"To the best of my professional knowledge, judgment, and belief, this document meets applicable NRCS standards."

90 PERCENT DESIGN REPORT

VOLUME I

WHITE TANKS FLOOD RETARDING STRUCTURE NO. 3
REMEDIATION DESIGN PROJECT
PHASE 1

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

CONTRACT FCD 2004C017

PCN 470.04.30

NRCS
Natural Resources Conservation Service

"To the best of my professional knowledge, judgment, and belief, this document meets applicable NRCS standards."

90 PERCENT DESIGN REPORT

VOLUME I

WHITE TANKS FLOOD RETARDING STRUCTURE NO. 3
REMEDIATION DESIGN PROJECT
PHASE 1

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

CONTRACT FCD 2004C017

PCN 470.04.30

URS

September 13, 2004

Larry K. Lambert, P.E.
Project Manager
Flood Control District of Maricopa County
2801 West Durango
Phoenix, Arizona 85009

Re: 90 Percent Submittal
Draft Design Report
White Tanks FRS No. 3 – Remediation Project
Phase 1
PCN 470.04.30
Contract Number: FCD2003C055
URS Job No. 23443748

Dear Mr. Lambert:

URS Corporation (URS) has prepared this 90 Percent Design Submittal for the referenced project. The 90 percent submittal includes a Draft Design Report, Plans, and Specifications. Different form previous submittals for this project, the 90 Percent Design addresses only Phase 1 of the project. The project has been separated into two phases to allow more time to address design issues related to the embankment cracking. Phase 1 design includes the following:

- Design of the South Fissure Risk Zone Embankment and Transition Embankments.
- Geotechnical analyses relevant to the design of the Phase 1 embankments.
- Hydrologic and hydraulic analyses relevant to design of both Phase 1 and Phase 2 embankments, the emergency spillway, and outlet works.
- Structural analyses required for outlet works design.

Phase 2 design will include the North Fissure Risk Zone Embankment, two non-fissure risk zone embankments, and the emergency spillway.

The 90 Percent Design Report is separated into two volumes. Volume 1 includes the report, tables, and figures. Volume 2 includes the appendices. The design plans are provided separately. Your review of the design report and drawings will show that certain elements remain to be completed, most notably the fissure erosion modeling design. The draft fissure erosion modeling report is currently being reviewed and will follow this submittal. The draft Geotechnical Data Report and ADWR Permit Application Checklist will also be provided separately and will follow this submittal.

o 0 o

URS Corporation
7720 North 16th Street, Suite 100
Phoenix, AZ 85020
Tel: 602.371.1100
Fax: 602.371.1615

We look forward to meeting with you on September 21, 2004 to discuss your comments on the 90 Percent Report. In the interim, should you have questions or require additional information, please do not hesitate to contact me at your convenience.

Sincerely,

URS

Todd E. Ringsmuth, P.E.
Project Manager

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

1.1 PURPOSE OF DESIGN REPORT

The purpose of this design report is to present the design details of the proposed modifications to White Tanks FRS No. 3. Modifications of the existing dam are being performed to address the following dam safety issues:

- Insufficient freeboard for the inflow design flood;
- Transverse cracking through the embankment;
- Potential for fissure formation in two separate, identified fissure risk zones.

To address these issues, the embankment will be modified by constructing soil cement embankments in the fissure risk zones and earthen raises in the non-fissure risk zones. The fissure risk zone embankment will be designed to address fissure formation beneath the embankment and transverse cracking. The non- fissure risk zone embankment design will address the issue of transverse cracking. The embankment modifications will provide crest elevations that address the freeboard issues and potential future subsidence.

The design and construction of the modifications to White Tanks FRS No. 3 have been separated into 2 phases. Phase 1 will include design and construction of the South Fissure Risk Zone Embankment and Transition Embankment, which connect the new embankment with the existing structure. Also included in Phase 1 is the design and construction of the new outlet works. Phase 2 will complete the design and construction of the modifications to White Tanks FRS No. 3. Phase 2 includes the non-fissure risk zone embankments, North Fissure Risk Zone Embankment, and emergency spillway. Phase 2 will also include the placement of aesthetic fill material and incorporation of landscaping on dam and borrow areas.

This design report is intended to provide the design basis and supporting documentation required for the Phase 1 modifications. Design details and analyses relevant to design of the overall Remediation Project (i.e., Geotechnical Investigations, Reservoir Routing, Emergency Spillway Design) are included in this design report. It is anticipated that Phase 1 and Phase 2 will be permitted and constructed separately.

1.2 PROJECT BACKGROUND

White Tanks Flood Retarding Structure (FRS) No. 3 is located on alluvial fan deposits east of the White Tank Mountains, approximately 20 miles west of Phoenix. The dam and its appurtenant facilities were designed and constructed by the Soil Conservation Service (SCS, now Natural Resource Conservation Service [NRCS]) in 1954. The facility is currently operated and maintained by the Flood Control District of Maricopa County (District).

The dam embankment was constructed as a homogenous earthfill with a crest width of approximately 11 feet, a maximum height above streambed elevation of approximately 30 feet, and 2:1 and 2.5:1 downstream and upstream slopes, respectively. Three gated, corrugated metal pipes (CMPs) through the embankment serve as the principal outlets for the dam. The secondary or emergency spillway is an unlined earthcut spillway located in at the right (south) abutment of the dam. In the 1980s, the NRCS designed and installed a granular filter along the centerline of the embankment. Several outlets were also installed to drain the center filter. In addition, the District installed sand diaphragm filters around the three principal outlets. The centerline filter does not extend to the foundation soils. Details of the dam modifications are provided in Sections 4.0 of this report.

Since the original design and construction of the dam, conditions at and in the vicinity of the dam have changed significantly. These changes include the following:

- Potential downstream consequences related to potential failure of the dam have increased significantly. The dam was originally intended to provide flood protection for agricultural lands. Since the original construction, significant urbanization has occurred, and is expected to occur at an increasing rate downstream from the dam.
- Withdrawal of groundwater for agricultural and domestic use has caused lowering of the water table and regional ground subsidence. A level survey along the crest of the dam performed by the District in November 2003 indicates that differential subsidence across the length of the embankment has lowered the north end of the embankment by nearly 4 feet from the original design crest elevation, while the loss of crest elevation (compared to design crest elevation) at the south end of the embankment is less than 1 foot.
- Differential subsidence has induced tensile stresses in the ground, creating the potential for earth fissuring. Investigative work performed by AMEC Earth & Environmental Inc. (AMEC) on behalf of the District has identified two fissure risk zones that intersect the existing and proposed embankment north extension. The fissure risk zones are located at existing Stations 30+00 and 55+00, and north of Station 0+00.

- Transverse cracks have developed across the embankment. The exact cause(s) of these cracks is not known. The cracks were likely caused by desiccation and shrinkage of the compacted soils, and perhaps to a lesser extent, because of hydro-collapse of relatively young (Holocene) soils underlying the embankment.

1.3 SCOPE OF WORK

The overall objective of this project is to design modifications to the dam and its appurtenant structures to mitigate risk related to dam safety concerns and to meet current regulations and standards as provided by the NRCS and the Arizona Department of Water Resources (ADWR). The objective also includes the restoration of flood protection and extending the structure life an additional 100 years. This overall objective will be achieved by completion of a series of tasks revolving around the implementation and design of a selected alternative. These tasks were discussed in detail in Scopes of Work for Work Assignments 1 and 2, dated January 21, 2004 and March 1, 2004, respectively. The key elements of URS' include the following:

- Subsidence evaluation
- Hydrologic and hydraulic analyses
- Geotechnical investigations and analyses
- Fissure erosion assessment and modeling
- Structural analyses and design
- Developing designs for embankments, emergency spillway, outlet works.
- Preparing construction plans and specifications.

1.4 COOPERATING AGENCIES

The following are the primary entities involved in this project:

- **Flood Control District of Maricopa County.** White Tanks FRS No. 3 is currently operated and maintained by the District. The District is a funding partner during the design and construction phases of the project. Larry K. Lambert, P.E. serves at the District's project manager for the design phase of the project.
- **Natural Resource Conservation Service.** The NRCS (then SCS) designed and built the dam in 1954. The NRCS has remained involved with this dam, currently serving as a major federal funding partner for the proposed rehabilitation. Mr. Ildefonso Chavez, Jr. of the NRCS is the designated Project Manager.

- **Arizona Department of Water Resources.** White Tanks FRS No. 3 is a jurisdictional structure with a "Significant Hazard" classification due to the structure height and reservoir capacity. ADWR currently provides regulatory oversight for jurisdictional dams in Arizona.

1.5 ADWR PERMIT APPLICATION CHECKLIST

The permit application checklist required to be submitted with the permit application is provided in Appendix A. **[Note to Reviewers: The permit application checklist will be included with the 100 Percent submittal.]**

1.6 CONSTRUCTION COST ESTIMATE

A construction cost estimate has been prepared for the Phase 1 design of the White Tanks FRS No. 3 – Remediation Project. The cost estimate is provided as a separate submittal. Material quantities used in developing the cost estimate are presented on the plans. Calculations of quantity estimates are provided in Appendix K.

The construction of the White Tanks FRS No. 3 remediation will be conducted in 3 phases to meet the funding constraints established by the District and NRCS. NRCS has provided funding in the amount of $6 million for Phase 1, to be spent on construction activities between January 2005 and October 2005. It is currently anticipated the NRCS will provide an additional $9 million dollars for Phase 2, with funds to be spent between November 2005 and October 2006. Phase 3 will be funded entirely by the District and will occur after November 2006.

2.0 SURVEY AND MAPPING

2.1 TOPOGRAPHIC MAPPING

The topographic mapping prepared for the project site consisted of three separate maps developed at different times and on different datums. The 2003 topographic mapping generally cover the area from the Bethany Home Road Alignment to the south, Beardsley Canal to the east, 199th Avenue to the west, and Orangewood Avenue to the north. The 2003 topography includes the existing dam and a majority of the reservoir flood pool. The 2003 topography was developed using the NAVD 88 Datum.

Additional surveying was performed in May of 2004 to provide topographic mapping of the area north of the existing dam along Beardsley Canal and the North Inlet Channel. This additional topography was required for design of the North Dam Extension. The 2004 topography was developed using the NAVD 88 Datum.

The 2003 and 2004 topographic mapping did not extend to include the entire reservoir pool. In order to develop the elevation-area-capacity data for reservoir routing, historic topography was modified. The District provided URS with topographic mapping and the base digital terrain mapping (DTM) files that included the additional areas. However, this topography was developed in 1998 and was based on the NGVD 1929 Datum. Another issue that potentially affected the 1998 topographic mapping is the subsidence that has likely occurred since 1998 and 2003. Therefore, URS manipulated the DTM file to develop topography that reflects the current NAVD 88 Datum and take into account subsidence.

The DTM file shift consisted of the following:

- Calculate the elevation shift between the NGVD 29 and NAVD 88 Datums at Benchmark USGS N475. This District estimated the shift to be an increase in elevation of 1.87 feet.
- Estimate the total subsidence that has occurred at the left abutment of the existing dam. The total subsidence that occurred between 1998 and 2003 at the dam crest benchmark SM-A1 (existing Station 10+00) was 0.027 feet.

Therefore, the DTM file was shifted up in elevation by 1.843 feet and a topographic map was developed.

2.2 VERTICAL DATUM

The design documents prepared for this project are developed using the North American Vertical Datum 1988 (NAVD88). Historical references and drawings for the White Tanks FRS No. 3 are based on the National Geodetic Vertical Datum 1929 (NGVD 29). The shift between the 1929 and 1988 datums has been identified by the District as 1.87. Due to the potential confusion, elevations presented in this report include the referenced datum is also provided.

3.0 FACILITY DESCRIPTION

White Tanks FRS No. 3 was constructed in 1954 by the NRCS to protect farmland and irrigation facilities from runoff collected off the White Tank Mountains. The dam is located on alluvial fan deposits east of the White Tank Mountains, approximately 20 miles west of Phoenix. The northern end of the embankment is approximately 1 mile south of the intersection of Northern Avenue and the Beardsley Canal in Maricopa County. The dam is a homogeneous earth embankment. The dam is currently maintained and operated by the District.

3.1 ORIGINAL CONFIGURATION

3.1.1 Embankment

The embankment is approximately 7,700 feet long, and was constructed using soils borrowed from the reservoir area. At its maximum section, the embankment is approximately 27 feet high. The crest width varies between 10 and 11 feet. The upstream and downstream faces are sloped at 2.5:1 (horizontal to vertical) and 2:1, respectively. The embankment soils are predominantly clayey sands with lesser amounts of sandy clays present.

3.1.1.1 Foundation Preparation

The foundation footprint was cleared and grubbed. There appears to have been no attempt to overexcavate and recompact the near-surface soils, or to remove granular channels that intersected the alignment. The soils underlying the embankment are predominantly silty and clayey sands with lesser amounts of sandy clays, and occasional layers of relatively clean sands.

3.1.2 Watershed

White Tanks FRS No. 3 was originally designed to impound runoff from a drainage area of approximately 24 square miles. A Phase II flood study performed by the District (1984) noted that portions of the watershed had been removed due to the breaching of training dikes and diversion channels north of Northern Avenue and the redirection of flows from the Caterpillar Test grounds. These changes reduced the tributary area of the structure to approximately 20.5 square miles, a reduction of 3.5 square miles (District 1984). The elevation of the watershed ranges from over 4,000 ft (NGCD 29) to the outlet works inlet elevation of approximately 1,188 ft (NGVD 29).

3.1.3 Flood Pool

The capacity of the reservoir at the time of construction was 2,655 ac-ft below the emergency spillway crest. The emergency spillway crest elevation was 1,210.0 feet (NGVD 29), or 1,211.87 feet (NAVD 88). The surface area of the flood pool at the emergency spillway crest was 280.6 acres.

3.1.4 North Inlet Channel

The north inlet channel runs for approximately 2 miles from north of Olive Avenue to the north end of the White Tanks FRS #3 embankment. The channel crosses Olive and Northern Avenues. The channel runs parallel to and on the west side of the Beardsley Canal. It is not clear when the channel was constructed. However, the channel serves to capture areas of the watershed that were included in the original design. The channel significantly increases the size of the watershed contained by White Tanks FRS #3: with the channel, the watershed is 20.49 square miles; without the channel, the watershed would be 9.72 square miles (NRCS 1998).

Historic subsidence has occurred at the north end of the dam, and along the North Inlet Channel, requiring that the dam be extended north to contain the design flood pool. The dam extension will be parallel to the channel and potentially require erosion protection along the upstream face of the dam.

3.1.5 Sediment Pool

The NRCS design incorporated sediment pool of 500 acre-feet (NRCS 1996) corresponding to a 100-year design life. The 500 ac-ft allowance for sediment accumulation corresponds to an elevation of 1,197 ft (NGVD 29), or a maximum of 21 ft above the current lowest surface behind the dam, as estimated from the elevation-capacity relationship shown on Figure 4-1. The upstream inverts of the existing North, Central, and South gated outlet pipes are at elevations of 1,190, 1,188, and 1,190 ft, respectively (NGVD 29).

3.1.6 Emergency Spillway

The emergency spillway is cut into natural ground at the south abutment of the dam. ADWR's inspection report (2002) indicates that the emergency spillway crest elevation is approximately 1,211.92 feet (NGVD 29). The unlined spillway was constructed 800-ft-wide for a design peak flow of 11,750 cubic feet per second (cfs).

Dames & Moore (1998) estimated that during discharge under the full probable maximum flood (PMF) conditions, the flow depths and velocities at the crest of the spillway were 4 feet and 6 feet per second (fps), respectively. Based on these depths and flow velocities, Dames & Moore (1998) predicted scour and head cutting at the emergency spillway.

3.1.7 Bethany Home Road Dike

The Bethany Home Road Dike begins at the south edge of the emergency spillway and runs eastward to the Beardsley Canal. The purpose of the dike appears to be for directing flows that pass through the spillway to a siphon crossing in the canal. The existing dike is located mostly off District property. Review of the design drawings suggests that the dike was intended to be constructed at heights ranging from 5 to 7 feet above the existing grade.

3.1.8 Principal Outlets

Three corrugated metal pipes (CMPs) serve as the principal outlets for the dam. These CMPs are located at stations 29+00, 46+00, and 63+80 (based on existing stationing). The two pipes at stations 29+00 and 46+00 are 48 inches in diameter, while the third outlet is 24 inches in diameter. One of the 48-inch outlets is connected to the Beardsley Canal via a concrete-lined channel, while the other two outlets discharge at the downstream toe of the dam. All three outlet pipes are provided with steel seepage collars. According to construction drawings, the collars are spaced at 20 foot centers and extend for a distance equal to the diameter of the pipe beyond the outlets. The outlets are provided with a protective asbestos-containing coating on inside and outside. The three outlets are regulated by control gates at the upstream end. The gates are manually operated and are fitted with stems which extend to the crest of the embankment.

3.2 DAM MODIFICATIONS

Since the original construction of White Tanks FRS No. 3, the facility has been modified to address dam safety issues that have arisen, and to improve the overall performance and safety of the dam. These modifications are discussed below. Additional details of previous modifications to the dam are provided in Section 4.0.

3.2.1 Central Filter and Outlet Drains

The NRCS designed and installed a granular filter along the centerline of the embankment to mitigate the impacts of the transverse cracking. The filter was installed for the entire length of the embankment and is approximately 30 inches wide. The center filter trench was backfilled with a medium to coarse sand. The filter does not extend to the foundation soils. However, it appears that outlets were installed at all locations where the transverse cracks extended below the bottom of the center filter trench. A total of about 68 outlets were installed. Each outlet includes a 2-foot by 2-foot section of open graded gravel to increase flow capacity. Additional information concerning the construction of the central filter and outlet drains is provided in Section 4.2 of this report.

3.2.2 Diaphragm Filters

In 2000, the District retained URS to design interim dam safety measures, that included installation of diaphragm filters around the three existing outlet pipes. The existing outlet pipes consist of corrugated metal pipes (CMPs) The diaphragm filters were designed and constructed in general accordance with NRCS guidelines. Details of the project are provided in a design report prepared by URS (2001).

All three conduits were extended. The extensions were encased in concrete to the spring-line. Sand diaphragms were constructed directly downstream of the embankment. The sand diaphragms were weighted down with buttress fill in order to counter potential hydrostatic pressures caused by a full reservoir. The design also included the design and installation of trash racks on the upstream end of the conduits.

3.2.3 Emergency Spillway Modifications

In 2000, the District retained URS to design interim dam safety measures, which included excavating a notch through the emergency spillway and provided erosion protection along the downstream toe of the embankment. The notch was excavated 75 feet wide and lowered the spillway crest to an elevation of 1,207.0 ft (NGVD 29). The notch elevation was set at this elevation to provide a minimum of 4 feet of dry freeboard below the lowest dam crest elevation of 1,211.39 ft (NGVD 29). The design notch elevation accounted for future potential lowering of the dam crest of 0.266 ft due to subsidence. The material excavated from the notch was used to construct the buttresses placed over the diaphragm filters at the outlets.

3.3 INTERIM OPERATIONAL PLAN

The District implemented an interim operational plan for the outlets following modification of the dam under the Interim Dam Safety Project (See Section 4.3). These modifications included constructing a notch lowering the emergency spillway crest, installing diaphragm filters near the downstream end of the outlets, and installing trash racks over the upstream end of the outlets. ADWR required that an interim operational plan be developed for permit approval of the interim design.

The Interim Operational Plan developed by the District details operational requirements that must be undertaken by the District during a reservoir filling event (FCDMC 2001). The Plan included the following requirements:

- The District's Operation & Maintenance Division (O&M) is notified by the District's ALERT staff and sent to the dam for around-the-clock watch when the reservoir is 25 percent full. The percentage full is measured as a volume of storage available below the emergency spillway crest. The Maricopa County Department of Emergency Management is also notified.
- When the volume reaches 50 percent full (a reading of 12 ft on the staff gage) the gate on the 48-inch Central Outlet is to be opened.

4.0 PREVIOUS STUDIES AND PROJECTS

4.1 ORIGINAL NRCS DESIGN

White Tanks FRS No. 3 was built as a flood control structure in 1954. It was a homogenous earth dam constructed by the NRCS (then the Soil Conservation Service [SCS]). The embankment was approximately 7,700 feet long and was constructed using material borrowed from the reservoir of the dam. The embankment was approximately 30 feet tall with a crest width of about 11 feet. The upstream and downstream slopes are constructed at 2.5:1 (horizontal:vertical) and 2:1, respectively. Three gated corrugated metal pipes (CMPs) placed through the embankment serve as the principal outlets form the reservoir, as described in Section 3.1.8 of this report. The emergency spillway was cut into natural ground at the right abutment of the dam and constructed with a crest elevation of 1,210 ft (NGVD 29).

4.2 MODIFICATIONS DESIGN PROJECT

Since the construction of White Tanks FRS No. 3 in 1954, the embankment has exhibited transverse, and to a lesser extend, longitudinal cracking. Multiple investigators studied the dam to evaluate the cause and potentially detrimental effects of these features. Fugro Inc. performed the most comprehensive investigation in 1979. The investigation identified that 60 percent of the embankment had experienced no cracking, 34 percent had a low degree of cracking, and 6 percent has a moderate to severe degree of cracking.

Between 1981 and 1982 NRCS (then the SCS) initiated a program to implement corrective actions. It was found during reconstruction that the cracking was more extensive than originally had been suspected. The section of the dam between Stations 56+10 and 59+90, which showed the worst cracking, was intentionally breached. This section was reconstructed using excavated materials, and additional soil from designated borrow sources. A central chimney filter was installed the entire length of the embankment. The design trench width was approximately 3 feet wide and extended 3 feet below the maximum depth of the cracks observed within the excavation, but did not extend into the foundation soils. Finger drains were provided at locations of selected cracks to convey water intercepted by the chimney filter.

4.3 INTERIM DAM SAFETY PROJECT

The Interim Dam Safety Project consisted of design and construction of interim dam safety measures in 2001 and 2002. The design was prepared URS (then Dames & Moore) and presented in the report *Interim Dam Safety Improvements – White Tanks FRS No. 3*. The project included the following activities:

- Excavation of a notch within the emergency spillway to provide a minimum dry freeboard of 4 feet.
- Construction of diaphragm filters around the outlet conduits on the downstream side of the dam.
- Installation of trash racks on the upstream end of the 3 outlets.
- Development of an Interim Operations Plan (prepared by the District).

4.4 BASINS ALTERNATIVES PROJECT

The District contracted with URS to evaluate the concept of replacing the White Tanks FRS No. 3 dam structure with one or more basins. The development of basin alternatives was presented in the *Design Issues/Basin Alternatives Report* prepared by URS in August 2001. Basin designs were developed and evaluated to provide alternatives to remediating the existing dam. Alternatives included engineering and multi-use recreation components. Cost estimates were developed for each alternative.

4.5 PRELIMINARY DESIGN CONCEPTS

Preliminary design concepts were developed for remediation of White Tanks FRS No. 3 and presented in *Preliminary Embankment Rehabilitation Concepts* (URS 2004). Design concepts were developed to address the potential failure modes related to transverse cracking and earth fissures. The concepts included the use of geomembranes, sand and graded filters, and hardened embankments.

4.6 DAM ALTERNATIVES PROJECT

AMEC evaluated various alternatives for remediation or replacement of the existing White Tanks FRS No. 3 dam. Their work was presented in the report *Realigned Dam Alternatives and Preferred Alternative Recommendation* (AMEC 2004a). Alternatives included realignment of the dam downstream of the existing site, modification of the existing dam, and replacement of the dam with a basin. A detailed geotechnical investigation was performed and is discussed in more detail in Section 7.0 of this report. A preferred alternative was selected through a screening process and included input from the District, NRCS, ADWR, and other interested parties. The preferred alternative was determined to be modification of the existing dam, which was the basis for design of the embankment presented in this report. The selected design consisted of a soil cement embankment in the fissure risk zone and an earthen raise with geomembrane in the two non-fissure risk zones.

5.0 PROJECT DESIGN LIFE

The design life for the project has been identified as 100 years in the *Rehabilitation Plan/Environmental Assessment for the White Tanks No. 3 Project* (NRCS 2004). The design developed to rehabilitate the existing dam will meet current design and safety criteria in order to provide continued flood protection. All elements of the design (i.e., sediment storage, material selection, hydrology, etc.) are intended to meet the 100-year design life.

6.0 LAND SUBSIDENCE

Section 6.0 of this report discusses historic and future subsidence predictions in the vicinity of White Tanks FRS No. 3. Prediction of future subsidence at the White Tanks area is an important factor in the ongoing design of remediation measures for the existing dam. Development of reasonably good estimates of future ground settlement (caused by regional subsidence) is critical for two main reasons:

- Establishing the new dam crest elevation (to ensure adequate freeboard in the future).
- Estimating risk of future fissure development, which could be related with the magnitude of future subsidence.

Underestimation of the future subsidence is not an immediate dam safety issue since, the dam crest could be incrementally raised in the future based on the observed subsidence trend. However, if underestimation of future subsidence leads to underestimation of fissure potential within the White Tanks area, then that would be a critical dam safety issue.

This section summarizes the results of subsidence evaluations from thee different sources:

- Geological Consultants Inc.'s (GCI) evaluation based on historical subsidence and a prediction of future groundwater withdrawal. GCI is a subconsultant to URS for this project.
- The District's independent evaluation performed by Dr. Dennis Duffy.
- URS's modified approach using the classical one-dimensional consolidation theory.

A general discussion is presented comparing the results from the three different studies and recommending a common conclusion of results. Information presented in this section regarding geologic setting, groundwater conditions, and historic subsidence was taken from GCI's technical memorandum.

6.1 GEOLOGIC SETTING

The White Tank Mountains are composed primarily of Precambrian igneous (granites, granodiorites, pegmatites) and metamorphic (gneiss, schist) rocks and Tertiary sedimentary and volcanic rocks. These north-south trending mountains form the western boundary of the western Salt River Valley (Arizona Geological Survey 1988). Surficial geology deposits in the area of FRS No. 3 include Holocene (0 to 10,000 years age) alluvial surfaces (Y) in larger drainages;

poorly sorted, angular to subangular admixture of silt, sand, and gravel of early to late Pleistocene (10,000 to 150,000 years age) alluvial fan material (M2) typically with a poorly to moderately developed desert pavement; and poorly sorted, angular to subangular admixture of silt, sand, and gravel of middle to late Pleistocene (150,000 to 300,000 years age) alluvial fans (M1b) with moderately to well developed cobble to pebble desert pavement. The middle to late Pleistocene surfaces are relatively thin but laterally extensive. The younger sediments are very thin and older units typically are exposed in small pockets in these areas.

White Tanks No. 3 FRS was constructed on a sequence of middle to late Pleistocene age (10,000 to 300,000 years) relic alluvial fan deposits. The alluvial fan deposits include interbedded lenses of poorly sorted, angular to subangular admixtures of silt, sand, and gravel. Beneath the poorly preserved gravel bar and swale topography of the alluvial fan surface, the soils are characterized by a weakly developed clayey horizon above a zone of caliche cemented (Stage II) soils. Because the genesis, or development, of these alluvial fan deposits is the result of periodic deposition of coarse-grained sediment during flash flood events, old buried coarse-grained alluvial stream channel deposits are likely included within the alluvial fan deposit. It is anticipated that the vertical and lateral distribution of the old, buried stream channel deposits would be variable; however, the alignment of the old channels would likely be similar to the Holocene channel regime (essentially normal to the axis of the FRS). Modern stream channels of recent to early Holocene age (<1,000 to 10,000 years) are incised into the Pleistocene age relic alluvial fan deposits. White Tanks No. 3 FRS is constructed across at least two major Holocene stream channels that contain coarse-grained, poorly sorted beds of silt, sand, and gravel. These Holocene channel deposits are unconsolidated and uncemented to very weakly cemented.

The basin or valley floor at the site is underlain by up to several thousands of feet of permeable alluvial sediments that comprise the alluvial aquifer system. These sediments were deposited by streams entering the valley from the west, north and east. The sediments store large volumes of groundwater and yield moderate to large volumes of water from deep irrigation and water supply wells. The sediments are also subject to subsidence or settlement and cracking as groundwater is withdrawn. A zone of potential cracking or fissuring has been delineated (AMEC, 2004) beneath the central portion of the embankment, based on the geometry of bedrock below the site, thickness of alluvium, location of groundwater withdraw, and location of a resulting tension zone in the alluvium.

The floor of the western Salt River Valley consists of coalesced alluvial fans or an alluvial pediment of Quaternary age. These alluvial deposits have been categorized in several different studies as three primary units: an upper alluvial unit (UAU), a middle alluvial unit (MAU), and a lower alluvial unit (LAU). These units vary in thickness in different portions of the basin. Because of the differing properties of these units, there are lateral and vertical variations in the subsurface stratigraphy across the basin. The LAU overlies or is in fault contact with the bedrock units of the mountains and consists of moderately to well-consolidated sand and gravel near the margins of the basins and grades laterally into mudstones and evaporite deposits in the central parts of the basin. The MAU overlies the LAU and consists of weakly consolidated sand and gravel near the margins of the basin, grading laterally into mudstone and evaporite deposits near the central part of the basin. The MAU is generally less permeable than the overlying UAU. The UAU consists of Quaternary gravel, sand, and lesser amounts of silt and clay.

6.2 GROUNDWATER CONDITIONS

Groundwater occurs in unconfined to semi-confined conditions in the alluvial sediments that underlie the valley floor. In 1923 the direction of groundwater flow was to the south, and then west, before large scale pumping began in the western Salt River Valley. Prior to pumping, the groundwater system was in equilibrium. Groundwater was recharged or replenished mainly by seepage and streamflow along mountain fronts and by groundwater underflow into the area.

Large scale pumping of groundwater began in the area in the 1930s primarily for irrigation of agricultural lands. By the 1950s, a cone of depression had developed southwest of Luke Air Force Base. This cone of depression became more pronounced and the center shifted as greater amounts of groundwater were withdrawn over the years. From 1923 to 1977, groundwater levels declined in the western Salt River Valley by up to 350 feet. Since the 1980s, regional groundwater levels have generally stabilized, and even rebounded in some cases. However, overall regional groundwater declines of up to 300 feet still are prevalent (Hammett and Herther 1995; Schumann and O'Day 1995).

The water levels in wells in the vicinity of White Tanks FRS No. 3 have generally declined since the 1940s. The greatest declines occurred from the 1940s through the mid-1970s. One well owned by Maricopa County Municipal Water Conservation District No. 1 and designated by ADWR as B-02-02-04 DCB is located near the Beardsley Canal and the northern end of White Tanks FRS No. 3. The groundwater levels fell by nearly 140 feet between 1946 and 1971. Since 1971, the groundwater levels have continued to decline but at a significantly lower rate compared to the pre-1971 conditions. Between 1971 and 2001, the groundwater levels in this well declined by approximately 30 feet.

6.3 HISTORIC SUBSIDENCE

Land subsidence is known to occur in alluvium filled valleys of Arizona where agricultural activities and urban development have caused substantial over-drafting or removal of groundwater from thick basin aquifers. The magnitude of subsidence is directly related to the subsurface geology, the thickness, and compressibility of the alluvial sediments deposited in the valleys, and the net groundwater decline. According to Bouwer (1977), land subsidence rate range from about one- hundredth to one-half feet per 10-foot drop in groundwater level, depending on the thickness and compressibility of the basin fill sediments.

White Tanks FRS No. 3 is located in an area of known ground subsidence. The subsidence is a response to groundwater withdrawal and corresponding consolidation of the basin alluvial fill. The Luke Air Force Base area, located approximately 5 miles east of the dam, recorded a cumulative subsidence of nearly 19 feet by 1996 (Schumann and O'Day 1995).

GCI reviewed data for a Benchmark (BM) H265 located on the Beardsley Canal at Glendale Avenue. The elevation of BM H265 as surveyed in 2001 was nearly 4 feet lower than the original elevation in 1948. Over the years, however, the rate of subsidence has decreased substantially. Periodically, the District surveys monuments along the crest and downstream toe of the White Tanks FRS No. 3 embankment. Approximately one foot of subsidence has been recorded at the southern end of the dam (relative to the design dam crest elevation). The maximum subsidence of approximately 4.5 feet has been recorded at the northern end of the embankment.

6.4 ESTIMATED GROUNDWATER WITHDRAWAL

Maricopa Association of Governments (MAG) has prepared population growth projections for the County that only go through year 2030. Using this data, GCI estimates that in the vicinity of White Tanks FRS No. 3, the total population is expected to grow to 143,817 through 2030, which equates to about 54,280 housing units. Using data provided by ADWR, GCI estimates that the residential water demand ranges from 1.206 acre-feet per year for 2000 to 12,238 acre-feet per year through 2030. Non-residential water demand ranges from 1,450 acre-feet per year for year 2000 to 10,772 acre-feet per year through 2030. The combined values equate to a total estimated water demand of about 23,010 acre-feet per year. It is anticipated that a significant portion of this future demand will be met through groundwater extraction.

Data from two groundwater modeling studies were used to estimate future groundwater declines in the vicinity of White Tanks FRS No. 3:

- Modeling by ADWR suggests that the decline in groundwater levels between 1983 and 2025 may range from 50 to 100 feet, corresponding to a rate of decline of approximately 1.2 feet per year to 2.4 feet per year, respectively. Assuming that the rates of decline remain unchanged, it is estimated that over the 100-year design life of the dam, groundwater levels at the dam could decline by 120 to 240 feet.
- Groundwater drawdown projections associated with a major land planning study for the development of approximately 2,000 acres parallel to the Beardsley Canal was recently conducted by Fluid Solutions of Phoenix, Arizona. The results of the Fluid Solution's drawdown study suggest the water demand for developments in the vicinity of White Tanks FRS No. 3 could cause a lowering of the water table of about 375 feet over the next 100 years.

6.5 GCI'S ESTIMATE OF FUTURE SUBSIDENCE

Several factors can affect future subsidence. These factors include, but may not be limited to, the following:

- On-going residual subsidence.
- Continuing groundwater use at present rates.
- Reduction in groundwater use.
- Future increases in groundwater use.
- Location of pumping centers.
- Water use practices.
- Increase or decrease in available surface water.
- Increase or decrease in groundwater level.
- Aquifer compressibility.
- Saturated thickness of aquifer.
- Depth to bedrock or subsurface structural feature.

Because of the wide variety of complex and interrelated factors that can affect subsidence predictions, the selection of appropriate prediction methods must consider simplifying assumptions to reduce complexities. At White Tanks FRS No. 3, GCI examined two cases:

1) Predicted subsidence with present-day groundwater conditions; and

2) Predicted subsidence with additional groundwater withdrawal.

Using this groundwater data, GCI provided the following estimates of subsidence for BM H265:

Groundwater Condition	Predicted Subsidence (feet)
Current groundwater trends continue	1.67 to 2.98
Additional drawdown using ADWR data	2.28 to 4.56
Additional drawdown using MWD data	7.17

The Historical and Predictive Subsidence Assessment prepared by GCI is provided in Appendix B.

6.6 DISTRICT'S SUBSIDENCE EVALUATION

[Note to Reviewers: The District has independently evaluated the potential for future subsidence at White Tanks FRS No. 3. Results will be summarized in the 100 Percent Submittal.]

6.7 URS' MODIFIED APPROACH

URS completed an independent settlement analysis for the White Tanks area using the classical one-dimensional consolidation approach. The intent of our independent analysis was to help resolve the differences between the earlier studies completed by Mr. Dennis Duffy on behalf of the District and Mr. Ken Euge of GCI. The major difference between our approach and the approach adopted in the previous analyses is that we have accounted for the transition between recompression and virgin compression using classical Cc and Cr values to achieve a much better match between the observed settlement values and our calculated trend. We recognize that the actual subsidence process is made inherently complex by the geology of the field, and the uncertain depth of pressure change and compressing strata. In the absence of any site-specific consolidation parameters, we have utilized the best current understanding of the key geologic parameters for our analyses.

Generally, the analysis methodology adopted by Dr. Duffy and that adopted by Mr. Euge are very similar except that Dr. Duffy has utilized an averaging scheme to reduce his final results. At this time we do not understand the justification behind this averaging scheme.

