URS

30 PERCENT
DESIGN REPORT
WHITE TANKS FLOOD
RETARDING STRUCTURE NO. 3
REMEDIATION PROJECT

Prepared for
FLOOD CONTROL DISTRICT OF
MARICOPA COUNTY

URS Job No. 23443748
APRIL 23, 2004

Flood Control District
of Maricopa County

INTEROFFICE MEMORANDUM

Date: April 26, 2004

To: **Tom Renckly, Joe Rumann, Bob Stevens, Dennis Holcomb**

From: Larry Lambert

Subject: White Tanks FRS No. 3 – 30% Design Submittal

Attached are the 30% Design Plans, Report and Construction Specifications from URS. A design review meeting is scheduled for May 4 at 8:30 AM at URS's Offices. Review comments are due to URS by May 7. Please provide me any specific comments no later than Thursday, May 6.

Enclosure

File: 470.04.30

URS

April 22, 2004

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

Re: 30 Percent Submittal
Draft Design Report
White Tanks FRS No. 3 Remediation Project
PCN 470.04.30
Contract Number: FCD2003C055
URS Job No. 23443748

Dear Mr. Lambert:

URS Corporation (URS) has prepared this Draft 30 percent Design Submittal for the referenced project. The 30 percent submittal includes a Draft Design Report, Plans, and Specifications.

During preparation of this submittal, we have identified certain aspects of the design configuration that we believe warrant your close attention. These items are discussed below:

- The overall configuration of the upstream liner in the non-fissure risk zone, including the sequence of the various components, the need to drain the sand layer, and the thickness of the sand layer.
- The need (or lack thereof) of a sand diaphragm filter around the principal outlet conduit through the embankment within the soil-cement section.
- The definition of the fissure risk zone as defined by the Preliminary Geotechnical Investigation Report prepared by AMEC Earth & Environmental.
- The approach employed by URS to develop topographic mapping for the area north of the existing dam. This issue is discussed in detail in Section 4.0 of the report.
- The cutoff walls for the embankment within the fissure risk zone includes a geomembrane to improve water tightness. This configuration should be checked for consistency with the configuration currently proposed for the McMicken Dam FRZR Project.
- The TR-20 models developed by NRCS and used in this design include diversions at Olive and Northern Avenues. This reduces the total volume of water and peak flow reaching the reservoir. The diversions are discussed in Section 8.3.7 of this report.
- During construction of the fissure risk zone dam (i.e., soil cement section) the existing dam crest will be lowered to the 100-year flood pool elevation during excavation. This approach should be evaluated for concerns of reduced dam height during construction.

URS

We look forward to meeting with you on May 4, 2004 to discuss your comments on the 30 percent submittal. 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

Page

LIST OF TABLES

[To be completed for 60 Percent Submittal]

LIST OF FIGURES

[To be included with 60 Percent Submittal]

LIST OF APPENDICES

[To be included with 60 Percent Submittal]

1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION

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 designed and installed sand diaphragm filters around the three principal outlets. The centerline filter does not extend to the foundation soils.

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, 2004) on behalf of the District has identified a fissure risk zone that intersects the embankment and extends for a length of approximately 2500 feet between Stations 30+00 and 55+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.

1.2 SCOPE OF WORK

The overall objective of this project is to design modifications to the dam and its appurtenant facilities to mitigate risk related to dam safety concerns at this facility and to meet current regulations and standards as provided by the NRCS and the Arizona Department of Water Resources (ADWR). This overall objective will be achieved by completion of a series of tasks. These tasks were discussed in detail in Scopes of Work for Work Assignments 1 and 2, dated January 21, 2004 and _________, respectively. The key elements of URS' scope of work are summarized below.

Hydrology & Hydraulics

- Review the existing Hydrologic Analysis Report prepared by the NRCS (1998)
- Develop a stage storage relationship for the reservoir
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Geotechnical Investigations

- Review previous geotechnical investigations performed by the NRCS (1992)
- Review preliminary geotechnical investigations performed by AMEC (2004)
- Prepare and implement a geotechnical work plan for additional investigations at the facility
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-

Design

- Design modifications to the embankment within and outside the fissure risk zone
- Design necessary modifications to the emergency spillway
- Design modifications to, or new principal outlets for the dam
- Evaluate historic ground subsidence and assist the District in selecting a design subsidence rate for the project
- Prepare construction drawings, plans, and specifications
- Prepare a Design Report in accordance with ADWR guidelines
- Prepare a Construction Quality Assurance Plan
- Prepare a Construction Schedule

1.3 AUTHORIZATION

URS is performing the design modifications for the dam under contract FCD 2003C055. Two Work Assignments have been reviewed and approved by the District. The Notice to Proceed (NTP) for Work Assignment No. 1 is dated January 21, 2004, and the NTP for Work Assignment No. 2 is dated ____________________.

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 because of its height and reservoir capacity. ADWR currently provides regulatory oversight for jurisdictional dams in Arizona.

1.5 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 is approximately 1.8 feet depending on location. Because of the potential confusion, whenever an elevation is provided in this report the relevant datum is provided.

2.0 FACILITY DESCRIPTION

White Tanks FRS #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.

2.1 ORIGINAL CONFIGURATION

2.1.1 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.

2.1.1.1 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.

2.1.2 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 to the outlet works inlet elevation of approximately 1,188 ft.

2.1.3 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 1,211.92 feet (NGVD 29). The surface area of the flood pool at the emergency spillway crest was 280.6 acres.

2.1.4 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. The channel significantly increases the size of the watershed contained by White Tanks FRS #3: with the channel, the watershed is 20.49 mi^2; without the channel, the watershed would be 9.72 mi^2 (NRCS 1998).

Subsidence at the north end of the dam and along the North Inlet Channel require 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.

2.1.5 Sediment Pool

The NRCS has estimated a design sediment pool of 500 acre-feet (NRCS 1996) corresponding to a 100-year design life. The basis of the 500 acre-ft estimate is not evident from the available documentation. The 500 acre-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).

2.1.6 Emergency Spillway

The emergency spillway for the facility 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 conditions, the flow depths and velocities at the crest of the spillway would range from 2 to

4 feet and 5 to 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.

2.1.7 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 dike consists of a ditch along Bethany Home Road bordered by embankments above the general terrain elevation. These embankments follow the ditch in two 90-degree turns: one about 1,500 ft southwest of the southwest end of the dam embankment; and the second about the same distance to the northwest of the southwest end of the dam embankment.

2.1.8 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. 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 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.

2.2 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.

2.2.1 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.

2.2.2 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 springline. 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.

2.2.3 Emergency Spillway Modifications

In 2000, the District retained URS to design interim dam safety measures, that 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 feet (NGVD 29).

