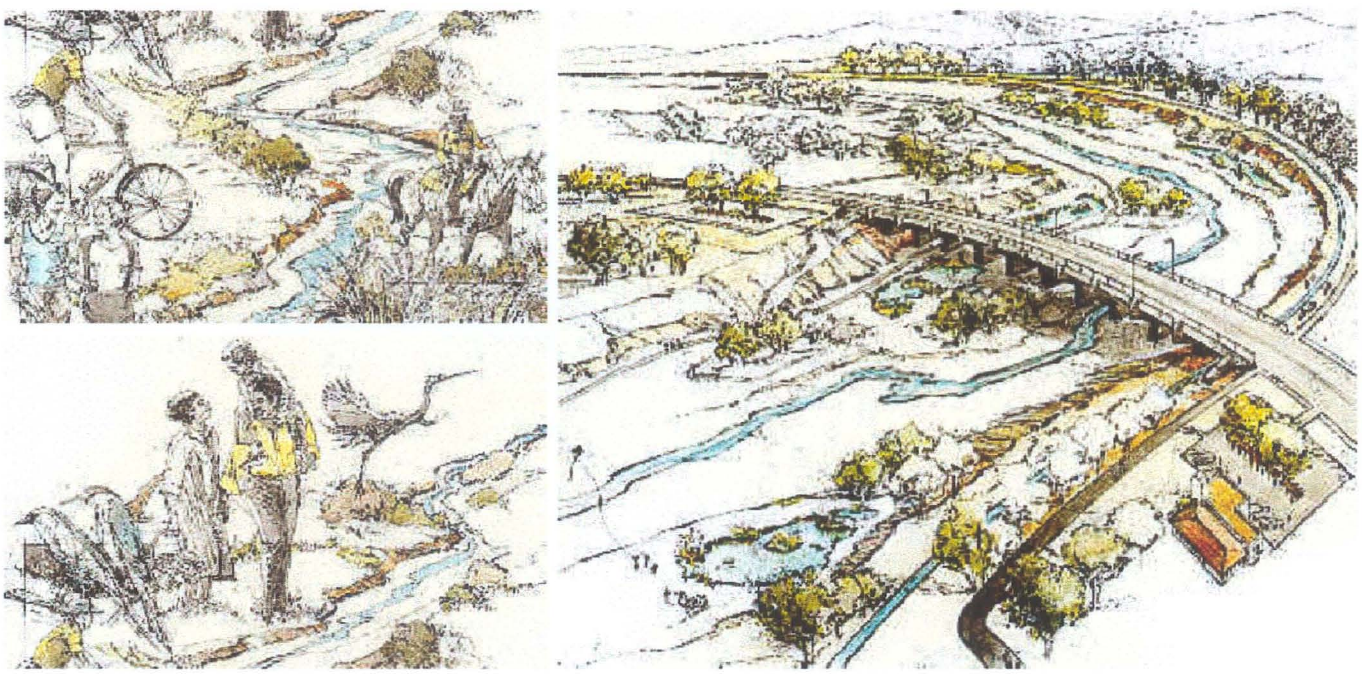
US ARMY CORPS
OF ENGINEERS
Los Angeles District

Design Documentation Report

Salt River, Arizona



Low Flow Channel, Phoenix Reach

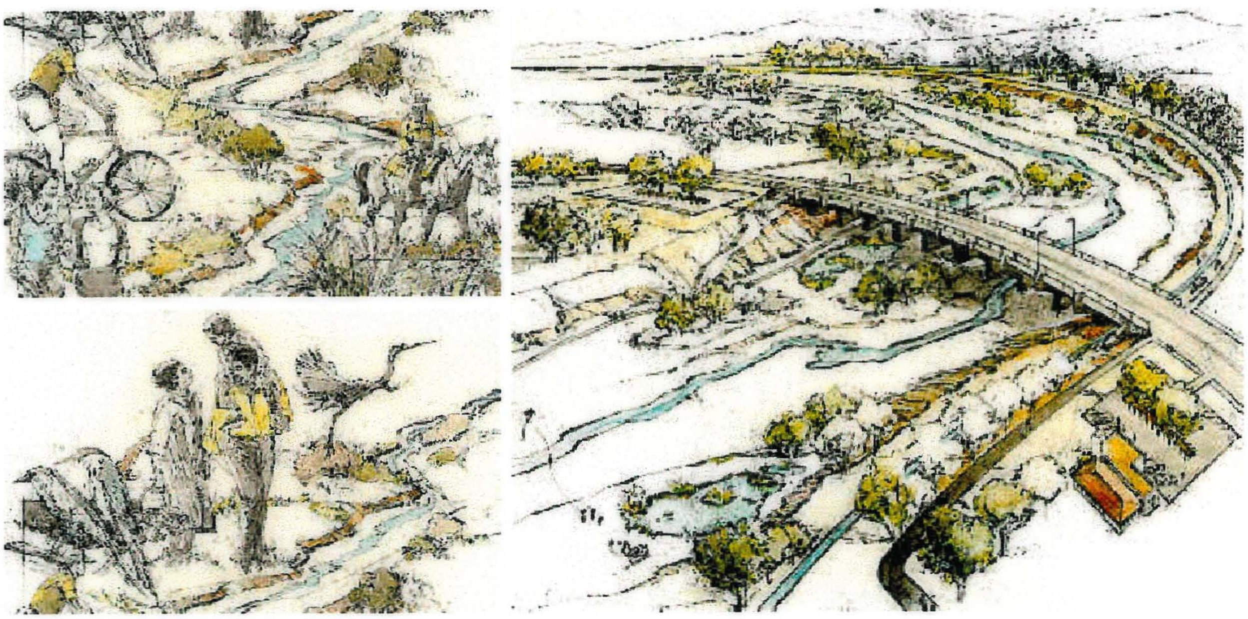
Prepared By:

MONTGOMERY WATSON

US ARMY CORPS
OF ENGINEERS
Los Angeles District

Design Documentation Report

Salt River, Arizona



Low Flow Channel, Phoenix Reach

Prepared By:

MONTGOMERY WATSON

Prepared for:

US ARMY CORPS
OF ENGINEERS
Los Angeles District

US Army Corps of Engineers,
LA District
911 Wilshire Blvd. (13111)
Los Angeles, CA 90017

RIO SALADO LOW FLOW CHANNEL

DESIGN DOCUMENTATION REPORT

Phoenix, Arizona

October 2000

Prepared by:

MONTGOMERY WATSON
4820 S. Mill Avenue, Suite 202
Tempe, Arizona 85282

Table of Contents

MONTGOMERY WATSON

TABLE OF CONTENTS

Section 1

MONTGOMERY WATSON

1.0 INTRODUCTION

In 1998 the City of Phoenix, City of Tempe, and the Corps of Engineers (Los Angeles District), completed a Feasibility Report and Environmental Impact Statement for Salt River Project within the City Limits of Phoenix and Tempe, Arizona. These Reports propose the restoration of the ecosystem function of the Salt River as it weaves through Phoenix and Tempe. Besides establishing a variety of habitats, the Reports propose a multitude of recreational opportunities. The targeted areas of improvement include five miles of the Salt River in Phoenix, 1.3 miles of the tributary Indian Bend Wash, and an additional 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 combine differing habitats with diverse recreational opportunities. This Design Documentation Report (DDR) details the design and construction process subsequent to the submittal of the Feasibility Report and Environmental Impact Statement for the Low Flow Channel portion of the Rio Salado Project, Phoenix Reach. This DDR has been prepared in the format as outlined in ER 1110-2-1150, Engineering and Design for Civil Works Projects (August, 1999), Appendix D, as applicable.

1.1 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, 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 was divided into the Phoenix Reach and Tempe Reach. The PED for the Phoenix Reach was authorized 31 July 1998. 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 monitor construction of the LFC.

1.2 PROJECT DESCRIPTION

The project consists of a five mile stretch of the Salt River, within the Phoenix City limits which extends approximately 500 feet upstream of the 19th Avenue Bridge to approximately 1,500 feet downstream of the Interstate 10 Bridge. In an effort to re-establish vegetative and wildlife habitat to this section of the Salt River, a low flow channel (LFC) will be constructed within the bank limits of the river. The intent of the LFC is to offset the 100-year flood capacity lost through the re-establishment of vegetation to the channel and associated volume modifications as a result of terrace construction on the banks of the river. Increasing the vegetation within the bank limits of the Salt River will increase the roughness of the river resulting in lower channel velocities and larger flow depths. To compensate, the LFC has been designed to safely transmit a flow rate of approximately 12,200 cubic feet per second.

The LFC is to be constructed of earthen materials. Roller Compacted Concrete (RCC) structures will be built to limit scour and local degradation of the LFC and adjacent environmental habitat restoration areas. The RCC structures to be designed for the channel include thirty-eight (38) Guide Dike Structures (GDS), four (4) Grade Control Structures (GCS), two (2) scour protection aprons and reinforcing bank protection, where required. Other minor structures typically associated with flood control channels will also be constructed. These include side drain conveyance structures, access roads, and other miscellaneous elements. Section 4.0 of this report documents various engineering investigations performed for the LFC. Section 5.0 details the engineering design of the aforementioned channel and structures.

The design and construction of the LFC was divided into two (2) phases. Phase 1 extended from upstream of the 19th Avenue Bridge to near the 7th Street Bridge. Phase 2 extended from near the 7th Street Bridge to just downstream of the ADOT grade control structure located near the Interstate 10 Bridge. In all, there exist a total of five roadway bridges and two earthen material conveyor bridge structures between each end of the project. Scour analyses were performed on the bridge structures to verify stability during a design flow event. Section 4.3.5 of this report discusses the scour estimations at the bridge structures and Section 4.3.6 of this report discusses the supporting structural analyses and designed retrofits for the bridges.

1.3 PROJECT DATA AND PROJECT DOCUMENT REFERENCES

1.3.1 PROJECT DATA

As indicated in the previous sections, the LFC was broken into separate phases for design and construction. The following table summarizes relevant statistics for the channel configuration:

TABLE 1
PROJECT SUMMARY

	PHASE 1	PHASE 2	TOTAL
GENERAL DATA:			
100-year flow (cfs)	166,000	166,000	166,000
LFC flow capacity (cfs)	12,200	12,200	12,200
Channel Length (miles)	2.0	2.8	4.8
Starting Station (river miles)	211.63	213.62	211.63
Ending Station (river miles)	213.62	216.40	216.40
Channel Section	Trapezoidal	Trapezoidal	Trapezoidal
Bottom Width (feet)	205	160 – 205	160 – 205
Side Slopes (H:V)	3H:1V/4H:1V	3H:1V	3H:1V/4H:1V
Channel Lining	None	None	None
Excavation Volume (cubic yards)	770,000	875,000	1,645,000
GRADE CONTROL STRUCTURES:			
Number of Grade Control Structures	1	3	4
Number with Scour Protection Aprons	1	1	2
Construction Material	RCC	RCC	RCC
Volume (cubic yards)	33,600	50,920	84,520
GUIDE DIKE STRUCTURES:			
Number of Structures	20	18	38
Construction Material	RCC/Gabions	RCC/Gabions	RCC/Gabions
Material Volume (cubic yards)	22,930	28,275	51,205
BANK PROTECTION STRUCTURES:			
Amount of Protection (linear feet)	2,250	0	2,250
Construction Material	RCC	N/A	RCC
Volume (cubic yards)	15,667	0	15,667
CONSTRUCTION COSTS:			
Cost	$6,111,406	$10,918,230	$17,029,636
Source	R.E. Monks (low bidder)	Engineers Est.	

1.3.2 PROJECT DOCUMENT REFERENCES

The following data sources and project document references were used in the preparation of this DDR.

AGRA Earth & Environmental, Inc. (May 16, 2000). *Geotechnical Investigation Report, Design of Grade Control Structures.* Prepared for WEST Consultants, Inc. Submitted to the Corps of Engineers.

Aspen Environmental Group (March, 1998). *Final Environmental Impact Statement, Rio Salado Environmental Restoration.* Prepared for the U.S. Army Corps of Engineers.

Dames & Moore (January 31, 2000). *Draft Data Analysis Report, Rio Salado Habitat Restoration Project.* Prepared for the US Army Corps of Engineers.

Dames & Moore (April 26, 2000). *Draft Data Analysis Report, Rio Salado Habitat Restoration Project.* Prepared for the US Army Corps of Engineers.

Flood Control District of Maricopa County (1992). *FCDMC ALERT System Precipitation Report.*

Flood Control District of Maricopa County (1993). *FCDMC ALERT System Precipitation Report.*

Sabol, G.V., J.M Rumann, D. Khalili, and S. D. Waters (September 1, 1990). *Hydrologic Design Manual for Maricopa County, Arizona.*

Sabol, G.V., J.M Rumann, D. Khalili, and S. D. Waters (June 1, 1992). *Documentation / Verification Report for the Drainage Design Manual for Maricopa County, Arizona – Volume 1 – Hydrology.*

Salt River Valley Water Users' Association (1985). *Zanjero Area Maps.*

SA&B (September 28, 1999). *Health and Safety Plan, Rio Salado Project Area Between 19th Avenue and the Interstate 10 Bridge, Phoenix, Arizona.* Prepared for the City of Phoenix.

SCS Engineers (February 19, 1999). *Phase I Environmental Assessment, Rio Salado Project Area, Phoenix, Arizona.* Prepared for the City of Phoenix.

SCS Engineers (February 29, 2000). *Phase II Environmental Assessment, Rio Salado Project Area, Phoenix, Arizona.* Prepared for the City of Phoenix.

US Army Corps of Engineers (April, 1998). *Feasibility Report and Environmental Impact Statement.* Prepared for the Rio Salado Project.

US Army Corps of Engineers (September, 1999). *Design Analysis.* Prepared for the Rio Salado Project.

US Army Corps of Engineers (September, 1999). *60% Submittal – Construction Solicitation and Specifications.* Prepared for the Rio Salado Project, Phoenix Reach, Low Flow Channel, Phase 1 (19th Avenue to 7th Street).

US Army Corps of Engineers (August 31, 1999). Engineering and Design for Civil Works Projects. *Regulation # ER 1110-2-1150.*

US Army Corps of Engineers (January, 2000). *95% Submittal – Construction Solicitation and Specifications.* Prepared for the Rio Salado Project, Phoenix Reach, Low Flow Channel, Phase 1 (19th Avenue to 7th Street).

US Army Corps of Engineers – Survey Section (July 14, 2000). *Rio Salado Phase II, Phoenix, Arizona.* Prepared for the Rio Salado Project.

WEST Consultants (June 21, 1999). *Pre-Final (90 Percent) Low Flow Channel Design Analysis Report for Rio Salado (Salt River), Arizona.* (Technical Appendices Only) Prepared for the U.S. Army Corps of Engineers, Los Angeles District.

WEST Consultants, Inc. (September 2, 1999). "Grade Control Structure Alternatives, Rio Salado (Salt River), Phoenix, Arizona". *Memorandum to the Corps of Engineers.*

WEST Consultants (January 11, 2000). *Low Flow Channel Design Analysis for Rio Salado (Salt River), Arizona.* Prepared for the U.S. Army Corps of Engineers, Los Angeles District. 4 Volumes (Final Report, Maps, Technical Appendices I (1-3), and Technical Appendices II (4-7)).

Section 2

MONTGOMERY WATSON

2.0 ENGINEERING STUDIES

Prior to the initiation of design for the LFC, several engineering studies were completed to address the feasibility and environmental impacts of the project. Following the identification of the selected alternative, a value engineering study was performed to identify potential cost saving design features to the channel. The following sections briefly describe these engineering studies and their recommendations.

2.1 ENVIRONMENTAL IMPACT STATEMENT

In conjunction with the Feasibility Study, an Environmental Impact Statement (EIS) (Aspen Environmental Group, March, 1998) was prepared for the channel. The purpose of the EIS was to identify potential environmental impacts, which would result from the construction, operation, and maintenance of the proposed habitat areas, LFC, and other improvements and facilities associated with the project.

Several environmental restoration alternatives were evaluated and discussed for the Phoenix Reach of the Salt River. These alternatives were evaluated based on criteria associated with the following:

- Geology and Geomorphology;
- Air Quality;
- Hydrology and Water Quality;
- Biological Resources;
- Land Use and Recreation;
- Cultural Resources;
- Hazardous, Toxic, and Radioactive Waste;
- Aesthetics;
- Noise;
- Transportation; and
- Cumulative Impacts.

2.2 PROJECT FEASIBILITY STUDY AND RECOMMENDED PLAN

The study efforts (U.S. Army Corps of Engineers, April, 1998) were directed toward establishing the feasibility of environmental restoration with incidental recreation along the Salt River in Phoenix and Tempe, Arizona. Restoration efforts are required because upstream water projects have curtailed year-round water flows and converted the once perennial Salt River into a dry riverbed devoid of habitat. The feasibility report intended to: (1) provide a complete presentation of study results and findings, including those developed in the reconnaissance phase so that readers can reach independent conclusions regarding the reasonableness of recommendations; (2) indicate compliance with applicable statutes, executive orders and policies; and (3) provide a sound and documented basis for decisions makers at all levels to judge the recommended solution(s).

The two non-Federal sponsors identified were the Cities of Tempe and Phoenix, Arizona. A key initial activity of the feasibility effort was to work with the non-Federal sponsors to identify the study area and focus on the environmental restoration opportunities, with associated incidental recreation opportunities, within the defined study area. Upon initiation of the feasibility effort, the entire 33-mile reach studied under the reconnaissance phase was evaluated for potential environmental restoration. However, after discussion with the non-Federal sponsors, two specific sites were identified which would be of immediate interest in a cost-shared construction project.

The first site is located in Tempe, Arizona, on portions of the Indian Bend Wash and the Salt River and is referred to as the "Tempe Reach." The second site studied in this feasibility report is located entirely in the Salt River within the City of Phoenix, Arizona and is referred to as the "Phoenix Reach." As this DDR is being prepared for the Low Flow Channel within the Phoenix Reach, the remaining discussion of the feasibility study will focus on the Phoenix Reach.

There is currently very little habitat found within the Phoenix Reach. The desired habitat types for this area include mesquite habitat, cottonwood/willow habitat, wetland marsh, aquatic strand/scrub habitat, and open edges. Integral to the restoration of riparian habitat is providing sufficient water to irrigate the desired vegetation. After evaluation of several alternative water sources, groundwater was the selected source of water for restoration activities within both reaches.

A number of habitat restoration alternatives were developed in cooperation with the non-Federal sponsor and evaluated relative to their effectiveness, acceptability, and incremental economic efficiency. From the array of alternatives a plan was selected for each reach which was determined to be technically feasible, economically efficient, and environmentally sound according to the Federal water resources planning criteria. The selected plans would provide riparian habitat, marginal surface and groundwater quality improvement from well-head treatment and the natural filtering ability of wetland vegetation, and incidental aesthetic and recreational opportunities. Restoration within the Phoenix Reach would consist of approximately 130 acres of mesquite, 99 acres of cottonwood/willow habitat, 58 acres of wetland marsh, 51 acres of aquatic strand/scrub habitat, and 187 acres of open edges.

The non-Federal sponsors have also expressed a desire to increase the recreation opportunities incidental to the restoration effort within the study area. The riparian habitat created by the selected restoration plans would be unlike any other resource in the metropolitan area. The selected recreation plans intend to create a wide variety of means to enjoy the resource, including viewing, picnicking, education, and exploring by foot, horseback or bicycle.

The analysis indicated that the selected plans are feasible and would provide environmental restoration benefits that serve the public interest. In order to maintain the existing 100-year flood capacity, the Feasibility Study suggests entrenching the LFC within the river's bottom to mitigate the capacity reduction induced by the restorative features. A maximum low flow discharge of 12,200 cubic feet per second (cfs) was agreed upon by the study team as being the design target discharge, based on a step 4 release schedule found in the Modified Roosevelt Dam Water Control Diagram. This particular discharge corresponded to between a 50- and 100-year flow for a duration time of 30 days. However, in terms of peak flow, a discharge of 12,200 cfs corresponds to less than a 5-year event.

The LFC, as presented in the Feasibility Study, was designed to have a natural vegetation bottom, consisting of opportunistic emergent vegetation. The channel would have a 2H:1V slope soil cement embankment throughout the channel except under each bridge crossing, where it would have a 3H:1V slope soil cement embankments. The embankments were suggested to have a minimum 5-foot embedment below the base grade of the LFC to limit scour and an 8-foot width to account for machinery operations during construction.

The selected alternative for the Phoenix Reach included approximately 2.5 miles of permanent open water features, within the LFC, as part of the riparian system. This open water in the channel would consist of low discharge perennial stream (5 cfs) that would connect four shallow ponds. The design features in support of these features included the over-excavation of the low flow channel at the pond locations, and small inlet and outlet structures upstream and downstream of the pond locations to guide the stream. It is expected that the LFC and associated structures would have to be restored periodically after major flood events.

The selected plan included a total of four (4) drop structures within the LFC and two additional drop structures located outside the low flow channel in side-drain outlet structures. The drop structures would be made of 30 inch thick Roller Compacted Concrete (RCC). The structures vary in height and would be constructed with a minimum embedment 5 feet below the LFC base elevation.

2.3 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.

Section 3

MONTGOMERY WATSON

3.0 ENGINEERING INVESTIGATIONS

3.1 GEOLOGICAL/GEOTECHNICAL INVESTIGATIONS

Several geotechnical investigations were performed to characterize materials within the banks of the Salt River for the Phoenix Reach. The Corps of Engineers excavated a series of test trenches within the project limits of the Phoenix Reach to obtain geotechnical samples of the existing material. Agra was subcontracted to perform a subsurface investigation of the materials below each of the grade control structures. The following documents the geological and geotechnical investigations performed for the project.

A total of 32 test trenches were excavated at locations indicated on the construction plan drawings for project. The test trenches were excavated to a depth of approximately 16 feet. The material is described primarily as alluvial deposits of sandy gravel to gravely sand, with some cobbles. Various test trenches indicated thin lens deposits of silt material at various depths. However, the majority of the test trenches suggest that the overall fine grain material content (< #200 Sieve) is less than 5 to 10% by weight. A summary of the geotechnical data collected is presented as Appendix A to this document.

Four boreholes were drilled in the approximate locations of the grade control structures. The goal of the subsurface investigation was to identify materials beneath the grade control structure to determine their foundation characteristics. As indicated in the geotechnical investigation report for the design of Grade Control Structures (AGRA, May 16 2000), presented in Appendix A, no samples were taken to verify the geotechnical characteristics of the soil due to the coarse nature of the alluvium. An effort was made to estimate the density of the material by counting the number of blows required to drive the 9-inch diameter casing used by an air percussion drill rig. Field identification of drill cuttings was performed. This field observation indicated that each of the structures would be founded on a dense alluvial deposit comprised primarily of sand, gravel, and cobbles. A portion of the draft report is presented in Appendix A, as applicable to this section.

3.2 ENVIRONMENTAL ASSESSMENT

As part of the investigation phase for the Phoenix Reach of the Salt River, SCS Engineers were subcontracted by the City of Phoenix to perform an environmental assessment of the river basin to be impacted both by the construction of the LFC and the Habitat Restoration. The details of the Environmental Assessment are presented in the reports prepared by SCS Engineers (February 19, 1999; February 29, 2000). As this DDR applies to only the LFC, the environmental assessment report was not provided under this cover.

Through the environmental assessment, historical landfilling within the project boundaries was identified for the Phoenix Reach. To identify the impacts to the LFC and supporting hydraulic structures, geophysical surveys, test pit excavations, and borings were performed as part of a second phase to the environmental assessment. The intent of the investigation was to characterize the nature and approximate the extent of buried materials, determine the presence of landfill gases associated with the buried materials, and characterize stained soil material within the project limits of the Phoenix Reach.

Geophysical surveys were performed to identify subsurface anomalies, which would indicate potential buried materials within the project area. Test trenches were excavated where the geophysical survey indicated the presence of these subsurface anomalies or at suspect locations determined by the field engineer. The trenches were excavated to identify the potential waste and the extent of cover over the material. As a result of the survey, trenches excavated identified areas of inert and municipal waste, which may impact the performance of the hydraulic structures associated with the LFC. These impacts are mostly associated with the guide dike structures adjacent to the riverbank, and portions of the trapezoidal section of the LFC. Design modifications associated with the proximity of waste materials to the hydraulic structures for the LFC are discussed in Section

4.0 of this report. Construction issues associated with the landfilling impacts are presented in Section 5.2 of this report.

3.3 CONSTRUCTION MATERIAL INVESTIGATION

The Corps of Engineers evaluated materials available within the project limits and those available from vendors for use on structural components of the LFC. As indicated in Section 3.1 of this report, the alluvial material within the bank limits of the Salt River is predominantly sand and gravel with some cobbles. There is very little fine-grained material available for use in the RCC mix design for the guide dike structures and grade control structures. Offsite aggregate suppliers were contacted to determine the availability of fine-grained material within reasonable proximity to the project. It was identified that the material is not readily available from vendors and that crushing, screening and washing of excavated soils from the LFC would provide the best and most cost effective source to produce the granular materials for the mix design. The improvement in gradation will supply a better more uniform material that will require less cement at a higher performance for the RCC.

In addition to the soil materials to be used in the construction of the LFC, fly ash was considered as a substitute to cement to reduce the overall cost of the structure. In general, it was determined that widespread availability of fly ash in the region in contrast with the high price of cement, makes the use of fly ash as a pozzolanic material a more efficient and cost effective alternative. The Corps of Engineers prepared RCC mix design studies to determine the gradation of materials to be used in the construction of the RCC structures, the required quantity of cement, and the appropriate percentage of fly ash. Appendix B details the evaluation of materials for the RCC Structures.

3.4 HYDROGEOLOGIC INVESTIGATION

In conjunction with the Environmental Assessment and geological field investigations, the hydrogeological conditions for the project site were examined. A potentiometric surface was generated for the project site based on water levels recorded during the various field investigations. The identified surface has been presented on the plan drawings for dewatering consideration of the contractor. The water surface, as indicated on the construction plans, suggests that portions of the LFC and associated structure excavations will need to be dewatered.

The quality of the ground water was evaluated for suitable use in the construction of the LFC and associated structures. It was determined that the ground water did not meet potable water standards and was not suitable for use with the RCC Mix design, however, the water could be used for dust control measures. It is expected that the contractor will obtain potable water for the RCC structures from an offsite source.

3.5 SURVEYING

An aerial survey was performed for both Phases 1 and 2 of the project to develop a 1-foot contour plan for the entire project site. The resulting topographic plan is presented on the construction drawings for both phases. The survey was flown in August of 1999 and based on Corps of Engineers and City of Phoenix Survey monuments.

In July 2000, the Phase II portion of the project was flown for a second time, as significant modifications to the existing conditions of the Salt River were made. During the 1st quarter of 2000, CALMAT, through an arrangement with the City of Phoenix, excavated approximately 295,000 cubic yards of soil material within the limits of the LFC. The updated aerial topography was used to represent existing conditions for the plan drawings for the Phase 2 portion of the project.

Section 4

MONTGOMERY WATSON

4.0 ENGINEERING DESIGN

4.1 PROJECT SITE PLAN

4.1.1 PROJECT LIMITS

As indicated previously, the project limits for the Rio Salado, Phoenix Reach extend between the ADOT Grade Control Structure (located 1,500 feet downstream of the I-10 Bridge) and approximately 500 feet upstream of the 19th Avenue Bridge. The North and South limits of the project are generally described as 50 horizontal feet outside of the current Salt River Banks. The plan drawings for the LFC present the limits of Right of Way as delineated by the City of Phoenix as of March, 2000.

4.1.2 REAL ESTATE IMPACTS

The City of Phoenix owns and maintains the property area described as the project limits in the previous section. Two private sand and gravel companies currently operate material quarries adjacent to the Rio Salado Project Area. One company has an abandon soil conveyor bridge that crosses the river at approximately River Mile Station 212.35. The other maintains both a low water, haul road crossing and a soil conveyor-bridge, which traverses the Salt River at approximately River Mile Station 215.10. As directed by the City of Phoenix, both conveyor structures will be demolished. The low water crossing at River Mile Station 215.10 will be maintained in equivalent condition for approximately 30 years at this location.

The existing low water crossing is comprised of two haul road ramps with grades not exceeding 10%. Two, eight (8) foot diameter culverts exist beneath the north access ramp to transmit nuisance flows from an existing ADOT side-channel discharge. It is the intent of the design to continue to convey nuisance flows from the ADOT structure through the existing culverts. No improvements will be made to the existing culverts; however, a shallow channel will be constructed to more efficiently transmit nuisance flows from the ADOT discharge to the culverts and the LFC.

A temporary construction easement located at the eastern end of University Drive has been provided to permit access to the eastern portion of the LFC project. The easement is approximately 75 feet wide and is shown on the plans and discussed in more detail in the following section.

4.1.3 SITE ACCESS

Contractor access to the river bottom is available at the following locations using City of Phoenix rights-of-way:

- On the south side of the river, from the west side of the 7th Avenue. There is an existing curb cut along the west side of the street south of the bridge, and a gradual slope down to the river bottom. The Contractor may find it necessary to construct a ramp to the river bottom in lieu of using the existing bank conditions.

- On the south side of the river, from the east side of the Central Avenue. There is an existing curb cut along the east side of the street south of the bridge. The Contractor will find it necessary to construct a ramp at this location to the river bottom. There is also an existing high clearance box culvert crossing under Central Avenue at this location that provides access from the east side to the west side of the bridge and the river bottom.

- On the north side of the river, west of 16th street, there is an existing ramp, which gains access from the crest of the Rio Salado River Bank to the River Bottom. The access has been paved with concrete and will likely permit only pick-up truck, or similar loads.

The Contractor may elect to obtain permission on his own for the use of other access locations to the river bottom. This would include the use of other existing ramps into the river bottom. However, the Contractor will obtain prior written approval of the property owner for such access use and submit a copy of the approval to the Engineer prior to use of the property and/or ramps.

Ramps to access the base of the LFC will be constructed at bridge crossing locations. Ramps will be primarily used for periodic maintenance of channel.