Based on several spot checks performed by us, we did not find any calculation errors in the previous work (spreadsheets). However, we do not totally agree with the way the previous authors have applied the consolidation theory in solving this particular problem. In particular, two of our major comments are as follows:

1) The previous analyses have utilized two straight lines to fit through the void ratio versus effective stress data taken from various sources when plotted on a linear scale. In our opinion, this approach is quite reasonable as long as the void ratio versus effective stress data is plotted on a semi-log scale and not on a linear scale. When the void ratio vs. effective stress data is plotted on a semi-log scale, as suggested by the classical consolidation theory, two straight lines can be drawn through the data to separate the recompression and virgin compression portions of the settlement curve. However, when two straight lines are drawn to fit the void ratio vs. effective stress data drawn on a linear scale, it forces the Cc value to change across the pressure range even for the same soil. This concept of varying the Cc value across a pressure range for the same soil is not consistent with the classical consolidation theory. Another problem associated with plotting the void ratio vs. effective stress on a linear scale is that it is extremely difficult, if not impossible, to identify the recompression portion of the settlement curve. Our second comment, as discussed below, is directly related to this issue.

2) Because the previous analyses have utilized the void ratio vs. effective stress relationships developed from the linear plots, they have inadvertently ignored the initial recompression portion of the settlement curve. As a result of this, the settlement trends calculated by the previous analyses are generally a depiction of the virgin compression behavior of the settlement and thus, do not match the observed data.

6.7.1 ONE-DIMENSIONAL CONSOLIDATION APPROACH

The universally accepted one-dimensional consolidation theory developed by Karl Terzaghi (Terzaghi, ~1939) models volume change caused by changes in effective stress in discrete layers. The magnitude of volume change in each layer is calculated based on the layer's initial void ratio, coefficients of compressibility (Cc) and recompressibility (Cr), the over-consolidation ratio (OCR), and the magnitude of changes in effective stress. The time-rate of settlement is governed by the coefficient of consolidation (Cv) of the compressible deposits and the drainage path. These characteristics are defined for each compressible layer and volume change computed. The volume changes in each layer are then summed to compute settlement.

Based on the above approach, we performed consolidation analysis for the North end of the dam using the computer program Consol 3.0: A Computer Program for 1-D Consolidation Analysis of Layered Soil, developed at Virginia Polytechnic Institute and State University by J. Michael Duncan et al. To be consistent with the previous analyses, we assumed the depth of the compressible soil equal to 1,100 feet below existing ground. Because of the sandy/silty nature of the soils, we assumed that any drop in the water level would translate into 100% consolidation settlement in relatively short period of time (large Cv value).

In the absence of any site-specific consolidation data, we set up our model using assumed material properties with an intent to match and back predict the historic data. By incrementally adjusting the key input parameters, mainly OCR, Cr, and Cc, we were able to achieve a reasonably good match between the observed values and our calculated settlement trend. We calibrated two such models: one with two sub-layers and the other with four sub-layers. Both of these calibrated models were then run to predict future settlements with anticipated drop in groundwater. Results from these simulations are presented on Figure 1 in Appendix B-1 and also summarized in the following table.

Drawdown Below Existing Groundwater Level (feet)	**Projected Subsidence (feet)**
100	2.9 – 4.3
200	4.4 - 7.4
300	5.1 – 9.5

The purpose of the two curves shown on Figure 1 in Appendix B-1 is not to necessarily show an upper and lower bound of the projected values but to demonstrate the large variation in results between the two assumed model geometries and material properties. To define the projected trend with greater precision and accuracy, the uncertainties in the model input parameters must to be reduced with site-specific soil investigations. The results from the current analytical effort have adequately demonstrated that the future subsidence at the White Tanks area can be effectively evaluated provided the model input parameters can be defined with some reasonable degree of confidence.

6.7.2 ANALYSIS LIMITATIONS

We believe our one-dimensional consolidation analysis provides a simple, yet analytically robust evaluation of observed subsidence behavior. The time-honored methodology has been validated over 75 years of observed geotechnical performance of all types of facilities. In particular, this approach has been used successfully in evaluating subsidence from ground water extraction in

California and subsidence caused by oil reservoir extraction in the North Sea. The method is strengthened by direct utilization of measured geotechnical parameters for the formation.

Because of the excellent match with observed behavior, the 1-D consolidation model should provide a sound basis for predicting future behavior. However, the precision and accuracy of the model would be greatly enhanced if site-specific consolidation data is made available through additional soil investigation efforts especially at greater depths. This could be accomplished by:

- Coring and oedometer testing of layers below 400 ft;
- Installing vibrating wire piezometers and keeping a more comprehensive record of actual ground water conditions.

However, given the large uncertainty in the sub-surface conditions and in predicting the rate and magnitude of future groundwater drawdown, it is our opinion that the cost of additional deep soil investigations is not warranted at this time.

6.8 CONCLUSIONS

The future subsidence predicted by URS and GCI differ significantly from that predicted by the District. Based on our interpretation of results, it is estimated that approximately 3 to 6 feet of subsidence could be experienced at the north end of the structure, with a low to moderate probability of exceedence over the life of the structure through 2100. It is anticipated that the subsidence along the embankment towards the south will decrease similar to historic subsidence patterns noted from previous dam crest surveys. As discussed above, the predicted subsidence estimates could be further improved with additional soil investigation efforts especially at deeper depths.

6.9 DESIGN SIGNIFICANCE

The impacts of subsidence on the crest elevation of the embankment occur of long periods of time and can be monitored through the District's survey program. Therefore, the District has the opportunity to respond to lowered dam crests due to subsidence through construction modifications. Therefore, the District has directed URS to incorporate a subsidence raise of 1.0 foot in the design of the embankment crest elevation above the required elevation determined from reservoir routing. In addition, the design will include a widened crest to allow for a future raise of an additional 1.0 ft along the entire crest length.

Methods used to estimate the adjustment using the historic subsidence data included the following:

- Using the ratio of subsidence occurring between 1990 and 2003 at the dam crest.
- Using the ratio of subsidence occurring between 1990 and 2003 at the dam toe.
- Using the ratio of subsidence occurring between dam construction (i.e., as-built) and surveys taken between 1990 and 2003.

Estimated adjustments to the subsidence raise for each of these methods are presented in Table 6-1, and provide similar results. Selection of design subsidence raise is detailed in Section 12.3.2.3 of this report.

7.0 GEOTECHNICAL CONSIDERATIONS

7.1 PREVIOUS INVESTIGATIONS

Between 1992 and April 2004, eight different geotechnical investigation programs were conducted at the project site of the White Tanks FRS No. 3. Apart from the NRCS program, the locations of the borings, test pits, test trenches, and various seismic survey lines that constituted the remaining geotechnical investigation programs are shown on Figure 7-1. A summary of the various elements of each investigation program are presented on Table 7-1.

7.1.1 SCS Design Investigation

URS researched existing documentation on White Tanks FRS No. 3 at the District, ADWR, and the Phoenix office of the NRCS. No documentation on geotechnical investigations pertaining to the original design of the facility in the 1950s was identified. Thus, it is unclear whether or not geotechnical investigations were performed as part of the original design.

7.1.2 NRCS Geologic Investigation

In the early 1990s, the NRCS performed a geologic investigation at the dam. The objectives of the program were to evaluate the foundation alluvium underlying the embankment, and identify depth intervals for future pressure meter testing (NRCS, 1992).

As part of the NRCS investigation, drilling was performed along the upstream and downstream toes of the embankment. The boreholes were spaced 600 feet apart, and staggered. Borehole locations were sometimes adjusted in order to investigate specific features (washes, for example) along the alignment. Standard Penetration Testing (SPT) and split spoon sampling were conducted in the boreholes. The soils encountered during the field investigation were visually examined and logged.

7.1.3 Dames & Moore Investigations

In 1998, the District retained Dames & Moore (now URS) to design rehabilitation measures for White Tanks FRS No. 3. Multiple geotechnical investigations were performed during various phases of the project. Investigative activities along with results of the exploration were discussed in detail in a Geotechnical Data Report prepared by URS (2001) and are summarized below:

- **Dam Modification Investigations:** A total of 22 hollow stem auger borings and 9 test pits were advanced along and in close proximity to the embankment between October and December 1998. The drilling was performed using a truck-mounted Mobil B-50 rig. SPT and split spoon sampling were performed at regular intervals in the borings using 3 inch diameter Dames & Moore Type U sleeves. The borings were grouted upon completion of the drilling and sampling activities. The test pits were backfilled with soil. Selected samples collected during the field investigation were forwarded to a soils laboratory for analyses.

- **Basins Alternatives Investigation:** The geotechnical field investigation program for the Basin Alternatives study included six borings, three test pits, and six refraction seismic survey lines. The six borings were drilled using a CME 75 truck-mounted drill rig equipped with hollow stem augers. The borings were backfilled with soil cuttings after drilling and sampling activities were completed.

  Six refraction seismic surveys were performed at the site. The field data was collected by Bird Seismic Services Inc. and processed and interpreted by Hasbrouck Geophysics Inc. The overall objective of the survey was to evaluate ease of excavation or ripability in the project area. The refraction seismic survey was performed using a 24-channel Bison Spectra signal-enhancement seismograph, Sensor Model SM-11-30Hz geophones, and a 16-pound sledgehammer source.

- **Interim Dam Safety Investigation:** The geotechnical investigation for the Interim Dam Safety project consisted of three test pits excavated at the emergency spillway. The test pits were excavated with a medium-sized backhoe under the supervision of a field engineer from URS. The test pits were excavated to evaluate and sample the soils at the emergency spillway. Logs were not prepared for the three test pits. The laboratory testing program during this phase of the project was limited to sieve analyses and Atterberg limits tests on selected samples collected during the field investigation.

- **Existing Filter Investigation:** Three exploratory borings were drilled on the crest of the dam on November 1, 1999 using a CME 75 with a 3 ¾-inch hollow stem auger. The borings were located at Stations 57+30, 58+00, and 59+00 and were drilled to depths of 30 feet. A test pit was excavated using a backhoe on the crest of the dam on March 31, 2000 to provide additional insight regarding the construction of the existing filter at this location. The test pit was located at approximately Station 58+90. The approximate dimensions of this pit were 6 feet wide, 8 feet long, and 5.5 feet deep. Mechanical sieve tests were performed on selected samples to obtain grain-size distributions. Four samples from the test pit and four samples from the borings were tested.

- **Crack Investigation:** URS performed a field investigation on March 31, 2000 to determine the lateral and vertical extent of transverse cracks observed during previous investigations. A test pit was excavated on the upstream side of the dam at Station 59+00. URS engineers directed the fieldwork. A mechanical sieve test was performed on the sample taken from the test pit.

7.1.4 AMEC Preliminary Investigations

In late 2003, the District retained AMEC to perform preliminary geotechnical investigations at White Tanks FRS No. 3. These investigations were largely focused on a new dam alignment to the south of the existing embankment. However, some of the investigative activities performed by AMEC were in close proximity of the existing dam. Details of this investigation are provided in AMEC's Preliminary Geotechnical Investigation Report (2004), and are summarized below:

- **Review of Existing Data:** AMEC compiled and reviewed data from previous investigations at White Tanks FRS No. 3. This review covered reports prepared by the Fugro (1979), the SCS (1982), NRCS (1992), FCDMC (1992), Dames & Moore (1998), and URS (2001). In addition, published geological, hydrological, and geophysical data was also reviewed.
- **Interferometric Synthetic Aperture Radar Data:** Upon request, ADWR provided AMEC with copies of four interferograms of the Salt River Valley. AMEC utilized these interferograms to characterize the distribution and rate of ground subsidence in the study area.
- **Relative Gravity Survey:** ADWR and AMEC jointly conducted a relative gravity survey to support the characterization of the subsurface geometry and help identify potential earth fissure hazard zones. The survey consisted of 128 gravity stations, and was completed using a Scintrex CG-3M gravimeter.
- **Resistivity Soundings:** AMEC completed five deep resistivity soundings using an Advanced Geosciences Inc. Sting R1 resistivity meter with a four point Wenner array configuration. Two layer interpretations, typically for a shallow and a deep interface, and when appropriate, an intermediate interface, were performed.
- **Analysis of Low-Sun Angle Aerial Photography:** AMEC acquired and analyzed specialized low-sun angle aerial photography. The imagery was evaluated for the purpose of identifying features indicative of the presence of earth fissures.
- **Ground Reconnaissance and Geological Mapping:** After completion of interpretation of the interferograms and the low-sun angle imagery, AMEC visited potential lineaments

on the ground. The alignment of some features were modified (or in some cases deleted) based on the ground reconnaissance.

- **Seismic Refraction Profiling:** AMEC performed twenty seismic refraction surveys to identify the presence of absence of potential fissures in the study area, and to investigate the geotechnical properties of the shallow soil profile. The seismic traces were inspected for a sudden decrease in signal amplitude, and/or an increase in arrival time. Both features were used to detect the potential presence of soil discontinuities.

- **Deep Shear Wave Profiling:** AMEC completed five deep vertical s-wave profiles using the refraction microtremor (ReMi) method. A Geometrics S-12 twelve channel signal enhancement seismograph with a 240-meter cable and 4.5 Hz vertical geophones were used.

- **Test Pit Investigation:** AMEC excavated twenty-two backhoe test pits using a CAT 446B Turbo and a John Deere 710D. The soils encountered were visually examined and continuously logged. The test pits were backfilled with soil cuttings.

- **Exploratory Drilling:** AMEC drilled a total of six hollow stem auger boring along, in the vicinity of, and downstream from the existing embankment. The drilling was performed using a CME 75 truck-mounted drill rig. SPT, split-spoon sampling, and CME continuous sampling were performed in the borings. The borings through the embankment were backfilled with grout while the remainder of the borings were backfilled with soil cuttings.

- **Test Trenching Program:** AMEC excavated two trenches in the vicinity of the existing dam embankment. The alluvial deposits exposed on the walls and upper benches of each excavation were characterized in regards to the geological properties. The test trenches were backfilled with soil cuttings.

7.1.5 URS Dam Rehabilitation Project Investigations

URS performed investigations at White Tanks FRS No. 3 in support of rehabilitation design for the dam and its appurtenant facilities. A detailed presentation of work conducted and results of these investigations can be found in a companion document titled *White Tanks FRS No. 3 Geotechnical Report* (URS 2004). Key aspects of the work performed are summarized as follows:

- **Review of Existing Information:** URS reviewed and summarized geotechnical data collected during previous investigations at White Tanks FRS No. 3. Key documents that were reviewed included the Geotechnical Data Report prepared by Dames & Moore

(2001), and the Preliminary Geotechnical Investigation Report prepared by AMEC (2004). In addition, URS reviewed other applicable published articles and reports on the geologic setting and the geotechnical conditions at White Tanks FRS No. 3.

- **Exploratory Drilling:** URS supervised the drilling of 24 test holes along the upstream toe of the existing embankment (B-1 through B-16) and in the emergency spillway discharge channel (B-17 through B-24) in April 2004. Nine of the 24 test holes were drilled in the FRZ (B-1 through B-9). Test hole depths ranged from 10 feet to 100 feet bgs. Selected samples (split spoon, ring, core, and Shelby tube) were collected for laboratory testing to estimate index properties, strength, compressibility, permeability, and erodibility of Holocene and Pleistocene soils. Laboratory testing included a suite of standard geotechnical tests as well as specialized erosion tests. The latter includes testing of extruded Shelby tube specimens using the Erosion Function Apparatus developed by Professor Jean-Louis Briaud of Texas A&M University; and the Hole Erosion Test procedure developed by the United States Bureau of Reclamation (USBR), with testing at the USBR laboratory in Denver, Colorado. The laboratory test program and test results are described in detail in the companion Geotechnical report.
- **Test Trenches and Vertical Jet Erosion Tests:** In July 2004, Geological Consultants Incorporated (GCI), a subconsultant of URS, supervised the excavation of 5 test trenches at the following locations: near the right abutment (TT-1), along the upstream toe of the existing embankment in the FRZ (TT-2 and TT-3), and near the right abutment (TT-4 and TT-5). Each trench was excavated to two depth levels (5 and 10 feet bgs), and Vertical Jet Erosion (VJT) tests were performed at both depth levels (9 total VJT tests). VJT equipment was provided by Engineering & Hydrosystems, Incorporated. A scraper, water truck and equipment operator were provided by the District. Encountered soils were field-classified and logged.
- **Test Pit Investigation:** In April 2004, URS and Terracon (subconsultant to URS) supervised the excavation of 33 test pits at the following locations: Borrow Area B (TP-1 through TP-7), Borrow Area A (TP-8 through TP-20), along the upstream toe of the existing embankment (TP-21 through TP-30), and along the right edge of the emergency spillway (TP-31 through TP-33). The primary purpose of test pit excavation was to evaluate the suitability of on-site surficial soils as borrow material for construction of the common fill embankment, soil cement embankment, and soil cement-bentonite cutoff walls. Test pits were excavated to a depth of 10 feet. Five of the 33 test pits were excavated in the FRZ (TP-24 through TP-28). An excavator and equipment operator were

provided by the District. Encountered soils were field classified and logged, and bulk samples were collected for laboratory testing.

- **Seismic Refraction Survey:** In April 2004, GCI performed a seismic refraction survey geophysical investigation to measure seismic velocities through Holocene and Pleistocene soils, and to identify any shallow bedrock and any anomalous subsurface features such as fissures. The survey consisted of twenty-five seismic lines located along the upstream toe of the existing embankment and along the control section of the emergency spillway. Locations of the seismic lines in plan view are shown on Figure 7-1, and the seismic velocity zones in section view are shown on Figure 7-2. GCI's Seismic Refraction Geophysical Survey (GCI 2004) report are summarized and included as an appendix in the companion Geotechnical Report.

7.2 SOIL CONDITIONS

Geotechnical conditions of the existing earth embankment and subsurface soils are summarized in this section. A detailed discussion of soil conditions is presented in the 2004 URS Geotechnical Data Report.

7.2.1 Existing Embankment Soils

Embankment soils were not included in the scope of URS' 2004 geotechnical investigations, but were investigated by Dames & Moore from 1998 through 2000 as part of the Dam Modification Investigation, Existing Filter Investigation, and Crack Investigation (Dames & Moore, 2001). Results of those investigations indicate that the embankment soils are predominantly clayey sands with lesser amounts of sandy clays present. The fines contents of the clayey sands range from 23 to 35 percent, and the PIs range from 6 to 17 percent. The gravel content is as high as 40 percent, but typically less than 10 percent. The sandy clays are of low to medium plasticity, with PIs ranging from 7 to 13, and with fines contents ranging from 53 to 70 percent, but typically less than 60 percent. The gravel content of the fine-grained soils is less than 5 percent.

Laboratory tests were performed as part of the Dam Modification Investigation to evaluate shear strength parameters for the embankment soils. Triaxial tests were performed on two relatively undisturbed samples of embankment soils. These tests were performed under consolidated, undrained conditions with pore pressure measurements. For effective stress conditions, the internal angle of friction ranged from 34 to 37 degrees, and the cohesion ranged from zero (0) to 150 pounds per square foot (psf). For total stress conditions, the internal angle of friction ranged from 22 to 32.5 degrees, and the cohesion ranged from 50 to 220 psf. Permeability of the triaxial specimens ranged from 1.7×10^{-6} to 9.6×10^{-6} cm/sec. These strength and permeability parameters were used to develop input parameters for slope stability and seepage modeling, presented in Section 12.6.

7.2.2 Subsurface Soils

Soil conditions of the foundation, emergency spillway, and flood pool borrow areas (A and B) are summarized in this subsection. A wide array of investigation data forms the basis for characterization of subsurface soil conditions:

- Published information on Holocene and Pleistocene alluvial fan deposits;
- Subsurface exploration data from test holes, test pits, and test trenches;
- Seismic refraction survey data;
- Field erosion test data from Vertical Jet Erosion tests;
- Laboratory geotechnical test data;
- Laboratory Hole Erosion Tests (HET) data; and
- Laboratory Erosion Function Apparatus test data.

Locations of all previous and current test and exploration locations are shown in plan view on Figure 7-1. A graphical representation of embankment foundation soil conditions is shown in section view on Figure 7-2. The focus of the characterization of subsurface soils in the following paragraphs is to establishing representative classifications, index properties, and engineering properties used in geotechnical analyses, modeling, and design.

7.2.2.1 Near-Surface Geology and Hydrogeology

Detailed discussions of site geology and hydrogeology are provided in Section 6 of this report, in the URS 2004 Geotechnical Data Report, and in previous investigation reports. The lateral extent of surficial Holocene and Pleistocene at the site has been mapped (see URS 2004 Geotechnical Data Report). Holocene surficial deposits appear to be present along about two-thirds the length of the embankment alignment. Pleistocene surficial deposits appear to be present along about one-third of the embankment alignment and throughout most of the flood pool and emergency spillway areas.

The vertical extent of Holocene deposits beneath the embankment has received considerable attention during previous investigations, as well as during the 2004 URS investigation. Holocene fine silts and sands have typically been correlated with lower strengths, higher erodibility, and higher collapse potential than Pleistocene soils. AMEC interpreted the surficial Holocene deposits to be up to 12 feet deep (AMEC, 2004). Geoconsultants interpreted Holocene alluvial deposits to roughly correlate with an upper seismic velocity zone (1,200 to 2,100 fps), which generally extends about 10 feet below ground surface within the FRZ, but extends to depths of 20 to 25 feet at several areas within the FRZ and throughout most of the southern portion of the embankment. It should be noted that during our investigation, identification of unsuitable foundation materials was not limited solely to delineation of Holocene deposits, but also took into consideration material classifications, index properties, and engineering properties.

Higher permeability "Paleo" stream channels (or paleochannels), consisting of less-cemented and less-compacted sediments than the surrounding material, may be present beneath the dam. Geoconsultants identified a potential paleochannel in the FRZ between Stations 31+60 and 33+90 (Geoconsultants, 2004).

Unsaturated soils extend to great depths below the site, as groundwater is approximately 300 feet below ground surface. Soils are typically dry to slightly moist, with an average moisture content of about 4 percent.

7.2.2.2 Material Classification

A tally of all Unified Soil Classification System (USCS) classifications for 135 total samples collected during previous investigations, as well as during the 2004 URS investigation, was performed. Classifications have been divided into four groups, based on approximate material similarities: [GP, GP-GM, SP-SM, SW-SM], [SM, SC, SC-SM], [SM/ML, ML, CL, CL-ML], and [CH]. The tally is presented in the following table:

Summary of USCS Material Classifications

USCS Classification	Tally of Shallow Samples (0 - 10 feet)	Tally of Deeper Samples (10 + feet)	Totals	Percent of Total
GP	0	1	1	0.75
GP-GM	0	1	1	0.75
SP-SM	2	6	8	5.9
SW-SM	2	4	6	4.4
Subtotal	**4**	**12**	**16**	**11.9**
SM	23	17	40	29.6
SC	13	11	24	17.7
SC-SM	4	5	9	6.6
Subtotal	**40**	**33**	**73**	**54.1**
SM/ML	0	2	2	1.5
ML	9	6	15	11.1
CL	13	9	22	16.3
CL-ML	6	0	6	4.4
Subtotal	**28**	**17**	**45**	**33.3**
CH	0	1	1	0.75
Subtotal	**0**	**1**	**1**	**0.75**
Grand Total	**72**	**63**	**135**	**100**

The predominant material classification group is the [SM, SC, SC-SM] group, which comprises 54 percent of all sample classifications. The [SM/ML, ML, CL, CL-ML] classification group is the next largest, comprising 33 percent of the sample classifications. The [GP, GP-GM, SP-SM, SW-SM] group comprises 12 percent of the material classifications. Lastly, only one sample was classified as CH, illustrating the paucity of high plasticity material at the site. It appears that there is a higher concentration of fine grain material at shallow (0-10 ft) depths, based on the USCS classifications. If all material on the site were a homogeneous blend, the material would most closely classify as SM, or as an AASTHO A-4 soil. Material classifications were used, in part, to develop input parameters for various geotechnical analyses, based on published correlations between engineering properties and USCS classifications.

7.2.2.3 Fines Content and Plasticity

Sieve analyses results for all near-surface (0-10 feet) soil samples obtained during URS' April 2004 investigation, as well as previous investigations, indicate that site-wide fines contents range from 7 to 94 percent, with an average fines content of 44 percent, for all samples tested (76 total). For samples collected at depths 10 feet or deeper, site-wide fines contents range from 3 to 77, with an average fines content of 34, for all samples tested (65 total). Average fines contents do not show significant lateral spatial variation across the site, but the average fines content of near-surface soils is 10 percent higher than the average fines content of samples collected at depths 10 feet or greater. This agrees with the observation that on average, shallow soils (0-10 feet) have higher fines contents than deeper soils based on USCS classifications, as discussed above. The percent fines for borrow area composite samples were 48.7 (Borrow Area A), 62.6 (Borrow Area B), and 40.4 (Embankment borrow area), with an average fines content of 50.6.

The plasticity indices (PI) of near-surface (0-10 feet) soils range from zero (non plastic) to 26, with an average PI of 5.5 percent, for all samples tested (73 total). The PI's of samples collected at depths 10 feet or deeper range from zero to 28, with an average PI of 6.2 percent, for all samples tested (52 total). On average, subsurface soils have low plasticity, and therefore also have a relatively low swell potential.

Fines content data was useful in assessing the suitability of on site materials as construction material for soil-cement, soil-cement-bentonite, and embankment fill. PI data was used to assess the potential for clay ball formation in soil-cement and soil-cement-bentonite mixes, and was also used as an input parameter in the NRCS SITES model of emergency spillway erodibility. These are discussed in separate sections in this report.

7.2.2.4 Strength

In-situ density, blow count values, percent core recovery, cementation, and triaxial test data are indicators of the strength of site soils, and are summarized below. In-situ densities were obtained from moisture-density density tests conducted on relatively undisturbed samples. Moisture-density test data from Dames & Moore and URS investigations are summarized in the following table.

Summary of In-Situ Density

Soil Material	No. of Samples	Dry Density (pcf)	Water Content (%)	Total Density (pcf)
0-10 foot depth				
ML or CL	13	100.5	4.1	104.5
SW-SM	1	119	2.0	121.4
SC or SM	18	106.3	3.3	109.9
Average		**108.6**	**3.1**	**111.9**
10-foot depth and greater				
ML or CL	3	115.5	7.3	123.9
SW-SM	0	----	----	----
SC or SM	2	109.5	3.9	113.5
Average[1]		**113.1**	**5.9**	**119.8**

Notes: 1. An ML sample with a very low dry density of 78.5 pcf was not included in the average.

Shallow depth (0-10 ft) blow count values obtained during our investigation generally ranged from 9 to 17 within the FRZ, generally indicating moderately firm material. In the 10 to 30 foot depth range, blow count values ranged from 15 to 30, generally indicating moderately firm to firm material. In the 30 to 60 foot depth range, blow count values ranged from 30 to 50, generally indicating firm to very firm material. Blow count values obtained outside of the FRZ along the embankment were generally in the same ranges as FRZ blow count values. These borings located outside the FRZ were only advanced to depths up to 20 feet.

Blow count values obtained in the emergency spillway ranged from 6 to 50/5". From 0 to about 7 feet bgs, blow count values were typically less than 10, generally indicating soft material. From 7 to 11 feet, blow counts ranged from 16 to 50/5", generally indicating firm to very firm material. URS borings were only advanced to depths of 11 feet in the emergency spillway.

Most of the URS borings within the FRZ were continuously triple-tube cored or continuously sampled with Shelby tubes either for the full extent of the boring, or at depths greater than about 20 feet. Therefore, no blow count values are available for these FRZ boring intervals, and percent core or Shelby tube sample recovery was used as an indication of the strength or consistency of the subsurface material. The degree of cementation was also reviewed, but recovery was considered a better indicator of strength because cementation data appeared inconsistent, and often conflicted with blow count values. Shelby tube sample recovery was generally good in material classified as SM and SC, even in the 30 to 60 foot depth interval, and poor in material with appreciable sands, gravels, or cobbles. Shelby tubes were only advanced in three borings at depths greater than 15 feet. The good recovery in SM and SC material at greater depths indicates that this material is dense/firm enough to be retained in the tube, but not so dense/firm that the Shelby tube cannot be advanced. Correlation between material type and triple-tube (ring) sample recovery was not always consistent. However, poor recovery was often associated with coarse-grained material (i.e., GM, GP-GM, SW-SM, SP-SM), probably indicating the presence of lenses or zones of loose coarse-grained alluvium beneath the embankment. Good recoveries were generally associated with ML, CL, ML-CL, materials, irrespective of the depth. SM and SC material core recovery ranged from poor to good, irrespective of depth.

As part of Dames & Moore's Dam Modification Investigation, a triaxial test was performed on one relatively undisturbed sample of foundation soil at the upstream toe of the existing embankment (Dames & Moore, 2001). The test was performed under consolidated, undrained conditions with pore pressure measurements. For effective stress conditions, the internal angle of friction was 36 degrees, and the cohesion was 120 psf. For total stress conditions, the internal angle of friction was 21 degrees, and the cohesion was 300 psf.

The strength indicators discussed above were used to develop input parameters for various geotechnical analyses, described separately in this report.

*7.2.2.5 **Permeability***

Horizontal lenses of coarse-grained, higher-permeability material interbedded with fine-grained material were observed in test pits and test trenches. Lenses and zones of coarse-grained, higher-permeability material were also observed in soil cores at shallow depths (0 to 10 feet), as well as greater depths (10 to 90 feet).

The average permeability of the triaxial sample of foundation soil described above was 1.1×10^{-5} cm/sec. The sample was taken at a depth of 10 feet. Laboratory permeability tests are more representative of vertical conductivity values, which are typically less than the horizontal conductivity values. Assuming an anisotropic ratio (k_H/k_V) of 10 to account for possible horizontal stratification, the corresponding horizontal permeability value is about 1×10^{-4} cm/sec. This is consistent with published with published permeability values for SM and SC material.

Therefore, the horizontal mass permeability of both Holocene and Pleistocene soils can be assumed to be on the order of magnitude of 1×10^{-4} cm/sec. Lenses of coarse-grained material may actually have higher permeability values, on the order of 10^{-3} cm/sec, but such lenses do not appear to be uniform or continuous across the site.

7.2.2.6 Compressibility

There are three potential modes of foundation soil compression at the site: consolidation, elastic compression, and collapse. Given the great depth to the current water table and the resulting thickness of unsaturated soils, it is judged that there is low potential for soil compression related to consolidation.

Regarding elastic compression, no direct measurements of elastic modulus of foundation soils were made. However, Beckwith and Hansen (1982) have established correlations between elastic modulus, blow count values, and cementation for Holocene and Pleistocene soils. An average elastic modulus of 8 ksi was selected for foundation soils, as discussed in Section 12.6.5 Settlement Analyses. This value corresponds to a moderate elastic soil compressibility.

Eight response-to-wetting or "collapse" tests were performed on relatively undisturbed samples of the Holocene soils at the upstream toe of the existing embankment, as part of our investigation. Additionally, Dames & Moore performed 7 collapse tests on soils obtained from the upstream and downstream toes of the existing embankment (Dames & Moore, 2001). For the 15 samples tested, axial strain (or percent collapse) ranged from about 0.5 percent to 4.8 percent of the sample height, with an average of about 3 percent. This corresponds to a moderate collapse potential. Settlement related to potential collapse is discussed in Section 12.6.5.

7.2.2.7 Erodibility

Field Vertical Jet Erosion Tests (VJT) were performed in test trenches at depths of 5 and 10 feet at five locations along the embankment alignment and near the right and left abutments. Hole Erosion tests (HET) were performed on undisturbed samples at the USBR geotechnical laboratory in Denver, Colorado. Erodibility testing using the Erosion Function Apparatus (EFA) were performed on undisturbed samples at a testing laboratory at Texas A&M University. Test procedures and results are described in the 2004 URS Geotechnical Report. Fissure erosion modeling was performed and documented in the report "White Tanks FRS No. 3 Foundation Fissure Modeling" (Engineering & Hydrosystems, 2004).

Field and laboratory erosion test results indicate that near-surface soils (0 to 10 ft) are moderately- to highly-erodible, soils at mid-level depths (20 to 40 ft) generally are moderately erodible, and soils at greater depths generally have moderately-low to very-low erodibility. However, there are likely local zones of highly-erodible material at depths greater than 20 feet, based on our review of borehole data. Erodible soils at shallow and mid-level depths will be cut off by a soil-cement structure and cutoff walls, which will extend to a depth of 40 feet below grade.

7.3 EARTH FISSURES

7.3.1 Mechanics of earth fissure development

Fissures occur in unconsolidated sediments, typically near the margins of alluvial valleys or near bedrock pediments where groundwater levels have dropped from 200 to 500 feet below ground surface. The main factors relating to the development of an earth fissure are the differential consolidation of unwatered sediment resulting from groundwater withdrawal. The differential consolidation may occur due to shallow bedrock irregularities, or changes in soil lithology.

Fissures are initiated underground when tensile stresses exceed the tensile strength of the ground. The fissures then propagate upwards to intersect the ground surface. The locations of earth fissures are controlled primarily by the configuration of the bedrock surface, variation in basin fill stratigraphy, and other factors. Early signs of earth fissures are small linear en echelon hairline cracks, irregularly spaced but aligned depressions, and large open holes. Other physical features associated with fissures are slump-related escarpments from one inch to a few inches in height, as well as a drainage pattern associated with the fissure that does not conform to the local area drainage pattern.

Field evidence indicates that fissures are exposed after overlying sediments are eroded by surface water runoff from rainfall or irrigation. The surface expressions of the fissures are exaggerated because the initial hairline crack is attached by water to create wide and deep erosional gullies that often have vegetation growing in them. The fissures are commonly perpendicular to natural drainage channels. The length of a fissure at the ground surface varies, typically less than one mile.

7.3.2 Fissure risk zones

The District retained AMEC Earth & Environmental Inc. to evaluate fissure risk at White Tanks FRS No. 3. Details of the study are documented in AMEC's report entitled "Preliminary Geotechnical Investigation Report, White Tanks FRS No. 3". Key findings of the investigation as documented in AMEC's report are summarized below:

- The north end of the dam has, and will probably continue to be, a region of greater subsidence as compared to the south end of the dam. It is more likely that this differential subsidence is a result of greater thickness of fine-grained deposits at the north end of the dam, rather than due to varying thickness of the underlying alluvium.

- AMEC performed a simplified analysis of horizontal strain using a method proposed by Lee and Shen. This analysis indicated that the greatest strain was calculated to occur between Stations 45+00 and 55+00, and a maximum strain of approximately 0.06 percent was reported.
- The general shape of the ground deformation as seen in the interferograms was generally consistent with the orientation and density of photolineaments identified during examination of aerial photographs. However, field inspection of the area by AMEC personnel did not identify earth fissures.
- Seismic refraction techniques and direct observations in trenches excavated in the area of the photolineaments did not detect the presence of earth fissures.

Based on these observations, AMEC identified three zones of fissure risk along the embankment:

- **Zone 1 – Station 30+00 to Station 55+00:** Region where alluvial basin characteristics, the distribution of probable soil discontinuities and past subsidence behavior indicates the presence of conditions favorable for future earth fissure development.
- **Zone 2 - Station 42+00 to Station 52+00:** Region of Zone 1 where the existence of deflation features in the Holocene alluvium, steeper interferometric gradients, an increased density of oriented photolineaments, and/or a significant break in the dam crest settlement profile may indicate a higher probability of earth fissure development.
- **Zone 3 – Remainder of Embankment:** Region of probable low fissure risk, with insignificant differential deformation indicated by the interferometry, where geologic conditions appear to preclude the development of large horizontal strains, and/or where compression is indicated in the subsidence profile.

Based on the AMEC investigation, the District selected a fissure risk zone that covers Zone 1 (Station 30+00 to Station 55+00) as identified by AMEC. It appears that the AMEC assessment did not evaluate the impacts of future groundwater withdrawal.

