
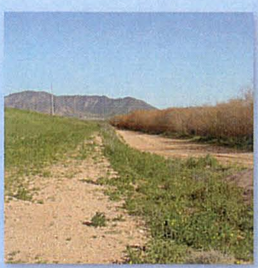
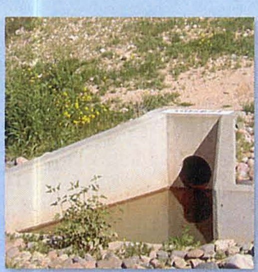
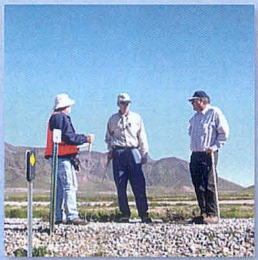

Saddleback Flood Retarding Structure

Individual Structure Assessment Report

Prepared for
Flood Control District of Maricopa County
On-Call Phase I Assessment
FCD 2003C015
PCN 050.03.01
October 2005

Kimley-Horn and Associates, Inc.

SADDLEBACK FLOOD RETARDING STRUCTURE INDIVIDUAL STRUCTURE ASSESSMENT REPORT

FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
FCD 2003C015

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Kimley-Horn
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INDIVIDUAL STRUCTURE ASSESSMENT REPORT
for
SADDLEBACK FLOOD RETARDING STRUCTURE

Table of Contents

Section No. **Page No.**

Section No. | **Page No.**

List of Tables (Appendix)

List of Figures (Appendix)

List of Appendices

1.0 EXECUTIVE SUMMARY

This Individual Structures Assessment (ISA) Report documents the results of a technical evaluation and field examination for one of the twenty-two Flood Control District of Maricopa County (District) flood control dams. The dam investigated as part of this project was **Saddleback Flood Retarding Structure**. The ISA Report is part of Phase I of the Structures Assessment Program. The technical evaluation of the dam consisted of engineering, geological and geotechnical reviews of structure historical reports and documents. The types of documents reviewed included original and subsequent design and analyses such as hydrology and hydraulic studies of the dam, foundation reports, boring logs, seismic studies, subsidence and earth fissure evaluations, construction plans (design and as-builts) and construction specifications, and any documents pertaining to repairs, modifications, or upgrades to the structure. Detailed visual field examinations were conducted for the structure and associated features. The purpose of the field examinations was to assist in the systematic technical evaluation of the structure and operational adequacy of the dam project features and to determine if signs of distress exist at the dam and appurtenant features. A Failure Modes and Effects analysis was conducted for Saddleback FRS. The FMEA qualitatively identified and evaluated potential failure modes and consequences of dam failure. The ISA report provides recommendations for the structure regarding work plans and actions for future engineering studies.

1.1 Dam Description

Saddleback FRS is located between Courthouse Road (approximately the McDowell Road Alignment) and Interstate 10 in Sections 17, 21, 27, 28, and 34 of Township 2 North, Range 8 West. The project consists of the Saddleback FRS embankment, a principal spillway, and four irrigation outlets. The principal spillway was designed to function as the auxiliary (emergency) spillway.

The Saddleback FRS reservoir has a capacity of 3,620 acre-feet. A permanent pool is not retained in the reservoir. The Saddleback FRS and reservoir are designed to detain the 100-year floodwater and store the impoundment for a slow release of approximately six days into the Saddleback Diversion channel. Reservoir capacity is then restored to detain future stormwater runoff events.

The Saddleback FRS is a 26,970-foot long (5.1 miles) homogeneous earthfill structure with 3:1 upstream slopes, 2:1 downstream slopes, and a crest width of 12 feet. The embankment also has a 5-foot wide vertical central filter. The structure ranges from 1 to 20.8 feet above the existing ground. The volume of fill used for the structure is 690,000 cubic yards. The cutoff trench is 12 feet wide. The combined principal/emergency spillway is an 8-foot tall by 10-foot wide concrete box culvert. The irrigation outlets are 12-inch diameter steel pipes in concrete.

The vertical central filter drain was placed in the structure with a drain outlet approximately every 400 feet starting at Station 8+00. Each drain outlet has a cross-section that is 2 feet tall, with a top width of 6 feet, bottom width of 2 feet, and a 6-inch diameter perforated asbestos cement pipe to facilitate internal drainage to the toe of the embankment. The central filter drain was designed to be inclined, but was constructed vertically. The reason for this change could not be located in the documentation.

The maximum recorded impoundment for Saddleback FRS is 102 acre-feet with a stage of 2.5 feet at the Saddleback FRS in July 1996.

Watershed

The dam was constructed across a number of local drainage washes conveying runoff from the Saddleback Mountains and the alluvial plain originating from the eastern slope of the Big Horn Mountains. An additional 55 square miles of the historic watershed was cut off from the rest of the watershed by the construction of the Granite Reef Aqueduct (GRA: also known at the Central Arizona Project or CAP) and was considered non-contributing area during the design of the FRS. The total contributing drainage area for the Saddleback is 22.3 square miles. The sediment storage requirement is 120 acre-feet, which is the estimated sediment that will be supplied by the watershed over a 50-year period.

Flood Pool

The total capacity of the reservoir is 3,620 acre-feet including a sediment capacity of 120 acre-feet, which is the estimated 50-year sediment yield. The FRS was designed to capture floodwater runoff from the 100-year storm and discharge it through the primary outlet over a period of approximately six days. The total surface area of the sediment pool is 61 acres and the total area of the floodwater pool is 760 acres.

The principal spillway inlet channel is a low flow channel that is at the upstream toe of the dam, which conveys low flow through the reservoir from Station 34+75 to the principal outlet at Station 15+83.08. The flood pool is divided into two separate basins. Basin No. 1 is the largest basin and near the principal spillway. Basin No. 2, upstream of Basin No. 1, is at the outlet of the Harquahala Floodway.

Principal/Emergency Outlet Works

The principal spillway was also designed to function as the emergency spillway. The principal/emergency spillway box culvert is located at Station 15+83.08 along the structure and is an 8-foot tall by 10-foot wide concrete box culvert that is 65 feet long. The principal spillway flows into the Saddleback Diversion Channel, which ultimately discharges into Centennial Wash. The Saddleback Diversion and Channel, which is a 25,000-foot long diversion structure and channel, were constructed in conjunction with the Saddleback FRS as part of the Watershed Work Plan.

There is a pressure transducer gage (ALERT gage) at the principal spillway location that was installed in 1988 as part the District's flood warning system. A staff gage is also located at the principal/emergency spillway.

Emergency Spillway

There is no separate emergency spillway associated with the Saddleback FRS. The Harquahala Valley Watershed Work Plan indicates that the principal spillway is dual purpose serving also as the emergency spillway. The combination spillway was designed to pass the freeboard hydrograph through the reservoir without overtopping the dam with no freeboard.

1.2 Hydrologic and Hydraulic Considerations

The "Watershed Work Plan–Harquahala Valley Watershed" was prepared by the NRCS in 1967 and updated in 1977. The final design report and final design report supplement were completed in 1980. The NRCS designed the Saddleback FRS to detain the 100-year stormwater runoff volume and route it though the combination principal and emergency spillway.

Principal Spillway

The principal spillway hydrograph (PSH) is the hydrograph used to determine the minimum crest elevation of the emergency spillway, to establish the principal spillway capacity, and to determine the associated minimum floodwater retarding storage capacity. The Saddleback FRS has a combination principal and emergency spillway that is an 8-foot wide by 10-foot tall concrete box culvert with an invert elevation set essentially at the bottom of the reservoir.

The watershed was split into four major subbasins and curve numbers and times of concentration were estimated for each subbasin. Curve numbers for the 6-hour storm ranged from 83 to 88, with an average of 86 for the entire watershed. Ultimately, in the development of the PSH using TR-20, the curve number reverted to an average of 83 because the total precipitation was less than six inches. This adjustment is based on the design standard from the National Engineering Handbook-4 (NEH-4) Table 21-2. Precipitation depths of 4.15 inches and 5.67 inches were translated to watershed runoff depths of 2.02 inches and 3.27 inches, respectively, based upon an areal correction and a channel loss factor in the routing.

NRCS design notes indicate that the reservoir was split into two major basins (Basin No. 1 and Basin No. 2) for the PSH. Basin No.1 is near the principal spillway and Basin No. 2 is near the outlet of the Harquahala Floodway. The principal spillway crest elevation is 1176.9 ft (N9VD29). The maximum reservoir outflow during the PSH is 1,100 cfs. The drawdown time of the reservoir is approximately six days.

There are two other sources of inflow in the Saddleback watershed in addition to the contributing watershed. A baseflow of 500 cfs from the Harquahala Floodway is discharged into Basin No. 2 in the TR-20 model. The maximum capacity of the floodway was estimated to be 500 cfs. As stated, the historic Saddleback FRS watershed is divided by the Central Arizona Project (CAP) canal, which intercepts nearly 55 square miles of watershed, of which a portion would have contributed directly to the Saddleback FRS watershed. The upstream embankment of the CAP is approximately 20 feet high and creates a basin that detains nearly 7,500 acre-feet of stormwater runoff. The runoff is controlled by five 72-inch diameter conduit overchutes that penetrate the CAP embankment. One of these overchutes discharges into the Saddleback FRS watershed.

Emergency Spillway

A combination principal/emergency spillway design was used in the routing of the Freeboard Hydrograph (FBH), Emergency Spillway Hydrograph (ESH), and Principal Spillway Hydrograph (PSH). The Saddleback FRS is designed to detain the entire FBH. The combination spillway was designed to pass the FBH with no freeboard. According to standard NRCS design, the FBH actually sets the minimum settled dam crest elevation. The FBH is also typically used to evaluate the structural integrity of the spillway system. The structure was classified by the NRCS as a Class A structure, but was designed to Class B standards. The FBH for a Class B Structure is a function of the PMP and 100-year precipitation.

Spillway Inundation Study. An emergency spillway delineation study was completed for the Saddleback FRS in May 1998 (Entellus, 1998). The study was completed for the District and was essentially a delineation of flows for the Saddleback Diversion Channel. The study extended from the Saddleback FRS combination spillway to the outfall of the Saddleback Diversion into Centennial Wash. The total length of the study was 6.5 miles.

The Saddleback Diversion Channel has a contributing watershed of 8.6 square miles and hence the anticipated discharge in the channel increases downstream. The discharges for the inundation study ranged from 1,120 cfs at the combination spillway to 6,060 cfs and its lower end.

Dambreak Analysis. A dambreak and flood routing analysis was completed for the Saddleback FRS by Carter Associates in February 1991 for the District (Carter and Associates, 1991). The analysis included a PMP hydrologic assessment using HEC-1, a dam breach analysis using National Weather Service (NWS) BREACH model, and a ½ PMF dambreak flood routing analysis using the NWS DAMBRK model. The BREACH and DAMBRK models have now been phased out of service by the NWS, combined, and replaced with the FLDWAV model.

The hydrologic phase of the dambreak study found that the Saddleback FRS would not be overtopped during the ½ PMF. Therefore the selected potential failure mechanism studied was from internal embankment erosion or piping. The study developed a PMF analysis for the 6-hour and 72-hour duration storm event on the Saddleback FRS watershed and then routed the ½ PMF flood through the reservoir to determine the maximum reservoir water surface elevation and to get the parameters for the dambreak analysis. The study indicated that the 72-hour storm was the most critical for the DAMBRK analysis since the 72-hour generated the largest runoff volume to the reservoir and produced the highest reservoir water surface elevation for the ½ PMF event. One approach would be to assume the combination principal/emergency spillway was plugged and the pool full to the top of the dam crest (with no inflow)

The dambreak breach parameters and resulting dambreak peak discharges appear to be more conservative than what would be expected. The breach widths and times of breach are not reasonable given the structural height of the dam. The breach occurs during the peak of the ½ PMF event and consequently is not a true 'sunny-day' failure. Usually 'sunny day' piping breaches are analyzed when the reservoir water surface elevation is at the emergency spillway crest elevation with no inflow. This should be analyzed for the Saddleback FRS since there not a traditional emergency spillway crest elevation. One approach would be to assume the combination principal/emergency spillway was plugged and the pool full to the top of the dam crest (with no inflow).

Kimley-Horn recommends that an updated dambreak analysis and inundation mapping be prepared for the Saddleback FRS. New integrated hydraulic models such as HEC-RAS (unsteady flow and dambreak options) could be used to prepare the updated study. The dambreak update should develop reasonable dambreach parameters using published guidelines and the District's dambreach model currently under development. The true "sunny day" failure defined as a full pool with no inflow should be considered as well as a dambreak for the ½ PMF and PMF events using ADWR guidelines for routing through a flood control dam.

1.3 Geologic and Geotechnical Considerations

Geologic Setting. The Saddleback FRS is located within the Sonoran Desert section of the Basin and Range physiographic province. This portion of the Basin and Range is characterized by north and northwest trending mountains that rise abruptly to form broad, elongated, deep, sediment-filled valleys produced by block faulting, tilting and folding. The structure lies in the east-central portion of the Harquahala Valley. The Harquahala valley is a northwest trending alluvial valley bounded on the north by the Harquahala Mountains, the northeast and east by the Big Horn and Saddle Mountains, the west by the Eagletail and Little Harquahala Mountains, and the south by the Gila Bend Mountains.

Seismic Evaluation. In 2002, a Seismic Exposure Evaluation was performed by AMEC Earth & Environmental, Inc. (AMEC, 2002) for the Dam Safety Program of the Flood Control District of Maricopa County. According to this report, the Saddleback FRS lies within the Southern Basin and Range Source Zone. A seismicity evaluation conducted for the Arizona Department of Transportation (Euge, 1992) describes this zone as the Sonoran Seismic Source Zone. This source zone appears to have a low level of seismicity and few active or potentially active faults. Within this source zone, the largest historical earthquake was a 1956 magnitude 5.0 event that occurred in the southern portion of the zone.