4.2 PROJECT HYDROLOGY

Hydrologic conditions for the Phoenix Reach of the LFC were determined during the Feasibility Study phase of the project. As indicated in Section 1.1 of this DDR, the proposed modifications were evaluated based on the estimated peak flow from a 100-year discharge event. In order to maintain the existing 100-year peak flow capacity, the Feasibility Study suggested entrenching the LFC within the river's bottom to mitigate the capacity reduction induced by the restorative features. A maximum low flow discharge of 12,200 cubic feet per second (cfs) was agreed upon by the study team as being the design target discharge, based on a step 4 release schedule found in the Modified Roosevelt Dam Water Control Diagram. This particular discharge corresponded to between a 50- and 100-year flow for a duration time of 30 days. However, in terms of peak flow, a discharge of 12,200 cfs corresponds to less than a 5-year event. The 100-year peak flow for the Phoenix Reach is approximately 166,000 cfs (U.S. Army Corps of Engineers, March, 1998).

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 not 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.

4.3 HYDRAULIC ANALYSIS

WEST Consultants, Inc. (WEST) were contracted by the Army Corps of Engineers to perform the hydraulic analysis of the Salt River to include the LFC. The final report, as prepared by WEST, details the hydraulic analysis and design of the LFC and supporting hydraulic structures. The report is entitled "Low Flow Channel Design Analysis for Rio Salado (Salt River), Arizona. January, 2000". A portion of this design report is provided in Appendix C to this exhibit.

The overall design slope of the LFC is 0.0025 ft/ft. The bottom width of the channel ranges from 205 feet in the lower reach of the project (below river mile station 215.65) to 160 feet in the upper reach. A portion of the 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 inch water line crossing (214.80) just upstream of the bridge. The low flow channel has a 3H:1V vegetated side slope. Portions of the side slopes for the LFC were modified to 4H:1V to maintain sinuosity of the channel banks. Channel bank protection, using RCC, will be provided between river mile station 212.84 and station 213.24. Grade control structures and guide dikes are included in the channel design and briefly discussed in the following sections.

4.3.1 CHANNEL ALIGNMENT

The channel alignment, as presented in the WEST report, was established based on following guidelines:

- 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).
- Avoid the top of the bank from being too close to the existing outer channel banks or levees at any given location.
- Align low flow channel to avoid bridge piers.
- Align the flow with the bridge piers.
- Minimize the change in the channel sinuosity.

For the 90% Plan submittal the channel alignment was modified from River Mile Stations 215.75 to 216.23 to more centrally locate the LFC GCS within this reach. The intent is to funnel braided flows upstream of the GCS more quickly into the LFC to limit impacts to the restoration portions of the project.

4.3.2 RIVER WATER SURFACE COMPARISON

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 existing conditions model for the 100-year peak discharge of 166,000 cfs. In this analysis, it was assumed that vegetation damage reduces the overbank Manning roughness coefficient value 'n' from 0.08 to 0.04. A sensitivity analysis shows that the overbank 'n' values could be as high as 0.06 upstream of river mile station 212.84 and still produce computed water surface elevations below those for the existing conditions. Baker proposed a Flood Insurance Study (FIS) model that had a water surface equal to, or higher, than WEST's model of existing conditions.

4.3.3 GRADE CONTROL STRUCTURES

Four grade control structures, constructed from RCC, are proposed for the project reach. The structures are located at River Mile Station 216.23 (below the ADOT GCS) and 215.80 (below 24th Street) to limit the scour in the upstream reach, at Station 214.65 to reduce scour at the 16th Street Bridge, and at River Station 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 (depth from finish ground surface to base of GCS) are 30 feet within the low flow channel and 16 feet in the overbank areas. The GCS at the 24th Street Bridge was originally located at River Mile Station 215.65 in the WEST report (January, 2000). The GCS was moved upstream to the 24th Street bridge to limit the scour potential at the bridge piers. To limit local scour as a result of LFC excavation, a scour protection apron 4 feet thick will be constructed between the piers of the 24th Street bridge within the outerbank limits of the LFC. The GCS at the Central Avenue Bridge was also modified to limit local scour based on structural and scour analyses. These analyses are briefly discussed in subsequent sections of this report. 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.

Consideration was given to the subgrade support of the structures. As indicated previously, boreholes were excavated in the proximity of the grade control structures. The exploration holes were constructed to verify the subsurface conditions for each of the structures. As a result of the exploration, it is not anticipated that waste materials will be encountered during the excavation for these structures. However, guidance has been provided to the contractor in the plans and specifications for the project. A discussion of waste handling procedures for the contractor is discussed in 5.2, Debris Removal.

4.3.4 GUIDE DIKE STRUCTURES

The selected low flow channel alternative includes 38 guide dikes at strategic locations within the overbank area. The guide dikes will be constructed 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 channels in the overbank areas. During receding flow events, the guide dikes will direct flow toward the LFC, which will help preserve the location of the original meander geometry and horizontal orientation of the LFC (West, January 2000).

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, and minimize the likelihood of encountering buried landfill and regulated materials. Differential settlement issues were considered in proximity to the existing riverbanks. Gabion mattresses will be used to further limit excavation requirements adjacent to the banks and promote a relatively dynamic revetment system less susceptible to differential settlement conditions.

4.3.5 BRIDGE SCOUR ANALYSES

Bridge scour analyses were 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 degradation was determined from the 25-year HEC-6T simulations using the Laursen-Copeland sediment transport method, which results in maximum scour. The scour elevations were evaluated against the allowable scour elevations required for structural integrity of each bridge. The following section briefly discusses the structural analyses of the bridge structures within the Phoenix Reach of the project.

4.3.6 BRIDGE STRUCTURAL ANALYSIS

Based on the results of the scour analyses, structural analyses were performed by TY Lin on each bridge to determine the necessity of a structural retrofit. The structural analyses determined that countermeasures would be required at the Central Avenue Bridge and the 24th Street Bridge. The remaining bridges within the project reach will not require structural retrofits according to the calculations performed by TY Lin and WEST Consultants.

To limit local scour at the 24th Street Bridge and maintain it's structural integrity, an RCC apron will be constructed between the bridge piers from bank to bank. The grade control structure (originally 800 feet downstream of the 24th Street Bridge) was moved upstream to tie into the scour protection apron. By reducing the scour potential at the bridge, the stability of the structure will not be compromised during the 100-year design flow event.

The Central Avenue Bridge will also have an RCC apron constructed between the bridge piers. A grade control structure will be constructed immediately downstream of the Apron to limit long-term scour degradation through the bridge piers. This apron will also extend bank to bank to limit the scour potential outside of the LFC.

Uplift pressures on the RCC apron and grade control structures were considered at both of these locations due to the depth of flow during the 100-year event. According to letter reports prepared by TY Lin, the structures will withstand uplift during the flow event without compromising their orientation or structural stability. The letter reports are presented in Appendix D.

4.3.7 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.

4.4 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 discussed in the previous sections.

Four over-excavation scenarios (1 -foot, 2-foot, 3-foot and 4-foot over-excavation depths) were evaluated. It was recommended that the channel be excavated 2 feet below the design invert downstream of the proposed grade control structure at 214.65 (16th Street). This 2-foot over-excavation scenario results in the least amount of scour at 16th Street Bridge and has a moderate number of annual maintenance events.

Section 5

MONTGOMERY WATSON

5.0 LFC CONSTRUCTION ASPECTS

In addition to the design of the LFC, construction aspects were considered. The following sections briefly discuss construction issues considered for the LFC.

5.1 ROUGH GRADING OF LOW FLOW CHANNEL

Prior to the preparation of the final design for the Phase 2 portion of the project, a local sand and gravel company was permitted to rough excavate portions of the LFC. Material was removed from the channel limits as delineated by the project team. Areas not excavated were in proximity to existing bridge structures, and preliminary locations of grade control structures. The rough excavation limits were defined horizontally by the base width of the channel at the given section. The side slopes of the channel were not excavated. The vertical limits of excavation were defined as approximately 1 foot above the design LFC invert. The topography presented on the 90% construction plans and final construction plans represent the post excavation limits of the work performed by the local sand and gravel company within the channel limits.

5.2 DEBRIS REMOVAL

As indicated in the Phase 1 and Phase 2 Environmental Assessments for the Rio Salado Project. Inert materials, construction materials, tires, and household wastes may be encountered during the construction of the LFC and its related structures.

Upon encountering any waste materials the Contractor is instructed to notify the Engineer of the location of the material and allow the Engineer and the City of Phoenix (COP) on-call environmental contractor full access to the site to inspect the wastes and recommend further procedures. The Contractor must provide all information necessary to comply with ARS 49-701 to the Engineer. Within 14 days of the removal and disposal of any solid waste, the Contractor must, unless otherwise directed by the Engineer, provide the following information, using the form provided in Appendix "D".

1. A narrative description and map of the location where the waste materials were encountered including station points and offsets.

2. A brief written description of the wastes and removal procedures, including the nature and approximate quantity of the wastes removed, approximate dimensions of the excavation, a description of waste handling, storage, and transportation practices, and a description of the disposal method and location.

3. Supporting documentation such as load receipts, manifests, etc.

All waste materials other than inert material and construction debris will be characterized by the COP, and if necessary segregated by a COP on-call environmental contractor. Once this has been accomplished the Contractor, at the direction of the Engineer, may be instructed to remove and dispose of all non-hazardous materials. This includes construction debris, inert material, special wastes and household wastes.

The limits of removal for household wastes and special wastes for placement of RCC structures shall be 3 feet beyond neat line limits and backfill and compact to neat line.

The limits of removal for household wastes and special wastes for LFC excavation shall be as follows:

1. Waste material located below the LFC invert shall be completely removed.

2. Waste material located along the side slope of the LFC shall be overexcavated at least 10 feet horizontally from the toe and finished face of the LFC side slope. The overexcavation will be performed to a slope of 6:1 through the waste accumulation. Portions of the overexcavated slopes, which do not contain waste, can be sloped as necessary to meet existing or final grade.

3. Waste material located below the invert of the LFC, that also extends under the side slope, shall be completely removed below the channel invert for a distance of at least 10 feet horizontally from the toe of the final LFC side slope.

The limits of removal for inert material and construction debris shall be as follows:

1. At the LFC side slope, such material shall be overexcavated and removed to a horizontal distance of at least 3 feet from the side slope neat line, and to a depth of at least 8 feet below the LFC channel invert.

2. For the placement of RCC structures, such material shall be removed at least 3 feet beyond neat line limits of the structure.

In all cases where waste materials of any type have been removed, the resulting void shall be backfilled and compacted to the neat line.

Any waste material characterized and found to be of a hazardous nature, including asbestos-containing material, will be disposed of by the COP on-call environmental contractor.

All tires removed during excavation activities or recovered from the ground surface shall be handled, stored, transported, and disposed of in accordance with applicable federal, state, and local regulations. Applicable state regulations include: Arizona Revised Statues (ARS) §§44-1301 et seq: §44-1301; §44-1302; §44-1303; §44-1304.01; §44-1305; §44-1306; §44-1307.

5.3 CONTRACTORS EQUIPMENT AND MATERIAL YARD LOCATION

The Contractor may establish a Contractor's Work Area (CWA) in the bottom of the Salt River for the purpose of parking and servicing equipment, as well as establishing a roller compacted concrete (RCC) production plant. The Contractor understands that his use of the river bottom for a CWA is solely at his own risk.

1. The CWA must cover the least amount of acreage possible to accomplish the tasks required for the production plant and servicing of equipment.

2. The Contractor will monitor on a daily basis all activities in the CWA that may result in the leakage of oils, fluids, fuels, etc. which may contaminate soils in the river bottom, and promptly report any suspected leaks to the Engineer.

3. The Contractor will remove or clean up to background concentrations, and in accordance with applicable regulations test and properly dispose of all such contaminated soils resulting from the Contractors activities within the CWA and the river bottom on at least a biweekly basis, or more frequently at the direction of the Engineer. The Contractor shall provide all necessary documentation to the Engineer, including at a minimum the location, quantity, test results, and documentation of disposal of any such contaminated soils within one month after removal. At the discretion of the Engineer, the Contractor may be required to provide a cleanup plan for approval prior to addressing such contaminated soils.

4. The Contractor must create low diversion berms to direct surface flows away from the CWA so as to minimize the transport of contaminated soils downstream.

The Contractor may stockpile aggregate materials for the production of RCC in the river bottom. However the following criteria will be applied to the stockpiles:

1. The stockpiles can be no more than 100 feet wide at the base.

2. The long axis of the stockpiles must be oriented parallel to the direction of flow in the river.

3. Any remnant materials remaining from the stockpiles after completion of the project must be completely removed from the river bottom.

The Contractor shall obtain approval of the Engineer when using property outside the project limits of the river to park and service equipment and store materials for use. The Contractor will obtain prior written approval of the property owner for such use and submit a copy of the approval to the Engineer prior to use of the property.

The Contractor must provide the Owners field office construction trailer area outside of the river bottom. City of Phoenix right-of-way is available along the West Side of Central Avenue on the south side of the river for such field office use and as a possible site for Contractor construction trailers and general parking. This site is out of the river bottom, is accessed from the East Side of Central, and goes under the bridge via an existing high clearance box culvert.

5.4 HEALTH AND SAFETY

In accordance with standard operating guidelines for civil works projects, a health and safety plan has been written for the project site. The plan addresses appropriate personal protective equipment, identified hazards within the project limits, recommended safe operating procedures, as well as safety documentation and reporting procedures.

5.5 DEWATERING

Based on the hydrogeological data collected for the project site, it is anticipated that dewatering of structural excavation will be required. The water discharged from the excavation will be permitted for use as dust control but may not be used in the construction of the RCC structures.

Section 6

MONTGOMERY WATSON

6.0 OPERATIONAL CONSIDERATIONS

All project features located within the Phoenix Reach of the Salt River, including vegetation and infrastructure, are potentially subject to damage from long periods of flood inundation and significant high flows. Annual maintenance will be required to ensure the continued success of the project. Periodic clearing of the low flow channel would also be necessary to maintain the existing channel capacity. The low flow channel is designed to contain aquatic strand/scrub habitat and not large trees.

Average Annual Vegetation Damage

Assumptions on average annual vegetation damage presented in the following paragraphs were coordinated with environmental specialists on the study team. Vegetation within the channel would be periodically damaged due to flows exceeding the capacity of the low flow channel. This periodic occurrence would result in an operation and maintenance cost. To provide a basis for estimating this cost, the average annual vegetation damage has been estimated. It was estimated that the 100-year flood would damage about 95 percent of the vegetation in the channel. The largest flood that would not cause appreciable vegetation damage was taken as the flood that would first exceed the capacity of the low flow channel (typically less than a 5-year flood). The damage-discharge relationship was assumed to be a straight line between this event and the 100-year flood. The average annual vegetation damage was calculated by mathematically integrating the area under the established damage-frequency curve. Table 2 shows the calculated average annual vegetation damage.

TABLE 2

Reach (miles)	Average Annual Vegetation Damage (%)
Phoenix Reach – Station 212.12 to 214.99	8
Phoenix Reach – Station 215.09 to 215.65	7
Phoenix Reach – Station 215.75 to 216.33	7

Section 7

MONTGOMERY WATSON

7.0 CONSTRUCTION COST ESTIMATE AND SCHEDULE

7.1 COST ESTIMATE

Project cost estimates were prepared using the Government MCACES computer program and associated crew rates, labor rates, and unit price book. The Corps of Engineers prepared a 100% cost estimate for the Phase 1 portion of the project. The initial estimate for this phase was approximately $12 Million. R.E. Monks submitted the low bid for the project at $6.1 million. The cost differential was accounted for by the aggressive bidding climate at the time of advertisement. The contractor began work in June of 2000. Montgomery Watson has prepared a 90% cost estimate for the Phase 2 portion of the project. The engineer's cost estimate has been provided under a separate cover.

7.2 CONSTRUCTION SCHEDULE

Anticipated construction schedules for each phase of the project were developed in conjunction with the project cost estimate. The construction of the Phase 1 portion of the project was estimated to have an overall duration of approximately 300 days. The Phase 2 portion of the project is estimated to require approximately 330 days to complete.

Section 8

MONTGOMERY WATSON

8.0 QUALITY CONTROL PLAN AND TECHNICAL REVIEW DOCUMENTATION

In accordance with U.S. Army Corps of Engineers Quality Management Plan (CESPD R 1110-1-8), a quality control plan was written for each phase of the project. A copy of the quality control plan is maintained on record by the Corps of Engineers.

Section 9

MONTGOMERY WATSON

9.0 STATEMENT OF TECHNICAL AND LEGAL REVIEW

To be provided at the conclusion of construction for Phases 1 and 2 of the project.

Appendix A

MONTGOMERY WATSON

APPENDIX A

Geotechnical Data

Geotechnical Investigation Report
Design of Grade Control Structures
AGRA Earth & Environmental, Inc.
(May 16, 2000)
Prepared for West Consultants

US Army Corps of Engineers
Logs of Exploration for the Rio Salado Project

GEOTECHNICAL INVESTIGATION REPORT
DESIGN OF GRADE CONTROL STRUCTURES
PHOENIX, ARIZONA

Submitted To:

WEST Consultants, Inc.
2151 East Broadway Road
Suite 116
Tempe, Arizona 85282-1705

Submitted By:

AGRA Earth & Environmental, Inc.
3232 West Virginia Avenue
Phoenix, Arizona 85009-1502

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13872
LAWRENCE A.
HANSEN
Date Signed 5/16/00
ARIZONA, U.S.A.

May 16, 2000
AGRA Job No. 0-117-001007

AGRA
ENGINEERING GLOBAL SOLUTIONS

AGRA
ENGINEERING GLOBAL SOLUTIONS

AGRA Earth &
Environmental Inc.
3232 West Virginia Avenue
Phoenix, Arizona 85009
Tel (602) 272-6848
Fax (602) 272-7239
Toll Free 1-800-248-AGRA

May 16, 2000
AGRA Job No. 0-117-001007

WEST Consultants, Inc.
2151 East Broadway
Suite 116
Tempe, Arizona 85282-1705

Attention: Dennis L. Richards, P.E.

Gentlemen:

**RE: GEOTECHNICAL INVESTIGATION
DESIGN OF GRADE CONTROL STRUCTURES
PHOENIX, ARIZONA**

Submitted herein is our final Geotechnical Investigation Report for the grade control structures planned for the Phoenix Rio Salado project. The report presents the results of a geotechnical drilling program, analytical chemistry analyses of samples of groundwater collected at the four locations investigated, general recommendations for design of the grade control structures, and specific recommendations for the grade control structure to be located near the Central Avenue bridge.

Should you have any questions regarding the recommendations presented in this report, please do not hesitate in contacting the undersigned.

Respectfully submitted,

AGRA Earth & Environmental, Inc.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13872
LAWRENCE A.
HANSEN
Date Signed 5/16/00
ARIZONA, U.S.A.

Lawrence A. Hansen, Ph.D., P.E.
Senior Vice President

c: Addressee (1)
U.S. Army Corps of Engineers, LAD
Attn: CESPL-ED-DB(13012, Desai) (8)

G:\Engineering-Development\2000Projects\0-117-001007GradeControlStruc\finalgrcreport.wpd

TABLE OF CONTENTS

APPENDICES

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13372
LAWRENCE A.
HANSEN
Date Signed 5/16/00
ARIZONA, U.S.A.

1.0 INTRODUCTION

This report is submitted pursuant to a geotechnical investigation performed by AGRA Earth & Environmental, Inc. (AGRA) of the planned locations of four grade control structures for the Phoenix Rio Salado project. The report presents the results of a geotechnical drilling program, analytical chemistry analyses of samples of groundwater collected at the locations of the grade control structures, general geotechnical recommendations for design of the grade control structures, and the results of specific geotechnical analyses for the grade control structure to be located near Central Avenue.

2.0 PROJECT DESCRIPTION

Details of the project were provided by Dennis L. Richards, P.E. of WEST Consultants, Inc. and in the scope of work developed by the U.S. Army Corps of Engineers (USACOE) for Task Order No. 0010 of Contract No. DACW09-97-D-0022. It is the understanding of AGRA that a low flow channel will be constructed withing the Salt River between about 7th Avenue to the west and the Interstate 10 (I-10) bridge to the east. Grade control structures will be constructed near the Central Avenue bridge and at river miles 214.65, 215.56 and 216.23. Construction plans (90 percent submittal) for the Central Avenue grade control structure were reviewed as part of the project.

3.0 INVESTIGATION

3.1 HEALTH & SAFETY PLAN

The project site generally is located within an area potentially impacted by adjacent facilities, possibly resulting in contamination of the soils and groundwater. Specific facilities that may have impacted the site include the old Del Rio Landfill located on the south side of the Salt River between 7th and 16th Streets, the Estes Landfill located upstream of the I-10 bridge and the south side of the Salt River, and the Motorola NPL site which includes Sky Harbor International Airport. Since the groundwater in the areas investigated may have been contaminated, AGRA maintained the site-specific health and safety plan prepared by the City of Phoenix. Ambient air quality monitoring was performed during the drilling operations.

3.2 EXPLORATION FOR GRADE CONTROL STRUCTURES

AGRA's field geologist was on-site between March 13, 2000 and March 15, 2000 to conduct field activities including ambient air quality monitoring, logging of borings and groundwater sampling. Subsurface borings were located 2,170 feet east of the upstream face of the 24th Street bridge on the north side of the channel (Boring B-1), 800 feet west of the downstream face of the 24th Street bridge in the center of the channel (Boring B-2), 650 feet west of the downstream face of the 16th Street bridge in the center of the channel (Boring B-3), and 150 feet west of the downstream face of the Central Avenue bridge in the center of the channel (Boring B-4). Boring locations are shown on the site plan included in Appendix A.

AGRA subcontracted an AP-1000 drill rig and auxiliary vehicles from Layne Christensen Company to complete the borings at the locations specified to a depth of 55 feet below existing subgrade. No water or drilling mud was added to the borings. The drill pipe was steam cleaned between borings to prevent cross-contamination. The borings were continuously logged in the field in accordance with the Unified Soil Classification System (USCS) ASTM D2488. The sediments in the Salt River consisted primarily of sand, gravel and cobbles. Because of the very coarse nature of the alluvium, soil samples were not collected. However, the effort required to advance the 9-inch diameter casing, as indicated by the number of blows per foot, was continuously recorded. The logs of the borings are provided in Appendix A, along with a brief description of drilling equipment and procedures, and a summary of the Unified Classification System.

As a precaution, drill cuttings from depths at or below the water table were containerized in 55-gallon drums because of potential soil contamination. These drums were placed in a secure location provided by the City of Phoenix at 7th Avenue and the Rio Salado until arrangements can be made for proper disposal pending laboratory analysis. The four borings were abandoned by backfilling with cement grout in accordance with Arizona Department of Water Quality standards.

Ambient air quality monitoring was conducted during the exploration program using a PID meter. Meter readings and the time of day when they were recorded are listed on the boring logs in Appendix A. Meter readings typically were less than 0.5, with a maximum reading of 1.1 recorded at the location of Boring B-3.

3.3 GROUNDWATER SAMPLING & TESTING

Groundwater samples were obtained for water quality testing during the geotechnical investigation. Groundwater sampling was not performed with the intent to fully characterize water quality; however, the geotechnical investigation provided an opportunity to obtain limited information regarding the potential impact to water quality from facilities and landfills located adjacent to the Salt River which are known or suspected to have contaminated groundwater. Water quality sampling would address the risk posed by potentially contaminated groundwater to the health of workers completing the Rio Salado Project.

Installation of monitor wells with perforated casing in the groundwater interval was not part of the scope of work. Groundwater was not purged from the boreholes because the drill casing was not perforated and purging would not have efficiently or effectively drawn formation water into the borehole to be sampled. In addition, purging each 9-inch diameter boring would have generated a significant volume of water, which would have required containerizing pending the results of the water quality analysis. If this water had been contaminated, it would have required proper disposal.

Grab samples were collected from the open drill casing of the AP-1000 drill rig, as depicted in the diagram in Appendix A. The water level was allowed to stabilize for at least thirty minutes prior to sampling. Each well was sampled using a new disposable bailer. Upon retrieval of the bailer, the groundwater was transferred to laboratory prepared containers, labeled, stored and handled in accordance with AGRA standard sampling protocol, which has been prepared in accordance with guidelines specified by the Arizona Department of Environmental Quality (ADEQ).

The samples were submitted to the AGRA laboratory in Portland, Oregon, a State of Arizona certified laboratory. The laboratory testing program completed by AGRA included analyses for volatile organic compounds (VOCs) by EPA Method 8260B; for semi-volatile organic compounds (SVOCs) by EPA Method 8270C; for polychlorinated biphenyls (PCBs) by EPA Method 8082; and for pesticides by EPA Method 8081A.

Samples also were submitted by AGRA to SVL Analytical, Inc. (SVL) in Kellogg, Idaho, a State of Arizona certified laboratory. The laboratory testing program completed by SVL included a target analyte list (TAL) of metals (23 metals) by EPA Method 6010B; hardness as calcium chloride by EPA Method 200.7; sulfate, sulfides, nitrates, nitrites and chlorides by EPA Method 300.0; and total dissolved solids, pH, sulfides and calcium carbonate by EPA 100-series methods.

Although groundwater samples were handled in general accordance with ADEQ protocol, samples were not collected under conditions which would provide a representative groundwater sample. As discussed above, collecting a representative groundwater sample would have required the installation of a groundwater well with screened casing in the saturated interval, and purging the well of three casing volumes prior to sample collection. Recent precipitation resulted in flowing water in the river channel prior to the geotechnical investigation and water quality sampling. There likely is some recharge of surface water to groundwater through the coarse alluvial material in the river channel. AGRA is of the opinion that the presence of surface flow may cause the local groundwater conditions to vary from times when surface flow is not present.

Since the drilling method uses air injected into the formation and because friction heats the casing, the concentration of VOCs and SVOCs in the groundwater may have been altered by sampling directly from the drill casing. However, AGRA is of the opinion that high concentrations of VOCs or SVOCs, or the presence of free product, would have been detected with the sampling method which was used. PCBs, pesticides, and total and dissolved metals concentrations likely were unaffected by the sampling method.

Detailed summaries of the laboratory analyses, including copies of chain of custody forms and laboratory sample receipt documentation forms, are presented in Appendix B. Level III Quality Assurance/Quality Control (QA/QC) procedures were performed by the analytical laboratories. The standard QA/QC data package is included in Appendix B. The remainder of the Level III QA/QC documentation is maintained in AGRA's project files.

4.0 DISCUSSION & SUMMARY

Based on the exploratory investigation, deposits of sand, gravel and cobbles were encountered to the full depths of the borings at the four locations planned for the grade control structures. Penetration resistance values typically varied from about 35 to 85. As indicated by these values, lenses and thin layers containing more sand and less gravel and cobbles were encountered at various depths throughout the borings. No significant or extensive predominately sand deposits are indicated by the penetration resistance profiles. Typically, the deposits contain less than about 5 to 10 percent silt and clay, based on AGRA's previous experience. Groundwater was encountered at depths varying from about 29 feet (Boring B-3) to between about 37 to 38 feet (Borings B-1, B-2 and B-4).

A summary of the analytical results is presented in Table 1. No VOCs, SVOCs, PCBs, or pesticides were detected at concentrations above their respective laboratory method reporting limits. Total dissolved solids concentrations varied from 410 to 665 milligrams per liter (mg/L), sulfate concentrations varied from 57.0 to 98.9 mg/L, and chlorides concentrations varied from 103 to 220 mg/L. Total concentrations of most metals were near or below their respective laboratory method reporting limits. However, total concentrations of sodium varied from 82.6 to 135 mg/L, total concentrations of magnesium varied from 21.1 to 53.2 mg/L, and total concentrations of calcium varied from 53.5 to 80.5 mg/L. Dissolved concentrations of these metals were of a similar order of magnitude. Total concentrations of aluminum varied from 5.5 to 103 mg/L, but the dissolved concentrations of this metal were either less than or only slightly above the detection limits.

5.0 ANALYSIS & RECOMMENDATIONS

Generally, roller compacted concrete structures are planned for the proposed low flow control facilities. The structures will include upstream aprons, bank and overbank protection dikes, channel weirs and channel grade control structures. The loads imposed by these structures are expected to be relatively low and spread over large areas. The maximum loads likely will be imposed by the channel weir structures. Allowable bearing pressures on the order of 10,000 to 12,000 pounds per square foot (psf) typically are assigned for design of isolated spread footings bearing on the sand, gravel and cobbles in order to limit settlements to acceptable values. For the general types of structures planned, neither bearing capacity nor settlement considerations will control the design.

The potential for uplift or flotation due to buoyant forces, and for sliding due to unbalanced lateral forces, however, are primary design considerations. AGRA completed evaluations of the stability of the grade control structure and apron to be located downstream of the Central Avenue bridge. These analyses were detailed in Letter No. 1 dated February 15, 2000 and Letter No. 2 dated March 3, 2000. These letter reports are included in Appendix C of this report. The analyses generally were completed in accordance with USACOE guidance manuals*.

As presented in Letter No. 2, the combined weight of the channel weir and the channel grade control elements, assuming no redeposition of granular materials following a flow event, is about equal to the buoyant force acting on these elements, resulting in a factor of safety near unity. If the flow event results in redeposition of 50 percent of the granular fill, the factor of safety against buoyancy is increased to 1.52 and the factor of safety against sliding is 1.65. If the upstream apron and connecting bank control elements are included in the overall structure, but again assuming no redeposition of granular materials, the factors of safety against buoyancy and sliding are 1.39 and 1.82, respectively.

Based on the analyses completed, it is concluded the structure will remain stable relative to sliding and buoyancy considerations for the case where the flood waters have receded to a level coincident with the top of the upstream apron. This case reflects the most extreme buoyant condition that can exist. Because of the distribution of forces acting on the various parts of the grade control structure, moments and shear forces would need to resisted by the roller compacted concrete, particularly at the connection between the channel grade control structure and the weir, and the connections between the channel grade control structure and the bank control elements. This analysis is not part of the scope of work reported herein.

---

*U.S. Army Corps of Engineers, Flotation Stability Criteria for Concrete Hydraulic Structures, ETL 1110-2-307, 20 August 1987.

U.S. Army Corps of Engineers, Sliding Stability for Concrete Structures, ETL 1110-2-256, 24 June 1981.