In **[MONTH]** 2004, the District retained AMEC to perform a fissure risk analysis for the area north of the existing dam. AMEC's scope of work for the northern extension included a review of interferograms, low-sun angle aerial photographs, and limited ground-truthing. Based on this analysis, AMEC identified a low to moderate fissure risk for the northern dam extension. The results of AMEC's study were summarized in **[REPORT TITLE]** (AMEC 2004). **[Note to**

Reviewers: The proper reference to the AMEC evaluation of the North Fissure Risk Zone will be provide with the 100 Percent submittal.]

7.3.3 Failure Modes Related to Earth Fissures

Failure modes related to earth fissures were evaluated in *Preliminary Embankment Rehabilitation Concepts* (URS 2004). The following two failure modes were identified:

1. **Embankment Construction:** Homogenous earth embankment. **Failure Mode:** Water flowing along a fissure across the embankment foundation erodes the Holocene (and possibly a portion of the embankment) soils. This erosion of the foundation and/or embankment soils causes a void to form under the upstream portion of the embankment. The embankment is unable to span this void, resulting in settlement and severe cracking of the upstream portion of the embankment.

2. **Embankment Construction:** Embankment constructed with materials capable of spanning a void formed by erosion of the Holocene soils. **Failure Mode:** Erosion of the Holocene soils progresses under the entire width of the embankment (upstream to downstream), forming a tunnel. The tunnel daylights at the downstream toe of the embankment, leading to an uncontrolled release of the reservoir.

7.4 TRANSVERSE CRACKING

An inspection by Fugro (1979) identified transverse cracking of the embankment. Based on this study, the embankment was "zoned" based on the degree of cracking. However, during construction of the center filter, it was discovered that the degree of cracking observed in the trench exceeded the surface observations during the Phase I Inspection. Therefore, the field observations by NRCS personnel (1981) during construction of the center filter have been summarized below:

- The NRCS mapped nearly 400 transverse cracks through the embankment.
- The width of the transverse cracks mapped by the NRCS ranged from 0.03125 inches (hairline) to 3 inches.
- The average crack width is estimated to be 0.13 inches.
- 95 percent of all cracks mapped by the NRCS were less than 0.5 inches in width.

Several agencies including the NRCS, the U.S. Army Corps of Engineers (COE), and various consultants on behalf of the District have investigated the phenomenon of transverse cracking of homogenous flood control dams in Arizona. Some of the key potential causes for transverse cracking as identified in studies completed by the above-mentioned agencies are summarized below:

- In the late 1970s, the NRCS assembled a team to study and report on transverse cracking of homogenous embankment flood control dams in Arizona. The report by the study team (NRCS 1978) identified desiccation of the embankment soils as the primary cause for transverse cracking of the embankment. Secondary causes identified by the study team included differential settlement of the foundation soils, regional subsidence associated with groundwater withdrawal, variability within the soil type and compaction within the embankment, and stresses induced by tremors and earthquakes.
- The NRCS study team (1978) also identified foundation settlement as a secondary cause of embankment cracking, but did not specifically identify collapsible soils as a possible cause of embankment cracking. Dams designed and constructed by the NRCS in Arizona prior to the 1978 NRCS crack study (For example, White Tanks FRS No. 3 and 4, constructed in the 1950s) had limited foundation treatment. There was no attempt to identify, evaluate, or treat potentially collapsible soils within the embankment footprint. Dam designs by the NRCS post-1978 appear to address (to varying degrees) potentially collapsible foundation soils under dam embankments.
- In the early 1970s, the Los Angles of the COE initiated an investigative program at McMicken Dam to present information pertinent of cracking of the embankment, and to recommend remedial treatment (1973). The study concluded that transverse cracking of the McMicken Dam embankment was a result of regional subsidence related to groundwater withdrawal. The COE (1973) further concluded that since the embankment soils were compacted at moisture contents below the shrinkage limits of the soils, it was unlikely that cracking was due to desiccation and shrinkage.
- In the early 1980s, Sergent, Hauskins & Beckwith Consulting Geotechnical Engineers Inc. (SHB) performed a comprehensive geotechnical investigation at McMicken Dam. SHB's (1982) report concluded that the transverse cracking of the embankment was primarily due to collapsible soils underlying the embankment. The report further stated that since most of the embankment soils were compacted at moisture contents below the shrinkage limits of the soils, it was unlikely that desiccation was a major factor contributing to the cracking of the embankment.

The exact cause of transverse cracking at White Tanks FRS No. 3 is not currently known. Based on available geotechnical data, it appears that transverse cracking is primarily due to desiccation and shrinkage of the embankment soils with time. The collapse of Holocene soils underlying the embankment may have contributed to the transverse cracking, albeit to a lesser degree than desiccation.

7.4.1 Cause(s) of Transverse Cracking

The exact cause of transverse cracking at White Tanks FRS No. 3 is not currently known. Based on available geotechnical data, it appears that transverse cracking is primarily due to desiccation and shrinkage of the embankment soils with time. The collapse of Holocene soils underlying the embankment may have contributed to the transverse cracking, albeit to a lesser degree than desiccation.

7.4.2 Failure Modes Related to Transverse Cracking

Failure modes related to transverse cracking were evaluated in *Preliminary Embankment Rehabilitation Concepts* (URS 2004). The following five failure modes were identified:

1. **Embankment Construction:** Homogenous earth embankment with no central filter. **Failure Mode:** Water flows along a transverse crack through the embankment. Continuous seepage erosion causes enlargement of the crack leading to an uncontrolled release of the reservoir.

2. **Embankment Construction:** Homogenous earth embankment with a partially penetrating center filter. **Failure Mode:** The filter functions as intended and protects a portion of the embankment against continuous seepage erosion. However, flow along the crack through the unprotected section of the embankment allows full development of the failure mode.

3. **Embankment Construction:** Homogenous earth embankment with a full-depth center filter that functions as designed and protects the entire embankment against continuous seepage piping. A cutoff trench at the upstream toe of the embankment does not extend into the Pleistocene soils. **Failure Mode:** Seepage along a transverse crack at the embankment-foundation interface causes erosion of the underlying Holocene soils, leading to failure of the dam.

4. **Embankment Construction:** Homogenous earth embankment with a full-depth center filter that functions as designed and protects the entire embankment against continuous seepage piping. A cutoff trench at the upstream toe of the embankment extends through the Holocene soils and into the Pleistocene soils. **Failure Mode:** Flow enters the

transverse crack at some height above the embankment-foundation interface along the upstream face of the dam.

5. **Embankment Construction:** Homogenous earth embankment with a full-depth granular filter along the centerline of the embankment. A cutoff trench at the upstream toe of the embankment extends through the Holocene soils and into the Pleistocene soils. **Failure Mode:** A defect in the center filter allows the transverse crack to extend through the entire width of the embankment. Potential causes for defects in granular filters include segregation, open cracks supported by cementation or re-cementation of the granular filter, and arching of the filter sand due to settlement of the sand after wetting.

7.5 SOIL CEMENT MIX DESIGN

7.5.1 Preliminary Assessment of Use of Soil Cement at White Tanks FRS No. 3

URS performed a preliminary assessment of the use of soil-cement for construction of the structural core of the embankment within the FRZ. The results of this preliminary assessment are documented in a technical memorandum attached in Appendix J titled "Preliminary Assessment of Soil Cement – White Tanks FRS No. 3 Remediation." Topics discussed in that technical memorandum include:

- Historic and current application of soil cement in dams;
- Guidelines and criteria for selection of suitable soil-cement mix materials, and a discussion of potentially suitable soil material at the project site;
- Engineering properties and standard tests for soil-cement;
- Performance criteria; and
- Summary of soil cement mix design testing conducted for the White Tanks FRS No. 3.

Based on a review of the historic and current industry uses of soil cement in dams, it was concluded that soil cement was considered feasible for use at White Tanks FRS. No. 3. Key soil-cement mix material information, guidelines, or criteria from the preliminary assessment are summarized in the following paragraphs.

7.5.1.1 Soil Fines Content

The amount of soil fines in soil-cement mixes used in actual historic projects has ranged from 4 to 38 percent. The USBR and USACE recommend fines contents ranging from 15 to 25 percent, and 5 to 35 percent, respectively. Maricopa County's soil cement fines content criteria for bank protection is 0 to 8 percent.

7.5.1.2 Soil Plasticity

Plasticity of soils used in soil cement is usually limited to a plasticity index (PI) of 8 or less. Clay balls tend to form when the PI is greater than 8. Maricopa County's criteria for maximum plasticity for soil cement use in bank protection is a PI of 25.

7.5.1.3 Cement Content

Cement requirements vary depending on the severity of climactic exposure, the desired properties of the soil, and type of soils. Cement contents usually range from 4 to 16 percent of the dry weight of soil. Once a cement content has been established based on strength and durability tests, and additional 2 percent of cement is generally specified for water control projects to account for the more severe effects of water exposure and field variations in the soil and mixing process.

7.5.1.4 Compressive Strength

The main engineering property used to evaluate performance of soil-cement mixes for the White Tanks FRS is compressive strength. Erosion resistance is another important property, but is essentially related to compressive strength. Durability was considered to be of lesser importance because the White Tanks FRS soil cement core will be blanketed with a thick layer of common fill on both sides, and will not be subjected to repeated cycles of freeze/thaw and wetting/drying, as is the case with water control structures with permanent pools, located in harsh climates. The laboratory tests used to measure durability – freeze/thaw and wet/dry tests – were therefore judged to not be representative of the climatic exposure that the FRS soil-cement core will experience, and consequently were not performed. Maricopa County's requirements for minimum 7-day compressive strengths for soil-cement banks and grade-control structures (e.g. channel bottoms and spillway crests) are 750 psi and 1,000 psi, respectively.

7.5.2 Summary of Mix Design Testing Program

Following the preliminary soil cement assessment, a mix design testing program was performed to evaluate the performance of soil-cement mixes prepared with on-site soils and a range of cement contents. Nine trial mixes were prepared using soil from three potential borrow areas and using three cement contents. The three potential borrow areas are shown on Figure 7-1 and include Borrow Area A (south borrow area), Borrow Area B (north borrow area), and the upstream toe of the existing embankment. Cement contents were 3, 6, and 9 percent by dry weight. Standard Proctor tests, grain size analyses, and Atterburg limit tests were performed on composite soil samples from the three borrow areas. Standard Proctor tests were also performed on each of the 9 soil-cement trial mixes. Test cylinders were subjected to unconfined compressive tests after 3, 7, 14, 28, and approximately 90 days of curing. A more detailed description of the mix testing program is provided in the companion Geotechnical Report.

7.5.3 Mix Design Test Results

Mix design test results are summarized below and are presented in detail in the companion Geotechnical Report. Of the three potential borrow sources, composite soil from the embankment borrow source (EBS) produced mixes that had the highest compressive strengths. The EBS composite sample also had the lowest fines content (40 percent). Composite samples from Borrow Areas A and B had fines contents of 49 and 63 percent, respectively. Composite samples from all three borrow sources produced mixes that achieved Maricopa County's 7-day compressive strength criteria of 750 psi. However, the cement content required to meet this criteria differed, depending on the composite soil borrow source: 5 percent cement content for the embankment borrow source, 6 percent for Borrow Area B, and 7.5 percent for Borrow Area A. A minimum cement content of 7.5 percent would therefore provide maximum flexibility during construction, so that soil from any of the three potential borrow sources could be used and still meet the 750 psi criteria. However, to account for field variations in the soil and mixing process, the minimum cement content should be increased to 9 percent.

Compressive strength results showed substantial increases at test time intervals of 14, 28 and approximately 90 days. Ninety-day compressive strengths for 9-percent cement mixes ranged from 1360 psi to 1610 psi (approximately double the 7-day strengths). Although the average on-site soil fines content is in the 45 to 50 percent range, and exceeds the industry-standard fines-content range of 4 to 38 percent and Maricopa County's criteria for 0 to 8 percent fines, use of on-site soils and a 9-percent cement content is judged to be acceptable, based on achievement of compressive strength criteria. Additionally, an average PI of 6.3, obtained by averaging PI's of 26 shallow soil samples (0 to 10 feet bgs), meets Maricopa County's maximum PI criteria of 25.

Athough no significant mixing problems occurred in the controlled laboratory conditions, a greater mixing effort may be required during field mixing due to the high fines content and variability of the on-site soils. Clay ball formation during field mixing is possible, and may require greater mixing energy to disperse the clay balls. However, the possible formation of clay balls in the field should not have a significant impact on workability and performance, based on the results of the mix design testing program.

7.6 SOIL CEMENT-BENTONITE MIX DESIGN

Cutoff walls are incorporated into the South FRZ Embankment with the design objective of controlling erosive subsurface flows through potential earth fissures beneath the dam within the FRZ. The soil cement-bentonite (SCB) cutoff wall alternative was selected as the preferred alternative because of its estimated cost, constructability, and performance characteristics (See Section 12.0). Performance objectives of the constructed SCB cutoff walls include seepage/fissure flow control, erosion resistance, and cracking resistance. The following sections provide details of the soil cement-bentonite mix design.

7.6.1 Summary of SCB Design Mix Testing Program

Design mix testing was performed to develop an SCB mix that would meet performance objectives by achieving a balance of impermeability, strength, and ductility. Initial trial mix proportions were developed based on our experience with previous design mixes that have achieved similar performance objectives. Nine SCB trial mixes were prepared consisting of three different soil materials and three cement contents. The first soil material is a locally available commercial aggregate termed "dirty" MAG AB with a fines content of about 9 percent. The second soil material is an on-site composited soil with a fines content of about 30 percent. The third soil material is an on-site composited soil with a fines content of about 45 percent. The composite soil materials were obtained from test pits excavated in Borrow Area A, the embankment borrow source, and in the discharge channel of the emergency spillway. The use of higher fines content aggregates (30 and 45 percent fines) is a departure from standard practice, but was included in the trial mix testing program, based on economic considerations, to evaluate if on-site material will yield mixes that meet the performance criteria. Cement contents were 6, 8 and 10 percent of the dry soil weight.

Specific performance and mixing criteria selected for the SCB material include:

- Unconfined Compressive Strength: 200 to 500 psi (28 days)
- Minimum Permeability: 10^{-6} cm/sec
- Slump: 7 to 9 inches
- Water-Bentonite Viscosity: In the range of 38 to 42 seconds
- Workability: Mixes with high fines content soils (30 and 45 percent fines) should be observed for clay ball formation. Clay ball formation may be considered prohibitive depending on the size of the clay balls and if excessive mixing energy is required to disperse clay balls.

The following tests were performed on the trial mixes to evaluate if the mixes meet performance criteria: unconfined compression tests, slump tests, and the Marsh Funnel test. Permeability testing was not performed, as similar mix designs from previous projects have easily met the performance criteria. Gradation tests were performed on the three soil materials. The test required to determine viscosity of the water-bentonite mixture is the Marsh Funnel test (API Code RP 13B procedure). The SCB mix design testing program is presented in detail in the 2004 URS Geotechnical Report.

7.6.2 SCB Mix Design Test Results

Mix design testing is currently in progress, and only the raw data for the 7-day and 14-day unconfined compression tests have been provided by the testing laboratory. All nine trial mixes met the 7 to 9 inch slump criteria and the Marsh Funnel test criteria. No clay balls were observed to form during mixing for any of the trial mixes, although the required mixing energy was greater to achieve a uniform mix for the higher fines content mixes (30 and 45 percent). Seven and 14-day unconfined compression test results are summarized below:

Soil Material	Cement Content (%)	7-day Compressive Strength (psi)	14-day Compressive Strength (psi)
Dirty MAG AB	6	70	80
	8	80	90
	10	120	140
30% Fines Mix	6	60	70
	8	70	120
	10	110	150
45% Fines Mix	6	60	80
	8	80	110
	10	120	140

[Note to Reviewers: Twenty-eight day unconfined test results and final mix proportions will be reported as part of the 100% Design Report submission, and test results will be discussed in greater detail. Based upon the preliminary results, it appears that at least a 10 percent cement content will be recommended.]

8.0 HYDROLOGY

8.1 GENERAL

The White Tanks FRS No.3 was constructed in 1954 by the SCS to protect farmland and irrigation facilities from runoff collected off the White Tank Mountains. The structure was built with a crest length of 1.5 miles and designed to impound runoff from a drainage area approximately 24 square miles. The capacity of the reservoir at the time of construction was 2,655 ac-ft below the crest of the emergency spillway.

Since the original design in 1954, several characteristics related to the hydrology and hydraulics for the structure have changed. A Phase II flood study performed by the Flood Control District in 1984 noted that portions of the watershed had been removed due to the breaching of training dikes and diversion channels north of Northern Avenue and the redirection of flows from the Caterpillar Test grounds. These changes reduced the tributary area of the structure to approximately 20.5 square miles, a reduction of 3.5 square miles. In addition, it was also found in previous studies that the portions of the White Tanks FRS No. 3 structure crest elevation are lower than the original design elevations due to subsidence caused by the extensive withdrawal of groundwater in the region. The current survey data shows a storage volume of 3,153 ac-ft below the emergency spillway crest elevation of 1,212 feet (NAVD 88).

As a part of the current study, URS reviewed existing hydrologic/hydraulic analysis and models developed by the Natural Resources Conservation Service (NRCS) and documented in the report titled as *Hydrologic Analysis of the White Tank Mountains on Flood Retarding Structure # 3* (NRCS 1998). URS staff conducted a site visit in April 2004 to verify watershed conditions. The NRCS hydrologic models reflect current watershed conditions. The models were updated to reflect anticipated future development. Additional models were developed as identified by the District. The procedures and methodologies used to develop the updated models are discussed in the following sections. Details of the modeling and calculations are provided in Appendices C, D, and E.

8.2 REVIEW OF EXISTING MODELS

URS reviewed the existing hydrologic/hydraulic analyses and models documented by Natural Resource Conservation Services (NRCS) in their hydrology report (NRCS 1998). NRCS developed flood hydrographs for a range of storms including the 100-year, 24-hour; 100-year, 10-day; Emergency Spillway Hydrograph (ESH); and the Probable Maximum Precipitation

(PMP). A summary of the results for the 100-year, 24-hour storm is provided in Table II of the NRCS hydrology report (NRCS 1998). The ESH hydrograph is based on a hyetograph that combines the 100-year, 6-hour and 6-hour Local PMP. NRCS developed Probable Maximum Flood (PMF) hydrographs based on PMP distributions for 6-hour Local and 6-, 12-, 18-, 24-, 48- and 72-hour General storms using TR-20 computer model. NRCS routed these inflow hydrographs through the reservoir with the spillway elevation set at 1210 feet (NGVD 29). The peak inflows and the corresponding outflows are summarized in Table III of NRCS hydrology report (NRCS 1998).

The derivation of the various PMFs presented in the NRCS hydrologic report (NRCS 1998) includes the generally accepted rainfall estimation procedures in Hydrometeorological Report No. 49 (HMR-49). The TR-20 input files provided by the District show that AMC II curve numbers were used in the PMF analysis. The derivation of the 100-year, 24-hour and 100-year, 10-day hydrographs appear to be developed in accordance with the cited references (Chapter 21 of NEH-4, and Hydrologic Notes PO-4 and PO-6). It should be noted that 100-year, 10-day hydrograph does not have a shape similar to that expected from a typical 10-day extreme rainfall. URS noted that in deriving the 100-year 10-day hydrograph, NRCS applied a Channel Loss Factor (CLF) to computed runoff to account for infiltration into the channel beds. This factor for this watershed is 0.55. The result is that the runoff volume from the 100-year, 10-day storm is less than that for the 100-year, 24-hour storm. In sum, NRCS's derivations of design hydrographs for the White Tanks FRS No. 3 watershed appear to be reasonable.

The electronic versions of the NRCS's TR-20 models provided by the District were also reviewed. Details of the TR-20 models and the results are summarized in Table 8-1. Peak inflows were compared for each storm obtained from the District provided output files with the ones tabulated in Tables II and III of NRCS hydrology report (NRCS 1998) and found an exact agreement between them (see Table 8-1). The input files provided by the District were executed and compared to the generated peak inflows with the NRCS results. Minor discrepancies were found for the 6-, 12-, 48-, 72-hour General PMP storms, ESH, and 100-year, 24-hour storm events (see Table 8-1). Although these discrepancies are of minor nature, they have been documented as a part of the review process.

8.3 DEVELOP DESIGN MODELS

The existing TR-20 computer models were modified to reflect anticipated future development. The steps involved in developing these models are described in the following sections.

8.3.1 Watershed Delineation

NRCS delineated the White Tank Watershed above FRS No. 3 into 7 basins, as shown on Figure 4 of the NRCS hydrology report (NRCS 1998). The drainage area of each basin is documented in Table I of NRCS hydrology report. The District was unable to provide the electronic version of the NRCS watershed map. However, the District provided URS with an electronic version of the watershed based on a modified version prepared by WLB, Inc. for a previous study. The modified map was not identical to the NRCS watershed map.

The watershed map developed by NRCS consisted of 7 major basin areas. The modified District delineations were placed onto USGS quadrangle maps and adjusted to match the contour lines (See Figure 8-1). The revised drainage areas, and those estimated by NRCS, are presented in Table 8-2. A review of Table 8-2 indicates that the drainage areas of each basin as determined by URS and NRCS are very similar, with the overall variation less than 0.5 percent. Therefore, the drainage areas developed by NRCS were used in the updated TR-20 models.

8.3.2 Reservoir Elevation-Area-Capacity Curve

8.3.2.1 Elevation-Area-Capacity Curve for the Existing Structure

An updated elevation-area-capacity curve for the existing White Tank FRS No. 3 was developed using the 2003 and 2004 topographic mapping in combination with the modified 1998 topographic mapping, both of which were provided by the District. The elevation-area-capacity curve for the existing structure was established using the end-area method as described in Table 17-2 of *NEH-4 – Hydrology Manual* (USDA 1985A).

8.3.2.2 Modifications to Elevation-Area-Capacity Curve

Construction of the remediation project will result in changes to the existing elevation-area-capacity curve during Phase 1 and Phase 2 construction. These changes will occur due to the excavation of soil from the borrow sources in the reservoir pool and the subsequent placement of fill materials on the upstream edge of the existing embankment.

An evaluation was performed to estimate the impact on the reservoir routing due to the modification of the curve. An estimate of the area removed from the reservoir volume was made based on the Phase 1 embankment cross-section and the preliminary Phase 2 embankment cross-section. To be conservative, the evaluation assumed that the volume below the emergency spillway elevation remained unchanged from the existing volume. The results of the evaluation show that the maximum water surface elevation estimates developed from reservoir routings would remain unchanged. Details of the evaluation are presented in a calculation package provided in Appendix C.

8.3.2.3 Design Elevation-Area-Capacity Curve

The existing elevation-area-capacity curve will be used for reservoir routing and estimating the design embankment crest elevation. The evaluation of impacts to the reservoir routing results due to estimated modifications of the elevation-area-capacity curve indicated no significant impact to the results. The elevation-area-capacity data is summarized in Table 8-3 and presented graphically on Figure 8-2. The detailed computations related to determination of elevation-area-capacity curve for White Tank FRS No.3 are provided in a calculation package in Appendix C. An as-built elevation-area-capacity curve will be developed following completion of both Phase 1 and Phase 2 construction.

8.3.3 Sediment Pool

8.3.3.1 NRCS Sediment Yield Estimate

NRCS used the Pacific Southwest Inter-Agency Committee (PSIAC) Sediment Yield Evaluation Model to estimate the 100-year sediment accumulation at White Tanks FRS No. 3 (NRCS 1994). The NRCS estimated an annual sediment yield of 0.244 acre-feet/square mile, or 5 acre-feet for the 20.5 square mile watershed, for a total sediment yield of 500 acre-feet.

8.3.3.2 Other Sediment Yield Studies

Estimates of annual sediment yield were presented in the Spook Hill ADMP Update that ranged from 0.07 to 2.16 acre-feet/square mile for various structures throughout Arizona. The annual sediment yield for the structures within the Spook Hill area ranged from 0.07 to 0.16 acre-feet/square mile. The NRCS estimate would appear conservative when compared to the Spook Hill area estimates.

A detailed study of annual sediment yield for McMicken Dam was presented in the Draft Wittman ADMS Update (District 2004). The study indicates that a reasonable estimate of annual sediment yield is provided by the PSIAC model and ranges from 0.21 to 0.40 acre-feet/square mile. The NRCS estimate falls within this range of sediment yield estimates.

8.3.3.3 *Design Sediment Accumulation*

The design sediment accumulation for 100 years is based on the NRCS estimate of 500 acre-feet. This estimate appears reasonable when compare to the Spook Hill and McMicken Dam studies. The 500 acre-feet of sediment corresponds to an elevation of 1,199.2 ft (NAVD 88).

8.3.4 Reservoir Infiltration

The TR-20 models developed by NRCS included a seepage component in the outflow rating curve. As a part of a previous study conducted by Dames & Moore for White Tank FRS No. 3, infiltration tests were conducted within the White Tanks reservoir to collect site-specific infiltration values for. The results of the infiltration tests were presented in the *Draft Design Issues Report (DIR) – White Tanks FRS # 3 Modifications Design Project* (Dames & Moore 1998). The results estimated an infiltration rate of 0.002 in/hr for the sediment pool, and 0.26 in/hr for the natural ground making up the remainder of the reservoir pool area. The estimated infiltration rate for natural ground was compared with similar studies performed in the area and determined to be reasonable. Estimated infiltration rates for different reservoir elevations are provided on Table 8-3.

8.3.5 Precipitation

As discussed previously, a review of the models prepared by NRCS and provided by the District indicated that the precipitation estimates appear to be derived in accordance with generally accepted procedures. Therefore, the precipitation values and rainfall distributions within the TR-20 models provided by the District were not modified.

Special note should be given to the 100-year, 10-day routing model, because it does not used precipitation within the model. To adjust for the longer duration storm, the 100-year, 10-day model uses a runoff hydrograph developed from a mass curve. The mass curve is derived using procedures presented in the *NEH-4 - Hydrology Manual* (USDA 1985a). With this approach, the runoff volume resulting from the precipitation is adjusted for an average watershed curve number. The 100-year, 10-day model was modified to reflect curve numbers estimated for future conditions (see Section 8.3.6).

8.3.6 Runoff Curve Number

The runoff curve numbers for the White Tanks FRS No. 3 watershed were developed by modifying the curve numbers previously developed by NRCS to account for anticipated future land use resulting from potential future development. The curve numbers previously developed by NRCS in the NRCS hydrology report (NRCS 1998) are presented in Table 8-4..

8.3.6.1 Land Ownership and Future Land Use

The future land use of the White Tanks FRS No. 3 watershed was derived based on current land ownership. Land ownership information was obtained from Figure 2 in the *Draft Design Issues Report (DIR) – White Tanks FRS # 3 Modifications Design Project* (Dames & Moore 1998). The current land ownership was overlain on the watershed delineation map (See Figure 8-2). The 4 categories of land ownership are:

- State Trust Land
- Private Property
- Maricopa County Regional Park
- District Property

An approach was developed to determine which areas would be considered as being developable and undevelopable. Any areas within the County Regional Park and District property were considered to by undevelopable. In addition, lands within the mountainous terrain (i.e., steep slopes) were determined to be undevelopable. Lands considered to be mountainous terrain are shown on Figure 8-3. Private Property and State Trust Land were considered to be developable.

Developable areas were separated into low-density and high-density areas based on the information available at Maricopa Association of Governments (MAG) website for Year 2030 growth projections. Based on this information, all the developable areas located north of Northern Avenue were considered to be low-density and all the developable areas located south of Northern Avenue were considered to be high-density. Details of the distribution of developable and undevelopable areas within the White Tanks FRS No.3 watershed is provided in Table 8-4 and shown on Figure 8-3.

Based on the criteria defined above, White Tank FRS No. 3 watershed was divided into 3 categories of future land use:

- Mountain Region (undevelopable)
- Valley Region (undevelopable)
- Valley Region (developable)

8.3.6.2 *Curve Number Estimates*

The White Tanks FRS No. 3 watershed is divided into 7 basins, as shown on Figure 8-1. The White Tanks FRS No. 3 watershed consists generally of undisturbed desert with mild slopes and mountain areas. Basins 1 and 3 are located entirely within the Mountain Region. Basins 2, 4, 5, 6, and 7 include both Valley and Mountain Region lands. Curve numbers for each basin were estimated using an area-weighted average.

A curve number of 87.2 was used for the Mountain Region, which was based on the NRCS estimate (NRCS 1998). The curve numbers estimated for the Valley Region vary depending on the proportion of developable and undevelopable lands. The curve numbers for the undevelopable portions of the Valley Region were calculated using the curve number estimates provided in the NRCS hydrology study (NRCS 1998). Curve numbers for developable land were estimated using *Urban Hydrology for Small Watersheds, Technical Release No. 55 (TR-55)* (USDA 1986). Because the difference between high-density and low-density development curve numbers was minor, the same curve number was applied to all developable areas.

The modified runoff curve number estimated for each basin is presented in Table 8-4. Details of the curve number derivations are presented in the calculation packages provided in Appendix C.

8.3.7 Diversions

The TR-20 models developed by NRCS included two diversions from the watershed. The diversions occur along the eastern edge of the watershed at Olive Avenue and Northern Avenue where the North Inlet Channel is restricted by culverts at the road crossings. The effect of the diversions is to reduce the peak flow and volume reaching the reservoir from the northern half of the watershed. In general, the full runoff volume and peak flow rates resulting from the 100-year, 24-hour storm event reach the reservoir. For storm events greater than the 100-year, 24-hour storm, the peak flow in the channel is limited by the diversions and the volume reaching the reservoir is reduced.

The TR-20 models incorporates a diversion at Olive Avenue where flows greater than 4,100 cfs are diverted out of the watershed. Flows less than or equal to 4,100 cfs at Olive Avenue are conveyed in the North Inlet Channel to Northern Avenue where the flows are combined with runoff from the basins between Northern and Olive Avenues. The TR-20 models incorporates a diversion at Northern Avenue where flows greater than 11,000 cfs are diverted out of the watershed. Flows less than or equal to 11,000 cfs at Northern Avenue are conveyed in the North Inlet Channel to the reservoir.

The base hydraulic calculations for these diversion estimates were not presented in the NRCS hydrologic report (NRCS 1998), nor were the flows out of the reservoir watershed quantified.

8.3.8 Other Model Parameters

It should also be noted that only the curve numbers were modified for the design models. Basin lag times and antecedent moisture condition (AMC) for the basins were not modified.

9.0 HYDRAULICS

9.1 GENERAL

The emergency spillway at White Tanks FRS No. 3 will be significantly modified to improve spillway efficiency and reduce the maximum water surface in the reservoir during the PMF. A broad-crested weir will be constructed across the existing open channel spillway. The hydraulics of the broad-crested weir provides a control section that determines the flow depth within the reservoir. The submergence analysis was performed to verify that the broad-crested weir would not be drowned-out by the flow depths downstream of the spillway crest

The spillway control section will be constructed of soil cement and extend the spillway crest length. A channel will be excavated downstream to provide the required flow depth and conveyance. Soil will also be excavated upstream to provide the required approach depth and width. The Bethany Home Road Dike will be reconstructed on District property south of the downstream channel. Details of the emergency spillway design are provided in the design drawings.

9.2 EMERGENCY SPILLWAY DESIGN

The emergency spillway control section will be constructed of soil cement and extend the spillway crest length. The soil cement section of the emergency spillway will be constructed with a 10-ft crest and 1:1 upstream and downstream slopes. The crest of the emergency spillway will be set at elevation 1,212.0 ft (NAVD 88). The effective crest length of the emergency spillway will be 1,200 ft. A soil cement apron will be placed immediately downstream of the spillway weir to contain the hydraulic jump and protect against erosion. A 5-ft deep cutoff wall and rip rap blanket will be placed at the downstream end of the soil cement apron to protect the against erosion within the natural spillway channel. Aesthetic fill will be placed over the upstream and downstream slopes no flatter than 10:1 and 4:1, respectively. The right and left abutments of the emergency spillway will be constructed at 2:1 slopes. No aesthetic fill will be placed over the abutments above the spillway crest elevation.

Details of the emergency spillway design, including a discussion of the SITES modeling, are provided in Section 13.0 of this report and shown on the plans. It is important to note that the emergency spillway structure has only been prepared to a preliminary design level. Detailed design of the structure will be completed during the Phase 2 Remediation Design Project.

9.3 DEVELOPMENT OF THE EMERGENCY SPILLWAY DISCHARGE RATING CURVE

The development of the discharge rating curve for the emergency spillway required the development of an initial rating curve for use in evaluating upstream and downstream flow conditions and their potential impacts on the rating. The final rating curve incorporates any potential upstream and downstream impacts.

9.3.1 Initial Rating Curve

The emergency spillway is designed to function as a broad-crested weir. The hydraulic analyses used to develop the spillway rating curve are based on the following weir equation:

$$Q = CLH^{3/2},$$

where 'C' is the weir coefficient,

'L' is the spillway crest length, and

'H' is the depth of flow over the weir as measured in the reservoir.

The design weir coefficient (C) was estimated to be 2.64 using engineering references. The weir coefficient takes into account the placement of aesthetic fill. Details of the weir coefficient selection are presented in the calculation packages provided in Appendix C.

A design spillway crest length (L) of 1,200 ft was used to develop the rating curve. Although the actual spillway crest length is approximately 1,223 ft, the rating curve was developed assuming the side slopes of the spillway abutments would impact flow. Therefore, the effective spillway crest length was reduced to 1,200 ft. The rating curve was developed using a range of depths (H).

Preliminary evaluations showed the 6-hr Local Storm PMF, under NRCS criteria, to be the worst-case condition resulting in a maximum water surface elevation of 1216.5 ft and a discharge of approximately 30,000 cfs (see Section 10.0).

9.1.1 Analysis of Downstream Flows

The design of the broad-crested weir requires that downstream flows do not drown-out the weir during the maximum spillway discharge. To prevent the weir from becoming drowned-out, the emergency spillway channel must be excavated to provide sufficient elevation drop from the spillway crest to the top of the channel water surface during maximum discharge. The spillway channel will be excavated to provide 0.5 percent bottom slope to convey flow away from the weir. The channel maintains this slope until it intersects with the existing grade.

9.1.1.1 Downstream Flow Depth Estimate

The natural conveyance downstream of the spillway channel is steeper and wider than the proposed spillway channel. This suggests that flow depths in the natural conveyance will be less than in the spillway channel. In addition, analyses indicate that the flow in both areas is subcritical. Therefore, normal depth calculations were used to estimate the flow depth in the spillway channel downstream of the spillway weir.

FLOWMASTER, a computer program, was used to determine the normal flow depth within the spillway channel. The flow depth was evaluated for flow rate of 30,000 cfs, a channel bottom width of approximately 845 ft, and side-slope of 2:1(H:V). The District used Manning's roughness values of 0.045 for channel flows and 0.060 for overbank flows in its HECRAS model. Based on a field visit performed by URS staff, these estimates of Manning's Roughness appeared to be a reasonable representation of the actual conditions. FLOWMASTER requires the use of a single roughness coefficient. Therefore, an average of 0.05 was used for the spillway channel flow evaluation. The FLOWMASTER analysis indicated a normal flow depth of 5.45 ft within the spillway channel with the mean flow velocity equal to 6.45 ft/sec. Details of the FLOWMASTER output results are presented in the calculation packages provided Appendix C.

9.1.1.2 Submergence Effects Evaluation

The broad-crested weir was also checked against the possible submergence effects caused by the downstream flow conditions in the spillway channel. The evaluation assumed a minimum drop from the spillway crest to the spillway channel of 4.0 ft. The submergence analysis indicate the downstream flow conditions in the spillway channel do not submerge the spillway weir and no modifications to the rating curve are required. Details of the submergence analysis are presented in the calculation packages provided in Appendix C.

9.1.2 Analysis of Upstream Conveyance Capacity

The design of the emergency spillway is a significant modification to the existing structure. The design required the analysis of the conveyance capacity from the reservoir to the spillway crest. Reduction in conveyance capacity can result in an increase in the reservoir water surface upstream of the spillway.

