boulders.
9.0		100	76	53	38	30	28	25	15	4	1	1		NP	2.6	same as above, light gray to brown, approximately 15% cobbles and 5% boulders.
13.0	SP	100	89	75	68	63	60	53	28	6	2	2		NP	3.4	POORLY GRADED SAND WITH GRAVEL, light gray to brown, very loose, less than 5% cobbles.
16.0	GW	100	82	43	34	29	27	24	14	4	2	1		NP	6.5	WELL GRADED GRAVEL WITH SAND, light brown, loose, approximately 5% cobbles.

TT99-12

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
2.0	GW	100	60	43	33	28	26	23	15	5	2	1		NP	2.3	WELL GRADED GRAVEL WITH SAND, light brown, loose, approximately 5% cobbles.
5.0	SP	100	99	93	89	87	87	85	66	28	5	1		NP	10.3	POORLY GRADED SAND, light brown, no cobbles, water encountered at 5 feet.
8.0		100	77	68	63	61	60	58	34	11	2	1		NP	11.1	POORLY GRADED SAND WITH GRAVEL, light brown. Terminated trench due to water in trench.

TT99-13

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	100	86	61	44	36	32	24	9	3	1	1		NP	4.9	POORLY GRADED GRAVEL WITH SAND, light brown, very loose, approximately 5% cobbles.
6.0		100	89	69	51	40	33	21	6	1	1	1		NP	2.0	same as above, less cobbles.
8.0		100	89	69	52	38	35	29	18	10	7	4		NP	5.0	same as above, light brown, less than 5% cobbles, some PCC debris.
11.0		90	80	66	51	40	33	24	12	4	2	1		NP	37.0	same as above, approximately 10% cobbles, 5% boulders, some PCC and AC debris.
16.0		100	77	51	40	32	29	25	12	3	1	1		NP	3.9	same as above, light brown, very loose, approximately 5% cobbles.

TT99-14

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
2.0	GP	100	80	59	44	36	32	29	19	5	1	1		NP	5.3	POORLY GRADED GRAVEL WITH SAND, very loose, approximately 15% cobbles.
6.0		100	60	24	21	19	17	13	7	3	2	1		NP	38.2	same as above, brown, approximately 20% cobbles, less than 5% boulders.
10.0		100	75	53	39	32	28	24	12	3	1	1		NP	n/s	same as above.
16.0	GW	100	69	44	32	25	21	16	8	2	1	1		NP	12.6	WELL GRADED GRAVEL WITH SAND, brown, very dense, approximately 25% cobbles, less than 5% boulders.

TT99-15

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
4.0	GP	100	73	55	40	33	27	21	10	3	1	1		NP	5.7	POORLY GRADED GRAVEL WITH SAND, light brown, loose, 10% cobbles.
7.0	SP	100	98	94	91	90	88	84	69	38	9	1		NP	8.1	POORLY GRADED SAND, light brown, very loose.
10.0	GP	100	75	55	45	39	37	34	20	7	2	1		NP	9.4	POORLY GRADED GRAVEL WITH SAND, light brown, loose, some cobbles, odor of fuel detected.
12.0	SP	100	93	88	86	84	83	76	19	2	1	1		NP	16.5	POORLY GRADED SAND WITH GRAVEL, light brown, dense.
16.0	GP	100	82	58	44	39	37	33	15	3	1	1		NP	13.7	POORLY GRADED GRAVEL WITH SAND, light brown, approximately 10% cobbles.

TT99-16

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
2.0	GP	100	87	63	49	43	42	38	20	5	1	1		NP	5.6	POORLY GRADED GRAVEL WITH SAND, light brown, some cobbles.
3.0	SP	100	89	82	73	67	63	58	49	31	9	1		NP	10.1	POORLY GRADED SAND WITH GRAVEL, light brown,
4.0		100	85	70	61	54	49	41	26	14	7	2		NP	15.9	very loose, percentage of cobbles increases with
5.0		100	89	84	81	79	58	29	13	6	2	1		NP	33.3	depth.
7.0	SP-SM	100	100	100	99	99	93	88	83	75	48	9		NP	26.7	POORLY GRADED SAND WITH SILT, brown, very loose.
11.0	GP	100	85	59	42	33	29	24	12	5	2	1		NP	16.2	POORLY GRADED GRAVEL WITH SAND, light brown, approximately 20% cobbles, hard digging due to large rock.
16.0	SP	100	75	64	57	53	49	42	21	6	1	1		NP	17.7	POORLY GRADED SAND WITH GRAVEL, more cobbles than above, approximately 10% boulders.

Figure 10

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

LOGS OF EXPLORATION
TT99-9 TO TT99-16

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: W H
DRAWN BY: W H
CHECKED BY: JDD

SUBMITTED BY:
ABBAS T. ROODSARI, P.E.
CHIEF, GEOTECHNICAL BRANCH

CADD FILE NAME: g02.dgn

DISTRICT FILE NO. NOT APPLICABLE
SPEC. NO. : NOT APPLICABLE

SHEET 16 OF 19 SHEETS

FIGURE 11

TT99-17

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GW	100	68	51	36	28	24	20	10	3	1	1		NP	9.3	WELL GRADED GRAVEL WITH SAND, light brown, loose, approximately 5% cobbles, 5% boulders, hard digging due to large rock.
6.0	GP	100	83	59	45	37	33	30	22	14	7	1		NP	12.0	POORLY GRADED GRAVEL WITH SAND, light brown, loose, approximately 15% cobbles, 5% boulders.
9.0		92	78	60	43	33	30	25	13	4	1	1		NP	13.5	same as above, light brown, approximately 20% cobbles and 10% boulders.
12.0		100	89	67	53	45	40	29	14	4	1	1		NP	16.3	same as above, brown, approximately 15% cobbles, and 5% boulders.
16.0	GW	92	75	50	36	29	26	23	13	4	1	1		NP	16.6	WELL GRADED GRAVEL WITH SAND, brown, approximately 15% cobbles and 5% boulders.

TT99-18

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	100	77	62	48	40	37	33	22	9	3	1		NP	7.4	POORLY GRADED GRAVEL WITH SAND, light brown, moist, loose to dense, 20% cobbles.
6.0		100	69	48	35	27	23	21	16	9	3	1		NP	7.3	same as above, less than 5% boulders.
10.0		100	71	54	42	34	30	26	16	7	2	1		NP	12.2	same as above, brown, very dense, approximately 15% cobbles, less than 5% boulders.
16.0	GW	100	73	51	38	29	24	20	13	5	2	1		NP	16.5	WELL GRADED GRAVEL WITH SAND, brown, very dense, approximately 20% cobbles.

TT99-19

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GP	100	73	55	43	37	35	30	17	7	3	1		NP	11.1	POORLY GRADED GRAVEL WITH SAND, brown, dense, approximately 20% cobbles.
6.0		100	67	45	36	33	32	28	13	2	1	1		NP	8.9	same as above, approximately 25% cobbles.
10.0	GW	100	58	43	31	26	24	19	7	2	1	1		NP	11.1	WELL GRADED GRAVEL WITH SAND, brown, approximately 20% cobbles, hard digging! Hit water at 10 feet.
14.0	GP	100	74	42	25	20	19	17	13	7	3	2		NP	NS	POORLY GRADED GRAVEL WITH SAND, brown, same as above. Stop trench due to caving and water inflow.

TT99-20

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
2.0	GP	91	81	64	51	41	36	30	18	9	6	3		NP	4.2	POORLY GRADED GRAVEL WITH SAND, light brown, loose, approximately 5% cobbles.
3.0	SM	100	99	87	73	62	57	54	44	35	29	23		NP	8.2	SILTY SAND WITH GRAVEL, brown, dense, debris of: asphalt-concrete and tires.
6.0	ML	100	99	98	98	98	98	97	96	95	91	75			8.6	SILT WITH SAND, light brown, very loose, powdery.
8.0		100	100	100	100	100	100	100	99	99	97	87			24.3	SILT, brown, chunky, falls apart on handling.
11.0	SM	100	100	100	100	100	100	99	99	97	73	31		NP	3.9	SILTY SAND, brown, dry.
13.0		100	100	99	98	98	97	97	95	93	81	46			23.2	SILTY SAND, brown.
16.0		100	79	79	74	71	69	67	59	53	44	25		NP	7.3	SILTY SAND WITH GRAVEL, light brown, loose, approximately 5% cobbles.

TT99-21

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GW	100	68	50	38	31	28	25	18	9	5	3		NP	5.9	WELL GRADED GRAVEL WITH SAND, light brown, loose, approximately 20% cobbles and 5% boulders.
6.0	SP	90	90	81	74	70	69	66	59	41	18	4		NP	5.5	POORLY GRADED SAND WITH GRAVEL, light brown, loose, approximately 20% cobbles and 5% boulders.
9.0	GP	100	87	63	51	44	41	37	28	18	7	2		NP	8.4	POORLY GRADED GRAVEL WITH SAND, light brown, very loose, approximately 20% cobbles and less than 5% boulders.
12.0		100	58	23	15	13	12	11	9	4	1	1		NP	14.6	POORLY GRADED GRAVEL, light brown, loose, approximately 10% cobbles, less than 5% boulders.
16.0		94	77	40	28	25	24	22	17	9	3	1		NP	7.3	POORLY GRADED GRAVEL WITH SAND, light brown, loose, approximately 10% cobbles, less than 5% boulders.

TT99-22

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
3.0	GW	100	76	51	38	30	27	23	16	7	3	1		NP	9.9	WELL GRADED GRAVEL WITH SAND, brown, very dense, approximately 25% cobbles and 20% boulders.
6.0		100	80	57	42	32	28	24	13	4	1	1		NP	13.3	same as above, brown, approximately 15% cobbles.
10.0	GP	100	64	41	28	22	20	18	9	2	1	1		NP	12.4	POORLY GRADED GRAVEL WITH SAND, brown,, approximately 15% cobbles, hard digging as above. Hit water at 10 feet.
16.0		100	72	45	30	24	22	20	17	11	5	2		NP	21.6	WELL GRADED GRAVEL WITH SAND, brown to black, approximately 40% cobbles and 15% boulders, hard to dig as above.

TT99-27

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
1.0	GP-GM	100	69	52	44	38	36	34	27	19	13	8		NP	10.4	POORLY GRADED GRAVEL WITH SILT AND SAND, brown, loose, approximately 15% cobbles.
6.0	GP	100	47	39	35	32	30	27	22	14	6	1		NP	11.2	POORLY GRADED GRAVEL WITH SAND, light brown to gray, loose approximately 15% cobbles, less than 5% boulders.
8.0	GW	88	60	47	36	29	27	24	18	11	7	4		NP	11.0	WELL GRADED GRAVEL WITH SAND, light brown to gray, loose, approximately 15% cobbles, less than 5% boulders.
10.0	SP	100	89	81	76	71	68	56	18	2	1	1		NP	16.0	POORLY GRADED SAND WITH GRAVEL, light brown to gray, very loose, less than 5% cobbles.
16.0	GW	89	57	40	32	27	26	23	13	5	2	1		NP	18.4	WELL GRADED GRAVEL WITH SAND, light brown to gray, loose, approximately 20% cobbles.

TT99-28

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
4.0	GP	100	91	74	45	36	33	29	19	8	3	1		NP	10.5	POORLY GRADED GRAVEL WITH SAND, light brown, loose, approximately 15% cobbles, less than 5% boulders.
8.0	GW	100	83	62	45	32	27	21	11	4	1	1		NP	6.6	WELL GRADED GRAVEL WITH SAND, brown, loose, approximately 20% cobbles and 5% boulders.
12.0	SP	100	82	64	57	52	49	45	29	11	3	1		NP	6.6	POORLY GRADED SAND WITH GRAVEL, brown, loose, approximately 20% cobbles and 5% boulders.
16.0	GP	100	55	23	16	13	11	10	7	3	1	1		NP	8.4	POORLY GRADED GRAVEL, brown, approximately 20% cobbles and 5% boulders, some trash.

Figure 11

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

LOGS OF EXPLORATION
TT99-17 TO 20, 27, AND 28

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: W H
DRAWN BY: W H
CHECKED BY: JDD

SUBMITTED BY: ABBAS T. ROODSARI, P.E.
CHIEF, GEOTECHNICAL BRANCH

CADD FILE NAME: g03.dgn

DISTRICT FILE NO. NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 17 OF 19 SHEETS

FIGURE 12

TT99-29

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
2.0	GP	100	64	52	40	33	30	23	11	3	1	1		NP	13.3	POORLY GRADED GRAVEL WITH SAND, brown, very loose, approximately 10% cobbles, less than 5% boulders.
8.0		100	66	42	31	25	20	16	9	1	1	1		NP	6.6	same as above, loose, approximately 20% cobbles, less than 5% boulders.
12.0	SP	100	92	88	84	82	80	78	66	23	8	1		NP	5.7	POORLY GRADED SAND WITH GRAVEL, light brown, very loose.
16.0	GP	100	87	69	56	48	44	40	29	11	3	1		NP	6.2	POORLY GRADED GRAVEL WITH SAND, light brown, very loose, less than 5% cobbles.

TT99-30

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
5.0	GP	90	45	33	27	23	21	16	8	3	1	1		NP	15.5	POORLY GRADED GRAVEL WITH SAND, brown, loose, approximately 10% cobbles, less than 5% boulders.
8.0	SW-SM	100	98	94	91	88	71	52	37	23	14	6		NP	14.1	WELL GRADED SAND WITH SILT, dark brown, dense.
12.0	GP	90	78	62	52	47	44	35	13	3	1	1		NP	12.2	POORLY GRADED GRAVEL WITH SAND, light brown, very loose, less than 5% cobbles.
16.0		100	55	32	23	20	18	16	11	4	1	1		NP	21.8	same as above.

TT99-31

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
6.0	GP	90	44	32	25	22	21	19	11	3	1	1		NP	14.3	POORLY GRADED GRAVEL WITH SAND, brown, loose approximately 20% cobbles, some PCC rubble.
12.0		100	67	43	32	26	24	21	10	2	1	1		NP	13.6	same as above, light brown, loose, approximately 25% cobbles.
17.0		100	58	41	26	20	18	15	7	2	1	1		NP	16.3	same as above, dark brown to black, loose, approximately 15% cobbles.

TT99-32

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
1.0	GP	91	75	58	50	44	42	40	30	16	8	3		NP	12.4	POORLY GRADED GRAVEL WITH SAND, brown, loose, approximately 15% cobbles, less than 5% boulders.
10.0		100	75	50	41	36	34	31	21	9	3	1		NP	15.7	same as above, light brown, loose, approximately 20% cobbles, some boulders, some PCC rubble.
16.0		100	74	57	46	37	32	26	15	5	2	1		NP	14.8	same as above, dark gray/brown, loose, approximately 25% cobbles, less than 5% boulders.

TT99-33

DEPTH (ft)	SOIL CLASS	3	1.5	3/4	3/8	4	8	16	30	50	100	200	LL	PI	MC	DESCRIPTION
1.0	GW	100	78	52	39	31	27	23	12	5	2	1		NP	5.9	WELL GRADED GRAVEL WITH SAND, brown, loose, approximately 15% cobbles, some PCC debris.
2.0	SP	100	100	95	81	52	46	38	25	15	7	3		NP	13.3	POORLY GRADED SAND WITH GRAVEL, light brown, loose, approximately 10% cobbles.
8.0	GP	100	96	88	68	49	42	33	21	11	6	3		NP	21.1	POORLY GRADED GRAVEL WITH SAND, dark green, loose, trash (wood, metal tires, brick), approximately 10% cobbles. Encountered water at 7 feet.
9.5	SM	100	100	100	100	100	92	81	75	70	65	42			44.0	SILTY SAND, brown to dark green. Stopped trench due to water.