TABLE 1
SUMMARY OF GROUNDWATER SAMPLE ANALYTICAL TEST RESULTS

ANALYSIS	CONSTITUENT	SAMPLE NAME			
		B-1-GW	B-2-GW	B-3-GW	B-4-GW
EPA 130.2	$CaCO_3$ (mg/L)	<1.0	<1.0	<1.0	<1.0
EPA 160.1	TDS (mg/L)	665	549	410	504
EPA 200.7	Hardness	284	420	288	220
EPA 180.1	Sulfide	<0.5	<0.5	<0.5	<0.5
EPA 150.1	pH	7.79	7.78	7.71	8.00
EPA 300.0	Sulfate (mg/L) Sulfides Nitrates (mg/L) Nitrites Chlorides (mg/L)	93.2 <0.5 2.6 <0.25 220	94.9 <0.5 2.66 <0.25 176	57.0 <0.5 1.48 <0.5 103	98.9 <0.5 <0.05 <0.1 105

TABLE 1 (CONT.)
SUMMARY OF GROUNDWATER SAMPLE ANALYTICAL TEST RESULTS

ANALYSIS	CONSTITUENT	SAMPLE NAME							
		B-1-GW		B-2-GW		B-3-GW		B-4-GW	
		Total	Dissolved	Total	Dissolved	Total	Dissolved	Total	Dissolved
Total Metals (mg/L)	Aluminum	5.5	<0.024	103	0.049	66.9	0.048	24.6	<0.024
	Antimony	<0.032	<0.032	<0.032	<0.032	<0.032	0.034	0.033	<0.032
	Arsenic	<0.04	<0.04	<0.04	<0.04	<0.04	<0.04	<0.04	<0.04
	Barium	0.156	0.101	0.842	0.063	0.959	0.080	0.248	0.069
	Beryllium	<0.002	<0.002	0.003	<0.002	0.002	<0.002	<0.002	<0.002
	Cadmium	<0.0024	<0.0024	0.0025	<0.0024	<0.0024	<0.0024	<0.0024	<0.0024
	Calcium	67.5	71.8	80.5	57.7	56.9	40.4	53.5	47.4
	Chromium	0.007	<0.005	0.129	<0.005	0.091	<0.005	0.037	<0.005
	Cobalt	<0.005	<0.005	0.056	0.006	0.050	<0.005	0.012	<0.005
	Copper	0.019	<0.003	0.283	<0.003	0.280	0.007	0.074	0.005
	Iron	10.1	<0.02	75.8	0.04	50.0	0.02	25.5	0.02
	Lead	<0.04	<0.04	<0.04	<0.04	<0.04	<0.04	<0.04	<0.04
	Magnesium	28.1	30.1	53.1	24.0	35.4	15.7	21.1	14.9
	Manganese	0.805	0.483	6.35	0.306	8.5	1.49	1.60	0.471
	Mercury	<0.0002	<0.0002	0.0003	<0.0002	<0.0002	<0.0002	<0.0002	<0.0002
	Nickel	<0.023	<0.023	0.187	<0.023	0.258	<0.023	0.035	<0.023
	Potassium	7.8	6.3	16.2	5.4	12.2	5.7	8.7	6.6
	Selenium	<0.04	<0.04	0.14	<0.04	0.04	<0.04	<0.04	<0.04
	Silver	<0.006	<0.006	0.007	<0.006	<0.006	<0.006	<0.006	<0.006
	Sodium	135	145	104	114	82.6	88.0	112	117
	Thallium	<0.1	<0.1	<0.1	<0.1	<0.1	0.1	<0.1	<0.1
	Vanadium	0.008	<0.005	0.139	<0.005	0.100	<0.005	0.042	<0.005
	Zinc	0.021	<0.003	0.162	<0.003	0.116	<0.003	0.042	<0.003

TABLE 1 (CONT.)
SUMMARY OF GROUNDWATER SAMPLE ANALYTICAL TEST RESULTS

ANALYSIS	CONSTITUENT	SAMPLE NAME			
		B-1-GW	B-2-GW	B-3-GW	B-4-GW
Volatiles EPA 8260B		ND*	ND*	ND*	ND*
Semi-Volatiles EPA 8270C		ND*	ND*	ND*	ND*
PCBs EPA 8082		ND*	ND*	ND*	ND*
Pesticides EPA 8081A		ND*	ND*	ND*	ND*

*ND - No analytes were reported at concentrations which exceeded their respective laboratory method detection limits. Refer to the laboratory analytical data.

Appendix B

MONTGOMERY WATSON

APPENDIX B

RCC Design

U.S. Army Corps of Engineers
Rio Salado Project – Report on RCC for Construction

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	Cemen t	Pozzol an	Water	W/(C+P)	(Secs)	(Pct)	7-Day	28-Day	56-Da y	90-Da y
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

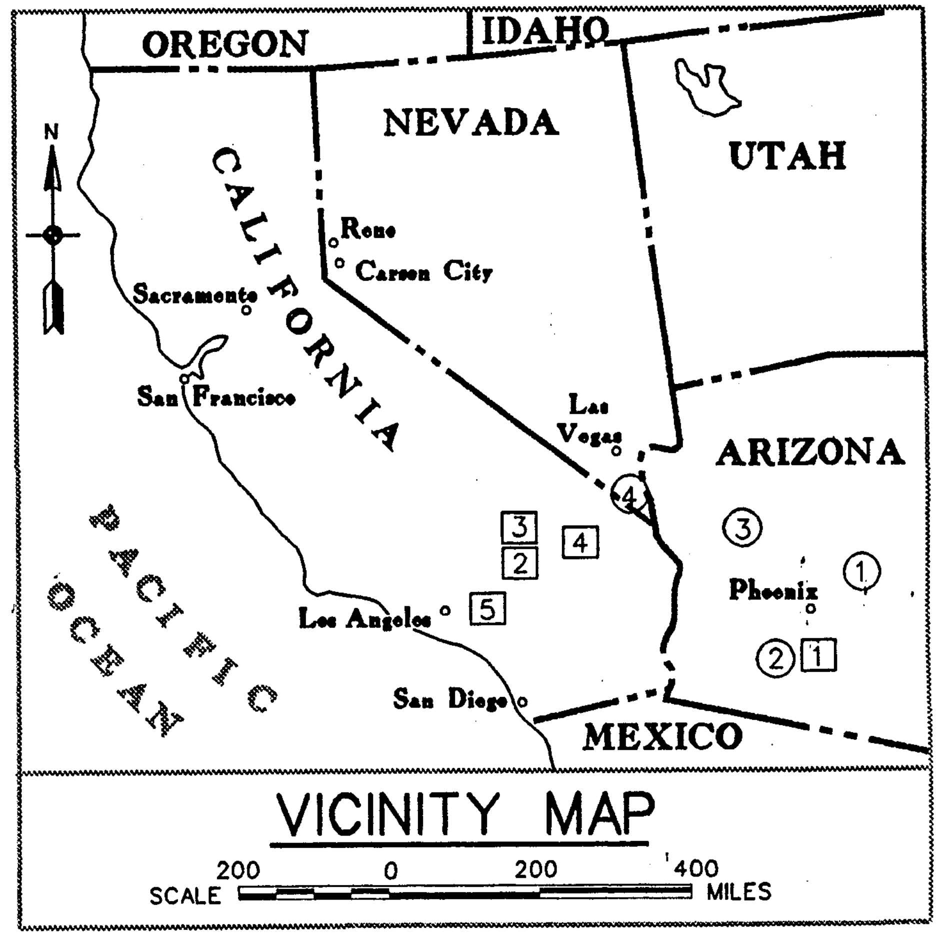
SOURCES OF CEMENTS AND POZZOLANS

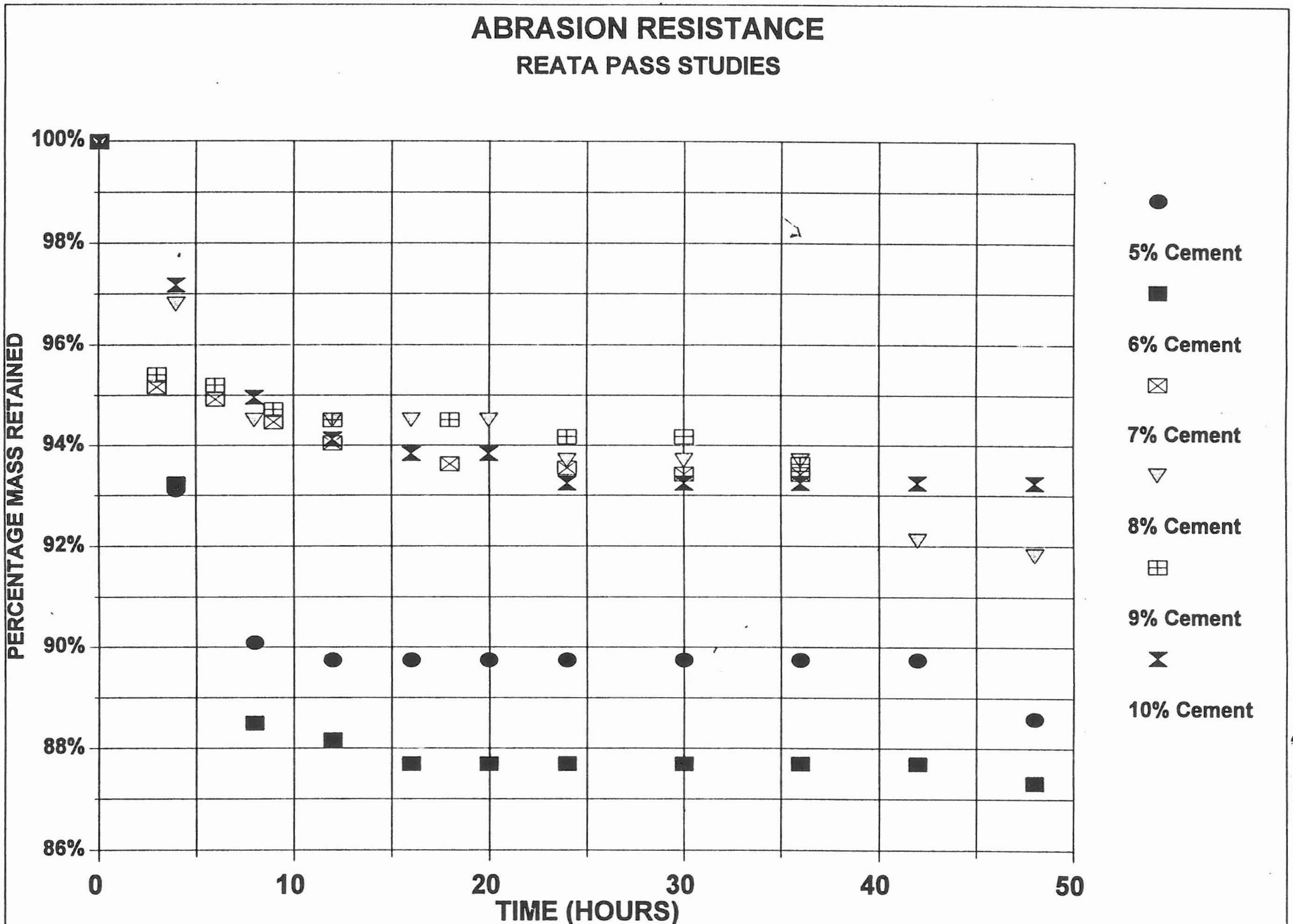
SOURCES OF CEMENT

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 BOREAL MATERIALS, LAUGHLIN, NEVADA





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

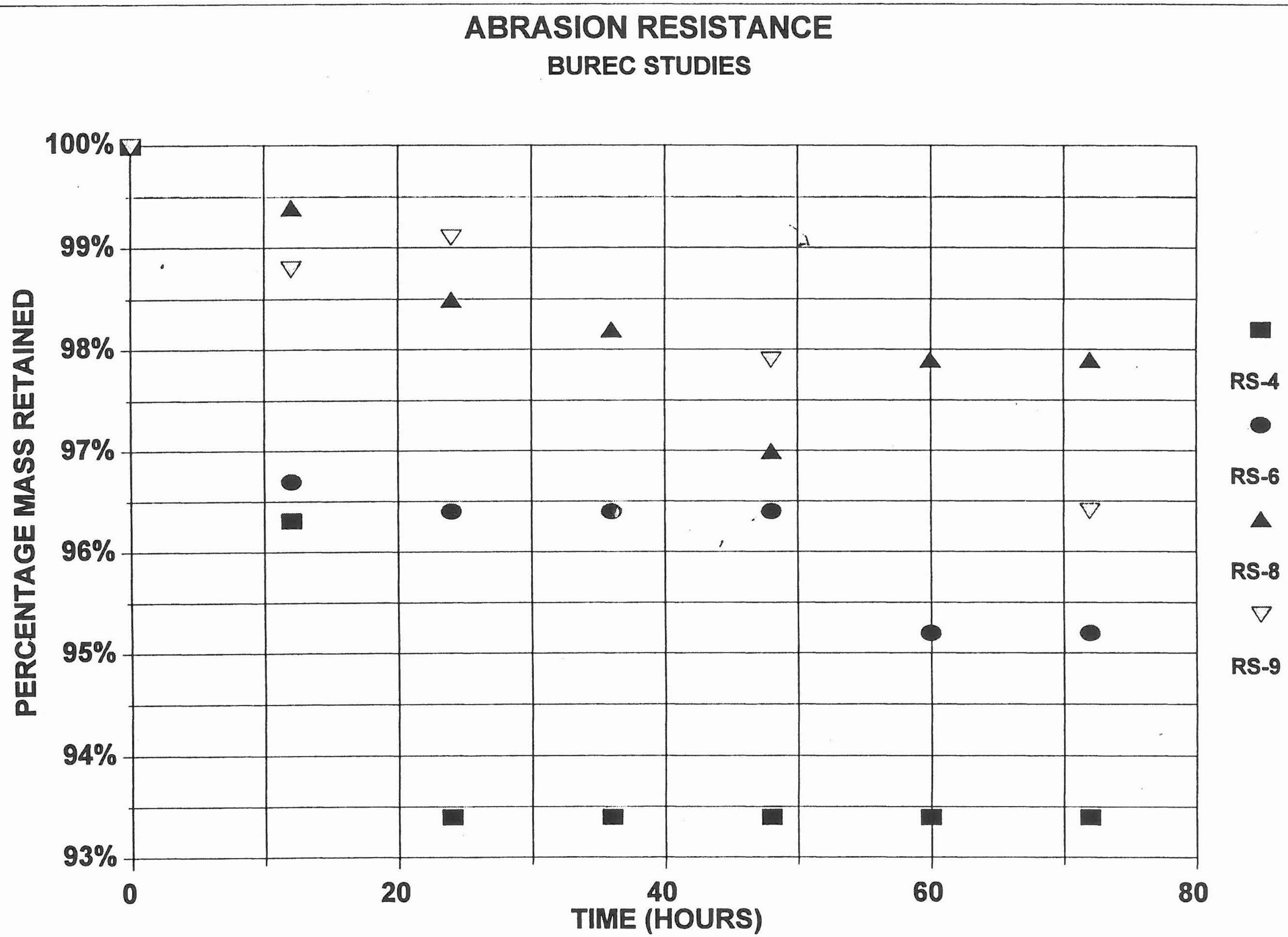

ABRASION RESISTANCE
BUREC STUDIES
PERCENTAGE MASS RETAINED
100%
99%
98%
97%
96%
95%
94%
93%
0
20
40
60
80
TIME (HOURS)
RS-4
RS-6
RS-8
RS-9

FIGURE 2

Rio Salado

USACE vs MCFCD

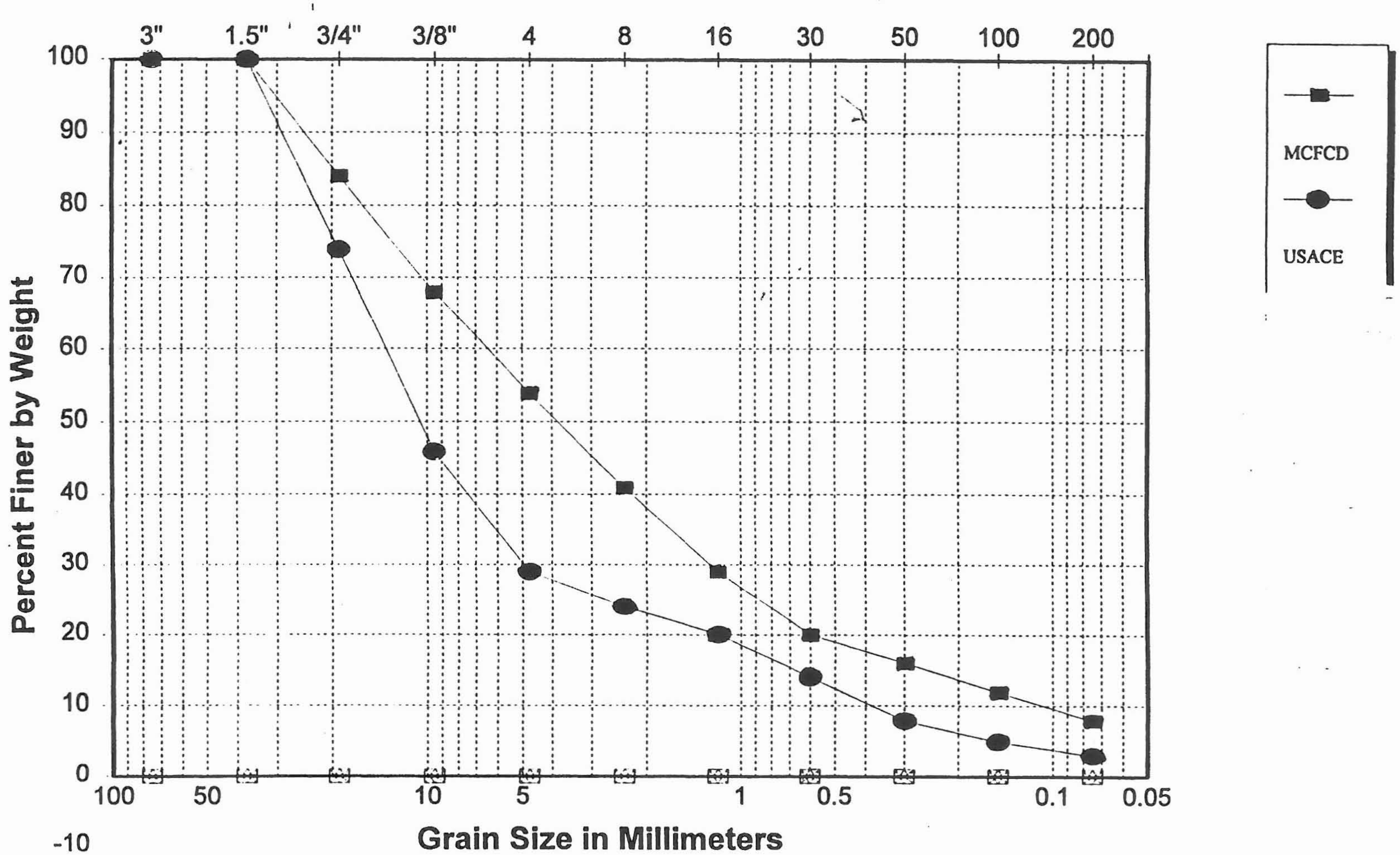


FIGURE 3

Appendix C

MONTGOMERY WATSON

APPENDIX C

Hydraulic Analysis

West Consultants (January 11, 2000).
Low Flow Channel Design Analysis
for Rio Salado (Salt River), Arizona.
Prepared for the U.S. Army Corps of Engineers,
Los Angeles District.

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).

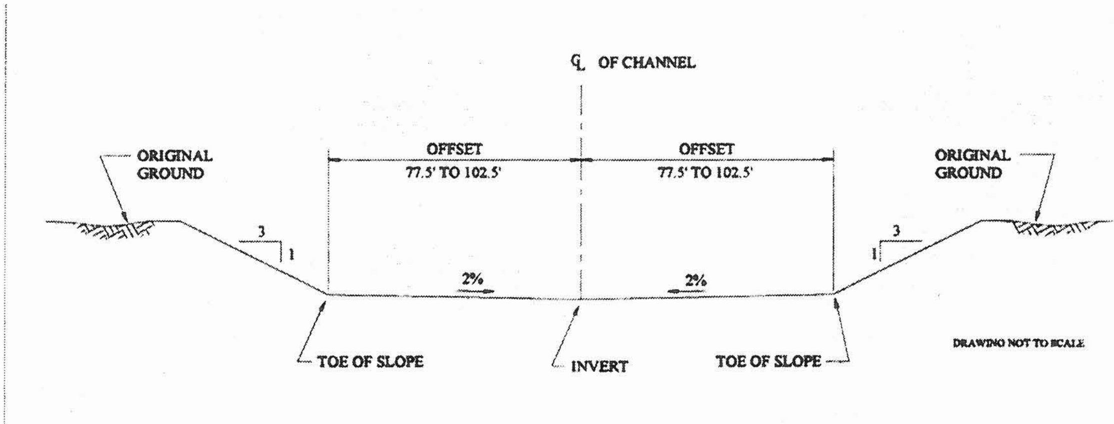


Figure 2-1. Typical cross section of the low flow channel.

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.

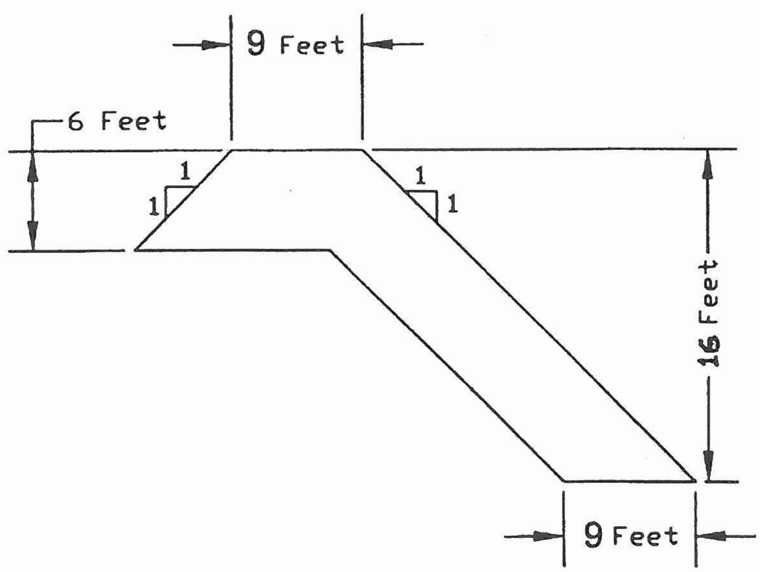


Figure 2-2. Typical cross section of soil cement grade control sill in overbank area, with a downstream toe-down depth of 16 feet. Within low flow channel, the toe-down depth should be 27 feet.

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.

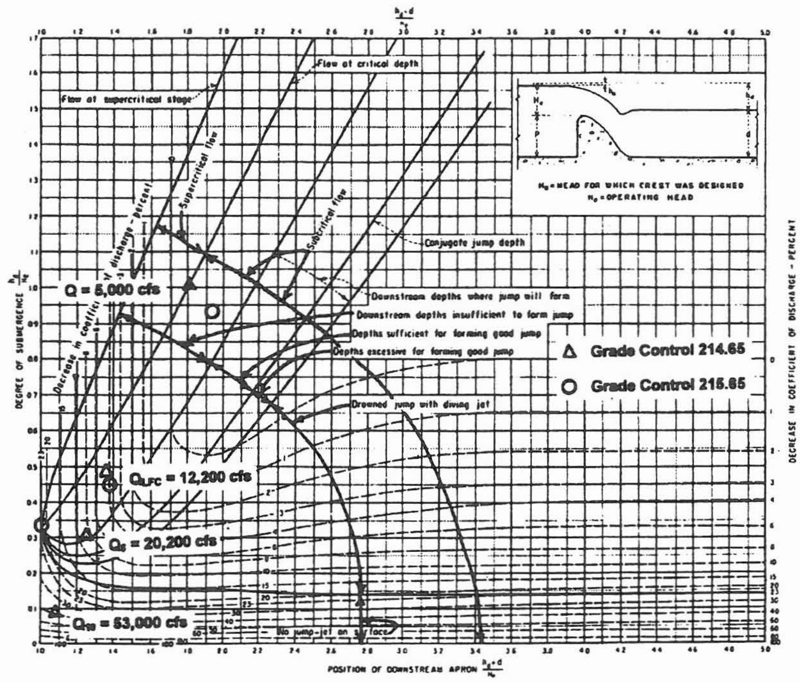


Figure 2-3. Plot of hydraulic drop-specific head ratio versus total head-specific head ratio used for evaluating grade control structure hydraulics (flow classification chart from USBR, 1977).

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.

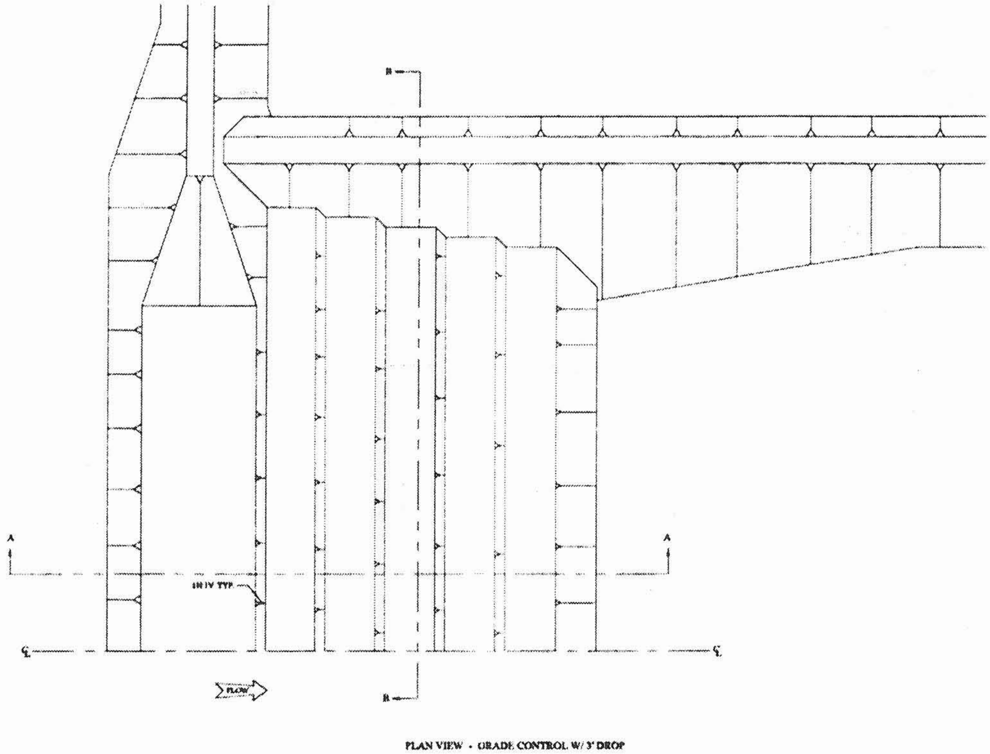


Figure 2-4. Typical plan view of grade control with 3 foot drop. Centerline of channel is at bottom of figure.

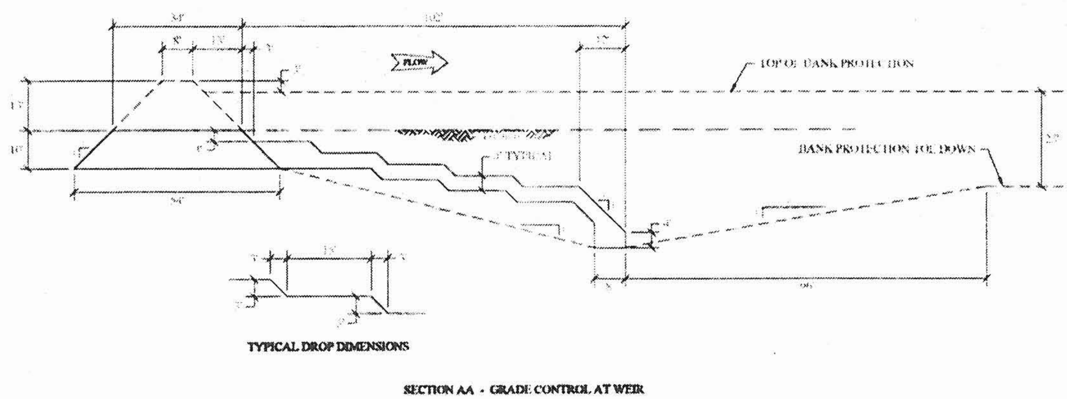


Figure 2-5. Cross section along low flow channel centerline of grade control structure with 3 foot drop. Soil cement apron is stepped to 10-year scour depth and then sloped at 1H:1V to the 100-year scour depth.

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.

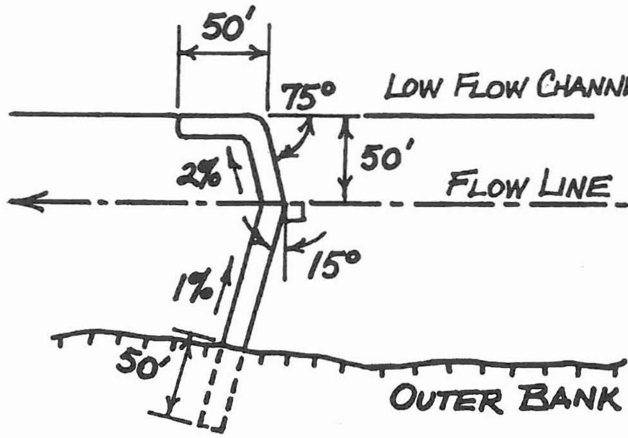


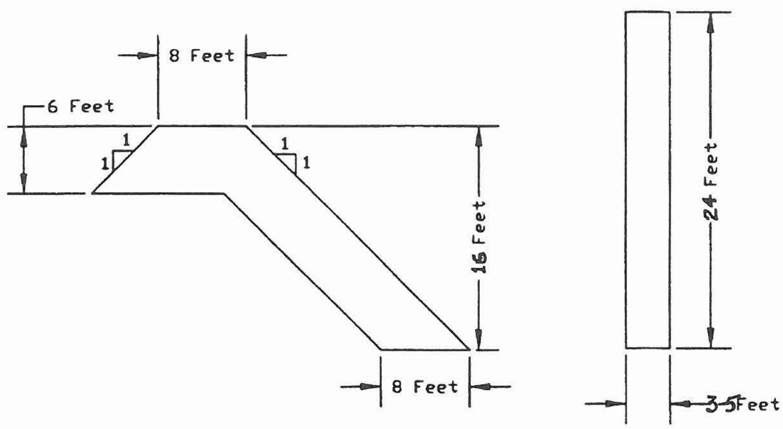
Figure 2-6. Typical plan view of originally proposed guide dike installation.