The conveyance analysis indicated that the water surface elevation rises by approximately 0.25 ft moving south to north along the emergency spillway. The analysis also indicated that this rise in water surface only occurs at the spillway and does not continue to increase further in to the reservoir. Details of the conveyance analysis are presented in the calculation packages provided Appendix C.

9.1.3 Design Discharge Rating Curve

The design rating curve was developed by modifying the initial rating curve through incorporation of upstream and downstream impacts. Analyses showed no downstream impacts for the proposed spillway design. However, the spillway design does result in upstream impacts due to reduced conveyance capacity. The upstream impacts require that the spillway rating curve account for the increasing water surface along the spillway. The initial rating curve was modified by reducing the discharge to reflect the reduced effective depth over the spillway. The effective depth was reduced by 0.12 ft, which was the average depth reduction of the conveyance impact. The emergency spillway design discharge rating curve is presented in Table 9-1 and Figure 9-1. Details of the rating curve development are presented in the calculation packages provided Appendix C.

10.0 RESERVOIR ROUTING

10.1 GENERAL

Reservoir routing was performed for selected storm events to determine water surface elevations for embankment design. Reservoir routing was performed using the revised TR-20 models and input parameters discussed in the previous sections of this report. The following storm events were modeled:

- 100-year, 24-hour; 200-year, 24-hour; and 500-year, 24-hour
- 100-year, 10-day
- Probable Maximum Precipitation (PMP) (6-hour local, 6-hour general, 12-hour, 18-hour, 24-hour, 48-hour, and 72-hour)
- Emergency Spillway Hydrograph (ESH)

10.2 DEVELOP ROUTING MODELS

10.2.1 Routing Conditions

The installation of a principal spillway represents a significant change in the routing conditions for White Tanks FRS No. 3. The District plans to install a downstream conveyance channel in the future to control outflow from the principal spillway. However, until the conveyance channel is constructed the District intends to close the principal spillway to prevent uncontrolled outflow. The following routing conditions potentially exist and were evaluated:

- Interim Condition
- Future Condition (Principal Spillway Open)
- Future Condition (Principal Spillway Closed)

The resulting water levels estimated from the routing models will be evaluated against the design criteria. These 3 routing conditions are discussed in the following sections.

10.2.1.1 Interim Condition

The Interim Condition represents the time period following construction of the facilities included in the Remediation Project and the installation of the downstream conveyance channel. Since this time period is anticipated to be short (less than 10 years), it is assumed that no additional sediment has accumulated in the reservoir. The principal spillway is closed and does not convey flow out of the reservoir. Under this condition the only outflow from the reservoir occurs through infiltration and flow through the emergency spillway.

10.2.1.2 Future Condition (Principal Spillway Open)

The Future Condition (Principal Spillway Open) represents the time period following construction of the downstream conveyance channel and the principal spillway is opened. The inlet to the principal spillway is set at elevation 1,200 feet (NAVD 88). It is assumed that the 100-year sediment volume (500 ac-ft) has accumulated in the reservoir. Under this condition outflow from the reservoir occurs through infiltration, flow through the principal spillway, and flow through the emergency spillway.

10.2.1.3 Future Condition (Principal Spillway Closed)

The Future Condition (Principal Spillway Closed) represents the condition where construction of the downstream conveyance channel does not occur and the principal spillway remains closed. It is assumed that the 100-year sediment volume (500 ac-ft) has accumulated in the reservoir. Under this condition outflow from the reservoir occurs through infiltration, and flow through the emergency spillway.

10.2.2 Routing Models

10.2.2.1 NRCS Models

Routing was performed for the 6-hour local, 6-hour general, 12-hour, 18-hour, 24-hour, 48-hour, and 72-hour PMF design floods, 100-year 10-day and ESH based on design criteria established by NRCS. The NRCS design criteria are detailed in Technical Release No. 60 (TR-60) (USDA 1985b), and include:

- For the Interim Condition and Future Condition (Principal Spillway Closed), routing was performed to verify that the water level resulting from back-to-back 100-year, 10-day storms did not exceed the emergency spillway crest elevation. This criteria is required because the reservoir has gated outlets and no principal spillway.
- For the Future Condition (Principal Spillway Open), routing was performed to verify that the principal spillway could convey a single 100-year, 10-day storm with the maximum water level below the emergency spillway crest and drain the associated water volume down to the principal spillway inlet elevation 10 days following the storm.

- For the Interim Condition and Future Condition (Principal Spillway Closed), the antecedent reservoir condition (ARC) for the PMF and ESH hydrographs will be based on the water surface elevation 10 days following the end of the 100-year, 10-day storm. Since the outlets are gated, drawdown of the reservoir for this case was the result of infiltration.
- For the Future Condition (Principal Spillway Open), the ARC for PMF and ESH hydrographs will be based on the inlet elevation of the principal spillway.
- The dam crest elevation will be set at an elevation above the maximum water level that results during routing of the required PMF hydrographs. The PMF hydrograph is considered the freeboard hydrograph for this structure.
- The emergency spillway must be shown to pass the ESH hydrograph at the safe velocity determined for the site. The ESH hyetograph is developed from a combination of the 100-year, 6-hour and 6-hour local PMP.

10.2.2.2 ADWR Models

Routing was performed for the 6-hour Local Storm PMF, 72-hour General Storm PMF, and 100-year 24-hour storm based on design criteria established by ADWR. The ADWR design criteria are provided in the *Draft Guidelines: Emergency Spillway Capacity, Reservoir Routing, and Freeboard Requirements* (ADWR 2004), and include:

- Based upon the size and hazard classification of the dam, the embankment crest will be set at an elevation equal to the maximum water level that results during routing of the required PMF hydrographs plus the residual freeboard.
- For the Interim Condition, the ARC for the PMF hydrographs will be based on the water surface elevation equal to the invert of the lowest outlet. This ARC was selected after verifying that a single 48-inch outlet pipe could draw down 85 percent of the peak storage volume at the end of 10^{th} day following the peak of 100-year 24-hour storm.
- For the Future Condition (Principal Spillway Closed), the ARC for PMF hydrographs will be based on the elevation of the 100-year sediment pool.
- For the Future Condition (Principal Spillway Open), the ARC for PMF hydrographs will be based on the inlet elevation of the principal spillway.

10.2.2.3 District Models

In addition to the 100-year, 24-hour storm event, the District requested that routing models be developed to evaluate reservoir conditions for the 200-year and 500-year, 24-hour storm events. TR-20 models for the 200-year and 500-year models were developed using the existing 100-year, 24-hour model. The storm events were modeled for the three routing conditions presented in Section 10.2.1.

The rainfall depth for 500-year, 24-hour storm was determined based on the methodology described in *Highway Drainage Design Manual – Hydrology, Arizona Department of Transportation* (ADOT 1993). However, ADOT manual did not provide necessary information required to develop 200-year, 24-hour rainfall depth. Therefore, 5-, 10-, 50- and 100-year, 24-hour rainfall depths were developed based on the methodology described in ADOT Drainage Manual. The depth-duration relationship of the 24-hour rainfall depths were plotted against the 5-, 10-, 25-, 100-, and 500-year duration on a semi-log scale. Based on this depth-duration relationship, the 200-year, 24-hour rainfall depth was estimated. The 200-year and 500-year, 24-hour rainfall depths were reduced for aerial reduction by the same factor by which the 100-year, 24-hour rainfall amount was reduced in the NRCS Hydrology Report (NRCS 1998). The computations related to development of the 200-year and 500-year, 24-hour rainfall depths are provided in a calculation package in Appendix C.

10.3 MODELING RESULTS

TR-20 modeling was performed using the criteria established by ADWR, NRCS, and the District. Reservoir routing results consisted of peak inflows, peak outflows, storage volume, and maximum reservoir stage. The results are summarized in Table 10-1. The TR-20 input and output files are presented in the calculation packages provided in Appendices D and E.

10.3.1 NRCS Models

Based on the reservoir routing results for the NRCS models, the maximum reservoir stage occurs during routing of the 6-hr Local Storm PMF. The maximum reservoir stage for the 3 routing conditions occurs for the Future Condition (Principal Spillway Closed) at an elevation of 1,216.4 feet (NAVD 88).

For the Interim Condition and Future Condition (Principal Spillway Closed), the routing results show that back-to-back 100-year, 10-day storm events will be contained below the emergency spillway crest. For the Future Condition (Principal Spillway Open), the maximum water level for a single 100-year, 10-day storm was shown to be below the emergency spillway crest. The results also show that the maximum water level resulting from routing of the ESH is below the was also routed through the reservoir. The antecedent reservoir condition for ESH was set same as of PMF design flood hydrographs.

10.3.2 ADWR Models

Based on the reservoir routing results for the ADWR models, the maximum reservoir stage occurs during routing of the 6-hr Local Storm PMF. The maximum reservoir stage for the 3 routing conditions occurs for the Future Condition (Principal Spillway Open) at an elevation of 1,216.0 feet (NAVD 88). The results show that the maximum water level during routing of the Future Condition (Principal Spillway Closed) model was 0.1 ft lower than the Future Condition (Principal Spillway Open) model. This lower water level is caused by the lower ARC elevation and the fact that the principal spillway outflow does not contribute significantly to lowering the water level during routing of the 6-hour Local Storm PMF.

10.3.3 District Models

TR-20 models were developed to provide hydrographs for the 100-year, 200-year, and 500-year 24-hour storm events. Results of the modeling are provided in Tables 10-1 and 10-2. Based on the reservoir routing results for the District models, the 100-year and 200-year, 24-hour storm events are contained below the emergency spillway crest for each of the routing conditions. The results indicate the 500-year storm event results in a discharge through the emergency spillway.

11.0 FISSURE EROSION MODELING

[Note to Reviewers: The fissure erosion modeling discussion is provided in a separate report prepared by Engineering & Hydrosystems Inc. Details of the report and technical reviews will be provided in the 100 Percent submittal.]

12.0 EMBANKMENT DESIGN

12.1 GENERAL

Section 12.0 of this report discusses the embankment configuration for the South Fissure Risk Zone (FRZ) Embankment and Transition Embankments. The South FRZ Embankment and transitions are to be completed as part of Phase 1 of the Remediation Project. Phase 2 of this project will complete the embankment components of the White Tanks FRS No. 3 structure and will consist of the North FRZ Embankment, South Non-Fissure Risk Zone (NFRZ) Embankment, and the North NFRZ Embankment.

The following sections focus on the details of the Phase 1 embankment including the new dam stationing, selection of the dam crest elevation, and physical dimensions of the embankment. In addition, discussions are presented on the rationale and basis of selection for the various components of the embankment (e.g., soil cement, geosynthetic elements, etc.). Brief discussions are also presented concerning the Phase 2 embankment to provide the reviewer some perspective as to the final anticipated configuration of the embankment.

12.2 STATIONING

The dam stationing has been modified to include the existing embankment, the South FRZ Embankment, and Transition Embankments. The new stationing is aligned along the centerline of the existing embankment and the new embankment where modifications are proposed. The new stationing places Station 0+00 to the right of the emergency spillway. The Stationing of the Phase 1 Embankment are shown on Table 12-1, with both New Dam Stationing and Existing Dam Stationing. The new embankment stationing has been changed in accordance with District drawing standards and runs the opposite direction from the original embankment stationing.

12.3 CREST ELEVATION

12.3.1 Existing Conditions

The original design by the NRCS (1952) shows a design crest elevation of 1,216 feet (NGVD 29); converted to 1,217.87 feet based on the NAVD 88 Datum. A survey along the crest of the dam by the District in November 2003 shows that that north end of the dam has subsided by approximately 4.7 ft, while the south end of the dam has subsided by approximately 1.0 ft.

12.3.2 Design Requirements

Determination of the design crest elevation is based on the results of reservoir routing plus freeboard. For the design of White Tanks FRS No. 3, the freeboard will be based on wave runup and subsidence.

12.3.2.1 IDF Routing

Routing of the Inflow Design Flood (IDF), or PMF, through the reservoir estimated the maximum water surface elevation behind the embankment for both ADWR and NRCS criteria, as discussed in Section 10.0 and summarized on Table 10-1. The maximum water surface elevation estimated based on the ADWR criteria is 1,216.0 feet (NAVD 88). The maximum water surface elevation estimated based on the NRCS criteria is 1,216.5 feet (NAVD 88). The maximum water surface elevations estimated based on ADWR and NRCS criteria are not equal due to differences in the ARCs used in the TR-20 models.

12.3.2.2 Freeboard

Freeboard is addressed for both ADWR and NRCS design criteria. ADWR design criteria requires freeboard be provided above the inflow design flood, or PMF. NRCS design criteria requires freeboard be based on an Emergency Spillway Hydrograph (ESH) and a Freeboard Hydrograph (PMF).

12.3.2.2.1 ADWR Freeboard

The requirement for total freeboard of an embankment is detailed in the Arizona Administrative Code (A.A.C.) R12-15-1216.A.2.d as:

An applicant shall ensure that the total freeboard is the largest of the following:

- *i. The sum of the inflow design flood maximum water depth above the spillway crest plus wave runup.*
- *ii. The sum of the inflow design flood maximum water depth above the spillway crest plus 3 feet.*
- *iii. A minimum of 5 feet.*

The District has submitted a request for variance to this rule requesting that the proposed embankment modifications only be required to meet part (i) and (iii) of this section. The request for variance is based on the following:

- The inflow design flood is the PMF, which is considered to have an extremely low frequency of occurrence.
- During impoundment of the inflow design flood, the maximum water surface occurs for only a short duration. The 72-hour General Storm PMF is estimated to result in a freeboard of less than 3 feet for a period of 5 hours.
- The wave runup for the reservoir was estimated to be 1 foot, with an exceedence frequency of 0.4 percent. Details of the wave runup calculation are provided in Appendix C.

For these reasons, the design freeboard has been selected to be 1.0 ft above the maximum water surface elevation resulting from the reservoir routing. The minimum dam crest elevation required based on this approach would be 1,217.0 feet (NAVD 88).

[Note to Reviewers: This section may be modified based upon the review and comment by ADWR of the variance request.]

12.3.2.2.2 NRCS Freeboard

The maximum water surface elevation for NRCS criteria occurs from routing for the Future Condition – Principal Spillway Closed. The water surface elevations for the ESH and PMF are 1,213.2 feet (NAVD 88) and 1,216.5 feet (NAVD 88), respectively. NRCS design criteria require that the dam crest be set at a minimum elevation of 1,216.5 feet (NAVD 88) to meet freeboard requirements.

12.3.2.3 Subsidence Raise

12.3.2.3.1 Overall Embankment Design

The subsidence raise is calculated based on a maximum of 1.0 ft and adjusted to reflect the variation in historic subsidence over the length of the dam (see Section 6.9). Table 6-1 presented subsidence adjustment estimates at various locations along the embankment which varied depending of the method used. The design subsidence raise was determined based on the information presented in Table 6-1 and incorporates a conservative approach. The design subsidence freeboard is summarized in the following table:

Existing Dam Station	Design Subsidence Raise (feet)
0+00 and North	1.0
0+00 to 55+00	1.0
55+00 and South	0.4

The conservative approach is reflected in the selection of a design subsidence raise of 1.0 ft between existing dam stations 30+00 and 55+00, which consists of the South FRZ Embankment being designed and constructed under Phase 1 of this project. A reduction of 0.1 ft could have been incorporated south of Station 30+00, but was excluded to account for the potential unknown impacts related to dam construction and freeboard (see Section 8.3.2). In addition, some variation in the subsidence freeboard could also have been accounted for between existing dam Station 30+00 and 55+00. However, the variation is minor (approximately 0.3 ft) and is not considered in order to provide a more efficient construction approach. Similarly, the variation in the subsidence freeboard from existing dam Station 55+00 and south varies from 0.3 to 0.4 ft, a constant increase of 0.4 ft will be used.

12.3.2.3.2 Phase 1 Design

The stationing provided in the table is shown as existing dam stations. The Phase 1 embankments (South FRZ and Transition Embankments) will be constructed as follows:

- The South FRZ Embankment will receive the design subsidence raise of 1.0 feet.
- The Transition Embankments will be constructed to the height of the adjacent existing dam and not receive a design subsidence raise during Phase 1. The embankments will be modified to include the appropriate design subsidence raise during Phase 2.

12.3.2.4 Design Crest Elevations

12.3.2.4.1 Overall Embankment Design

The design crest elevations of the modified embankment are based on the results of the IDF routing, freeboard requirements, and the design subsidence raise. The results of the IDF routing and freeboard requirements indicate that the ADWR criteria result in a dam crest elevation of 1,217.0 feet (NAVD 88), which is greater than the dam crest elevation of 1,216.5 feet (NAVD 88) based on the NRCS criteria. Therefore, the minimum crest elevation based on IDF routing and freeboard requirements will be 1,217.0 feet (NAVD 88). Incorporating the design subsidence raise, the design crest elevations for the overall embankment are summarized in the following table.

Existing Dam Station	Design Embankment Crest Elevation (feet)
0+00 and North	1,218.0
0+00 to 55+00	1,218.0
55+00 and South	1,217.4

12.3.2.4.2 Phase 1 Design

The Phase 1 design consists of the South FRZ and Transition Embankments as shown on the plans. The South FRZ Embankment crest elevation is set at 1,218.0 ft (NAVD 88). The crest of the Transition Embankments are set at the adjacent existing embankment crest and slope from the existing crest to adjoin the soil cement embankment. The crest elevations where the South and North Transition Embankments meet the existing embankment are 1,216.0 ft (NAVD 88) and 1,214.0 ft (NAVD 88), respectively. The Transition Embankment elevations were set in this manner to provide some flexibility during Phase 2 of the project when transitions will be completed as part of the NFRZ Embankment design.

12.4 PHASE 1 EMBANKMENT DESIGN

Phase 1 of this project consists of the South FRZ Embankment and the Transition Embankments. The Transition Embankments connect the South FRZ Embankment to the existing dam. The following sections provide details of the embankments and supporting design analyses.

12.4.1 South Fissure Risk Zone Embankment Design

The South FRZ Embankment consists of a soil cement embankment constructed upstream of the existing embankment and located between the New Dam Stations indicated on Table 12-1. The location of this embankment was selected to correspond with the South Fissure Risk Zone located between Existing Dam Stations 30+00 and 55+00 (See Section 7.0). The embankment design includes cutoff system intended to prevent failure of the embankment due to erosion of a fissure beneath the embankment. The cutoff system includes parallel cutoff walls at the upstream and downstream toes of the embankment and a coarse sand filter at the upstream toe. The outlet works will be constructed with conveyance pipes through the soil cement section of the embankment. The soil cement embankment will be covered with common fill during construction.

12.4.1.1 Foundation Preparation

The objective of foundation preparation within the fissure risk zone is to remove and replace collapsible, erodible, and other soils that could potentially have an adverse impact on the long-term performance of the embankment. Relatively young (Holocene) soils and coarse-grained channel deposits are considered unacceptable foundation conditions. As currently proposed, the foundation preparation for the South FRZ Embankment will include the following steps:

- The entire footprint of the proposed foundation excavation as shown in the design drawings will be cleared and grubbed in order to remove vegetation and other deleterious materials.
- Over-excavate and remove a portion of the existing embankment, ensuring that the existing dam crest is not lowered below elevation 1,213.5 feet (NAVD 88).
- Over-excavate and remove the underlying Holocene soils. The excavation depths shown on the plans were estimated based on the information developed from the geotechnical investigation.
- Over-excavate and remove coarse-grained channel deposits exposed within the foundation excavation. The exact location of these channel deposits is not known and will be identified by the Engineer during excavation. The excavation side slopes for this purpose will be no steeper than 2:1.
- Following construction of the cutoff walls, the area between the cutoff walls will be excavated an additional 2 feet prior to construction of the soil cement embankment.
- The foundation excavation will be thoroughly inspected and approved by the Engineer prior to construction of the embankment.

12.4.1.2 South FRZ Embankment

The South FRZ Embankment consists of two components: a soil cement core and a surrounding common fill zone. The design intent is to provide a soil cement core that is stable without the surrounding common fill zone. The cross-section of the embankment changes at the left and right ends where the soil cement meets the Transition Embankments. The following discussions pertain to the soil cement component:

- The soil cement component will be designed to serve as the structural core of the embankment, independent of the surrounding common fill.

- The crest length of the soil cement core was designed to extend between the defined limits of the fissure risk zone. The toes of the left and right abutments extend beyond this zone.
- The alignment of the soil cement core includes a curve at the left end to follow the existing dam alignment.
- The soil cement core has a crest width of 10 feet. The total crest width of the embankment is 18 feet, which provides sufficient width for raising the embankment crest for address future subsidence.
- The side slopes of the soil cement core are 0.6:1 (horizontal to vertical) along most of the embankment. At the left and right ends of the core the upstream side slope flattens to 2.5:1, allowing for connection to the Transition Embankments. The left and right abutments of the soil cement core will be constructed at 3:1 to allow proper placement and compaction of structural fill.
- The soil cement will be designed to withstand erosive forces resulting from potential seepage flows along transverse cracks through the embankment. The erosion resistance of the soil cement will be estimated in terms of its Erodibility Index (Annandale, 1996). The applied erosive forces will be estimated using the breach model developed by Annandale (2003) during a previous project for the District.
- Because of the relatively infrequent impoundment occurrences, as well as the presence of a significant fill surrounding the soil cement core, deterioration of the soil cement due to wet-dry cycles is considered to be unlikely. As such, wet-dry durability tests were not performed for the soil cement mix design. Similarly, due to relatively mild winter temperatures at the site as well as infrequent impoundment, deterioration of the soil cement due to freeze-thaw cycles is considered to be unlikely, and as such, freeze-thaw durability tests were not performed for the soil cement mix design.
- Based on the gradation of the soils used for the soil cement, it is anticipated that compacted soil or other mechanical methods will be required to compact the soil cement lifts during construction. Upstream of the soil cement core with slopes of 0.6:1, the common fill will have finished side slopes of 2:1. Downstream of the soil cement core, the common fill will be integrated with the existing embankment and have a tapered finished slope as indicated on the plans. It is important to note that the existing embankment crest cannot be lowered below elevation 1,213.5 ft (NAVD 88) until the structural fill within the Transition Embankments is placed.

12.4.1.3 Cutoff Walls

Cutoff walls are incorporated into the South FRZ Embankment with the design objective of controlling erosive subsurface flows through potential earth fissures beneath the dam within the FRZ. URS performed preliminary evaluation of three cutoff wall alternatives for depths up to 60 feet:

- A combination geomembrane and controlled low-strength material (CLSM) backfill cutoff wall;
- A soil-cement-bentonite (SCB) cutoff wall; and
- A plastic concrete cutoff wall.

Dual cutoff walls (upstream and downstream) were incorporated into the evaluation, as two cutoff walls were considered necessary to create an impermeable or low-seepage zone beneath the soil cement structure and reduce the risk of subsurface erosion along a potential earth fissure. The cutoff walls are incorporated in the design as a mechanism to force flow within a fissure down into the less-erodible Pleistocene soils found at greater depth beneath the dam.

The SCB cutoff wall alternative was selected as the preferred alternative because of its estimated cost, constructability, and performance characteristics. Performance objectives of the constructed SCB cutoff walls include seepage/fissure flow control, erosion resistance, and cracking resistance. The walls will be constructed in a trench 3 ft wide and extend 30 feet into the Pleistocene soils beneath the dam. In the event that fissure flow occurs, the potential exists for some erosion at the edge of the upstream cutoff wall. As a measure to help minimize this erosion, the design includes a coarse aggregate apron. The material in this coarse aggregate apron is intended to fall into the fissure and provide some additional protection against widening of the fissure. Section 11.0 of this report discusses the fissure erosion modeling performed to develop the design basis. Details of the cutoff wall design are shown on the plans.

12.4.1.4 Fissure Instrumentation and Monitoring

The South FRZ Embankment is located in the fissure risk zone, as discussed in Section 7.3 of this report. The design of the embankment will incorporate instrumentation to detect future fissure formation. Monitoring for fissures will include the instrumentation and ground observation. Details of the fissure instrumentation and monitoring provided in the following sections has been taken from *Fissure Zone Instrumentation & Monitoring Plan, McMicken Dam Fissure Risk Zone Remediation Project* (AMEC 2004). Where appropriate, the text has been modified to reflect the remediation design of White Tanks FRS No. 3.

12.4.1.4.1 Fissure Instrumentation

Fissure instrumentation for the South FRZ Embankment will consist of Time Domain Reflectometry (TDR) Arrays installed within a trench cut in the top of each cutoff wall. TDR utilizes a pulsed electromagnetic signal along a coupled coaxial cable to detect reflected changes resulting from deformation. Both travel time and signal strength are measured. Travel time is used to determine position, with signal strength being an indication of the severity of the strain. Although signal strength is a rough measure of strain, TDR should viewed as a means to detect but not fully quantify ground deformation.

The TDR arrays will be comprised of two parallel 50-ohm coaxial cables of like construction. Each will be composed of solid, copper-clad aluminum conductor, encased in foam polyethylene dielectric, wit an outer, smooth aluminum conducting cover. This will provide for redundancy in the sensing component of the system. Crimps will be placed at 250-ft intervals along the entire length of each cable to provide each reference point in the TDR waveform signatures. The cables will be installed in a trench cut in the top of each cutoff wall and run from the left abutment of the soil cement core, up to the crest of the embankment, and terminate in a weather-proof box near the outlet works. Details of the cable installation and alignment are shown on the plans. **[Note to Reviewers: Details of the fissure instrumentation design may not be included with the 90 Percent submittal, but will be provided with the 100 Percent submittal.]**

The coaxial sensor cable runs will be connected to 8:1 multiplexers (Campbell Scientific, Inc. [CSI] Model SDMX500, housed in an environmental enclosure located at the crest of the dam near the outlet works. The low-loss transmission cable will be standard type RG58, with a solid inner conductor, polyethylene dielectric, and outer braided copper sheathing. Waterproof connections will be prepared between the sensing coaxial cables and multiplexer, with BNC connectors encased in silicone gel and shrink-wrapped. The system will employ dedicated TDR pulser/samplers (CSI Model TDR100 reflectometer), supported by dataloggers (CSI CR10X).

The system will be powered by a 20-watt solar panel recharging a voltage-regulated 12-volt deep-cycle battery. All of the electronic and electrical components will be housed in a vandal-proof ground vault. The solar panels will be mounted above a 15-ft pole placed at the crest of the dam. The pole tower that houses the solar panel will also be fitted with a directional VHF antenna. This antenna will service a dedicated VHF radio (CSI Maxom Model RF310), supported by a modem (CSI Model RF310m). Appropriate software will be acquired to enable the system to be fully functional as a remote detection system with a remote link to the District's ALERT network. The District may combine portions of this system with that developed to monitor reservoir water levels and discharge through the outlet works.

12.4.1.4.2 Ground Inspection

Visual ground inspection should be performed by an experienced person walking the fissure risk zone looking for cracks, potholes or other features which may indicate earth fissuring in the embankment and/or native soils. Visual inspections should be performed as close in time as practicable to the TDR array monitoring. Inspections should also be performed after major storm runoff events. Locations and descriptions of cracks, potholes, and other erosional features should be documented with sketches, maps, and photographs as appropriate including locations dimensions, and orientations. Features should be marked with stakes, small flags, or whiskers nailed into the ground for location by survey.

12.4.1.4.3 Monitoring Schedule

A reasonable monitoring schedule for the TDR arrays must take into account initial calibrations and limits of resolution and repeatability as compared to actual ground movement. Quarterly reading for the first year can provide a basis for overall baseline calibration and personnel training. After the first year of monitoring, the schedule should be revised as appropriate. It is anticipated that annual readings of the system may be adequate and sufficient, especially if remote readings of the instrumentation at a much more frequent schedule is implemented. Unusual events or movements indicated by remotely read instrumentation would trigger a monitoring cycle to verify that the remove measurements indicate a true need for response.

12.4.1.4.4 Response

The District should establish relevant response levels for potential earth fissuring. Initial response levels should include:

- Alert by the TDR instrumentation.
- Observation of unusual erosional features at or near the dam; and
- Observation of a fissure near or projecting toward the dam.

Action guidelines in response to the triggering of response levels might include:

- Notification of regulatory authorities and mitigation of surface features;
- Re-measurement of parameters of interest;
- Modification and/or intensification of monitoring schedule; and
- Acquisition and analysis of new low-sun angle photography.

Other actions that may be considered in consultation with regulatory authorities could include:

- Critical re-evaluation of response levels in light of the measurement data;
- Performance of additional deformation analysis;
- Seismic refraction evaluation to determine if subsurface anomalies may be present;
- Trenching of suspected discontinuities, documentation of geologic observations, and refinement of fissure maps; and
- Implementation of defensive or protective actions.

12.4.1.5 Subsidence Monuments

Subsidence monuments will be installed on the crest of the South FRZ Embankment within the soil cement core. The monuments will be placed along the centerline of the embankment near the abutments and at 250-ft intervals along the length of the dam. Details of the subsidence monument installation are provided on the plans.

[Note to Reviewers: The details of the subsidence monuments may not be included in the 90 Percent submittal, but will be provided with the 100 Percent submittal.]

12.4.1.6 General Discussions

As noted in the previous sections, the soil cement component will be designed as the structural core of the embankment, independent of the common fill around the soil cement core. Removal of the Holocene soils as part of the foundation preparation measures is limited to the footprint required for construction of the proposed soil cement core. Outside of those areas excavated for construction of the soil cement core, foundation treatment will be limited to clearing and grubbing of the surface soils, and scarification, moisture conditioning, and compaction of the upper 8 inches of soil, leaving a portion of the existing Holocene soils under the common fill.

This will also be applicable to the aesthetic fills placed during Phase 2 of this project. Wetting of these Holocene soils may lead to collapse-type settlement and consequent cracking of the aesthetic fill. These cracks may require periodic maintenance measures to maintain the aesthetic appearance of the fill, but are not expected to adversely impact the performance of the embankment.

Similarly, the cutoff walls are located at the upstream and downstream toes of the soil cement core to protect the soil cement core in the event of seepage and erosion along an earth fissure.

However, seepage along an earth fissure may cause damage to the common and aesthetic fill, requiring maintenance after significant impoundments.

12.4.2 Transition Embankment Design

The Transition Embankments join the South FRZ Embankment to the existing dam, thus maintaining a complete structure. Activities related to excavating the downstream slope of the existing dam for the outlet works or general grading cannot be performed until the Transition Embankments are completed to the elevations designated on the plans. The Transition Embankments consist of structural fill and include a graded filter at the downstream end of the soil cement/structural fill connection. Structural fill will be placed overlapping the soil cement embankment on the downstream side to provide sufficient embankment volume. The Transition Embankments will become part of the Non-Fissure Risk Zone Embankment design being developed in Phase 2 of the project.

The South Transition Embankment is located at the right abutment of the South FRZ Embankment; The North Transition Embankment is located at the left abutment of the South FRZ Embankment. The New Dam Stationing for the Transition Embankments is shown on Table 12-1.

12.4.2.1 Foundation Preparation

The objective of foundation preparation within the non-fissure risk zone is to remove the collapsible soils that could potentially have an adverse impact on the long-term performance of the embankment. Relatively young (Holocene) soils are considered unacceptable foundation conditions. As shown in the design drawings, the Holocene soils and the top 2 feet of the Pleistocene soils within the footprint of the proposed upstream embankment will be over-excavated and removed. The foundation preparation for the Transition Embankments will include the following steps:

- The entire footprint of the proposed foundation excavation as shown in the design drawings will be cleared and grubbed in order to remove vegetation and other deleterious materials.
- Excavate at the existing upstream toe at a 2.5:1 slope to remove the Holocene soils beneath the modified embankment footprint.

12.4.2.2 Transition Embankments

The Transition Embankments consist a structural fill embankment connecting the soil cement core of the South FRZ Embankment with the existing embankment. The upstream slope of the transitions are designed to match the upstream slopes of the soil cement embankment. Structural fill will also be placed between the soil cement embankment and existing embankment as shown on the plans. This overlap provides for a minimum embankment thickness of the transition. It is important to note that the Transition Embankments are temporary and will be integrated into the Phase 2 design to address minimum crest elevation requirements and embankment transverse cracking as a potential failure mode.

12.4.2.3 Graded Filter

The graded filter is located downstream of the interface between the South FRZ Embankment and Transition Embankments. The filter is intended to control seepage that may occur along the contact of the soil cement-structural fill contact at the abutments of the South FRZ Embankment. The graded filter will be constructed to overlap the contact of soil cement and structural fill 10 feet in both directions at the downstream edge of the soil cement core. A drain is not provided from the filter. The graded filter is intended to serve only as a temporary component to protect against seepage until completion of the Phase 2 construction. Details of the graded filter design are shown on the plans.

12.5 PHASE 2 EMBANKMENT DESIGN CONCEPT

This design report will only briefly address components of the Phase 2 embankment design to provide an understanding of how the completed embankment will integrate with the Phase 1 embankment design. As discussed earlier in this section, the major Phase 2 embankment components will consist of the following:

- South NFRZ Embankment;
- North NFRZ Embankment; and
- North FRZ Embankment.

In addition, completion of the dam will include the construction of a fill zone north of the left abutment and covering of the embankment slopes with aesthetic fill. Modifications to the emergency spillway will also occur during Phase 2 construction. A general plan view of the Phase 2 components is shown on Figure L1 in Appendix L. Typical cross sections of the design concepts for the NFRZ and North FRZ Embankments are also provided in Appendix L. These concepts may be significantly modified during the Phase 2 design project.

12.5.1 Non-Fissure Risk Zone Embankments

The NFRZ Embankments will be constructed outside of the fissure risk zones as discussed in Section 7.0. The South NFRZ is located between Existing Dam Stations 76+67 and 55+00 and the North NFRZ is located between Existing Dam Stations 30+00 and 0+00. The South NFRZ Embankment will complete the dam between the emergency spillway and the South FRZ Embankment. The North NFRZ Embankment will complete the dam between the South FRZ Embankment and the North FRZ Embankment.

The design of the NFRZ Embankment is intended to perform the following:

- Raise the crest of the embankment in order to prevent overtopping of the embankment during the IDF.
- Reduce the risk of seepage and erosion along transverse cracks of the embankment.

Previous design submittals during this project have proposed addressing the concerns related to transverse cracks through the installation of a geomembrane at the upstream slope of the embankment. However, questions raised by ADWR have necessitated that the District perform a more detailed evaluation of the geomembrane concept and potentially other concepts during Phase 2 design. The Phase 2 design will be integrated with the Transition Embankments constructed during Phase 1.

12.5.2 North Fissure Risk Zone Embankment

The North FRZ Embankment will be constructed in the north fissure risk zone identified north of the Existing Dam Station 0+00. The embankment will consist of a soil cement core and SCB cutoff walls. The embankment will be constructed east of the North Inlet Channel and Dike, then aligned to connect with the existing embankment. The Holocene soils will be excavated prior to construction of the embankment in a manner similar to the South FRZ Embankment. ***Preliminary modeling suggests that the depth of the cutoff walls could be approximately XX feet.*** **[Note to Reviewers: The modeling to determine the depth of the cutoff walls in the North FRZ is not yet complete. Information will be provided in the 100 Percent Design Report.]** A detailed analysis of the cutoff wall design will be performed during Phase 2 design.

12.5.3 Fill Zone

The Fill Zone is an area north of the left dam abutment where depressions exist downstream of the North Inlet Channel Dike. In this area, the dike will impound the reservoir pool above the existing grade at heights up to 8 feet. The lowest point is at elevation 1,210 feet (NAVD), which is 2 feet below the emergency spillway crest. To raise the reservoir pool into the fill zone, a storm event with a frequency greater than the 100-year, 24-hour event would be required. To raise the reservoir pool up to 6 feet deep in the fill zone, a storm event with a frequency greater than the 500-year, 24-hour event.

The Fill Zone will consist of structural fill placed to an elevation of 1,218 feet (NAVD), which is equal to the maximum reservoir pool during routing of the PMF. The east edge of the Fill Zone will end at the District property line with a 2:1 slope. Placement of the structural fill, in conjunction with the existing fill placed for the dike, would provide a mass of soil downstream of the reservoir pool with a thickness of approximately 100 feet.