The closest active fault to the Saddleback FRS, Sand Tank Fault, is approximately 83.3 miles southeast of the structure. Sand Tank Fault lies in south-central Maricopa County, east of the town of Gila Bend. Sand Tank Fault is a normal fault with a slip rate of less than 0.02 millimeters per year and a recurrence interval of approximately 100,000 years. This fault may be capable of producing a maximum credible earthquake of magnitude 5.7 and an associated maximum peak horizontal acceleration at the Saddleback FRS equal to 4 percent of the gravitational acceleration (g). The recommended peak horizontal acceleration design criteria calculated by AMEC for the Saddleback FRS is 0.10 g.

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

Groundwater in the Harquahala Groundwater Basin

The Saddleback FRS is located in the Harquahala groundwater basin in west-central Arizona. The lithology of the basin varies widely, but is generally composed of a heterogeneous mixture of clay, silt, sand and gravel. The alluvium may range from 0 feet deep at the base of the mountains to more than 5000 deep in the center of the basin. The alluvial deposits grade from coarse-grained sand and gravel in the southeast to fine-grained deposits in the center of the basin. Fine-grained clay deposits, over 1000 feet thick, occur in the western part of Township 2 North, Range 9 West. The fine-grained beds grade toward the west into an alternating sequence of fine-grained and coarse-grained layers from 800 to 850 feet thick, overlying a conglomerate unit.

The main use of groundwater in the Harquahala basin is for agricultural purposes. Prior to 1951, groundwater in the basin flowed from the northwest to southeast. By 1963, three cones of depression had developed in the southeastern part of the basin which, by 1966, had coalesced into one large cone in the center of the valley. By 1986, the basin had experienced a decline in the groundwater level in some areas of as much as 300 to 500 feet.

Study Area Subsidence

Historic National Geodetic Survey (NGS) level line data is not available in the vicinity of the Saddleback FRS. However, recent historic subsidence-settlement is available from the Flood Control District of Maricopa County using crest and toe monument elevations recorded between 1984 and 2003.

According to available data, it appears that negligible settlement or subsidence has occurred across most of the dam and a very minimal amount is recorded on one crest section between A-5 and A-10, A-21 and A-23 and the toe at B-25, from 1983 to 2003. The change in elevation in this area ranges from -0.001 to -0.300 feet. Overall the entire Saddleback FRS appears to be relatively stable in terms of settlement and subsidence. This is not surprising when considering the structure is located within the volcanic bedrock pediment area that is overlain by well cemented fanglomerate.

The minimal subsidence recorded from 1983 to 2003, along with the static or increasing groundwater levels in the area would suggest that future land subsidence in the vicinity of the Saddleback FRS would be minimal. This is subject to change if increased pumping of the groundwater caused the water level to decline thereby increasing land subsidence.

Known Earth Fissures in the Project Vicinity

There have been three earth fissures reported in the Harquahala Valley. The closest fissure to the Saddleback FRS lies approximately 2.5 miles west of the structure in Section 36, Township 2 North, Range 9 West. There is no current information on the status of this fissure. An examination of recent aerial photographs of the area did not display any feature that would be indicative of the fissure. This is probably due to the fact that the reported fissure is located in an agricultural area and any surface expression of an earth fissure would be destroyed during agricultural activity.

Another fissure lies approximately 4.7 miles northwest of the structure in Section 9, Township 2 North, Range 9 West. This fissure was first discovered in 1958, visible in an aerial photo. The fissure was examined in 1978 and appeared to have been dormant for many years.

The Rogers fissure was discovered in 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 11 miles west of the dam, when it made an abrupt appearance during an unusually heavy rainfall event. The fissure is approximately 4,400 feet long, averages 5 to 15 feet deep and 5 to 10 feet wide, with prominent near vertical side slopes. Development of the surface expression of the Rogers fissure was unusual in that there were no reported precursor features, such as small surface cracks, aligned potholes, linear depressions or linear vegetation, in the area that would have indicated the fissure was present.

In 2001, another earth fissure appeared suddenly, following a heavy rain. This fissure appeared in the West Salt River Valley, west of the Palo Verde Generating Station. This fissure is about 14.4 miles southeast of the Saddleback FRS.

Foundation Conditions

Surface soils along the centerline of the dam alignment were described in the Report of Geologic Investigation (SCS, 1978) as consisting of very loose to loose silty or clayey sands and sandy silts and clays with a variable amount of gravel. These loose materials generally extend to depths of between 3 and 5 feet, and up to 9 feet in some locations. These loose materials are underlain by relatively firm and incompressible caliche of cemented sandy gravels, clayey, silty and/or gravelly sands, and sandy silts and clays. Relatively high permeability channel fill deposits of gravelly sands and sandy gravels were reportedly interbedded with, or incised into, the lower permeability caliche at several locations along the dam alignment. There was reportedly no surface expression of these channel fill deposits.

A bedrock high was reported by SCS in the vicinity of Station 115+00. According to SCS, the loose surface soils were observed to a depth of approximately 6 feet. Cemented fangolmerate overlying volcanic bedrock was present beneath the surface soils. SCS concluded that the area of the bedrock high was relatively localized because the depth to the fanglomerate reportedly increased to approximately 12 to 13 feet at Stations 112+78 and 117+14, respectively. The as-built plan set also shows this to be a relatively localized feature. According to the as-built plan set, the bedrock high was encountered near Station 110+00. The reason for the discrepancy between the original Report of Geologic Investigation Report and the construction records is unknown.

Embankment

The Saddleback FRS was designed as a homogeneous (3H:1V upstream (Zone I) and 2H:1V downstream (Zone III) with an inclined, 5-foot wide filter/drain (Zone III). The as-built construction documentation indicates that the filter/drain was constructed as a centrally-located vertical filter/drain. The reason for the installation of a vertical filter/drain rather than the inclined filter/drain is unknown. The vertical filter/drain extends 1 foot into the cutoff over most of the alignment. However, it extends to the limit of foundation excavation at three locations (Station 46+00 to Station 76+00, Station 104+00 to Station 114+00, and Station 238+20 to Station 265+70). The filter/drain was designed to terminate 3 feet below the crest of the embankment; however as-built drawings of the filter/drain rehabilitation performed in 1995 indicate that the depth to the top of the filter/drain is approximately 3.5 ft to 5.5 ft. The filter/drain is connected to a system of 6-inch diameter perforated outlet pipes surrounded by drain rock installed at 400-ft intervals beginning at Station 8+00.

According to the as-built plan set, the bottom of the foundation cutoff trench was grouted using dental grout between Stations 108+00 and 110+00, in the vicinity of the bedrock high described in the previous section. The grouting was presumably performed to fill cracks and surface irregularities and to provide a uniform base for construction of the cutoff trench.

A cut-off trench was installed along the entire dam to depths of up to 11 feet. Protective berms were constructed along two upstream sections at locations that coincide with two of the three filter/drain extension locations (Station 46+00 to Station 76+00 and from Station 240+00 to Station 265+00). It is assumed that the berms were incorporated into the design to improve stability during rapid drawdown loading conditions.

Filter Compatibility

Zone II is shown on the as-built drawings as a 5-ft wide, vertical chimney filter/drain with the centerline of the filter/drain coincident with the centerline of the dam crest. The most important function for the Zone II materials is to serve as a filter to protect against potential internal erosion and piping of the Zone I materials in the event of transverse crack development.

Because of its critical function as a filter, the Zone II gradation was checked against current filter criteria in accordance with the NRCS, National Engineering Handbook, Chapter 26 "Gradation Design of Sand and Gravel Filters". Gradation curves for Zone I "Base Soil" materials were developed for embankment soil samples collected during a geotechnical investigation of Saddleback FRS in 2001. The samples for which gradation curves were developed (designated B-1 @ 4.5-6.0', B-3 @ 2.0-3.0', and #10 @ 4.5-5.5') were collected in an area where cracking has been documented and rehabilitation work has been performed making this location critical with respect to material compatibility. The filter/drain is compatible with the Zone I materials.

Dam Rehabilitation

In 1995, rehabilitation of the dam was conducted between Stations 44+00 and 52+00 to remediate longitudinal cracking in the dam crest, above the vertical filter/drain. During rehabilitation work, ADWR noted that longitudinal cracks along the centerline of the dam crest were between ½-inch and ¾-inch wide and extend entirely through the soil cover over the filter/drain and at some locations extend up to 12 inches into the underlying filter/drain. It was also noted that the upper 6 to 12 inches of filter/drain material was contaminated with fine-grained embankment materials.

The rehabilitation consisted of excavating a 2 ft wide trench along the crest of the dam between Stations 44+00 and 52+00 and backfilling and compacting the excavation with filter/drain material up to the crest. The excavation and manual removal of contaminated drainfill materials continued 1 foot into the uncontaminated material.

The gradation of the drainfill material placed during the rehabilitation was designed to be identical to the originally specified filter/drain material. It was reported by ADWR reports that laboratory testing confirmed that at least two of the three soil samples tested during the rehabilitation met the design specification for gradation however, the laboratory results were not available for review during this Phase I Structures Assessment.

1.4 Land Use

Existing land uses in the study area generally are characterized as active open space, agriculture, residential, commercial, or as public facilities. This information is summarized as follow:

- Interstate 10 is a major road through the project area and contains a large portion of land designated as open space and residential. This road is located approximately 0.5 miles upstream of Saddleback FRS and runs perpendicular to the dam.
- Major agriculture and irrigation canals are located south of Interstate 10.
- There is a power generation station (Allen Generating Station) located at 491st Avenue and Thomas Road.
- No new residential development was recorded near this dam at this time. However, new single family lots are being developed.

The major significant change under future land use is that the agriculture and vacant lands changes to single family residential. The residential land use change is shown to completely encompass Saddleback FRS.

1.5 Field Inspection

Based on the field inspection performed by the Kimley-Horn team, previous inspection reports by ADWR and the District and the results of FMEA for the FRS, major signs of distress in the form of confirmed transverse and longitudinal cracking have been identified at Saddleback FRS. Erosion holes on the dam crest are located over the central filter for approximately the full length of the dam.

Based on the field inspection performed by the Kimley-Horn team, previous inspection reports by ADWR and the District and the results of FMEA for the FRS, no major safety deficiencies have been identified for Saddleback FRS. An EAP for Saddleback FRS needs to be prepared and developed to meet the minimum guidelines from ADWR and FEMA

1.6 Failure Modes and Effects Analysis

Kimley-Horn conducted a FMEA for Saddleback FRS as part of the Phase I Assessment. The objective of the FMEA was to qualitatively assess the identified risks associated with potential failure modes to Saddleback FRS.

The FMEA developed only one Category I and four Category II potential failure modes. These are:

- Overtopping During Major Flood Event (Category I).
- Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks under the Filter (Category II).
- Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks that extend above the Filter (Category II).
- Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks that extend through the Filter (Category II).
- Internal Erosion Leading to a Breach in the Upper Embankment due to the Eroded Character of the Crest (Category II).

The qualitative risk of the overtopping failure mode ranges from low likelihood, medium consequences (for the PMF event) to high likelihood, lower consequences. The range of risk for this failure mode is dependent on the storm frequency, magnitude, and downstream consequences. The potential failure mode of internal erosion leading to a breach in the upper embankment has a risk range of medium likelihood and low to medium consequence. None of the potential failure modes have a high likelihood, high consequence.

1.7 Recommendations

The following additional studies and investigations are recommended based on updating existing studies, results of the FMEA, and other issues during the Phase I Assessment:

A. Hydrologic and Hydraulic Recommendations

(1) Kimley-Horn recommends that an updated dambreak analysis and inundation mapping be prepared for the Saddleback FRS. New integrated hydraulic models such as HEC-RAS (unsteady flow and dambreak options) could be used to prepare the updated study. The dambreak update should develop reasonable dambreach parameters using published guidelines and the District's dambreach model currently under development. The true "sunny day" failure defined as a full pool with no inflow should be considered as well as a dambreak for the ½ PMF and PMF events using ADWR guidelines for routing through a flood control dam.

(2) Evaluate effects of CAP canal and CAP upstream embankment on flows contributing to Saddleback FRS. Confirm profile of upstream embankment to check if the extra recommended height of dam was constructed as stated in project documents.

(3) A quantitative risk assessment for the facility will require development of stage-frequency and emergency spillway discharge frequency relationships.

(4) Probable Maximum Precipitation. Prepare PMP/PMF using 24-hr and 72-hour durations for the Saddleback FRS watershed. Compare routings of these events to PMP 6-hr duration flood to verify that they are less critical (or determine that they are more critical). Evaluate impact of the CAP canal on the PMF routings at the dam.

(5) Evaluate the need for a trash rack on the combined principal/emergency spillway.

(6) Input updated stage/storage/discharge rating curves into models. Evaluate impacts on routing IDF.

(7) Dynamic routing is recommended due to length of dam and geometry. Conduct dynamic routing for 100-year, ½ PMF, and PMF.

(8) Update the sediment yield analysis for the watershed. Typical sediment yield studies in Maricopa County have provided yield rates on the order of 0.2 to 0.3 acre-feet per square mile per year. The Watershed Workplan sediment yield for Saddleback FRS is 0.11 acre-feet per square mile per year.

B. Geotechnical and Geological Recommendations

Phase II Additional Evaluation of Zone II Drain Materials

The compatibility of the embankment materials and the ability of Zone II to adequately act as a filter for Zone I was evaluated for this Phase I Structures Assessment and found to be compatible. No gradation data for the as-built filter materials were available for review, therefore the assessment of filter compatibility was based only on the specified filter gradation with no confirmation that the original filter was built within the specified gradation range. According to ADWR (1995), three filter gradation samples were collected during the dam rehabilitation work and were within specifications, however these test results were not available for review. It is recommended that filter gradation data be obtained and the compatibility with the filter and embankment materials by confirmed.

Phase II Documentation of Slope Stability and Seepage Analyses

Under reasonable loading conditions for Saddleback FRS, it is expected that both upstream and downstream slopes will be stable. However, adequate documentation of slope stability factors of safety for specified loading and design criteria established by appropriate jurisdictional agencies is not available. Additional slope stability analyses are recommended to document the slope stability factors of safety for Saddleback FRS.

(1) **End of construction (upstream and downstream slope)**: The original factor of safety calculated for the downstream slope under end of construction loading conditions achieved the minimum ADWR criteria of 1.3. Evaluation of the downstream slope stability under these loading conditions was not performed.

(2) **Rapid drawdown (upstream slope)**: The original stability analysis for this loading condition resulted in calculated factors of safety that are currently acceptable under ADWR rules. Additional analyses are not required.