TT99-34

DEPTH (ft)	SOIL CLASS	DESCRIPTION
2.0	SP	POORLY GRADED SAND WITH GRAVEL, brown, very dry, dense to loose, fine to medium grained sand, 5% gravels.
9.0	GP	POORLY GRADED GRAVEL WITH SAND, light gray to brown, dry to moist, loose, medium to coarse grained sand, 5% cobbles, less than 5% boulders.
13.0	GP	GRAVELLY SAND WITH COBBLES, dark brown, very moist, coarse grained sand, approximately 25% cobbles and 5% boulders, hard digging, due to many large particles.
14.0	GP	same as above, approximately 10% boulders.

TT99-35

DEPTH (ft)	SOIL CLASS	DESCRIPTION
3.0	GP	POORLY GRADED GRAVEL WITH SAND, light brown, dry to moist, loose, medium to coarse grained sand, approximately 20% cobbles and 10% boulders, less than 5% chunks of concrete rubble, asphalt, and brick.
7.0	GP	same as above, approximately 25% cobbles, no trash, sand is very clean.
11.0	GP	same as above, with approximately 5% boulders. Terminated hole at 11 feet due to layer of cobbles.

TT99-36

DEPTH (ft)	SOIL CLASS	DESCRIPTION
3.0	GM	POORLY GRAVEL WITH SAND AND SILT, brown, moist, dense, fine to medium grained sand, approximately 15% cobbles and 5% boulders.
7.0	SP	SANDY GRAVEL, brown, moist, dense, medium to coarse grained sand, hard digging, approximately 10% cobbles.
16.0	SP	same as above, approximately 5% cobbles.

Figure 12

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

LOGS OF EXPLORATION TT99-29 TO TT99-36

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SUBMITTED BY: ABBAS T. ROODSARI, P.E.
CHIEF, GEOTECHNICAL BRANCH

DESIGNED BY: W H
DRAWN BY: W H
CHECKED BY: JDD

CADD FILE NAME: g04.dgn

DISTRICT FILE NO. NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 18 OF 19 SHEETS

FIGURE 13

TT99-37

DEPTH (ft)	SOIL CLASS	DESCRIPTION
4.0	GP	POORLY GRADED GRAVEL WITH SAND, brown, dry, loose, medium to coarse grained sand, approximately 20% cobbles.
5.0	GP	POORLY GRADED GRAVEL, light brown, wet, fine grained sand, approximately 20% cobbles, less than 5% boulders.
16.0	GP	SANDY GRAVEL, brown, moist to wet, dense, coarse grained sand, hard digging, approximately 20% cobbles.

TT99-38

DEPTH (ft)	SOIL CLASS	DESCRIPTION
1.0	SM	SILTY SAND, very fine to fine sands.
6.0	GP	GRAVELLY SAND WITH COBBLES, brown, moist, loose, medium grained sands, approximately 35% gravels 40%cobbles and 5% boulders.
10.0	GP	SANDY GRAVEL, brown, moist, dense, medium grained sand, approximately 40% gravels, 35% cobbles, and 5% boulders.
16.0	GP	same as above.

TT99-39

DEPTH (ft)	SOIL CLASS	DESCRIPTION
4.0	SP	POORLY GRADED SAND WITH GRAVEL, brown, moist, loose, fine to medium grained sand, approximately 35% gravels, 20% cobbles, and 5% boulders. PCC rubble on surface.
9.0	GP	SANDY GRAVEL, brown, moist, very loose, medium grained sands, approximately 45% gravels, 20% cobbles, and 5% boulders.
12.0	GP	same as above, denser, some silt.
16.0	GW	SANDY GRAVEL, brown,very moist, dense, approximately 50% gravels, 15% cobbles, 5% boulders, medium grained sands.

Figure 13

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

LOGS OF EXPLORATION
TT99-37 TO TT99-39

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: W H
DRAWN BY: W H
CHECKED BY: JDD

SUBMITTED BY:
ABBAS T. ROODSARI, P.E.
CHIEF, GEOTECHNICAL BRANCH

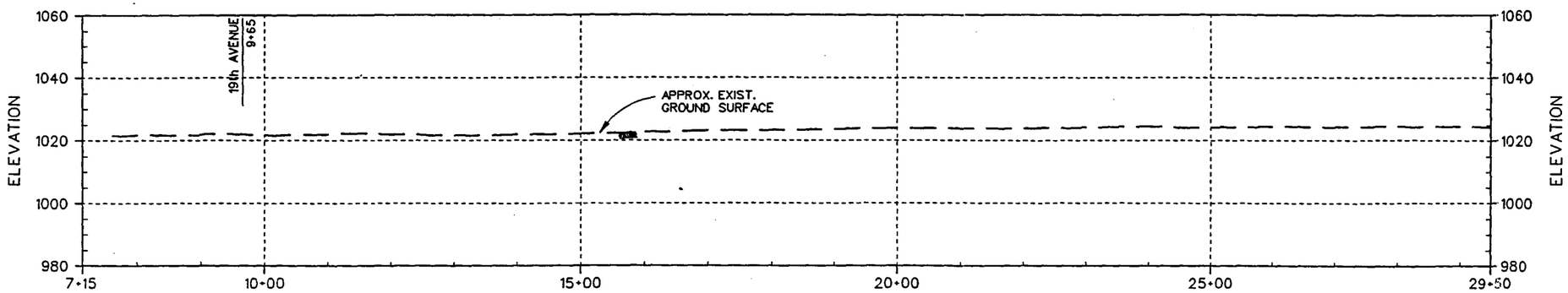
CADD FILE NAME: g06.dgn

DISTRICT FILE NO. NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 19 OF 19 SHEETS

FIGURE 14

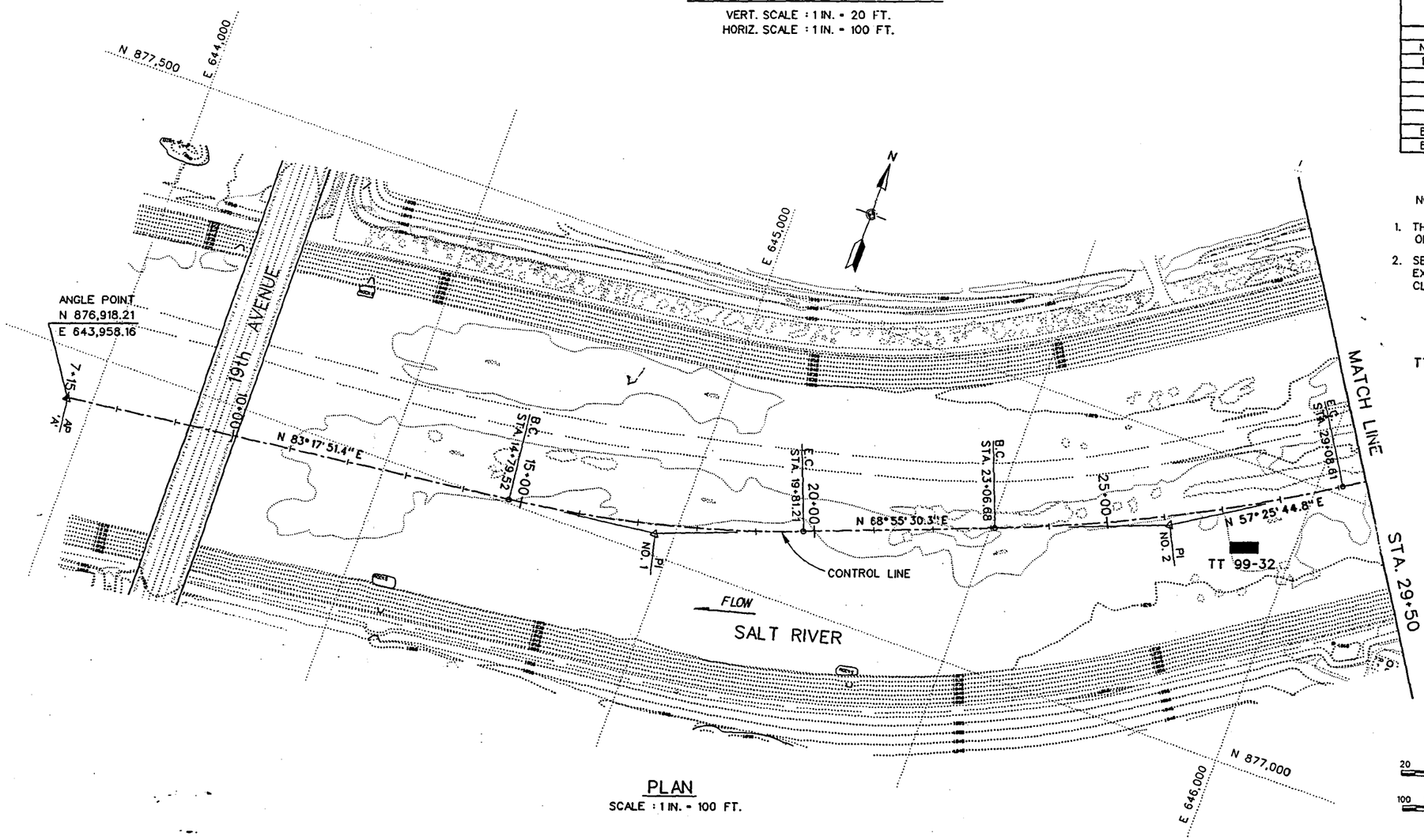
Figure 14



PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE		
P.I. NO.	1	2
NORTHING	877,036.87	877,353.17
EASTING	644,967.90	645,788.68
Δ=	14° 22' 21.1"	11° 29' 45.5"
R=	2,000'	3,000'
T=	252.17'	301.98'
L=	501.70'	601.93'
B.C. STA.	14+79.52	23+06.68
E.C. STA.	19+81.21	29+08.61



PLAN

SCALE : 1 IN. = 100 FT.

NOTES:

1. THIS SHEET IS FOR INFORMATION PURPOSES ONLY. NO GRADING IN THIS AREA.
2. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.

TT 99-32 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'

100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 7+15 TO STA. 29+50

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: CC / MO
DRAWN BY: MO
CHECKED BY: DC

SUBMITTED BY:
THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

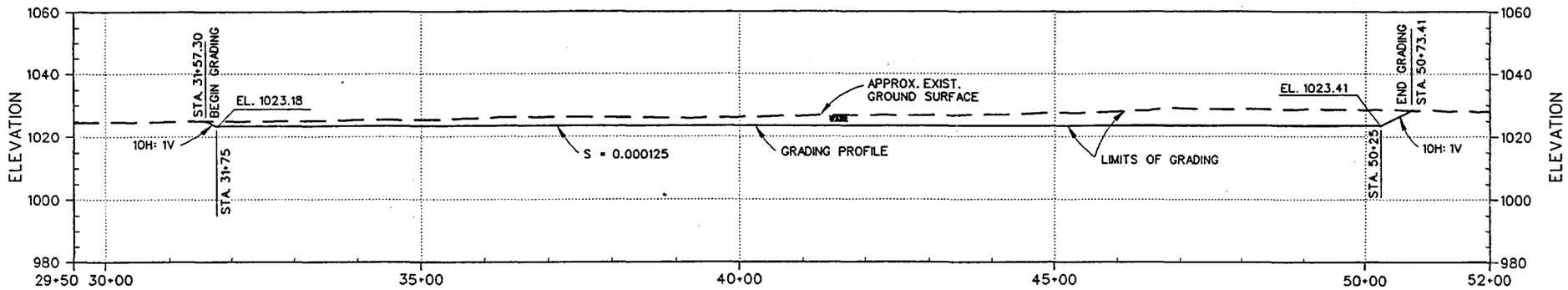
CADD FILE NAME: C14.dgn

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 3 OF 19 SHEETS

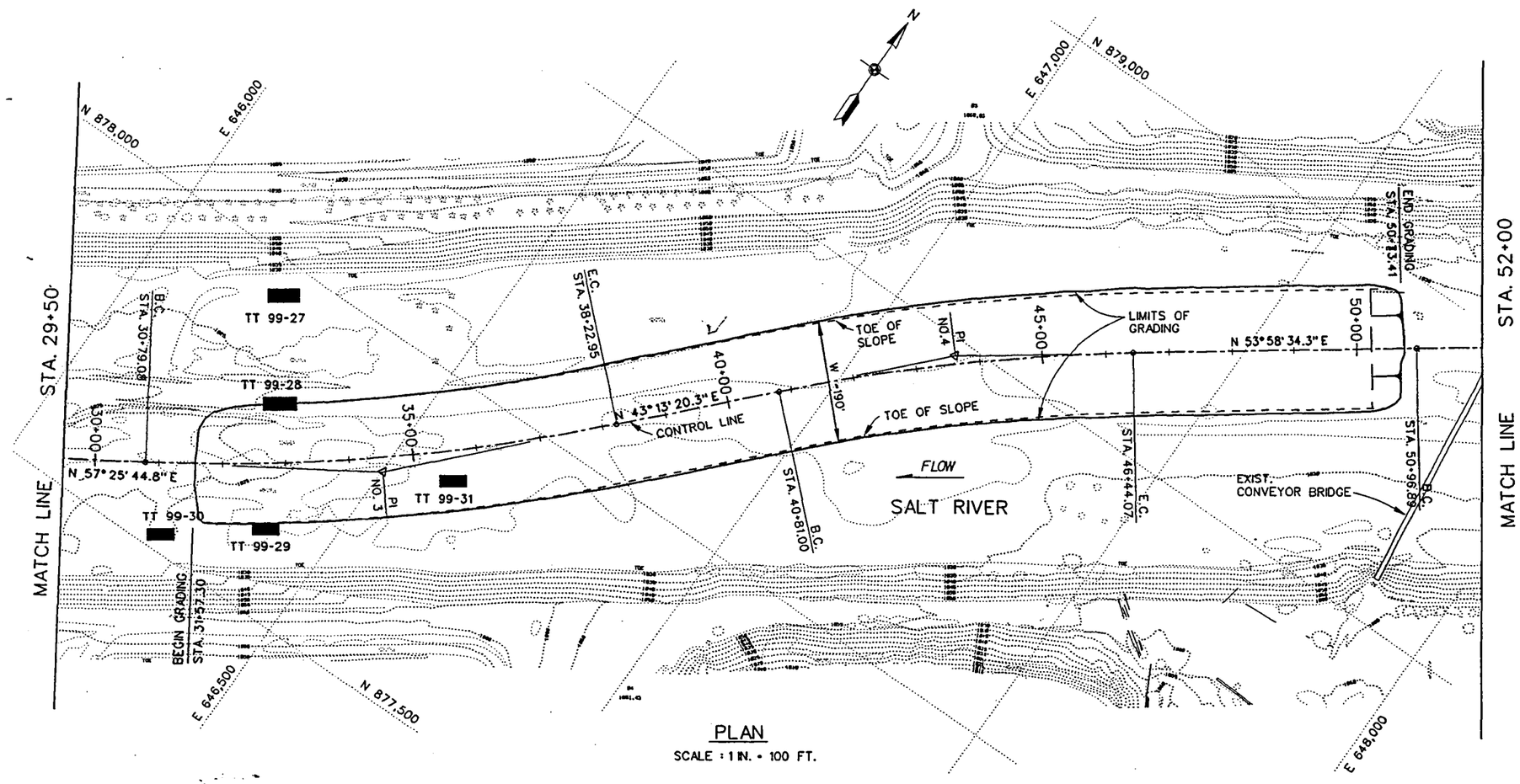
SAFETY PAYS

Figure 15



PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.