12.5.4 Aesthetic Fill

The entire dam structure will be covered with a zone of fill material to modify the aesthetics of the dam. The aesthetic fill will be placed on the dam upstream and downstream of the crest at varying slopes. Within the fissure risk zones (i.e., downstream of the soil cement core) portions of the existing dam will be removed to match the design of the aesthetic fill. The aesthetic fill will not be placed on the embankment crest in order to allow inspections.

Since the aesthetic fill does not serve as a structural component of the dam, the fill will consist of random backfill material (common fill). In addition, the Holocene soils beneath the footprint of the aesthetic fill but outside of the dam footprint will not be over-excavated. Therefore, it is anticipated that some cracks may appear within the aesthetic fill but these would not be considered a dam safety concern.

Aesthetic fill material will also be placed on the Bethany Home Road Dike in a similar manner as placed on the dam. The extent of aesthetic fill on the dam and dike will be accounted for in the hydraulic analysis of the emergency spillway.

12.6 GEOTECHNICAL ANALYSES

The geotechnical analyses presented in this design report address the analyses required for the design of the Phase 1 embankments: the South FRZ Embankment and Transition Embankments. The following sections detail the analyses performed and summarize the results.

12.6.1 General

Geotechnical analyses were completed for the rehabilitation design of the White Tanks FRS No. 3 proposed soil cement section and transition sections in accordance with the AWDR and NRCS standards and regulations and standard engineering practice. Analyses performed included:

- Steady state seepage analyses to estimate the phreatic surface and pore water conditions through/under the embankment;
- Slope stability analyses to evaluate the minimum factors of safety for various design loading conditions;
- Evaluation of the sliding and overturning stability;
- Estimation of immediate and post-construction settlement of the embankment and foundation and potential collapse settlement of foundation soils.
- Evaluation of liquefaction potential;
- Structural beam analysis to estimate the theoretical distance the soil cement section could span a hole in the foundation, potentially caused by fissure erosion; and
- Dispersive soil evaluation for common fill embankment and foundation soil material.

Analyses data, including input/output files, calculation packages, and figures are provided in Appendix G.

12.6.2 Seepage Analyses

Steady-state seepage analyses were performed to estimate the phreatic surface and pore water conditions through/under the FRZ maximum height soil cement section and transition section of the dam. The computer program SEEP/W (Geo-Slope International, 2000) was used to perform the steady-state seepage analyses.

As a flood control structure, White Tanks FRS No. 3 will retain water only during extreme flood events. Floodwater retention time is estimated to be about 13 days. For these loading conditions, it is extremely unlikely that steady state seepage conditions would ever develop. When water is impounded behind FRS No. 3, a phreatic surface within the dam will begin to develop, but before it could reach steady state conditions, the reservoir will be lowered, if not completely emptied. A steady state seepage analysis is considered to be conservative and was performed to model the most extreme seepage conditions envisioned during an extreme flood event. Transient seepage analysis was not performed, as it is considered to be less conservative than steady state seepage. The use of steady state phreatic surfaces for the slope stability analyses is also considered to be conservative, and is discussed later in this section.

Fissure flow was not modeled using SEEP/W because the effective hydraulic conductivity in a potential fissure zone would be extremely high, and is outside the range of values capable of being modeled by SEEP/W that would produce reasonable results. Fissure flow is discussed separately in Section 11.0.

Steady state seepage analyses were performed for two study sections - one representing the FRZ maximum height soil cement section and the other representing the transition embankment for Phase 1 construction. The design geometries of the two study sections are shown in Appendix G as Figures G.1-1 and G.1-2.

For the FRZ maximum height section, an upstream pool elevation of 1216 ft was used, which corresponds to PMF flood conditions with 2 feet of freeboard. Seepage analyses were performed without common embankment fill downstream of the soil cement section, and both with and without the upstream common fill in place. Modeling without common fill is considered to be conservative, and represents a worst-case scenario where common fill has completely eroded, leaving only the soil cement section standing. Two SCB cutoff walls were included in the model, one at the upstream toe of the soil cement embankment, and the other at the downstream toe. The coarse aggregate apron along the upstream toe of the soil cement section and any filter drain along the downstream toe of the existing embankment were ignored in the seepage analyses, because their impact on the overall seepage behavior is considered to be negligible.

For the transition section, the upstream pool elevation was assumed to be same as the Phase 1 design crest elevation of 1213.5 ft (NAVD 88). It is our understanding that during Phase 2 construction, the crest of this transition section will be raised to an elevation of 1218.0 ft (NAVD 88), consistent with the maximum height section of the dam. However, for the analyses presented in this section, we have analyzed only the Phase 1 geometry of the transition section. The zero freeboard assumption was made because the temporary crest elevation is lower than PMF reservoir elevation, and because the transition section can be considered as a temporary cofferdam.

12.6.2.1 Hydraulic Conductivity

The materials modeled in the seepage analyses include the foundation soils to a depth of approximately 80 feet, existing embankment, soil cement material, soil cement-bentonite material, and common fill. The hydraulic conductivity (k) values used for these materials were selected on the basis of site investigations, laboratory testing, and published data on similar materials. A summary of the hydraulic conductivity values used in the seepage analyses are presented in Table 12-2.

Permeability tests were performed on undisturbed tri-axial test specimens taken from shallow foundation soils and the existing embankment as part of the Dam Modifications Investigation in 1998 (Dames & Moore, 2001). Hydraulic conductivity values ranged from 1×10^{-5} to 2×10^{-6} cm/sec for the samples. Laboratory permeability test results are more representative of vertical conductivity values, which are typically less than the horizontal conductivity values. We therefore used an anisotropic ratio (k_H/k_V) of 10 for the foundation soils and a conservative horizontal hydraulic conductivity value of 1×10^{-4} cm/sec. It was conservatively assumed that the mass permeability of Holocene and Pleistocene soils was the same, based on the presence of low-blow count material and sand and gravel lenses in both soils. Holocene and Pleistocene soils were therefore modeled as one uniform layer in SEEP/W.

Based on published data, the hydraulic conductivity of the soil cement material can range from 1×10^{-5} to 1×10^{-8} cm/sec, depending on the fines content of soil in the mix, cement content of the mix, delay time between lift compaction, and whether flow is normal or parallel to the lifts. We selected a hydraulic conductivity of 1×10^{-7} cm/sec for use in modeling, which is representative of a mix with AASHTO A-4 type soil, a soil cement content of 8, and assumes flow parallel to the lifts. Blended on-site soil, which will be used in the soil cement mix, compares most closely with AASHTO A-4 soil.

Horizontal hydraulic conductivity values for the existing embankment and common fill were estimated from permeability test results, general material descriptions, and published correlations. The horizontal hydraulic conductivity value for the SCB cutoff walls was estimated using published data.

12.6.2.2 Seepage Analyses Results

Results from the seepage analyses are presented on Figures G.1-1 for the FRZ soil cement section and on Figure G.1-2 for the transition section. SEEP/W input and output data files are also included in Appendix G.1. Computed phreatic surfaces drop sharply in the soil cement structure and exit through the base of the structure, well upstream from the downstream face. Immediately downstream of downstream cutoff wall, the phreatic surface is located about 20 feet below ground surface, and continues to drop downstream from the dam, largely due to the presence of the cutoff walls. The computed phreatic surface through the soil cement structure is higher for the case where the upstream common fill is ignored. To be conservative, this higher phreatic surface for the maximum height section was used in the slope stability analyses discussed below. Results for the transition section are similar to those for the FRZ soil cement section, with the phreatic surface exiting through the embankment base well upstream of the downstream face, and continuing to drop downstream from the dam. Because there is no exit gradient at the downstream toe for both sections, it appears that piping is not considered a dam safety concern for an extreme flood event.

12.6.3 Slope Stability Analyses

Slope stability analyses were performed for various design loading conditions for the two design study sections discussed above - one representing the FRZ maximum height soil cement section, and the other representing the transition embankment for the Phase 1 construction. Both of these study sections were evaluated for the following loading conditions:

- End of construction case
- Steady state seepage case
- Instantaneous drawdown case
- Pseudo-static seismic case

Slope stability computations were performed using the UTEXAS3 (Wright, 1991) computer program.

12.6.3.1 Shear Strength Characterization

The most critical material property for the slope stability evaluations is the shear strength of the materials. For the purposes of slope stability analyses, White Tanks FRS No. 3 consists of the following materials:

- Existing embankment;
- Soil cement material;
- Common fill; and

- Foundation soils.

The contribution to shear resisting force of soil cement-bentonite material comprising the cutoff walls was conservatively not included in the model. Further, as a conservative assumption, the model for the FRZ soil cement section does not include common fill buttress placed upstream of the soil cement structure.

Generally, for slope stability analyses, shear strengths of the various materials are classified into three categories depending upon the material type and the loading condition to which the material will be subjected. The three broad categories of shear strength are drained, undrained, and post-seismic.

12.6.3.1.1 Drained Shear Strength

Drained shear strength represents the long-term steady state strength of a material assuming fully "drained" conditions. This is the strength mobilized when changes in stress conditions and/or pore pressures are not large enough or sudden enough to induce excess pore water pressures within saturated materials. The drained shear strength is generally the highest shear strength that a material is capable of generating, and generally increases with an increase in confining stress (overburden).

12.6.3.1.2 Undrained Shear Strength

Undrained shear strength represents the short-term strength of a material assuming "undrained" conditions. This strength is applicable only for relatively fine grained materials that are below the phreatic surface and are capable of generating excess pore water pressures when sheared rapidly under conditions that do not allow sufficient time for drainage to occur. The undrained shear strength is generally lower than the drained strength, especially for loosely placed materials.

12.6.3.1.3 Post-Seismic Shear Strength

Post-seismic shear strength represents the material shear strength immediately after an earthquake loading. During a strong seismic event, saturated cohesionless soils such as clean sands and gravels can experience a large loss of strength and stiffness associated with seismically induced pore pressure buildup. This phenomenon, which can lead to slope failure, lateral spreading, and settlement is commonly called liquefaction. If the earthquake loading is large enough to "liquefy" a material, then the post-seismic shear strength is classified as the residual shear strength, which is the lowest shear strength that the material can mobilize under cyclicallyloaded undrained conditions. If the earthquake loading is not large enough to liquefy a material but high enough to re-mold it, then the re-molded shear strength is used as the post-seismic shear strength. Generally, the re-molded shear strength is about 10 to 20 percent less than the peak shear strength estimated under static conditions.

12.6.3.2 Material Properties

Depending on the loading condition, drained, undrained, or post-seismic shear strength values have been modeled. Material properties of the four material types are estimated based on field and laboratory test data that were collected, from published empirical relationships, and from our experience. Material properties used in the slope stability analyses are presented in Table 12-3.

Unit weights for the various materials were selected based on the results of moisture-density tests or Standard Proctor tests, as applicable. Tri-axial tests were performed on three undisturbed samples and three re-molded samples taken from shallow foundation soils as part of the Dam Modifications Investigation in 1998 (Dames & Moore, 2001). Strength parameters from the undisturbed and remolded test samples were used as the basis for selecting lower-bound strength envelopes, as shown on Figure G.2-8. Strength parameters for the soil cement material were selected based on the results of mix design testing. The drained strength was taken as half the 28-day unconfined compressive strength of 1000 psi, and the undrained strength was estimated as half the 1-day strength of 400 psi, which is typically 50 to 60 percent of the 7-day strength. The friction angle was conservatively assumed to be zero.

12.6.3.3 Loading Conditions and Corresponding Shear Strengths

The FRS No. 3 was evaluated for four general loading conditions: steady-state drained, end of construction, post-seismic, and instantaneous drawdown.

12.6.3.3.1 Steady-State Drained Case

The steady-state drained loading condition was used to estimate long-term static stability under steady state pore pressure conditions. As discussed above, we have conservatively assumed a steady-state phreatic surface through the FRS No. 3, even though steady state conditions are unlikely to ever develop at the structure. For this case, effective stress drained shear strengths are used for all materials.

12.6.3.3.2 End of Construction Case

The end of construction loading condition was used to estimate short-term stability immediately after construction. For this case, "unconsolidated undrained" strengths, which are typically less than drained strengths, are used for all fine-grained materials that are placed wet, which includes common fill placed downstream of the soil cement structure, and common fill placed upstream of the existing embankment at the transition section. It was assumed that foundation soils will be slightly moist or dry during construction and were therefore modeled using drained shear strength. The soil cement material was modeled using an estimated 1-day unconfined compressive strength value of 200 psi to simulate during-construction conditions.

12.6.3.3.3 Post-Seismic Case

The post-seismic loading case was used to estimate stability immediately and after the design earthquake loading. FRS No. 3 was considered not to be susceptible to liquefaction for two main reasons: (1) clean sands and gravels are not horizontally or vertically extensive throughout the site, and (2) the probability of a major earthquake occurring at the same time as an extreme flood is extremely remote. Seismicity and liquefaction are further discussed in Section 12.6.6. Therefore, in accordance with AWDR regulations, a pseudo-static seismic analysis was performed. Based on the results of the seismic exposure evaluation study completed by AMEC (AMEC, 2002) a peak ground acceleration (PGA) of 0.1g was selected, and 60% of 0.1g was used as the pseudo-static coefficient. The post-seismic shear strength of the materials was estimated by reducing the static drained shear strength by 20 percent to account for potential re-molding of the materials due to seismic shaking. A phreatic surface was not modeled for the pseudo-static analyses because of the remote possibility of an extreme flood and major earthquake occurring simultaneously.

12.6.3.3.4 Instantaneous Drawdown Case

The instantaneous drawdown loading condition was used to estimate stability during rapid drawdown of the reservoir from steady state conditions to an empty reservoir. The stability of the maximum height soil cement section was checked assuming no upstream common fill under instantaneous drawdown conditions. Because of the extremely high strength of the soil cement material, it was not possible to obtain a factor of safety value using the limit equilibrium procedure. However, using simplified hand calculations, we were able to confirm that the factor of safety for this case was very high.

12.6.3.4 Minimum Acceptable Factors of Safety

The minimum acceptable factors of safety for the various loading conditions were established based on AWDR regulations:

- End of construction case – Minimum FS = 1.3
- Steady state seepage case – Minimum FS = 1.5
- Instantaneous drawdown – Minimum FS = 1.2
- Pseudo-static seismic case – Minimum FS = 1.2

12.6.3.5 Slope Stability Analyses Results

Results from the slope stability analyses, for the various loading conditions, are presented on Figures G.2-1 through G.2-3 for the soil cement section and on Figures G.2-4 through G.2-7 for the transition section. Input and output data files are also included in Appendix G.2. These figures show the computed minimum factor of safety values along with the associated critical shear surfaces for the various loading conditions. A summary of the computed minimum factor of safety values for the two study sections is presented in Table 12-4.

Because the shear strength of the soil cement material is extremely high, the computed critical shear surfaces for all loading cases do not pass through the soil cement embankment. The stability of the stand-alone soil cement embankment obviously achieves all slope stability criteria, and therefore only needs to be evaluated for sliding and overturning stability (see Section 12.6.4). The steady-state phreatic surfaces, computed in the seepage analyses, are relatively deep on the downstream side for both sections. Therefore, the downstream critical shear surfaces for the steady state seepage case, for both sections, are not impacted by the location of the phreatic surfaces.

All computed minimum factor of safety values, for both design sections, are higher than the minimum acceptable factor of safety values dictated by the AWDR regulations.

12.6.4 Sliding and Overturning Stability Analyses

12.6.4.1 Introduction and Background

Analyses been performed to evaluate sliding and overturning stability for the FRS No. 3. The fundamental component of the dam – the soil cement embankment – was modeled as a conventional concrete gravity dam for these analyses. Three possible loading conditions, as outlined in "Design Criteria for Concrete Arch and Gravity Dams" (USBR, 1977), were considered:

- Usual loading condition: normal reservoir elevation with applicable loads;
- Unusual loading condition: maximum design reservoir elevation with applicable loads; and
- Extreme loading condition: the usual loading condition plus the effects of the Maximum Credible Earthquake (MCE).

However, because White Tanks FRS No. 3 will function as a flood control dam only, and therefore will not impound a permanent pool. The following assumptions are therefore made with respect to the three loading conditions:

1. The usual loading condition corresponds to an empty reservoir;
2. The unusual loading condition corresponds to the PMF reservoir elevation; and
3. The extreme loading condition corresponds to an empty reservoir plus the design seismic load.

However, because the usual and extreme loading conditions involve an empty reservoir, and subsequently lack the necessary hydrostatic forces to potentially induce sliding or overturning failure, these two loading conditions were judged to not be applicable and were therefore not considered in the analyses. The unusual loading condition corresponding to the PMF reservoir elevation was deemed to be the critical loading condition for White Tanks FRS No. 3, and was therefore the only loading condition considered in the overturning and sliding stability analyses.

12.6.4.2 Sliding and Overturning Stability Criteria

We have reviewed various criteria for sliding and overturning stability analyses, including those of the ADWR (ADWR, 2000), NRCS (NRCS, 1990), the U.S. Army Corps of Engineers (COE, 1995), the U.S. Bureau of Reclamation (USBR, 1977), and the Federal Energy Regulatory Commission (FERC, 2002). The most stringent minimum sliding stability FOS of 2 (criteria of the COE and FERC) was used for comparison with our computed sliding stability FOS. The COE criteria that the location of the resultant force be located in the middle half of the base for the unusual loading condition was used to evaluate overturning stability.

12.6.4.3 Model Development

The soil cement embankment will have a 35-ft height, 10-ft crest width at Elevation 1218, and a 58-ft base width. The upstream and downstream faces will have a 0.6:1 horizontal to vertical slope. SCB cutoff walls will be located at the upstream and downstream toes and will extend 30 ft beneath the soil cement base. Common fill will be placed on the upstream slope of the soil cement embankment at a 2:1 horizontal to vertical slope, and will be placed relatively flat on the downstream slope. The maximum reservoir elevation is estimated as elevation 1216.0 ft (NAVD 88).

The internal angle of friction between the soil cement section and the foundation soil was assumed to be 35°. This corresponds to the mid-range of the strength envelope for foundation soil, as shown on Figure G.2-8. A 30° friction angle for foundation soil was conservatively used for slope stability analyses; this lower friction angle corresponds to the lower-range of the same strength envelope. A higher friction angle of 35° was used for sliding and overturning stability analyses because it was assumed that the upper 5 feet of foundation soil would be heavily densified during compaction and as a result of the load of the overlying soil cement structure.

The analyses were performed using the conservative assumption that the soil cement embankment be treated as a stand-alone structure, without common fill on the upstream and downstream slopes.

A triangular uplift seepage pressure distribution was conservatively applied at the base of the soil cement structure, assuming 33 feet of pressure head at the upstream toe of the soil cement structure, and ignoring the effect of the cutoff walls. In reality, the cutoff walls would drive the seepage flow path down and around the walls, significantly reducing the seepage uplift pressure applied at the base of the structure.

Pressurized flow through a fissure was considered, but was judged to not add appreciable uplift pressure based on several considerations. First, fissure flow would occur at least 30 ft below the base of the structure, below the cutoff walls. Uplift pressure applied at the base of the structure would be greatly reduced due to head loss. Second, uplift pressure from fissure flow would be applied over a relatively small distance laterally along the structure base, considering the relatively small width of a potential fissure. Compared to the laterally-extensive uplift pressure from seepage along the entire length of the FRZ, uplift pressure associated with fissure flow could be considered as a point load. At worst, such a point load may result in localized stresses, but considering the 3-dimensional effect of the mobilized sliding and overturning resistance of the soil cement structure (which can be considered to act as a rigid beam), these localized stresses are judged not to be significant.

12.6.4.4 Results

Pressure and weight forces, moment arms, and moment forces for each segment of the design model have been calculated. Total resultant forces in the horizontal plane were calculated, and the resisting forces were divided by the driving forces to calculate the sliding FOS. The overturning stability was calculated by dividing the sum of moments about the downstream toe by the sum of vertical forces. The location of the resultant force was compared with the overturning stability criterion established by the COE. The results are presented in the table below. Calculations are provided in Appendix G3.

Analysis	Result	Criteria
Sliding FOS	2.1	2.0
Location of Resultant Force (overturning)[1]	22 feet	Middle ½[(2)]

Notes:

a. As measured from the downstream toe.

b. The middle third of the base would be from 17.33 to 34.67 feet.

It should be noted that White Tanks FRS No. 3 meets sliding and overturning stability criteria under the most conservative of assumptions. Therefore, it can reasonably be inferred that if the effects of the cutoff wall, common fill surcharge, and 10-foot embedment of the soil cement structure were added into our analyses, the criteria would be achieved by even a greater margin.

12.6.5 Settlement Analyses

An evaluation of the settlement of the soil cement embankment section was made and can be broken into two components: settlement of the embankment materials, and settlement of the foundation soils beneath the embankment. Settlement of the soil cement material, due to the layered placement and relatively high compressive strength (1000 psi) will take place as the embankment is constructed, and post-construction settlement should be negligible (<1 inch). Immediate settlement of the common fill embankment material will also take place during construction, as part of the compaction process, and is also considered to be negligible (<1 inch).

Typically, the settlement of foundation soils would consist of both elastic compression and consolidation. However, given the great depth to the current water table and the resulting thickness of unsaturated foundation soils (approximately 300 ft), only elastic compression, or immediate settlement, will be considered. The total immediate settlement was estimated using an average soil elastic modulus of 8 ksi, assuming that a mid-range modulus value for Class 2 calcareous soil (Beckwith and Hansen, 1982) is representative of foundation soils. Immediate foundation settlement was estimated to be 6 inches for the soil cement and common fill embankment loading. Post-construction elastic settlement of foundation soils is considered to be negligible.

Finally, recognizing the unsaturated condition of the foundation, we have considered that collapse settlement that could occur in the foundation soils during flood inundation of the FRS. However, considering an estimated travel distance of the wetted front of 20 feet during the 13 day inundation period, and the presence of the upstream SCB cutoff wall, it is judged that foundation soils beneath the soil cement embankment will not become saturated; thus there is no calculated collapse induced settlement.

Settlement calculations are provided in Appendix G.4, and are summarized in Table 12-5. Based on the settlement results, it is recommended that soil cement and common-fill material quantities be increased to account for 6 inches of potential settlement during construction.

12.6.6 Seismic Load and Liquefaction Potential Analysis

A limited seismic evaluation was performed for White Tanks FRS No. 3 by estimating seismic loads and the likelihood of liquefaction. A seismotectonic study and quantitative liquefaction analysis were not performed, based on the following considerations:

- The structure will impound water only during extreme flood events, and only for a brief duration (less than 13 days);
- The likelihood of an extreme flood event and a strong earthquake occurring simultaneously is extremely remote; and
- A design earthquake is judged to be capable of causing only minimal damage or consequences to FRS No. 3, based on a low estimated peak ground acceleration of 10 percent g, the relatively high strength of the soil cement structure, and the allowance for some damage to the common fill embankment based on the intended aesthetic function of the common fill.

12.6.6.1 Seismic Load

A maximum horizontal peak ground acceleration (PGA) of 10 percent g is recommended for the White Tanks FRS No. 3 site in Table 4 AMEC's Seismic Exposure Evaluation report (AMEC, 2002). A copy of Table 4 from this report is included in Appendix G. A PGA of 10 percent g appears to be appropriate for the site, as it is consistent with, or higher, than other published seismic hazard values for the area. The National Earthquake Hazards Reduction Program (NEHRP) identifies the Phoenix area with a 10 percent g seismic hazard rating. Recent United States Geologic Survey (USGS) seismic hazard maps depict the Central Arizona and the Phoenix area with a seismic hazard ranging from of 6 percent g to 8 percent g, with a 10 percent probability of exceedance (a 2 percent probability of exceedance equates to about 10 percent g). The Arizona Geologic Survey places the Phoenix area in the "Low" seismic hazard category on a four-tiered subdivision including low, low to moderate, moderate, and high.

An adjusted PGA of 6 percent g is used as the pseudo static coefficient and as the seismic design load for the structure. Rule 12-15-1216 (B)(2)(c) of the Arizona Administrative Code states that 60 percent of the maximum peak bedrock acceleration at the site shall be used for pseudo static stability analysis. It was also judged appropriate to use 6 percent g as the seismic design load for the spillway.

12.6.6.2 Liquefaction Potential

Liquefaction is a concern when the following conditions are present:

- Loose, saturated sands, sensitive silts, or quick clays are present;
- Such materials are subjected to shear deformations from seismic loading or other loading sources.

The likelihood of a liquefaction failure occurring in the common fill embankment or embankment foundation that would result in reservoir release during an extreme flood event is judged to be very low. As mentioned previously, the likelihood of an extreme flood event (that produces saturated foundation and embankment soils) and a strong earthquake (that produces shear loading) occurring simultaneously, is extremely remote. Secondly, if any liquefaction did occur during such an unlikely event, it would likely occur only in local lenses or zones of loose saturated sands or low-blow count sandy silt or silty sandy silt material. Such material is indeed present in the foundation, especially at shallow depths, but is not laterally or vertically extensive enough to warrant concern for liquefaction-induced dam failure. Material in the soil cement structure foundation footprint will be excavated to a minimum depth of 10 feet (or deeper as necessary), removing such deleterious material present at shallow depths, which includes soils that have exhibited a low to moderate collapse potential.

Based on the foregoing discussion, the common fill embankment and foundation material are judged to have a very low susceptibility to liquefaction, and no additional liquefaction analysis is warranted.

12.6.7 Dispersive soil evaluation

Soil samples from borrow fill and foundation soils were not tested for the presence of dispersive soils. White Tanks FRS No. 3 will be used solely for flood-control purposes, with no permanent pool impounded by the embankment, and with no permanent phreatic surface extending through the structure. Steady state seepage analyses indicate that the phreatic surface from the design flood does not advance all the way through the soil cement embankment, nor is there an exit gradient in the embankment. In addition, the presence of the two 30-ft deep cutoff walls provide defense against dispersive action. It is therefore judged that the impacts of dispersive soil, if even present in the foundation soils or the future embankment, would be negligible or minimal.

12.7 DESIGN ANALYSES

12.7.1 Soil Cement Beam Analysis

An analysis was performed to determine the maximum theoretical length that the soil cement section could span if a hole developed in the foundation, potentially caused by fissure erosion. Conservatively, the soil cement section was modeled as a simply supported, uniformly loaded beam. Laboratory test results were used to estimate the modulus of elasticity, unit weight, and compressive strength as 1,000 ksi, 125 pcf, and 1,000 psi, respectively.

The only load considered in the analysis was the dead weight of the soil cement section. Span lengths varying from 20 feet to 80 feet were evaluated and the shear stresses, moments, and displacements were determined at 2-foot intervals along each beam. The maximum tensile stress in each span was also determined based on the standard beam flexure relationship. The maximum tensile stress was compared with a typical range of tensile stress for soil cement materials. This range was based on two publications (Portland Cement Association (PCA, 1988) and the U.S. Bureau of Reclamation (USBR, 1990)), which compared the tensile strength of concrete with the compressive strength of concrete. Based on these publications, the limits for the allowable tensile strength of concrete that was used in the analyses ranged from 10 percent of its compressive strength (100 psi) to 5 times the square root of the its compressive strength (158 psi). Based on these allowable tensile stress limits, the soil cement section is estimated to span about 61 to 76 feet. The actual length that this material could span is most likely somewhat less due to anticipated shrinkage cracking, considering that the above analysis only considers intact soil cement material. Considering the most conservative estimated width of an eroded fissure is approximately **XX feet** (See Section 11.0), the geometry and estimated strength of the soil cement section appear to be adequate. **[Note to Reviewers: The results of the fissure erosion modeling are provided separately. Details will be incorporated in this report for the 100 Percent submittal.]**

The analyses are included as Appendix G.6, which includes plots of shear, moment, and displacement for the various span lengths. Appendix G.6 also includes a plot of span length versus tensile strength in which the calculated tensile strength is shown along with the estimated allowable range of tensile strength.

12.7.2 Graded Filter Design

The graded filter is located downstream of the interface between the South FRZ Embankment and Transition Embankments. Calculations have been performed to match the filter with the structural fill and foundation materials. Detailed calculations are provided in Appendix G. **[Note to Reviewers: The filter match calculation is not included with the 90 Percent Submittal. The calculation and assessment of the filter design will be included with the 100 Percent Submittal.]**

13.0 EMERGENCY SPILLWAY DESIGN

13.1 GENERAL

The emergency spillway will be constructed during Phase 2 of the project. A significant level of detail is provided in this report because of the implications that the spillway design has on the setting the crest elevation used for the Phase 1 design. SITES modeling has been performed during Phase 1 design to provide an understanding of soil erodibility in the emergency spillway. Additional analyses to be performed during Phase 2 design of the emergency spillway include: structural, stability, and soil cement erodibility.

13.2 EXISTING CONDITIONS

The White Tanks FRS No. 3 emergency spillway is an earth-cut spillway located at the right dam abutment. The spillway has a width of 800 feet. The spillway crest is turned approximately 25 degrees downstream from the dam centerline. The spillway cut is sloped upstream and downstream from the crest to match existing grade with slope of 0.2 percent and 0.45 percent, respectively. The spillway crest is at an elevation of 1,212 feet (NAVD 88).

The existing Bethany Home Road Dike is located downstream of the spillway crest and was originally intended to contain spillway flows from the spillway to the Beardsley Canal. The dike was not constructed as shown on the design drawings. The dike is no longer entirely located on District property.

13.3 DESIGN CONFIGURATION

The emergency spillway design incorporates a significant modification from the current configuration. This modification will be made to improve the hydraulics of the structure and address potential erosion issues that exist for the earth-cut spillway. The modified structure will consist of trapezoidal weir, emergency spillway channel, and upstream excavation. The trapezoidal weir will be installed across the existing spillway and extend from the right dam abutment in a curved shape. An apron will be installed at the downstream toe of the weir to minimize erosion during discharge events. The Holocene soils within the footprint of the spillway weir and apron will be removed and replaced with structural fill. A concrete wall cutoff wall and rip rap will be installed at the end of the apron for erosion protection. A Details of the emergency spillway structures are provided in Appendix L.

13.3.1 Spillway Weir

The trapezoidal weir will have a total crest length of approximately 1,223 feet. The weir will be constructed of soil cement with a crest width of 10 feet, and have 1:1 side slopes. The overall height of the weir above the downstream apron will vary from 4 to 8.5 ft to match the slope of the emergency spillway channel. Discharge from the reservoir is forced to pass through critical depth over the weir, thus creating a control section for hydraulic analyses. Details of the hydraulic analyses are provided in Section 9.0 of this report.

It is anticipated that the upstream and downstream slopes of the weir will be covered will aesthetic fill in a manner similar to the dam. This fill should be maintained at slopes no flatter than 4:1 and 10:1 on the downstream and upstream slopes, respectively, to maintain spillway hydraulics.

13.3.2 Spillway Apron

An apron will be constructed at the downstream toe of the weir structure to create and contain the hydraulic jump. The apron is a critical component of the design because it contains the most energetic portion of flow on a non-erodible surface. Flow leaving the apron will be in the subcritical flow regime to minimize erosion in the emergency spillway channel.

The apron will be constructed of soil cement with a thickness of 2 ft, include blocks protruding upward into the flow to force the hydraulic jump to occur, and extend beyond the toe of slope created by the aesthetic fill. A reinforced concrete cutoff wall will be constructed at the downstream end of the apron to provide protection against erosion within the emergency spillway channel. In addition, rip rap will extend beyond the apron and wall for additional protection.

13.3.3 Emergency Spillway Channel

The emergency spillway channel consists of an excavated channel extending away from the spillway weir and apron at a slope of 0.5 percent. A channel slope of 0.5 percent maintains subcritical flow within the channel. Hydraulic modeling also indicates that overland flow downstream of the channel will remain subcritical. Rip rap will be installed along the dam face and toe to protect against potential erosion during a spillway discharge event. The extent, depth, and size of rip rap will be determined during Phase 2 design.

13.3.4 Bethany Home Road Dike

The Bethany Home Road Dike will be relocated onto District property and aligned parallel to the left bank of the spillway channel. The dike will be constructed to contain the peak flow during the PMF and have a maximum height of 10 feet above existing grade. The dike will extend approximately 300 feet past the end of the emergency spillway channel to a point designated by the District. Erosion protection will not be placed on the dike. Details of the Bethany Home Road Dike are shown on the figures provided in Appendix L.

13.3.5 Upstream Excavation

The area upstream of the spillway will be excavated to provide sufficient approach depth to the weir control section. This material will be used for construction of the embankment, as appropriate. Details of the hydraulic evaluation related to the upstream excavation are presented in Section 9.0.

13.3.6 Aesthetic Fill

The embankment, spillway, and Bethany Home Road Dike will be covered with aesthetic fill. The aesthetic fill was assumed not to erode for purposes of hydraulic routing. However, aesthetic fill material will likely be washed away during an emergency spillway discharge and require maintenance activities to replace.

13.4 EMERGENCY SPILLWAY EROSION MODELING

13.4.1 General

The Water Resource Site Analysis Computer Program (SITES) was used to estimate the erosion depth of soils in the emergency spillway channel. The depth of erosion will provide guidance in estimating the depth of cutoff wall required at the downstream edge of the apron. The design includes a reinforced-concrete cutoff wall that remain stable following erosion within the spillway channel during the maximum design spillway discharge.

13.4.2 Model Parameters

The discharge hydrograph through the emergency spillway was developed using the TR-20 models discussed in Section 10.0. The PMF hydrograph resulting in the maximum water surface elevation of 1,216.5 ft (NAVD 88) a peak flow of approximately 30,000 cfs was used in the SITES modeling. As discussed earlier in Section 13.0, the emergency spillway is designed to control the emergency spillway discharge and ensure subcritical flow within the spillway channel.

The emergency spillway intersects both Holocene and Pleistocene soils. A review of the soil profile along the cutoff wall alignment indicates a variable depth of Holocene. From approximately the centerline of the spillway to the right end, Holocene soils will be removed during construction of the emergency spillway channel. From approximately the centerline of the spillway to the left end, the depth of Holocene soils ranges from 0 to 5 feet below the finished emergency spillway channel.

Two separate SITES models were utilized in this analysis because of the varying soil conditions along a typical cross section of the spillway. One model was used to analyze erosion occurring only in the Holocene and one analyzing the Pleistocene erosion.

The typical parameters required in each of the SITES models include:

- Spillway outflow hydrograph
- Spillway dimensions
- Surface Conditions
- Soil Properties

13.4.2.1 Spillway Outflow Hydrograph

The 72 hour PMF outflow hydrograph developed by TR-20 was used as input into the SITES model. The SITES model was not used for flood routing.

13.4.2.2 Spillway Dimensions

The average spillway dimensions are required input into the SITES model and are used to develop a normal depth unit flow over the spillway. A and width of 850 feet and side slope ratio of 2:1 was used as input into the model. These dimensions are taken just downstream of the proposed soil cement drop structure; the structure will ensure subcritical flow within the spillway channel.

13.4.2.3 Surface Conditions

There are four surface parameters that are used as input to into the SITES model to define the surface of the modeled spillway. The surface conditions used in this analysis are defined below and summarized in Table 13-1.

1. Vegetal Retardance Curve Index is the flow resistance for the reach indicated by the beginning and ending stations on the same line. The flow resistance of the reach was entered as a Manning's n of 0.02 corresponding to minimum vegetation cover.
2. Vegetal Cover Factor describes the uniformity of vegetal cover in the immediate vicinity of the erodible bed. The cover factor ranges from zero for non-vegetated surfaces to 0.87 for typical turf grass sod covers. The vegetal cover factor used in this analysis was 0.3 corresponding to light vegetation.
3. Maintenance Code describes the overall uniformity of the cover in the channel. The acceptable values and their meaning are:
    a) Uniform cover over the entire area subject to flow;
    b) Minor discontinuities in the cover; and
    c) Major discontinuities in the cover.