(3) **Steady state seepage without seismic forces**: The original factor of safety calculated for this loading condition in Reach 2 (1.23) did not achieve the minimum criteria of 1.5. Additional analyses, including confirming the shear strength of embankment soils, either by review of additional data not available during this Phase I Structures Assessment or by field sampling, and reevaluating the critical failure surface on the downstream slope are recommended to document the stability of the downstream slope.

(4) **Steady state seepage, partial pool elevation (upstream slope)**: The original analysis did not evaluate upstream slope stability under this loading condition. The ADWR criteria for partial pool conditions are intended for water retention dams, in which a steady state phreatic line may develop for intermediate pool elevations. The factor of safety may be lower for the intermediate pool conditions than the steady state condition under maximum pool. The following analysis could be done to document the minimum partial pool factor of safety,

under the scenario that the outlet works are clogged such that the steady state phreatic line develops:

a. Perform seepage analyses under various partial pool elevations to establish the steady state pore pressure distributions within the dam at each pool elevation.
b. Conduct slope stability analyses for each partial pool seepage analysis result, and graph the results as factor of safety versus pool elevation.
c. Report the minimum factor of safety and corresponding pool elevation.

(5) **Steady state seepage with seismic forces (downstream slope)**: No seismic stability analysis was documented for Saddleback FRS. To document seismic stability under current design criteria, a pseudo-static stability analysis is recommended. The analysis should use a peak ground acceleration (PGA) of 0.1g and the ADWR recommendation of a pseudo-static coefficient equal to 60% of the PGA.

C. Erosion Holes Along Crest Centerline

Recommendations for Further Investigations and Monitoring

The Kimley-Horn team, as part of this Phase I assessment, is recommending the following actions to ascertain the nature of the erosion hole formation, monitoring of the erosion holes, additional geotechnical investigations for filter compatibility, filter material testing, and a potential repair alternatives to be evaluated further under a Phase II investigation.

1. An erosion hole monitoring program should be developed to note the locations and sizes of the holes along the crest. The program should map the locations on a set of as-built plans, obtain GPS coordinates of the erosion holes, and download the locations into the District GIS system. In this manner, the District may monitor, over time, hole locations and sizes, as well as the sections of the dam where erosion hole formation is most prevalent.
2. A field investigation program should be developed to investigate the compatibility of the filter with the embankment soil, the in-place density of the filter, and the current moisture and stress state of the Holocene soils underlying both the upstream and downstream zones of the dam. This would include:

    a. Drilling, sampling, and in-place density testing (standard penetration tests, or SPTs) of the foundation soils at the upstream and downstream toes of the dam; and drilling, sampling and SPTs of the embankment, filter zone and foundation soils from the crest of the dam,

    b. Test trenching and bulk sampling along the crest; samples should be collected from both the filter and the embankment zones,

    c. Mapping of cracks (transverse and longitudinal) in the test trenches (similar to what was done at Buckeye),

d. Laboratory testing for grain size characteristics, Atterberg limits, consolidation-collapse potential, and in-place moisture content.

3. Conduct high-resolution ground penetrating radar (GPR) surveys across the embankment crest to locate anomalies that may represent erosion hole, cracks or other voids. Excavate selected anomalies to determine what they represent and their possible relationship to erosion holes. Representative test GPR surveys could be run across areas of known erosion holes or voids to define their radar signature. This will assist with the characterization of other anomalies that are not coincident with surface expressions of erosion holes or other voids. The information gathered during the GPR surveys can be integrated into the data base generated in Item 1, above.

4. Using the data gathered during the previous site investigations and dam safety inspections, quantitatively and qualitatively identify the sections of the dam to define zones of high, moderate, low, and no concentrations of erosion holes. With this information, develop a phased mitigation program. The phasing of the mitigation program could be a function of available funding starting with the zone of high concentration.

5. Potential Interim and Permanent Repairs for further evaluation as part of a Phase II investigation may include:

   a. Interim measure: Filling the erosion holes with sand and allow sand to migrate into filter. Continue to apply sand until further sand will not be accepted. Cover sand at crest with compacted borrow materials from pool area.

   b. Potential permanent repair: remove the embankment from the crest to one foot below the elevation of the centerline filter. Where the filter is not in-place, remove the embankment to one-foot below the design flood maximum elevation. Rebuild the affected embankment sections and drain to the design crest elevation with engineered compacted fill.

   c. Potential permanent repair: Excavate by trenching entire centerline filter. Install geofabric in trench (to encase drainfill) and backfill trench to 1 foot below crest elevation with clean new drainfill material. Cap trench with compacted borrow materials from pool area.

D. Additional Recommendations from Inspection Report

(1) Provide Additional Means for Flood Warning. Add more gauges in contributing watershed, outside watershed, and stream gauges. Consider use of Doppler radar and satellite imaging.

(2) Conduct a Phase II study to evaluate converting Saddleback FRS and floodway into a true FEMA certified levee system. This scenario would remove the structure from state jurisdictional oversight. The potential of converting this dam and floodway to a levee system is promising.

(3) Develop an Emergency Action Plan to meet FEMA 64 and ADWR requirements;

(4) Develop a repair plan for erosion holes on the crest of the dam;

(5) Evaluate gravel mulching of embankment slopes;

(6) Evaluate effect of Solome Road over crest of dam. It is recommended that the District consider stabilizing Solome Road using concrete asphalt paving for the entire roadway surface over the dam and ¼ mile upstream and downstream from the dam. Paving could be applied to the dam crest at the intersection of the road for approximately 100 feet in either direction.

(7) Locate the toe drains at Station 204+00 and 248+00;

(8) Determine source of material deposited in the toe drain outlets.

(9) Due to the length of the structure (5 miles), an additional staff gage should be added. A recommended location would be at the Solome Road crossing of the embankment.

(10) Map all cracks on set of as-built plans and profiles as well as aerial photo of dam. Continue to map cracks after all dam safety inspections. Enter GPS coordinate crack location into District HIS system. Monitor, over time, reaches of dam where there has been a noted propensity of cracks.

E. Recommendations from FEMA Report

(1) Identify And Quantify The Existence Of Transverse Cracks. Identified transverse cracks should be noted on a set of as-built plans. Over time the District will be provided with a map that indicates the higher frequency of crack occurrence along the dam indicating reaches of the dam that may need a remedial repair.

(2) Determine The Cause Of Crest Holes. Several mechanisms of the formation of the crest erosion holes were discussed by the FMEA team. The team agreed that a likely causative mechanism for erosion hole formation is the downward movement of crest materials in the filter drain as a result of settlement of the filter.

(3) Dynamic Routing Is Recommended Due To Dam Length And Geometry. Unsteady flow modeling will provide better insight in the response of the structure and floodway from large storm events including the IDF and PMF. The dam design was based on the method of level pool routing. Due to the length of the structure, the water surface at one end of the dam may not be the same as at the other end during an impoundment event.

(4) Continue Monitoring Of Drain Outlets Where Silt Has Been And Continues To Be Observed. The inspection team observed silt in several of the downstream drain outlets. Future dam safety site inspection should document the presence (on not) of silt in every drain outlet. Large amounts of silt could be an indicator of stress within the dam.

(5) Verify Utility Relocations, Add Fiber Optic Line To The Plans, And Locate All Drain Outlets On The Plans And Replace If Not Found. The as-built plans indicate that several utilities were relocated prior to dam construction. A set of as-built plans should be used to record utility crossing of the dam and floodway. A fiber optic cable was recently constructed at the Salome Highway crossing of the dam.

(6) Evaluate Drain Outlet Video Tape. The District conducted a video survey of the drain outlet system located at the principal spillway.

(7) Evaluate Hydrologic Routing With Salome Road Culverts Plugged. This action item may be conducted as a modeling scenario for dynamic routing for large storm events. This approach could check for potential overtopping of the dam crest under the IDF and PMF.

(8) Perform Multi-Frequency Analysis To Determine Incipient Overtopping. A multi-frequency analysis of large storms will provide an indication of the frequency storm to just cause overtopping of the dam.

(9) Locate Utility As-Built Files. The dam construction plans indicate that several utilities were relocated prior to dam construction at Salome Highway. However, the as-built plans for the utility relocations could not be located in District or NRCS files. The locations shown on the dam as-built plans are assumed to be the as-built relocations from the as-built utility plans.

(10) Maintain Annual Surveys In The Area Due To Potential Fissure Risks. Three earth fissures have been documented in the Harquahala and Centennial valleys. It is prudent to continue with dam crest surveys and toe monument surveys to monitor for local land subsidence.

(11) Develop IGA With ADWR On Fissure Monitoring And Include Saddleback FRS In The Study To Develop Baseline INSAR Data. A fissure monitoring program may be developed in conjunction with other state and federal agencies. One agency is ADWR who has state oversight on groundwater use and groundwater pumping in the state. One of ADWR monitoring and interpretive techniques is the use of INSAR data and imagery.

(12) Regular Inspections In The Vicinity Of The Harquahala Floodway Discharge Into The Saddleback FRS And At The Roadside Drainage Next To Courthouse Road Are Recommended. The Phase I dam safety site inspection observed at Courthouse Road that a roadside drainage channel appeared to be migrating toward the right abutment. Monitoring of both abutments and the inflow location from the Harquahala floodway to Saddleback FRS is recommended during dam safety site inspections.

(13) Review Existing Instrumentation (Rainfall And Streamflow Gages) And Recommend Changes And Modifications If Necessary. A non-structural alternatives measure could include evaluation of added rainfall and streamflow instrumentation in the upstream watershed. A staff gage is recommended at the Salome Highway crossing. A stream gage should be considered at the CAP canal overchute that monitors floodwaters released from the CAP embankment.

F. Recommendations for Monitoring and Inspection of Longitudinal Cracks in Reservoir Area

(1) Continue to monitor for longitudinal cracks in pool area after major rainfall events and/or impoundments in pool area.

(2) Develop a crack investigation and repair method for longitudinal cracks.

(3) Develop rainfall criteria amount to trigger a site inspection of dam and pool area.

2.0 DESCRIPTION OF DAM

The Saddleback Flood Retarding Structure (FRS) is a structural plan element of the Harquahala Valley Watershed Work Plan for the Harquahala Valley Watershed, Maricopa and Yuma (former portion of Yuma County now La Paz County) Counties, Arizona. The Watershed Work Plan was prepared by the Wickenburg and Buckeye-Roosevelt Natural Resource Conservation Districts and the Flood Control District of Maricopa County with assistance from the Natural Resources Conservation Service (NRCS; formerly the Soil Conservation Services, SCS) in January 1967 (NRCS, 1967). The Watershed Work Plan was updated in March 1977 (NRCS, 1977) with additional assistance from the State of Arizona, Arizona Water Commission (now Arizona Department of Water Resources). The plan was developed under the authority of the Watershed Protection and Flood Prevention Act (Public Law 566, 83d Congress, 68 Stat. 666).

The Harquahala Valley watershed is in west central Arizona about 70 miles west of downtown Phoenix, Arizona. The watershed is in Maricopa and La Paz Counties between the Harquahala, Big Horn and Saddleback Mountains and a broad alluvial plain that drain into Centennial Wash. The total original watershed area was over 374 square miles.

2.1 Purpose of Dam

The Saddleback FRS is one of two flood retarding structural measures designed and constructed under the Watershed Work Plan. The other structure is the Harquahala FRS. The purpose of the Saddleback FRS is to provide flood protection and erosion control benefits to over 19,000 acres of farmland in the Harquahala Valley, as well as agricultural infrastructure, county roads, gas and phone utilities, and residential and commercial properties. The Saddleback FRS was designed to control runoff from the 100-year event.

The Saddleback FRS was constructed under the supervision of the NRCS beginning in 1979 under the local sponsorship of the Flood Control District of Maricopa County and the Wickenburg and Buckeye-Roosevelt Natural Resource Conservation Districts. PRC Toups of Phoenix, Arizona prepared the design calculations, plans, and contract documents under contract from the NRCS. Saddleback FRS was designed and constructed in conjunction with the Saddleback Diversion and Channel. Construction was completed in 1982.

2.2 Dam Location

Saddleback FRS is located between Courthouse Road (approximately the McDowell Road Alignment) and Interstate 10 in Sections 17, 21, 27, 28, and 34 of Township 2 North, Range 8 West. **Figure 1 (Appendix Figures)** provides a location map of Saddleback FRS. The project consists of the Saddleback FRS embankment, a principal spillway, and four irrigation outlets. There is no separate emergency or auxiliary spillway.

Saddleback FRS collects runoff from 22.3 square miles of the north and western slope of Saddleback Mountain and the eastern slopes of Burnt Mountain. The principal spillway from the Harquahala FRS discharges into Saddleback FRS reservoir via the Harquahala Floodway. The drainage area for the Harquahala FRS is 102.3 square miles.

The Saddleback FRS reservoir has a capacity of 3,620 acre-feet. A permanent pool is not retained in the reservoir. The Saddleback FRS and reservoir are designed to detain the 100-year floodwater and store the impoundment for a slow release of approximately six days into the Saddleback Diversion. Reservoir capacity is then restored to detain future stormwater runoff events.

The principal spillway is an 8-foot tall by 10-foot wide concrete box culvert. The irrigation outlets are 12-inch diameter steel pipes protected in a 2-ft. by 2-ft. concrete encasement

2.3 Physical Features

The Saddleback FRS is a 26,970-foot long (5.1 miles) homogeneous earthfill structure with 3:1 upstream slopes, 2:1 downstream slopes, and a crest width of 12 feet. The embankment also has a 5-foot wide vertical central filter/drain. The structure ranges from 1 to 20.8 feet above the existing ground. The volume of fill used for the structure is 690,000 cubic yards. The cutoff trench is 12 feet wide.

The dam crest elevation varies as shown in **Table 1 (Appendix Tables)**. There are 5-foot long transitions between the changes in dam crest elevation. The dam was constructed with a six-inch camber above the design crest elevation along its entire crest to facilitate drainage from the dam crest.

The vertical central filter/drain was placed in the structure with a drain outlet approximately every 400 feet starting at Station 8+00. Each drain outlet has a cross-section that is 2 feet tall, with a top with of 6 feet, bottom width of 2 feet, and a 6-inch diameter perforated asbestos cement pipe to facilitate internal drainage to the toe of the embankment. The central filter/drain was designed to be inclined, but was constructed vertically.