PLAN

SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE		
P.I. NO.	3	4
NORTHING	877,808.77	878,475
EASTING	646,501.88	647,128
Δ=	14° 12' 24.5"	10° 45' 14.1"
R=	3,000'	3,000'
T=	373.85'	282.37'
L=	743.87'	563.07'
B.C. STA.	30+79.08	40+81.00
E.C. STA.	38+22.95	46+44.07

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-27 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'
100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 29+50 TO STA. 52+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

DESIGNED BY: CC / MD
DRAWN BY: MD
CHECKED BY: DC

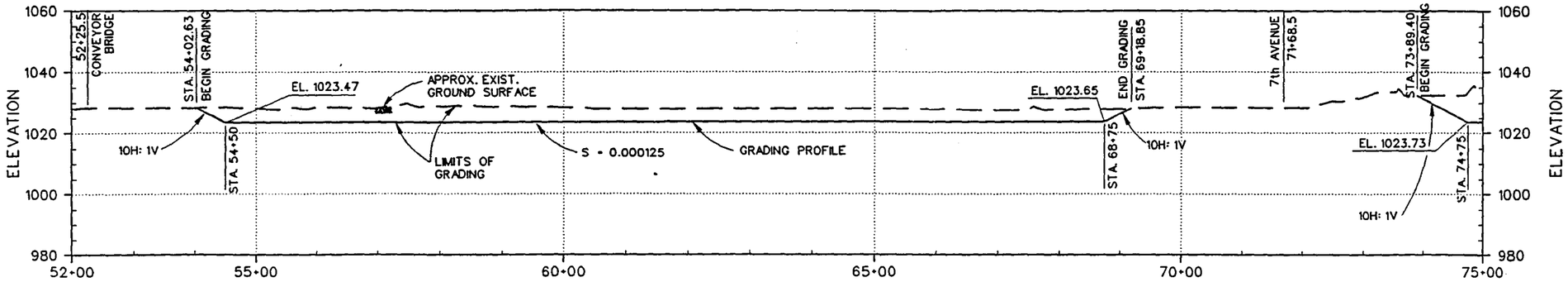
CADD FILE NAME: C2.dgn

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 4 OF 19 SHEETS

SAFETY PAYS

Figure 16

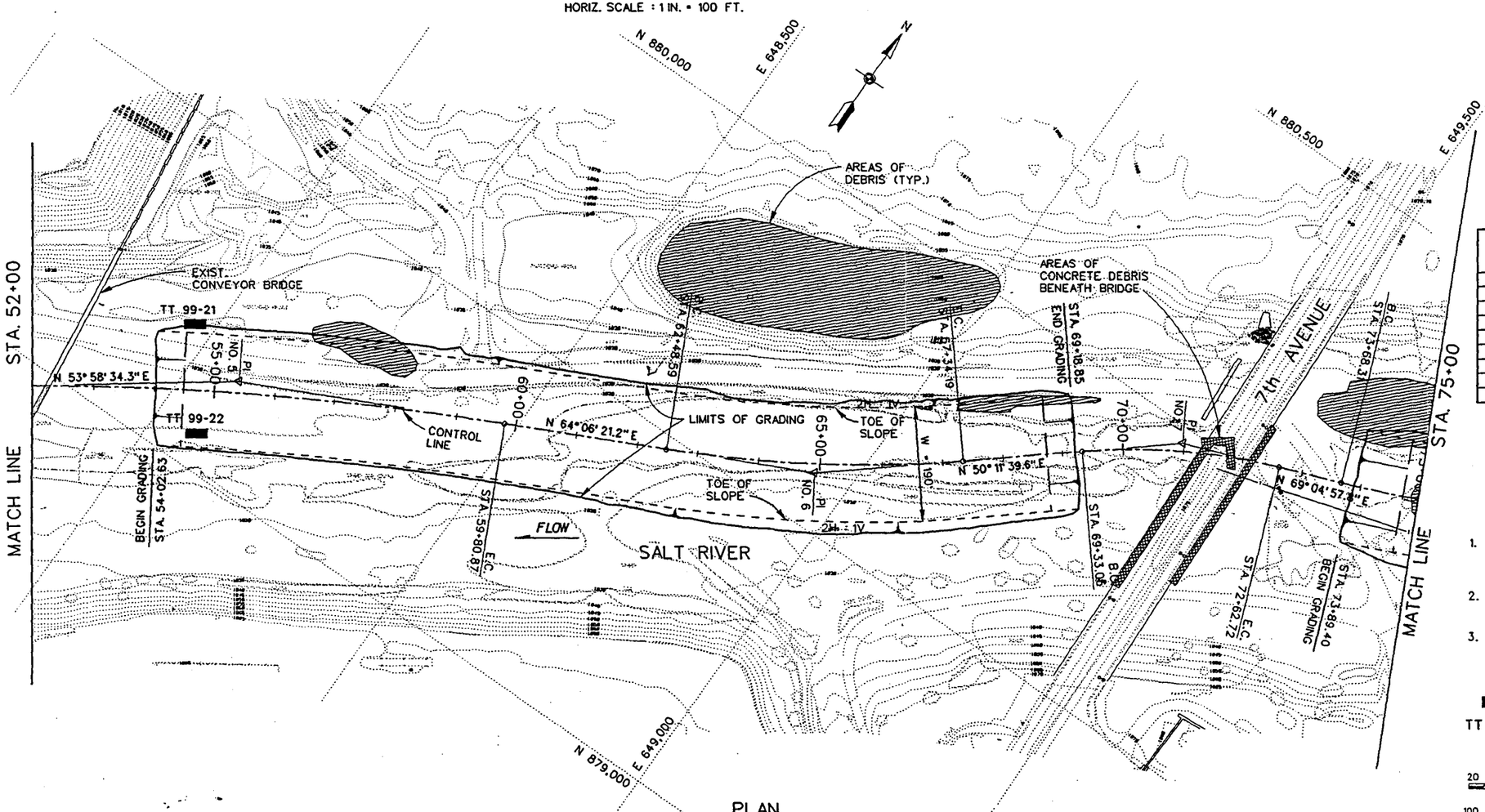


PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE	
P.I. NO.	5
NORTHING	879,168
EASTING	648,081
Δ=	10° 07' 46.7"
R=	5,000'
T=	443.14'
L=	883.98'
B.C. STA.	50+96.89
E.C. STA.	59+80.87

CURVE DATA AT CONTROL LINE		
P.I. NO.	6	7
NORTHING	879,585	879,975
EASTING	648,940	649,408
Δ=	13° 54' 41.3"	18° 53' 18.4"
R=	2,000'	1,000'
T=	244.00'	166.34'
L=	485.60'	329.67'
B.C. STA.	62+48.59	69+33.05
E.C. STA.	67+34.19	72+62.72



PLAN

SCALE : 1 IN. = 100 FT.

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-21 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'

100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)
ROUGH GRADING, PLAN AND PROFILE
STA. 52+00 TO STA. 75+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

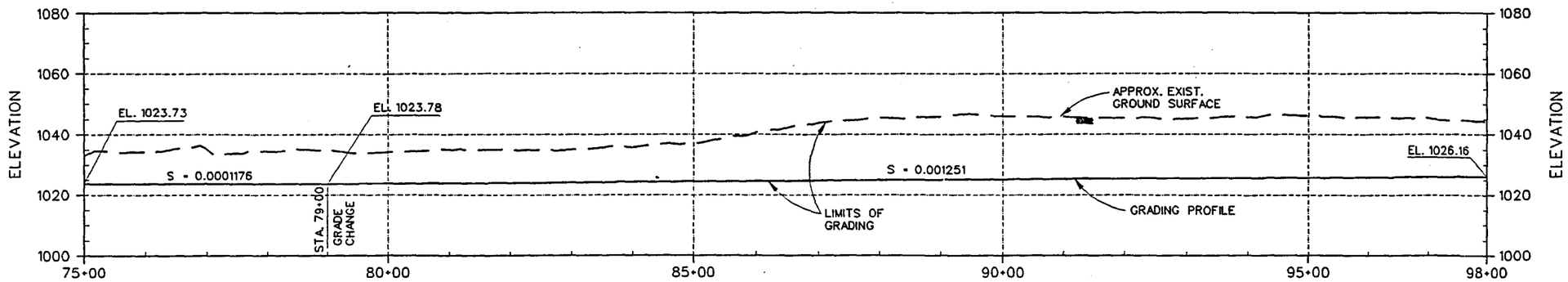
DESIGNED BY: CC / WD
DRAWN BY: WD
CHECKED BY: DC

CADD FILE NAME: 03.dgn

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 5 OF 19 SHEETS

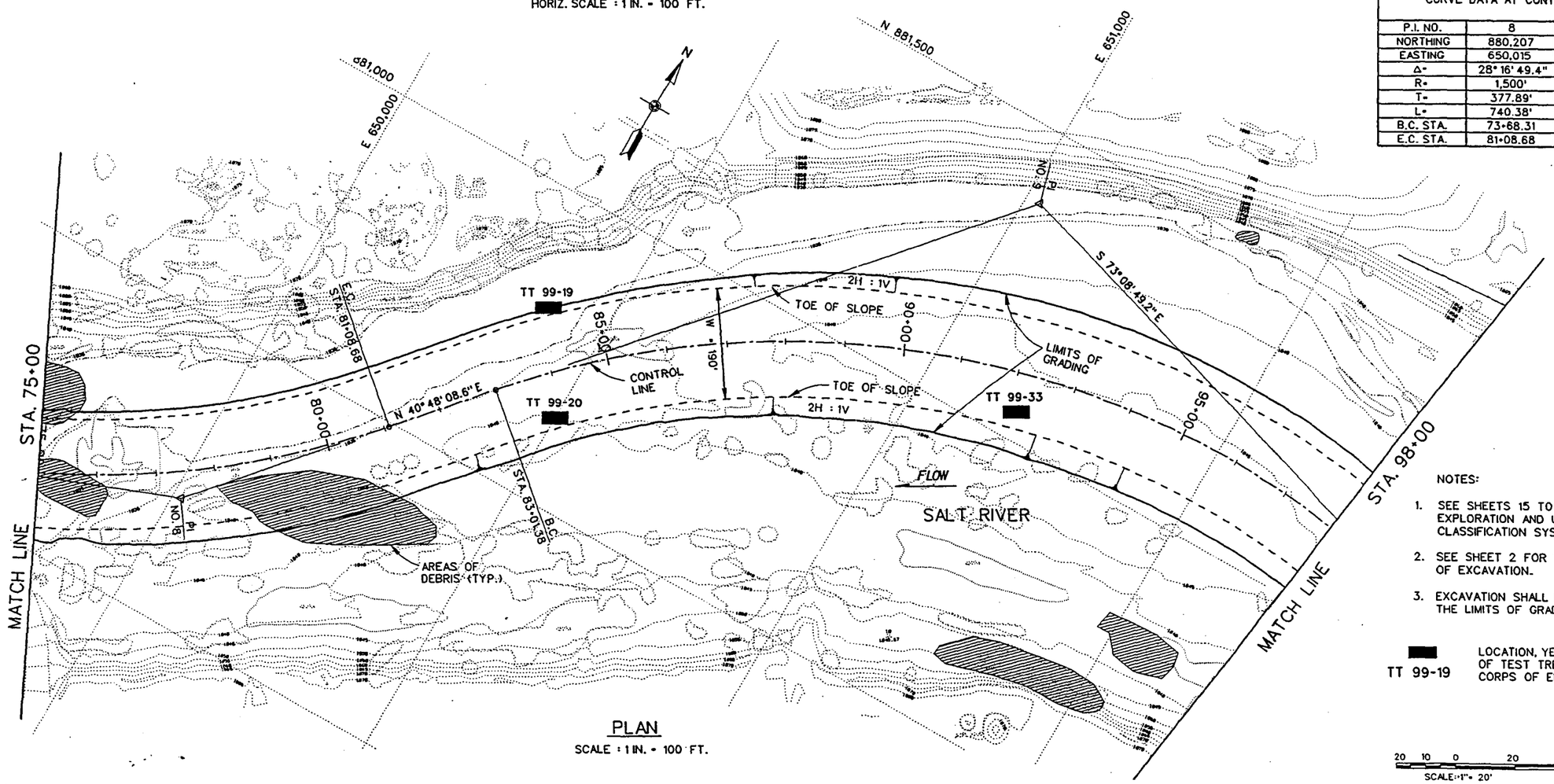
Figure 17



PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE		
P.I. NO.	8	9
NORTHING	880,207	881,377
EASTING	650,015	651,025
Δ=	28° 16' 49.4"	66° 03' 02.2"
R=	1,500'	1,500'
T=	377.89'	975.05'
L=	740.38'	1,729.20'
B.C. STA.	73+68.31	83+01.38
E.C. STA.	81+08.68	100+30.58



PLAN

SCALE : 1 IN. = 100 FT.

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-19 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1" = 20'

100 50 0 100 200FT
SCALE: 1" = 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 75+00 TO STA. 98+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

DESIGNED BY: CC / MO
DRAWN BY: MD
CHECKED BY: DC

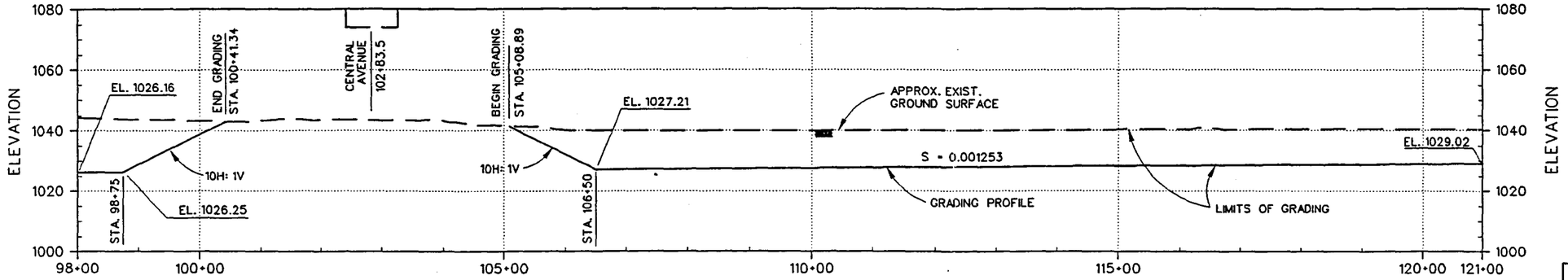
CADD FILE NAME: c4.dgn

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 6 OF 19 SHEETS

SAFETY PAYS

Figure 18

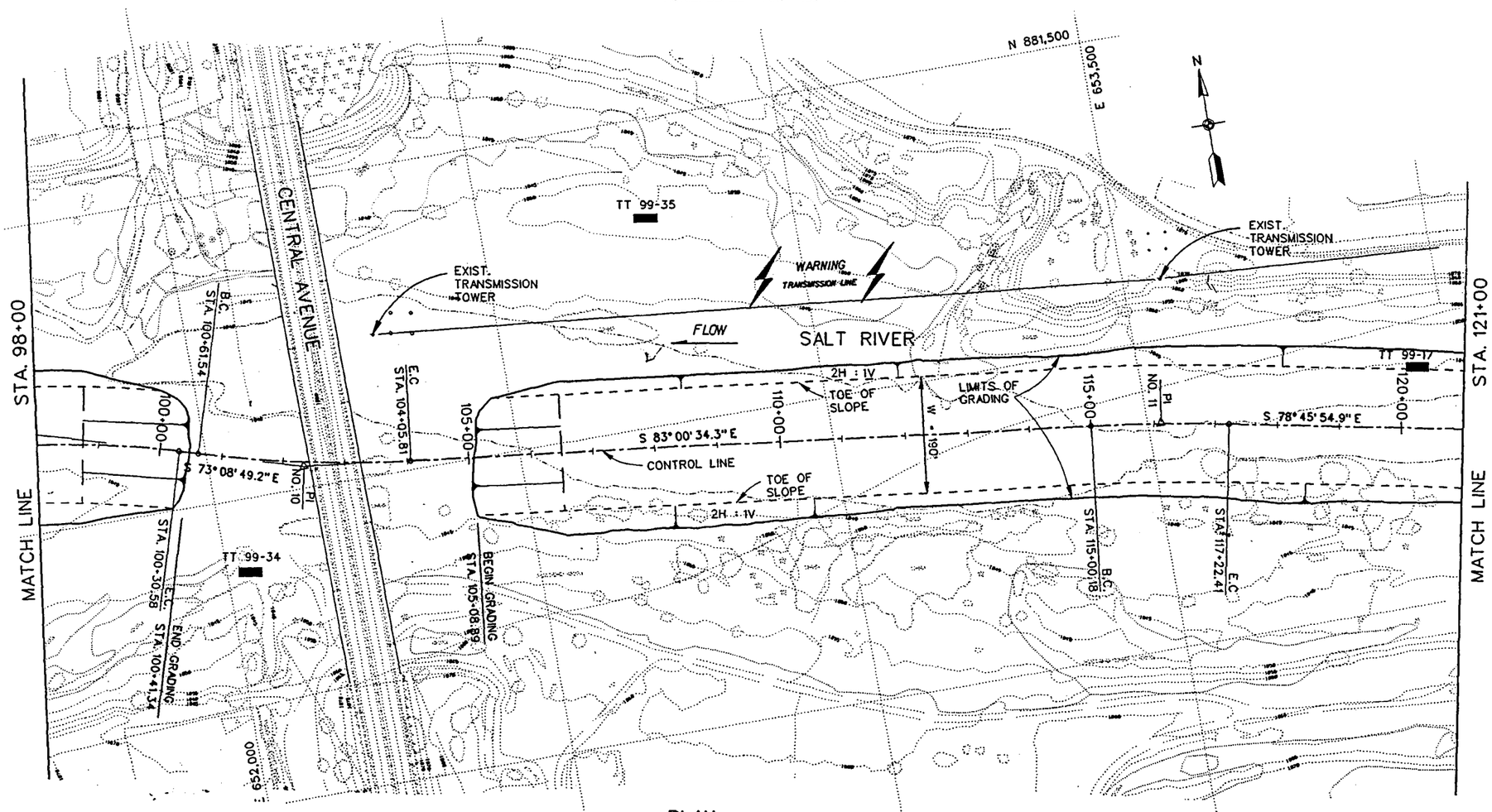


PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE	
P.I. NO.	10
NORTHING	881,035.31
EASTING	652,152.96
Δ=	9° 51' 45.1"
R=	2,000'
T=	172.56'
L=	344.27'
B.C. STA.	100+61.54
E.C. STA.	104+05.81

CURVE DATA AT CONTROL LINE	
P.I. NO.	11
NORTHING	880,867.59
EASTING	653,520.81
Δ=	4° 14' 39.4"
R=	3,000'
T=	111.17'
L=	222.23'
B.C. STA.	115+00.18
E.C. STA.	117+22.41



PLAN

SCALE : 1 IN. = 100 FT.