    A maintenance code of 1 was used in this analysis. A maintenance code of 1 may be used for non-vegetated conditions, since the cover is uniformly non-existent as will be indicated by other parameters.
4. Potential rooting depth is the depth to which roots could reasonably penetrate under good growing conditions. The potential rooting depth is used in the identification of cover conditions susceptible to sod stripping or rafting by the flow. Therefore, this parameter becomes significant for computations only when the value is less than approximately one foot. The Potential rooting depth was set to 0.5 for the Holocene and 0 for the Pleistocene in this analysis.

13.4.2.4 Soil Properties

There are five soil properties that are used as input to into the SITES model to define the sub-surface soil properties of the modeled spillway. The soil properties used in this analysis are defined and summarized in Table 13-1.

1. Plasticity index of the material being described. The plasticity index was estimated from previous soil investigations to be 5 for the Holocene and 13 for the Pleistocene.

2. Dry bulk density of the material being described in pounds per cubic foot. The dry bulk density was estimated from previous soil investigations to be 110 for the Holocene and 117 for the Pleistocene.

3. Headcut erodibility index of the material being described. The headcut erodibility index is a measure of the strength of the material and its resistance to headcut advance. It ranges from 0.01 for sand to greater than 10,000 for massive rock. Erodibility factors have been estimated for both soils based on information collected during the geotechnical investigation which included soil gradations, blow counts, EFA testing, and field vertical jet testing (VJT). A subsequent technical memorandum titled Erodibility of Spillway authored by George Annandale was used to determine the input values used. A 0.001 erodibility index was used for the Holocene and a range from 0.01 to 0.4 was analyzed for the Pleistocene.

4. Percent clay of the material being described. Used in computing the surface detachment rate coefficient for the material. The percent clay was estimated from previous soil investigations to be 20 for the Holocene and 35 for the Pleistocene.

5. Representative diameter in inches for the material being described. The diameter being sought is the diameter representative of the "particle" being detached during erosion. The representative diameter was estimated as the d75 recorded from previous soil investigations a value of 0.004 inches was used for the Holocene and 0.008 inches for the Pleistocene.

13.4.3 Model Results

The SITES models and results are provided in Appendix I. The SITES modeling in the emergency spillway provided the following results:

- The modeling shows the Holocene soils will be completely eroded away within the downstream Emergency Spillway Channel. It is anticipated that the Holocene soils will erode to the depth of Pleistocene within the channel downstream of the spillway weir structure.
- The modeling shows the Pleistocene soils will erode to depth ranging from approximately 3 to 5 ft at the downstream end of the spillway apron structure. Greater erosion depths may be seen downstream of the Emergency Spillway Channel.

13.4.4 Design Recommendations

The cutoff wall will be constructed at the downstream end of the spillway apron to protect the spillway weir and apron structures from failure during the design flow event. Excavation for the emergency spillway will expose Pleistocene soils along the right half of the cutoff wall alignment. The right half of the cutoff wall alignment will be constructed through Holocene soils with depths ranging from 0 to 5 feet. The cutoff wall design will extend the cutoff wall 5 feet into the Pleistocene soils. The cutoff wall depth will range from 5 to 10 feet along the length of the spillway, depending on the depth of Holocene soils.

The intent of the cutoff wall is to prevent a failure of the spillway structure during the design storm event. Erosion occurring downstream and away from the spillway weir structure can be addressed through maintenance activities following storm events. Details of the cutoff wall are shown on the figures provided in Appendix L. Structural calculations required for the design of the wall will be provided with the Phase 2 design.

14.0 OUTLET WORKS

14.1 EXISTING CONDITIONS

White Tanks FRS No. 3 currently has 3 outlets, identified as the North, Central, and South Outlets. The outlets are corrugated metal pipes constructed through the earthen embankment. The outlets were extended and had diaphragm filters installed in 2001 as part of the Interim Dam Safety Project. Each outlet has a mechanically operated slide gate covered by a trash rack on the upstream end. Details of the location and diameters of the existing outlets are presented in Table 14-1.

14.2 CLOSURE OF EXISTING OUTLETS

14.2.1 Phase 1

The North and Central Outlets are located in the area impacted by Phase 1 construction activities. Since both outlets are downstream of the proposed South FRZ Embankment, removal of the outlets is not necessary. Therefore, the design approach for closure of the outlets consists of leaving the outlets in place and filling each with cement grout. The diaphragm filters and drain pipes will remain in place but are not longer required for safe operation of the dam.

During excavation for the embankment construction, a section of this outlet will be intersected and removed. Prior to their removal and decommissioning of the slide gate, the conduits will be plugged and filled with grout from the downstream end. When the grout has hardened sufficiently (in accordance with the specifications), the upstream portion of the outlets will be removed in accordance with the excavation plan. The grout will consist of a cellular-concrete grout, which is a positive-filling (non-shrink) grout. Traditional grout is not recommended due to the potential for shrinkage after placement and the higher cost. The diaphragm filter and drain pipes will be abandoned. Details of the existing outlet decommissioning are shown on the plans.

The slide gates, control mechanisms, and trash racks will be removed and preserved for future use by the District. Previous studies have indicated that the outlet pipe is potentially coated with asbestos. Special conditions must be met for its removal and handling. The South Outlet will remain operational during construction and until the proposed new outlet works are operational.

[Note to reviewers: The concept of grouting the entire pipe and prior to removal has been included only recently. The plans may not reflect this concept in the 90 Percent submittal.]

14.2.2 Phase 2

Phase 1 includes the construction of the new outlet works, which consists of a gated outlet and principal spillway. Therefore, the South Outlet will be decommissioned as part of the Phase 2 project. Due to the complexities involved with breaching the dam to allow removal of the outlet, the South Outlet will be closed in the same manner as the North and Central Outlets, by filling the conduit with cellular-concrete grout. The major difference with the closure of the South Outlet is that the diaphragm filter and drain installed during the Interim Dam Safety Project will remain operational. The filter and drain will be maintained because the conduit, although filled with grout, will remain within the modified embankment. The drain pipes will be extended to the limits of the aesthetic fill placed during Phase 2 construction. Details of the closure design will be provided with the Phase 2 Design Report.

The slide gates, control mechanisms, and trash racks will be removed and preserved for future use by the District. Previous studies have indicated that the outlet pipe is potentially coated with asbestos. Special conditions must be met for its removal and handling.

14.3 DESIGN OF NEW OUTLET WORKS

The proposed new outlet works will consist of 2 separate conduits through the embankment: the Gated Outlet and Principal Spillway. The Gated Outlet consists of a 48-inch reinforced concrete-cylinder pipe (RCCP) with a sluice gate constructed on the upstream end. The Principal Spillway consists of a 48-inch RCCP with a riser structure constructed at the upstream end. The Principal Spillway conduit is also connected to a bypass conduit with a sluice gate constructed at the upstream end. Both conduits discharge to a connected concrete stilling basin designed to dissipate the flow energy prior to entering the outlet channel. The outlet channel will convey outlet works discharges to the existing wash located adjacent to the Beardsley Canal.

The outlet works will be through the South FRZ Embankment. The outlet works will be installed on a soil cement foundation. The existing soils beneath the soil cement will be excavated to remove the Holocene soils in the same manner as detailed for the South FRZ Embankment. The conduits will be partially encased within the structural and common fill, and fully encased within the soil cement as shown on the plans. It was determined seepage along the outside of the outlet conduits would be minimal since the conduits will be constructed through the soil cement embankment and be surrounded by concrete. Therefore, a diaphragm filter around the outlet conduits would not be necessary. Details of the outlet works design are shown on the plans.

14.3.1 Gated Outlet

The gated outlet will be constructed with an upstream invert elevation of 1,197 feet (NAVD 88). The inlet structure at the upstream end of the conduit will consist of a concrete encased steel pipe, sluice gate, and trash rack. The conduit consists of RCCP for most of its length, but will be a steel pipe within the inlet structure.

The trash rack will be installed to prevent debris from clogging the conduit. Design calculations performed for the trash rack include the following:

- Flow velocity calculations to verify the flow velocities through the rack do not exceed 2.5 feet per second, as per NRCS criteria.
- Structural calculations under full head and complete (100 percent) blockage of the trash rack.

The sluice gate will be operated from the embankment crest with a manually controlled mechanism installed on the embankment side slope. Supports for the gate mechanism will be installed on the embankment slope. Details of the slide gate and trash rack are shown on the design drawings. Details of the trash rack design are presented in the calculation packages provided in Appendix H.

The discharge rating curve for the Gated Outlet is presented in Table 14-2 and on Figure 14-1. The Gated Outlet has sufficient capacity to drain down the reservoir from the emergency spillway crest elevation in approximately 7 days. A vent pipe is installed at the upstream end of the conduit and extends to the crest of the embankment.

14.3.2 Principal Spillway

The Principal Spillway will consist of an NRCS-type riser with an inlet elevation of 1,200 feet (NAVD 88). The inlet elevation has been set at this elevation to be above the 100-year sediment pool. The riser structure will be constructed of reinforced concrete and incorporate trash racks at the inlet. The foundation of the riser structure will consist of soil cement, which will be constructed in a manner similar to the soil cement core of the South FRZ Embankment. The principal spillway conduit will connect to the riser at the based of the structure. The Principal Spillway has sufficient capacity to drain down the reservoir to elevation 1,200 ft (NAVD 88) from the emergency spillway crest elevation in approximately 6.5 days. The discharge rating curve for the Principal Spillway is presented in Table 14-2 and on Figure 14-1. Details of the riser design are provided in the plans.

The Principal Spillway will be blocked during construction to reflect the requirements detailed under the Interim Condition and Future Condition (Principal Spillway Closed). Uncontrolled outflow from the reservoir through the spillway cannot occur until construction of a downstream conveyance channel. The method used to block the Principal Spillway will consist of the following:

- Steel plates will be bolted over the inlet to the spillway at elevation 1,200 ft (NAVD 88).
- Epoxy grout will be inserted around the edges of the plate to minimize flow past the plates. It is anticipated that an minor amount of flow will pass around the plates and through the conduit.

[Note to reviewers: The design concept for blocking the Principal Spillway is not reflected in the plans included with the 90 Percent submittal. The design will be finalized for the 100 Percent submittal.]

14.3.3 Principal Spillway Bypass Gated Outlet

During the Interim Condition and Future Condition (Principal Spillway Closed), the ability to discharge water through the Principal Spillway conduit will be achieved with the Bypass Gated Outlet. The Bypass consists of a separate conduit and sluice gate system connected to the Principal Spillway conduit. The invert elevation of the gated bypass outlet is 1,197 feet (NAVD 88). A trash rack will be installed over the sluice gate. The design of the sluice gate and trash rack will be the same as that detailed in Section 14.3.1 of this report. A vent pipe is installed at the upstream end of the conduit and extends to the crest of the embankment.

The discharge rating curve for the Bypass Gated Outlet is presented in Table 14-2 and on Figure 14-1. The Bypass Gated Outlet has sufficient capacity to drain down the reservoir from the emergency spillway crest elevation in approximately 7 days. The combined flow capacity of the Gated Outlet and Bypass Gated Outlet allows for drain down of the reservoir in approximately 3.5 days. A vent pipe will be installed at the upstream end of the Principal Spillway conduit to protect against pressure buildup during flow through the Bypass Gated Outlet.

14.3.4 Outlet Works Conduit

The Outlet Works conduit will consist of 48-inch diameter reinforced concrete-cylinder pipe (RCCP). The RCCP is designed to be water-tight and will extended from the upstream inlet structures to the downstream stilling basin. Within the soil cement core of the embankment, the conduit will be fully-encased in concrete. Within the earth-fill sections of the embankment the conduit will be partially encased in concrete. The concrete encasement is used to allow for proper compaction around the conduits. The conduits and encasement are constructed on a foundation of soil cement, which rests on Pleistocene soils. Details of the Outlet Works conduit design are shown on the plans.

14.3.5 Stilling Basin

The impact stilling basin works to dissipate the energy of the discharge flows from the two outlet conduits and reduce velocities entering the downstream outlet channel. The stilling basin structure consists of two typical basin designs sharing a common wall. A concrete baffle constructed opposite the conduit opening dissipates the flow, while the walls contain the flow in the basin. The invert elevation of the outlet conduit at the stilling basin and the invert of the downstream edge of the stilling basin is 1,190 feet (NAVD 88). Details of the stilling basin design are shown on the design drawings.

14.3.6 Outlet Channel

The outlet channel will convey discharge flows from the stilling basin to a natural wash downstream of the embankment. The channel will be excavated at a 0.5 percent slope through the existing dam to the point where the channel daylights with the existing ground surface. Erosion protection will be placed on the channel banks from the stilling basin and through the existing dam. Details of the outlet channel design are shown on the plans.

14.4 CONSTRAINTS

The Outlet Works have been located near the north end of the South FRZ to direct outflow from the reservoir away from the central area of the fissure risk zone. In addition, placing the outlet works through the soil cement core provides protection against seepage along the conduit causing a failure of the embankment due to the erosion resistance of the soil cement.

The construction of the Outlet Works must be performed in stages because breaching of the existing dam is required. The structures located upstream of and within the soil cement core can be constructed with the South FRZ and Transition Embankments. The structures located downstream of the soil cement core cannot be construction until after the South FRZ and Transition Embankments are completed. At that time, a new embankment will exist upstream of the existing embankment in the South FRZ and the existing dam can be breached safely.

15.0 BORROW SOURCE

15.1 BORROW SOURCE INVESTIGATION

A detailed borrow source investigation was performed upstream of White Tanks FRS No. 3 on District property. This investigation consisted of areas identified as Borrow Areas A and B, as shown on Figure 7-1. Limited investigations were performed in the emergency spillway area to identify potential borrow areas. Portions of the existing embankment and upstream toe area that will be excavated during embankment construction were also evaluated as potential construction materials. Details of the geotechnical investigation program and material analyses are provided in the companion document to this design report titled *White Tanks FRS No. 3 Geotechnical Report* (URS 2004).

15.2 BORROW MATERIAL DESCRIPTION

Locations of cross sections showing subsurface information for Borrow Areas A and B, and the emergency spillway, are shown on Figure 7-1 and presented as Figures 7-3 and 7-7. Subsurface information at the existing embankment is shown in the longitudinal cross-section provided on Figure 7-2. Materials excavated from Borrow Areas A and B, and the existing embankment will generally consist of surficial Holocene soil deposits. Materials excavated from the emergency spillway area will consist of both Holocene and Pleistocene deposits.

On-site Holocene and Pleistocene sediments generally contain a high percentage of fine-grained material (on average, 45 to 50 percent). Information obtained from test pits and test holes suggests both the Holocene and Pleistocene sediments consist of an interbedded complex of channel, bar, overbank, and mudflow deposits. Channel and bar deposits often consist of stratified, interbedded and cross-bedded sand, silty sand, sandy gravel, and gravel. The overbank deposits typically consist of poorly stratified beds of silt. Mud flow deposits consist of non-stratified and well graded or poorly sorted admixtures of clay, silt, sand, gravel.

15.3 PHASE 1 BORROW

Construction of the South FRZ and Transition Embankments will be performed using mainly on-site borrow material. Based on the results of the borrow investigation, material excavated from the embankment, embankment toe, and upstream borrow areas can be used for common fill, structural fill, and soil cement. On-site soil material is suitable for use for the soil cement structures, based on the results of soil cement mix design testing (see Section 12.7). Upstream borrow will be limited during Phase 1 to a small portion Borrow A, as shown on the plans.

Where practicable, the contractor should use the more coarse material found in the excavation for use in construction of soil cement and soil cement-bentonite (SCB) cutoff walls. The borrow investigation indicated the potential availability of only a limited amount of material available on-site for use in construction of the SCB cutoff walls. Therefore, based on the investigations performed to date, it is anticipated that the soil required for the cutoff walls would need to be imported from local aggregate suppliers. Material for construction of the graded filter and coarse aggregate apron would likely need to be imported from off-site as well.

[Note to Reviewers: Laboratory testing of on-site materials for use in the SCB cutoff walls is on-going. Results will be incorporated in the 100 Percent submittal.]

15.4 PHASE 2 BORROW

Phase 2 will consist of the construction of the North FRZ, South NFRZ, and North NFRZ Embankments. Similar to the construction of the Phase 1 embankments, a large quantity of fill material will result from the excavation of the Phase 2 embankments. In addition, fill material will come from the emergency spillway. With placement of aesthetic fill on the downstream slope, as well as the upstream slope, it is anticipated that borrow sources will be identified downstream of White Tanks FRS No.3 during the Phase 2 design. Additional fill material will be taken from Borrow Area A, and Borrow Area B will be utilized.

Similar to Phase 1 construction, the on-site borrow will be used for common fill, structural fill, and soil cement. It is anticipated that material required for construction of the SCB cutoff walls, graded filter, and coarse aggregate apron would be imported from off-site.

15.5 BORROW QUANTITIES

Estimated quantities of materials required for construction during Phase 1 are provided on the plans. Plan sheets also provide material quantities for all components of the Phase 1 construction project. Calculations performed to estimate material quantities are provided in Appendix K.

16.0 REQUIREMENTS DURING CONSTRUCTION

16.1 SURFACE WATER CONTROL DURING CONSTRUCTION

[Note to Reviewers: This section will be completed for the 100 Percent submittal.]

16.2 OPERATIONAL PLAN DURING CONSTRUCTION

[Note to Reviewers: This section will be completed for the 100 Percent submittal.]

17.0 REFERENCES

ADWR 2000. *Arizona Administrative Code, Title 12–Natural Resources, Chapter 15–Department of Water Resources, Article 12–Dam Safety Procedures*. Arizona Department of Natural Resources. June 2000.

AMEC 2002. *Seismic Exposure Evaluation, Dam Safety Program*. Flood Control District of Maricopa County, May 2002.

AMEC 2004. *Fissure Zone Instrumentation & Monitoring Plan, McMicken Dam Fissure Risk Zone Remediation Project*. AMEC, August 2004.

AMEC 2004a. *Realigned Dam Alternatives and Preferred Alternative Recommendation*. AMEC, Earth & Environmental, Inc. March 29, 2004.

ADWR 2002. *Embankment Dam Safety Inspection Report – White Tanks FRS No. 3*. Prepared by the Arizona Department of Water Resources, November 6, 2002.

ADWR 2004. *PMF Studies for Evaluation of Spillway Adequacy – General Guidelines*. ADWR. Revised March 2004.

Beckwith and Hansen 1982. *Calcareous Soils of the Southwestern United States*. 1982.

COE 1978. *Embankment Cracking Report. Trilby Wash Detention Basin, Lower Agua Fria River*. Prepared by the U.S. Army Corps of Engineers, Los Angeles District, January 1973.

COE 1995. *Gravity Dam Design (EM 1110-2-2200)*. U.S. Army Corps of Engineers. June 1995.

Dames & Moore 1998. *Draft Design Issues Report – White Tanks FRS No. 3 Modifications Design Project*. Prepared by Dames & Moore for the Flood Control District of Maricopa County. November 1998.

Dames & Moore 2001. *Geotechnical Data Report, White Tanks FRS #3*. Prepared by Dames & Moore for the Flood Control District of Maricopa County. May 2001.

FCDMC 2001. Interim Operational Plan for White Tanks Flood Retarding structure No. 3 Outlet Pipes. Flood Control District of Maricopa County. February 2001.

FERC 2002. *Engineering Guidelines for the Evaluation of Hydropower Projects, Chapter III – Gravity Dams*. October 2002.

Fugro 1979. *Crack Location Investigation, White Tanks FRS No. 3, Maricopa County, Arizona.*

Geo-Slope International. 2000. *SEEP/W VER 4*. Software Manual. Calgary, Alberta, Canada.

GCI 2004. *Seismic Refraction Geophysical Survey, Work Assignment No. 2, White Tanks FRS No. 3 Remediation Project, FCD 2003C055*. Geological Consultants Inc. August 30, 2004

NRCS 1990. *Earth Dams and Reservoirs (TR-60)*. Natural Resources Conservation Service, Revised October 1985, Amended October 1990.

NRCS 1992. *Report of Geologic Investigation of White Tanks Flood Retarding Structure #3*. Prepared by the Soil Conservation Service, July 21-24, 1992.

NRCS 1994. *White Tanks FRS No. 3 – Sediment Rate*. Prepared by Blake Covey, NRCS. 1994.

NRCS 1996. *GM 390 PDM White Tanks Watershed - Floodwater Retarding Structure #3 - Evaluating Alternatives - National Planning Process Step 6.* Gregory E. Perez EIT, Planning Engineer, March 1, 1996

NRCS 1998. *Hydrologic Analysis of the White Tank Mountains on Flood Retarding Structure #3. United States Department of Agriculture*. Natural Resources Conservation Service. Revised August 1998.

PCA 1988. *Design and Control of Concrete Mixtures*, Thirteenth Edition, Portland Cement Association. 1988

Soil Conservation Service, 1982. *White Tank No. 3 and No. 4 Repair.* Flood Control District of Maricopa County, Arizona. 1982.

SHB 1982. *Design Report – McMicken Dam Restoration Study.* Prepared by Sergent, Hauskins & Beckwith Consulting Geotechnical Engineers Inc. for the Flood Control District of Maricopa County. December 29, 1982.

URS 2001. *Geotechnical Data Report, White Tanks FRS #3.* Prepared by URS Corporation, for the Flood Control District of Maricopa County. May 11, 2001.

URS 2004. *Preliminary Embankment Rehabilitation Concepts.* Prepared by URS Corporation, for the Flood Control District of Maricopa County. January 6, 2004.

USBR 1977. *Design Criteria for Concrete Arch and Gravity Dams.* U.S. Bureau of Reclamation, 1977.

USBR 1990. *Design Standard No. 13, Embankment Dams, Chapter 13, Soil cement Slope Protection.* U.S. Bureau of Reclamation, 1990.

USDA 1985a. National Engineering Handbook, Section 4 – Hydrology. U.S. Department of Agriculture, Soil Conservation Service. March 1985.

USDA 1985b. Technical Release No. 60 (TR-60). Earth Dams and Reservoirs. U.S. Department of Agriculture, Soil Conservation Service. Revised October 1985.

Wright, S.G. 1991. UTEXAS3 – A Computer Program for Slope Stability Calculations. Software Manual, Shinoak Software. Texas. 158 pages.

TABLES

TABLES

TABLE 6-1
Embankment Subsidence Adjustment Evaluation

Existing Embankment Station	Crest Monuments (1990 - 2003)		Toe Monuments (1990 - 2003)		Crest Monuments (as-built - 2003)	
	Subsidence (feet)	Adjustment Ratio	Subsidence (feet)	Adjustment Ratio	Subsidence (feet)	Adjustment Ratio
10+00	0.325	1.0	0.335	1.0	4.67	1.0
20+00	0.325	1.0	0.332	0.9	4.38	0.9
30+00	0.274	0.8	0.252	0.8	4.09	0.9
40+00	0.236	0.7	0.199	0.6	3.35	0.7
50+00	0.158	0.5	0.121	0.4	1.86	0.4
60+00	0.093	0.3	0.088	0.3	1.53	0.3
70+00	0.085	0.3	0.079	0.2	0.96	0.2

Notes:

1. Existing embankment stations are approximate.
2. Subsidence data for 2003 was taken from Survey 2003b.

TABLE 7-1
Summary of Geotechnical Investigations Performed Between October 1998 and April 2004

Testing Program	Work Scope	Driller/ Investigator	Date Performed	Field Tests	Sampling/ Sounding Method	Lab Tests	Original ID	Normalized ID
Dam Modification Investigation	22 Borings, 9 Testpits	ATL Inc.	October - December 1998	SPT	Splitspoon, bag and bulk	Index tests, S/C, Mod. Density, TXL, and pore pressure measurements	DMB 1 - DMB 22	B 101 - B 122
							DMP1 - DMP 19	TP 101 - TP 119
Basin Alternatives Study	6 Borings, 3 Testpits	ATL Inc.	November 1999	SPT	Splitspoon, bag and bulk	Sieve analyses, Atterberg limits, and Moisture Density tests	B1 - B6	B 201 - B206
							TP 1- TP 3	TP 201 - TP 203
	6 Seismic Refraction lines	Bird Seismic Services Inc./ Hasbrouck Geophysics Inc.		Seismic Refraction survey	24 channel Bison Spectra signal enhancement seismograph	N/A	SL 1 - SL 6	SL 101 - SL 106
Interim Dam Safety Project	3 Testpits	ATL Inc.	November 1999	None	Bulk	Sieve analyses and Atterberg limits	TP A - TP C	TP 301 - TP 303
Existing Filter Investigation	3 Borings	ATL Inc.	November 1999	SPT	Splitspoon and bulk	Sieve analyses	57+30, 58+00, 59+00	B 301 - B 303
	1 Test pit				Bulk	None	58+90	TP 401
Crack Investigation	1 Test pit	ATL Inc.	March 2000	None	Bulk	Sieve analyses	59+00	TP 501
Preliminary Geotechnical Investigation	5 Resistivity Soundings (deep)	AMEC Earth & Environmental, Inc.	October - December 2002 and 2003	Resistivity soundings	Sting R1 (Advanced Geosciences) resistivity meter with a 4-point Wenner array	N/A	RWT 1, RWT 2, R 3-R 5	SR 101 - SR 105
	20 Seismic Refraction surveys (shallow)			Seismic Refraction surveys	12 channel Geometrics ES-1225/ Smartseis S-12 signal enhancement seismograph		L 1 - L 20	SL 201 - SL 220
	5 S-wave vertical profiles (deep)			Refraction microtremor (REMI) surveys			L 21 - 25	RM 121 - RM 125
	6 Borings, 22 Testpits, 2 Test Trenches		November - December 2003	SPT	Bag and Bulk	Moisture content, Sieve Analyses, and Atterberg limits	B 1 - B 6	B 401 - B 406
							TP 1 - TP 22	TP 601 - TP 622
							TT 1, TT 2	TT 101, TT 102
White Tanks No. 3 FRS Rehabilitation Project	25 Seismic Refraction surveys (deep)	Geological Consultants, Inc.	April 12 - April 15, 2004	Seismic Refraction surveys	24 channel Geometrics ES-1210F seismograph	N/A	S 1 - S 25	SL 301 - SL 325
	5 Test Trenches		July 21, 2004	Visual classification, pocket penetrometer, and vertical jet testing	None	None	TT 1 - TT 5	TT 201 - TT 205
	24 Borings	Enviro-Drill, Inc./ Crux (Lab testing by Terracon)	April 6, 7, 20-24, 29,30, 2004	SPT	Split spoon, ring samples, core (HQ), and Shelby tubes	Consolidation, collapse, sieve analyses, Atterberg limits, and EFA testing	B 1 - B 24	B 501 - B524
	33 Testpits	Terracon / Quackenbush Construction	April 20, 21, 26-28, 2004	None	Bulk	Sieve analyses, and Atterberg limits	TP 1 - TP 33	TP 701 - TP 733

TABLE 8-1
Summary of TR-20 Computer Model Review

Storm	NRCS MODELS[1]		FCDMC MODELS[2]		URS MODELS[3]	
	Peak Inflows (cfs)	Peak Inflow Volumes (acre-ft)	Peak Inflows (cfs)	Peak Inflow Volumes (acre-ft)	Peak Inflows (cfs	Peak Inflow Volumes (acre-ft)
6-Hour Local PMP	66,122	9,202	66,122	9,202	66,122	9,190
6-Hour General PMP	34,212	6,913	34,212	6,913	34,216	6,913
12-Hour General PMP	32,435	9,142	32,435	9,142	32,278	9,150
18-Hour General PMP	26,905	10,327	26,905	10,327	26,905	10,327
24-Hour General PMP	23,800	11,229	23,800	11,229	23,800	11,229
48-Hour General PMP	31,819	13,411	31,819	13,411	31,696	13,413
72-Hour General PMP	32,300	14,225	32,300	14,225	32,296	14,228
100-Year, 24-hour	10,835	N/A	10,835	2,204	10,468	2,204
Emergency Spillway Hydrograph (ESH)	21,685	3,567	21,685	3,567	21,674	3,567
Principal Spillway (100-year 10-Day)	3,290	1,614	3,290	1,614	3,290	1,614

Notes

1. These peak inflows and inflow volumes are tabulated in Table II and III of the NRCS Report Hydrologic Analysis of the White Tank Mountains on Flood Retarding Structure # 3 (NRCS, August 1998).
2. These peak inflows are inflow volumes are obtained by opening up the TR-20 output files provided by FCDMC to URS.
3. These peak inflows and inflow volumes are based on the output files generated by URS by executing the input files provided by FCDMC.

TABLE 8-2
Watershed Basin Drainage Areas

Basin	Drainage Areas Estimated by NRCS[1] (square miles)	Drainage Areas Estimated by URS (square miles)	Difference in Drainage Areas (%)
1	2.45	2.46	0.41
2	2.34	2.38	1.68
3	3.96	3.94	-0.51
4	2.02	2.06	1.94
5	4.76	4.78	0.42
6	1.5	1.47	-2.04
7	3.46	3.48	0.57
Total	**20.49**	**20.57**	**0.39**

Notes:

1. These drainage areas are tabulated in Table I of the NRCS Report *Hydrologic Analysis of the White Tank Mountains on Flood Retarding Structure #3* (NRCS, August 1998).

TABLE 8-3
Elevation-Area-Capacity Data and Infiltration Estimates

Reservoir Elevation (NAVD 88) (feet)	Surface Area (acres)	Average Surface Area (acres)	Reservoir Storage (acre-feet)	Cumulative Storage (acre-feet)	Estimated Infiltration Rate – Interim Condition (cfs)[1]	Estimated Infiltration Rate – Future Condition (cfs)[2]	Comments
1178.0	0.10	0.10	0.00	0.00	0.026	0.000	
1179.0	0.51	0.302	0.30	0.30	0.133	0.001	
1180.0	1.05	0.778	0.78	1.08	0.275	0.002	
1181.0	1.76	1.403	1.40	2.48	0.461	0.004	
1182.0	3.07	2.415	2.42	4.90	0.806	0.006	
1183.0	4.79	3.932	3.93	8.83	1.256	0.010	
1184.0	5.82	5.305	5.30	14.14	1.525	0.012	
1185.0	6.60	6.210	6.21	20.35	1.731	0.013	
1186.0	7.44	7.022	7.02	27.37	1.951	0.015	
1187.0	8.65	8.048	8.05	35.42	2.269	0.017	
1188.0	10.32	9.487	9.49	44.90	2.706	0.021	
1189.0	11.93	11.125	11.13	56.03	3.128	0.024	
1190.0	13.93	12.930	12.93	68.96	3.652	0.028	
1191.0	15.98	14.955	14.96	83.91	4.189	0.032	
1192.0	22.46	19.220	19.22	103.13	5.888	0.045	
1193.0	**27.82**	**25.140**	**25.14**	**128.27**	**7.293**	**0.056**	**Gated Outlet Invert**
1194.0	33.83	30.825	30.83	159.10	8.869	0.068	
1195.0	44.05	38.940	38.94	198.04	11.548	0.089	
1196.0	56.65	50.350	50.35	248.39	14.852	0.114	
1196.8	67.18	61.914	50.77	299.16	17.612	0.135	
1197.0	69.49	68.334	12.30	311.46	18.218	0.140	
1198.0	83.88	76.685	76.69	388.14	21.991	0.169	
1199.0	98.77	91.325	91.33	479.47	25.894	0.199	
1199.2	**102.00**	**100.385**	**20.58**	**500.05**	**26.741**	**0.206**	**100-Year Sediment Pool Level**
1200.0	**112.95**	**107.475**	**85.44**	**585.49**	**29.612**	**4.516**	**Principal Spillway Inlet**
1200.1	115.02	113.985	13.79	599.28	30.154	5.187	
1201.0	130.05	122.535	107.71	706.99	34.095	10.165	
1202.0	147.55	138.800	138.80	845.79	38.683	15.989	
1203.0	165.40	156.475	156.48	1002.26	43.362	21.895	
1204.0	183.11	174.253	174.25	1176.52	48.004	27.758	

TABLE 8-3 (CONTINUED)
ELEVATION-AREA-CAPACITY DATA AND INFILTRATION ESTIMATES

Reservoir Elevation (NAVD 88) (feet)	Surface Area (acres)	Average Surface Area (acres)	Reservoir Storage (acre-feet)	Cumulative Storage (acre-feet)	Estimated Infiltration Rate – Interim Condition (cfs)[1]	Estimated Infiltration Rate – Future Condition (cfs)[2]	Comments
1205.0	199.09	191.098	191.10	1367.61	52.195	33.257	
1206.0	216.66	207.875	207.88	1575.49	56.801	38.912	
1207.0	234.81	225.735	225.74	1801.22	61.559	44.601	
1208.0	253.15	243.980	243.98	2045.20	66.367	50.266	
1209.0	274.77	263.960	263.96	2309.16	72.036	56.556	
1210.0	294.10	284.435	284.44	2593.60	77.103	62.352	
1211.0	313.41	303.755	303.76	2897.35	82.166	68.094	
1212.0	**327.85**	**320.630**	**320.63**	**3217.98**	**85.951**	**72.698**	**Emergency Spillway Crest**
1213.0	350.00	338.925	338.93	3556.91	91.758	78.958	
1214.0	368.69	359.345	359.35	3916.25	96.658	84.411	
1215.0	387.54	378.115	378.12	4294.37	101.600	89.861	
1216.0	409.55	398.545	398.55	4692.91	107.370	96.018	
1217.0	431.48	420.515	420.52	5113.43	113.120	102.143	
1218.0	454.15	442.815	442.82	5556.24	119.063	108.430	

Notes:

1) Estimated Infiltration Rates – Interim Condition refers to the existing reservoir conditions.
2) Estimated Infiltration Rates – Future Condition refers to the reservoir condition assuming 500 ac-ft of sediment has accumulated.

TABLE 8-4
Urban Growth Projections and Curve Number Estimation

Basin No.	Land Ownership Category	Urban Growth Status (Year 2030)	Drainage Area (sq mi)	NRCS Curve Numbers (Existing) (CN)	URS Curve Numbers (Future) (CN)	Average Curve Numbers (Future) (CN)
1	Regional Park	Undevelopable	2.460	87.2	87.2	87.2
2A	Regional Park	Undevelopable	1.020		87.2	
2B	Regional Park	Undevelopable	0.070	78.2	71.45	79.9
2C	State Trust Land	Developable (Low Density Population)	1.291		74.6	
3	Regional Park, Private Land, and State Trust Land	Undevelopable (Mountains)	3.940	87.2	87.2	87.2
4A	Regional Park Area	Undevelopable	0.430		87.2	
4B	Regional Park Area	Undevelopable	0.440	75.5	72.41	77.3
4C	State Trust Land and Private Land	Developable (Low Density Population)	1.190		75.5	
5A	Regional Park	Undevelopable	1.000		87.2	
5B	Regional Park	Undevelopable	0.879		73.67	
5C	State Trust Land and Private Land	Developable (Low Density Population)	0.978	76.5	76.6	78.8
5D	State Trust Land	Developable (High Density Population)	1.700		78.5	
5E	District Poperty	Undevelopable	0.222		73.67	
6A	State Trust Land	Developable (High Density Population)	0.310	87.2	87.2	87.7
6B	Regional Park Area, Private Land, and State Trust Land	Undevelopable (Mountains)	1.160		87.2	
7A	State Trust Land and Private Land	Undevelopable (Mountains)	1.098		87.2	
7B	FCDMC Area	Undevelopable	0.278	78.9	75.07	81.7
7C	State Trust Land and Private Land	Developable (High Density Population)	2.104		79.7	

TABLE 9-1
Emergency Spillway Discharge Rating Curve

Elevation (feet) (NAVD 88)[1]	Discharge (cfs)[2]
1,212	0
1,213	2,593
1,214	8,134
1,215	15,443
1,216	24,165
1,217	34,099
1,218	45,113

Notes:

1. The emergency spillway crest elevation is set at 1212.0 feet.
2. The emergency spillway discharge was estimated using the weir formula. The weir coefficient is 2.64. The spillway crest length is 1,200 feet.
3. The discharges tabulated above take into account the conveyance effects upstream of the White Tank FRS No.3 spillway.