There is a 3-foot thick protective berm on the upstream toe of the dam from Stations 46+00 through 76+00 and 240+50 through 265+00. From Station 46+00 to 76+00 the berm extends to elevation 1179.6 ft (NGVD29) and from Station 240+50 to 265+00 it extends elevation 1187.2 ft.

The maximum recorded impoundment for Saddleback FRS is 102 acre-feet with a stage of 2.5 feet at the Saddleback FRS in July 1996. **Table 2 (Appendix Tables)** provides a summary of the physical data for Saddleback FRS.

Watershed

The dam was constructed across a number of local drainage washes conveying runoff from the Saddleback Mountains and the alluvial plain originate from the eastern slope of the Big Horn Mountains. An additional 55 square miles of the historic watershed was cut off from the rest of the watershed by the construction of the Granite Reef Aqueduct (GRA: also known at the Central Arizona Project or CAP) and was considered non-contributing area during the design of the FRS. The total contributing drainage area for the Saddleback is 22.3 square miles. The sediment storage requirement is 120 acre-feet, which is the estimated sediment that will be supplied by the watershed over a 50-year period

Flood Pool

The total capacity of the reservoir is 3,620 acre-feet including a sediment capacity of 120 acre-feet, which is the estimated 50-year sediment yield. The FRS was designed to capture floodwater runoff from the 100-year storm and discharge it through the primary outlet over a period of approximately six days. The total surface area of the sediment pool is 61 acres and the total area of the floodwater pool is 760 acres.

The principal spillway inlet channel is a low flow channel that is at the upstream toe of the dam, which conveys low flow through the reservoir from Station 34+75 to the primary outlet at Station 15+83.08. The flood pool is divided into two separate basins. Basin No. 1 is the largest basin and near the principal spillway. Basin No. 2, upstream of Basin No. 1, is at the outlet of the Harquahala Floodway.

Principal Outlet Works

The principal spillway was also designed to function as the emergency spillway. The principal/emergency spillway box culvert is located at Station 15+83.08 along the structure and is an 8-foot tall by 10-foot wide concrete box culvert that is 65 feet long. The principal spillway flows into the Saddleback Diversion Channel, which ultimately discharges into Centennial Wash. The Saddleback Diversion and Channel, which is a 25,000-foot long diversion structure and channel, were constructed in conjunction with the Saddleback FRS as part of the Watershed Work Plan.

The principal spillway consists of two segments connected by a bell and spigot joint that was designed to be watertight. The bell end is on the upstream side of the joint and spigot end is on the downstream side. There are two anti-seepage collars on the principal spillway that are 15.33 feet tall and 18.33 feet wide. The principal spillway inlet consists of concrete wingwalls, a concrete apron, and riprap. The outlet also consists of wingwalls, a concrete apron, and riprap and it has a 16-foot long Saint Anthony Falls (SAF) stilling basin.

There is a pressure transducer gage (ALERT gage) at the principal spillway location that was installed in 1988 as part the District's flood warning system. A staff gage is also located at the principal spillway.

Emergency Spillway

There is no separate emergency spillway associated with the Saddleback FRS. The Harquahala Valley Watershed Work Plan (NRCS 1967) indicates that the principal spillway is dual purpose serving also as the emergency spillway. The combination spillway was designed to pass the PMF through the reservoir without overtopping the dam with no freeboard. For a Class B structure, the FBH is a function of the PMP and 100-year.

Irrigation/Vegetation Maintenance Outlets

There are four 12-inch diameter gated steel pipe irrigation/vegetation maintenance outlets on the Saddleback FRS. The pipes are each protected in a 2-ft by 2-ft concrete easement. **Table 3 (Appendix Tables)** shows the locations and details of the conduits. The conduits consist of 8-foot and 16-foot long segments with bell and spigot joints that were designed to be watertight. The typical -*section from the as-built plans indicated that the bell end is on the upstream side of the joint and spigot end is on the downstream side.

Concrete anti-seepage collars that are 7 feet tall and 10 feet wide were constructed with the irrigation/vegetation outlets. They are fully encased in concrete with about 4.5 feet of structural backfill on top of the conduit (2 feet about the top of the anti-seepage collars).

The inlets for the irrigation outlets at Station 103+70 and 124+10 are standard NRCS Size C structures with trashracks. The inlets at 60+50 and 256+00 are special riser units that are taller because the 50-year sediment yield would bury a standard Size C inlet. The inlets at Station 60+50 and 256+00 are elevated by 2.29 feet and 3.79 feet, respectively, to account for the 50-year sediment yield.

The outlets for the irrigation outlets consist of concrete wingwalls, a concrete apron, a 3.5–foot long stilling basin, and downstream riprap. The channel downstream of the outlets has a bottom width of 6 feet, sideslopes of 2:1, 6 inches of bedding material and 18 inches of riprap.

3.0 TECHNICAL REVIEW

The purpose of the technical review was twofold. The first purpose was for the project assessment team to review the existing and available engineering records related to the Saddleback FRS and its' construction. Through this review, the project assessment team became familiar with the Saddleback FRS, the history of the Saddleback FRS, and the basis of the original analysis and design, which will assist in the engineering assessment of the Saddleback FRS. The second purpose was to review original design criteria and design guidelines under which the Saddleback FRS was constructed and compare them to current standards.

This section of the ISA report presents a discussion of the data review and dam design criteria under which the dam was originally constructed. The original dam design criteria will be compared to the current NRCS standards, current ADWR dam safety rules and regulations for jurisdictional dams, and any pertinent District guidelines.

The review of the technical documentation was limited to the available reports, studies, investigations, construction plans and as-builts, specifications, and office correspondence collected as part of this study. The data reviewed for this assessment were collected from several sources and repositories, which included the libraries and office files of the District, NRCS, and Arizona Department of Water Resources (ADWR)-Office of Dam Safety. Kimley-Horn has prepared under separate cover, a data collection report, summarizing the information collected for Saddleback FRS.

The basis for the structure assessment was through this technical review, the field examination, and the failure mode and effects analysis (FMEA). The assessment provides an evaluation the operational adequacy and structural stability of the dam and a review of the current dam safety rules and regulations compliance of the structure.

3.1 Dam Design Criteria

Saddleback FRS was designed by PRC-Toups, Inc. of Phoenix, Arizona under contract from the NRCS in the late 1970's. The basis of the Saddleback FRS design was originally founded in the NRCS publication "Engineering Memorandum EM-27" which is the precursor manual to "Technical Release TR-60: Earth Dams and Reservoirs" the present NRCS design guideline for earth dams. The Saddleback FRS was designed according to EM-27 and TR-60.

The purpose of the Saddleback FRS along with the Harquahala FRS was to provide a 100-year level of flood protection and erosion protection to over 19,000 acres of farmland in the Harquahala Valley, as well as agricultural infrastructure, the granite Reef Aqueduct, Interstate-10, county roads, an El Paso Natural Gas Line, an AT&T phone line, and residential and commercial properties (NRCS, 1977). The 100-year design event was used to size the principal spillway and reservoir storage volume according to

NRCS design standards. The hydrology for the emergency spillway design and freeboard design flood is discussed below in the Hydrology Section.

According to ADWR criteria, the Saddleback FRS Inflow Design Flood (IDF) is the ½ Probable Maximum Flood (PMF). PRC-Toups designed the structure to convey the freeboard hydrograph, which is generally a certain percentage of the PMP/PMF, without overtopping the top of the structure without any freeboard. **Table 4 (Appendix Tables)** provides a summary of the original NRCS design criteria (based on EM-27) and current TR-60 criteria for the dam and compares these criteria with current ADWR dam safety rules and regulations for jurisdictional dams.

3.2 Dam Classification

The NRCS, based on EM-27 and TR-60 guidelines, uses a three-category "hazard" classification system. The three categories or classes (Class A, B, or C) are established to permit the association of criteria with the damage that might result from a sudden major breach of the earth dam embankment.

The NRCS classifies Saddleback FRS as a Class A structure, though the design documents indicate that the Saddleback FRS was designed as a Class B structure (PRC, 1979). Class A structures are defined as those structures located in predominantly rural or agricultural areas where failure may damage farm buildings, agricultural land, or township and county roads.

The ADWR rules and regulations for jurisdictional dams classified the Saddleback FRS as a significant hazard, medium size dam. This classification is based on the size of the dam and the potential downstream hazard.

The Flood Control District conducted a downstream hazard and classification review of Saddleback FRS and Harquahala FRS (FCD, November 2004). The District review was based on December 2003 aerial photography, the dambreak report titled "Final Dambreak Analysis for the Harquahala and Saddleback Flood Retarding Structures" by Carter Associates (Carter Associates, 1991), and the U.S. Bureau of Reclamation's ACER Technical Memorandum No. 11 titled "Downstream Hazard Classification Guidelines" (Bureau of Reclamation, 1988). The purpose of the review was to assess whether either structure potentially could be elevated to a high hazard structure.

The District superimposed the dambreak inundation areas over the 2003 aerial mapping for each dam to identify areas of land used that were of significant economic value or have human inhabitance and subject to inundation during a dambreak. The District review identified three impacted areas: (1) Interstate 10 (impacted by Harquahala FRS); (2) Residential structures at Harquahala Valley Road and south of Salome Highway (impacted by Harquahala FRS only); and (3) Harquahala Generating Station at the intersection of Thomas Road and 491st Avenue (impacted by both structures).

The dambreak cross sections used in the 1991 dambreak study were superimposed on the 2003 aerial mapping. Elevations of the impacted areas were determined for use in estimating the maximum depth of dambreak flooding. The maximum flow depths and flood velocities at each cross section location in the impacted areas were then plotted on the flood danger level relationship plots from the Bureau memorandum. The District review included the results of the flood danger assessment. Note that the District's assessment was based on the 2003 aerial photography and did not assess the potential of future development in the downstream inundation areas (as required by ADWR the development downstream is taken 10 years from the present time).

The District concluded that there was insufficient information to support a change in the downstream hazard classification for both Harquahala FRS and Saddleback FRS from the existing classifications of Significant hazard. Kimley-Horn is of the opinion based on this Phase I assessment and the review conducted by the District, that there is not overwhelming support at this time for an elevated hazard classification for either dam. Sufficient information for a more formal engineering hazard assessment may be acquired at future time. These actions include:

- Preparation of updated dambreak analysis for each structure including revised dambreach parameters
- Evaluation of the impacts of the CAP canal immediately downstream of the Harquahala FRS
- Evaluation of the impacts of I-10 on the breach floodwave from Harquahala FRS
- Updating the available topographic mapping (the District study used the 20-ft contour mapping from USGS quadrangle maps)
- Dynamic routing of the IDF into the flood pool then through the breach.
- Modeling scenarios: (1) empty pool with breach; (2) full pool with breach; and (3) full pool with breach and no inflow "Sunnyday" failure.

3.3 Hydrology and Hydraulics Review

3.3.1 Hydrology. The "Watershed Work Plan–Harquahala Valley Watershed" was prepared by the NRCS in 1967 and updated in 1977 (NRCS, 1967, 1977). The final design report and final design report supplement were completed in 1980 (PRC, 1979).

The Watershed Work Plan identifies the structural elements of the watershed project including the Saddleback FRS, the Harquahala FRS, the Harquahala Floodway, the Saddleback Diversion, and the Centennial Levee. The two flood retarding structures capture and impound stormwater from their respective upstream watersheds. Primary outlet discharge from the Harquahala FRS is routed to the Saddleback FRS reservoir through the Harquahala Floodway. The Saddleback FRS discharges into the 5-mile long Saddleback Diversion which ultimately discharges into Centennial Wash. The Centennial Levee is approximately 5 miles long and directs stormwater runoff away from the developed farmland on the west side of the Harquahala Valley to Centennial Wash and is not hydraulically connected to the other structural work plan elements.

PRC designed the Saddleback FRS to detain the 100-year stormwater runoff volume and route it though the combination principal and emergency spillway. The 100-year runoff volume was calculated using the principles outlined in Chapter 21, National Engineering Handbook, Section 4 (NEH-4) based on the 24-hour and 10-day duration storms. Technical Release (TR) TR-20 was used to develop the inflow hydrographs into the reservoir. The National Oceanic and Atmospheric Administration (NOAA) atlas was used to develop precipitation data for 100-year return frequency storm event. Probable Maximum Precipitation (PMP) depths were determined using the document, "Probable Maximum Thunderstorm Precipitation Estimates in the Southwest States" (PRC, 1979). The runoff curve numbers were calculated from the SCS soil and cover reconnaissance surveys using procedures outlined in Chapters 7, 8, and 9 of NEH-4.

Times of concentration were derived from stream channel hydraulics. Channel cross sections were taken at several locations and velocities computed. Procedures outlined in Chapter 15 of NEH-4 were used. Rainfall depths, curve numbers, and times of concentration for the Saddleback FRS are similar to other structures in Central Arizona and Maricopa County during this time period.

Principal Spillway

The principal spillway hydrograph (PSH) is usually the hydrograph used to determine the minimum crest elevation of the emergency spillway, to establish the principal spillway capacity, and to determine the associated minimum floodwater retarding storage capacity. The Saddleback FRS has a combination principal and emergency spillway that is an 8-foot wide by 10-foot tall concrete box culvert with an invert elevation set essentially at the bottom of the reservoir. A PSH developed for the Saddleback FRS is based on the 100-year precipitation depth (P_{100}) and was routed through the reservoir using TR-20. The PSH did not include a TR-48 (DAMS2) run. **Table 5 (Appendix Tables)** summarizes the principal spillway hydrologic data from the design report (PRC, 1979).

The watershed was split into four major subbasins and curve numbers and times of concentration were estimated for a subbasin. Curve numbers for the 6-hour storm ranged from 83 to 88, with an average of 86 for the entire watershed. Ultimately, in the development of the PSH using TR-20, the curve number reverted to an average of 83 because the total precipitation was less than six inches. This adjustment is based on the design standard from the NEH-4 Table 21-2 (PRC, 1979). Precipitation depths of 4.15 inches and 5.67 inches were translated to watershed runoff depths of 2.02 inches and 3.27 inches, respectively, based upon an areal correction and a channel loss factor in the routing.