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-34 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'
100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 98+00 TO STA. 121+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

SUBMITTED BY:

DESIGNED BY: CC / WD
DRAWN BY: WD
CHECKED BY: DC

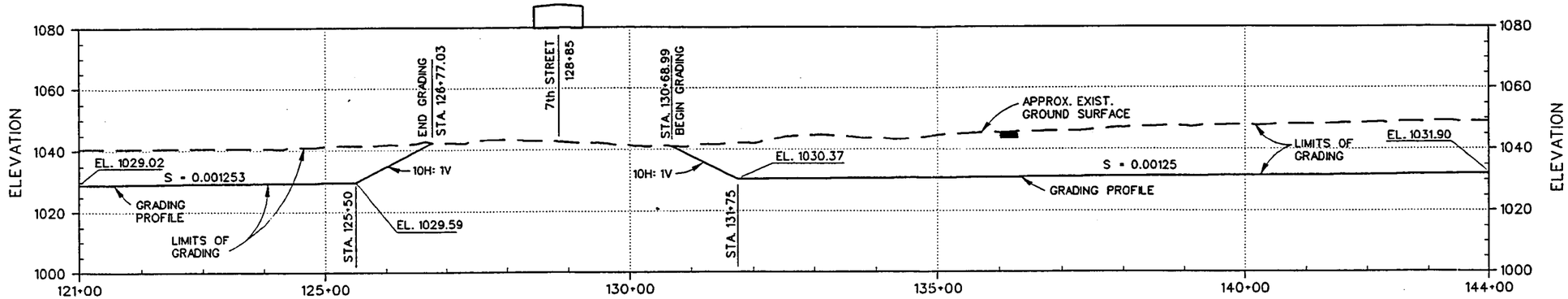
CADD FILE NAME: C5.dgn

SPEC. NO.: NOT APPLICABLE

DISTRICT FILE NO.: NOT APPLICABLE

SHEET 7 OF 19 SHEETS

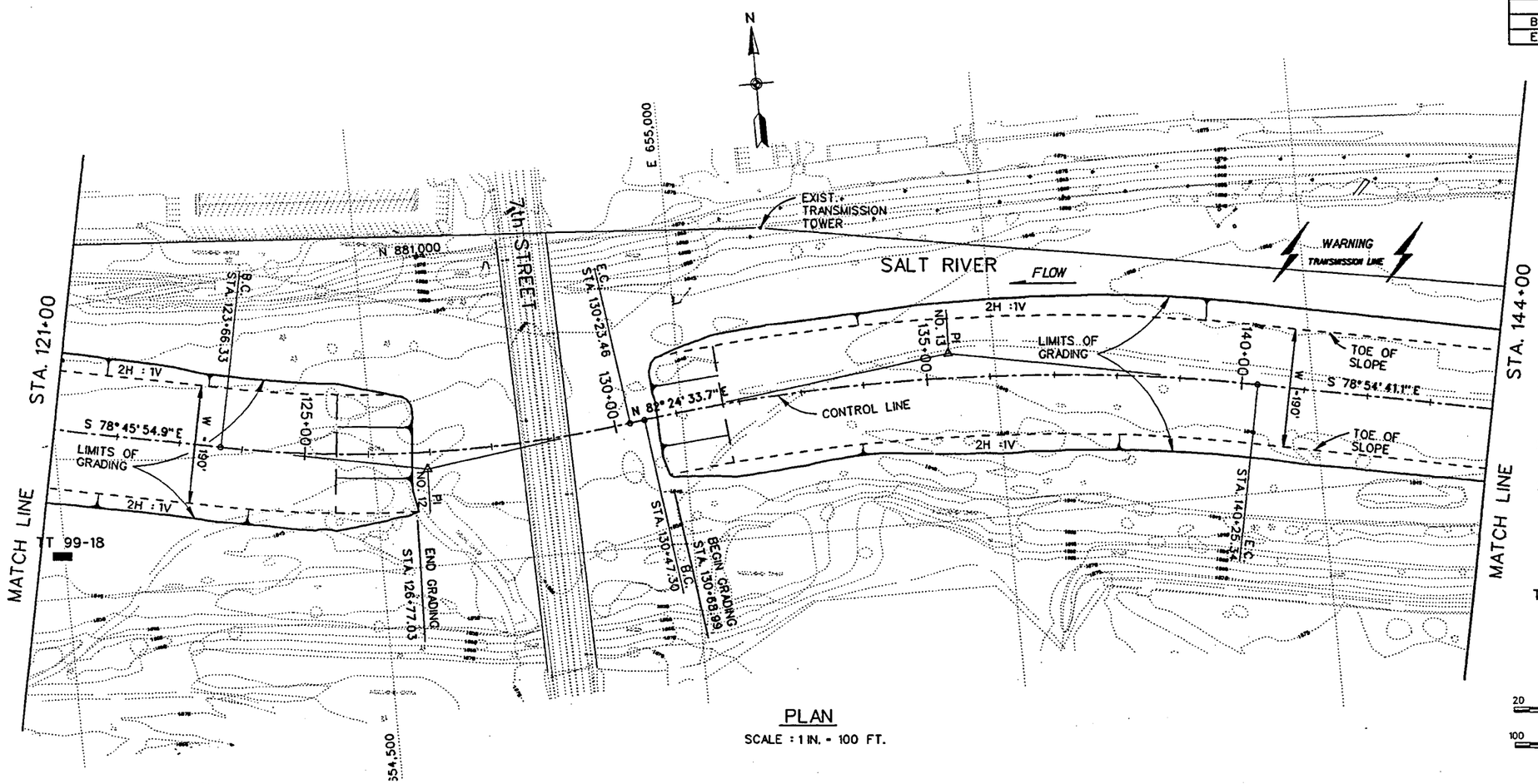
Figure 19



PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE		
P.I. NO.	12	13
NORTHING	880,655.88	880,768
EASTING	654,586.63	655,427.98
Δ=	18° 49' 31.4"	18° 40' 45.2"
R=	2,000'	3,000'
T=	331.55'	493.40'
L=	657.13'	978.04'
B.C. STA.	123+66.33	130+47.30
E.C. STA.	130+23.46	140+25.34



PLAN

SCALE : 1 IN. = 100 FT.

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-18 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'

100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 121+00 TO STA. 144+00

DESIGNED BY: CC / MD
DRAWN BY: MD
CHECKED BY: DC

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

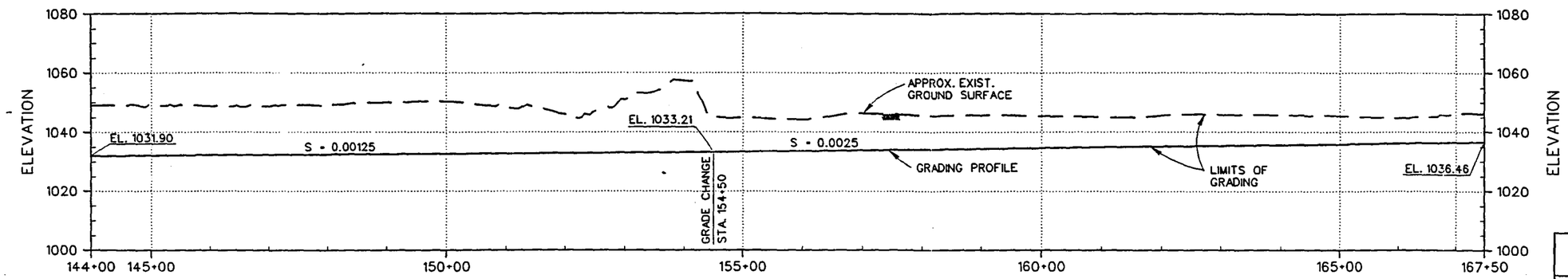
SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

CADD FILE NAME: ch4.dgn

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 8 OF 19 SHEETS

Figure 20

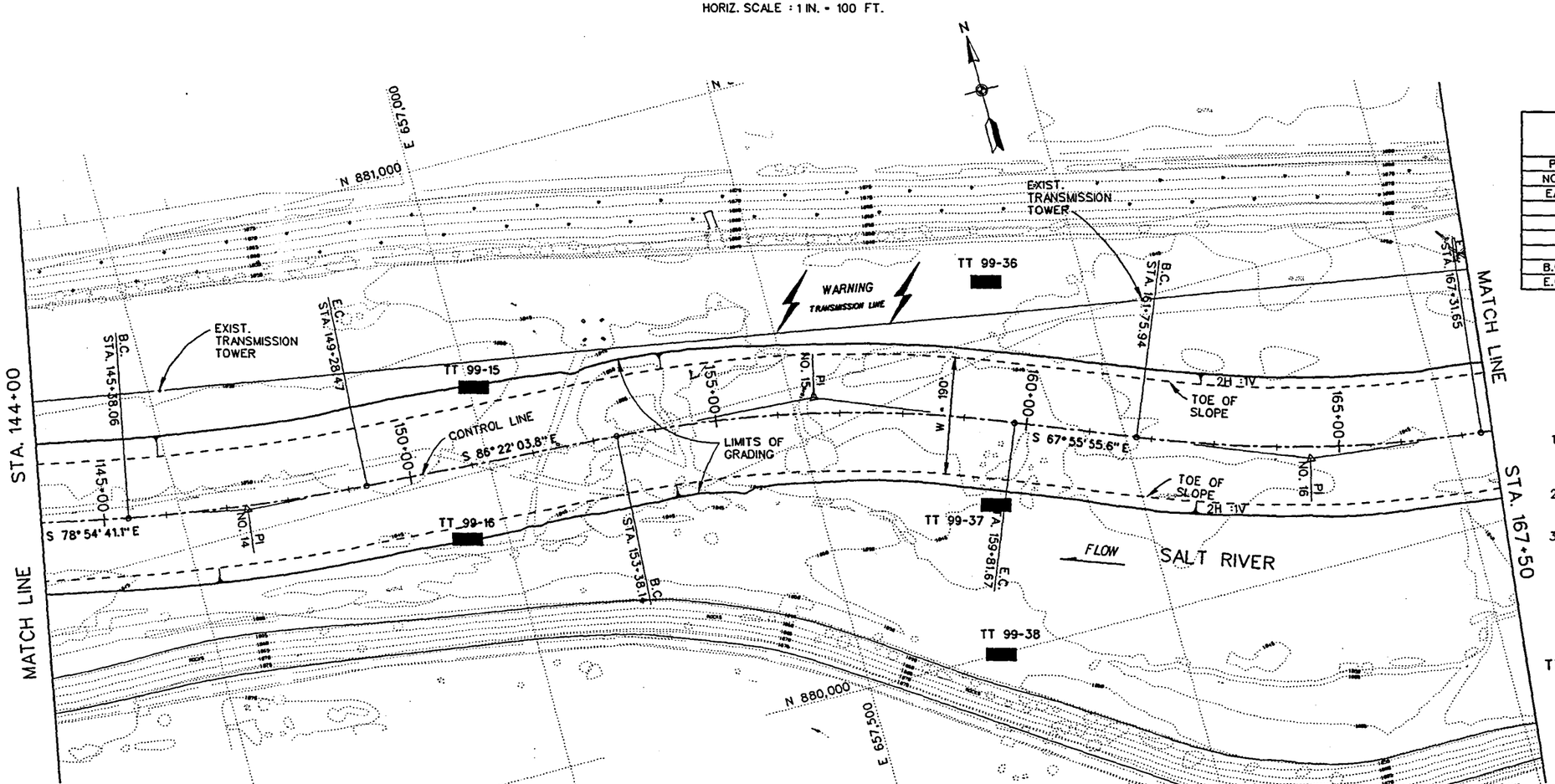


PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE	
P.I. NO.	16
NORTHING	880,178
EASTING	658,275
Δ=	15° 55' 12.4"
R=	2,000'
T=	279.66'
L=	555.72'
B.C. STA.	161+75.94
E.C. STA.	167+31.65

CURVE DATA AT CONTROL LINE		
P.I. NO.	14	15
NORTHING	880,536.90	880,478
EASTING	656,607.15	657,535
Δ=	7° 27' 22.7"	18° 26' 08.3"
R=	3,000'	2,000'
T=	195.48'	324.57'
L=	390.41'	643.52'
B.C. STA.	145+38.06	153+38.14
E.C. STA.	149+28.47	159+81.67



PLAN

SCALE : 1 IN. = 100 FT.