3.0 PREVIOUS STUDIES AND PROJECTS

[Section 3.0 to be completed with the 60 percent submittal]

3.1 ORIGINAL NRCS DESIGN

3.2 NRCS MODIFICATIONS WORK PLAN

3.3 MODIFICATIONS DESIGN PROJECT

3.4 BASINS ALTERNATIVES PROJECT

3.5 INTERIM DAM SAFETY PROJECT

3.6 PRELIMINARY DESIGN CONCEPTS

3.7 DAM ALTERNATIVES PROJECT

3.7.1 Preliminary Geotechnical Investigations

3.7.2 Dam Alternatives Analysis

4.0 TOPOGRAPHIC MAPPING

The topographic mapping prepared for the project site consisted of 2 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 include the existing dam and a majority of the reservoir flood pool. The 2003 topography was developed using the NAVD 88 Datum.

Early design evaluations indicated that the left abutment of the dam would need to be extended 2,500 feet north to include the maximum flood pool. The extension placed approximately 1,500 feet of dam off the 2003 topography. In addition, pool elevations above 1,212 feet (NAVD 88) extended off the 2003 topography.

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 effected the 1998 topographic mapping is the subsidence that has likely occurred since 1998 and 2003. Therefore, URS manipulated the DTM file and shifted data to match the 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 shift was calculated to be an increase in elevation of 1.873 feet using the NGS VERTCON calculator.

- 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 0.069 feet.

Therefore, the DTM file was shifted up in elevation by 1.80 feet and a topographic map was developed. It is important to note that the design of the dam in the areas where the topographic mapping is based on the shifted 1998 topography may result in potential errors in quantity estimates.

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, subsidence prediction, hydrology, etc.) will meet the 100-year criteria.

6.0 LAND SUBSIDENCE

[Section 6.0 to be completed with the 60 percent submittal]

6.1 GEOLOGIC SETTING

6.2 GROUNDWATER CONDITIONS

6.3 FUTURE SUBSIDENCE

6.3.1 Projected Population Growth

6.3.2 Projected Water Demand

6.3.3 Projected Groundwater Conditions

6.3.4 Projected Subsidence

6.3.5 Uncertainties

6.3.6 Design Subsidence Rate and Magnitude

7.0 EARTH FISSURES

[Section 7.0 to be completed with the 60 percent submittal]

7.1 MECHANICS OF FISSURE DEVELOPMENT

7.2 FISSURE RISK ZONE

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. It is unclear if the design storage below the spillway crest included the sediment pool storage of 500 ac-ft.

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 Appendix __.

8.2 REVIEW OF EXISTING MODELS

URS reviewed the existing hydrologic/hydraulic analyses and models documented by Natural Resource Conservation Services (NRCS) in a report titled as (NRCS 1998). NRCS developed PMF flood 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. In addition, NRCS also developed inflow hydrographs for Emergency Spillway (ESH) and the 100-year, 10-day storm. The ESH hydrograph is based on a hyetograph that combines the 100-year, 6-hour and 6-hour Local PMP. 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).

NRCS also developed a model for the 100-year, 24-hour storm event. A summary of the results for the 100-year, 24-hour storm is provided in Table II of the NRCS hydrology report (NRCS 1998). Based on these results, the 6-hour Local PMF was determined to the critical design storm. URS verified the derivation of the above-mentioned inflow hydrographs. 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.

URS also checked the derivation of the 100-year 24-hour and 100-year 10-day hydrographs as presented in NRCS hydrologic report (NRCS 1998) and confirmed that the presented hydrographs are derived correctly per 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 peak/volume from the 100-year, 10-day storm is less than that for the 100-year, 24-hour storm.

In summary, URS found the NRCS's derivation of design hydrographs for the White Tanks FRS No. 3 watershed (NRCS 1998) to be reasonable with no major objection.

URS also reviewed the electronic versions of the NRCS's TR-20 models provided to by the District. 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, Emergency Spillway Hydrograph (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

URS modified the existing TR-20 computer models 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. reference

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 9-1). The revised drainage areas, and those estimated by NRCS, are presented in Table 9-1. A review of Table 9-1 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 (see Table 9-1). Therefore, the drainage areas developed by NRCS were used in the updated TR-20 models.

8.3.2 Reservoir Elevation-Storage Curve

A new elevation-storage curve for the White Tank FRS No. 3 was developed using the 2003 topographic map in combination with the modified 1998 topographic map, both of which were provided by the District. The elevation-storage curve was established using the end-area method as described in Table 17-2 of *National Engineering Handbook, Section 4 Hydrology* (USDA 1985). The elevation-storage data is summarized in Table 9-2 and presented graphically on Figure 9-2. The detailed computations related to determination of elevation-storage curve for White Tank FRS No.3 are provided in a calculation package in Appendix __.

8.3.3 Sediment Pool

The sediment pool volume and elevation are important for determining the antecedent reservoir condition for routing of certain design storms. The *Plan and Environmental Assessment for White Tank Mountains Watershed* (NRCS 1996) indicates that the sediment pool is 500 acre-feet. It is unclear whether this is a 50-year or 100-year sediment pool. Additional research suggests that the 100-year sediment pool might be 600 acre-feet. The 600 acre-ft allowance for sediment accumulation corresponds to an elevation of 1200.1 feet (NAVD 88).

Current routing will be performed assuming the 100-year sediment pool is 600 acre-feet. NRCS is researching their design documents to help resolve this question. In addition, URS will evaluate other sediment volume estimates developed by the District for similar watersheds. The 600 acre-ft allowance for sediment accumulation corresponds to an elevation of 1200.1 ft.

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 a weighted average of 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 9-2.

8.3.5 Precipitation

The Probable Maximum Precipitation (PMP) estimates presented in the NRCS hydrologic report (NRCS 1998) were developed using the generally accepted procedures in Hydrometeorological Report No. 49. The rainfall estimates for the 100-year 24-hour and 100-year 10-day storm were also verified. The methods used to estimate the precipitation values for these storms were derived correctly per the cited references (Chapter 21 of NEH-4, and Hydrologic Notes PO-4 and PO-6).

It should noted that for routing purposes, URS did not modify the precipitation values or rainfall distributions within the TR-20 models provided by the District.

8.3.6 Curve Number Estimation

8.3.6.1 Existing Conditions

The White Tanks FRS No. 3 watershed consists generally of undisturbed desert and mountain areas. The curve numbers presented in the NRCS hydrology report (NRCS 1998) are presented in Table 8-2. The curve numbers developed by NRCS are reasonable for this application.

8.3.6.2 Future Conditions

To address the issue of impacts resulting from potential future development in the watershed, the TR-20 models were updated to incorporate anticipated future land use. The future urban growth potential of different basins in White Tank 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 modified watershed delineation map (See Figure 8-3). 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 determined to by undevelopable. Private Property and State Trust Land were deemed to have the potential for development. However, a further distinction was made where any of the potentially developable land within the mountainous terrain (i.e., steep slopes) was determined to be undevelopable. A line dividing the mountain and valley regions is shown on Figure 8-4.