TABLE 10-1
Reservoir Routing Results

Storm Event	Inflow to White Tanks FRS No. 3		Outflow from White Tanks FRS No. 3 (Interim Condition)						Outflow from White Tanks FRS No. 3 (Future Condition - Principal Spillway Open)						Outflow from White Tanks FRS No. 3 (Future Condition - Principal Spillway Closed)					
			NRCS Criteria			ADWR Criteria			NRCS Criteria			ADWR Criteria			NRCS Criteria			ADWR Criteria		
	Precipitation (inches)	Peak Inflow (cfs)	Antecedent Reservoir Condition (ARC)	Peak Outflow (cfs)	Maximum Reservoir Elevation (NAVD 88) (feet)	Antecedent Reservoir Condition (ARC)	Peak Outflow (cfs)	Maximum Reservoir Elevation (NAVD 88) (feet)	Antecedent Reservoir Condition (ARC)	Peak Outflow (cfs)	Maximum Reservoir Elevation (NAVD 88) (feet)	Antecedent Reservoir Condition (ARC)	Peak Outflow (cfs)	Maximum Reservoir Elevation (NAVD 88) (feet)	Antecedent Reservoir Condition (ARC)	Peak Outflow (cfs)	Maximum Reservoir Elevation (NAVD 88) (feet)	Antecedent Reservoir Condition (ARC)	Peak Outflow (cfs)	Maximum Reservoir Elevation (NAVD 88) (feet)
6-hr General PMP	8.80	35,610	1,201.4	14,991	1,214.9	NR[5]	NR	NR	1,200.0[4]	14,678	1,214.9	NR	NR	NR	1,204.8	15,970	1215.1	NR	NR	NR
12-hr General PMP	11.00	33,225	1,201.4	17,654	1,215.2	NR	NR	NR	1,200.0	17,229	1,215.2	NR	NR	NR	1,204.8	19,022	1215.4	NR	NR	NR
18-hr General PMP	12.20	27,370	1,201.4	21,744	1,215.7	NR	NR	NR	1,200.0	21,338	1,215.6	NR	NR	NR	1,204.8	22,744	1215.8	NR	NR	NR
24-hr General PMP	12.90	24,210	1,201.4	20,545	1,215.6	NR	NR	NR	1,200.0	20,306	1,215.5	NR	NR	NR	1,204.8	21,065	1215.6	NR	NR	NR
48-hr General PMP	15.00	32,200	1,201.4	21,360	1,215.7	NR	NR	NR	1,200.0	20,577	1,215.6	NR	NR	NR	1,204.8	23,461	1215.9	NR	NR	NR
72-hr General PMP	15.80	32,700	1,201.4	24,182	1,216.0	1,193.0	21,911	1,215.7	1,200.0	23,197	1,215.9	1,200.0	23,197	1,215.9	1,204.8	26,054	1216.2	1,199.2	23,435	1,215.9
6-hr Local PMP	12.70	68,290	1,201.4	25,640	1,216.1	1,193.0	21,921	1,215.7	1,200.0	24,230	1,216.0	1,200.0	24,229	**1,216.0**	1,204.8	28,968	**1,216.5**	1,199.2	23,720	1,215.9
ESH	5.29	23,556	1,193.0[4]	2,106	1,212.8	-	-	-	1,200.0	3,912	1,213.2	NR	NR	NR	1199.2[4]	3,652	1,213.2	NR	NR	NR
100-year 10-Day	6.40[1]	2,179	1,193.0	55.4[3]	1205.7	NR	NR	NR	1,200.0	216	1,206.1	NR	NR	NR	1199.2	48.2[3]	1207.6	NR	NR	NR
Back-To-Back 100-year 10-day storms	6.40	2,179	1,204.6[2]	71.0[3]	1208.9	NR	NR	NR	NR	NR	NR	NR	NR	NR	1206.9[2]	67.5[3]	1210.8	NR	NR	NR

Notes:

1) The TR-20 model for the 100-year, 10-day storm events is set up different from the models for the other storm events. Due to the extended duration of the storm, the runoff depth of 1.64 inches is input to the model to reflect the total anticipated runoff.
2) The ARC for the second 100-year 10-day storm is based upon the reservoir elevation at the end of 10th day of the reservoir routing of first 100-year 10-day storm.
3) The peak outflow for 100-year 10-day and Back-Back 100-year 10-day storms reflects the infiltration amount only.
4) The ARC Elevation of:

   1193.0 ft corresponds to the invert level of the lowest outlet work,

   1199.2 ft corresponds to the100-year sediment pool level (500 acre-feet of sediment storage), and

   1200.0 ft corresponds to the crest elevation of the principal spillway.
5) NR stands for Not Required.
6) The water surface elevations shown in **BOLD** indicate the elevations used for design.

TABLE 10-2
Results of 24-Hour Storm Routing

Storm Event	Peak Inflow (cfs)	Antecedent Reservoir Condition (ARC)		Maximum Reservoir Elevation (NAVD 88)[1] (feet)
		Condition	Elevation (feet) (NAVD 88)	
100-year, 24-hour	11,750	Interim Condition	1,193.0[2]	1,209.3
		Future Condition (Principal Spillway Open)	1,200.0[2]	1210.1
		Future Condition (Principal Spillway Closed)	1,199.2[2]	1,210.6
200-year, 24-hour	14,530	Interim Condition	1,193.0	1,210.6
		Future Condition (Principal Spillway Open)	1,200.0[2]	1211.3
		Future Condition (Principal Spillway Closed)	1,199.2	1,211.9
500-year, 24-hour	17,782	Interim Condition	1,193.0	1,212.1
		Future Condition (Principal Spillway Open)	1,200.0	1212.2
		Future Condition (Principal Spillway Closed)	1,199.2	1,212.3

Notes:

1) Emergency spillway crest is at 1212.0 feet (NAVD 88).
2) The ARC Elevation of:

   1193.0 ft corresponds to the invert level of the lowest outlet work,

   1199.2 ft corresponds to the100-year sediment pool level (500 acre-feet of sediment storage), and

   1200.0 ft corresponds to the crest elevation of the principal spillway.

TABLE 12-1
Embankment Stationing

Embankment Section	New Dam Stationing		Corresponding Existing Dam Stationing	
	From	To	From	To
South Transition	31+51	35+22	58+59	55+00
South Fissure Risk Zone	35+22	60+35	55+00	29+20
North Transition	60+35	63+78	29+20	25+87

TABLE 12-2
Summary of Hydraulic Conductivity Values

Material	Hydraulic Conductivity k_H (cm/sec)	k_H/k_V
Existing Embankment	$1x10^{-5}$	10
Soil-Cement Material	$1x10^{-7}$	1
Common Fill	$1x10^{-5}$	10
SCB Cuttoff Wall	$1x10^{-7}$	1
Foundation Soils	$1x10^{-4}$	10

TABLE 12-3

Material Properties for Slope Stability Analysis

Material	Moist Unit Weight (pcf)	Drained Shear Strength		Undrained Shear Strength		Post-Seismic Shear Strength	
		Cohesion	Friction Angle (degrees)	Cohesion	Friction Angle (degrees)	Cohesion	Friction Angle (degrees)
Existing Embankment	125	0	33	0	33	0	28
Soil-Cement Material	135	500 psi	0	200 psi	0	400 psi	0
Common Fill	120	0	33	500 psf [1]	19 (2)	0	28
Foundation Soils	118	0	30	0	30	0	25

Notes:

1. Estimated Unconsolidated Undrained Strength
2. p': Effective Overburden Pressure

TABLE 12-4
Summary of Slope Stability Analyses Results

Section	Loading Case	Computed Minimum FS	Required Minimum FS
Maximum Height Soil Cement Section	End of Construction	2.24	1.3
	Steady-State Seepage	2.21	1.5
	Instantaneous Drawdown	See Note 1	1.2
	Pseudo-static seismic	1.55	1.2
Transition Section	End of Construction	2.54	1.3
	Steady-State Seepage	2.75	1.5
	Instantaneous Drawdown	1.66	1.2
	Pseudo-static seismic	1.83	1.2

Notes:

1. Based on simplified calculations, the FS for this case is estimated to be well above the required minimum value.

TABLE 12-5
Summary of Embankment and Foundation Settlement

Settlement Condition	Settlement During Construction (inches)	Post –Construction Settlement (inches)
Soil cement Embankment Settlement	< 1	< 1
Consolidation Settlement, Foundation Soils	< 1	< 1
Immediate Settlement, Foundation Soils	6	0
Collapse Settlement, Foundation Soils	0	0

TABLE 13-1
SITES Model Parameters

Surface Conditions	Holocene	Pleistocene
Vegetal Retardance Curve Index	0.02	0.02
Vegetal Cover Factor	.3	0
Maintenance Code	1	1
Potential rooting depth (ft)	0.5	0

Soil Properties	Holocene	Pleistocene
Plasticity index	10	13
Dry bulk density (lbs/cu ft)	110	117
Headcut erodibility index	0.001	0.01-0.4
Percent clay (%)	20	35
Representative diameter (in)	0.004	0.008

TABLE 14-1
Existing Outlets

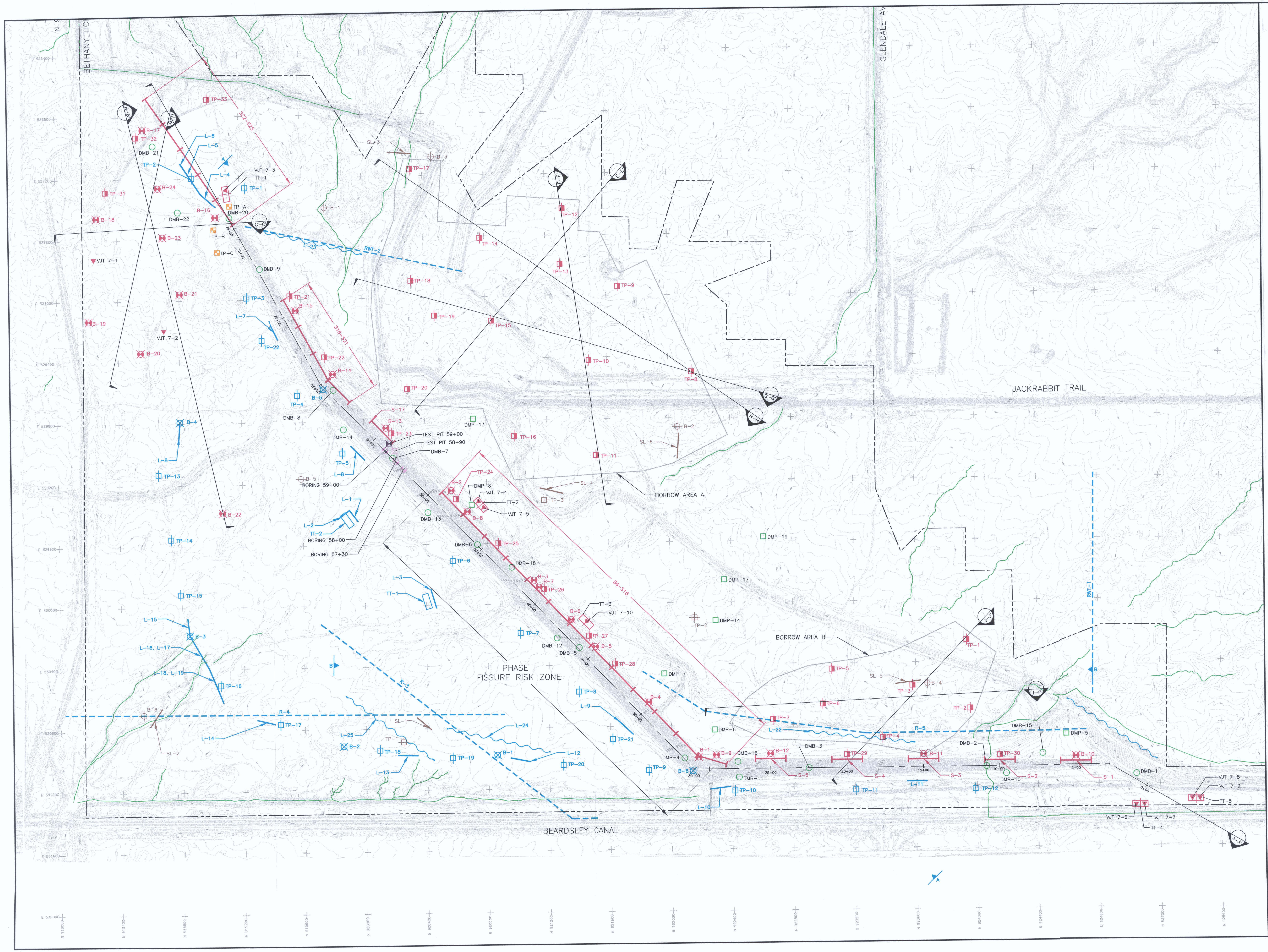
Outlet	Location		Diameter (inches)
	Existing Dam Stationing	New Dam Stationing	
North	63+87	26+22	48
Central	45+97	44+23	48
South	29+06	60+46	24

TABLE 14-2
Outlet Works Discharge Rating Curve

Elevation (feet) (NAVD 88)	Future Condition (Principal Spillway Open) Discharge (cfs)			Interim Condition & Future Condition (Principal Spillway Closed) Discharge (cfs)		
	Gated Outlet (48-inch)	Gated Principal Spillway Bypass outlet (48-inch)	Combined	Gated Outlet (48-inch)	Principal Spillway[1] (48-inch)	Combined
1,195	0	0	0	0	-	0
1,197	24	24	48	24	-	24
1,199	87	87	174	87	-	87
1,201	123	123	246	123	76	199
1,203	150	150	300	150	181	331
1,205	173	173	346	173	200	373
1,207	194	194	388	194	218	412
1,209	212	212	424	212	234	446
1,211	229	229	458	229	250	479
1,212	237	237	474	237	257	494
1,213	245	245	490	245	264	509
1,214	253	253	506	253	270	523
1,215	260	260	520	260	277	537
1,216	267	267	534	267	284	551
1,217	274	274	548	274	290	564
1,218	281	281	562	281	296	577

Notes:
1. Principal spillway crest elevation is set at 1,200 ft (NAVD 88).

FIGURES



LEGEND

DAM MODIFICATION INVESTIGATION (OCTOBER - DECEMBER 1998)
- BORINGS (DMB-1 TO DMB-22)
- TEST PITS (DMP-1 TO DMP-19)

BASIN ALTERNATIVES STUDY (NOVEMBER 1999)
- BORINGS (B-1 TO B-6)
- TEST PITS (TP-1 TO TP-3)
- SEISMIC REFRACTION PROFILE (SL1-SL6)

INTERIM DAM SAFETY PROJECT (NOVEMBER 1999)
- TEST PITS (TP-A TO TP-C)

EXISTING FILTER INVESTIGATION (NOVEMBER 1999)
- BORINGS (57+30, 58+00, 59+00)
- TEST PIT (58+90)

CRACK INVESTIGATION (MARCH 2000)
- TEST PIT (59+00)

PRELIMINARY GEOTECHNICAL INVESTIGATION (OCTOBER 2002 - DECEMBER 2003)
- BORINGS (B-1 TO B-6)
- TEST PITS (TP-1 TO TP-22)
- TEST TRENCH (TT-1, TT-2)
- SEISMIC REFRACTION PROFILE (120-FOOT LINES)
- SEISMIC RESISTIVITY PROFILE
- REFRACTION MICROTREMOR (REMI) SURVEYS
- CONCEPTRUAL GEOLOGIC CROSS SECTIONS DEVELOPED BY AMEC

WHITE TANKS NO. 3 F.R.S. REMEDIATION PROJECT (APRIL AND JULY 2004)
- BORINGS (B-1 TO B-24)
- TEST PITS (TP-1 TO TP-33)
- TEST TRENCHES (TT-1 TO TT-5)
- SEISMIC REFRACTION PROFILE (230-FOOT LINES)
- JET EROSION INDEX TEST (VJT 7-1 TO VJT 7-9)

NOTES

1. SECTION A-A' IS LOCATED ALONG THE UPSTREAM TOE OF THE DAM.
2. BASE MAP OF WHITE TANKS TOPOGRAPHIC MAP RECEIVED FROM FCDMC 5/2004.
3. ORIGINAL SITE PLAN EXPLORATION LOCATIONS ARE COLOR CODED.
4. NEW STATIONING HAS BEEN ESTABLISHED FOR THE DAM REHABILITATION PROJECT. HOWEVER, THIS DRAWING SHOWS THE OLD STATIONING SYSTEM.

NORTH

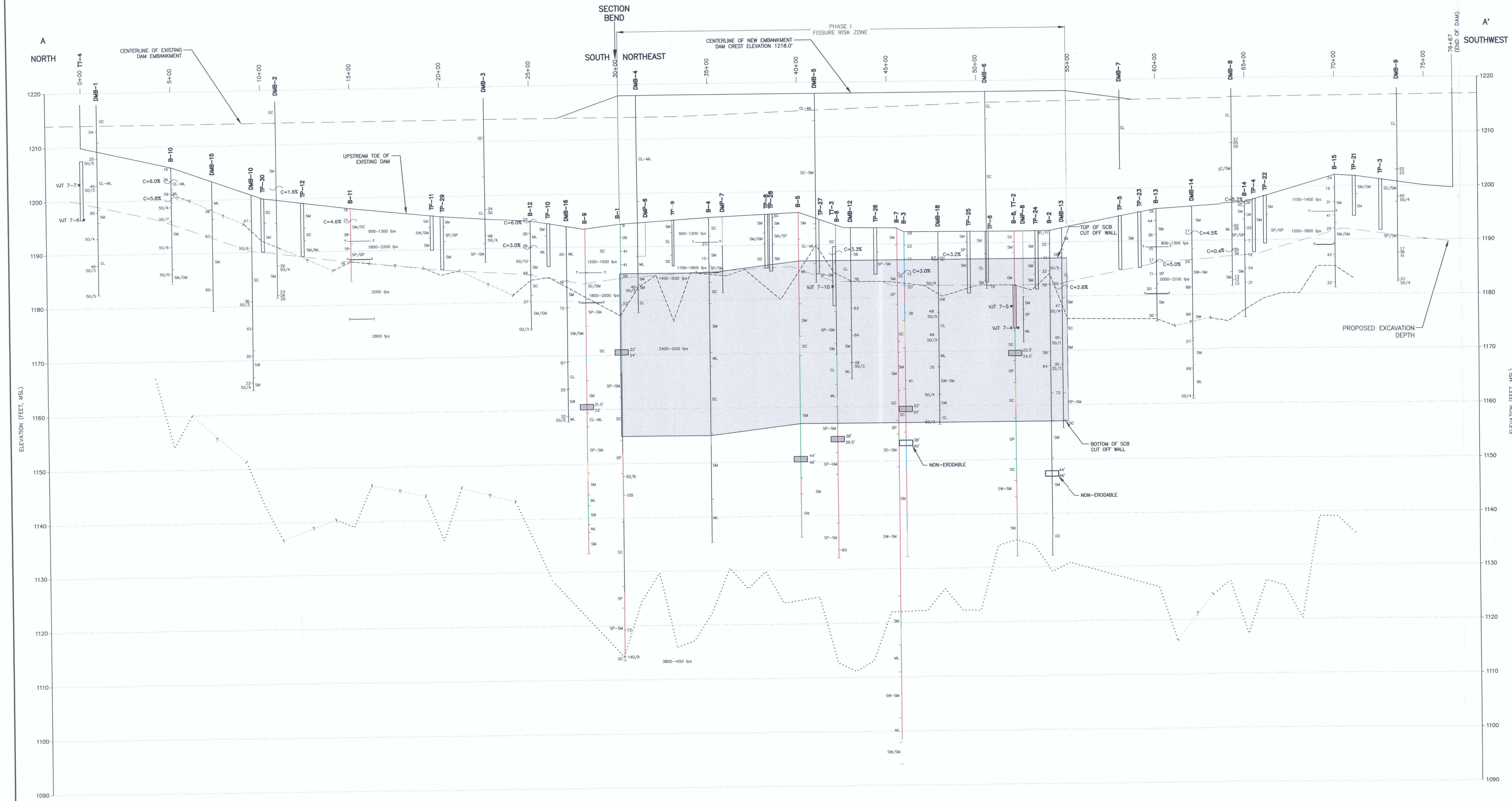
0 150 300 450 600
Scale in Feet

WHITE TANKS F.R.S. NO. 3
FIGURE 7-1
URS JOB NO. 23443748

URS

Flood Control District of Maricopa County

WHITE TANKS FRS # 3



LEGEND

GEOCONSULTANTS SEISMIC REFRACTION LINES (JULY 2004)

UPPER SEISMIC LINE 1230-2100 fps / 2400-2850 fps

LOWER SEISMIC LINE 2400-2850 fps / 3400-4650 fps

AMEC'S SEISMIC REFRACTION LINES (2002-2003)

UPPER SEISMIC LINE 800-1500 fps / 1500-2200 fps

MIDDLE SEISMIC LINE 1500-2200 fps / 2000-3100 fps

LOWER SEISMIC LINE 2000-3100 fps / 2800 fps

CLASSIFICATION OF CEMENTATION

UNCEMENTED

STAGE I

STAGE I+

STAGE II

STAGE II+

STAGE III

STATE OF CEMENTATION UNKNOWN

EFA TESTS, TEXAS A&M UNIVERSITY (JUNE 2004)

HOLE EROSION TEST, USBR DENVER (AUGUST 2004)

COLLAPSE TEST LOCATION AND RESULTS C=4.5%

EXPLORATORY BORING WITH REPRESENTATIVE N-VALUES AND USCS LITHOLOGIC CLASSIFCATIONS B-10 50/5 SM/GM

EXPLORATORY TEST PIT WITH USCS LITHOLOGIC CLASSIFICATIONS TP-11 SM SW/SM

LOCATION OF VERTICAL JET TEST VJT 7-4

NOTES

1. BLOW COUNTS FOR DMB-1 THROUGH DMB-22 WERE OBTAINED USING A DAMES AND MOORE TYPE U 3-INCH DIAMETER SAMPLER. TO OBTAIN EQUIVALENT SPT-N VALUES THE DMB BLOW COUNTS SHOULD BE DIVIDED BY 2.
2. ALL TEST LOCATIONS PROJECTED ONTO PROXIMAL SECTION A-A' LOCATED ALONG THE UPSTREAM TOE OF THE DAM. SEE FIGURE 3-1 FOR TEST AND EXPLORATION LOCATIONS IN PLAN VIEW.
3. AMEC TEST PITS - TP1 TO TP12
4. TERRACON TEST PITS - TP21 TO TP30
5. TEST TRENCH PROFILES ARE PRESENTED IN REPORT APPENDICES
6. AMEC'S SEISMIC REFRACTION LINES HAVE A DEPTH PENETRATION OF 35 FEET. THE SEISMIC LINES DRAWN REPRESENT THE AVERAGE DEPTH OF THE SEISMIC PROFILE AT THE SEISMIC LINE LOCATION.
7. GEOCONSULTANT'S SEISMIC REFRACTION LINES HAVE A DEPTH PENETRATION OF 80 FEET.
8. ALL SEISMIC VELOCITIES ARE IN FEET PER SECOND.
9. CONSOLIDATION VALUES ARE ROUNDED TO THE NEAREST TENTH OF A PERCENT.
10. NEW STATIONING HAS BEEN ESTABLISHED FOR THE DAM REHABILITATION PROJECT. HOWEVER, THE OLD STATIONING SYSTEM IS SHOWN ON THIS DRAWING.

0 150 300 450 600
SCALE IN FEET
HORIZONTAL SCALE: 1"=300'
VERTICAL SCALE: 1"=10'

WHITE TANKS F.R.S. NO. 3

GENERALIZED CROSS SECTION A-A'

FIGURE 7-2

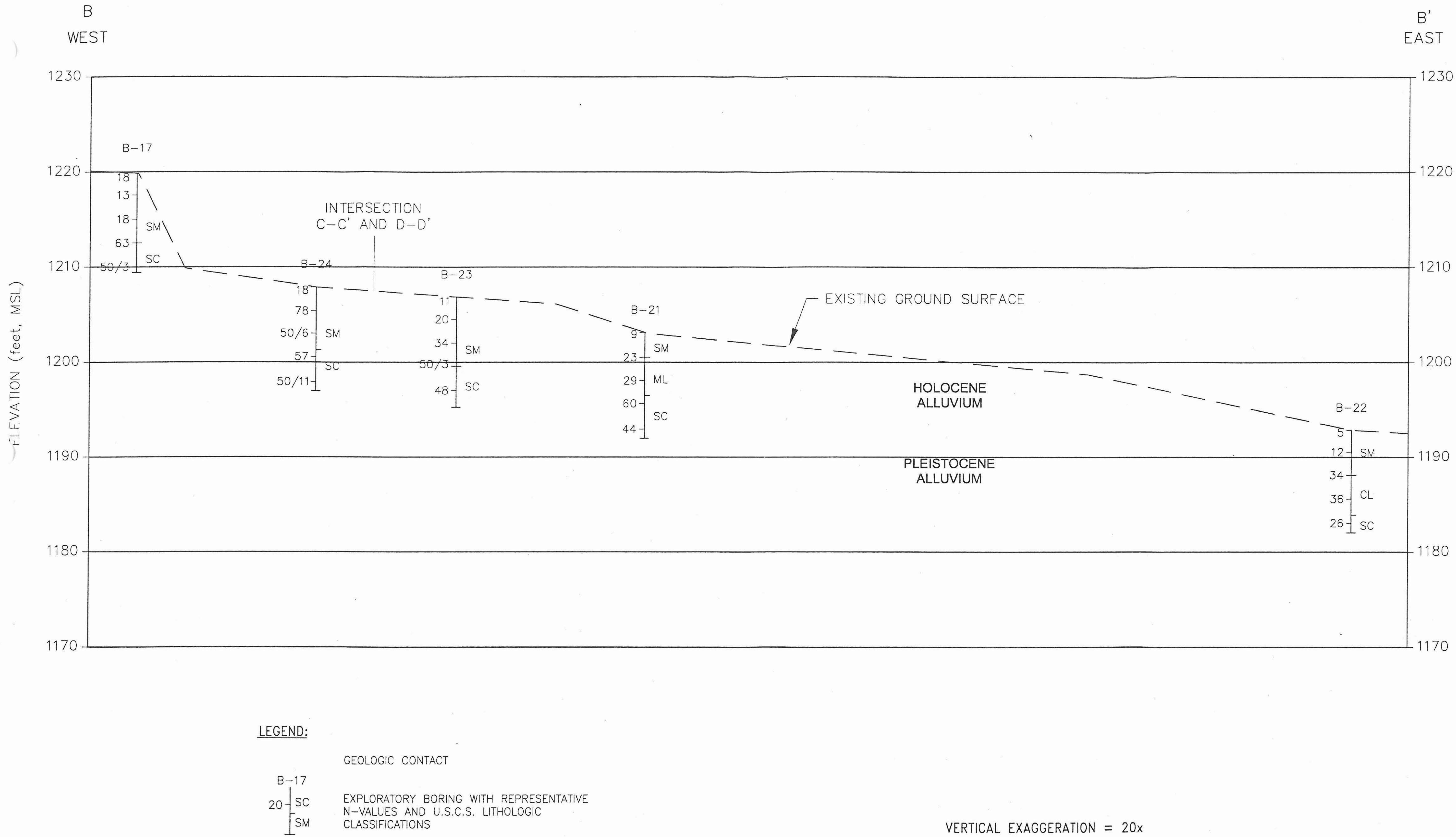
URS JOB NO. 23443748

URS

NRCS
Natural Resources Conservation Service

Flood Control District of Maricopa County

WHITE TANKS FRS # 3



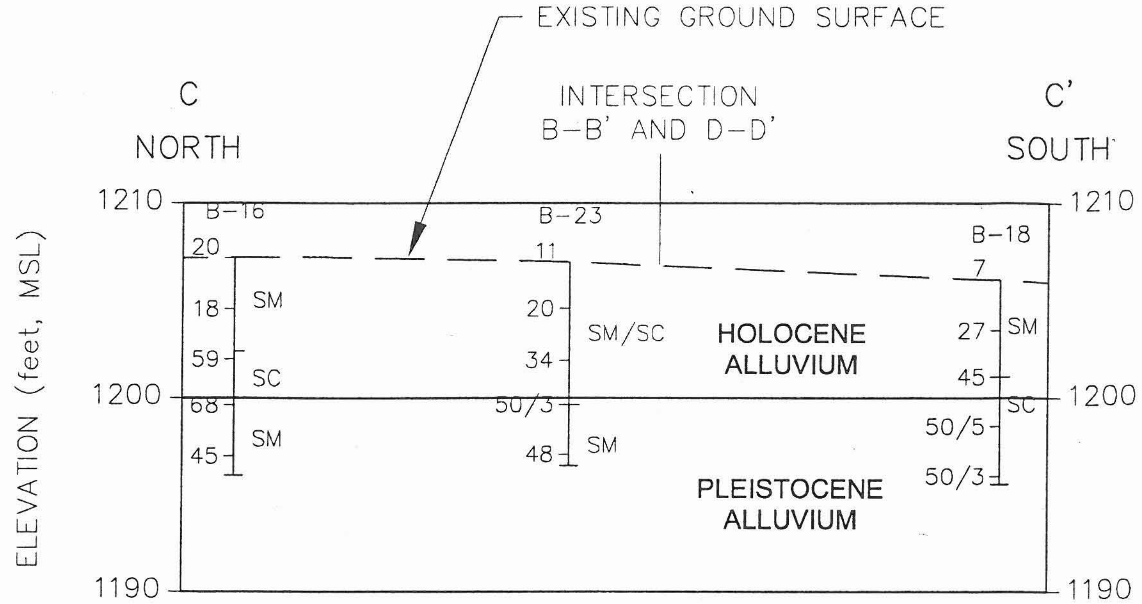
VERTICAL EXAGGERATION = 20x

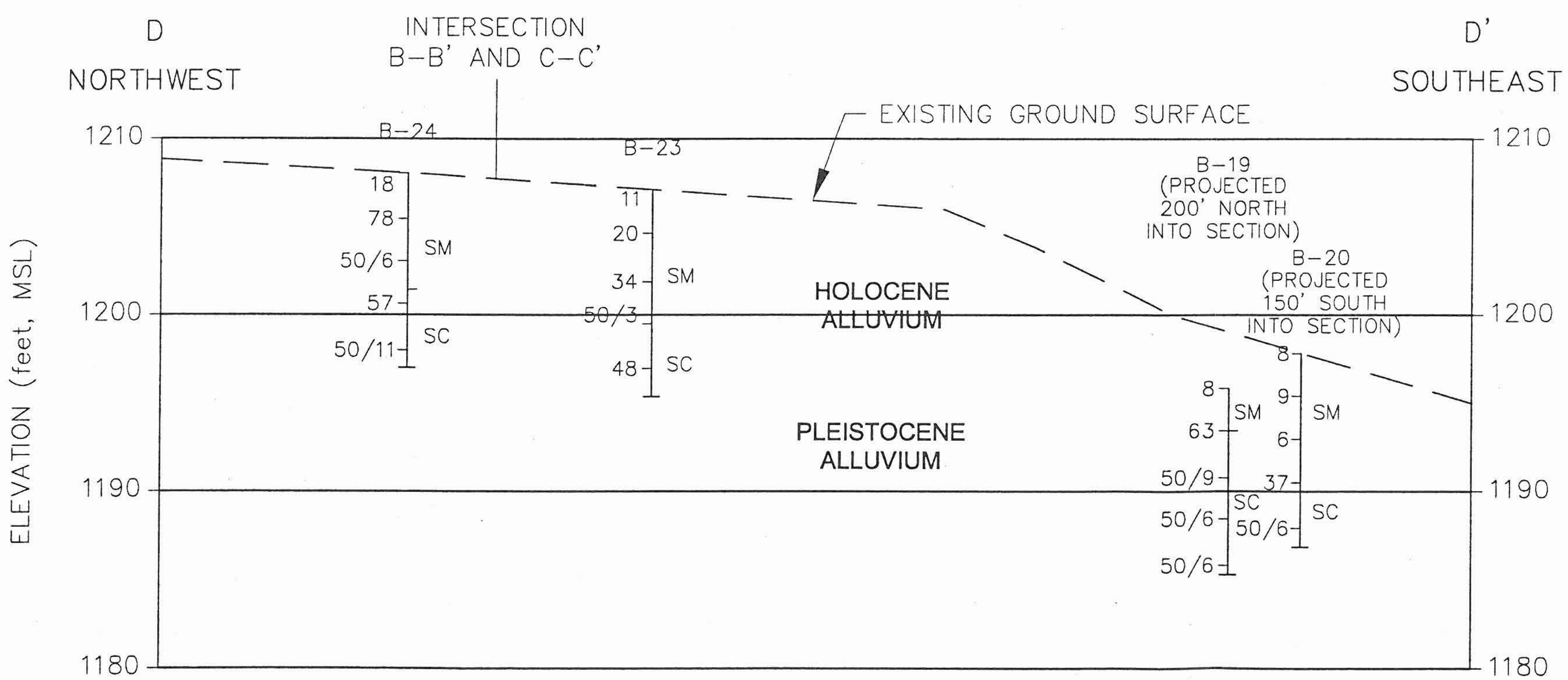
EMERGENCY SPILLWAY
GENERALIZED CROSS SECTION B-B'
WHITE TANKS F.R.S #3

URS

100 0 100 200 FEET SCALE HOR: 1"=200', VERT: 1"=10'	CHECKED BY: TER	DATE: 5-20-04	FIG. NO:
		PROJ. NO: 23443748.00037	7-3

P:\FCDMC\23443698 WHITE TANKS\CADD\FROM DENVER\9-10-04\FISSURE PROFILE-URSDEN-design-reva.dwg 9-10-04





LEGEND:

GEOLOGIC CONTACT

B-17
20 SC
SM

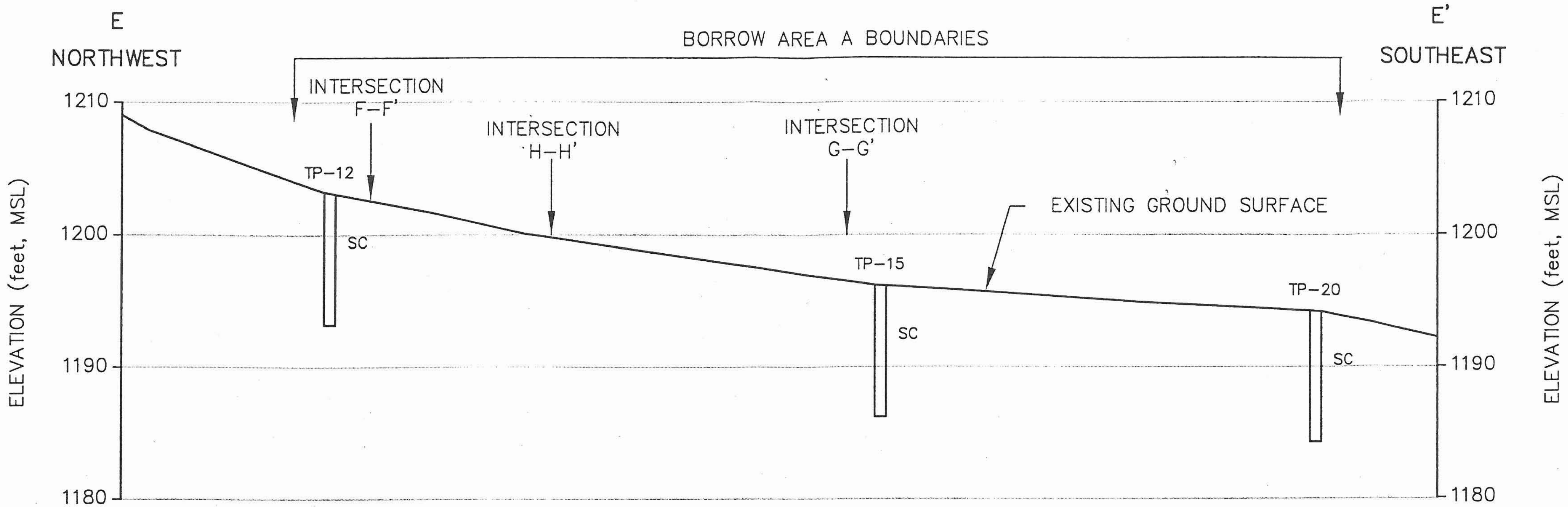
EXPLORATORY BORING WITH REPRESENTATIVE N-VALUES AND U.S.C.S. LITHOLOGIC CLASSIFICATIONS