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-38 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'

100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 144+00 TO STA. 167+50

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

SUBMITTED BY:

DESIGNED BY: CC / MB
DRAWN BY: MB
CHECKED BY: DC

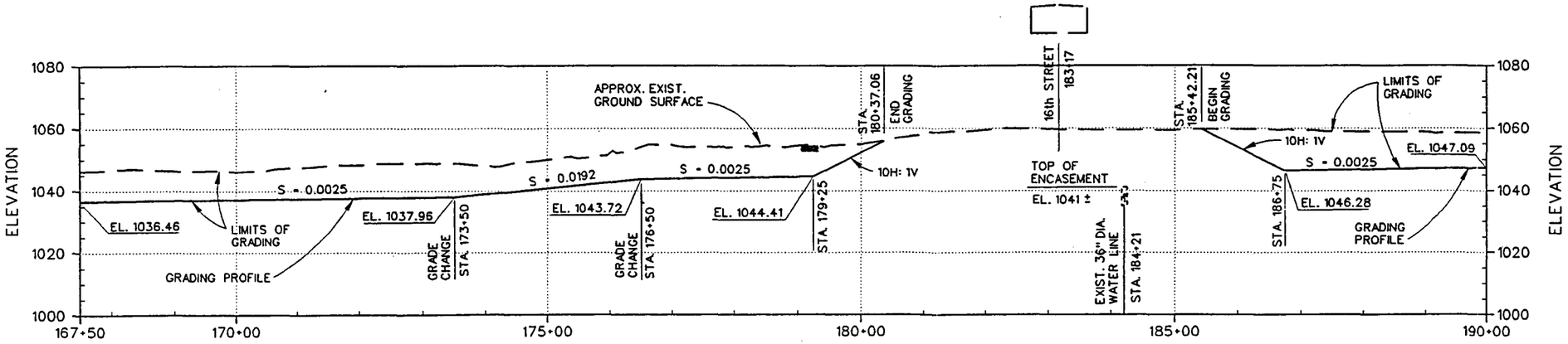
CADD FILE NAME: 67.dgn

DISTRICT FILE NO.: NOT APPLICABLE

SPEC. NO.: NOT APPLICABLE

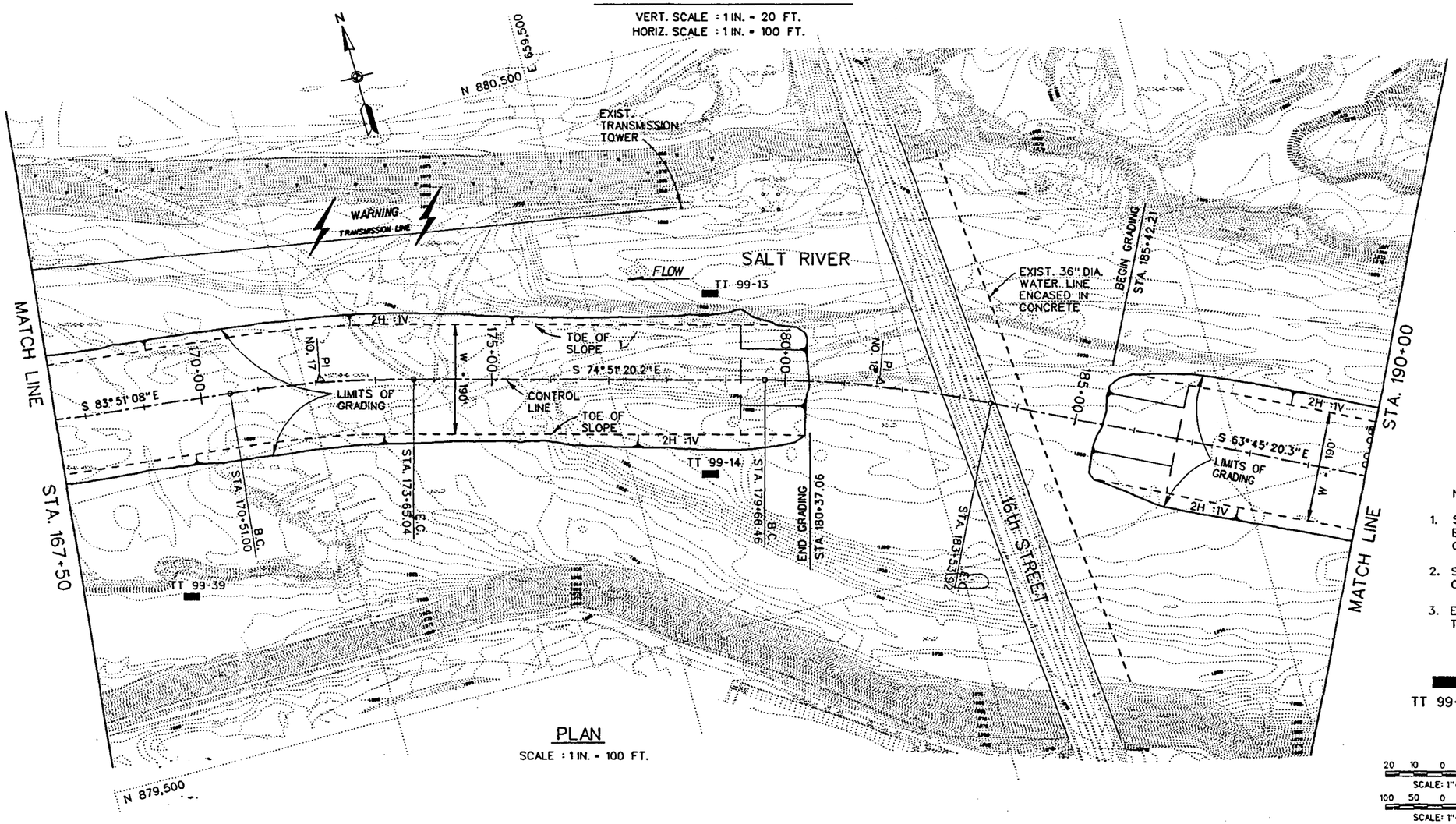
SHEET 19 OF 19 SHEETS

Figure 21



PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.



PLAN

SCALE : 1 IN. = 100 FT.

℄ CURVE DATA	
P.I. NO.	17
NORTHING	880,097
EASTING	659,027
Δ=	8° 59' 47.8"
R=	2,000'
T=	157.34'
L=	314.04'
B.C. STA.	170+51.00
E.C. STA.	173+65.04

℄ CURVE DATA	
P.I. NO.	18
NORTHING	879,848
EASTING	659,947
Δ=	11° 05' 59.9"
R=	2,000'
T=	194.34'
L=	387.46'
B.C. STA.	179+66.46
E.C. STA.	183+53.92

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-13 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'
100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL

REVISIONS

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 167+50 TO STA. 190+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

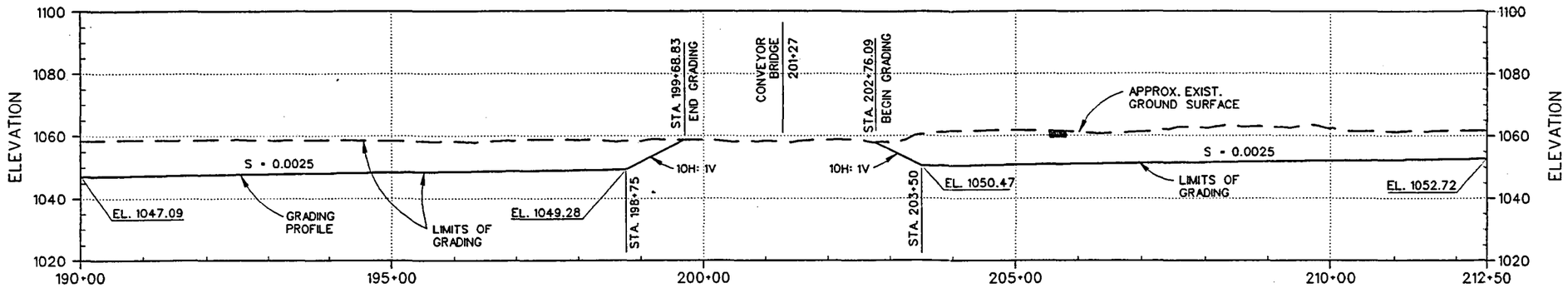
DESIGNED BY: CC / MD
DRAWN BY: MD
CHECKED BY: DC

CADD FILE NAME: 08.dgn

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

SHEET 19 OF 10 SHEETS

Figure 22



PROFILE AT CONTROL LINE

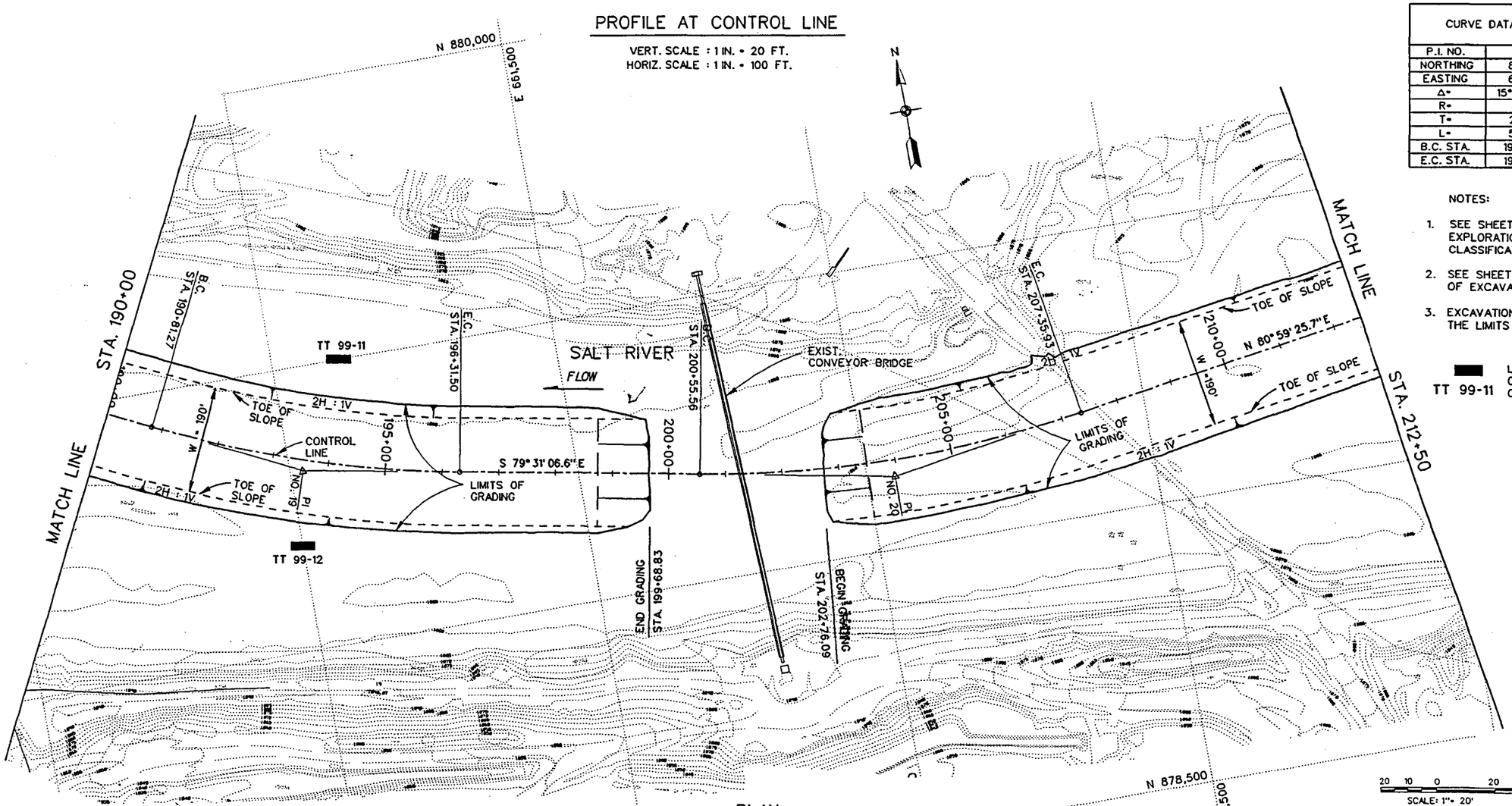
VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE		
P.I. NO.	19	20
NORTHING	879,318	879,128
EASTING	661,022	662,049
Δ=	15° 45' 46.4"	19° 29' 27.7"
R=	2,000'	2,000'
T=	276.86'	343.50'
L=	550.23'	680.36'
B.C. STA.	190+81.27	200+55.56
E.C. STA.	196+31.50	207+35.93

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-11 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS



PLAN

SCALE : 1 IN. = 100 FT.

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 190+00 TO STA. 212+50

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

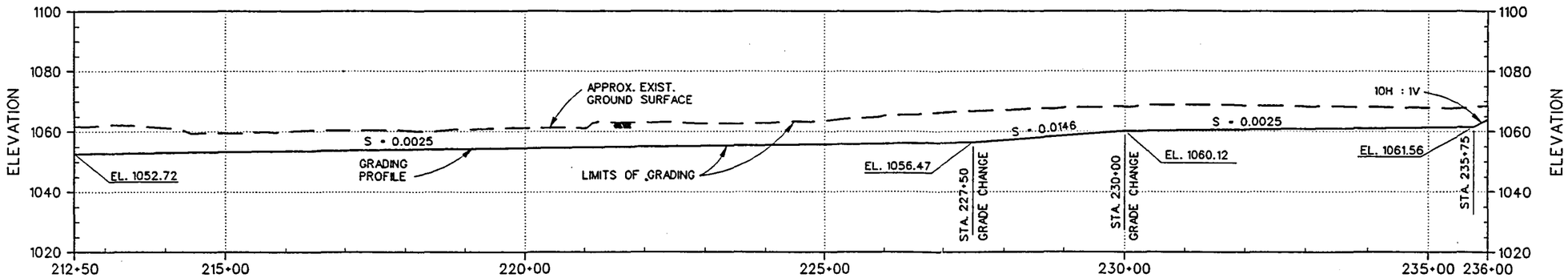
SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

DESIGNED BY: CC / WD
DRAWN BY: RD
CHECKED BY: DC

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE
CADD FILE NAME: 18.dgn

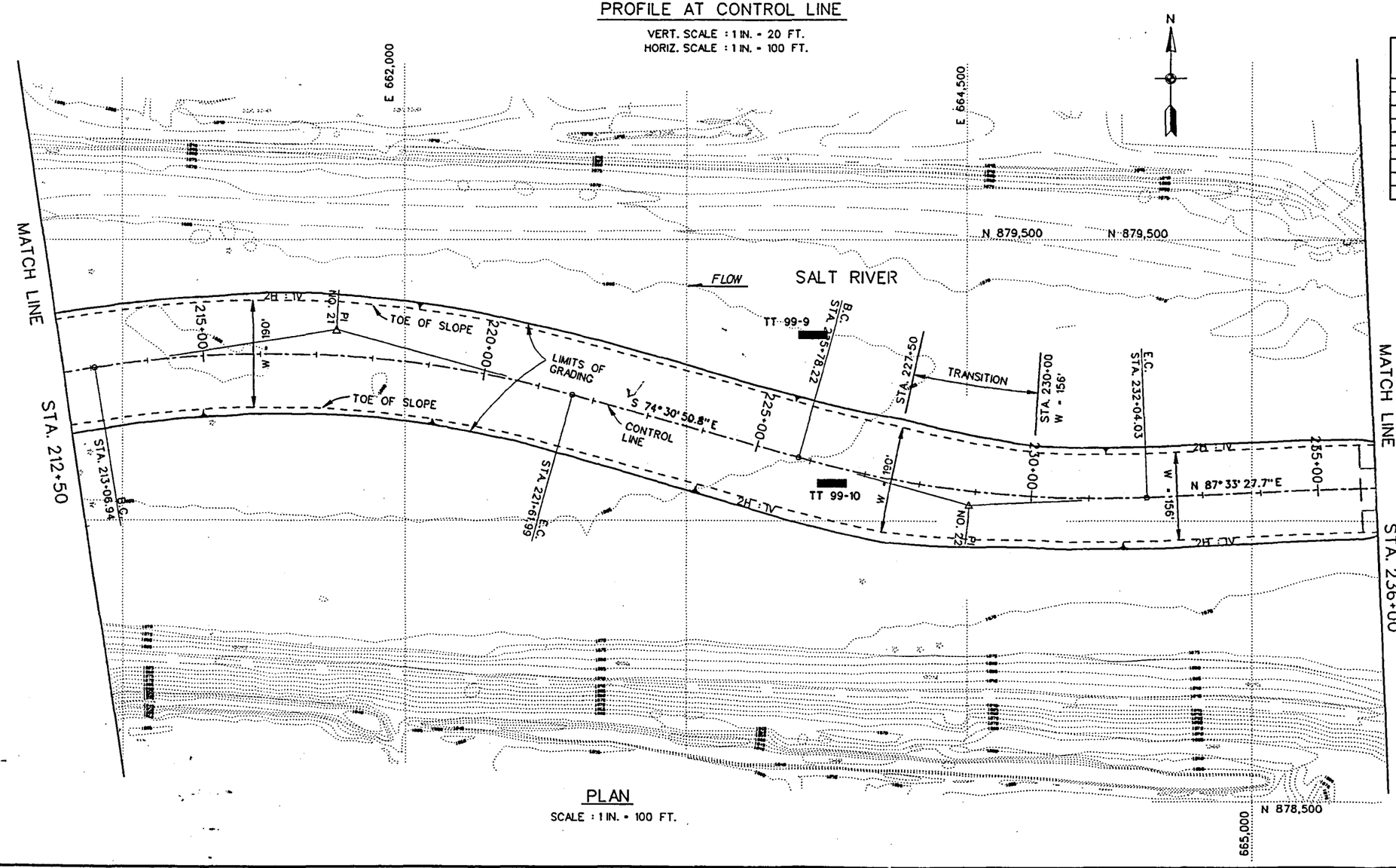
SHEET 11 OF 19 SHEETS

Figure 23



PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.