Within the potentially developable properties

Developable areas were separated into low-density and high-density areas based on the information provided 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 8-2 and shown on Figure 8-4.

Based on the criteria defined above, White Tank FRS No. 3 watershed was primarily divided into 3 categories:

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

8.3.6.2.1 Curve Number for the Mountain Regions

A curve number of 87.2 was used in the mountain region, which was based on the NRCS estimate (NRCS 1998). This curve number was applied to Basins 1,3, and 6 as shown on Figure ___.

8.3.6.2.2 Curve Numbers for the Valley Region (Undevelopable)

Basins 2, 4, 5, and 7 have sub-basins within the valley region which are considered undevelopable based on land ownership conditions. For these areas, curve numbers were estimated using the following relationship:

$$\text{Curve Number} = [(\text{Area}_{total} * \text{CN}_{NRCS}) - (\text{Area}_{Mountain} * 87.2)] / \text{Area}_{Valley}$$

Where

Area_{total} = Total area for the basin;

CN_{NRCS} = Curve number assigned for that basin in NRCS hydrology report (NRCS 1998);

$\text{Area}_{Mountain}$ = Area within the mountain region;

Area_{Valley} = Area within the valley region

The CN is ?

8.3.6.2.3 Curve Numbers for the Valley Region (Developable)

Developable lands areas which are located in the valley region and located on Arizona State Land or Private Property. These areas are further classified as either low-density or high-density populated areas based on the criteria already defined in Section 4.1. Curve numbers for developed areas 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. Details of the curve number derivation is presented in the calculation packages provided in Appendix __.

8.3.6.2.4 Curve Number Estimates for Future Conditions

The curve number estimates for the 3 categories of surface conditions were used to develop a curve number for each basin using an area-weighted average. The resulting curve numbers were incorporated into the TR-20 models used for design.

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 a stormwater 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, 100-year, 24-hour flows are allowed the reach the reservoir, but flows exceeding the restrictions are diverted out of the watershed.

At Olive Avenue, flows greater than 4,100 cfs are diverted out of the watershed. At Northern Avenue, flows greater than 11,000 cfs are diverted out of the watershed. 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. use N/C project

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

An elevation-discharge relationship for the White Tank FRS No. 3 emergency spillway was developed based on the 2003 topographic mapping provided by the District. The hydraulics of reaches upstream and downstream of the spillway were defined using a Corps of Engineers program HEC-RAS. This program allows for computation of water surface elevation in irregular, natural channels, using a gradually varied flow assumption and basic energy and momentum equations.

9.2 REVIEW OF EXISTING MODEL

The HEC-RAS model developed by the District modeled the reach down stream of the emergency spillway with the furthest upstream cross-section taken at the spillway crest. The District model was based on a spillway crest elevation of 1,210 feet (NGVD 29). The cross-sections indicated the presence of two low flow channels: one near the Bethany Home Road Dike and another near the dam. The District used Manning's roughness values of 0.045 for channel flows and 0.060 for overbank flows. Based on a field visit performed by URS staff the District's estimates for Manning's roughness appeared to be a reasonable representation of the actual conditions.

Overall, the District modeling results appear reasonable. However, Flood District Model indicated a sub-critical flow at the spillway crest and for most of the downstream reach. Therefore, it was determined that the model should be modified to incorporate additional cross-sections upstream of the crest to evaluate potential backwater effects on the reservoir pool elevation. In the process of adding cross-sections, the downstream cross-sections were also modified.

9.3 DEVELOP DESIGN MODEL

A new HEC-RAS model was developed using the 2003 topographic mapping provided by the District. The HEC-RAS modeling was performed with the following assumptions:

- The Bethany Home Road Dike will be reconstructed on District property. The modeling assumes that the dike will not fail during passage of the spillway design flood, even though it is not protected against erosion. The non-failure scenario was considered to be the most conservative for estimating the maximum flood pool elevation. HEC-RAS cross-sections were developed assuming that a 3:1 slope started at the toe of aesthetic fill covering the dike.

- Aesthetic fill will be placed on the dam. This will effect both upstream and downstream cross-sections. HEC-RAS cross-sections were developed assuming that a 3:1 slope started at the toe of aesthetic fill covering the dike.

- The notch within the spillway will be filled and a hardened structure will be installed to maintain the spillway crest at 1,212 feet (NAVD 88).

9.3.1 Cross-Section Development

The 2003 topographic mapping provided by the District was imported into AutoCAD. The river modeling software, BOSS RMS (River Modeling System) was used to draw cross-sections for the modeled reach and develop the necessary cross-section data for input to HEC-RAS.

In previous studies the cross sections were taken only up to the spillway crest. However, in order to incorporate the backwater effects, for this particular study, additional cross sections were also taken upstream of the emergency spillway crest. The cross-sections developed for the HEC-RAS model shown on Figure 8-1. Cross-section 11 is located at the spillway crest.

9.3.2 Roughness Coefficients

Based on the field visit and a review of the District HEC-RAS model, the Manning's roughness values were not modified. The model uses Manning's roughness values of 0.045 for the channel areas and 0.060 for overbank areas.

9.3.3 Flow Rates

The HEC-RAS modeling was performed for a range of flow rates between 1,000 to 30,000 cfs. This range was selected to provide an spillway rating curve appropriate for the modeled storms and anticipated flow rates.

9.3.4 Development of Emergency Spillway Rating Curve

Examination of flow rates and velocities was performed for the higher flow rates to determine which cross-section best represented the reservoir flood pool. Model results showed that cross-section 14 and subsequent upstream cross-sections showed minimal variation in the energy grade line and water surface elevations. Therefore, the spillway rating curve was developed based on the water surface elevations calculated at cross-section 14. Cross-section 14 is located approximately 900-feet upstream of the spillway crest. The detailed HEC-RAS modeling results are presented in a calculation package provided in Appendix __.

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
- 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 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 1985), and include:

- The sediment storage volume will be based on the estimated 100-year sediment inflow.
- The emergency spillway crest will be set at an elevation to contain back-to-back 100-year, 10-day storms. This condition is required because the reservoir has gated outlets and no principal spillway.
- The dam crest elevation will be set at an elevation above the maximum water depth during routing of the "worst-case" PMF hydrograph.
- The antecedent reservoir condition for the PMF flood 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.

The 100-year 10-day hydrograph was routed through the reservoir by setting the starting routing elevation corresponding to 100-year sediment pool. URS then obtained the reservoir elevation at the end of 10^{th} day following the peak of a 100-year 10-day storm and set that elevation as starting routing elevation for PMF flood hydrographs.