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.



(a) (b)

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

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

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

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

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

2.3.2. Economic Justification

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

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

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

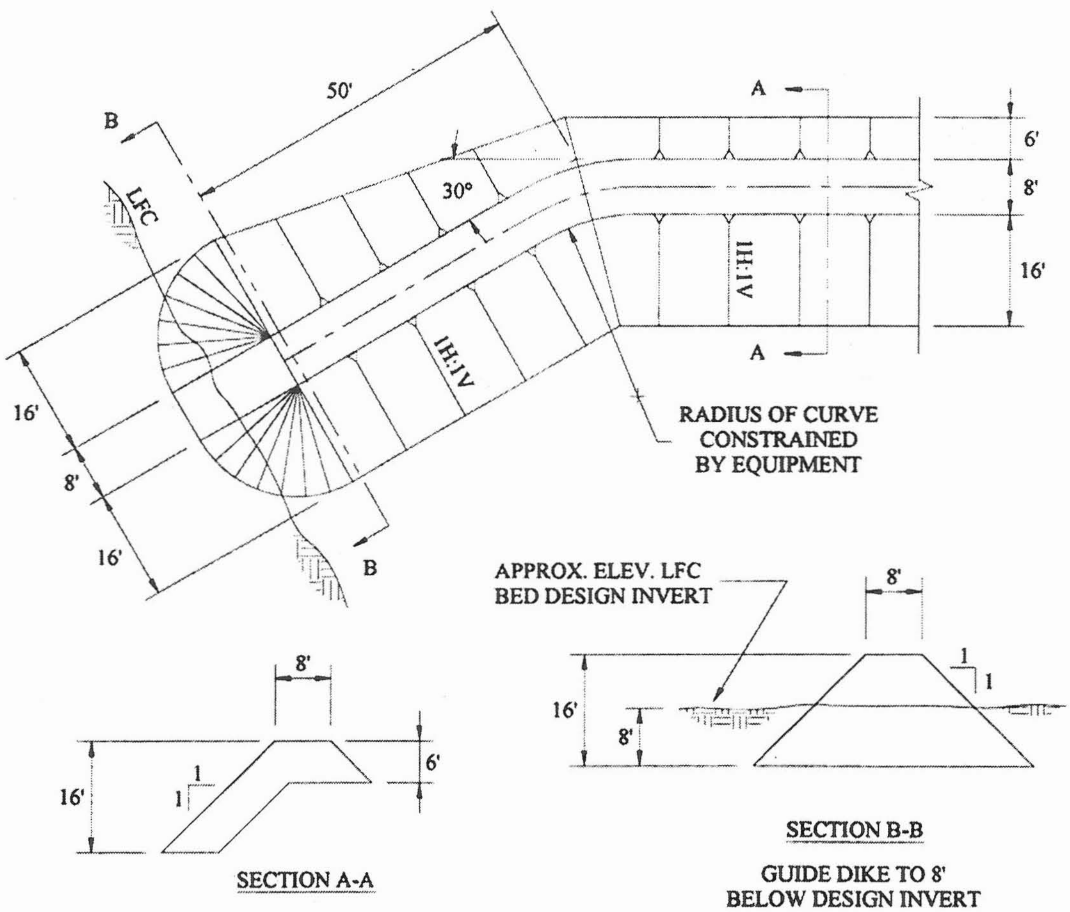


Figure 2-8. Plan view of revised guide dike design, end detail for low flow channel side.

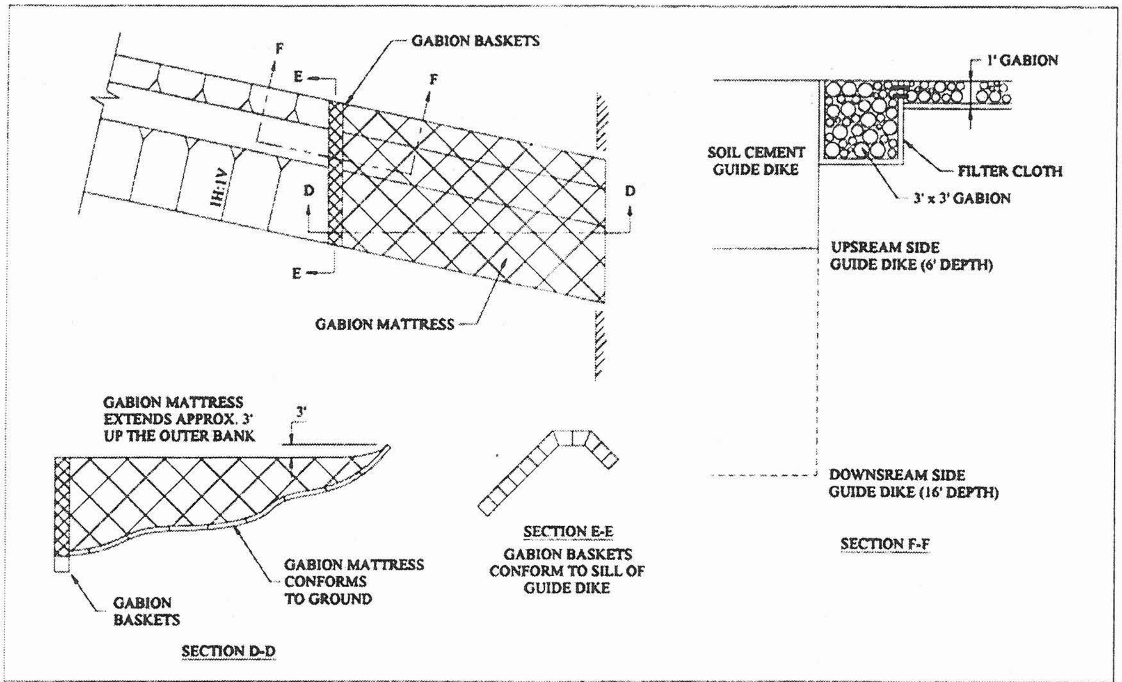


Figure 2-9. Plan view of revised guide dike design, end detail for transition from soil cemebt to gabions near outer bank.

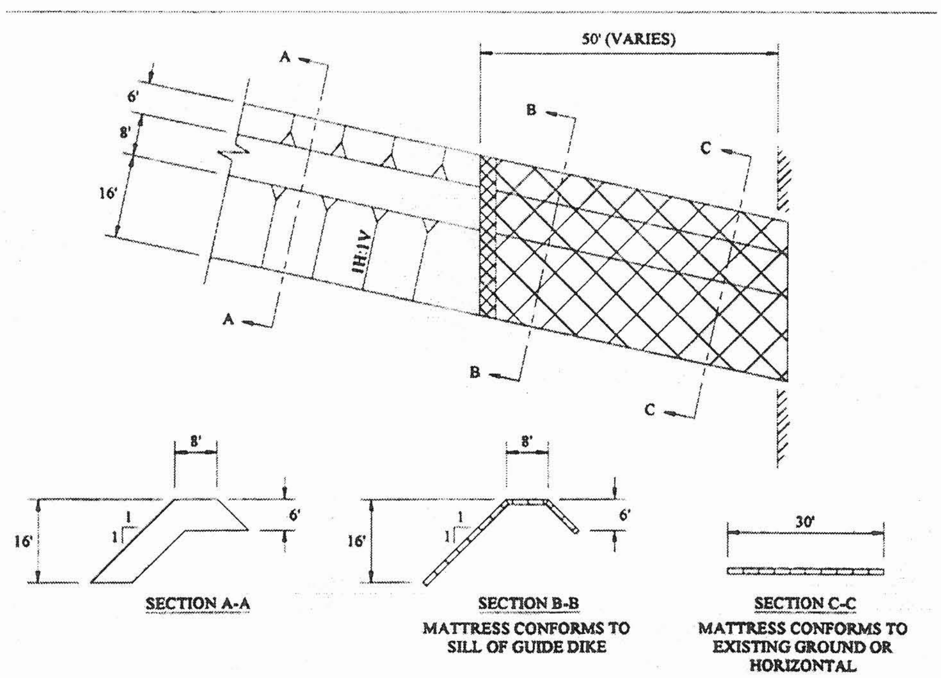


Figure 2-10. Plan view of revised guide dike design, end detail for layout of gabion matress near outer bank.

2.4. ADDITIONAL BANK PROTECTION MEASURES

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

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

2.5. OTHER CONSIDERATIONS

APS transmission towers are located within the Salt River from west of 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

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

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

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

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

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

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

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

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

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

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

3. HYDRAULIC ANALYSIS

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

3.1. Existing Conditions HEC-RAS Model

3.1.1. Model Description

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

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

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

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

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

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

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

A comparison of WEST's existing condition 100-year water surface profile and the 1998 proposed FIS 100-year water surface profile is shown in Figure 3-1. The water surface elevations in WEST's model are generally lower than those of the 1998 proposed FIS model, except near 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.

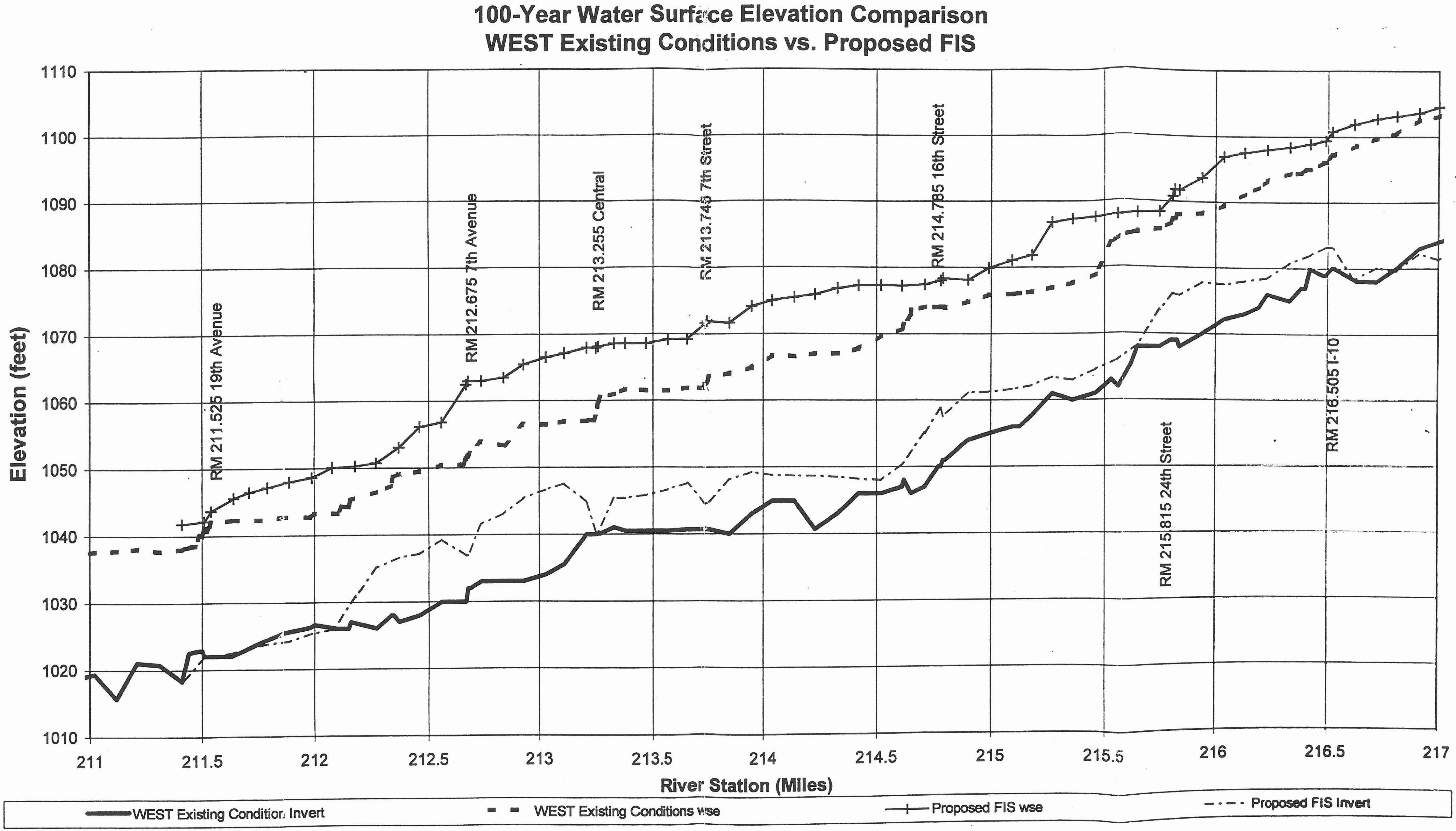
Given the long distance and length of time it will take to reach the project reach, the flood wave from a dam failure at Tempe Town Lake would be greatly attenuated. And since the flood wave attenuates much faster than that of a normal flood event, it would not have an effect any worse than a normal 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

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

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

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



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

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

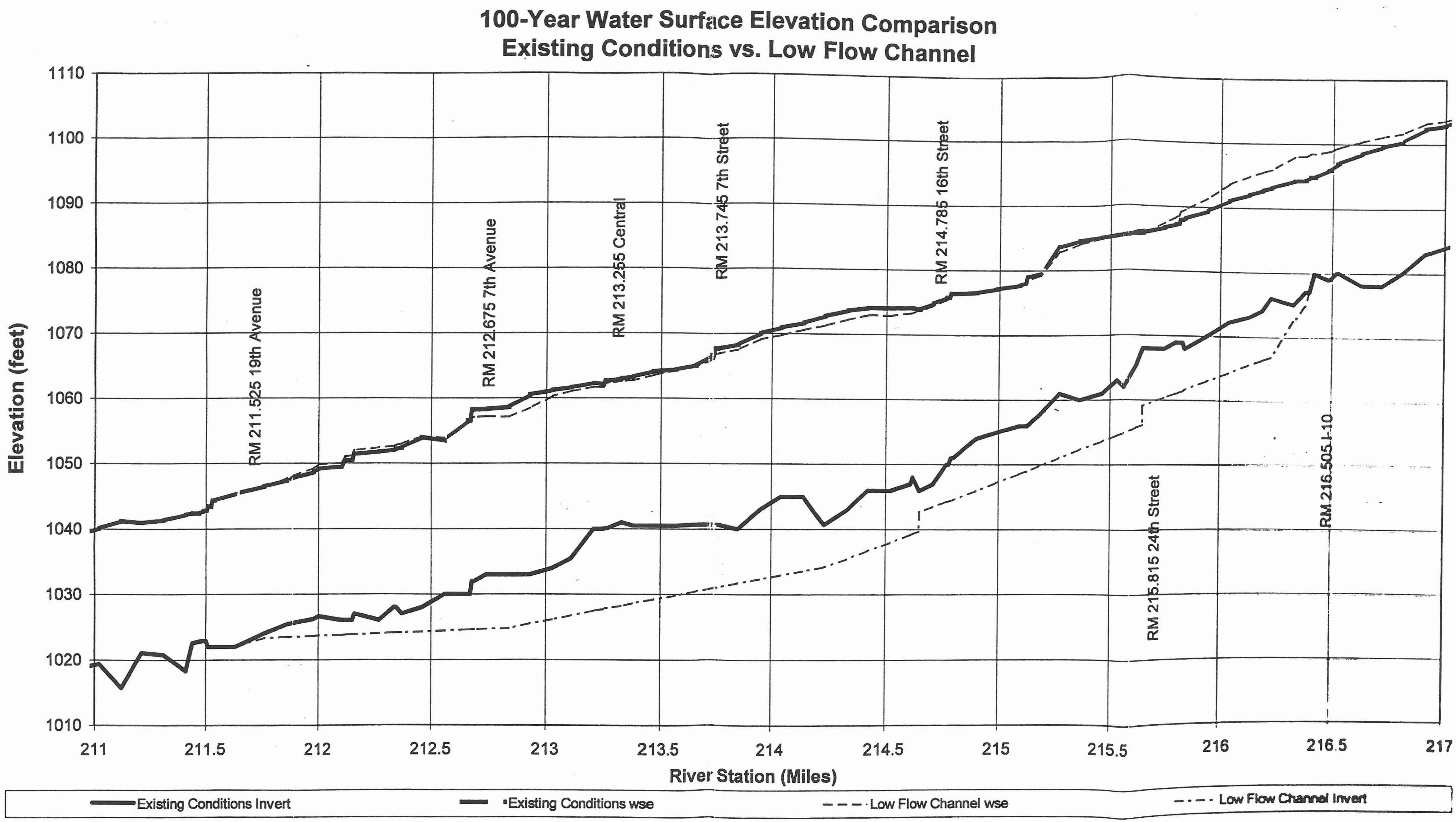


Figure 3-2. Comparison of WEST's existing condition and low flow channel water surface profiles.

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

4. SEDIMENT TRANSPORT ANALYSIS

4.1. INTRODUCTION

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

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

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

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

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

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

4.2. GRAVEL MINING ANALYSIS

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

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

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

4.3. HEC-6T Model Inputs

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

4.3.1. Geometric Data

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

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

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

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

4.3.2. Sediment Data

4.3.2.1. Existing Conditions Bed Gradations

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

4.3.2.2. Low Flow Channel Gradations

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

4.3.3. Sediment Transport Method

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

4.3.4. Inflowing Sediment Load

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

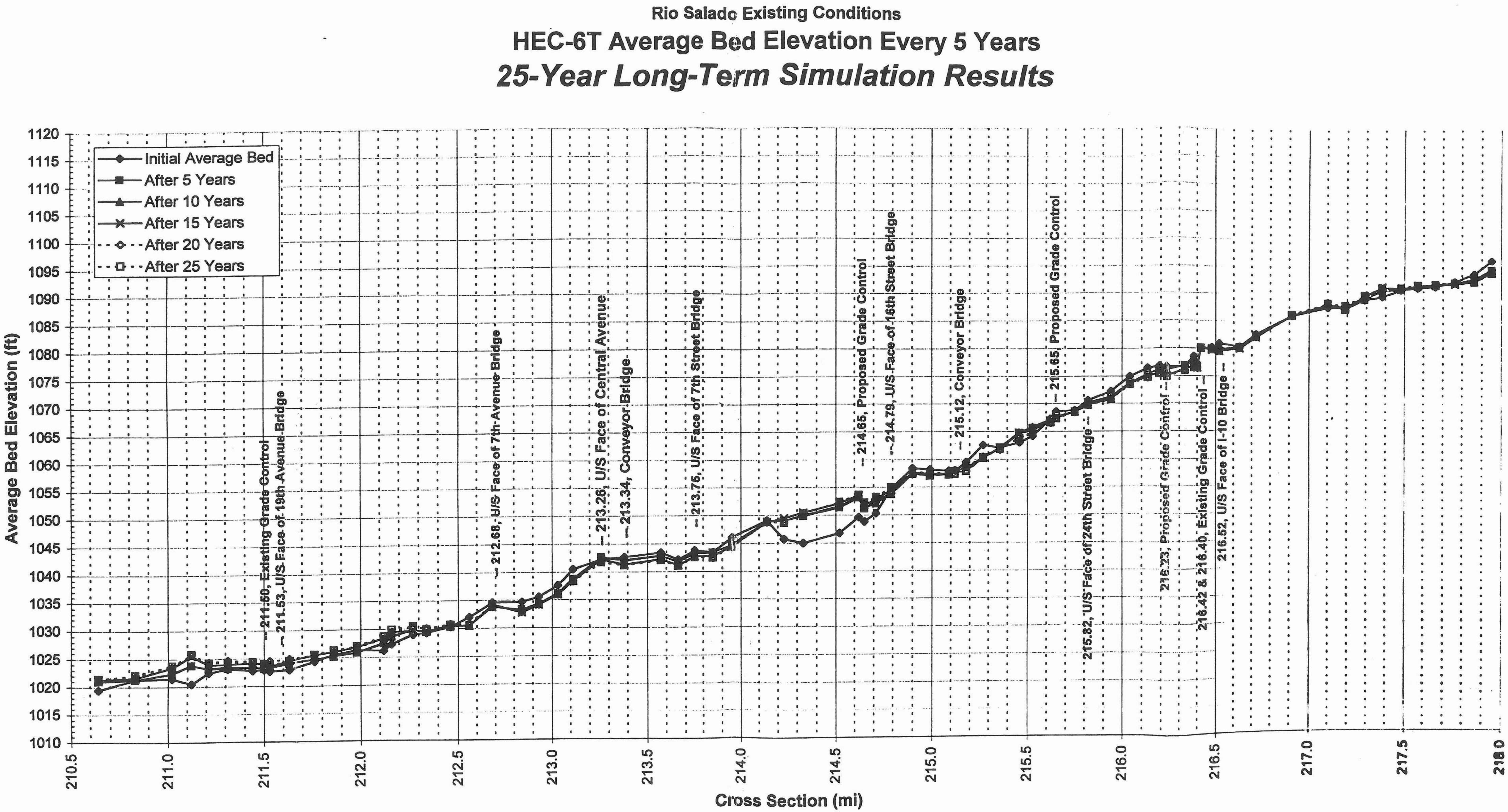


Figure 4-1. Average bed elevation at 5-year intervals for the existing conditions 25-year long-term simulation.

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).

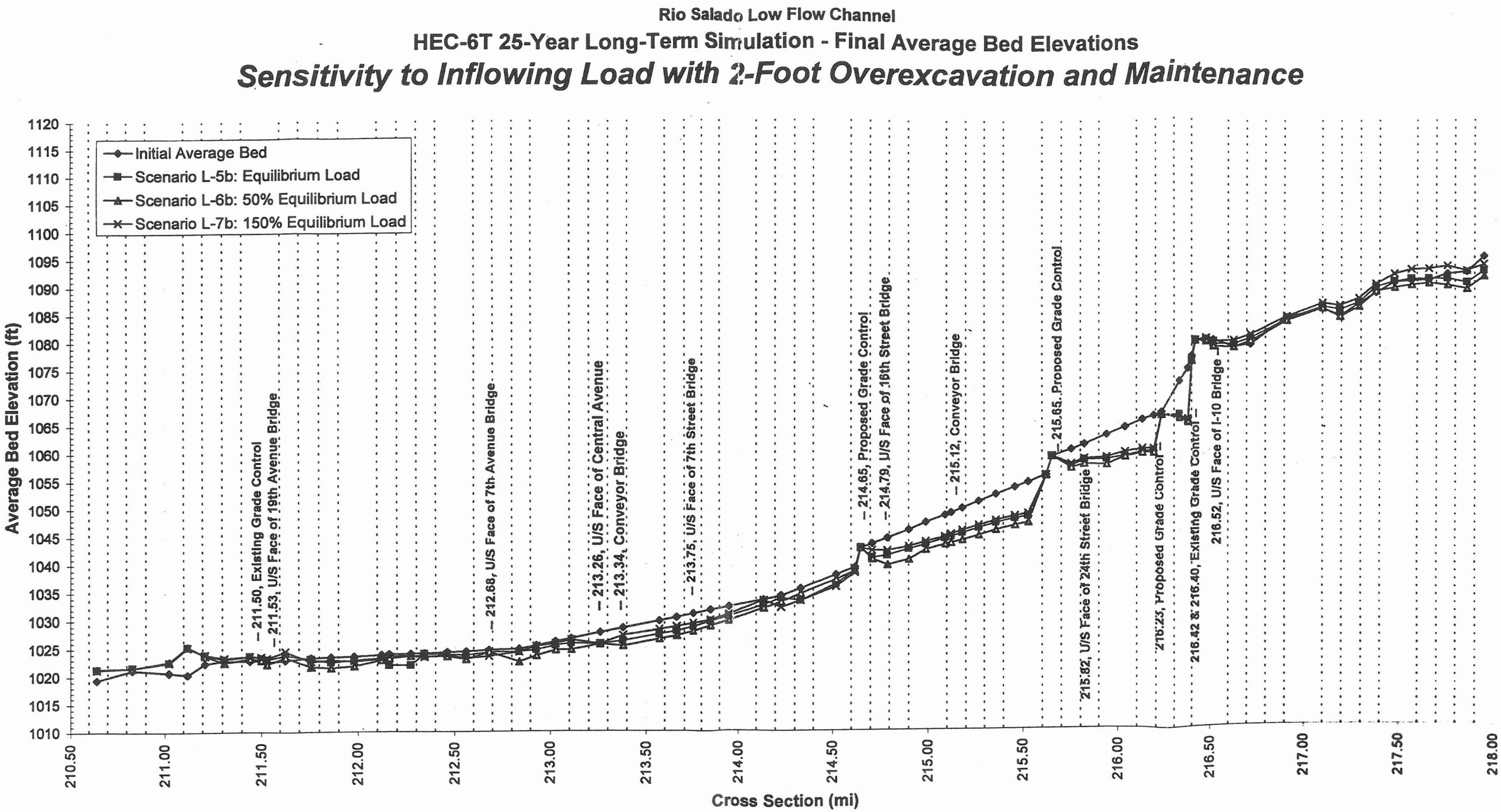


Figure 4-4. Sensitivity of the sediment transport results to the inflowing sediment load for the low flow channel with the four proposed grade control structures and 2-foot overexcavation and maintenance. A comparison of the average bed elevations at the end of the 25-year simulation period for 50%,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

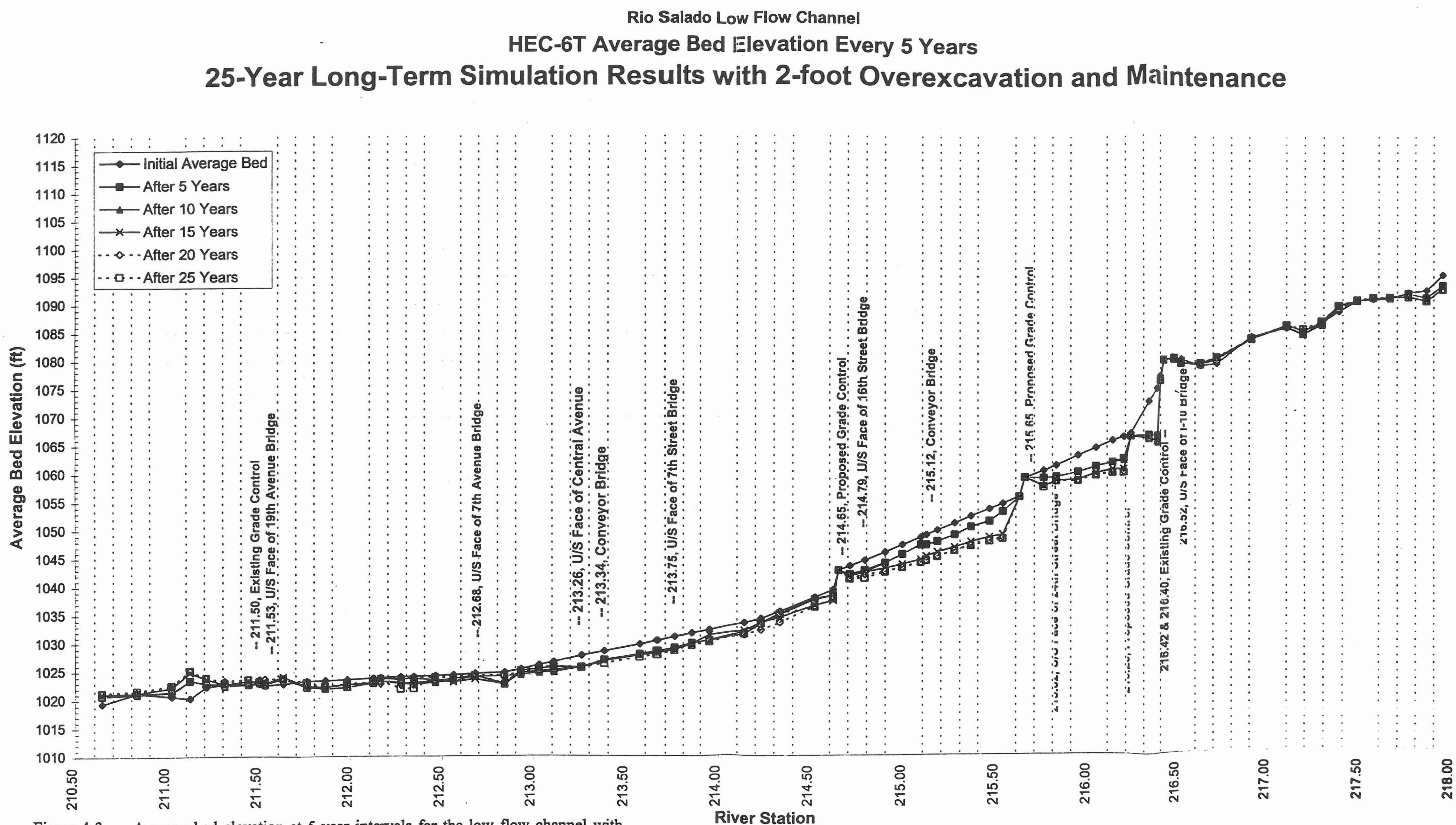


Figure 4-3. Average bed elevation at 5-year intervals for the low flow channel with the equilibrium inflowing sediment load, the four proposed grade control structures, and 2-foot overexcavation and annual maintenance.