PLAN

SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE		
P.I. NO.	21	22
NORTHING	879,339.20	879,027.91
EASTING	663,381.03	664,504.58
Δ=	24° 29' 43.5"	17° 55' 41.5"
R=	2,000'	2,000'
T=	434.16'	315.48'
L=	855.05'	625.81'
B.C. STA.	213+06.94	225+78.22
E.C. STA.	221+61.99	232+04.03

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-9 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'

100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)
ROUGH GRADING, PLAN AND PROFILE
STA. 212+50 TO STA. 236+00

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

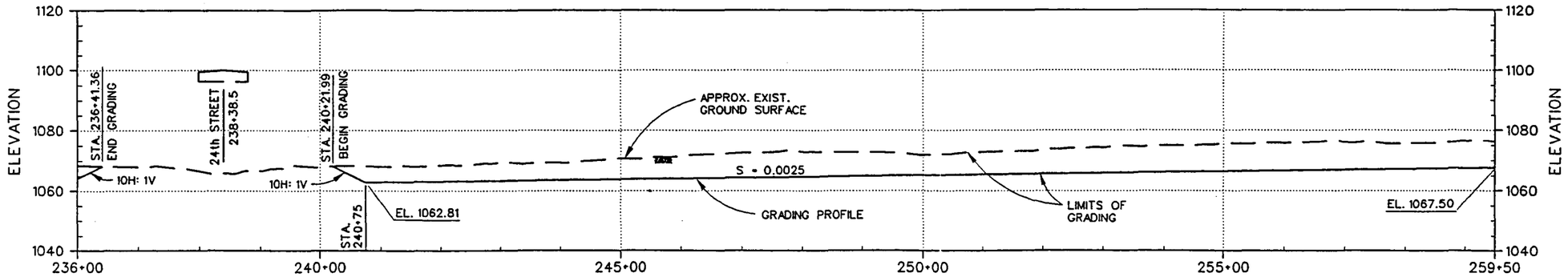
SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

DESIGNED BY: CC / MD
DRAWN BY: MD
CHECKED BY: DC

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE
CADD FILE NAME: 10.dgn

SHEET 19 OF 12 SHEETS

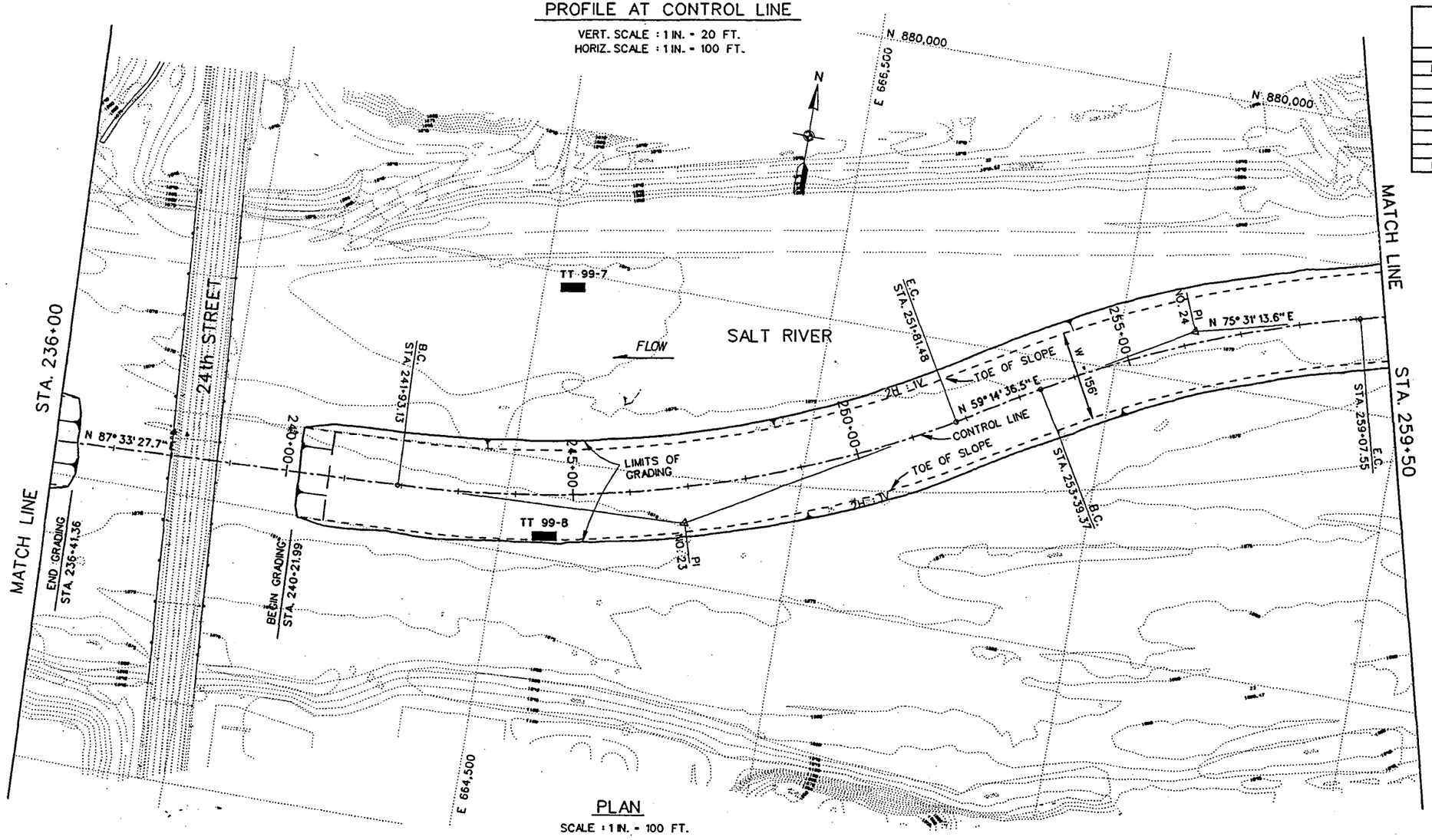
Figure 24



PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.

CURVE DATA AT CONTROL LINE		
P.I. NO.	23	24
NORTHING	879,105	879,590
EASTING	666,312	667,127
Δ=	28° 18' 51.3"	16° 16' 37.2"
R=	2,000'	2,000'
T=	504.49'	286.01'
L=	988.35'	568.17'
B.C. STA.	241+93.13	253+39.37
E.C. STA.	251+81.48	259+07.55



PLAN

SCALE : 1 IN. = 100 FT.

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-8 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'

100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 236+00 TO STA. 259+50

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

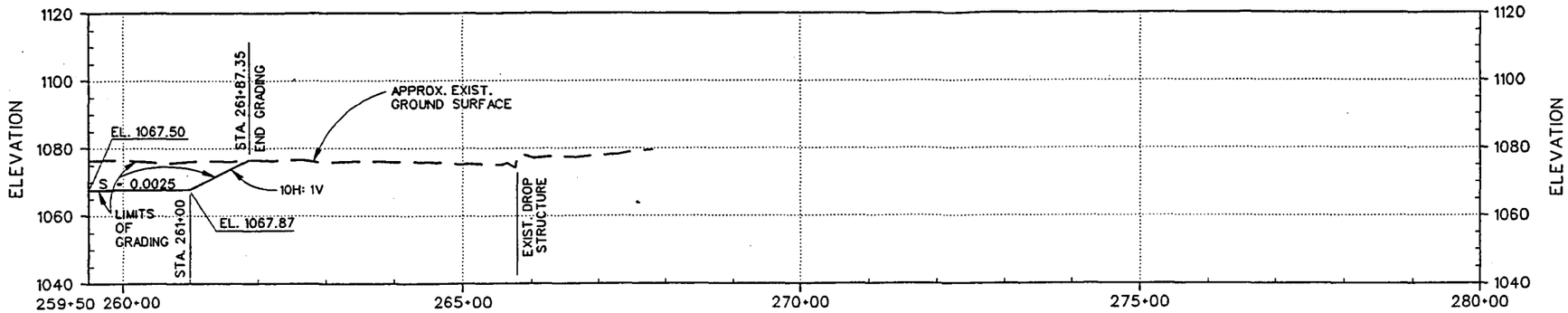
DESIGNED BY: CC / MD
DRAWN BY: MD
CHECKED BY: DC

CADD FILE NAME: CH.dgn

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE

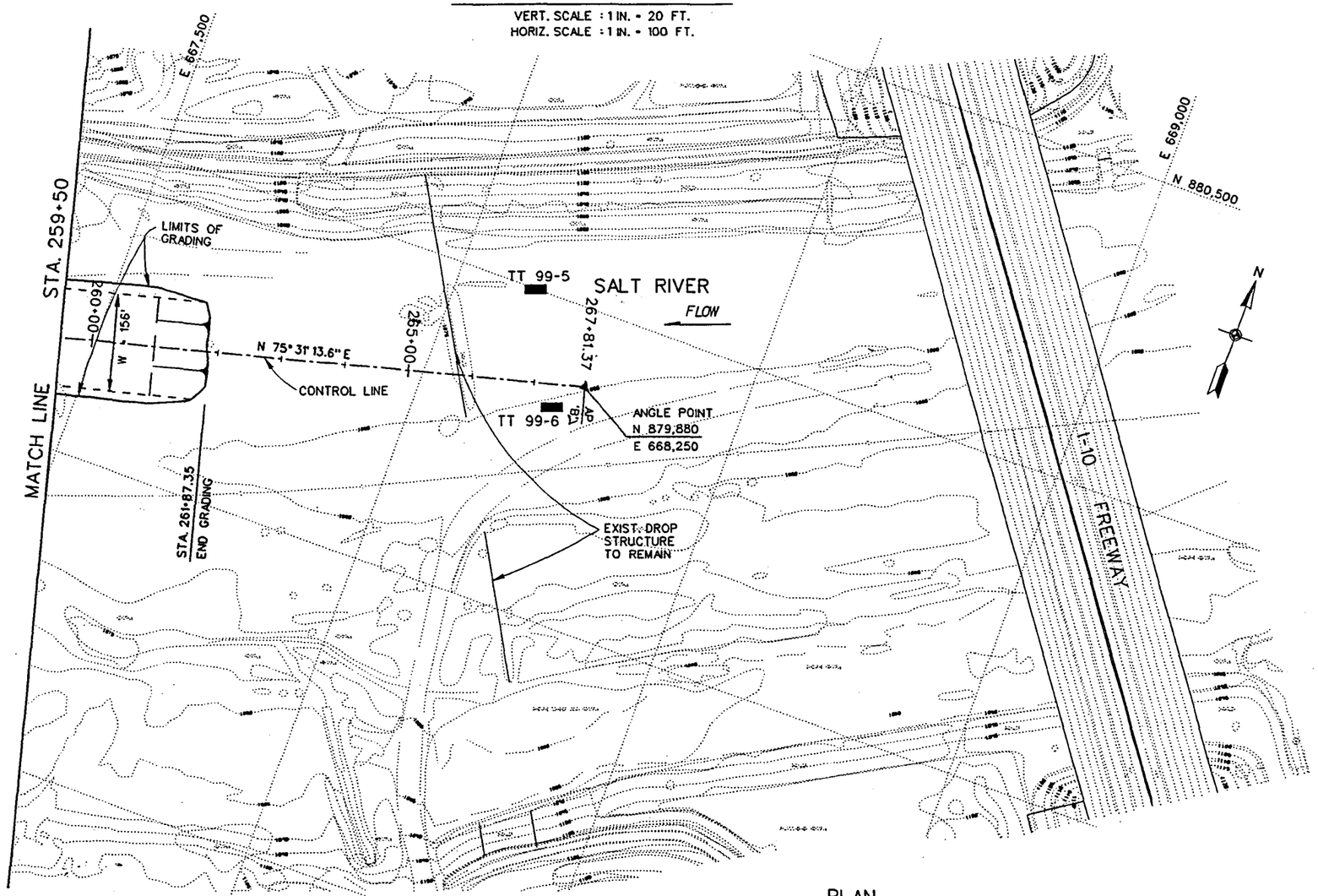
SHEET 19 OF 13 SHEETS

Figure 25



PROFILE AT CONTROL LINE

VERT. SCALE : 1 IN. = 20 FT.
HORIZ. SCALE : 1 IN. = 100 FT.



PLAN

SCALE : 1 IN. = 100 FT.

NOTES:

1. SEE SHEETS 15 TO 19 FOR LOGS OF EXPLORATION AND UNIFIED SOIL CLASSIFICATION SYSTEM.
2. SEE SHEET 2 FOR TYPICAL SECTION OF EXCAVATION.
3. EXCAVATION SHALL NOT EXCEED THE LIMITS OF GRADING SHOWN.

TT 99-5 — LOCATION, YEAR AND NUMBER OF TEST TRENCH BY CORPS OF ENGINEERS

20 10 0 20 40FT
SCALE: 1"= 20'

100 50 0 100 200FT
SCALE: 1"= 100'

SYMBOL	DESCRIPTIONS	DATE	APPROVAL
	REVISIONS		

SALT RIVER, MARICOPA COUNTY, ARIZONA
RIO SALADO, PHOENIX REACH
(19TH AVENUE TO I-10 FREEWAY)

ROUGH GRADING, PLAN AND PROFILE
STA. 259+50 TO STA. 267+81.37

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

SUBMITTED BY: THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

DESIGNED BY: CC / MD
DRAWN BY: MD
CHECKED BY: DC

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE
CADD FILE NAME: c12.dgn

SHEET 19 OF 14 SHEETS

Section B

MATERIALS

RIO SALADO
REPORT ON RCC FOR CONSTRUCTION

1. PURPOSE AND SCOPE
2. EXPLORATION
3. CEMENTITIOUS AND POZZOLANIC MATERIALS
    - 3.1 GENERAL
    - 3.2 PORTLAND CEMENT
        - a. *Types.*
        - b. *Testing Requirements.*
    - 3.3 FLY ASHES
        - a. *Class F.*
        - b. *Testing Requirements.*
    - 3.4 WATER
4. BORROW MATERIALS
    - 4.1 SAMPLING
    - 4.2 FIELD PROCESSING (by The Bureau of Reclamation {BUREC})
    - 4.3 PROCESSING BY TANNER-UNITED METRO
    - 4.4 LABORATORY PROCESSING
5. LABORATORY PROGRAM
    - 5.1 GENERAL
    - 5.2 PHYSICAL PROPERTIES OF AGGREGATES
    - 5.3 MIX DESIGN STUDIES
    - 5.4 CEMENT FLY ASH REPLACEMENT STUDIES
    - 5.5 ABRASION EROSION TEST DATA
    - 5.6 MIX DESIGN EVALUATIONS
6. AGGREGATE PROCESSING COSTS
    - 6.1 COMMERCIALLY AVAILABLE PRICE OF MATERIALS
    - 6.2 ESTIMATED COST OF PROCESSING AGGREGATES
7. DESIGN RECOMMENDATIONS
    - 7.1 AGGREGATE SELECTION
    - 7.2 CEMENT/FLY ASH COMBINATION

RIO SALADO PROJECT
REPORT ON RCC FOR CONSTRUCTION

1. PURPOSE AND SCOPE

The purpose of this report is to supply information and design alternatives for construction of Roller Compacted Concrete (RCC) structures as part of the Rio Salado Restoration Project. The report covers field investigations for potential sources of aggregates, laboratory and economic analysis of materials available, and recommendations for production of RCC for use in the subject project.