10.2.2 ADWR 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, 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 1994), and include:

- Based upon the size and hazard classification of the dam, the crest of the embankment will be based on the "worst case" elevation from routing of the PMF and the addition of a residual freeboard.
- As per District recommendations, the residual freeboard will be 1 foot.
- The antecedent reservoir condition for the PMF flood hydrographs will be based on the water surface elevation 10 days following the end of the 100-year, 24-hour storm. Since the outlets are gated, drawdown of the reservoir for this case was the result of infiltration.
- ADWR criteria requires that the reservoir following the 100-year, 24-hour storm peak should be emptied in 10 days or less or contain less than 15 percent of the maximum retarding volume storage at the end of 10^{th} day. The maximum retarding volume storage would be the peak volume for 100-year 24-hour storm. If this criterion is met, the starting routing elevation for PMF flood hydrographs would be the same as for 100-year 24-hour storm (i.e., the sediment pool). If this criterion cannot be met, the starting routing elevation for the PMF floods would be the reservoir elevation at the end of 10th day following the peak of a 100-year 24-hour storm. Since the outlets are gated, drawdown of the reservoir for this case was the result of infiltration.

The 100-year 24-hour hydrograph was routed through the reservoir by setting the starting routing elevation corresponding to 100-year sediment pool. URS found out that the White Tank reservoir not been able to draw down the 85 percent of the peak storage volume at the end of 10^{th} day following the peak of 100-year 24-hour storm. Therefore, the starting routing elevation for the PMF floods was set at the reservoir elevation at the end of 10th day following the peak of a 100-year 24-hour storm.

10.2.3 District Models

TR-20 models were developed for the 200- and 500-year, 24-hour storm events based on conditions. 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 (see Figure 9-5). Based on this depth-duration relationship, the 200-year, 24-hour rainfall depth was estimated. The computations related to development of the 200- and 500-year, 24-hour rainfall depths are provided in a calculation package in Appendix __. The 200- 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).

10.3 MODELING RESULTS

The TR-20 modeling developed based on ADWR, NRCS, and District requirements was performed. Reservoir routing results i.e. peak inflows, outflows, storage volume and reservoir stage for each hydrograph are summarized in Table 9-3. Detailed output results of TR-20 modeling for 200- and 500-year, 24-hour storms are provided in an attached calculation package under Appendix __.

10.3.1 NRCS Models

Based on the reservoir routing results, the 6-hr local PMF storm was determined to have the highest reservoir pool elevation. The reservoir routing results also indicated that the Whit Tank FRS No.3 reservoir could contain two 100-year, 10-day back-to-back storms below the emergency spillway crest. The Emergency Spillway Hydrograph (ESH) was also routed through the reservoir. The ESH hydrograph was based on a hyetograph that was combination of the 100-year, 6-hour and 6-hour local PMP. The antecedent reservoir condition for ESH was set same as of PMF design flood hydrographs. URS found out that the ESH is less than the free board hydrograph.

10.3.2 ADWR Models

Based on the reservoir routing results, the 6-hour local PMF storm was determined to have the highest reservoir pool elevation. As per the ADWR design criteria, 1 foot of freeboard was added to the maximum water surface elevation of the 6-hour local PMF flood hydrograph. The reservoir routing results indicated the 100-year 24-hour storm was contained below emergency spillway crest. The antecedent reservoir level for the routing purpose for the 100-year 24-hour storm was set at 100-year sediment pool level.

10.3.3 District Models

The antecedent reservoir elevation for the routing purposes for the 200-year and 500-year 24-hour storms was set at 100-year sediment pool level. Based on the reservoir routing, the 200-year and 500-year 24-hour resulted in a discharge through the emergency spillway.

10.4 ADDITIONAL ROUTING MODELS

[To be completed with 60 Percent Submittal]

10.4.1 Existing Condition

10.4.2 Original Spillway Condition

10.4.3 Bethany Home Road Dike Condition

11.0 GEOTECHNICAL CONSIDERATIONS

11.1 PREVIOUS INVESTIGATIONS

11.1.1 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.

11.1.2 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.

11.1.3 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. 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. 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.

11.1.4 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.

11.1.5 Current URS Investigations

URS has prepared this work plan for a geotechnical investigation at White Tanks FRS No. 3 in support of rehabilitation design for the dam and its appurtenant facilities. The formal Work Plan (URS 2004) describes in detail, the geotechnical investigation for this project. Key aspects of the proposed work plan are summarized below:

- **Review of Existing Information:** URS will review and summarize geotechnical data collected during previous investigations at White Tanks FRS No. 3. Key documents that will be reviewed include the Geotechnical Data Report prepared by URS (2001), and the Preliminary Geotechnical Investigation Report prepared by AMEC (2004). In addition, URS will also review other applicable published articles and reports on the geologic setting and the geotechnical conditions at White Tanks FRS No. 3.

- **Depth of Holocene Soils at the Embankment:** The depth of the Holocene soils at the embankment will be investigated through seismic refraction surveys, exploratory drilling using hollow stem augers, and test pit excavation. Selected samples collected during the field investigation will be forwarded to a soils laboratory for testing. The number and types of tests will depend on the conditions encountered during the field investigation.

- **Erodibility of Pleistocene Soils:** URS proposes to evaluate the erodibility of the Pleistocene soils in the fissure risk zone through a combination of geophysical surveys, exploratory drilling, and laboratory testing. a continuous seismic refraction survey will be performed along the upstream toe of the embankment within the fissure risk zone. Up to 10 borings will be drilled to depths ranging from 40 to 100 feet using either a modified Pitchers sampler, or the triple tube coring technique. The borings will be grouted upon completion. Selected samples collected during the field investigation will be forwarded to a soils laboratory for testing.

- **Borrow Source Investigation:** The District has identified two potential borrow sources within the flood pool of the dam. The field investigation for the borrow sources will include approximately 20 test pits to depth of 10 feet. Up to 40 bulk samples will be collected in plastic buckets for laboratory classification and testing. Upon completion of the excavation and sampling activities, the test pits will be backfilled with soil cuttings.

- **Emergency Spillway Investigation:** Surface seismic refraction surveys will be performed in the emergency spillway. A continuous seismic profile will be generated along the spillway crest alignment. The field investigation at the emergency spillway will include approximately 10 relatively shallow borings (5 to 10 feet in depth) to evaluate the erosion resistance of the soils in the emergency spillway. Upon completion of drilling and sampling activities, the borings will be backfilled with a cement-bentonite grout. Selected samples collected during the field investigation will be forwarded to a soils laboratory for testing.

11.2 EMBANKMENT CONDITIONS

11.2.1 Soils

Previous investigations by Dames & Moore (1998) indicate that the embankment soils are predominantly clayey sands with lesser amounts of sandy clays present. The fines contents of the clayey sands vary from 23 to 35 percent, and the PIs vary 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 (PI = 7 percent to 13 percent) 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.