PRC design notes indicate that the reservoir was split into two major basins for the PSH. Basin No.1 is near the principal spillway and Basin No. 2 is near the outlet of the Harquahala Floodway. The principal spillway crest elevation is 1176.9 ft. The maximum reservoir outflow during the PSH is 1,100 cfs. The drawdown time of the reservoir is approximately six days. PRC expected the maximum reservoir water surface elevation during the PSH 1-day and 10-day events to be 1188.7 ft. and 1190.1 ft., respectively. At elevation 1190.1 ft., the volume of floodwater stored in the reservoir will be 3,620 acre-feet. The stage-storage curve developed for the Saddleback FRS was developed prior to construction, so the curve capacity did not include any additional storage capacity created by the excavation of borrow material from the reservoir area during construction. Current stage/storage/capacity curves are provided in the **Appendix Figures.** These curves are the latest curves from the District and are available on the District website.

There are two other sources of inflow in the Saddleback watershed in addition to the contributing watershed. A baseflow of 500 cfs from the Harquahala Floodway is discharged into Basin No. 2 in the TR-20 model. The maximum capacity of the floodway was estimated to be 500 cfs. As stated, the historic Saddleback FRS watershed is divided by the Control Arizona Project (CAP) canal, which intercepts nearly 55 square miles of watershed, of which a portion would have contributed directly to the Saddleback FRS watershed. The upstream embankment of the CAP is approximately 20 feet high and creates a basin that detains nearly 7,500 acre-feet of stormwater runoff. The runoff is controlled by five 72-inch diameter conduit overchutes that penetrate the CAP embankment. One of these overchutes discharges into the Saddleback FRS watershed. Consequently, PRC completed a hydrologic analysis for the 55 square mile upstream watershed, routed it through the CAP overchutes, 20% of which was added to the Saddleback FRS watershed as baseflow. It was estimated that 20% of the controlled release for the 100-year storm from the CAP basin would be equivalent to a baseflow of 241 cfs (PRC 1979). Separate rainfall depths, curve numbers, and times of concentration were developed for the watershed upstream of the CAP.

Emergency Spillway

A combination principal/emergency spillway design was used in the routing of the Freeboard Hydrograph (FBH), Emergency Spillway Hydrograph (ESH), and Principal Spillway Hydrograph (PSH). The Saddleback FRS is designed to detain the entire FBH. The combination spillway was designed to pass the FBH with no freeboard. According to standard NRCS design, the FBH actually sets the minimum settled dam crest elevation. The FBH is also typically used to evaluate the structural integrity of the spillway system. As stated previously, the structure was classified by the NRCS as a Class A structure, but was designed to Class B standards. Under the Class B standard, the FBH is based on a design storm precipitation depth derived from a combination of the 100-year precipitation depth, P_{100}, and the probable maximum precipitation (PMP). The FBH precipitation depth is equal to $P_{100} + 0.4*(PMP- P_{100})$.

For the Saddleback FRS, the 6-hour duration thunderstorm was used as the design storm for the FBH. The P_{100} (100-year 6-hour rainfall) was 3.33 inches was taken from NOAA Atlas 2 Volume VIII for the 6-hour duration event. The PMP value for a 6-hour duration event was 13.0 inches. The PMP for a 1-hour, 1 square mile thunderstorm was estimated to be 11.23 inches. A factor of 135% was applied for the 6-hour PMP and an areal reduction factor of 83% was applied for the 22.3 square mile watershed to obtain the value of 13.0 inches. The Class B FBH was generated by TR-20 and input into TR-48 (DAMS2) to determine the required top of dam elevation. **Table 6 (Appendix Tables)** shows a summary of FBH and ESH data and **Table 7 (Appendix Tables)** shows a summary of the reservoir and storage data taken from the TR-48 output.

The maximum peak discharge through the combination spillway during the FBH would be 1,300 cfs and the maximum water surface elevation in the reservoir would be 1192.8 ft. An FBH rainfall depth of 6.86 inches translates to a watershed runoff depth of 4.9 inches based on a curve number of 86.

A PMP rainfall value for the watershed upstream of the CAP was determined in a similar fashion using an areal reduction factor from the 55 square mile contributing watershed. During the development of the FBH, the upstream watershed was routed through the CAP overchutes and 20% of the discharge was input to the FRS watershed as baseflow, similar to the PSH. The FBH precipitation for the watershed upstream of the CAP was 6.0 inches, which translated to a runoff depth of 4.41 inches based upon a curve number of 86. PRC initially showed that the upstream CAP embankment would be overtopped by approximately one-foot during the FBH flood event. As a result, the overtopping was discussed with the United States Bureau of Reclamation and the portion of the CAP embankment that would be overtopped and would drain into the FRS watershed was raised by one-foot by to minimize the potential for overtopping (PRC, 1979).

The ESH was used to establish the dimensions of the emergency spillway. For a Class B hazard structure, the ESH is based on a watershed precipitation depth according to the following formula: $\{P_{100} + 0.12*(PMP - P_{100})\}$. Again, the storm duration was 6-hours. A precipitation depth of 3.99 inches translates to a runoff depth of 2.54 inches based on a curve number of 86. The ESH precipitation and runoff for the upstream CAP watershed was 3.49 and 2.09 inches, respectively.

3.3.2. Spillway Inundation Study. An emergency spillway delineation study was completed for the Saddleback FRS in May 1998 (Entellus, 1998). The study was completed for the District and was essentially a delineation of flows for the Saddleback Diversion Channel. The study extended from the Saddleback FRS combination spillway to the outfall of the Saddleback Diversion into Centennial Wash. The total length of the study was 6.5 miles.

The Saddleback Diversion Channel has a contributing watershed of 8.6 square miles (PRC, 1979a) and hence the anticipated discharge in the channel increases downstream. The discharges for the inundation study ranged from 1,120 cfs at the combination spillway to 6,060 cfs. The Saddleback Diversion Report indicates these discharges are based upon the maximum discharge from the combination spillway and the 50-year discharges from the contributing watershed of the Diversion Channel. The maximum outflow from the combination spillway is actually 1,300 cfs so it appears that the said discharge used in the inundation study is based upon the PSH discharge and the 50-year discharge from the contributing watershed.

The discharges and topographic information were provided to Entellus by the District. The study was completed as a steady-state one-dimensional analysis using the Corps of Engineers HEC-RAS program. **Figure 2 (Figures Appendix)** illustrates the emergency spillway inundation limits from the spillway to Centennial Wash.

The Manning's roughness coefficients, N-values, that were used in the study ranged from 0.028 to 0.07. The roughness coefficient was 0.035 for the majority of the diversion channel but did vary from 0.028 to 0.045 immediately downstream of the spillway and near the Courthouse Road culvert crossing. The roughness coefficients was 0.03 for the majority of the overbank area, but was 0.055 immediately downstream of the Courthouse Road crossing. The roughness coefficient was 0.07 for the main channel and overbank areas in the natural was segment between the Diversion outfall structure and the main channel of Centennial Wash. The study included a field reconnaissance report that followed District guidelines in determining Manning's N-values and a photographic record of the report. The study was assumed that the culvert at Courthouse Road was in good working order and would be fully functional during any discharge event.

3.3.3. Dambreak Analysis. A dambreak and flood routing analysis was completed for the Saddleback FRS by Carter Associates in February 1991 for the District (Carter and Associates, 1991). The analysis included a PMP hydrologic assessment using HEC-1, a dam breach analysis using National Weather Service (NWS) BREACH model, and a ½ PMF dambreak flood routing analysis using the NWS DAMBRK model. The BREACH and DAMBRK models have now been phased out of service by the NWS, combined, and replaced with the FLDWAV model. **Figure 3 (Figures Appendix)** illustrates the dambreak inundation area estimated during the study.

The hydrologic phase of the dambreak study found that the Saddleback FRS would not be overtopped during the ½ PMF. The dambreak study did not conduct an overtopping analysis for the PMF event. Therefore the selected potential failure mechanism studied was from internal embankment erosion or piping. The Corps of Engineers HEC-1 rainfall-runoff modeling program was used to estimate the hydrologic characteristics of the watershed. The study developed a PMF analysis for the 6-hour and 72-hour duration storm event on the Saddleback FRS watershed and then routed the ½ PMF flood through the reservoir to determine the maximum reservoir water surface elevation and to get the parameters for the dambreak analysis.

As with the original design, runoff from the watershed upstream of the Central Arizona Project (CAP) canal was routed through the CAP embankment and 20% was discharged into the Saddleback FRS watershed. Curve numbers ranged from 84 to 87 and times of concentration ranged from 0.4 to 1.15 hours.

The study indicated that the 72-hour storm was the most critical for the DAMBRK analysis since the 72-hour generated the largest runoff volume to the reservoir and produced the highest reservoir water surface elevation for the ½ PMF event. The 6-hour duration storm produced the greatest reservoir peak inflow. The PMP total precipitation depths developed for the study and applied to the watershed were 9.5 and 15.4 inches for the 6-hour local thunderstorm PMP and 72-hour general storm, respectively. The general storm PMP included an orographic rainfall component that totaled 4.1 inches and was developed for the month of August.

All of the physical characteristics of the structure described in the previous sections were used in this study with a few minor exceptions. The maximum discharge from the emergency/principal spillway was 1,315 cfs. The study used SCS soil loss methods and the SCS unit hydrograph for rainfall transformation. It appears that level pool routing was used to route the inflow hydrograph through the reservoir. **Table 8 (Appendix Tables)** shows the hydrologic summary data from the dam break study.

The dambreak study used the results from the hydrologic analysis as a basis in determining the dam breach parameters for the structure. The dam breach analysis was completed for the 6-hour and 72-hour ½ PMF events even though a 'sunny day' failure was the only failure mechanism identified. The dam breach was initiated at the maximum reservoir water surface elevation as shown in **Table 8 (Appendix Tables).** Two breach locations were chosen for the structure; one at the south end of the dam near Basin No. 1 by the spillway and another near the Basin No. 2 location on the north end of the dam. The elevation at which piping was initiated was half the distance between the crest of the dam and bottom of the reservoir. Soils information used in the BREACH analysis was not identified or located in the report. **Table 9 (Appendix Tables)** shows some of the dam break parameters used in the study.

The dambreak breach parameters and resulting dambreak peak discharges appear to be conservative than what would be expected. The breach widths and times of breach are not reasonable. The breach occurs during the peak of the ½ PMF event and consequently is not a true 'sunny-day' failure. Usually 'sunny day' piping breaches are analyzed when the reservoir water surface elevation is at the emergency spillway crest elevation with no inflow. This should be analyzed for the Saddleback FRS since there not a traditional emergency spillway crest elevation.

Kimley-Horn recommends that an updated dambreak analysis and inundation mapping be prepared for the Saddleback FRS. New integrated hydraulic models such as HEC-RAS (unsteady flow and dambreak options) could be used to prepare the updated study. The dambreak update should develop reasonable dambreach parameters using published guidelines and the District's dambreach model currently under development. The true "sunny day" failure defined as a full pool with no inflow should be considered as well as a dambreak for the ½ PMF and PMF events using ADWR guidelines for routing through a flood control dam.

3.3.4 Sedimentation. The estimated 50-year sediment volume was identified as 120 acre-feet in the Watershed Work Plan. The sediment was distributed between the two major basins of the reservoir. The sediment volume provided at the dam was 110 acre-feet. It was estimated that 80 acre-feet of sediment would settle into Basin No. 1 and the top of the sediment pool would be at 1178.13 ft. Basin No. 2 will retain 30 acre-feet of sediment and the top of the sediment pool will be 1185.7 ft. No reservoir storage was designed below the top of the sediment pools. The top of the sediment pool is above the invert of the combination spillway, so it is assumed that the sediment pool may not fully form in Basin No. 1 as it will be carried downstream through the combination spillway.

The annual sediment yield was estimated by the NRCS to be 0.11 acre-feet per square mile per year. This annual sediment yield over a 50-year design life of the structure provides the 120 acre-foot sediment storage volume.

A review of the computed annual sediment yield for Harquahala FRS was conducted by Kimley-Horn to compare to the annual yield for Saddleback FRS. The Watershed Workplan indicated that the Harquahala FRS watershed would yield 914 acre-feet of sediment in 50-years. This corresponds to an average annual sediment yield of 0.0081 acre-feet per square mile per year.

Kimley-Horn recently completed a sediment yield study for two earth embankment dams located in Pinal County, Arizona (Kimley-Horn, November 2003). As part of the study, Kimley-Horn reviewed the sediment yields for several dams with Maricopa County and Pinal County. The average sediment yield was determined to be 0.2 acre-feet per square mile. Based on this observation further evaluation of sediment yield is required for Saddleback FRS but at a future time. Further re-evaluation may be considered pending upstream land use changes.

3.4 Geological and Geotechnical Review

This section summarizes the review of the geological and geotechnical aspects of Saddleback FRS. The full presentation of the geologic and geotechnical review is provided in **Appendix A** and **Appendix B**, respectively. The geologic review was conducted by Geological Consultants, Inc., on behalf of Kimley-Horn and Associates, Inc. The geotechnical review was conducted by Gannett Fleming, Inc., on behalf of Kimley-Horn and Associates, Inc. This section of the report provides a summary of the major discussion and findings presented in **Appendix A** and **Appendix B**. The reader is referred to these two appendices for further discussion.

3.4.1 Geologic Setting. The Saddleback FRS is located within the Sonoran Desert section of the Basin and Range physiographic province. This portion of the Basin and Range is characterized by north and northwest trending mountains that rise abruptly to form broad, elongated, deep, sediment-filled valleys produced by block faulting, tilting and folding.

The structure lies in the east-central portion of the Harquahala Valley. The Harquahala valley is a northwest trending alluvial valley bounded on the north by the Harquahala Mountains, the northeast and east by the Big Horn and Saddle Mountains, the west by the Eagletail and Little Harquahala Mountains, and the south by the Gila Bend Mountains. The most prominent geologic feature near the Saddleback FRS is Saddle Mountain to the south and southeast of the structure and Burnt Mountain to the north and northeast. Saddle Mountain is composed predominately of mid-Tertiary volcanic rocks with underlying Proterozoic crystalline rocks (Ort, 1993). Basalt and basaltic andesite are the most common volcanic rocks, along with a volcanic breccia. Proterozoic crystalline rocks include granodiorite and slaty metavolcanic rocks. Burnt Mountain is a late Tertiary volcanic center of mainly andesitic composition (Stimac, 1994).