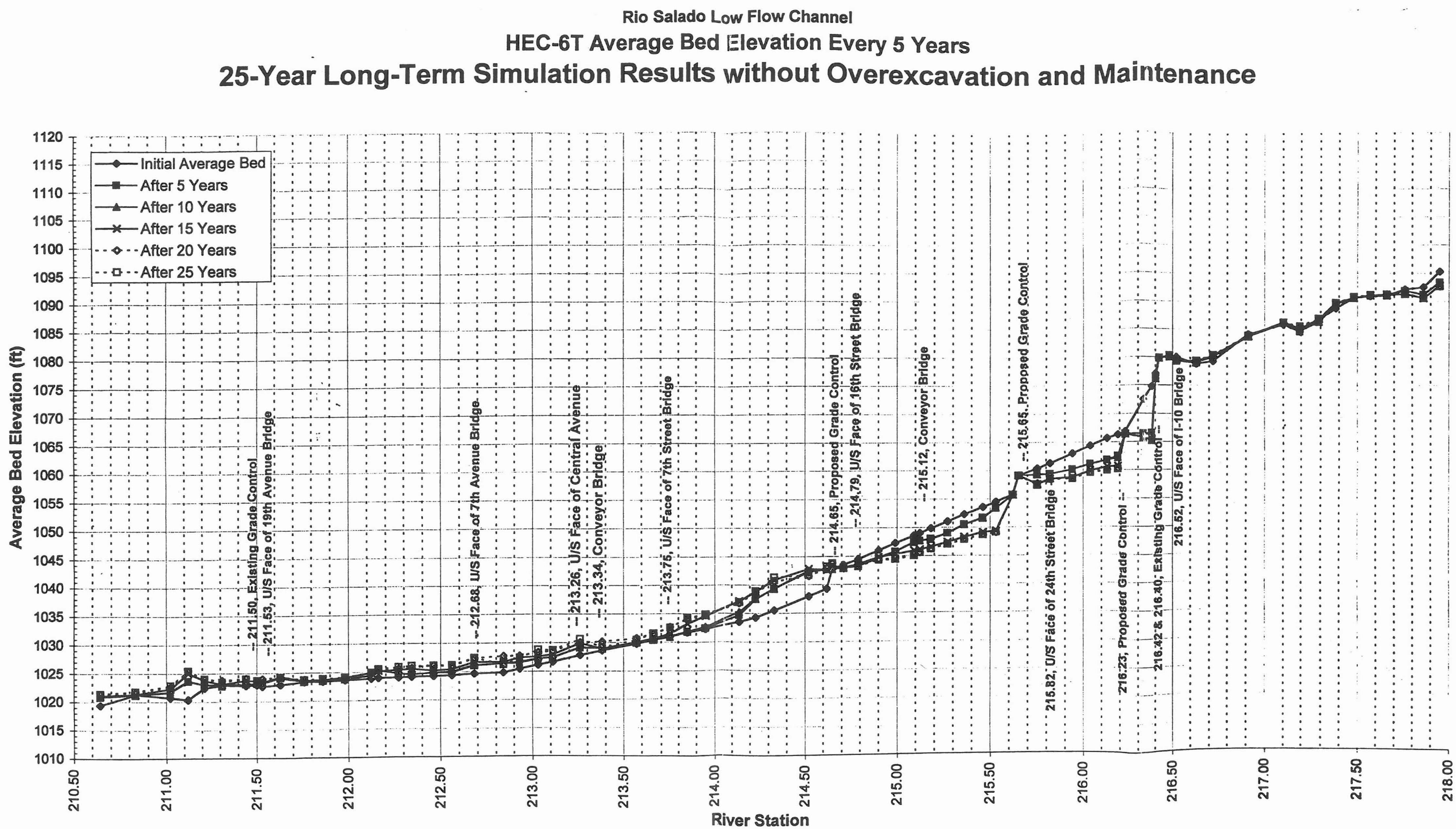


Figure 4-2. Average bed elevation at 5-year intervals for the low flow channel with the equilibrium inflowing sediment load and the four proposed grade control structures. Overexcavation and annual maintenance is not modeled.

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

TABLE OF CONTENTS

SECTION B
MATERIALS

FIGURES

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

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

E. Small amounts of <u>residential</u> 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.

8.0 References

ABM, 1957, Geologic Map of Maricopa County, Arizona.

AGS MAP GI-2-A-B-C, Geology of the Tempe Quadrangle, by C.S. Wellendorf and others, 1986.

Brown, J.G. and Pool, D.R., USGS WRIR 88-4202, Hydrogeology of the Western Part of the Salt River Valley Area, Maricopa County, Arizona, 1989.

City of Phoenix, Public Works Department, Solid Waste Disposal Management Division, Del Rio Landfill Phase II Remedial Action Plan, Ground Water Monitoring Report, April 1993.

Corkhill, E.F. and others, Arizona Department of Water Resources, A Regional Ground Water Flow Model of the Salt River Valley-Phase I, Phoenix Active Management Area, Hydrogeologic Framework and Basic Data Report, April 1993.

Dames & Moore, 1990, Remedial Action Plan for Del Rio Landfill, prepared for the City of Phoenix, AZ, April 1991

Dames & Moore, 1989, Remedial Action Plan for 19th Avenue Landfill, prepared for the City of Phoenix, AZ, June 1989.

Kokalis, P.G., Terraces of the Lower Salt River Valley, Arizona, September 1971, Masters Thesis.

Lee, W. Thomas USGS WSIP 136, Underground Waters of Salt River Valley, Arizona, 1905.

Reynolds, S.J., AGS, Geology of the South Mountains, Central Arizona, 1985.

Schumann, H.H., USGS MAP I-845-H, Land Subsidence and Earth Fissures in Alluvial Deposits in the Phoenix Area, Arizona, 1974.

Unified Soil Classification System, USCS, according to ASTM D24-87, 1987.

U.S. EPA, 1993, Record of Decision for South Indian Bend Wash Superfund Site, USEPA Region IX, September 1993.

U.S. Army Corps of Engineers, Rio Salado, Salt River, Arizona, Reconnaisance Report, March 1995.

U.S. EPA, 1990, Record of Decision for North Indian Bend Wash Superfund Site, USEPA Region IX, 1990.

Wellendorf and others, C.S., AGS MAP GI-2-F, Ground Water Map of the Tempe Quadrangle, 1986.

Wilson, R.P., USGS IR 90-4179, Summary of Ground-Water Conditions in Arizona 1985-86, 1991.

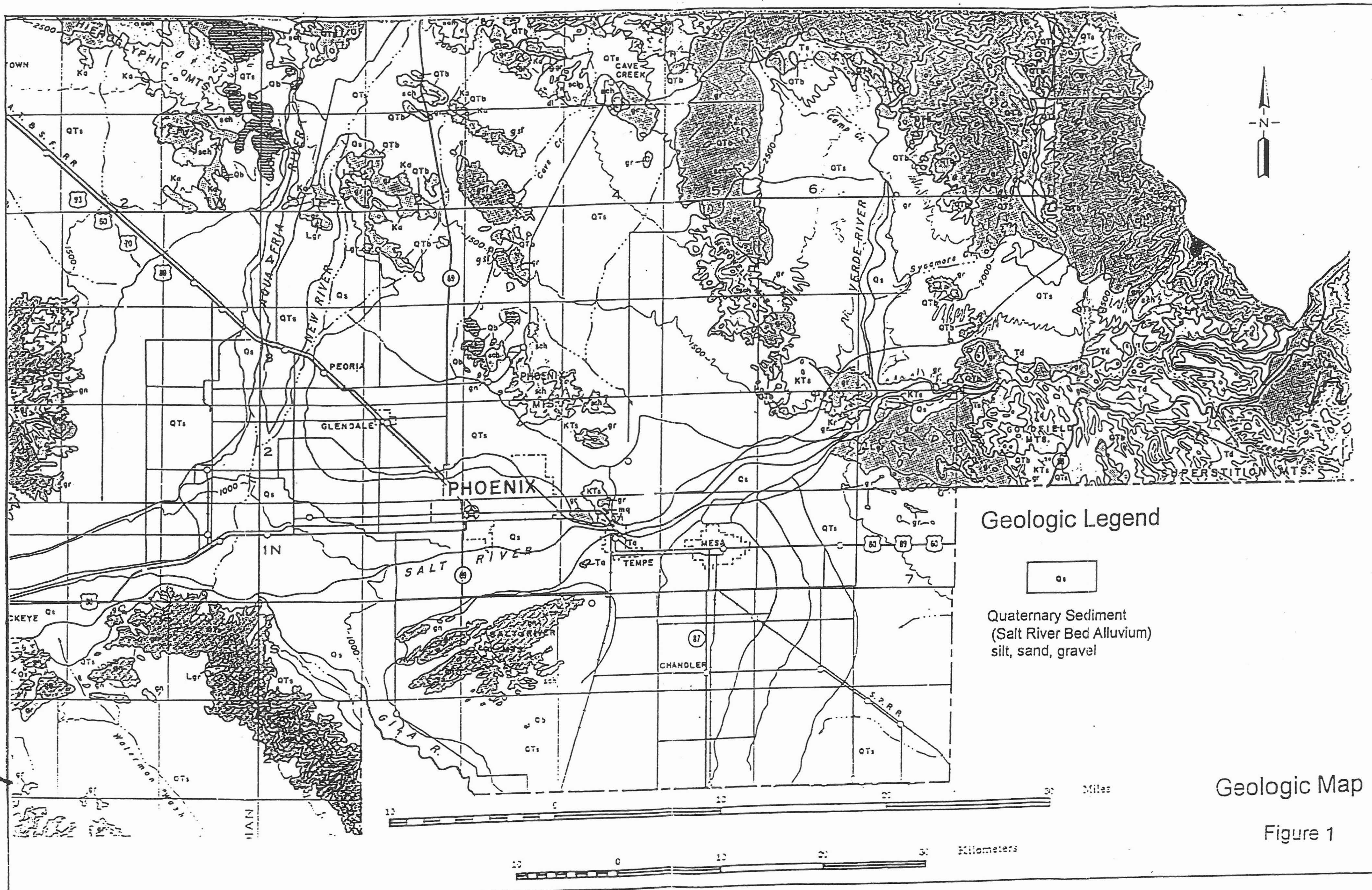
Wolcott, H.N. USGS OFR 1952, Ground Water in the Gila River Basin and Adjacent Areas, Arizona-A Summary, 1952.

U.S. Army Corps of Engineers, Los Angeles District, Interim Report on Survey for Flood Control, Gila and Salt Rivers, Gillespie Dam to McDowell Dam Site, Arizona, 1957.

International Conference of Building Officials, Uniform Building Code, Chapter 16, 1997.

Sharma, Hari D., Waste Containment Systems, Waste Stabilization and Landfills, 1994.

U.S. Army Corps of Engineers, Engineering Regulation ER 1110-2-1806, 31 July 1995.



Geologic Legend

Qs

Quaternary Sediment
(Salt River Bed Alluvium)
silt, sand, gravel

Geologic Map

Figure 1

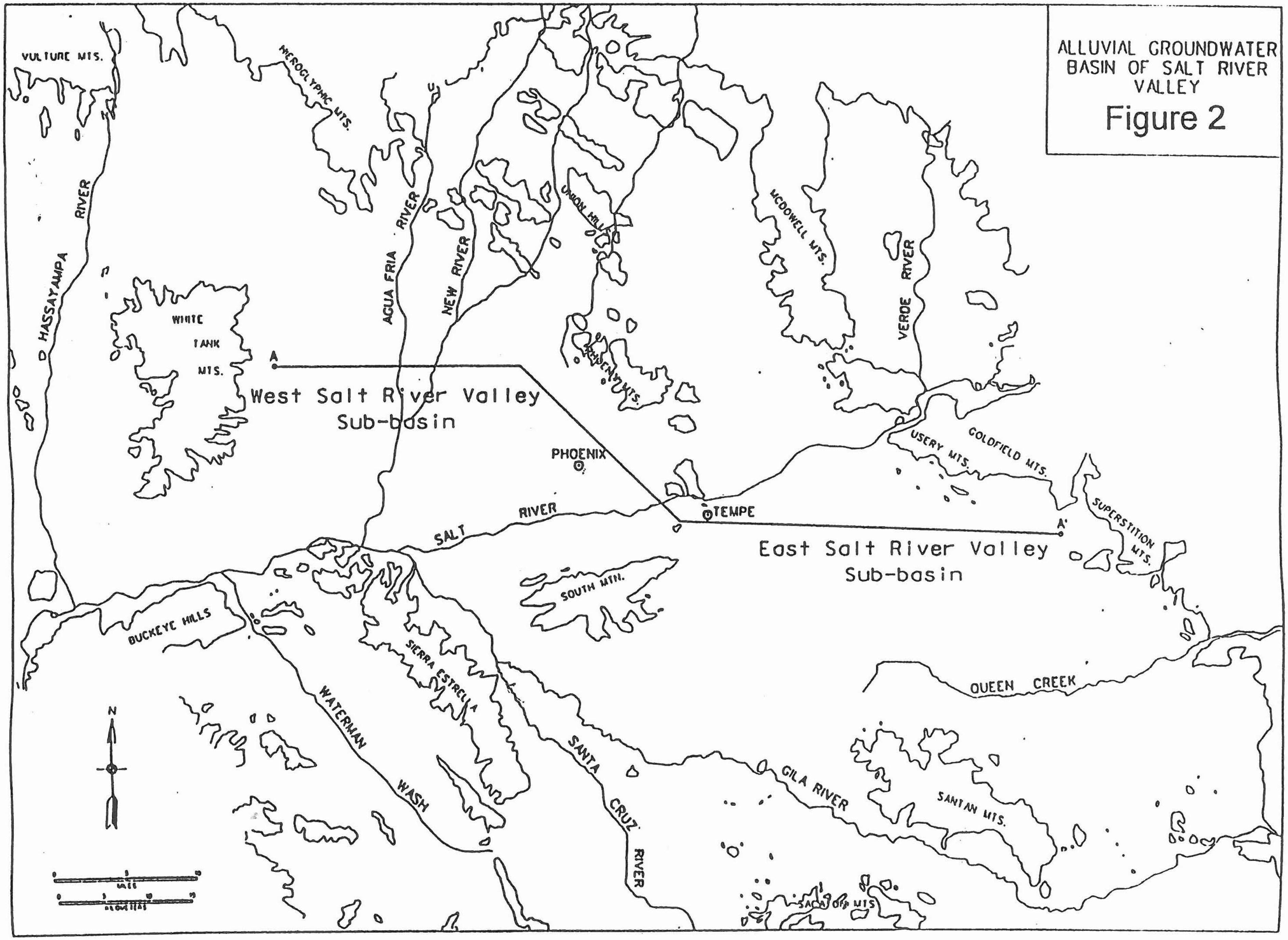
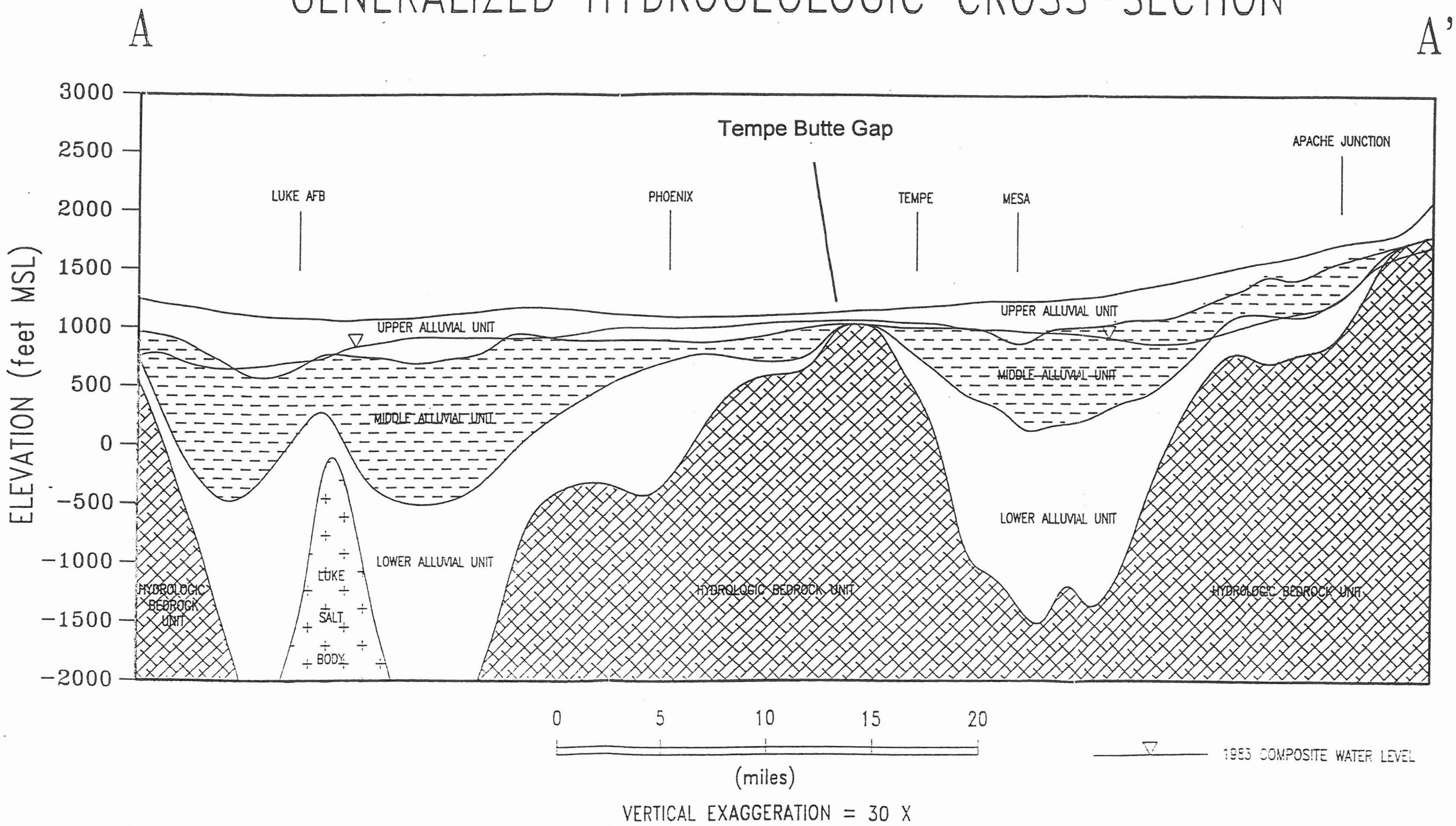

ALLUVIAL GROUNDWATER
BASIN OF SALT RIVER
VALLEY
Figure 2
VULTURE MTS.
HIEROGLYPHIC MTS.
HASSAYAMPA RIVER
AGUA FRIA RIVER
NEW RIVER
UNION HILLS
McDOWELL MTS.
VERDE RIVER
WHITE TANK MTS.
PHOENIX MTS.
West Salt River Valley
Sub-basin
PHOENIX
USERY MTS.
GOLDFIELD MTS.
SUPERSTITION MTS.
A
A'
SALT RIVER
TEMPE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

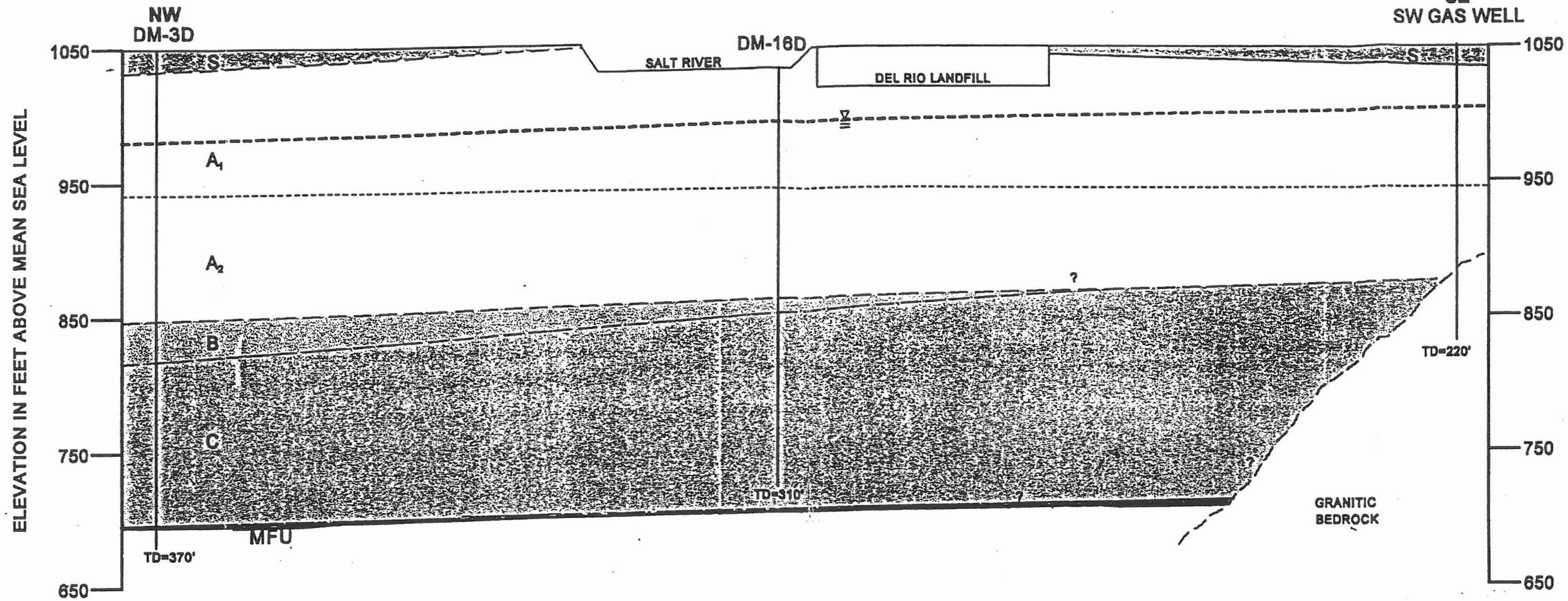
Figure 3

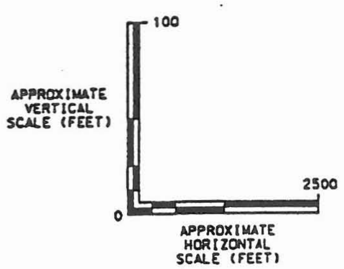
Rio Salado Habitat Project, Phoenix Portion

GENERALIZED HYDROGEOLOGIC CROSS-SECTION



CROSS SECTION





EXPLANATION

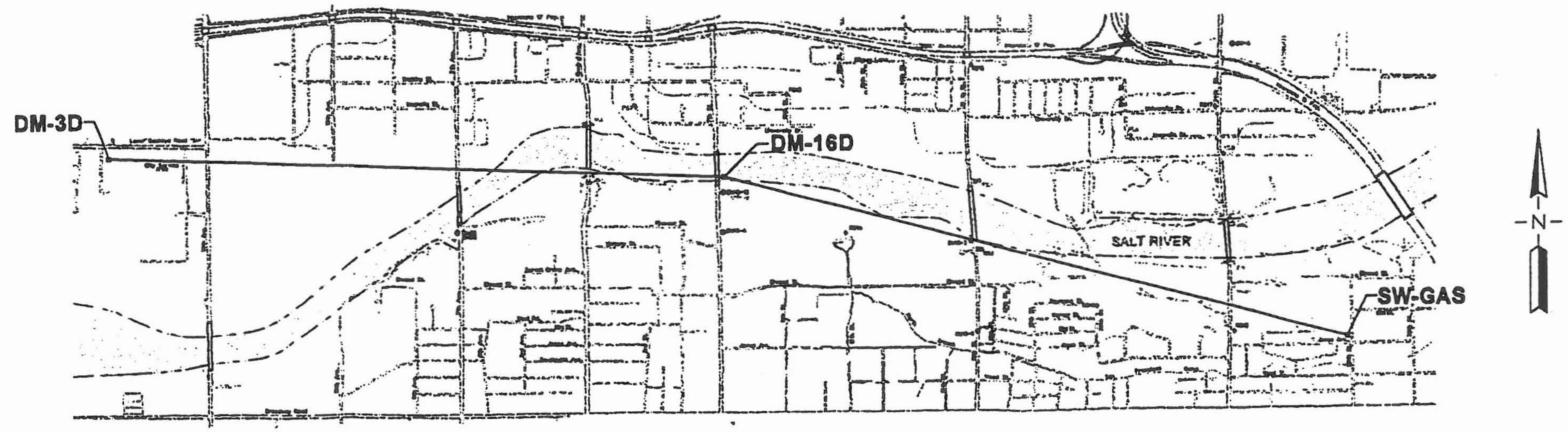
INFERRED UNIT CONTACT

INFERRED SUBUNIT CONTACT (TRANSITIONAL)

APPROXIMATE WATER TABLE
DAMES AND MOORE (1991)

	Geologic Subunit	DESCRIPTION
	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 A1: GRAVEL, 80-90% FINE TO COARSE GRAVEL AND COBBLES, 20-10% MEDIUM TO COARSE SAND, LITTLE TO NO FINES SUBUNIT A2: 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
	B	SILTY SAND WITH GRAVEL, REDDISH BROWN, 40-50% FINE TO MEDIUM SAND, 40-50% SILT WITH CLAY, 10-15% FINE GRAVEL
	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

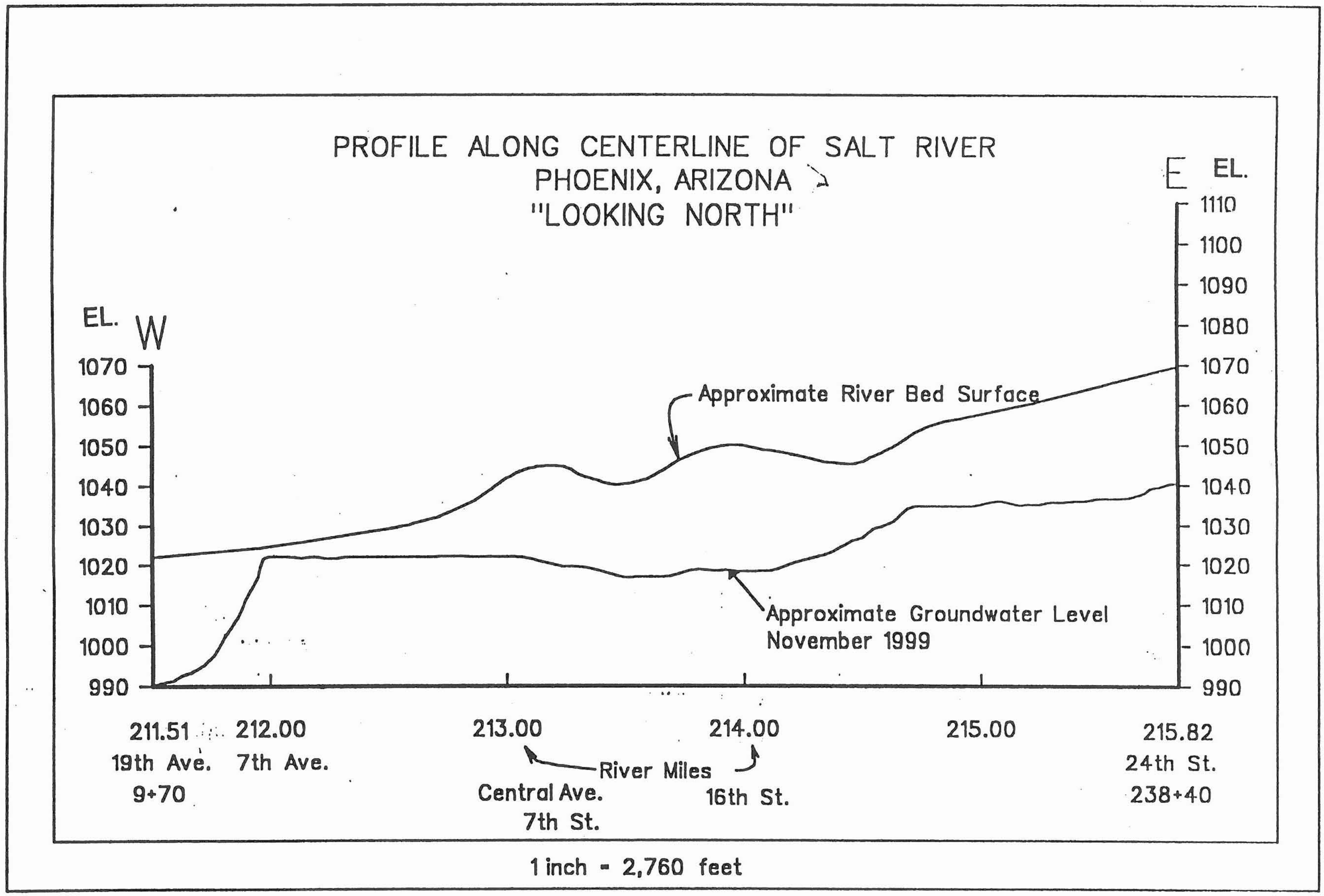
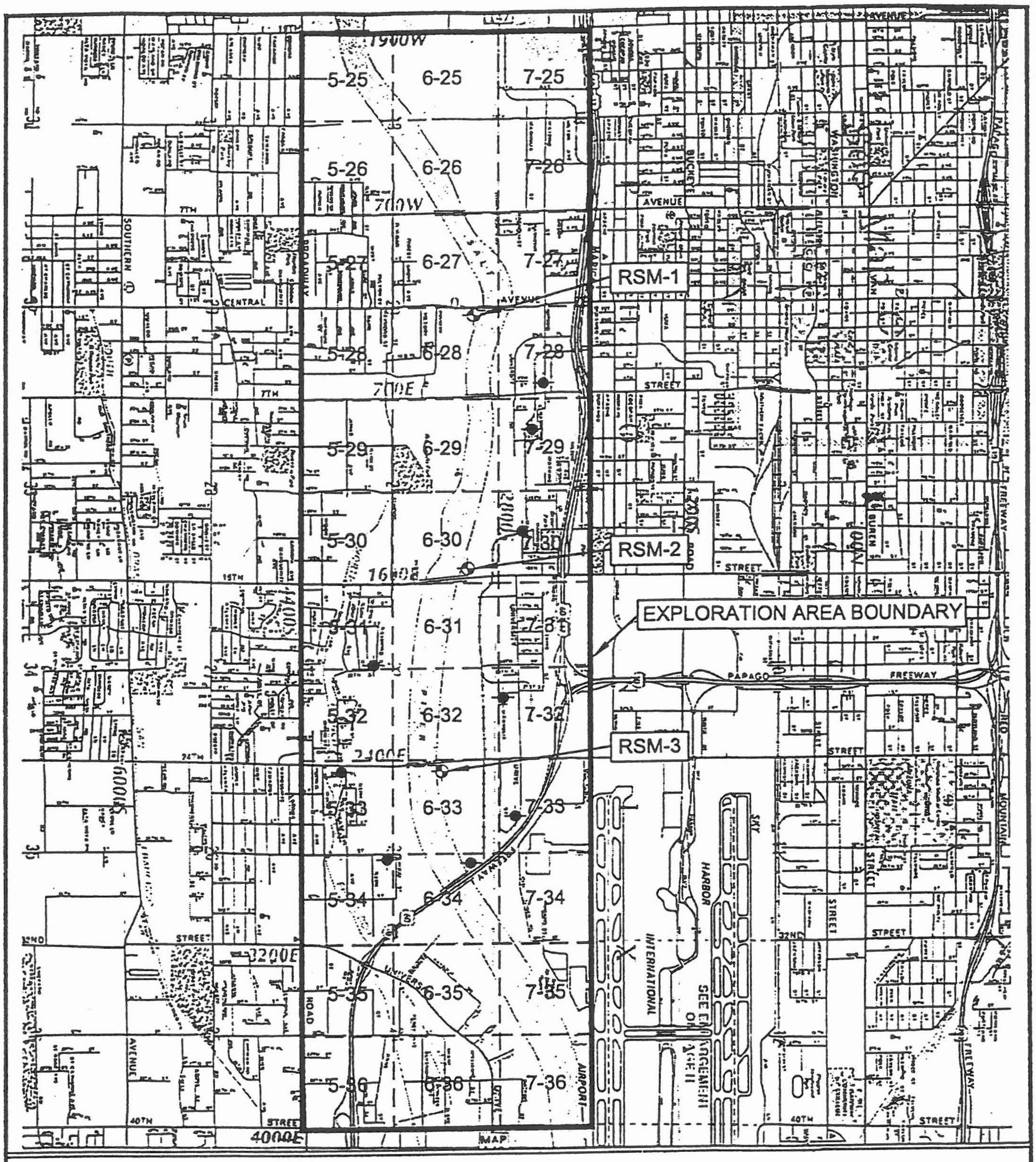