Dames & Moore (1998) performed laboratory tests to evaluate shear strength parameters for the embankment soils at White Tanks FRS No. 3. Triaxial tests were performed on three 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 36 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 21 to 31 degrees, and the cohesion ranged from 50 to 300 psf.

11.2.2 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.

11.2.2.1 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.

11.2.2.2 Failure Modes Related to Transverse Cracking

[Remaining sections to be completed with the 60 percent submittal]

11.3 FOUNDATION CONDITIONS

11.3.1 Soils

11.3.2 Impact on Embankment Design

11.4 SLOPE STABILITY

11.4.1 General

11.4.2 Cases Considered

11.4.3 Shear Strength Parameters

11.4.4 Seepage Considerations

11.4.5 Seismic Considerations

11.4.6 Analyses

11.4.7 Stability Analyses Results

11.4.8 Discussions

11.5 FLOW ALONG A FISSURE

11.5.1 General

11.5.2 Soil Erodibility

11.5.3 Erosion Modeling

11.5.4 Discussions

11.6 SETTLEMENT

11.6.1 Embankment Soils

11.6.2 Foundation Soils

11.6.3 Discussions

12.0 EMBANKMENT DESIGN

12.1 GENERAL

Section 12.0 of this report discusses the proposed embankment configurations for the White Tanks FRS No. 3 Remediation Design. In addition to the physical dimensions of the embankment sections, discussions pertaining to the rationale and basis of selection for the various components of the embankment (e.g., Soil cement, geosynthetic elements, etc.) are also included.

12.2 STATIONING

The stationing at the right and left abutments of the new embankment are approximately 10+00 and 110+00, respectively. Station 0+00 is located to the right of the emergency spillway. The new stationing is aligned along the centerline of the new embankment. The new embankment stationing has been rotated and runs the opposite direction from the original embankment stationing. The conversion from old to new stations is detailed in Table 12-1.

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 _____ feet based on NAVD 88. 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 ____ feet, while the south end of the dam has subsided by approximately _____ feet.

12.3.2 Design Requirements

The selection of design elevations for the embankment crest is derived based on the results of routing of the IDF, and minimum freeboard requirements based on routing of the IDF through the reservoir, and an estimate of future subsidence.

12.3.2.1 IDF Routing

Routing of the IDF through the reservoir estimated the maximum water surface elevation behind the embankment for both ADWR and NRCS criteria, as discussed in Section 10.0. The maximum water surface elevation estimated based on the ADWR criteria is 1,218.6 feet (NAVD 88). The maximum water surface elevation estimated based on the NRCS criteria is 1,218.3 feet

(NAVD 88). The maximum water surface elevations estimated based on ADWR and NRCS criteria are not equal due to differences in the antecedent reservoir condition at the beginning of the IDF.

12.3.2.2 Freeboard

ADWR requires that the crest be set at an elevation equal to the maximum water surface elevation plus freeboard. Guidance provided by ADWR suggests that a reservoir used for the single purpose of flood control can use a freeboard of 1 foot. The minimum dam crest elevation required based on the ADWR criteria would be 1,219.6 feet (NAVD 88). NRCS does not require freeboard above the IDF.

12.3.2.3 Subsidence

The evaluation of subsidence is continuing. The drawings presented with this 30 percent design submittal are based on the guidance provided by the District, which assumes that the north end of the dam will experience an additional 1 foot of subsidence in the future. In addition, the north end of the dam has been increased in width to allow for an additional 1 foot raise in the future. The south end of the dam has experienced significantly less historical subsidence. Currently, the rate of subsidence at the south end of the dam is approximately one-quarter that being experienced at the north end of the dam. Therefore, we have incorporated 0.25 feet of additional dam height at the south end of the dam. The south end of the dam has also been increase in width to provide a future raise of 1 foot.

Dam stationing has not been developed for the new dam alignment. Therefore, all references to stations are based on the existing dam stationing. Portions of the new dam located north (left) of Station 30+00 will have a crest elevation based on a future subsidence of 1 foot. The crest elevation of the new dam at Station 76+67 will have a crest elevation based on a future subsidence of 0.25 foot. The crest elevation of the new dam between Station 30+00 and Station 76+67 will be based on a straight line interpolation between the future subsidence values of 0.25-feet and 1.0-foot.

12.3.2.4 Design Crest Elevations

The design crest elevations of the new dam are based on the results of the IDF routing, freeboard requirements, and subsidence predictions. The results of the IDF routing and freeboard requirements indicate that the ADWR criteria result in a dam crest elevation of 1,219.6 feet (NAVD 88), which is greater than the elevation based on the NRCS criteria. The predicted future subsidence will be added to the elevation determined from the routing analysis. Therefore, the design crest elevations for the new dam will consist of the following (Stations are based on existing dam stationing):

- For Station 30+00 and north, the new dam will have a crest elevation of 1,220.6 feet (NAVD 88).
- For Station 76+67, the new dam will have a crest elevation of 1,219.85 feet (NAVD 88).
- Between Station 30+00 and Station 76+67, the new dam will have crest elevations ranging from 1,220.6 feet (NAVD88) at Station 30+00 to 1,219.85 feet (NAVD 88) at Station 76+67.

12.4 FISSURE RISK ZONE

The following sub-sections provide information on the proposed embankment configuration for the section of the dam within the fissure risk zone. As discussed earlier in Section 7.0, the fissure risk zone is defined from Station 30+00 to Station 55+00 (based on existing dam stations), based on investigations performed by AMEC (2004) under a separate contract with the District.

12.4.1 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 section of embankment within the fissure risk zone (See Section 7.0 for definition and description) will include the following steps:

- The entire footprint of the proposed foundation excavation as shown in the design drawings (Appendix __) will be cleared and grubbed in order to remove vegetation and other deleterious materials.
- Overexcavate and remove the underlying Holocene soils. The extent of the Holocene soils will be estimated during the on-going geotechnical investigation. For the purpose of the current 30 percent design phase, URS has assumed a depth of excavation of 10 feet.
- Overexcavate and remove coarse-grained channel deposits exposed within the foundation excavation. The excavation side slopes for this purpose will be no steeper than 2:1 and will be backfilled with soil cement.
- The foundation excavation will be thoroughly inspected and approved by the engineer prior to construction of the embankment.

12.4.2 Embankment

The proposed embankment section for fissure risk zone consists of two components – a soil-cement core, and a surrounding earthfill provided for aesthetic purposes. 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 aesthetic earthfill.
- As currently planned, the soil-cement core has a crest width of 13.2 feet, with 0.6:1 (horizontal to vertical) side slopes. These slopes may be modified depending on the results of slope and lateral stability analyses to be performed during subsequent phases of the project. The crest width of the soil-cement core has be increased in width from the minimum required crest width of 12 feet to provide for a future dam raise of 1 foot.
- Based on limited geotechnical data available from previous investigations, it is anticipated that soil needed for construction of the soil-cement core will be obtained from near-surface, on-site borrow sources. Geotechnical investigations are currently underway to evaluate two potential borrow sources within the reservoir. The geotechnical program also includes soil-cement mix design.
- 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 earthfill 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 are not proposed 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 are not proposed for the soil-cement mix design.