The valley basin consists of late Tertiary and Quaternary deposits. The Saddleback FRS lies primarily in Quaternary or Tertiary age old alluvium composed of caliche-cemented, unconsolidated to semi-consolidated sand and gravel deposits (ADWR, 2004). These deposits include a sedimentary sequence that varies in thickness from 0 feet to more than 5,000 feet and is generally divided into three units, the upper alluvial unit, the middle alluvial unit, and the lower conglomerate unit.

The upper alluvial unit may range from 0 feet to greater than 1,300 feet in depth and is composed primarily of late Pliocene to recent deposits. The unit consists of unconsolidated sand and gravel with some interbedding of silt and clay (Bureau of Reclamation, 1976). The middle alluvial unit consists of fine-grained interbedded sand and silty clay overlying a silt and clay layer containing some reworked evaporates, over a layer of primarily evaporates containing minor silt and clay (Bureau of Reclamation, 1976). The middle alluvial unit varies in thickness and because of the proximity of the underlying volcanic bedrock, the middle alluvial unit probably does not underlie the Saddleback FRS. The lower conglomerate unit consists of pebble to cobble size, variably cemented clasts of middle to late Tertiary age (Bureau of Reclamation, 1976). This unit is the primary aquifer in the Harquahala Valley.

According to the SCS (1978), the Saddleback FRS traverses a bajada that consists of primarily of unconsolidated and semi-consolidated (caliche cemented) alluvial fan deposits derived from the Burnt Mountain area and unnamed hills to the east composed of schist and volcanic rock that overlie mixed older alluvial fan deposits and layered volcanic rock. Several ephemeral washes cross the FRS alignment. These channels are incised into the caliche cemented fan deposits and the channels have been subsequently filled with unconsolidated sand and gravel. The surface expression of many of these channels is subdued.

A. Dam Centerline Surficial Geology

The following describes the surficial geology along the centerline of the dam. These descriptions are excerpted from the SCS Report of Geologic Investigation for the Saddleback FRS (1978). The descriptions are deduced from the SCS site investigation that included sampling and logging of 20 backhoe pit and 19 drill holes.

Station 0+00 to 33+00: The surficial soils are described as a loose to very loose silty gravelly sands to sandy gravels. Desert pavement is moderately well developed suggesting a relatively old and stable surface. The percentage of cobbles in the soils increases with depth. A loose surface horizon is about three feet thick and it is underlain by caliche cemented gravelly sands and sandy gravels with a small percentage of low plasticity fines. Cementation is moderate to strong.

Station 33+00 to 125+00: The soils along this portion of the FRS alignment are very loose to loose consisting of non-plastic silty sands to sandy silts with some fine gravel. The surface soil ranges from about 4 feet to 9 feet thick that are underlain by moderately to strongly cemented sandy silts, silty sand, clayey sand, and sandy clay with varying amounts of fine to coarse gravel. Several buried stream channels containing loose, highly permeable gravelly sands were found along this segment. Some of these loose deposits are interbedded with caliche cemented soil layers while other channels are incised into the cemented older alluvial fan deposits. Desert pavement is lacking along this segment suggesting the surface soils are relative young.

Around Station 115+00 the dam centerline passes between two volcanic rock outcrops. The surface soils in this area are underlain by cemented fanglomerate at a depth of about 6½ feet. Boring B-9 encountered pink porphryritic tuff beneath the fanglomerate. The contact boundary between the tuff and the fanglomerate drops off gradually to greater depths on both sides. At Station 117+14, the contact between the two units is about 13 feet below grade and at Station 112+78 it is about 12 feet below grade.

Station 125+00 to 280+00: Desert pavement is either very well developed or totally lacking along this segment of the alignment. The surface soils consist of loose to very loose sandy clays, silty sands, and clayey sands with varying amount of fine to coarse gravel. The surface soil zone is about 4 feet thick and overlies sandy clay, silty sand and gravelly sand that is well cemented with earthy caliche. Buried channel fill deposits containing loose to firm, permeable gravelly sand and sandy gravel are found locally along this segment.

B. Principal Spillway Surficial Geology

Geologic conditions in the Principal Spillway area are described as very loose to loose silty sand to an average depth of about 4 feet. Beneath the surficial soils (that were removed before the structure was constructed), the underlying material consists of consolidated, indurated caliche cemented fanglomerate. The degree of cementation reportedly increases with depth and the material becomes coarser grained grading into a cobbly fanglomerate at a depth of 8½ feet. The soils in this zone had very high blow count values from standard penetration testing conducted in this area.

3.4.2 Seismic Evaluation. In 2002, a Seismic Exposure Evaluation was performed by AMEC Earth & Environmental, Inc. for the Dam Safety Program of the Flood Control District of Maricopa County. According to this report, the Saddleback FRS lies within the Southern Basin and Range Source Zone. A seismicity evaluation conducted for the Arizona Department of Transportation describes this zone as the Sonoran Seismic Source Zone **(Figure 3 Appendix A)** (Euge, Schell, & Lam, 1992). This source zone appears to have a low level of seismicity and few active or potentially active faults. Within this source zone, the largest historical earthquake was a 1956 magnitude 5.0 event that occurred in the southern portion of the zone (AMEC, 2002).

The closest active fault to the Saddleback FRS, Sand Tank Fault, is approximately 83.3 miles southeast of the structure **(Figure 3 Appendix A)**. Sand Tank Fault lies in south-central Maricopa County, east of the town of Gila Bend. Sand Tank Fault is a normal fault with a slip rate of less than 0.02 millimeters per year and a recurrence interval of approximately 100,000 years (AMEC, 2002). This fault may be capable of producing an maximum credible earthquake of magnitude of 5.7 and an associated maximum peak horizontal acceleration at the Saddleback FRS equal to 4 percent of the gravitational acceleration (g) (AMEC, 2002). The recommended peak horizontal acceleration design criteria calculated by AMEC for the Saddleback FRS is 0.10 g. **Figure 4 (Appendix A),** the Horizontal Acceleration Map (from Euge et al, 1992), shows a 0.03 g horizontal acceleration of bedrock with 90 percent probability of non-exceedance in 50 years in the vicinity of the Saddleback FRS.

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

A. Groundwater

The major human-induced factor contributing to subsidence is the large scale pumping and removal of groundwater. Nearly all of the populated southern Arizona basins from Phoenix to Tucson have experienced at least a 100 plus foot drop in groundwater level, and an area surrounding the town of Stanfield, Arizona has dropped more than 500 feet (Schumann, 1986).

1. Groundwater in the Harquahala Groundwater Basin

The Saddleback FRS is located in the Harquahala groundwater basin in west-central Arizona. The lithology of the basin varies widely, but is generally composed of a heterogeneous mixture of clay, silt, sand and gravel (Corkhill, 1998). The alluvium may range from 0 feet deep at the base of the mountains to more than 5000 deep in the center of the basin. The alluvial deposits grade from coarse-grained sand and gravel in the southeast to fine-grained deposits in the center of the basin. Fine-grained clay deposits, over 1000 feet thick, occur in the western part of Township 2 North, Range 9 West (Corkhill, 1998). The fine-grained beds grade toward the west into an alternating sequence of fine-grained and coarse-grained layers from 800 to 850 feet thick, overlying a conglomerate unit.

The main use of groundwater in the Harquahala basin is for agricultural purposes. Prior to 1951, groundwater in the basin flowed from the northwest to southeast. By 1963, three cones of depression had developed in the southeastern part of the basin which, by 1966, had coalesced into one large cone in the center of the valley (ADWR, 2005). By 1986, the basin had experienced a decline in the groundwater level in some areas of as much as 300 to 500 feet (Schumann, 1986).

2. Groundwater in the Project Vicinity

Hydrographs for 26 wells within approximately 2.5 miles of the Saddleback FRS were obtained from the Arizona Department of Water Resources, with the oldest dating back to 1957 **(Figure 5 Appendix A)**. These hydrographs show an overall decline in groundwater levels of between 49 and 280 feet. From the early to mid 1980's, the wells in this area have experienced an increase of between 28 and 220 feet, but have not recovered to pre-pumping levels.

B. Regional Subsidence

Prior to the utilization of groundwater in south-central Arizona, the water table was higher and hydrogeological conditions were in equilibrium. Water levels within the aquifers were lowered when pumping was initiated and the basin fill sediments were dewatered. In the arid southwest, the water in the aquifer may be removed by pumping faster than it can be naturally replenished causing a net water table decline. As a result, the weight of the soil column is gradually increased as the buoyant effects and aquifer pressures induced by the water acting on the soil column are decreased. This condition causes increased loading stresses to consolidate portions of the thick compressible sediments that result in the lowering (subsidence) of the land surface over a large area.

Land subsidence was first documented in Arizona in 1934 following the releveling of first-order survey lines by the Coast and Geodetic Survey (now the National Geodetic Survey (NGS)). Subsequent leveling by the NGS, the U.S. Geological Survey, the Bureau of Reclamation, and the ADOT has documented substantial land surface subsidence in south-central Arizona including the Salt River Valley, the Queen Creek-Apache Junction area, the Eloy-Casa grande-Stanfield area, and the Harquahala valley area as overdrafting of the aquifer continues.

Subsidence and earth fissures in urban areas can cause a variety of problems. Structures built across fissures may be damaged, streets may crack, flow in gravity water and sewer lines can be reversed, and differential subsidence (although rare) can rupture buried utilities (Arizona Geological Survey, 1987). However, design measures can be implemented to mitigate the effects of land subsidence. Some of these measures can include additional structural reinforcement, over-sized pipes, surface drainage controls, bridging the subsidence feature, and avoidance.

1. Study Area Subsidence

Historic National Geodetic Survey (NGS) level line data is not available in the vicinity of the Saddleback FRS However, recent historic subsidence-settlement is available from the Flood Control District of Maricopa County using crest and toe monument elevations recorded between 1984 and 2003. A summary of the settlement that has occurred along the dam is shown in **Table 1 (Appendix A)** (FCDMC, 2004) and **Figure 6** of **Appendix A**.

According to this data, it appears that negligible settlement or subsidence has occurred across most of the dam and a very minimal amount is recorded on one crest section between A-5 and A-10, A-21 and A-23 and the toe at B-25, from 1983 to 2003 **(Figure 6 Appendix A).** The change in elevation in this area ranges from -0.001 to -0.300 feet. Overall the entire Saddleback FRS appears to be relatively stable in terms of settlement and subsidence. This is not surprising when considering the structure is located within the volcanic bedrock pediment area that is overlain by well cemented fanglomerate.

The minimal subsidence recorded from 1983 to 2003, along with the static or increasing groundwater levels in the area would suggest that future land subsidence in the vicinity of the Saddleback FRS would be minimal. This is subject to change if increased pumping of the groundwater caused the water level to decline thereby increasing land subsidence.

3.4.4. Earth Fissures. Fissures occur in unconsolidated sediments, typically near the margins of alluvial valleys or near the bedrock pediment edge where land water levels have dropped from about 200 feet to 500 feet below land surface (Schumann, 1986).

Fissures are initiated deep underground when tensile stresses exceed the strength of the soils. Tensile stresses induced by the subsidence continue to increase until the ground breaks to form earth fissures. The fissure then propagates upwards to intersect the ground surface. Examples of typical earth fissure characteristics are provided in **Figure 7 Appendix A.** Early signs of earth fissuring are small, en echelon, hairline cracks and irregular spaced depressions at the surface. As fissures develop the cracks grow in length to create fissures 1 foot to more than 10 feet deep when subject to erosion caused by surface runoff. The fissures often have vegetation growing in them because the ground is commonly moister along the earth fissure. Other physical features associated with fissure are slump-related escarpments from one inch to a few inches in height, as well as a drainage pattern associated with the fissure that does not conform to the areas local drainage pattern.

Field evidence indicates fissures propagate upward and are exposed after overlying sediments are eroded by surface water runoff from rainfall or irrigation (Pewe, 1982). The surface expressions of the fissures are exaggerated because the initial hairline crack is attacked by water to create wide (10 to 20 feet) and deep (more than 15 feet) erosional gullies that often have vegetation growing in them. The fissures are commonly perpendicular to natural drainage channels. The length of the fissure at the ground surface varies; usually less than one mile but one fissure near Picacho is more than 9 miles long. These features are easily recognizable on aerial photographs and in the field except where the ground surface is modified by agricultural activities or urban development.

A regional gravity survey was conducted that included the Saddleback FRS vicinity (Oppenheimer, 1980). The Oppenheimer map estimated the depth to crystalline bedrock under the study area to be from 400 to 600 below ground surface, with the depth to bedrock depth increasing away from the mountain front. The depth to volcanic bedrock ranges from zero were exposed at the ground surface to probably less than 400 feet.

Figure 8 Appendix A is a modified Bouguer Anomaly map and a modified Structure Contour Map, from the Bureau of Reclamation, Geology and Groundwater Resources Report (1976). As depicted in **Figure 8 Appendix A,** a relatively prominent bedrock boundary condition can be deduced that reflects the approximate buried limit of the volcanic rock west of the Saddleback FRS. It is possible that this boundary between the volcanic bedrock and the basin fill alluvial sediments could be the focus for earth fissure development; however, the trend of this boundary does not appear to cross the Saddleback FRS alignment. Therefore, it is unlikely that earth fissures could develop that would adversely impact the Saddleback FRS.

A. Known Earth Fissures in the Project Vicinity

There have been three earth fissures reported in the Harquahala Valley. The closest fissure to the Saddleback FRS lies approximately 2.5 miles west of the structure in Section 36, Township 2 North, Range 9 West **(Figure 9 Appendix A)**. There is no current information on the status of this fissure. An examination of recent aerial photographs of the area did not display any feature that would be indicative of the fissure. This is probably due to the fact that the reported fissure is located in an agricultural area and any surface expression of an earth fissure would be destroyed during agricultural activity.