PROFILE ALONG CENTERLINE OF SALT RIVER
PHOENIX, ARIZONA
"LOOKING NORTH"
EL.
W
E
EL.
1110
1100
1090
1080
1070
1060
1050
1040
1030
1020
1010
1000
990
Approximate River Bed Surface
Approximate Groundwater Level
November 1999
211.51
19th Ave.
9+70
212.00
7th Ave.
213.00
Central Ave.
7th St.
River Miles
214.00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

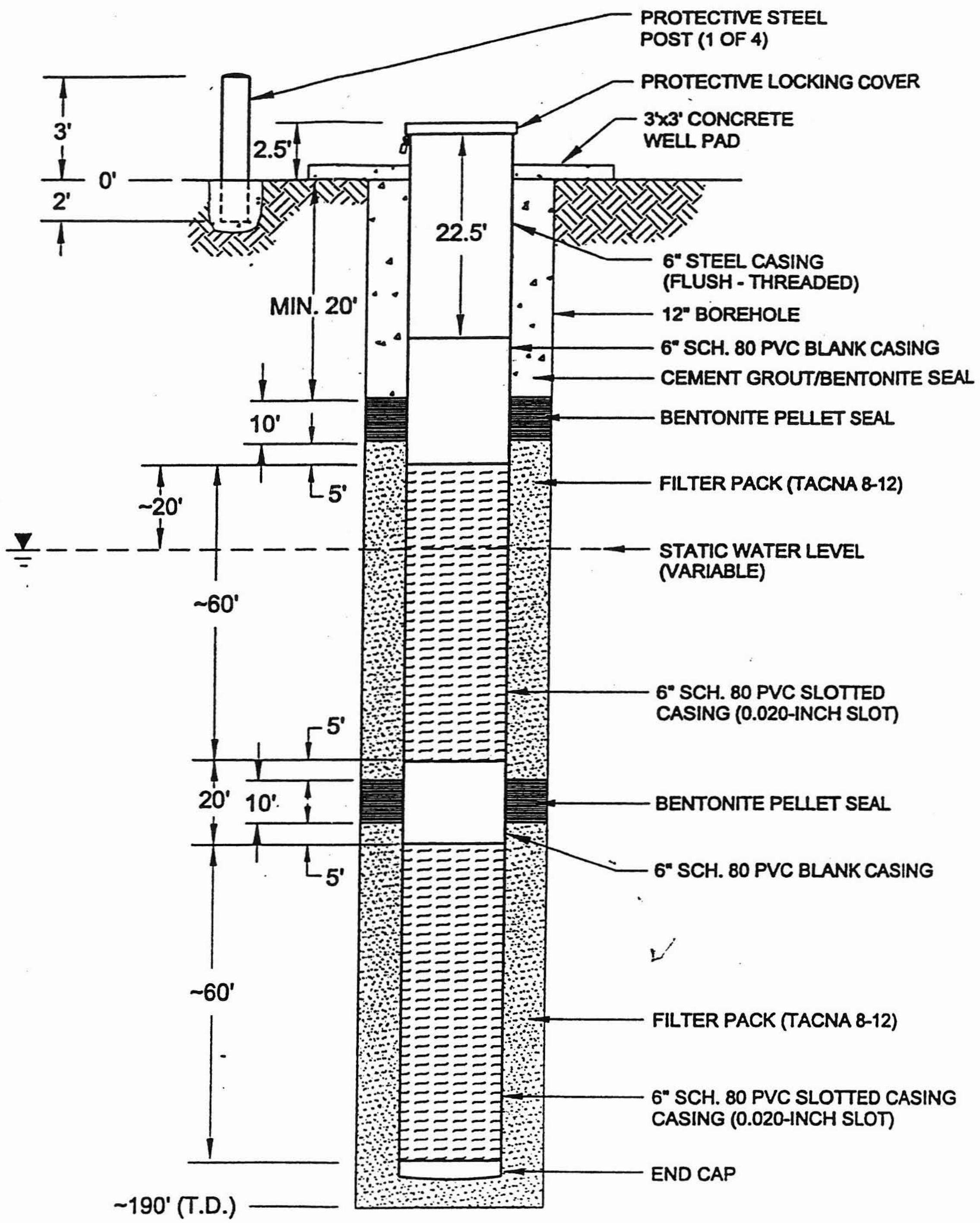
EXPLANATION:

- Currently Proposed Monitor Well Location
- Future Monitor Well Location

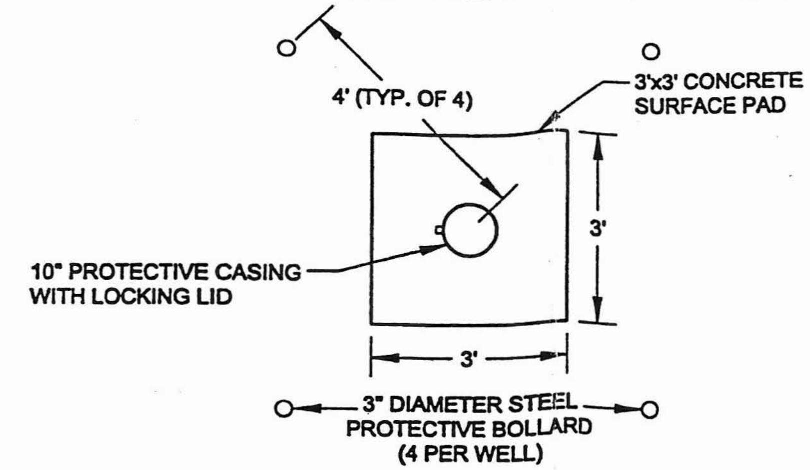
0 1/2 1

Scale in Miles

Figure 6

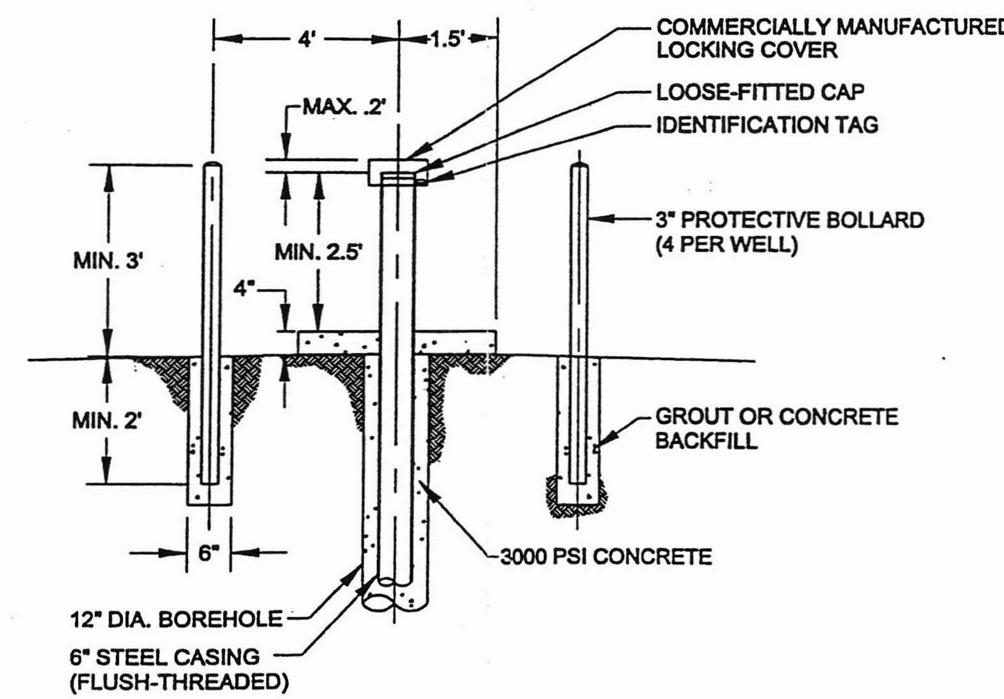


NOT TO SCALE



PLAN VIEW

NOT TO SCALE



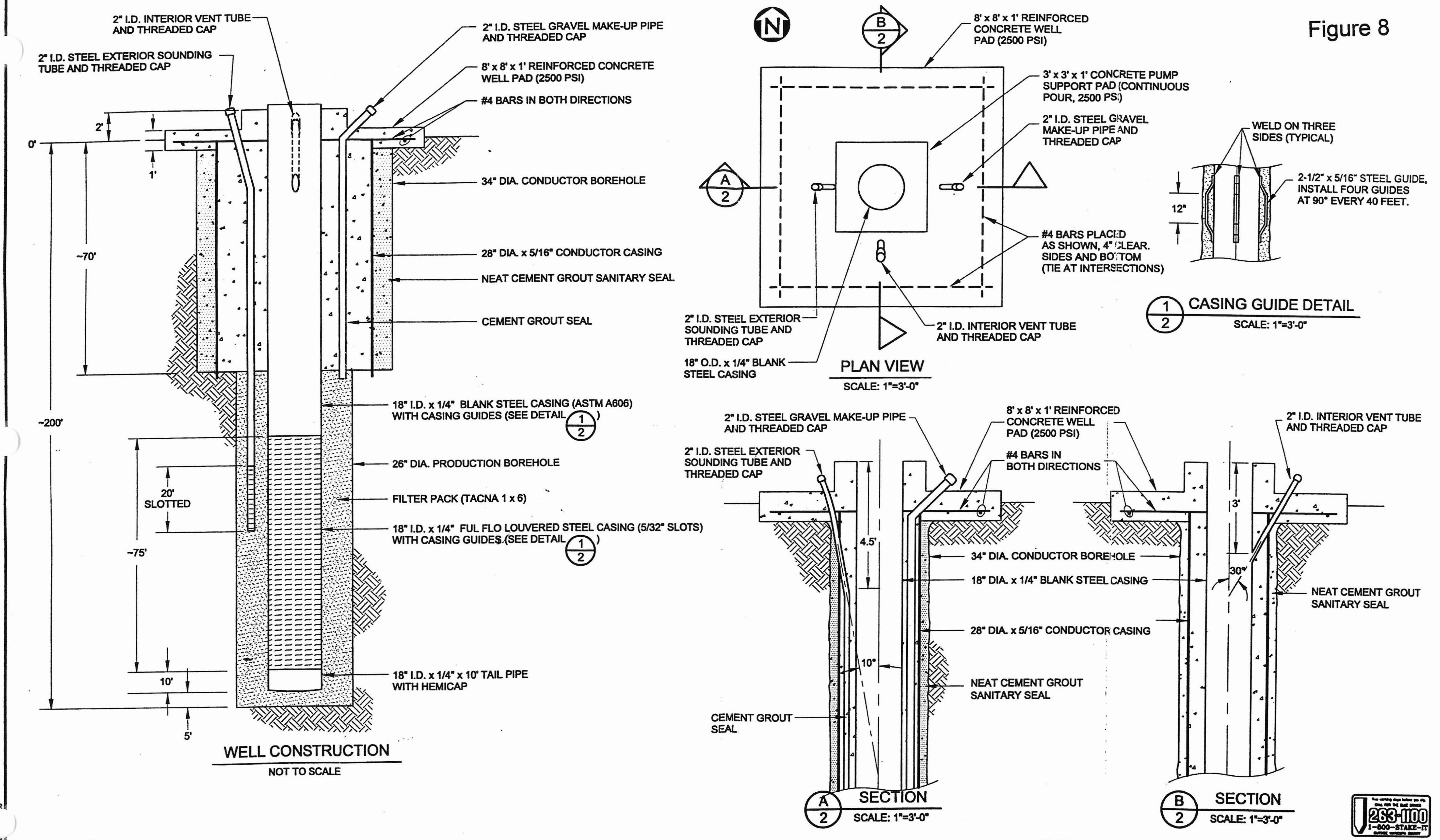
SURFACE COMPLETION DETAILS FOR MONITORING WELLS

NOT TO SCALE

Monitor Well Construction Diagram
and Surface Completion Detail

Figure 7

Figure 8



REFERENCES TITLE	REFERENCES TITLE	REVISIONS NO.	BY	DATE	DESCRIPTION
		△	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

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	
6.0	SP	100	94	91	88	84	81	75	53	23	7	3		NP	18.9	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	8	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 material is 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.

SCALE 5 0 5 10 15 FEET

Figure 9

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

DESIGNED BY: W H
DRAWN BY: W H
CHECKED BY: JDO

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

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

SAFETY PAYS

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, grey., 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, grey, 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, grey/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	26	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	26	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	56	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	39	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, aproximately 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	58	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, very loose, percentage of cobbles increases with depth.
4.0		100	85	70	61	54	49	41	26	14	7	2		NP	15.9	
5.0		100	89	84	81	79	58	29	13	6	2	1		NP	33.3	
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

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

LOGS OF EXPLORATION
TT99-9 TO TT99-16

SYMBOL	DESCRIPTIONS	DATE	APPROVED

REVISIONS

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

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE
CADD FILE NAME: g62.dgn

SHEET 16 OF 19 SHEETS

FIGURE 11

SAFETY PAYS

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	GP	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	GP	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	GP	100	69	48	35	27	23	21	16	9	3	1		NP	7.3	same as above, less than 5% boulders.
10.0	GP	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	GP	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, debri 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	ML	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	SM	100	100	99	98	98	97	97	95	93	81	46			23.2	SILTY SAND, brown.
16.0	SM	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	19	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	GP	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	GP	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	GW	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	GP	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	18.0	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	18	13	11	10	7	3	1	1		NP	8.4	POORLY GRADED GRAVEL, brown, approximately 20% cobbles and 5% boulders, some trash.

Figure 11

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

SYMBOL | DESCRIPTIONS | DATE | APPROVAL
REVISIONS

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

CADD FILE NAME: g03.dgn

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

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

DISTRICT FILE NO. NOT APPLICABLE | SPEC. NO. NOT APPLICABLE

SHEET 17 OF 19 SHEETS

FIGURE 12

SAFETY PAYS

TT99-29

DEPTH (ft)	SOIL CLASS	3	1.5	¾	⅜	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	GP	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	¾	⅜	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	GP	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	¾	⅜	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	GP	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	GP	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	¾	⅜	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	GP	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	GP	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	¾	⅜	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

SAFETY PAYS

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

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

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

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

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

CADD FILE NAME: g05.dgn

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

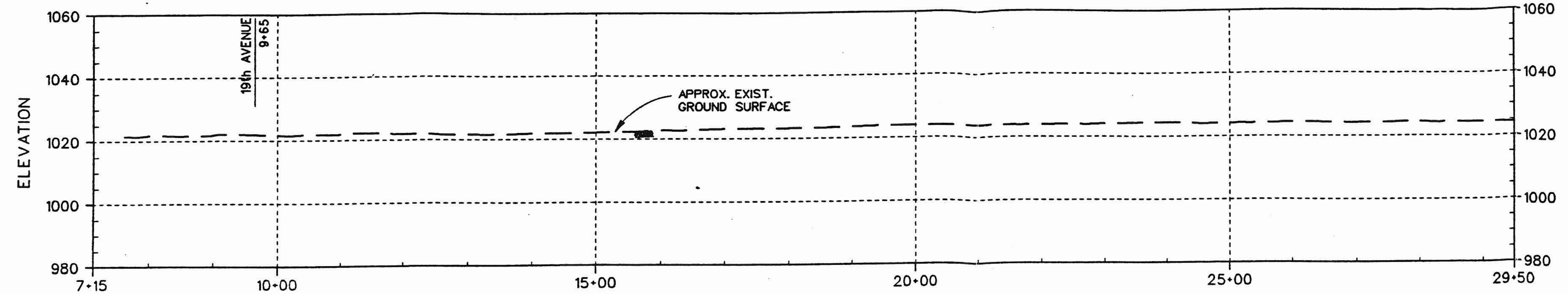
SUBMITTED BY:
ABBAS T. ROODSARI, P.E.
CHIEF, GEOTECHNICAL BRANCH

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

SHEET 19 OF 19 SHEETS

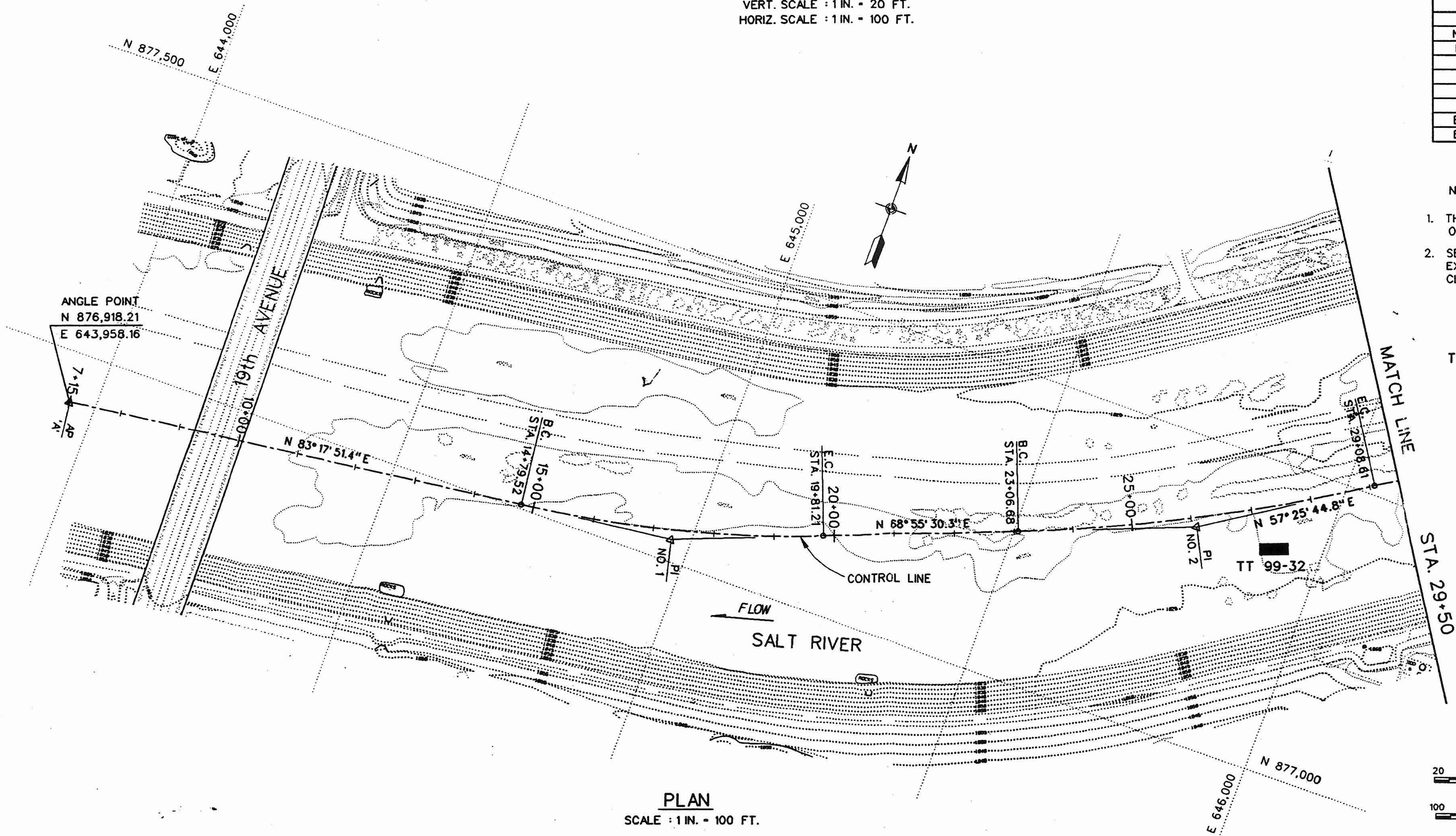
FIGURE 14

Figure 14



PROFILE AT CONTROL LINE

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



PLAN

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

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

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

CADD FILE NAME: CL.dgn

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

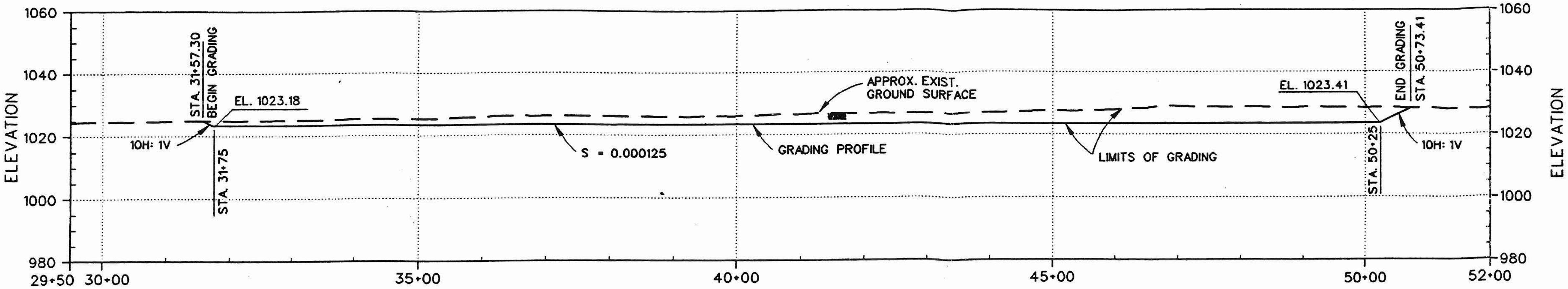
SUBMITTED BY:
THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

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

SHEET 3 OF 19 SHEETS

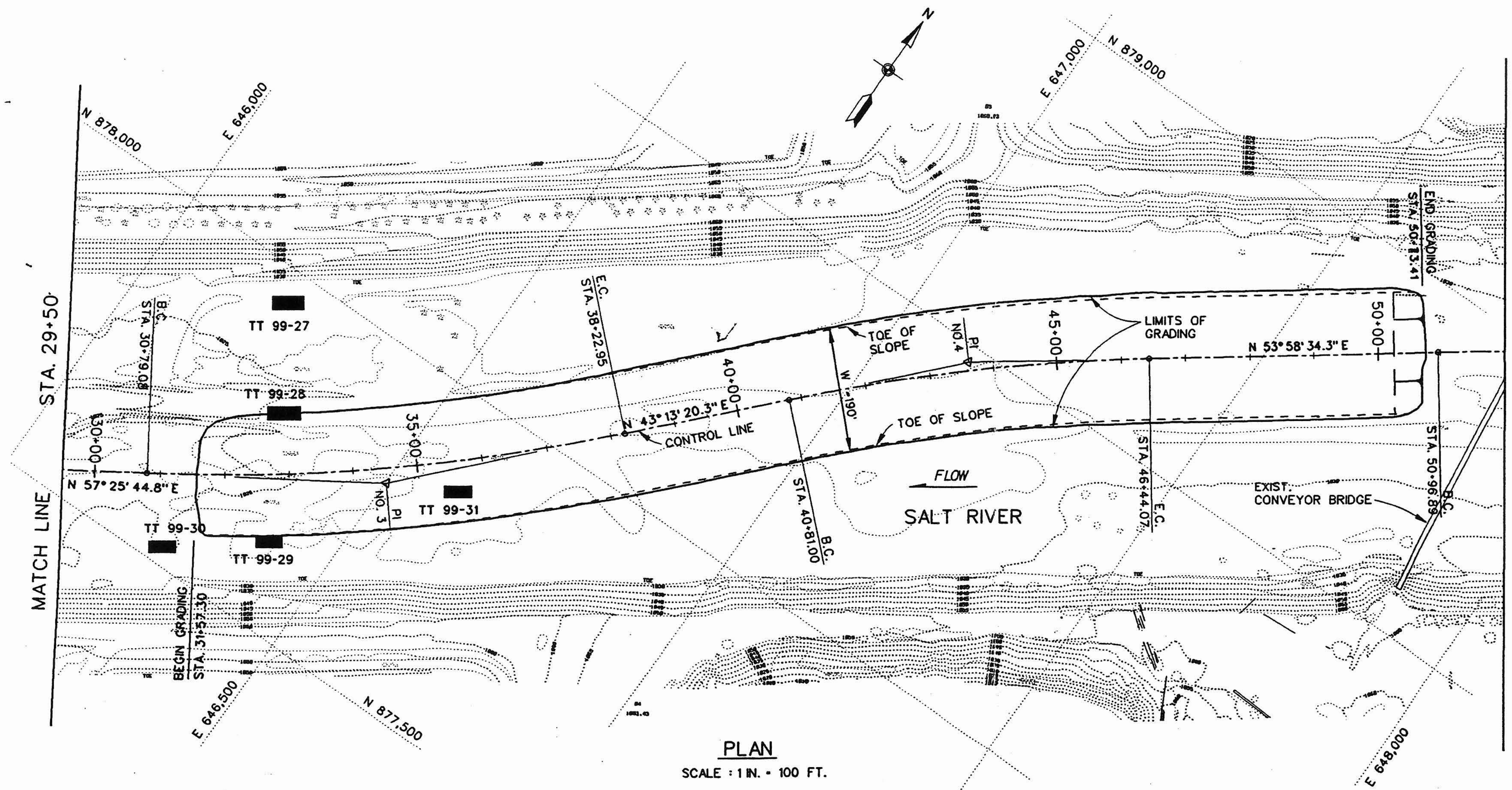
SAFETY PAYS

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 | APPROV.
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

DESIGNED BY: CC / WD
DRAWN BY: WD
CHECKED BY: DC
CADD FILE NAME: C2.dgn

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

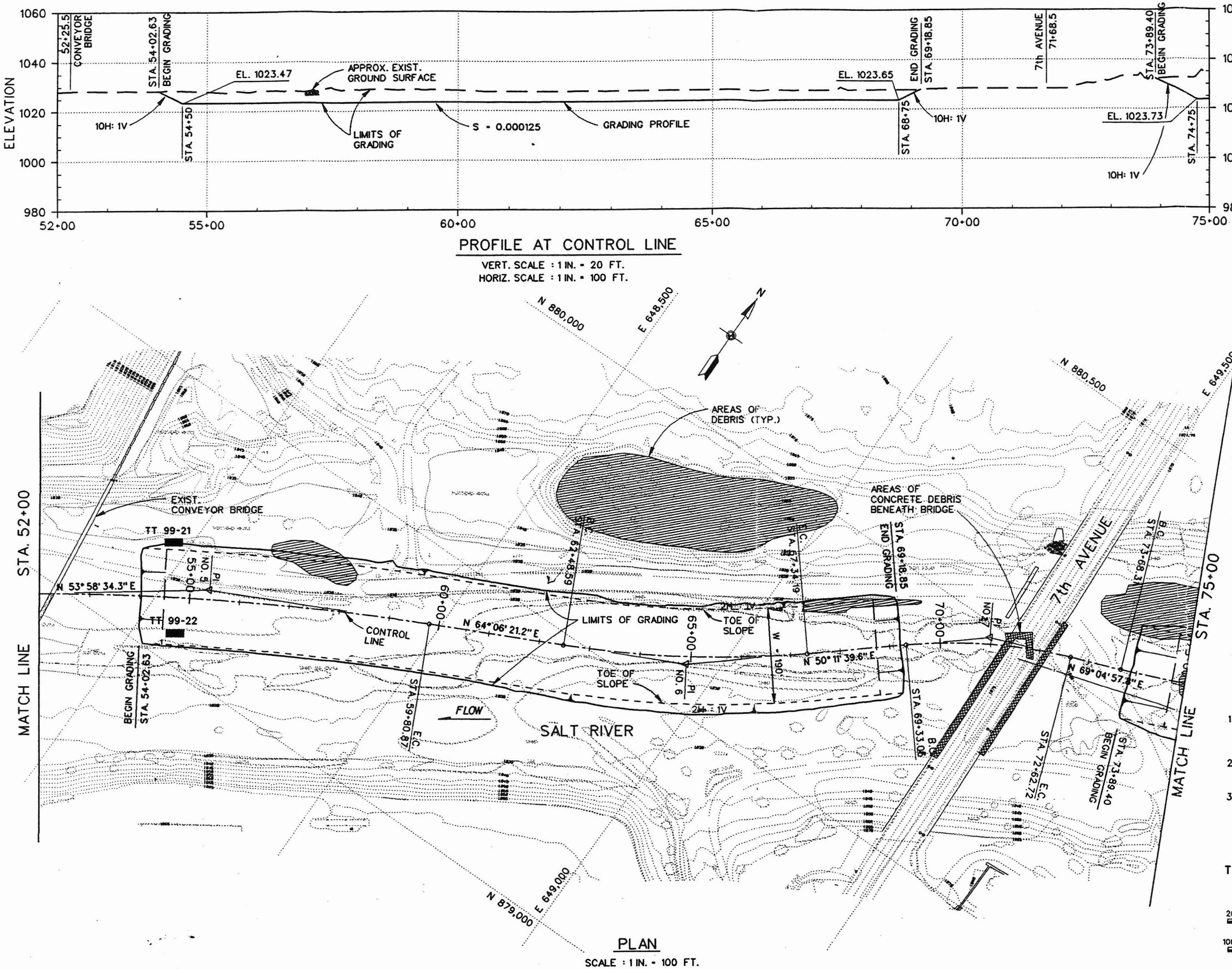
SUBMITTED BY:
THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