2. EXPLORATION

Due to the relative uniformity of the materials available, the relatively shallow and short nature of the structures to be built, explorations, for both the foundation investigations and potential borrow sources for RCC aggregates were carried out at the same time. In conjunction with these studies a detailed study of material types and stratification for a sediment transport study was carried out. Additional fine grained materials suitable to support planting and lining of ponds and channels were desired for construction of the planned project.

The materials encountered during the explorations were generally, cobbles, gravels, and sands. Naturally occurring fine grained materials, eg fine sands and silts, were not found in significant amounts in any of the on-site explorations.

Based on this lack of materials additional surveys of the existing aggregate suppliers in the immediate vicinity of the project were made. These surveys also confirmed that the local sources were short of fine grained materials. Most of the fine grained materials used by the sand and gravel operators is produced from crushing and screening operations.

3. CEMENTITIOUS AND POZZOLANIC MATERIALS

Based on the high cost of cement available, in the area, combinations of cement and fly ash were investigated to determine the most economical proportions of materials for construction. Detailed laboratory results for those studies are reported hereinafter.

3.1 GENERAL

Cementitious and pozzolanic materials needed for the proposed construction will

be Portland cement and pozzolanic admixtures such as fly ash. The use of fly ash is recommended based on the cost of cement, approximately $100 per ton and on the widespread availability and quality of fly ashes available in the region.

3.2 PORTLAND CEMENT

Potential sources of Portland cements are indicated on plate 1.

a. *Types.* Type II, low alkali cement conforming to the requirements of ASTM C-150, will be specified. This cement would be available in suitable quantities for any construction anticipated. All of the plants indicated would be capable of producing sufficient cement to meet the proposed construction requirements. The current costs of cement vary from approximately $98 to $105 per ton, from the Phoenix Cement Plant at Clarkdale and the Arizona Portland Cement Co plant at Rillito, Arizona.

b. *Testing Requirements.* Portland cement will be accepted based on the results of tests submitted by the supplier. The government reserves the right to perform Quality Assurance sampling and testing during the execution of any construction contracts.

3.3 FLY ASHES

The primary types of pozzolans available, in this region, are fly ashes. Fly ashes have been used extensively by the Los Angeles District in the past and are readily available in the area. Potential sources of fly ashes are indicated on plate 1.

a. *Class F.* Class F pozzolans, conforming to the requirements of ASTM C-618, will be specified and the special requirements from table 1A shall be invoked. Additionally, from table 2A the following requirements shall be added: (1) the limit on increase of drying shrinkage, and (2) mortar expansion at 14 days. The requirement for mortar expansion at 14 days will be modified so that specimens prepared with the selected fly ash will supply expansions less than or equal to those of specimens prepared using the selected cement alone. The pozzolan would be available in suitable quantities from the sources listed below. The current costs of pozzolan vary from approximately $35 to $40 per ton, from various locations throughout the state.

b. *Testing Requirements.* Fly ash be accepted based on the results of tests submitted by the supplier.

c. The government reserves the right to perform Quality Assurance sampling and testing during the execution of any construction contracts.

3.4 WATER

Water suitable for use in RCC construction would be available from existing city sources.

4. BORROW MATERIALS

Borrow materials proposed for use in production of the RCC will come from the required project excavation. This source has provided suitable quality materials for use in production of concrete and asphaltic aggregates in the past. The project site is currently being exploited by CALMAT and the Tanner-United Metro Co's for production of various classes of aggregates for construction throughout the region.

4.1 SAMPLING

Materials for particle size analysis were obtained from test trenches excavated with a CASE Model 580K, rubber tired backhoe. The depths of materials explored was limited to approximately 15 feet based on the design information available at the time of the exploration. An insignificant difference in material size is anticipated below those depths. Materials larger than 3 inches in size were visibly estimated, and small bag samples were obtained to return to the Los Angeles District Laboratory, for detailed particle size analysis. Materials for preparation of mix design studies were obtained from the United Metro plant at 19th Ave. The materials obtained from the United Metro plant were from existing borrow site in the stream bottom, currently being exploited by United Metro. A review of available data from United Metro and observations of explorations and stockpiles at the United Metro and CALMAT plant indicate that the materials are similar, and should be reasonably respresentative of materials available for borrow throughout the project limits.

4.2 FIELD PROCESSING (by The Bureau of Reclamation {BUREC})

An approximate 8 ton sample was delivered to the BUREC facilities in Phoenix. The sample was a composite sample obtained from TT99-26. A bulk gradation was performed at the BUREC facilities. The results of that gradation are reported in Table 4-1. Based on field observations of other contemporary excavations and examination of working materials pits in the immediate vicinity

the gradation indicated should be representative of potential excavation within the project limits.

Table 4.1
Composite Gradation
Test Trench 99-26

SIEVE SIZE	PERCENTAGE PASSING
12"	100
5"	90
3"	75
2"	64
1-1/2"	53
3/4"	45
3/8"	41
No 4	27
No 8	25
No 16	22
No 30	16
No 50	8
No 100	5
No 200	3

4.3 PROCESSING BY TANNER-UNITED METRO

Materials obtained from Tanner-United Metro were materials available from the planned construction site. The materials were excavated and then transported to the Tanner plant. At this location the materials are stockpiled and then fed into a primary crushing system. The system reduces the maximum particle size to approximately 3-inches and the materials are then stockpiled. From this point materials are transported for additional crushing, screening and classification to produce the desired commercial products. A bulk sample representing the materials available in the primary crush stockpile was obtained. Additionally, bulk samples of an Arizona Department of Transportation (ADOT) Class II, road

base and an unwashed sand were obtained for additional processing and use in

the planned studies. The procedure used for production of aggregates at this plant is similar to procedures used at other plants along the Salt River through the Phoenix area.

4.4 LABORATORY PROCESSING

The samples were transported to the BUREC facility in Denver Colorado, for additional processing and preparation of RCC mixtures for additional testing and analysis. The following materials were delivered to the laboratory for analysis: (1) an aggregate road base, conforming to ADOT standards for ABC was selected and transported to the laboratory; (2) the primary crush product from the United Metro production plant; and (3) an unwashed sand sized material. The primary crushed product from the United Metro Plant was screened, by the BUREC, to produce a 2" X 1-1/2", 1-1/2" X 3/4", 3/4" X No. 4, and a minus No. 4 material. The 1-1/2" X 3/4" and 3/4" X No. 4, were recombined to make the coarse aggregate indicated in Table 5.2. After examination the unwashed fine grained materials were washed to produce a more desirable gradation. The washed fine grained materials were recombined with the coarse aggregates to produce the final gradation used in the mix design RS-9. A complete description of mix design selection and evaluation is included below.

5. LABORATORY PROGRAM

5.1 GENERAL

Laboratory studies were conducted to evaluate the selected materials for production of RCC. All laboratory studies, except bulk gradations, were performed at the BUREC's laboratory facilities in Denver, Colorado.

5.2 PHYSICAL PROPERTIES OF AGGREGATES

Only a minimal number of tests were performed on the aggregates. Tests performed primarily to determine the mixture proportioning properties of the aggregates available. Table 5.1 summarizes physical properties of the aggregates used in the studies. Table 5.2 summarizes aggregate gradations used in the various trials.

Table 5.1
Physical Properties of Aggregates

Material	Sp Gr	Absorption
ABC	2.58	2.05
1-1/2" x No 4	2.62	1.18
Wash Sand	2.61	1.25

Table 5.2
Laboratory Gradations

Sieve Size	Aggregate Gradations				
	ABC	Washed River Sand	ABC+ Coarse Agg	Coarse Agg	Mix RS-9
1-1/2-inch	100	-	100	100	100
3/4-inch	99	-	74	57	74
3/8-inch	61	100	46	17	50
No. 4	38	99	29	2	41
No. 8	32	80	24	0	32
No. 16	26	61	20	0	24
No. 30	19	43	14	0	17
No. 50	11	23	8	0	9
No. 100	6	9	5	0	4
No. 200	4	3	3	0	1

5.3 MIX DESIGN STUDIES

RCC Mix design studies were performed based on the moisture-density relationships. A summary of mix designs and corresponding plastic and hardened properties are supplied in Table 5.3 below. The original studies were laid out based on targeting a compressive strength of approximately 3000 lb/in^2 at 28 days and supplying a Vebe consistency of approximately 30 to 45 seconds. This consistency has proven to be suitable for RCC construction in the past. In order to minimize costs of construction and processing costs a readily available gradation was selected for processing and production. The gradation selected was a gradation conforming to that generally in use for production of Aggregate Base Course (ABC) materials in Maricopa County. Mixes RS-1 to

RS-4 were developed to investigate this initial selection of materials, and to evaluate plastic and hardened properties. Based on the strengths achieved, mix RS-5 was developed to evaluate higher cement contents to achieve higher compressive strengths at comparable Vebe times.

Table 5.3
Summary of RCC Mix Designs and Properties

Mix Name	Lab. No.	Aggregate Quantities		Cementitious Materials									
		0-3/4 In	3/4 To 1-1/2 In	Cement	Pozzolan	Water	W/(C+P)	(Secs)	(Pct)	7-Day	28-Day	56-Day	90-Day
ABC-180	RS-1	3284	-	274	-	165	0.60	180	9.5	1420	1680	1475	2470
ABC-220	RS-2	3414	-	294	-	215	0.73	120	3.1	1660	2015	2490	1560
ABC-260	RS-3	3347	-	296	-	257	0.87	30	2.2	1400	2010	2115	2200
ABC-250	RS-4	3363	-	296	-	246	0.83	33	2.4	1510	2050	2150	2470
ABC-250.6WC	RS-5	3236	-	406	-	244	0.60	54	3.4	2360	3275	3510	3470
ABC1.5-250	RS-6	2537	859	297	-	248	0.83	24	1.9	1300	2195	2350	2520
ABC-250.63WCP	RS-7	3268	-	291	125	261	0.63	120	0.7	2140	2475	-	3690
ABC1.5-250.65WCP	RS-8	2489	842	280	120	252	0.63	50	0.4	1705	2445	-	3125
TAN1.5-200.6WCP	RS-9	1416	2132	234	100	201	0.60	33	0.6	2105	3065	-	3890

As an alternative, mix RS-6 was developed to determine if cement demand and abrasion resistance could be reduced by adding additional coarse aggregates, thereby improving the total aggregate gradation.

Based upon the first six mixtures (RS-1 to RS-6), the projected water content for a 30 to 45 second Vebe time is about 255 lb/yd^3 using the ABC aggregate and about 240 lb/yd^3 using the using the 1-1/2 inch NMSA aggregate. For a W/C of 0.6, the projected cement content would be about 425 and 400 lb/yd^3, respectively for each aggregate size to yield a 3000 lb/in^2 at 28 days age.

The strengths achieved and the estimated costs of production, primarily based on cement contents, was determined to be excessive based on previous Maricopa County experience with soil-cement mixtures. Target compressive strengths at 7 days were selected to be 1000 psi for the armoring of the guide dikes and 2000 psi for the drop structures. Mixes RS-7 to RS-9 were developed to examine the following effects: (1) increasing the maximum nominal coarse aggregate size, (2) substitution of pozzolan for cement, and (3) refine the aggregate gradation in the mix designs. The purpose of these analyses were to reduce the amount of Portland cement necessary to achieve the desired properties.

Prior to selecting pozzolan percentage rates to be used in the mix design studies, mortar cubes were manufactured to select a desired replacement level of cement with pozzolan. The results of that study are reported in section 5.4 below.

Further review indicated the that RCC with compressive strengths of 2000 psi for the grade control structures and 750 psi would be appropriate for armoring the guide dikes and slope protection.

5.4 CEMENT FLY ASH REPLACEMENT STUDIES

Due to the high cost of Portland Cement and the potential for reducing the cost of construction, studies to determine the potential for replacing Portland Cement in the RCC mixtures with fly ash were performed. These studies were completed based on 2-inch cube specimens, manufactured in accordance with ASTM C-109. These specimens were manufactured with various combinations of cement, fly ash, sand, and water. The mixtures were designed to supply approximately the same flows when tested in accordance with procedures outlined in ASTM C-87. The mix design and plastic properties of the various mixtures are reported in table 5.4 below. All mixtures contained 2063 gms (4.54 lbs) of sand.

Table 5.4

MIX ID	PCT Fly Ash Replacement	Batch Quantities (grms)			w/(c+p)	7-Day Strengths Avg	28-Day Strengths Avg
		Cement	Fly Ash	Water			
100C-0FA	0	750	0	355	0.473	5000	5970
90C-10FA	10	675	75	346	0.461	4107	6073
80C-20FA	20	600	150	337	0.449	3127	5253
70C-30FA	30	525	225	324	0.432	2773	3943
60C-40FA	40	450	300	315	0.420	2900	3697

Based on the above information and previous experience, mixtures may be prepared which will substitute substantial amounts of fly ash for cement, for economic reduction in production of the RCC. Target replacements used in the subsequent studies were 30 percent fly ash in substitution of cement.

5.5 ABRASION EROSION TEST DATA

Two separate investigations of abrasion erosion were used in this study. The first study was completed by the Maricopa County Flood Control District (MCFCD) and their consultants, for previous work completed by the MCFCD. The second study was performed by the government (USACE) during the current study. The MCFCD studies investigated abrasion erosion loss, generally in accordance with ASTM C-1138, based on a fixed gradation and varying amounts of cement. The current study examined abrasion erosion loss from a variation in aggregate properties, primarily gradation. The results of both studies have been used to select the optimum blend of aggregates and cementitious materials for use in the planned construction, based on the materials available within the project limits.

Abrasion erosion loss data developed by MCFCD is reported in figure 1. The results generally indicate that for the specific aggregate gradation selected, a minimum of 7 percent cement is recommended for use in soil cement to minimize abrasion erosion loss. Current studies (USACE) are reported in figure 2. The current abrasion-erosion study examined the effects of aggregate size and compressive strength. The current studies indicate that higher compressive strengths and larger NMS coarse aggregates increase abrasion-erosion resistance.

5.6 MIX DESIGN EVALUATIONS

A comparison between the USACE recommended gradation and the previously used MCFCD gradation is shown in figure 3. The decreased cement demand and the

increased performance in the abrasion test is most likely attributable to the coarser

nature of the recommended USACE gradation.
Not only does the USACE gradation increase abrasion resistance, but the mixture will most likely be more economical than the previously used gradation. The following table summarizes the cost of cementitious materials for RCC and soil cement. Costs of cement and fly ash were made based on currently available rates in the area. The RCC mixture and attributable costs is from the current (USACE) study. The soil-cement costs reported are based on a report prepared by AGRA Earth and Environmental for MCFCD in 1998.

Mixture	Cement (lbs/yd^3)	Fly Ash (lbs/yd^3)	7-Day Compressive Strength (psi)	Cost per yard Cementitious Materials*
SC (7%)	250	-	1300	$12.50
RCC	234	100	2100	$13.60
SC (10%)	356	-	2060	$17.80

* $100/ton for cement and $38/ton for fly ash.

Assuming approximately the same proportions, the cementitious materials cost of 750 psi RCC would be approxiamtely $9 per cubic yard.

6. AGGREGATE PROCESSING COSTS

6.1 COMMERCIALLY AVAILABLE PRICE OF MATERIALS

A brief survey of the cost of manufacturing the required aggregates from locally available commercial sources was completed. The price of the primary crushed product (the minus 3-inch material indicated above) used in the study would be available from local suppliers at a price of approximately $4 per ton. The specified ABC materials are generally available for $5.50 to $7.50 dollars per ton. A nominal 1-inch MSA would be available for approximately $5.00 per ton. Transportation to the site would be an additional cost not reflected in the prices quoted above.

6.2 ESTIMATED COST OF PROCESSING AGGREGATES

The costs quoted above represent retail costs for bulk purchases. A detailed cost estimate should be prepared for the desired materials specified above, but it can reasonably be concluded that total costs for processing aggregates would amount to $2

to $3 per ton. This would amount to an approximate cost of aggregates (based on mix RS-9) of approximately $3.50 to $5.25 per cubic yard.