Another fissures lies approximately 4.7 miles northwest of the structure in Section 9, Township 2 North, Range 9 West. This fissure was first discovered in 1958, visible in an aerial photo. The fissure was examined in 1978 and appeared to have been dormant for many years (Graf, 1980).

The Rogers fissure was discovered in 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 11 miles west of the dam, when it made an abrupt appearance during an unusually heavy rainfall event. The fissure is approximately 4,400 feet long, averages 5 to 15 feet deep and 5 to 10 feet wide, with prominent near vertical side slopes (Photos 1 & 2, next page) (Corkhill, 1998). Development of the surface expression of the Rogers fissure was unusual in that there were no reported precursor features, such as small surface cracks, aligned potholes, linear depressions or linear vegetation, in the area that would have indicated the fissure was present.

In 2001, another earth fissure appeared suddenly, following a heavy rain. This fissure appeared in the West Salt River Valley, west of the Palo Verde Generating Station. This fissure is about 14.4 miles southeast of the Saddleback FRS.

Photo 1: View of Rogers earth fissure with gulley headcutting upslope along the fissure alignment.

Photo 2: Well developed fissure gulley along portion of Rogers earth fissure. Note slump blocks in bottom center of view generated from the tabular failure of the over-steepened fissure side slopes.

3.4.5. Review of Previous Geotechnical Documentation. A comprehensive review of existing geotechnical reports was performed. The following documents were reviewed:

- Watershed Workplan for the Harquahala Valley Watershed (Flood Control District of Maricopa County (FCDMC), 1967)
- Supplemental Watershed Workplan, Harquahala Valley Watershed (FCDMC, 1977)
- Engineering Report, Saddleback Diversion Harquahala Valley Watershed (U.S. Department of Agriculture Soil Conservation Service, 1987)
- Report of Geologic Investigation, Harquahala Valley Watershed, Saddleback Diversion and FRS (SCS, 1978) – including Appendix entitled Harquahala Valley Watershed Earth Crack Investigation by Ronald L. graner, Geologist, October 10, 1966
- Materials Testing Report section of Geologic Investigation Report, Saddleback FRS and Diversion
- Saddleback Floodwater Retarding Structure, Harquahala Valley Watershed Engineering Documentation, Phase II (PRC Toups Corporation, 1979)
- Structural Stability of Embankments, Appendix to Design Report for the Saddleback FRS (Toups Corporation, 1979)
- Saddleback Floodwater Retarding Structure as-built plan set
- Operation and Maintenance Manual, Saddleback FRS and Diversion (U.S. Department of Agriculture Soil Conservation Service, 1980)
- Plan set (12 sheets) for Saddleback Diversion repair (U.S. Department of Agriculture Soil Conservation Service, 1989)
- A New Earth Fissure Opens in the Harquahala Plain of West-Central Arizona (September 25, 1997) (Arizona Department of Water Resources Hydrology Division and Groundwater Management Support Division, 1998)
- Dam Construction Inspection records
- Annual dam inspection checklists
- Documentation of the rehabilitation of central drain between Stations 44+00 and 52+00
- Special Inspection Reports and documentation regarding longitudinal cracking (Flood Control District of Maricopa County, October through December 2000)
- Geotechnical Investigation Report Saddleback FRS (AMEC Earth and Environmental, 2001)
- Downstream Hazard & Classification Review (Flood Control District of Maricopa County, 2004)

The following sections provide a discussion of findings from that review.

A. Regional Setting

Information on the regional setting of the Saddleback FRS was summarized and/or excerpted from FCDMC (1967) and SCS (1978).

The Harquahala Plain overlies a broad elongated alluvium-filled groundwater basin located about 60 miles west of Phoenix, Arizona. The plain is bounded to the north by the Harquahala Mountains, to the west by the Little Harquahala Mountains, to the southwest by the Eagletail Mountains, to the south by the Gila Bend Mountains, to the east by Saddle Mountain, and to the northeast the Big Horn Mountains. The Harquahala Plain and surrounding mountains cover an arid desert area of about 750 square miles. Centennial Wash, the major surface-water drainage in the basin, is an ephemeral stream which flows only in response to rainfall events. The average annual precipitation is about 6 inches per year.

The alluvium of the Harquahala basin is composed of a heterogeneous mixture of clay, silt, sand and gravel. The thickness of the alluvium varies from 0 feet at the mountain fronts to over 5,000 feet in the deepest part of the basin. The alluvial deposits generally grade from coarse sand and gravels in the southeastern portion of the basin to fine-grained deposits in the central portions of the basin. Fine-grained clay deposits exceeding 1,000 feet in thickness occur in the western portion of T2N, R9W. Farther west, near Sections 34-36, T3N, R11W, the fine-grained beds appear to grade into an alternating sequence of fine-grained and coarse-grained layers that overlie a conglomerate beginning at a depth of about 800 feet.

The area is within the Sonoran Desert Section of the Basin and Range physiographic province. The portion of the Harquahala Mountains included in the watershed area is composed mainly of Precambrian granite gneiss and schist, Paleozoic and Mesozoic shale, quartzite, and limestone, and Laramide granite and related crystalline rocks. The portion of the Big Horn Mountains included in the watershed is made up of Cretaceous andesite and andesitic tuff, Precambrian granite and granite gneiss, and Quarternary basalt with small areas of rhyolite, shale, quartzite, and limestone. The Saddleback Mountains are composed mainly of Precambrian schist, Cretaceous andesite and Quaternary basalt. Gentle alluvial slopes extend basinward from the mountains. Quaternary-Tertiary sand, gravel and conglomerate are present near the mountain fronts with Quaternary clay, silt, sand, and gravel occurring at the lower elevations.

Deep or moderately deep soils are present on the relatively flat-lying (1-5% slope) alluvial plains. Medium or moderately fine surface soils and subsoils are on the smoother slopes near the center of the valley. Coarse or moderately-coarse soils are present on the upper fans of washes from the granitic mountains. Along the foot of the mountains, there is usually an area of shallow to moderately deep residual soils. These often have a medium textured surface with gravel that is covered with dark desert varnish. They have slightly finer subsoils underlain at 12 to 28 inches by a strongly cemented lime hardpan. Alluvium for the valley fill soils originates in the granite, granite gneiss, schist, limestone, andesite, basalt, and shale rocks of the adjacent mountains. The soils in the plain are slightly to moderately erosive. Since the land surface is relatively flat and a sheet flow runoff condition prevails, erosion is generally not significant. Erosion is active in some of the channels and diversions constructed in and around the cultivated areas where flood flows are concentrated. Generally, the soils have a slow to very slow rate of water transmission and a slow to very slow infiltration rate when thoroughly wetted because of moderately fine to fine texture or a layer that impedes downward movement of water.

The Saddleback FRS is located in a bajada area that consists primarily of unconsolidated and semi-consolidated (caliche-cemented) regolith which overlies mixed alluvium and intercalated igneous rocks. The regolith is primarily Quaternary-Tertiary alluvial fan deposits derived from the surrounding mountains. The regolith is composed of very loose to very dense semi-consolidated sands, silts, clays and gravels. Calcareous cementation governs the degree of apparent cohesion. Generally, density and cementation increase with depth. The cemented materials consist of gravelly to earth caliches with fairly high in-place densities.

B. Foundation Conditions

Surface soils along the centerline of the dam alignment were described in the Report of Geologic Investigation (SCS, 1978) as consisting of very loose to loose silty or clayey sands and sandy silts and clays with a variable amount of gravel. These loose materials generally extend to depths of between 3 and 5 feet, and up to 9 feet in some locations. These loose materials are underlain by relatively firm and incompressible caliche of cemented sandy gravels, clayey, silty and/or gravelly sands, and sandy silts and clays. Relatively high permeability channel fill deposits of gravelly sands and sandy gravels were reportedly interbedded with, or incised into, the lower permeability caliche at several locations along the dam alignment. There was reportedly no surface expression of these channel fill deposits.

A bedrock high was reported by SCS (1978) in the vicinity of Station 115+00. According to SCS (1978), the loose surface soils were observed to a depth of approximately 6 feet. Cemented fangolmerate overlying volcanic bedrock was present beneath the surface soils. SCS (1978) concluded that the area of the bedrock high was relatively localized because the depth to the fanglomerate reportedly increased to approximately 12 to 13 feet at Stations 112+78 and 117+14, respectively. The as-built plan set also shows this to be a relatively localized feature. According to the as-built plan set, the bedrock high was encountered near Station 110+00. The reason for the discrepancy between the original Report of Geologic Investigation Report and the construction records is unknown.

The dam design called for removal of the loose surface soils to an average depth of 6 feet, with the final depths to be determined by the engineer after inspection of the materials encountered. The as-built plans indicate that the surface soils were removed to depths of between 3 and 9 feet.

C. Embankment

The designers reported that adequate borrow materials were available upslope from the dam centerline and that the soils in these locations correlated well with the soils along the centerline. Abundant fine-grained material was reportedly available for the embankment core and the non-plastic fines, reportedly the most common surface material, were suitable for selective use in the embankment.

The Saddleback FRS was designed as a homogeneous (3H:1V upstream (Zone I) and 2H:1V downstream (Zone III) with an inclined, 5-foot wide filter/drain (Zone II). The as-built construction documentation indicates that the filter/drain was constructed as a centrally-located vertical filter/drain. The reason for the installation of a vertical filter/drain rather than the inclined filter/drain is unknown. No documentation could be located in the project literature that explained the changed from an inclined to a vertical filter/drain. The filter/drain extends 1 foot into the cutoff over most of the alignment. However, it extends to the limit of foundation excavation at three locations (Station 46+00 to Station 76+00, Station 104+00 to Station 114+00, and Station 238+20 to Station 265+70). The filter/drain was designed to terminate 3 feet below the crest of the embankment; however as-built drawings of the filter/drain rehabilitation performed in 1995 indicate that the depth to the top of the filter/drain is approximately 3.5 ft to 5.5 ft. The filter/drain is connected to a system of 6-inch diameter perforated outlet pipes surrounded by drain rock installed at 400-ft intervals beginning at Station 8+00.

According to the as-built plan set, the bottom of the foundation cutoff trench was grouted using dental grout between Stations 108+00 and 110+00, in the vicinity of the bedrock high described in the previous section. The grouting was performed to fill cracks and surface irregularities and to provide a uniform base for construction of the cutoff trench.

A cut-off trench was installed along the entire dam to depths of up to 11 feet. Protective berms were constructed along two upstream sections at locations that coincide with two of the three filter/drain extension locations (Station 46+00 to Station 76+00 and from Station 240+00 to Station 265+00). It is assumed that the berms were incorporated into the design to improve stability during rapid drawdown loading conditions. A typical cross-section of the embankment is shown as Figure 1.

1. Embankment Materials

The embankment earth fill (Zones I and III) and filter/drain materials (Zone II) have the characteristics summarized on **Table 1 Appendix B**, based on the project design specifications.

The materials used to construct Zones I and III were derived from local borrow sources in the vicinity of the dam. The borrow materials were described by SCS (1978) as consisting of sandy clays, clayey sands, and silty sands with variable amounts of fine gravel. Fines in the borrow materials were reported to be non-plastic silts and slightly plastic clays. Logs of soil borings and test pits and laboratory test results for bulk samples obtained from the sediment pool approximately 500 ft to 800 ft from the dam centerline were provided by SCS (1978). At least twenty-two samples were collected for laboratory analysis from within the impoundment area along the alignment. The results of the laboratory testing reported by SCS (1978) are summarized on **Table 2 Appendix B**. SCS concluded that there was abundant material available for use in the embankment however as-built gradation data were not available for review.

Although no design report for the Saddleback FRS was available for review during this Phase I Structures Assessment, it is assumed that the filter/drain was designed based on the results of laboratory testing of soils presented above, in a manner similar to the design of nearby flood retarding structures constructed by SCS at a similar time (for example, Harquahala FRS). The compatibility of the Zone II fill as a filter for Zone I/III was assessed and is discussed in the next section.

D. Filter Compatibility

Zone II is shown on the as-built drawings as a 5-ft wide, vertical chimney drain with the centerline of the drain coincident with the centerline of the dam crest. The most important function for the Zone II materials is to serve as a filter to protect against potential internal erosion and piping of the Zone I materials in the event of transverse crack development.

Because of its critical function as a filter, the Zone II gradation was checked against current filter criteria in accordance with the NRCS, National Engineering Handbook, Chapter 26 "gradation Design of Sand and gravel Filters" (NRCS, 1994). **Figure 2 Appendix B** shows gradation curves for Zone I "Base Soil" materials (graphed with solid symbols) developed for embankment soil samples collected during a geotechnical investigation of Saddleback FRS in 2001 (AMEC, 2001). The samples for which gradation curves were developed and are shown on **Figure 2 Appendix B** (designated B-1 @ 4.5-6.0', B-3 @ 2.0-3.0', and #10 @ 4.5-5.5') were collected in an area where cracking has been documented and rehabilitation work has been performed (see Section 1.3.4), making this location critical with respect to material compatibility.

Soil sample B1 @ 4.5-6.0', a sandy clay having the Unified Soil Classification System (USCS) classification of CL, was collected from upstream of the filter/drain. Soil samples B-3 @ 2.0-3.0' (clayey silt, CL-ML) and #10 @ 4.5-5.5' (silty sand, SM) were collected from downstream of the filter/drain.

The base soil gradation curves (solid symbols) were adjusted for gravel content, per NRCS guidelines (NRCS, 1994). The adjusted gradation curves are shown in

Figure 2 Appendix B with open symbols. The filtering and permeability criteria for the adjusted curves are shown by the solid circles on the 15% passing line. These criteria are the basis for developing filter gradation requirements. As can be seen in **Figure 2 Appendix B**, the original design specification band falls within the NRCS permeability and filtering criteria. Therefore, the Zone II filter material is compatible as a protective filter against piping of Zone I materials, assuming placement in accordance with the specified gradation limits.

E. Dam Rehabilitation

In 1995, rehabilitation of the dam was conducted between Stations 44+00 and 52+00 to remediate longitudinal cracking in the dam crest, above the central filter/drain. During rehabilitation work, ADWR (1995) noted that longitudinal cracks along the centerline of the dam crest were between ½-inch and ¾-inch wide and extend entirely through the soil cover over the filter/drain and at some locations extend up to 12 inches into the underlying filter/drain. It was also noted that the upper 6 to 12 inches of drainfill material was contaminated with fine-grained embankment materials (see **Section 3.5** below for further discussion).