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

SHEET 4 OF 19 SHEETS

SAFETY PAYS

Figure 16



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

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

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

CADD FILE NAME: o3.dgn

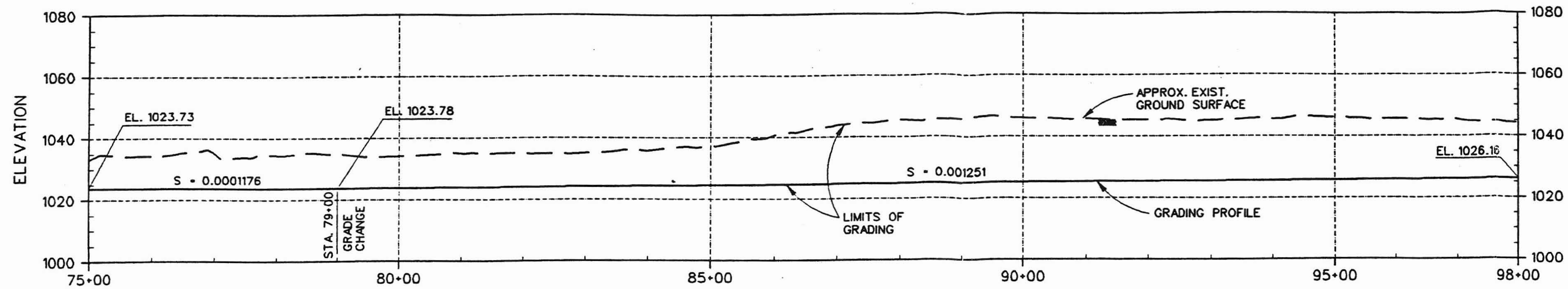
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

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

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

SHEET 5 OF 19 SHEETS

Figure 17

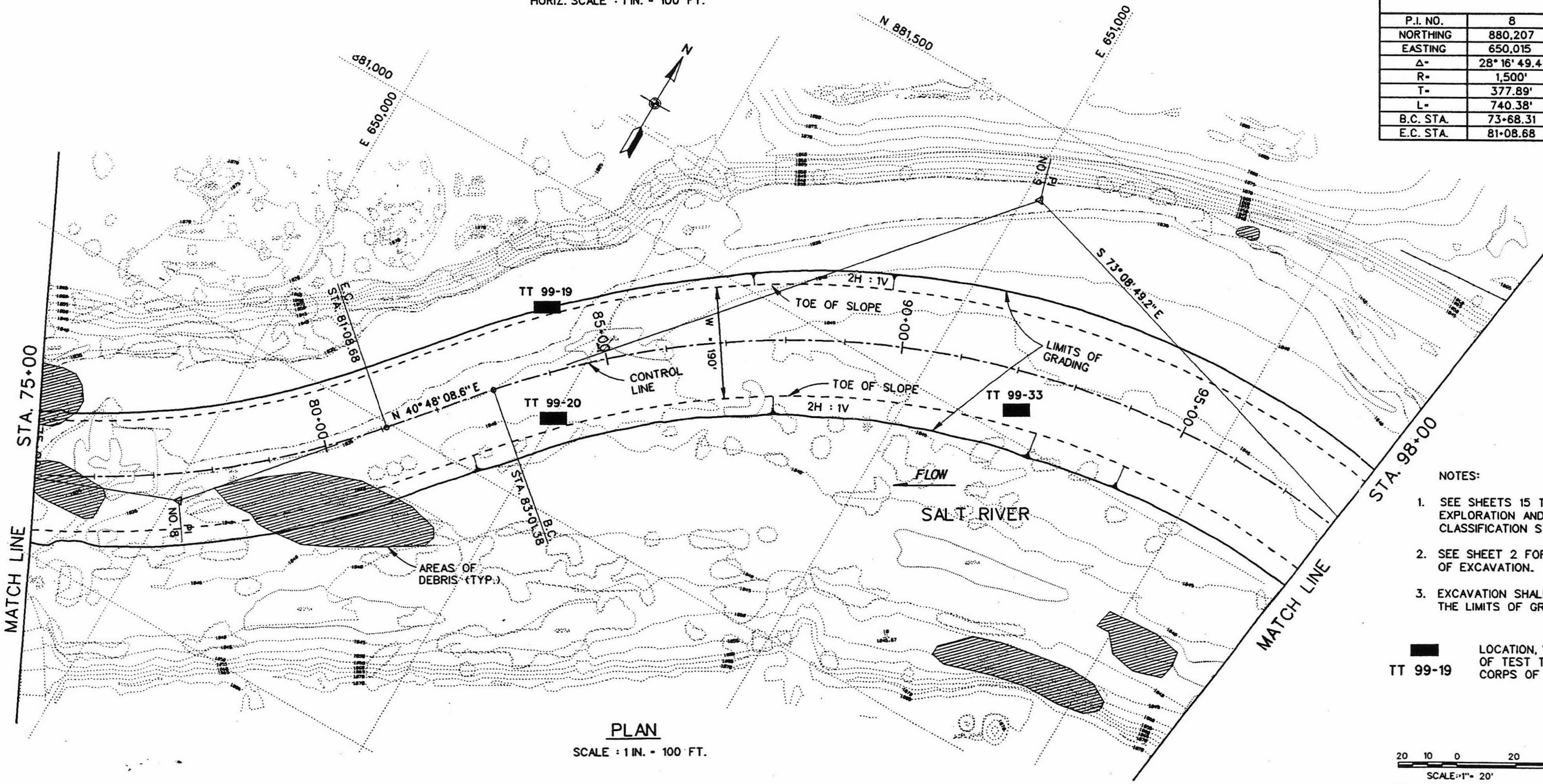


PROFILE AT CONTROL LINE

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

SCALE: 1" = 20'

SCALE: 1" = 100'

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

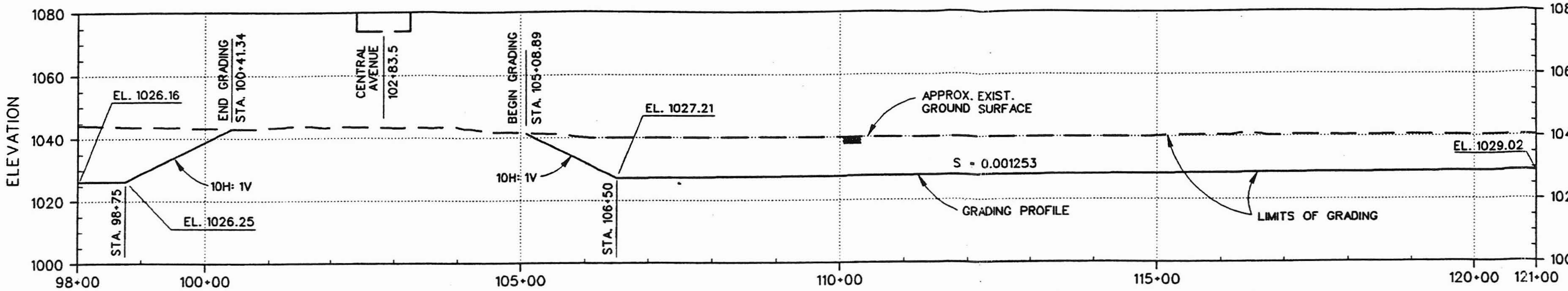
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

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

SHEET 6 OF 19 SHEETS

SAFETY PAYS

Figure 18

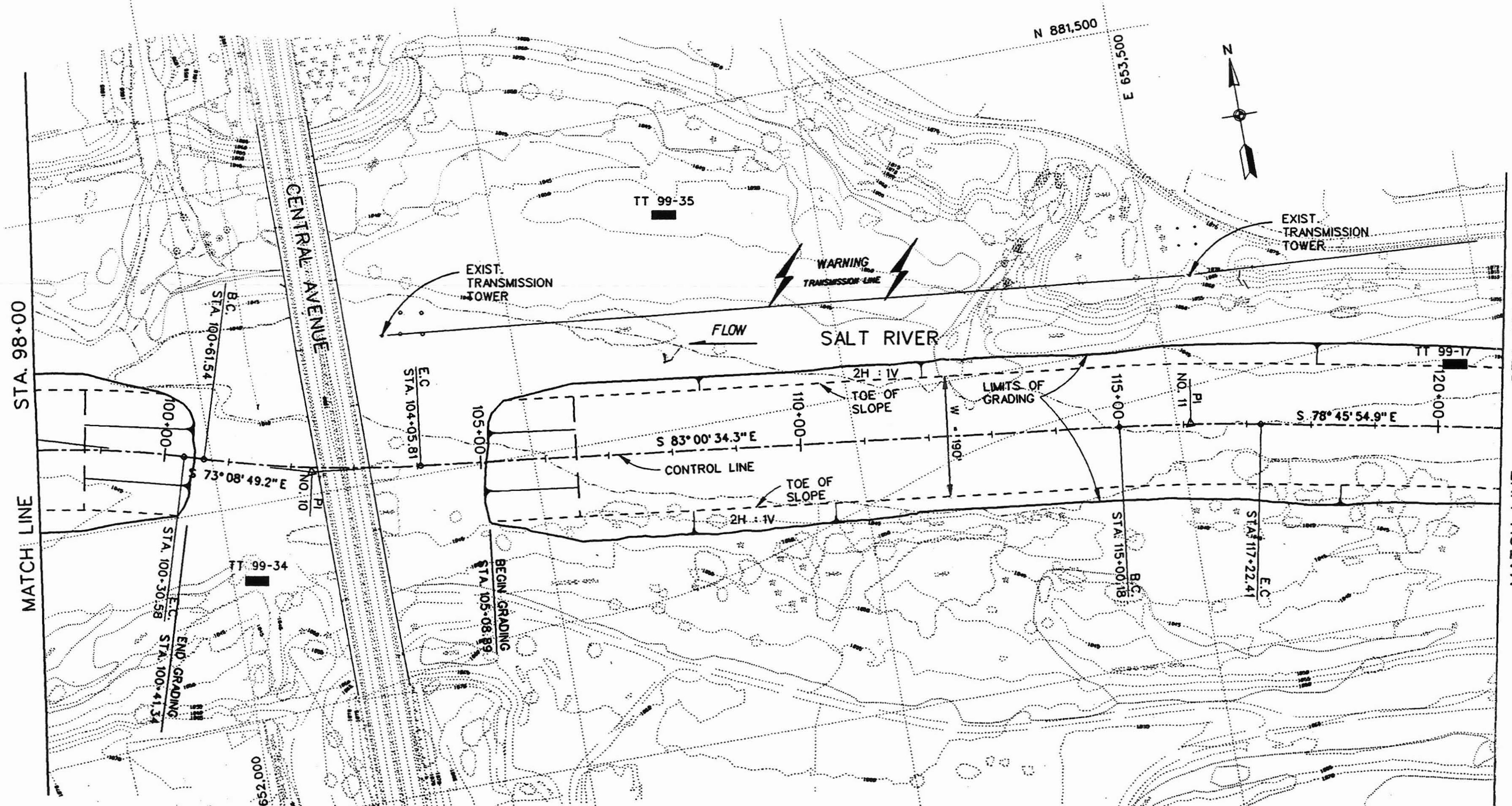


PROFILE AT CONTROL LINE

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

DESIGNED BY: CC / RD
DRAWN BY: RD
CHECKED BY: DC
CADD FILE NAME: C5.dgn

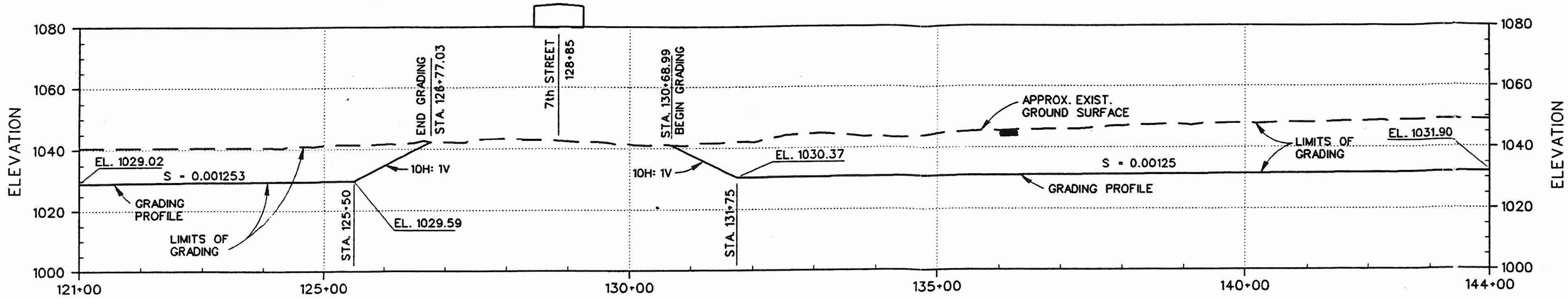
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

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

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

SHEET 7 OF 19 SHEETS

Figure 19

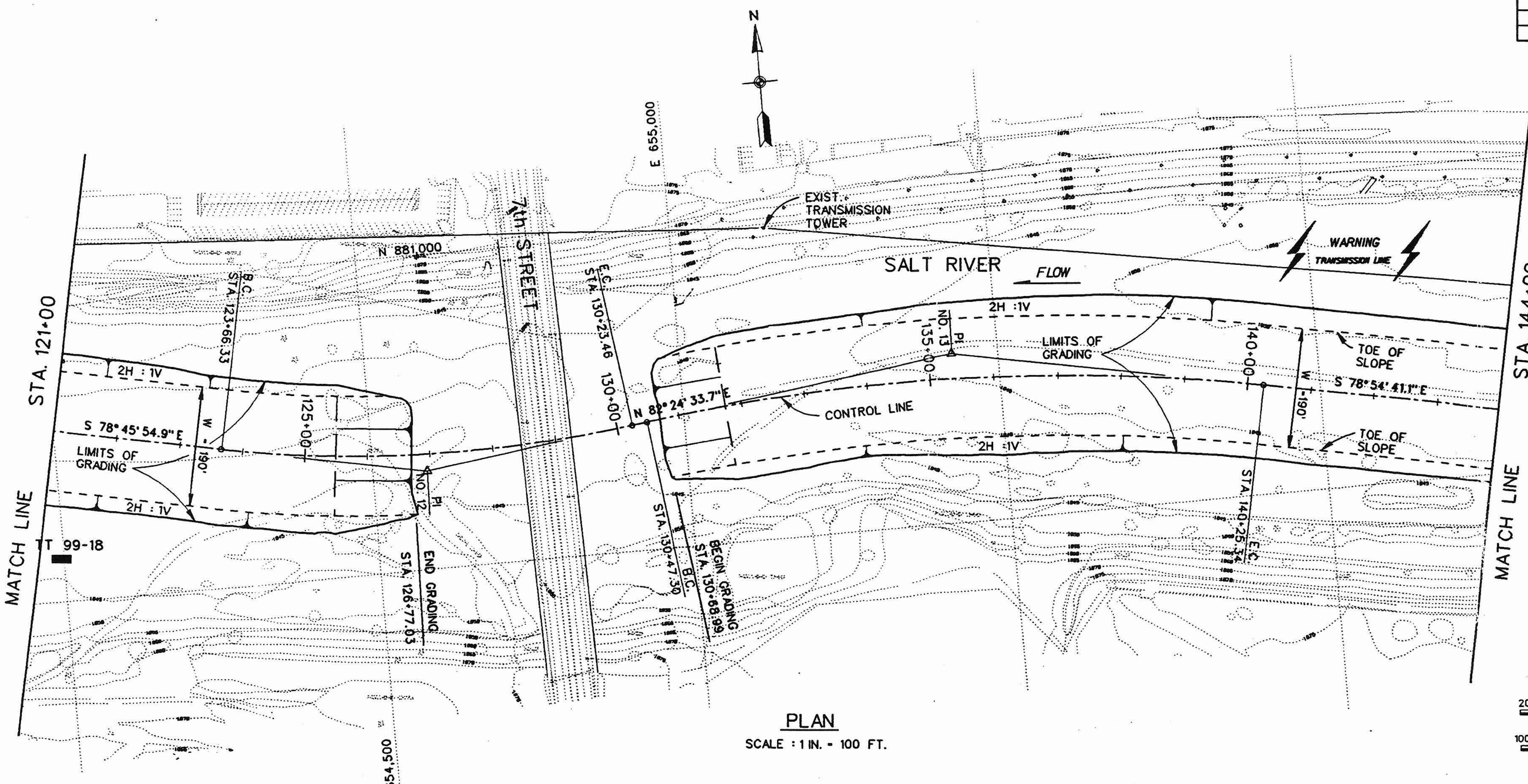


PROFILE AT CONTROL LINE

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 / WD
DRAWN BY: WD
CHECKED BY: DC
CADD FILE NAME: c8.dgn

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

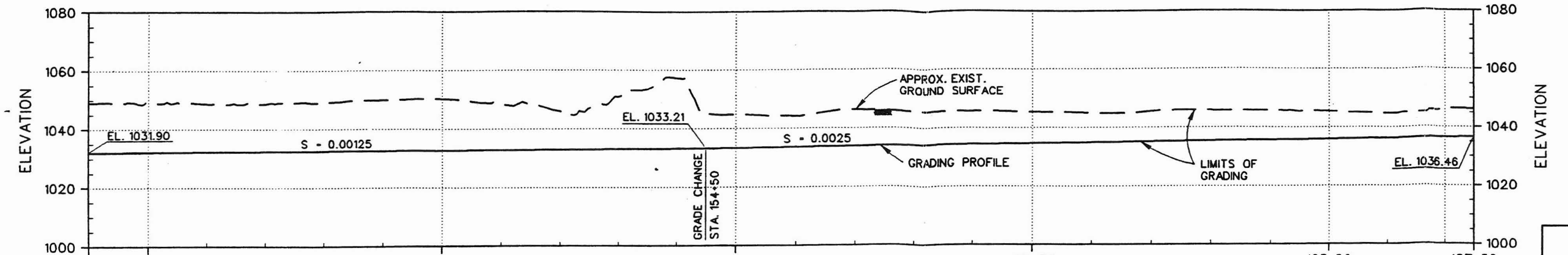
SUBMITTED BY:
THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

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

SHEET 8 OF 19 SHEETS

SAFETY PAYS

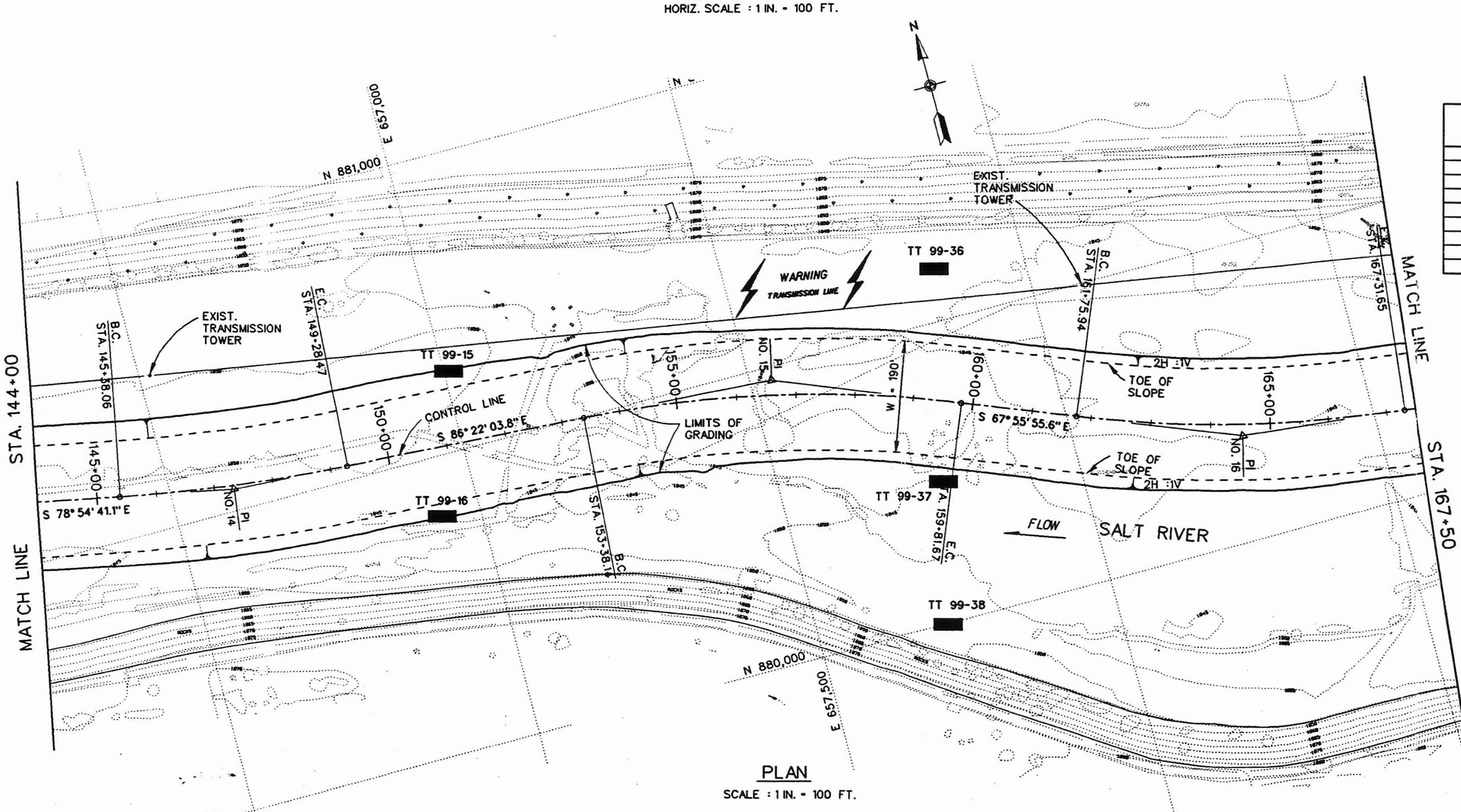
Figure 20



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



PLAN

SCALE : 1 IN. - 100 FT.

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

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

DESIGNED BY: CC / HD
DRAWN BY: HD
CHECKED BY: DC
CADD FILE NAME: c7.dgn

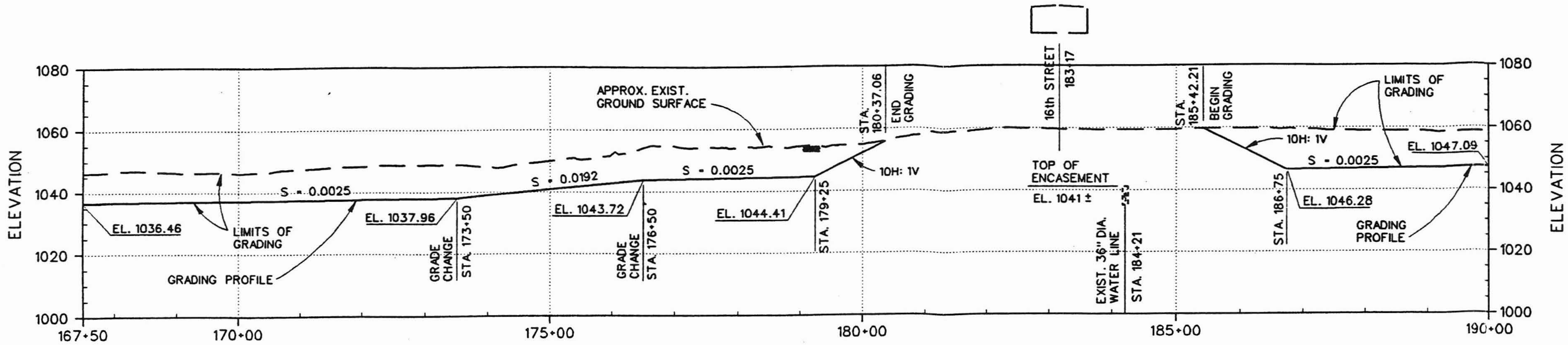
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

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

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

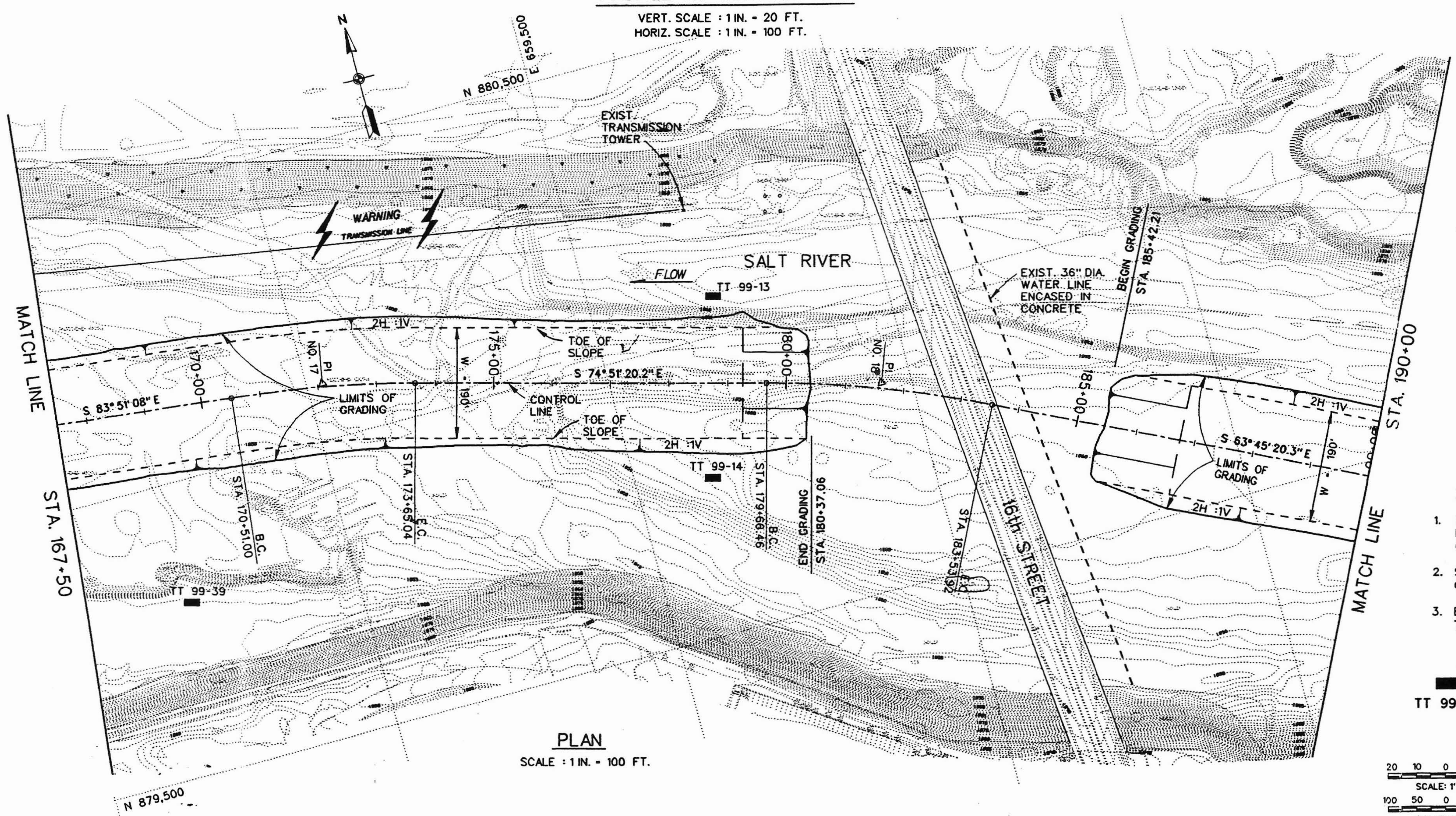
SHEET 9 OF 19 SHEETS

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'

SAFETY PAYS

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

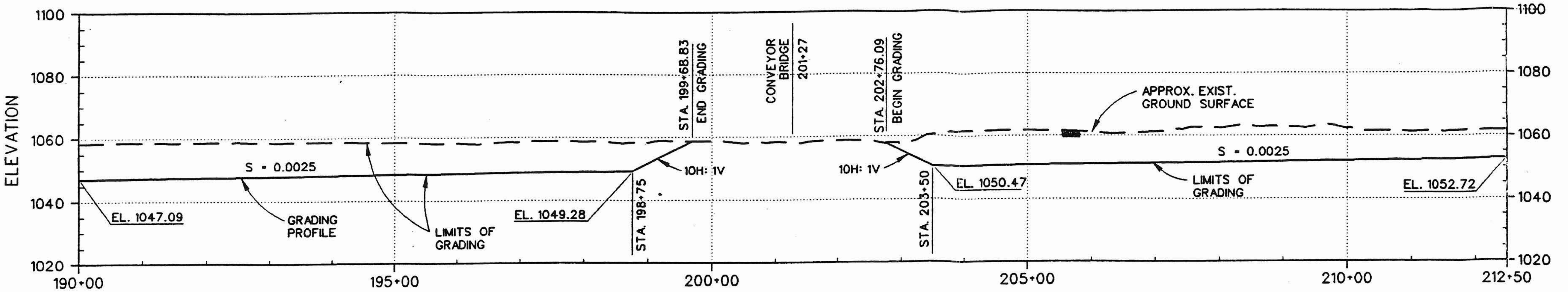
DESIGNED BY: CC / WD
DRAWN BY: ED
CHECKED BY: DC

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

DISTRICT FILE NO.: NOT APPLICABLE | SPEC. NO.: NOT APPLICABLE | CADD FILE NAME: c8.dgn

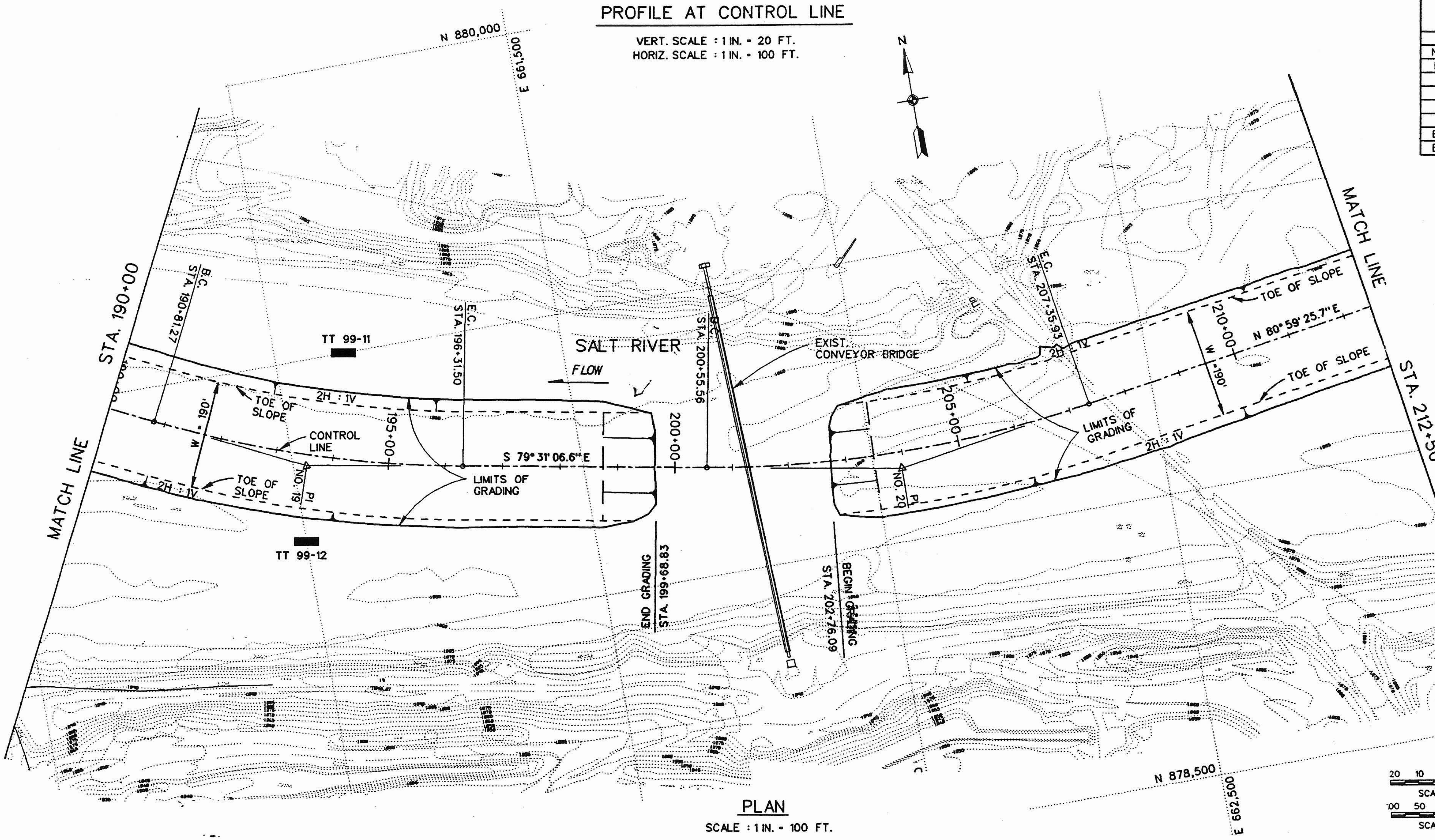
SHEET 10 OF 19 SHEETS

Figure 22



PROFILE AT CONTROL LINE

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



PLAN

SCALE : 1 IN. = 100 FT.