12.4.3 Cutoffs

The current configuration includes two vertical cutoffs – one each at the upstream and downstream toes of the soil-cement component of the embankment. Construction of the cutoff walls will involve excavation of a trench – 3 feet in width to the design depth. A liner system comprising of a geomembrane will install within each trench, and the trench will be backfilled with a flowable fill. The use of a flowable fill will eliminate the need for personnel entry into the trenches.

The depth of the cutoffs will depend on the nature of the foundation soils, and more specifically, on the erosion resistance of the underlying soils. A geotechnical investigation is currently underway to help define these parameters.

12.4.4 Discussions

As noted in Section 12.3.2, the soil-cement component will be designed as the structural core of the embankment, independent of the aesthetic earthfill around the soil-cement core. Removal of the Holocene soils as part of the foundation preparation measures (see Section 12.3.1) is limited to the footprint of the proposed soil-cement core. Within the footprint of the aesthetic fill, 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 aesthetic fill. 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 earthfill, but are not expected to adversely impact the performance of the embankment.

Similarly, the cutoffs 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 aesthetic earthfill, requiring maintenance after significant impoundments.

12.5 NON-FISSURE RISK ZONE – EXISTING EMBANKMENT

For the section of the embankment outside the fissure risk zone, the intent of the proposed design configuration is twofold:

- 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 by providing a composite liner system on the upstream face of the dam.

Figure 12-1 depicts a conceptual configuration of the upstream raise of the embankment outside the fissure risk zone. Construction of the upstream buttress will involve the following steps:

- Clear and grub the upstream face of the existing embankment.
- Construct the inner buttress fill in horizontal lifts.
- Cut or trim the compacted buttress to a 3:1 slope.
- Place a 2-foot thick layer of sand on the buttress fill.
- Install a geomembrane over the sand layer.
- Construct the outer buttress fill in horizontal lifts.

The components of the proposed configuration are discussed below.

12.5.1 Foundation Preparation

As shown in the design drawings, the Holocene soils within the footprint of the proposed upstream soil buttress will be overexcavated and removed. The trapezoidal trench will also serve as the anchor trench for the geosynthetic elements of the composite drain.

12.5.2 Inner and Outer Buttress Fill

Prior to placement of the inner buttress fill, the upstream face of the embankment will be cleared and grubbed. The buttress fill will be placed in horizontal lifts and keyed into the existing embankment. It is anticipated that soils for construction of the inner buttress will be obtained from borrow sources within the reservoir. The outer buttress fill will be placed in horizontal lifts over the geomembrane.

12.5.3 Geomembrane

A geomembrane will be installed on the upstream face of the inner buttress fill. The design drawings in Appendix __ show an 80-mil HDPE textured geomembrane. URS is currently evaluating various candidate membranes for this application.

Relative to the geomembrane, critical issues that will be examined during the on-going geotechnical investigation include:

- Interface strength and stability between the geomembrane and inner buttress fill
- Interface strength and stability between the geomembrane and the sand filter
- Crack simulation testing of the geosynthetic elements

12.5.4 Sand Filter

The design includes a sand filter directly beneath the geomembrane. The sand filter will have a nominal thickness of approximately 2 feet and will be placed in lifts directly on the 3:1 slope of the inner buttress fill. The gradation of the sand filter will meet the soil retention criteria described by the NRCS (______). Filter match analyses will be performed upon completion of the on-going geotechnical investigation.

12.6 NON-FISSURE RISK ZONE – NEW EMBANKMENT

As currently proposed, the dam embankment will be extended to the north of the existing left abutment. Section 12.5 discusses the key features of this new segment of embankment.

12.6.1 Geometry

The new embankment will have a total length of approximately 2,500 feet, a crest width of 18.5 feet, and a crest elevation of 1,220.6 feet (NAVD 88). The upstream and downstream slopes of the embankment will be 3:1 and 2:1, respectively. The dam crest has been increased in width over a minimum required crest width of 14 feet to allow for a future dam raise of 1 foot.

12.6.2 Foundation Preparation

The foundation preparation for the new embankment will include overexcavation and removal of the Holocene soils, and replaced with moisture conditioned embankment fill. The extent of the overexcavation will be established after completion of the on-going geotechnical investigation.

12.6.3 Geomembrane

A geomembrane will be installed on a 3:1 slope within the upstream zone of the new embankment. The design drawings in Appendix __ show an 80-mil HDPE textured geomembrane. URS is currently evaluating various candidate membranes for this application.

Relative to the geomembrane, critical issues that will be examined during the on-going geotechnical investigation include:

- Interface strength and stability between the geomembrane and outer buttress fill.
- Interface strength and stability between the geomembrane and the sand filter.
- Crack simulation testing of the geosynthetic elements.

12.6.4 Sand Filter

The design includes a central sand filter and drain above the existing ground surface. The gradation of the sand filter will meet the soil retention criteria described by the NRCS (______). Filter match analyses will be performed upon completion of the on-going geotechnical investigation.

12.6.5 Erosion Protection

The new embankment will run parallel to the north inlet channel. A certain portion of the new dam length will need to be protected against erosion during flood inflow events. The design includes the placement of rip rap on the upstream face with a D_{50} of __ inches (D_{50} to be determined for 60 percent design). The rip rap will be placed up to the dam crest and 3 feet below existing grade to protect against scour.

12.7 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 zone (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 dam 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. 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 crack 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.

13.0 EMERGENCY SPILLWAY DESIGN

13.1 CURRENT 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 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 on District property.

13.2 DESIGN REQUIREMENTS

The emergency spillway will be modified to address potential erosion issues that exist for the earth-cut spillway. A vertical cutoff wall will be installed to maintain the spillway crest elevation at 1,212 feet (NAVD 88). The notch through the spillway excavated in 2001 will be filled in to match the design crest elevation. Erosion protection measures will be installed in the vicinity of the wall and downstream to protect the cutoff wall structural integrity. Erosion protection will also be placed on the dam to prevent erosion during a spillway flow event.

SITES ANALYSIS

The Bethany Home Road Dike will be relocated onto District property and extended to a point downstream that protects existing properties. The dike height will be set 1 foot above the flow depth resulting from the spillway design flood. The dike alignment will be parallel to the property line. Erosion protection will not be placed on the dike.

13.3 CONSTRAINTS

The dam and Bethany Home Road Dike will be covered with aesthetic fill. The placement of aesthetic fill in the area of the spillway will be limited to maintain a width of 800 feet at the spillway, and upstream and downstream of the spillway.