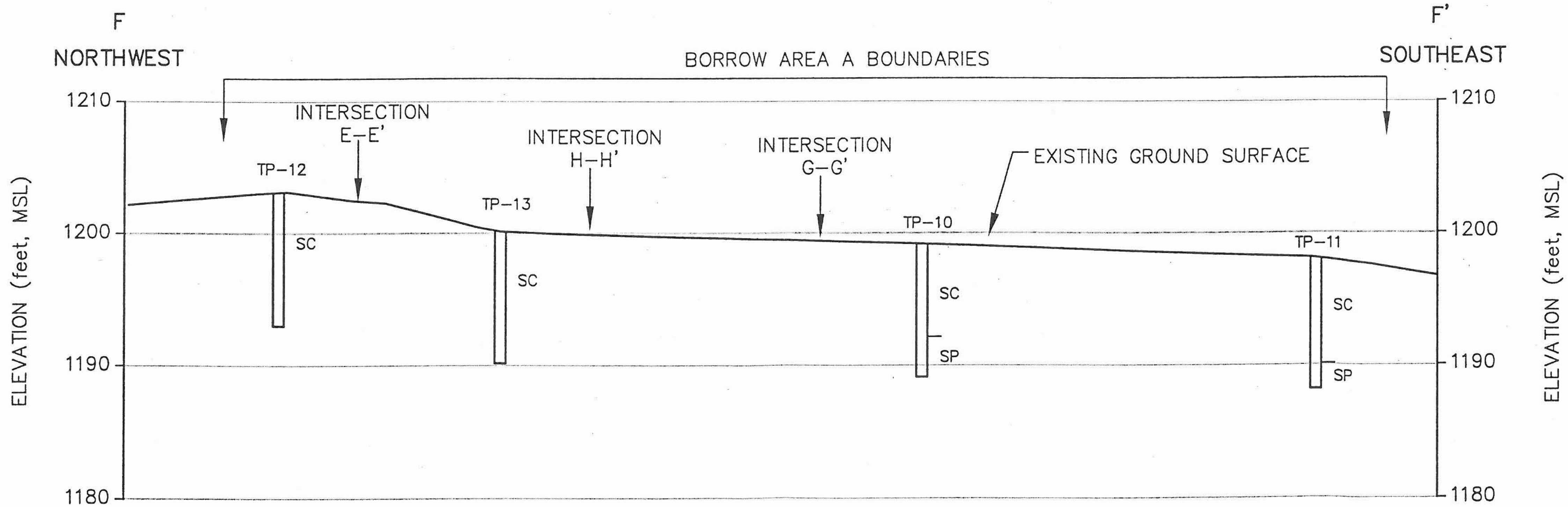
VERTICAL EXAGGERATION = 20x

EMERGENCY SPILLWAY
GENERALIZED CROSS SECTION C-C' AND D-D'
WHITE TANKS F.R.S #3

URS	100 0 100 200 FEET SCALE HOR: 1"=200', VERT: 1"=10'	CHECKED BY: TER	DATE: 5-20-04	FIG. NO:
			PROJ. NO: 23443748.00037	7-4

P:\FCDMC\23443698 WHITE TANKS\CADD\FROM DENVER\9-10-04\FISSURE PROFILE-URSDEN-design-reva.dwg 9-10-04





LEGEND:

TP–2

SC

SM

EXPLORATORY TEST PIT WITH USCS LITHOLOGIC CLASSIFICATIONS

VERTICAL EXAGGERATION = 20x

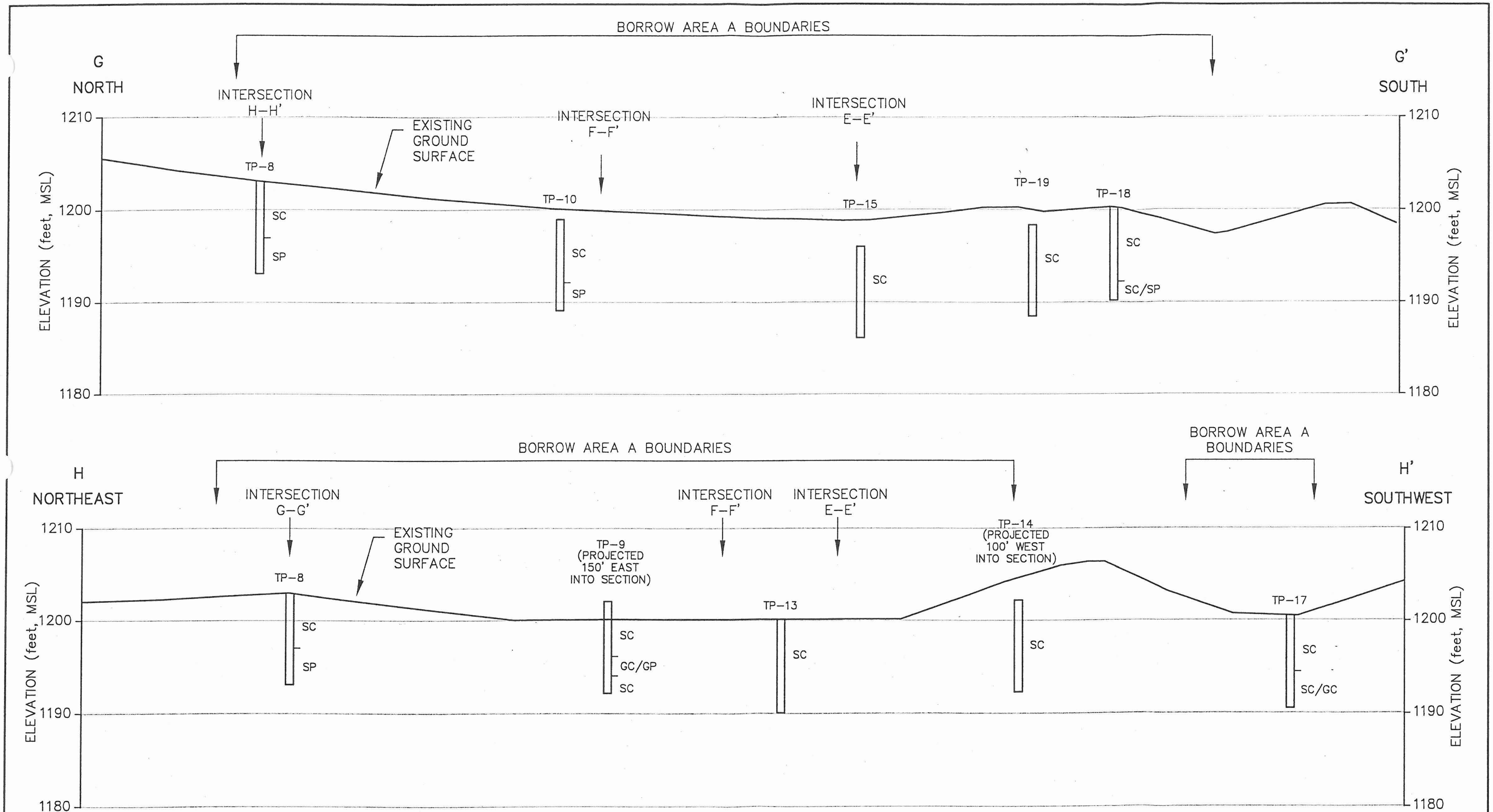
EMERGENCY SPILLWAY (BORROW AREA A)
GENERALIZED CROSS SECTIONS E-E' AND F-F'
WHITE TANKS F.R.S. #3

URS

100 0 100 200 Feet
SCALE, HOR: 1"= 200', VERT: 1"= 10'

CHECKED BY:	DATE: 8–20–04	FIG. NO:
PM:	PROJ. NO: 23443748.00037	7-5

X:\23443748\WT3 generalized xsect.dwg Aug 20, 2004 – 11:40am



LEGEND:

TP–2

SC

SM

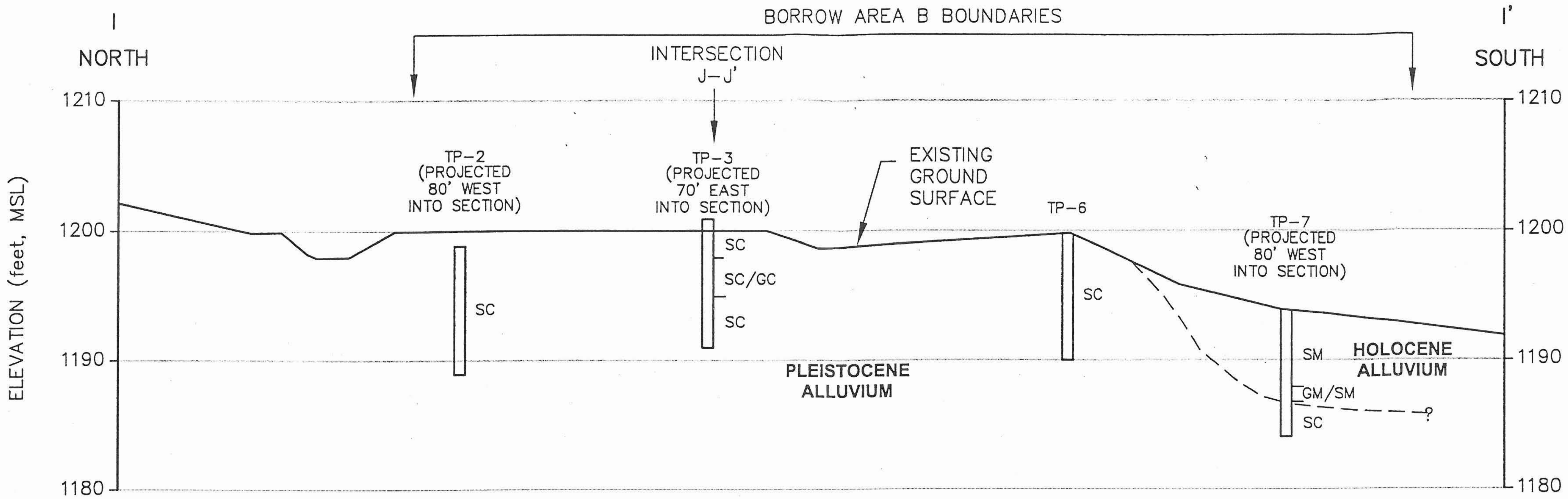
EXPLORATORY TEST PIT WITH USCS LITHOLOGIC CLASSIFICATIONS

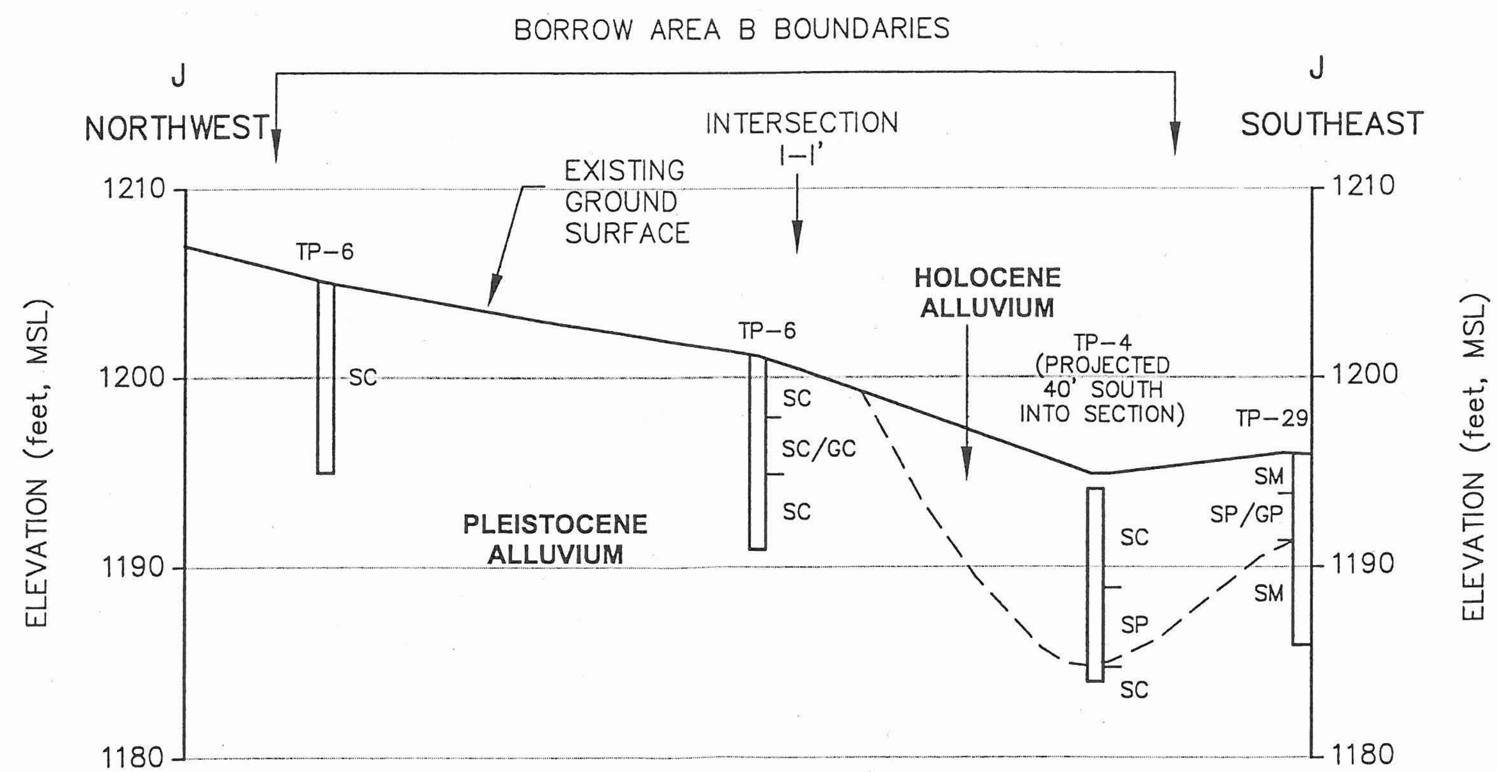
VERTICAL EXAGGERATION = 20x

EMERGENCY SPILLWAY (BORROW AREA A)
GENERALIZED CROSS SECTIONS G-G' AND H-H'
WHITE TANKS F.R.S. #3

URS	100 0 100 200 Feet SCALE, HOR: 1"= 200', VERT: 1"= 10'	CHECKED BY:	DATE: 8–20–04	FIG. NO:
		PM:	PROJ. NO: 23443748.00037	7-6

X:\23443748\WT3 generalized xsect.dwg Aug 20, 2004 – 11:40am





LEGEND:

TP–2

SC

SM

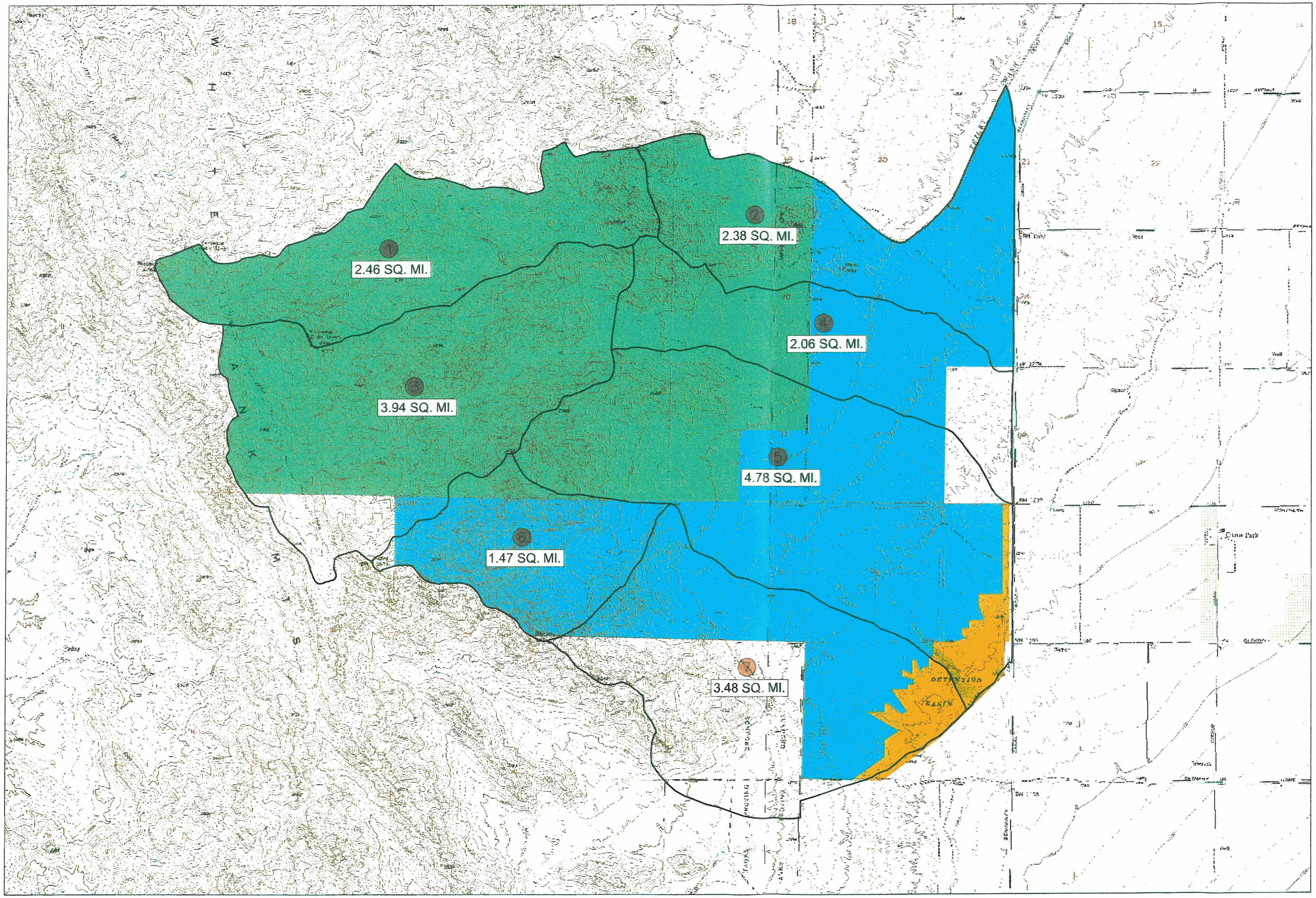
EXPLORATORY TEST PIT WITH USCS LITHOLOGIC CLASSIFICATIONS

VERTICAL EXAGGERATION = 20x

EMERGENCY SPILLWAY (BORROW AREA B)
GENERALIZED CROSS SECTIONS I-I' AND J-J'
WHITE TANKS F.R.S. #3

URS	100 0 100 200 Feet	CHECKED BY:	DATE: 8-20-04	FIG. NO:
	SCALE, HOR: 1"= 200', VERT: 1"= 10'	PM:	PROJ. NO: 23443748.00037	7-7

X:\23443748\WT3 generalized xsect.dwg Aug 20, 2004 – 11:39am



Legend
Surface Ownership
- Private
- State Trust
- Flood Control District of Maricopa County
- Regional Parks

0 2000 4000
Scale in Feet

Source: Provided by FCDMC
Reference: USGS Topographic 7.5 Minute Quadrangles
Weddall, AZ 1957, Photorevised 1971 and
White Tank Mts., AZ 1957, Photorevised 1971.

P:\FCDMC\23443698 WHITE TANKS\CADD\FIGURES\A16206.DWG 6-17-04

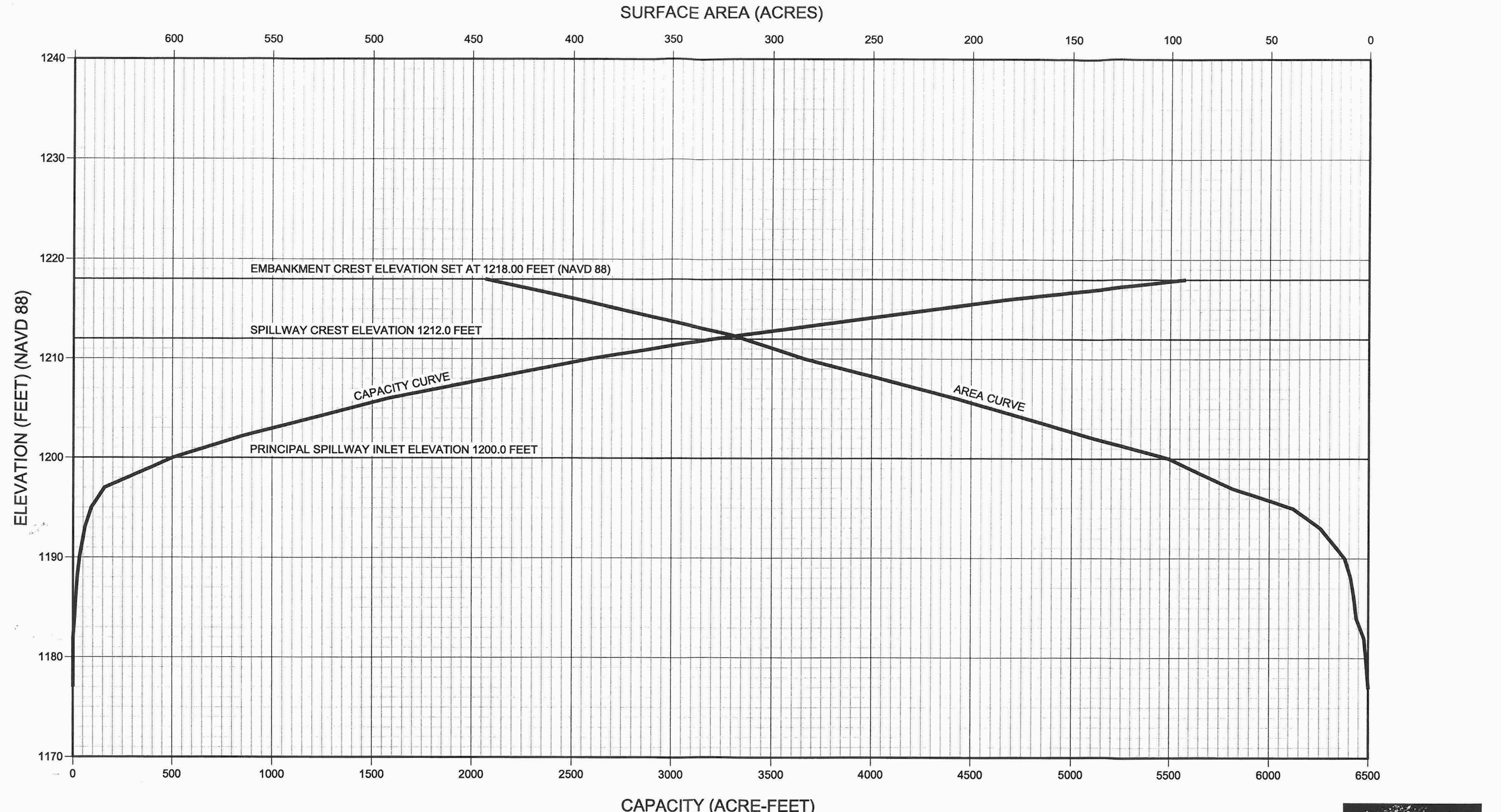
URS

NRCS
Natural Resources Conservation Service

Flood Control District of Maricopa County

WHITE TANKS
FRS # 3

White Tanks FRS No. 3 Watershed Delineation Map
Figure 8-1



P:\FCDMC\23443698 WHITE TANKS\CADD\FIGURES\A16271.DWG 9-10-04

URS

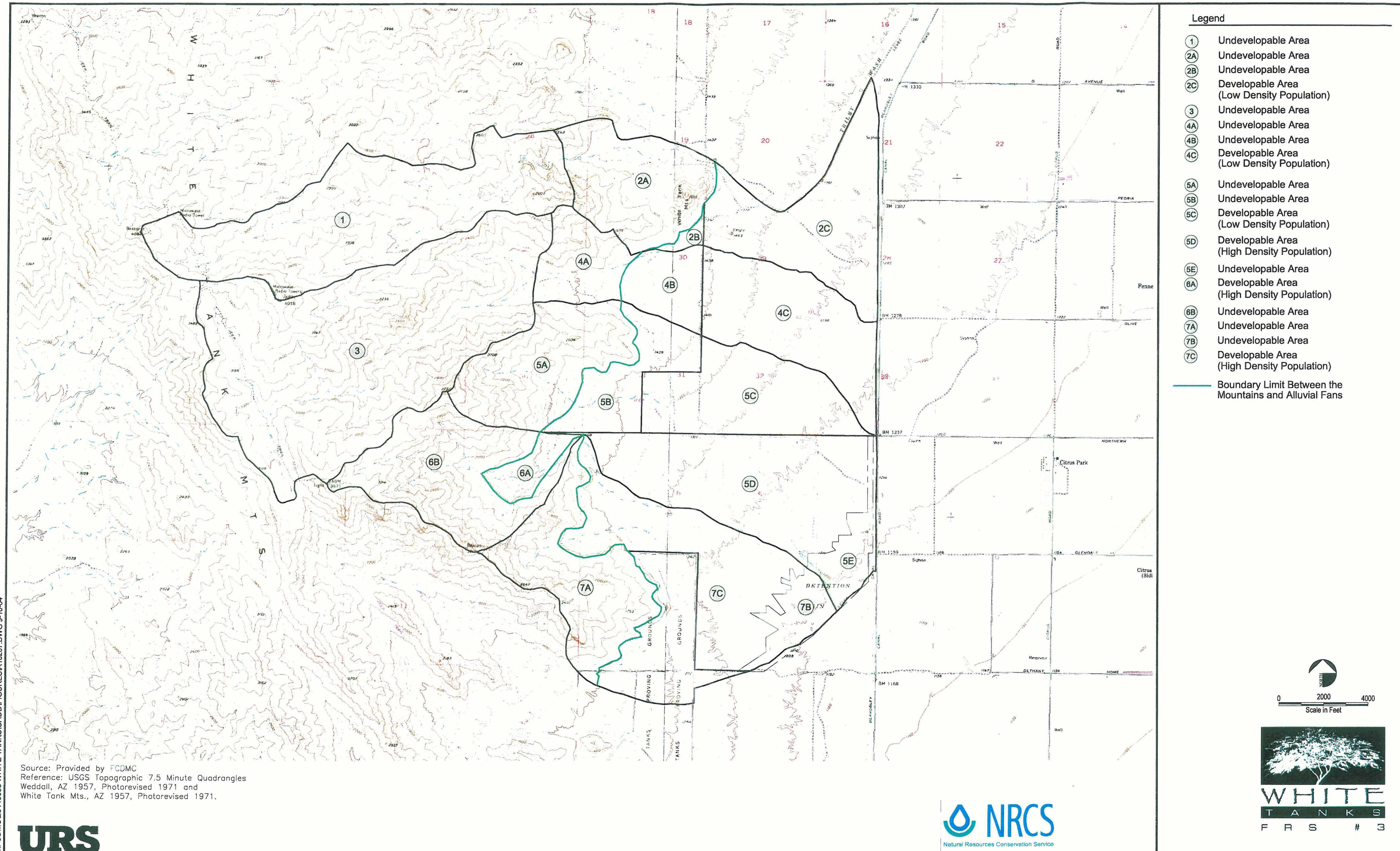
NRCS
Natural Resources Conservation Service

Flood Control District of Maricopa County

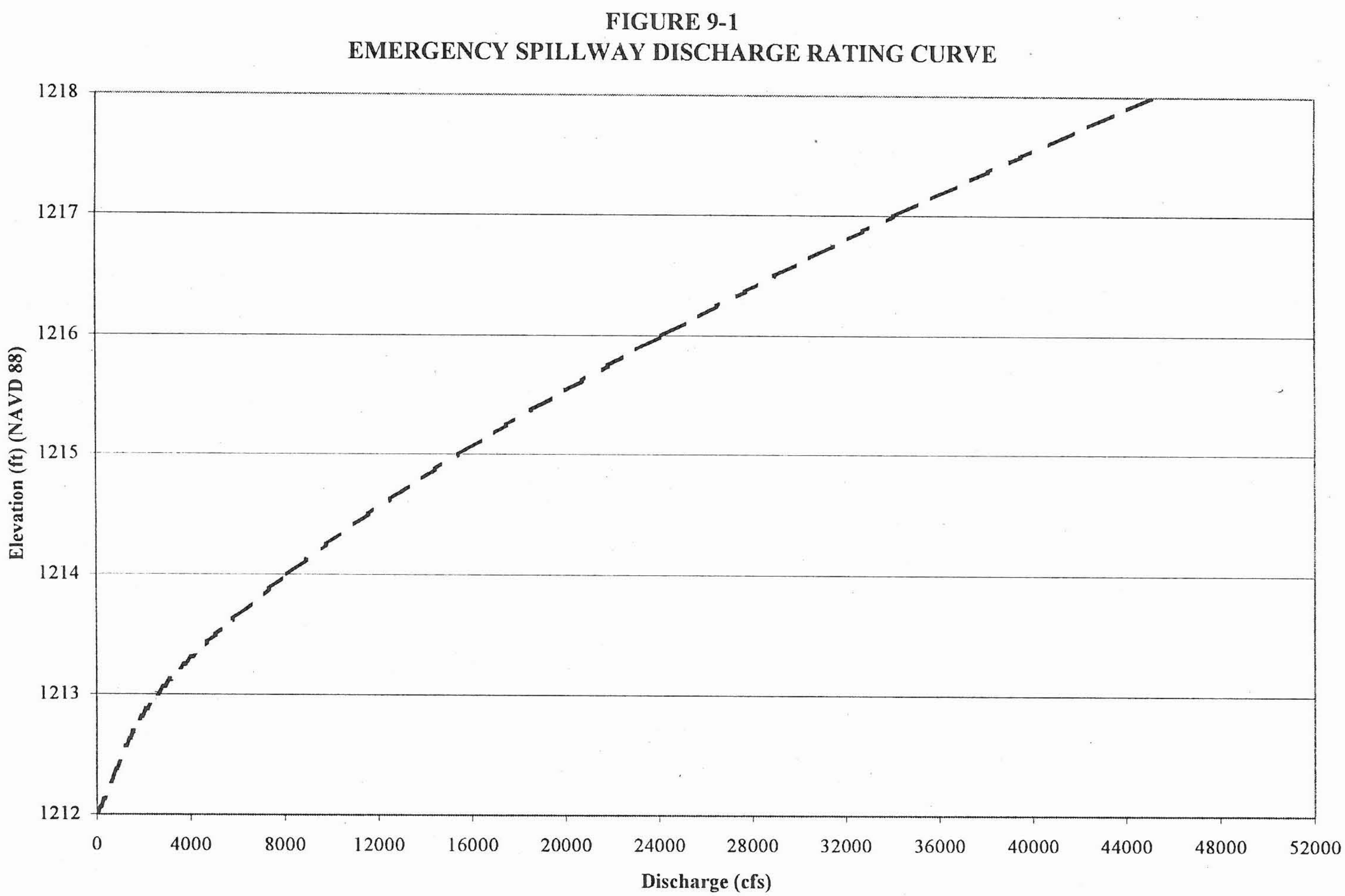
WHITE TANKS FRS # 3

Elevation-Area-Capacity Curve

Figure 8-2



Basin Distribution of White Tank FRS No. 3 Watershed
Based on Urban Growth Projection By Year 2030
Figure 8-3



P:\FCDMC\23443698 WHITE TANKS\CADD\FIGURES\A16316.DWG 9-10-04

URS

NRCS
Natural Resources Conservation Service

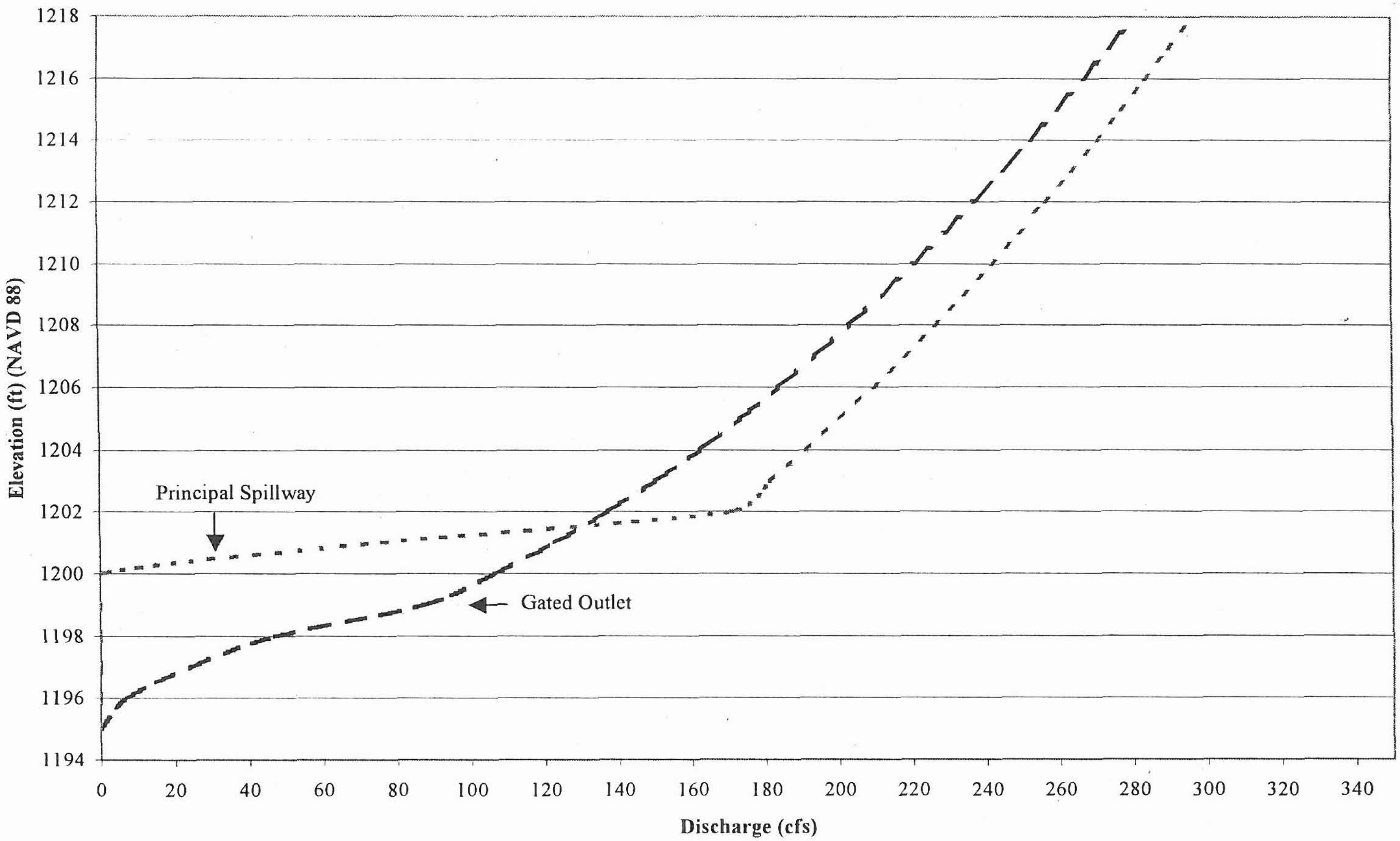
Flood Control District of Maricopa County

WHITE TANKS FRS #3

Emergency Spillway Discharge Rating Curve

Figure 9-1

FIGURE 14-1
OUTLET WORKS DISCHARGE RATING CURVE



P:\FCDMC\23443698 WHITE TANKS\CADD\FIGURES\A16315.DWG 9-10-04

URS

NRCS
Natural Resources Conservation Service

Flood Control District of Maricopa County

WHITE TANKS FRS # 3

Outlet Works Discharge Rating Curve

Figure 14-1