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

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

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

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

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

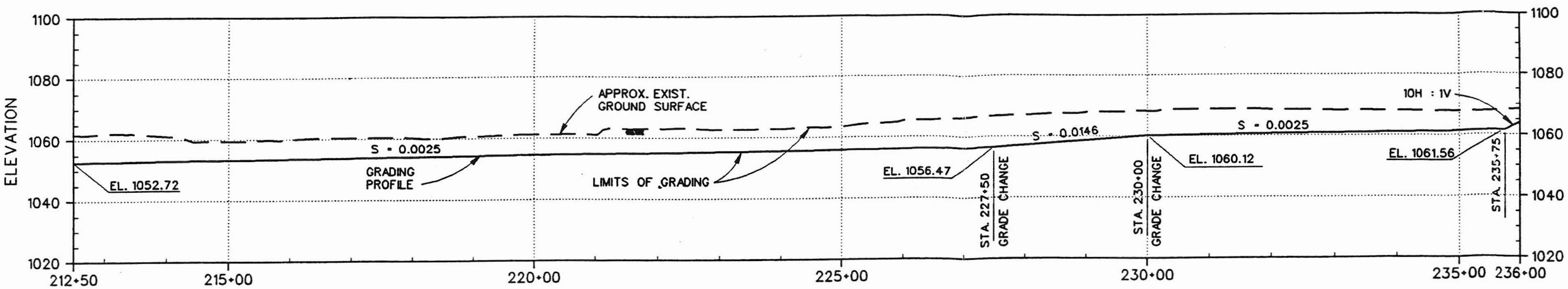
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

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

DISTRICT FILE NO.: NOT APPLICABLE
SPEC. NO.: NOT APPLICABLE
CADD FILE NAME: s8.dgn

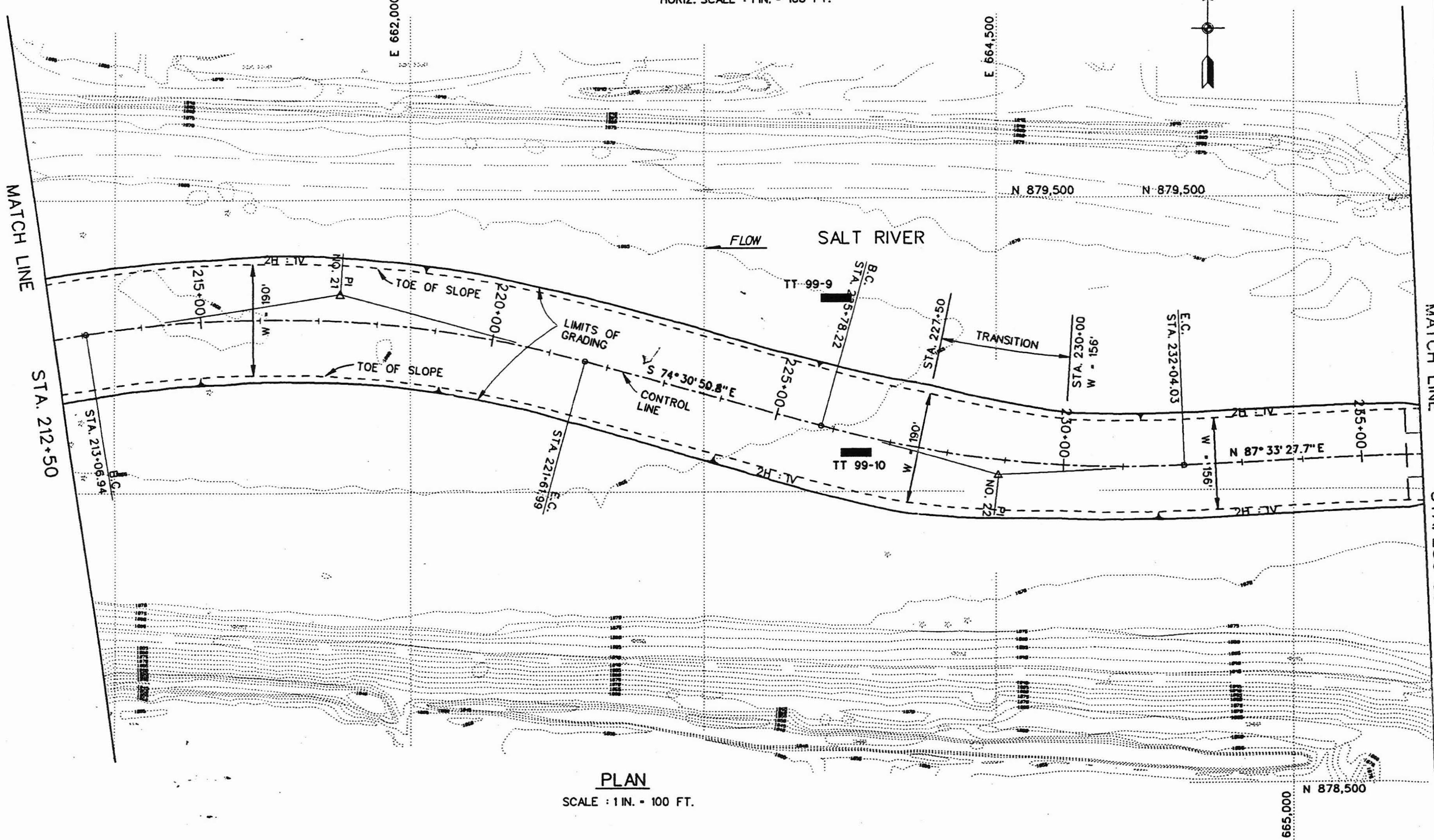
SHEET 11 OF 19 SHEETS

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

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

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

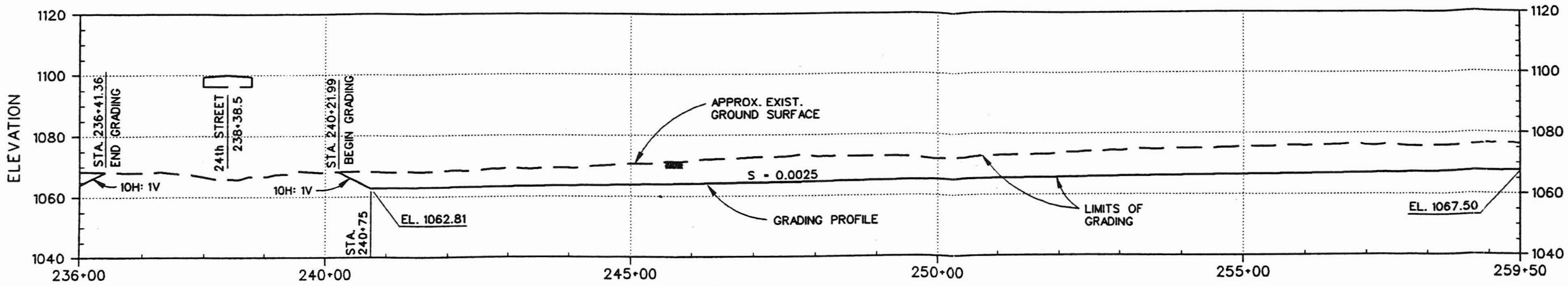
SUBMITTED BY:
THOMAS H. SAGE, P.E.
CHIEF, DESIGN BRANCH

DISTRICT FILE NO.: NOT APPLICABLE | SPEC. NO.: NOT APPLICABLE | CADD FILE NAME: s10.dgn

SHEET 12 OF 19 SHEETS

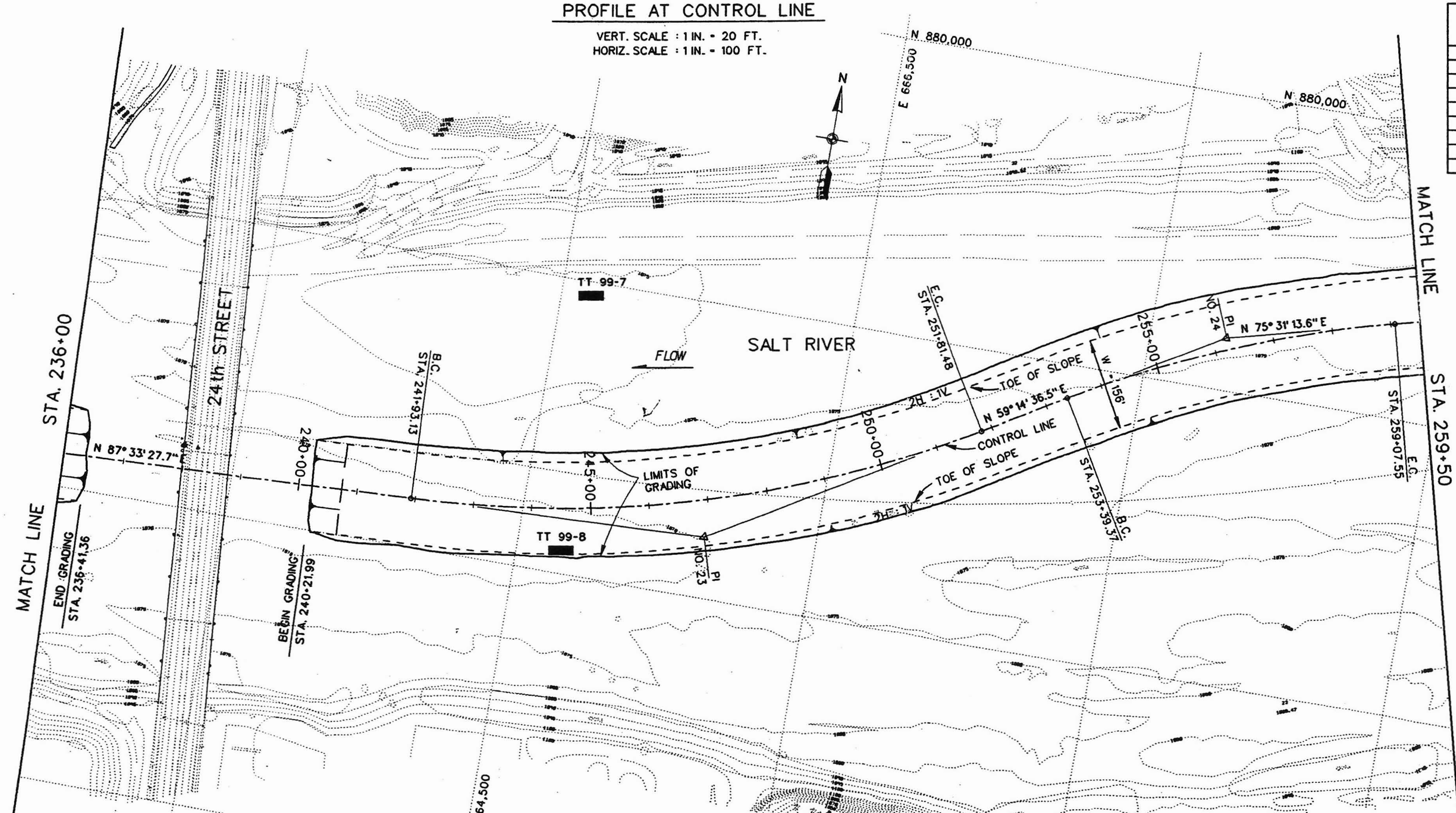
SAFETY PAYS

Figure 24



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.	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

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

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

CADD FILE NAME: C11.dgn

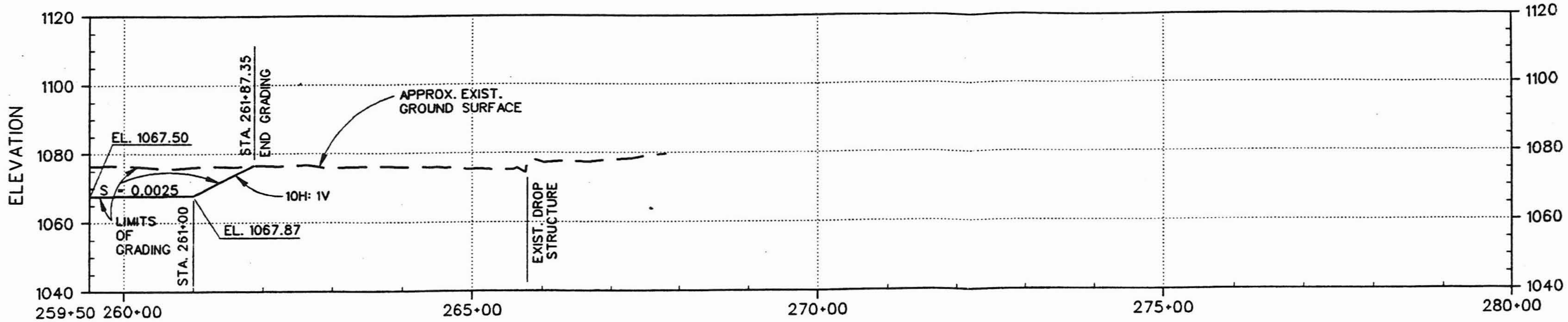
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

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

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

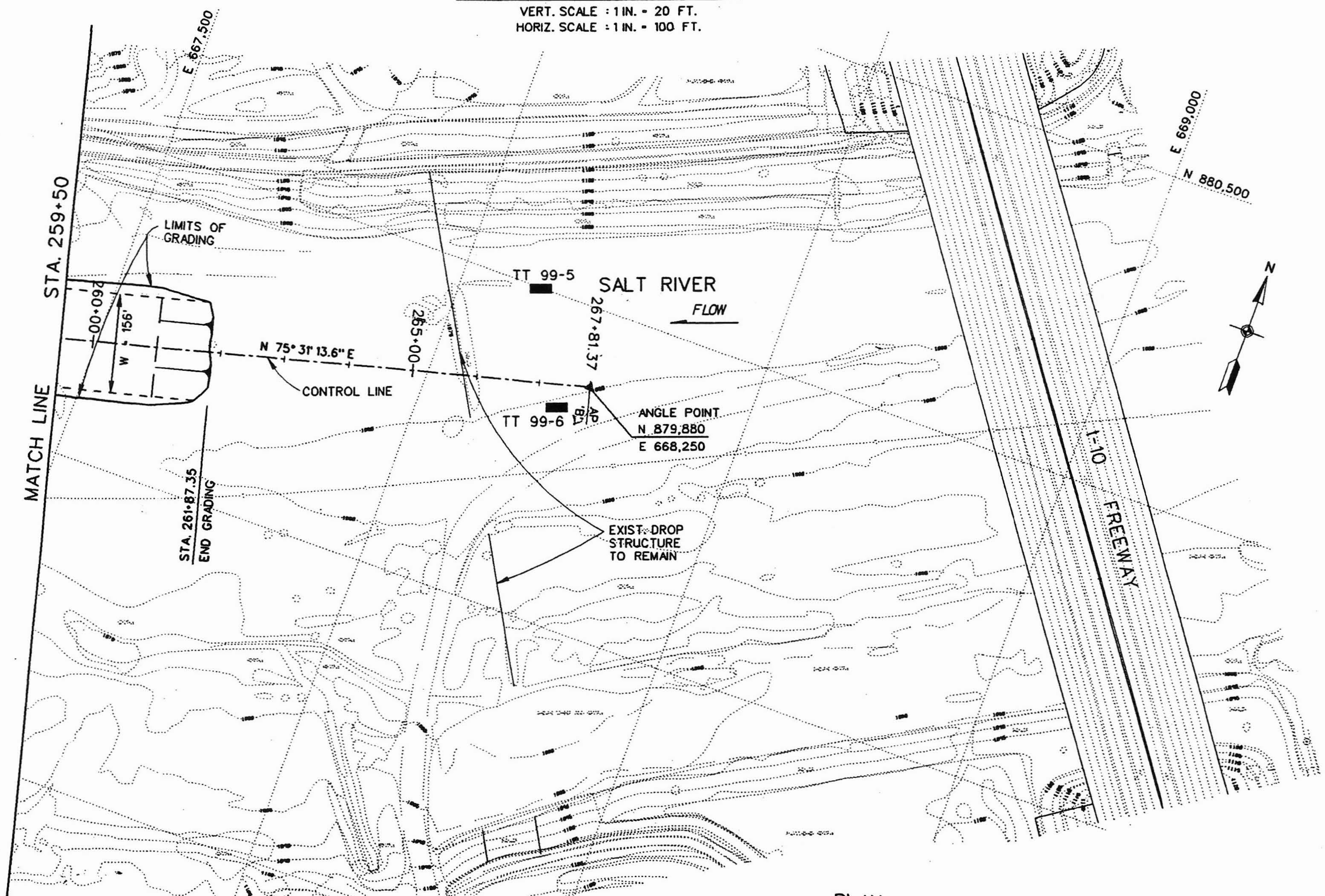
SHEET 13 OF 19 SHEETS

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

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

DISTRICT FILE NO.: NOT APPLICABLE | SPEC. NO.: NOT APPLICABLE | CADD FILE NAME: s12.dgn

SHEET 14 OF 19 SHEETS

Appendix D

MONTGOMERY WATSON

APPENDIX D

Uplift on RCC Reports

Buoyancy and Uplift Considerations for RCC Structures
Prepared by AGRA Earth & Environmental, Inc.
Prepared for West Consultants

AGRA
ENGINEERING GLOBAL SOLUTIONS

AGRA Earth &
Environmental Inc.
3232 West Virginia Avenue
Phoenix, Arizona 85009
Tel (602) 272-6848
Fax (602) 272-7239
Toll Free 1-800-248-AGRA

February 15, 2000
AGRA Job No. 0-117-001007
Letter No. 1

West Consultants, Inc.
2500 South Lakeshore Drive
Suite 210
Tempe. Arizona 85282-7054

Attention: Dennis L. Richards, P.E.

Gentlemen:

**RE: GEOTECHNICAL INVESTIGATION
DESIGN OF GRADE CONTROL STRUCTURES
PHOENIX, ARIZONA**

As requested, AGRA Earth & Environmental, Inc. (AGRA) has evaluated the stability of the grade control structure and upstream apron planned for the Salt River in the vicinity of the Central Avenue bridge. The analysis is based on the plans for the project developed by T.Y. Lin International (Sheets W-1 through W-8, 90 percent submittal, undated). The grade control structure includes a stepped-down roller compacted concrete (RCC) structure having a nominal thickness of 4 feet with RCC bank protection structures. The upstream apron is a RCC structure with a planned thickness of 4 feet extending about 241 feet upstream from the centerline of the grade control structure, and with a width of 360 feet. The upstream end of the apron will have a toe-down extending to a depth of 10 feet.

The stability analyses considered the following specific considerations:

- the effect of buoyancy on the upstream apron,
- the effect of buoyancy on the grade control structure,
- the potential for sliding of the grade control structure,
- the recommended thickness of the upstream apron, and
- the need for vertical weep holes in both the upstream apron and the grade control structure.

The analyses considered the condition of a storm flow event having occurred, resulting in a ground water level coincident with the elevation of the top the upstream apron at its juncture with the upstream end of the grade control structure (1025.53 feet). At the downstream bottom of the toe-down of the grade control structure (elevation 998.40 feet), the water pressure would be equivalent to 1,693 pounds per square foot (psf), or 27.13 feet. In the analyses, this differential head was distributed over the base of the base control structure, dependent on the elevation of the bottom of the components of the grade control structure.

Considering the effect of buoyancy on the upstream apron, the nominal pressure imposed by the RCC structure is 560 psf, which assumes a RCC unit weight of 140 pounds per cubic foot (pcf). The uplift pressure for the condition assumed is about 250 psf, resulting in a safety factor of 2.24 for the case of uplift, or buoyancy, of the apron. Considering a required factor of safety of 1.5 against uplift, then the required thickness of the apron is about 2.67 feet (2 feet and 8 inches). This would reduce the thickness of the apron by 1.33 feet (1 foot and 4 inches), and the quantity of RCC apron by 33 percent. The analysis also indicates that vertical weep holes in the upstream concrete apron are not required to provide for the stability of the apron.

Analysis of the grade control structure considered its upstream end at Station 101+34.00 (elevation 1015.53 feet) and its downstream end at Station 99+88.00 (elevation 998.40 feet), a distance of 146.00 feet and an elevation difference of 27.13 feet. Considering a cross section along the centerline of the structure (and not considering the bank control structures on either side of the Salt River channel or the connection to the upstream apron), and the groundwater conditions described in the previous paragraph, the weight of the RCC grade control structure (not considering the weight of the backfill to be placed above the structure) is approximately equal to the uplift pressure imposed by the water. Thus, the factor of safety relative to buoyancy is about 1.0, and the factor of safety relative to sliding of the structure (not considering its structural connection to the upstream apron) is less than one, which is not acceptable.

Considering the additional weight of the granular backfill (assumed in-place density of 140 pcf) that will be placed above the grade control structure (top of fill elevation of about 1024.53 feet), the factor of safety relative to buoyancy is increased to 2.1. Additionally, the factor of safety relative to sliding is increased to 3.94. Thus, the stability of the grade control structure, for the conditions assumed, is very dependent on the assumption that the backfill will be placed above the grade control structure and, more important, that the action of the high flow event will result in the scoured backfill being replaced by re-deposition of soil to the elevation of the low flow channel. If this is not the case, then the stability of the grade control structure should be reconsidered, or the placement of weep holes in the grade control structure to relieve water pressures should be considered.

Should you have any questions concerning the recommendations presented in this letter, please do not hesitate in contacting the undersigned.

Respectfully submitted,

AGRA Earth & Environmental, Inc.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13872
LAWRENCE A.
HANSEN
Date Signed 2/15/00
ARIZONA, U.S.A.

Lawrence A. Hansen, Ph.D., P.E.
Senior Vice President

c: Addressee: (3)

G:\Engineering-Development\2000Projects\0-117-001007GradeControlStruc\westltr1.wpd

AGRA
ENGINEERING GLOBAL SOLUTIONS

AGRA Earth & Environmental Inc.
3232 West Virginia Avenue
Phoenix, Arizona 85009
Tel (602) 272-6848
Fax (602) 272-7239
Toll Free 1-800-248-AGRA

March 3, 2000
AGRA Job No. 0-117-001007
Letter No. 2

West Consultants, Inc.
2500 South Lakeshore Drive
Suite 210
Tempe, Arizona 85282-7054

Attention: Dennis L. Richards, P.E.

Gentlemen:

RE: GEOTECHNICAL INVESTIGATION
DESIGN OF GRADE CONTROL STRUCTURES
PHOENIX, ARIZONA

As requested, AGRA Earth & Environmental, Inc. (AGRA) has completed additional evaluations of the stability of the grade control structure and upstream apron planned for the Salt River in the vicinity of the Central Avenue bridge. In our previous correspondence, it was concluded that the factor of safety relative to buoyancy for the grade control structure was about 1.0, if the scoured granular backfill is not re-deposited by the high flow event. However, that analysis considered only the stepped down portion of the grade control structure, and did not include the adjacent bank protection. The present analysis considers all components of the proposed structure.

Based on the plans for the project developed by T. Y. Lin International (Sheets W-1 through W-8, 90 percent submittal, undated), the weights of the various components of the grade control structure were estimated, assuming a unit weight of 140 pounds per cubic foot (pcf) for the roller compacted concrete (RCC). Assuming the same groundwater conditions described in our Letter No. 1, dated February 15, 2000, the buoyant forces acting on the various components of the grade control structure were also determined. These are presented in Table 1, along with the estimated weight of the granular backfill, assuming a fill unit weight of 140 pounds and a top-of-fill elevation of 1024.5 feet.

As discussed in our Letter No. 1, the combined weight of the channel weir and the channel grade control elements (40,000 kips) is about equal to the buoyant force acting on these elements (39,200 kips), resulting in a factor of safety near unity. Thus, considering only these elements of the overall structure, redeposition of the granular fill is required to provided sufficient weight to

TABLE 1

Estimated Grade Control Structure Weights & Buoyant Water Forces

Structure Element	Weight (1,000 kips)	Buoyant Force (1,000 kips)
Upstream Apron	25.2	11.2
Channel Weir	21.7	9.0
Channel Grade Control	18.3	30.2
Bank Protection	12.3	5.5
Overbank Grade Control	13.6	0.9
Granular Fill	39.2	---

offset the buoyant forces that will act on the these elements of the structure as the high flow event recedes. If about one-half the granular fill were redeposited, the factor of safety against buoyancy would be 1.52 and the factor safety against sliding would be 1.65.

However, if the weight of the connecting bank protection elements and the upstream apron are included, the total weight of the combined structure is 77,500 kips, compared to a buoyant force of 55,900 kips, resulting a factor of safety of 1.39. Thus, even without redeposition of the granular fill, the weight of the integral parts of the grade control structure, but excluding the over bank protection, is sufficient to resist the buoyant water forces that may exist for a very short period of time. For this case the factor of safety against sliding is 1.82. Because of the distribution of forces acting on the various parts of the grade control structure, moments and shear forces would need to be resisted by the RCC, particularly at the connection between the channel grade control structure and the weir, and the connections between the channel grade control structure and the bank protection elements.

Relief of the buoyant forces could be accomplished by placing weep holes in the channel grade control structure. Of concern would be the long-term integrity of the weep holes. Although intended to be a passive system, plugging of the weep holes could occur. However, based on the above simplified analysis of buoyancy and sliding, it is concluded that weep holes are not required.

Should you have any questions concerning the analyses or recommendations presented in this letter, please do not hesitate in contacting the undersigned.

Respectfully submitted,

AGRA Earth & Environmental, Inc.

Registered Professional Engineer (Civil)
CERTIFICATE NO.
13872
LAWRENCE A.
HANSEN
Date Signed 3/3/00
ARIZONA, U.S.A.

Lawrence A. Hansen, Ph.D., P.E.
Senior Vice President

c: Addressee (3)

G:\Engineering-Development\2000Projects\0-117-001007GradeControlStruc\gcsltr2.wpd

TABLE 1

Estimated Grade Control Structure Weights & Buoyant Water Forces

Structure Element	Weight (1,000 kips)	Buoyant Force (1,000 kips)
Upstream Apron	25.2	11.2
Channel Weir	21.7	9.0
Channel Grade Control	18.3	30.2
Bank Protection	12.3	5.5
Overbank Grade Control	13.6	0.9
Granular Fill	39.2	---