13.4 DESIGN ALTERNATIVES

Design alternatives will be considered for the 60 percent design submittal. Alternatives will include:

- Increasing the spillway width; and
- Modifying the spillway to function as a sharp crested weir. This would involve excavating soils from the spillway and exposing the cutoff wall.

These alternatives will be evaluated for the effect of lowering the IDF flood pool elevation and potentially reducing construction costs.

13.5 DESIGN CONFIGURATION

The design will consist of a buried cutoff wall at the existing spillway crest location and have a crest elevation of 1,212 feet (NAVD). Grouted rip rap will be placed downstream for approximately 40 feet to protect against erosion. Secondary cutoff walls will be constructed at selected intervals downstream of the main cutoff wall. The secondary wall dimensions and spacing will be determined through the erosion analysis performed using the SITES model.

Rip rap will be installed on the dam to protect against erosion during spillway flow events. The rip rap will be placed 3 feet below grade for scour protection. The height of the rip rap will be determined based on flow depths for the IDF. Aesthetic fill will be placed over the dam slope and rip rap.

The Bethany Home Road Dike is aligned parallel to the south property line and located to provide room for an access road on the north and south side of the dike. The upstream end of the dike is adjacent to the emergency spillway and continues to a point designated by the District. The top of the dike is approximately 6.5 feet above existing grade, which corresponds to the water surface of the spillway design flood. Erosion protection is not included in the dike design. Aesthetic fill will be placed over the dike side slopes.

should be

14.0 PRINCIPAL OUTLETS

14.1 CURRENT 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 under 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 DESIGN REQUIREMENTS

14.2.1 Existing Outlets

The existing outlets will be decommissioned and left in place or removed. The North and South Outlets will be removed because they are located within the non-fissure risk zone and would be within the raised embankment. The Central Outlet will be left in place because it is within the fissure risk zone and will be located downstream of the new embankment.

14.2.2 New Outlet

A new outlet will be installed through the embankment to replace the three existing outlets. The new outlet will be sized to drain the reservoir from the emergency spillway crest to the outlet invert in 10 days. The outlet will be installed within the soil cement embankment to eliminate the need for a diaphragm filter. A gate and trash rack will be installed on the upstream end of the outlet.

14.3 CONSTRAINTS

Design constraints guiding the design of the new outlet consist of the following:

- The outlet must be capable of draining the reservoir within 10 days with the water level starting at the emergency spillway crest.
- The upstream invert of the pipe will be located at the lowest possible elevation to allow draining of the greatest volume in the reservoir.
- The outlet will attempt to avoid directing downstream flows into the fissure risk zone.
- The outlet will be constructed through the soil cement embankment.

14.4 DESIGN CONFIGURATION

14.4.1 Closure of Existing Outlets

[To be completed at 60%]

14.4.2 New Outlet

The new outlet will be located near the left end of the soil cement embankment. A zone of structural fill will be placed upstream of the soil cement section to provide a 2.5:1 slope on the upstream face. In addition, structural fill will be placed along the length of the pipe downstream of the soil cement section. The pipe will be installed with a slope of 1 percent towards the downstream end. The upstream invert is at an elevation of 1,195 ft.

The outlet will be constructed of welded steel pipe supported on a concrete base. The pipe will be completely encased in concrete through the soil cement embankment. Anchors will be used during placement of the concrete to prevent floating of the pipe. Construction of the concrete components is detailed on the drawings.

A concrete pad will be constructed at the upstream end of the outlet to provide a base for the gate and trash rack. Supports for the gate mechanism will be installed on the embankment slope. Attempts will be made to reuse an existing gate, mechanism, and trash rack. The downstream end of the pipe will protrude from the embankment a sufficient distance to prevent erosion back into the embankment during operation. Erosion protection will be installed at the end of the outlet. Details of the new outlet design are provided on the design drawings.

15.0 CONSTRUCTION PHASING AND COST ESTIMATE

15.1 PHASING

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 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. The design drawings and construction cost estimate have been separated into 3 phases to allow the District to solicit construction bids on each phase of work separately.

15.1.1 Phase 1

Construction activities performed in Phase 1 will include:

- The soil cement component of the dam.
- The earthen embankment component south of the soil cement component to the right abutment.
- The earthen embankment component for approximately __ feet north of the soil cement component.
- The principal outlet and closure or removal of the existing outlets.
- The emergency spillway.
- The Bethany Home Road Dike.

15.1.2 Phase 2

Construction activities performed in Phase 2 will include:

- The earthen embankment component from the end of Phase 1 north to the right abutment. This will include the new earthen embankment constructed north of the existing dam.
- Aesthetic fill downstream of the dam crest and on the Bethany Home Road Dike.

15.1.3 Phase 3

Construction activities performed in Phase 3 will include:

- Aesthetic fill upstream of the dam crest.
- Completion and aesthetic treatment of the borrow sources.

15.2 CONSTRUCTION COST ESTIMATE

A separate construction cost estimate will be developed for each phase to provide the District with a basis for evaluating construction bids. **[To be completed for the 60 percent submittal]**

16.0 REFERENCES

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

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.

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

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

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.

TABLES

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

Storm	Peak Inflows (NRCS Report, August 1998)[1] (cfs)	Peak Inflows (FCDMC Provided Output Files)[2] (cfs)	Peak Inflows (URS Execution of Input Files)[3] (cfs)
6-Hour Local PMP	66,122	66,122	66,122
6-Hour General PMP	34,212	34,212	34,216
12-Hour General PMP	32,435	32,435	32,278
18-Hour General PMP	26,905	26,905	26,905
24-Hour General PMP	23,800	23,800	23,800
48-Hour General PMP	31,819	31,819	31,696
72-Hour General PMP	32,300	32,300	32,296
100-Year, 24-hour	10,835	10,835	10,468
Emergency Spillway Hydrograph (ESH)	21,685	21,685	21,674
Principal Spillway (100-year 10-Day)	3,290	3,290	3,290

Notes

1. These peak inflows 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 obtained by opening up the TR-20 output files provided by FCDMC to URS.
3. These peak inflows are based on the output files generated by URS by executing the input files provided by FCDMC.