7. DESIGN RECOMMENDATIONS

7.1 AGGREGATE SELECTION

Aggregates available from the streambed should be crushed, screened and washed to produce a more desirable gradation. The improvement in gradation will supply a better more uniform material that will require less cement and higher performance for the RCC.

7.2 CEMENT/FLY ASH COMBINATION

The laboratory studies indicate that significant amounts of fly ash may be substituted for cement and still achieve a suitable product. The MCFCD has indicated that generally strengths of 750 psi at 7 days would be suitable for the guide dikes. The Grade Control Structure will require RCC with a 7-day compressive strength of 2000 psi. Detailed laboratory trials during construction could lead to even lower quantities of cement and higher fly ash replacement rates. These alternatives should be developed in more detail during the actual construction laboratory trials.

RIO SALADO

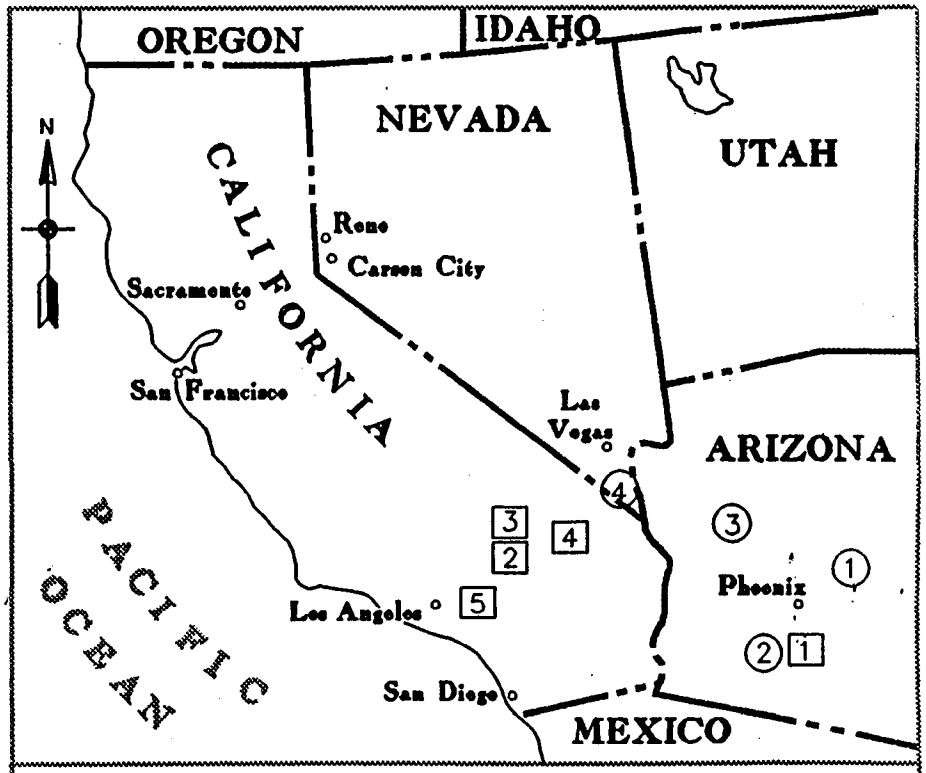
SOURCES OF CEMENTS AND POZZOLANS

SOURCES OF CEMENT

1. ARIZONA PORTLAND CEMENT, RILLITO, ARIZONA
2. SOUTHWESTERN PORTLAND CEMENT CO., VICTORVILLE, CALIFORNIA
3. RIVERSIDE CEMENT CO., ORO GRANDE, CALIFORNIA
4. MITSUBISHI CEMENT CO., LUCERNE VALLEY, CALIFORNIA
5. CALIFORNIA PORTLAND CEMENT CO., COLTON, CALIFORNIA

SOURCES OF POZZOLAN

1
2. ARIZONA PORTLAND CEMENT, RILLITO, ARIZONA
3. PHOENIX PORTLAND CEMENT, JOSEPH CITY, ARIZONA
4. BORAL MATERIALS, LAUGHLIN, NEVADA



VICINITY MAP

SCALE 200 0 200 400 MILES

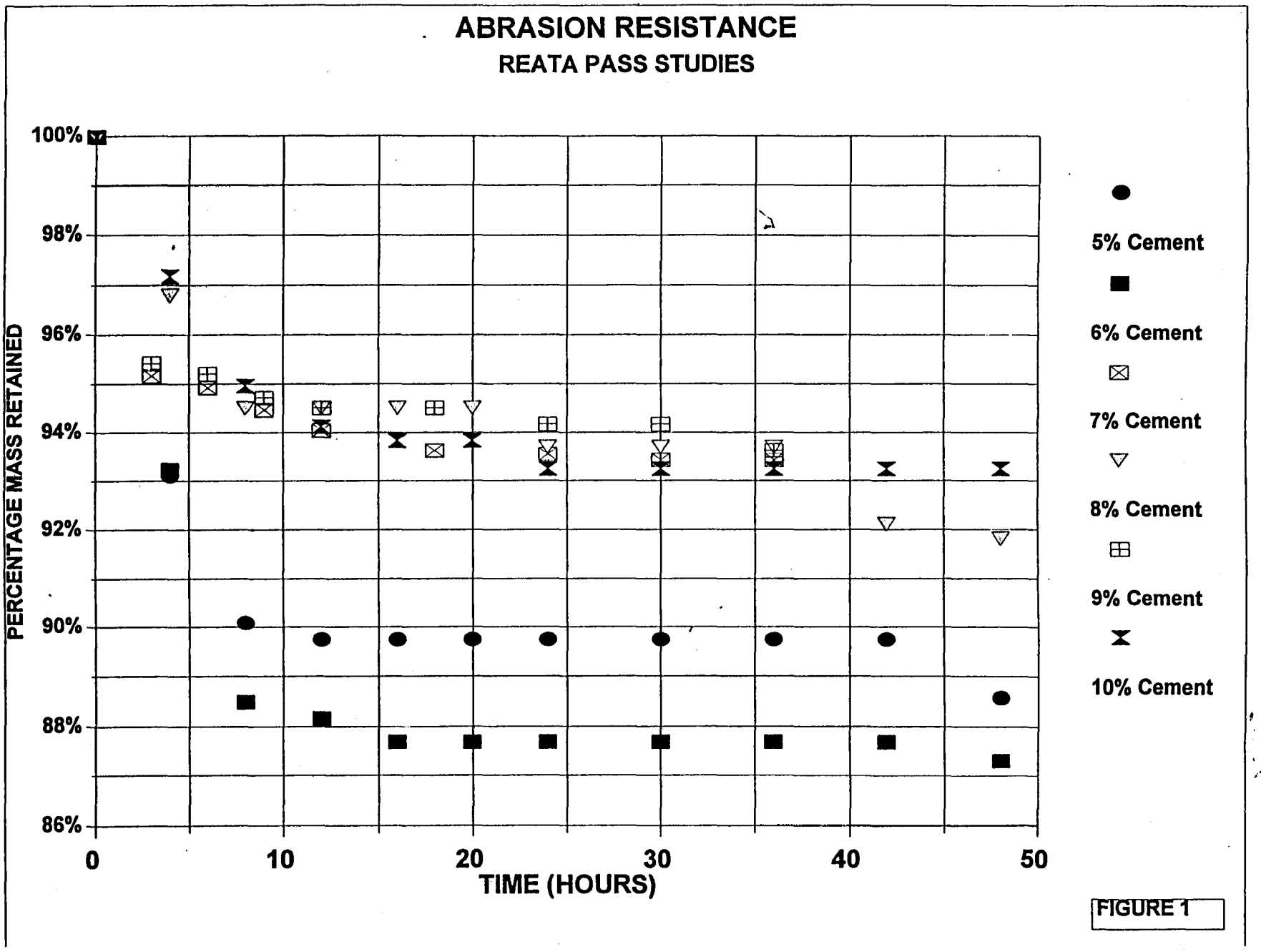

ABRASION RESISTANCE
REATA PASS STUDIES
PERCENTAGE MASS RETAINED
100%
98%
96%
94%
92%
90%
88%
86%
0
10
20
30
40
50
TIME (HOURS)
5% Cement
6% Cement
7% Cement
8% Cement
9% Cement
10% Cement

FIGURE 1

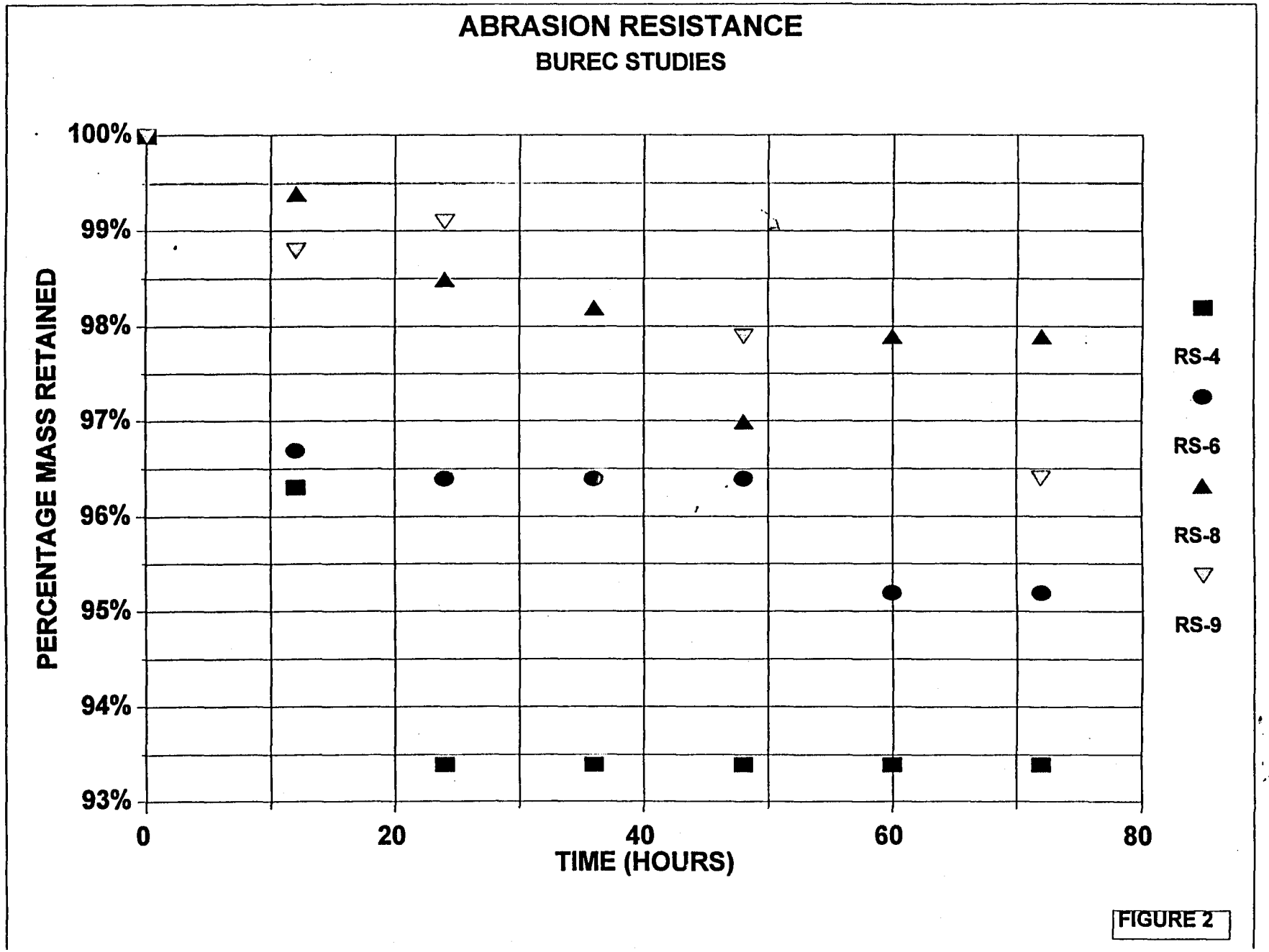

ABRASION RESISTANCE
BUREC STUDIES
PERCENTAGE MASS RETAINED
100%
99%
98%
97%
96%
95%
94%
93%
0
20
40
60
80
TIME (HOURS)
RS-4
RS-6
RS-8
RS-9

FIGURE 2

Rio Salado

USACE vs MCFCD

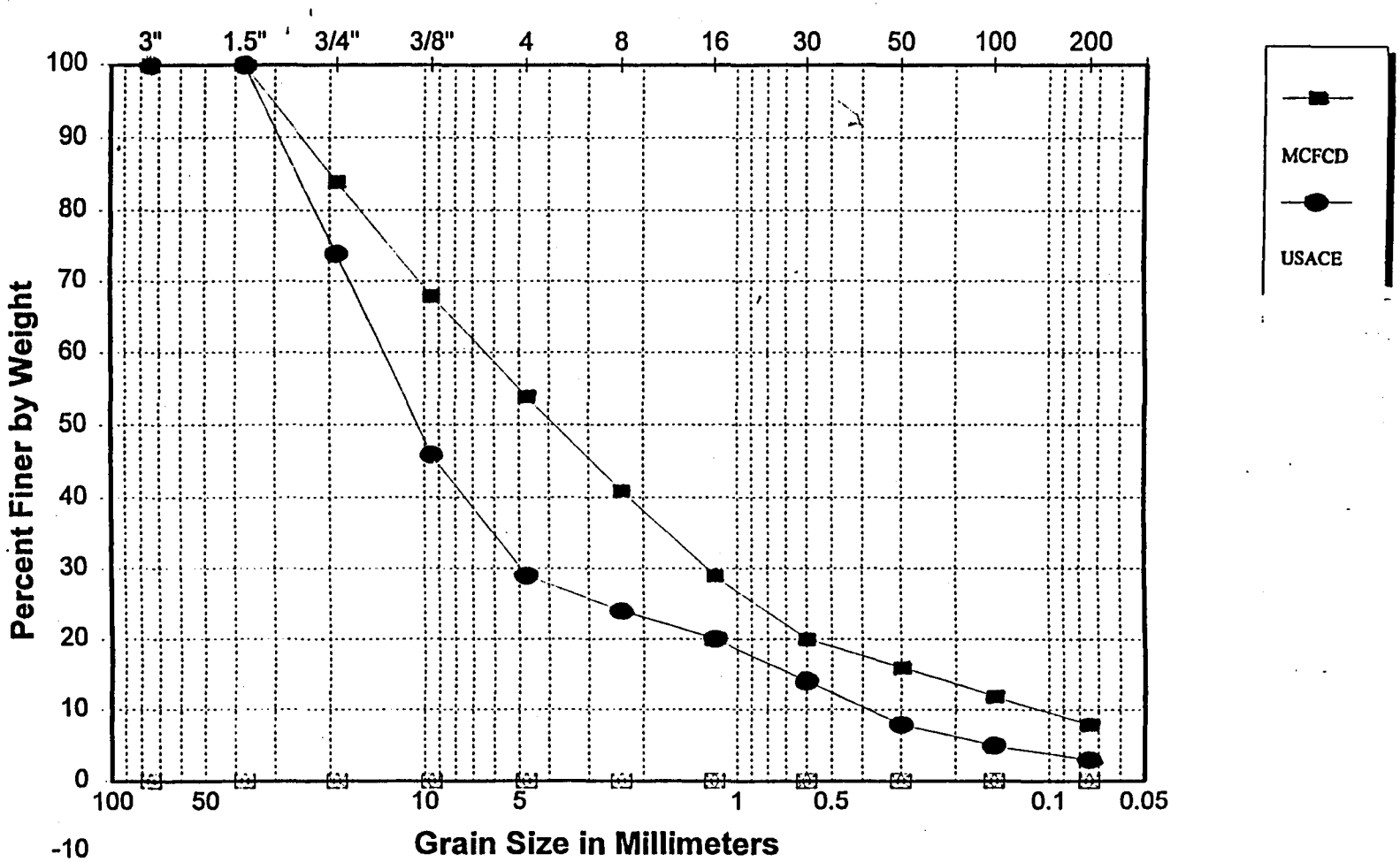


FIGURE 2