TABLE 8-2
Urban Growth Projections and Curve Number Estimation

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

TABLE 9-1
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 9-2
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 (cfs)	Estimated Infiltration Rate (acre-feet/hour)	Comments
1178	0.00						
1179	0.51	0.25	0.25	0.25	0.00	0.00	
1180	1.05	0.78	0.78	1.03	0.00	0.00	
1181	1.75	1.40	1.40	2.43	0.00	0.00	
1182	3.07	2.41	2.41	4.84	0.00	0.00	
1183	4.79	3.93	3.93	8.77	0.00	0.00	
1184	5.81	5.30	5.30	14.07	0.00	0.00	
1185	6.60	6.21	6.21	20.28	0.00	0.00	
1186	7.44	7.02	7.02	27.30	0.00	0.00	
1187	8.65	8.05	8.05	35.34	0.00	0.00	Principal Outlet Invert (Gated)
1188	10.32	9.49	9.49	44.83	0.021	0.002	
1189	11.93	11.13	11.13	55.95	0.024	0.002	
1190	13.93	12.93	12.93	68.88	0.028	0.002	
1191	15.98	14.96	14.96	83.84	0.032	0.003	
1192	22.46	19.22	19.22	103.06	0.045	0.004	
1193	27.82	25.14	25.14	128.20	0.056	0.005	
1194	33.83	30.83	30.83	159.02	0.068	0.006	
1195	44.05	38.94	38.94	197.96	0.089	0.007	
1196	56.65	50.35	50.35	248.31	0.114	0.009	
1196.84	**67.18**	**61.92**	**52.01**	**300.32**	**0.135**	**0.011**	50-year Sediment Pool Level
1197	69.49	63.07	10.09	258.40	0.140	0.012	
1198	83.88	76.69	76.69	335.09	0.169	0.014	
1199	98.77	91.33	91.33	426.41	0.199	0.016	
1200	112.95	105.86	105.86	532.27	0.228	0.019	
1200.13	**115.02**	**113.99**	**14.82**	**547.09**	**0.232**	**0.019**	100-year Sediment Pool Level
1201	130.05	121.50	105.70	637.98	5.372	0.444	
1202	147.55	138.80	138.80	776.78	11.445	0.946	
1203	165.40	156.48	156.48	933.25	17.597	1.454	
1204	183.11	174.25	174.25	1107.50	23.706	1.959	
1205	199.09	191.10	191.10	1298.60	29.468	2.435	

TABLE 9-2 (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 (cfs)	Estimated Infiltration Rate (acre-feet/hour)	Comments
1206	216.66	207.88	207.88	1506.48	35.332	2.920	
1207	234.81	225.74	225.74	1732.21	41.209	3.406	
1208	253.15	243.98	243.98	1976.19	47.045	3.888	
1209	274.77	263.96	263.96	2240.15	53.460	4.418	
1210	294.10	284.44	284.44	2524.59	59.402	4.909	
1211	313.41	303.76	303.76	2828.34	65.280	5.395	
1212	**335.63**	**324.52**	**324.52**	**3152.86**	**71.730**	**5.928**	Emergency Spillway Crest Elevation
1213	352.37	344.00	344.00	3496.86	76.954	6.360	
1214	374.59	363.48	363.48	3860.34	83.324	6.886	
1215	395.31	384.95	384.95	4245.29	89.334	7.383	
1216	415.92	405.62	405.62	4650.91	95.285	7.875	
1217	437.26	426.59	426.59	5077.50	101.373	8.378	
1218	460.05	448.66	448.66	5526.15	107.775	8.907	
1219	481.05	470.55	470.55	5996.70	113.736	9.400	
1220	505.14	493.10	493.10	6489.80	120.409	9.951	
1221	527.54	516.34	516.34	7006.14	126.665	10.468	
1222	551.69	539.62	539.62	7545.75	133.326	11.019	
1223	576.26	563.98	563.98	8109.73	140.078	11.577	
1224	601.61	588.94	588.94	8698.66	147.011	12.150	
1225	628.92	615.27	615.27	9313.93	154.420	12.762	
1226	656.88	642.90	642.90	9956.83	161.984	13.387	

TABLE 9-3
Peak Discharges and Maximum Reservoir Pool Elevations

Storm Event	Inflow to White Tanks FRS No. 3			Outflow From White Tanks FRS No. 3			
				NRCS Criteria		ADWR Criteria	
	Precipitation (inches)	Peak Inflow (cfs)	Maximum Reservoir Storage (acre-feet)	Peak Outflow (cfs)	Maximum Reservoir Elevation (NAVD 88) (feet)	Peak Outflow (cfs)	Maximum Reservoir Elevation (NAVD 88) (feet)
6-hr General PMP	8.80	35,610	7,084	13,170	1,217.0	13,946	1,217.2
12-hr General PMP	11.00	33,225	9,340	15,496	1,217.5	16,300	1,217.6
18-hr General PMP	12.20	27,370	10,525	18,843	1,218.1	19,620	1,218.2
24-hr General PMP	12.90	24,210	11,435	18,431	1,218.0	18,970	1,218.1
48-hr General PMP	15.00	32,200	13,620	18,147	1,218.0	19,120	1,218.1
72-hr General PMP	15.80	32,700	14,440	19,914	1,218.3	20,920	1,218.4
6-hr Local PMP	12.70	68,170	9,358	20,287	1,218.3	22,138	1,218.6
ESH	5.29	23,420	3,704	4,075	1,214.8		
100-year 24-hour	3.85	11,655	2,335	65	1,210.9		
200-year 24-hour	4.28	13,917	2,732	121	1,212.1		
500-year 24-hour	4.84	17,046	3,268	633	1,212.8		
100-year 10-Day	6.40[3]	3,290	1,614	48	1,207.4		
Back-to-back 100-year 10-day storms	12.8[3]	3,290	1614	60	1,210.2		

Notes:

1. The antecedent reservoir condition (ARC) for routing of the PMF and Emergency Spillway Hydrograph was different for NRCS and ADWR models. The ARC for the NRCS routing models was based on 10 days of infiltration drawdown following the 100-year, 10-day storm. The ARC for the ADWR routing models was based on 10 days of infiltration drawdown following the 100-year, 24-hour storm.
2. The ARC for routing of the 24-hour and 10-day storms was the top of the 100-year sediment pool.
3. 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 1.48 inches is input to the model to reflect the total anticipated runoff.

TABLE 9-4
Results of 24-Hour Storm Routing

Storm Event	Antecedent Reservoir Condition	Maximum Reservoir Storage (feet)	Maximum Reservoir Elevation (NAVD 88) (feet)	Flow Through Spillway
100-year 24-hour	Outlet Invert[1]	To Be Determined in 60% Design		
	100-Year Sediment Pool[2]	11,655	1210.9[3]	No
200-year 24-hour	Outlet Invert	To Be Determined in 60% Design		
	100-Year Sediment Pool	13,917	1,212.1	Yes
500-year 24-hour	Outlet Invert	To Be Determined in 60% Design		
		17,046	1,212.8	Yes

Notes:

1. Outlet invert elevation is 1187.5 feet (NAVD 88).
2. 100-year sediment pool level is 1200.1 feet (NAVD 88).
3. Emergency spillway crest is set at 1212.0 feet (NAVD 88).

TABLE 12-1

Conversion from Old to New Dam Stationing

[To be completed for 60 Percent design submittal]

TABLE 14-1
Existing Outlets

Outlet	Station Location	Diameter (inches)
North		48
Central		48
South		24

FIGURES

[Figures to be included with 60 Percent Submittal]

APPENDICES

[Appendices to be included with 60 Percent Submittal]