The rehabilitation consisted of excavating a 2 ft wide trench along the crest of the dam between Stations 44+00 and 52+00 and backfilling and compacting the excavation with filter/drain material up to the crest. The excavation and manual removal of contaminated drainfill materials continued 1 foot into the uncontaminated material.

The gradation of the drainfill material placed during the rehabilitation was designed to be identical to the originally specified filter/drain material. It was reported in ADWR (1995) that laboratory testing confirmed that at least two of the three soil samples tested during the rehabilitation met the design specification for gradation however, the laboratory results were not available for review during this Phase I Structures Assessment.

3.4.6. Original Slope Stability Analysis. Slope stability analyses were performed in general accordance with SCS guidelines (SCS 1985). The stability analyses utilized dry and saturated embankment soil unit weight values of 122 pounds per cubic foot (pcf) and 132 pcf, respectively. **Table 3 Appendix B** summarizes the shear strength parameter values used by designers for the embankment slope stability analyses. Direct shear testing (CD testing) provided the drained (effective) shear strength parameters and undrained shear strength parameters were determined from the triaxial testing (CU testing). Laboratory reports indicated that seven of the ten direct shear tests and all three of the triaxial tests were performed on remolded samples. The values shown in **Table 3 Appendix B** were adopted by the designers as composite values for the embankment soils based on the results of shear strength data from twelve tests (three CU tests and nine CD tests) of embankment materials (Toups Corporation, 1979).

The slope stability analysis assumed a circular arc failure surface extending through the toe of the embankment slope would have the minimum factor of safety. Slope stability analyses were conducted for this critical slip surface for loading conditions at the end of construction, during rapid reservoir drawdown and for steady-state conditions. Factors of safety were calculated for each of these loading conditions and are summarized on **Table 4 Appendix B**.

Although recommended by SCS (1985), the designers did not conduct a pseudo-static stability analysis of the downstream slope to assess the embankment slope stability under earthquake loading conditions.

A. End of construction

Due to the relatively low embankment height and the placement of materials at below optimum moisture content, pore pressures were not included in the end of construction analysis and the limiting strength values for drained conditions were used in the slope stability analysis (Toups, 1979). The factor of safety computed for the downstream slope on the basis of three trials was 2.3, which is above the allowable minimum value of 1.3 for end of construction loading conditions.

B. Steady-state seepage condition

Slope stability was assessed for steady-state seepage conditions using both drained (effective stress analysis (ESA)) and undrained (total stress analysis (TSA)) shear strength parameters. Factors of safety of 2.9 and 3.3 were calculated for ESA and TSA, respectively.

C. Rapid drawdown condition

For this analysis it was assumed that a full phreatic line could develop on the upstream slope to the elevation of the emergency spillway. Factors of safety were calculated for the upstream slope using both drained and undrained shear strength parameters. The factor of safety calculated using drained strength was 2.1 and the factor of safety calculated using undrained strength was 3.7.

3.4.7. Geotechnical Assessment Recommendations.

A. Phase II Additional Evaluation of Zone II Drain Materials

The compatibility of the embankment materials and the ability of Zone II to adequately act as a filter for Zone I was evaluated for this Phase I Structures Assessment and is discussed in Section 3.4.5.D. No gradation data for the as-built filter materials were available for review, therefore the assessment of filter compatibility was based only on the specified filter gradation with no confirmation that the original filter was built within the specified gradation range. According to ADWR (1995), three filter gradation samples were collected during the dam rehabilitation work (see Section 3.4.5.E) and were within specifications, however these test results were not available for review. It is recommended that filter gradation data be obtained and the compatibility with the filter and embankment materials by confirmed.

B. Phase II Documentation of Slope Stability and Seepage Analyses

Under reasonable loading conditions for Saddleback FRS, it is expected that both upstream and downstream slopes will be stable. However, adequate documentation of slope stability factors of safety for specified loading and design criteria established by appropriate jurisdictional agencies is not available. Additional slope stability analyses are recommended to document the slope stability factors of safety for Saddleback FRS. **Table 5 Appendix B** shows the definitions of various loading conditions and a comparison between the current NRCS design criteria that are outlined in TR-60 (SCS, 1985), and the current criteria as presented in the Arizona Department of Water Resources (ADWR) dam safety rules and regulations for jurisdictional dams.

The original stability analysis does not completely document factors of safety for all the loading conditions required under current NRCS or ADWR criteria. **Table 6 Appendix B** summarizes the results from the original stability analysis and indicates where additional analyses are required.

(1) **End of construction (upstream and downstream slope)**: The original factor of safety calculated for the downstream slope under end of construction loading conditions achieved the minimum ADWR criteria of 1.3 (see **Table 5 Appendix B**). Evaluation of the downstream slope stability under these loading conditions was not performed.

(2) **Rapid drawdown (upstream slope)**: The original stability analysis for this loading condition resulted in calculated factors of safety that are currently acceptable under ADWR rules. Additional analyses are not required.

(3) **Steady state seepage without seismic forces**: The original factor of safety calculated for this loading condition in Reach 2 (1.23) did not achieve the minimum criteria of 1.5 (see **Table 5 Appendix B**). Additional analyses, including confirming the shear strength of embankment soils, either by review of additional data not available during this Phase I Structures Assessment or by field sampling, and reevaluating the critical failure surface on the downstream slope are recommended to document the stability of the downstream slope.

(4) **Steady state seepage, partial pool elevation (upstream slope)**: The original analysis did not evaluate upstream slope stability under this loading condition. The ADWR criteria for partial pool conditions is intended for water retention dams, in which a steady state phreatic line may develop for intermediate pool elevations. The factor of safety may be lower for the intermediate pool conditions than the steady state condition under maximum pool. The following analysis could be done to document the minimum partial pool factor of safety, under the scenario that the outlet works are clogged such that the steady state phreatic line develops:

a. Perform seepage analyses under various partial pool elevations to establish the steady state pore pressure distributions within the dam at each pool elevation.
b. Conduct slope stability analyses for each partial pool seepage analysis result, and graph the results as factor of safety versus pool elevation.
c. Report the minimum factor of safety and corresponding pool elevation.

(5) **Steady state seepage with seismic forces (downstream slope)**: No seismic stability analysis was documented for Saddleback FRS. To document seismic stability under current design criteria, a pseudo-static stability analysis is recommended. The analysis should use a peak ground acceleration (PGA) of 0.1g and the ADWR recommendation of a pseudo-static coefficient equal to 60% of the PGA.

3.5 Longitudinal Cracking and Erosion Holes Along Crest Centerline

Saddleback FRS has experienced the formation of numerous erosion holes and longitudinal cracking along the entire length of centerline of the dam crest. The erosion holes and cracking were first noted and reported in the District inspection report dated March, 1984 (these were noted in the report as "watch for cracking on the dam face"). Longitudinal and transverse cracking and erosion holes have been reported in subsequent inspection reports. During the Phase I inspection of the dam, numerous crest erosion holes and centerline crest cracks were noted and reported (see **Appendix E Field Inspection Report**). The sizes of the holes ranged from small depressions of 6-inches in diameter and 1-inch deep to 2 feet in diameter and the depth undetermined (as the steel probe was only 3 feet long). In some instances it is believed that the top of filter drain could be observed at the bottom of some holes.

A repair program of the longitudinal cracking was initiated by the District with guidance from the NRCS in 1985. The dam crest was plated with 3 to 4-inches of gravel. Subsequent inspection reports for a period of a year or two indicated that dam crest longitudinal cracks were not being expressed through the newly placed gravel plating.

In February 1994, as noted in an SCS memorandum, a site visit was conducted by the SCS with the District to observe longitudinal cracks between Stations 45+00 and 72+00. As a result of the site inspection a repair program of the erosion holes and longitudinal cracks was completed by the District in November, 1995. Under this program, the longitudinal cracks and erosion holes from Station 45+00 to Station 47+15 were repaired. The repair method consisted of excavating a trench two-foot wide along dam centerline approximately 3 to 6 foot in depth. The soil cover was removed as well as the top 1 to 2 feet of drainfill. The trench was then backfilled and hand compacted with new clean drainfill to the top of dam. An Arizona Department of Water Resources memorandum dated December 7, 1995 provides photographs of the trench excavation during the repair of the dam. Several photograph captions and notations by ADWR indicated observations of longitudinal cracks in the trenches that extended deeper than the depth of the trench.

The February 1994 SCS memorandum provided an opinion of the causative mechanism for the formation of the longitudinal cracking and erosion holes over the centerline dam crest. The SCS stated that the cracks were forming as a result of "moisture moving through the foundation and the resulting stress transfer causing a crack at the center of the dam in the relatively brittle material 3 feet above the transition zone. The crack location corresponds/correlates well with the lowest elevation of the dam. This low area is the broad drainage through the valley and would be where a moisture front would first come through the foundation".

Potential Causative Mechanisms for Erosion Hole Formation

As part of the Failure Mode and Effects Analysis (FMEA) for Saddleback FRS, the FMEA team identified a potential failure mode associated with the centerline erosion holes on the dam crest. This potential failure mode is described as:

> "Failure Mode Description: During impoundment, water begins to saturate the upstream embankment soils and migrate through the upstream embankment soils and/or transverse cracks on the slope. The erosion holes (closely aligned), and/or longitudinal cracks that have been observed and documented along the centerline of the crest of the embankment, intersect the flow and provide a conduit to initiate internal erosion. There is uncertainty regarding the connectivity and network of existing erosion beyond that which is easily observed at the crest. However, any connectivity from the observed erosion features, particularly toward the downstream section of the embankment, presents the possibility of eventual seepage on the downstream face, internal erosion through the seepage features, and ultimate failure of the dam by breaching during sustained impoundment events".

This potential failure mode was identified based on observed erosion features that appear to be associated with a defect of the vertical central filter and/or adjacent embankment zones. A potential contributory causative mechanism of erosion hole formation could potentially be due to downward seepage during precipitation events leading to erosion of crest material into voids that are present under the central portion of the dam crest. This process has resulted in a zone of porous, weakened material in the upper 3 to 4 feet of the crest. The FMEA team generally agreed that without further investigation the real cause of the observed distress could not be determined (see below for further discussion of causative mechanisms).

The FMEA team noted adverse factors that may contribute to the likelihood of this failure mode associated with the erosion holes. The team also identified positive factors that may reduce or mitigate the potential for this failure mode. These factors are:

Adverse Factors:

(1) There is evidence of large voids/longitudinal cracking in the crest over substantial length of the dam.

(2) If the void formation is likely or possibly due to a lack of filter compatibility between the filter and the embankment soil a major flood event will accelerate the process.

(3) The potential for transverse cracks exacerbates this failure mode.

(4) There is a higher concentration of erosion holes in the areas where the filter was extended the full depth.

Positive Factors:

(1) Most voids are present only on/near the centerline and may not be present upstream and downstream from the filter.

(2) The erosion voids/longitudinal cracks have only been observed on the dam crest and not on the slopes.

(3) A large storm event (100-year or larger) is necessary to initiate this type of failure mode.

The FMEA team discussed the potential mechanisms for the formation of the erosion holes. The formation of the holes may be due to one or more in combination of the following:

- Suffusion of embankment materials into the filter due to non-compatible filter criteria (internal migration of fines through broadly graded cohesionless drain rock).
- Piping of fine-grained embankment soils by percolating water during precipitation events.
- Settlement of the filter (settlement of the drain itself due to improper or poor compaction at installation).
- Settlement of the embankment adjacent to the filter.
- Rotation of the upstream zone due to collapse of alluvial sediments, particularly in the area of buried paleo-channel areas where tension cracks have been observed on the upstream slope. The rotation of the upstream zone is manifested as a tensile separation between the upstream embankment and vertical filter/drain zones.
- Settlement of the upstream fill due to saturation on inundation.

Recommendations for Further Investigations and Monitoring

The Kimley-Horn team, as part of this Phase I assessment, is recommending the following actions to ascertain the nature of the erosion hole formation, monitoring of the erosion holes, additional geotechnical investigations for filter compatibility, filter material testing, and a potential repair alternatives to be evaluated further.

1. An erosion hole monitoring program should be developed to note the locations and sizes of the holes along the crest. The program should map the locations on a set of as-built plans, obtain GPS coordinates of the erosion holes, and download the locations into the District GIS system. In this manner, the District may monitor, over time, hole locations and sizes, as well as the sections of the dam where erosion hole formation is most prevalent.

2. A field investigation program should be developed to investigate the compatibility of the filter with the embankment soil, the in-place density of the filter, and the current moisture and stress state of the Holocene soils underlying both the upstream and downstream zones of the dam. This would include:

    a. Drilling, sampling, and in-place density testing (standard penetration tests, or SPTs) of the foundation soils at the upstream and downstream toes of the dam; and drilling, sampling and SPTs of the embankment, filter zone and foundation soils from the crest of the dam,

b. Test trenching and bulk sampling along the crest; samples should be collected from both the filter and the embankment zones,

c. Mapping of cracks (transverse and longitudinal) in the test trenches (similar to what was done at Buckeye),

d. Laboratory testing for grain size characteristics, Atterberg limits, consolidation-collapse potential, and in-place moisture content.

3. Conduct high-resolution ground penetrating radar (GPR) surveys across the embankment crest to locate anomalies that may represent erosion hole, cracks or other voids. Excavate selected anomalies to determine what they represent and their possible relationship to erosion holes. Representative test GPR surveys could be run across areas of known erosion holes or void to define their radar signature. This will assist with the characterization of other anomalies that are not coincident with surface expressions of erosion holes or other voids. The information gathered during the GPR surveys can be integrated into the data base generated in Item 1, above.

4. Using the data gathered during the previous site investigations and dam safety inspections, quantitatively and qualitatively identify the sections of the dam to define zones of high, moderate, low, and no concentrations of erosion holes. With this information, develop a phased mitigation program. The phasing of the mitigation program could be a function of available funding starting with the zone of high concentration.

5. Potential Interim and Permanent Repairs for further evaluation as part of a Phase II investigation may include:

   a. Interim measure: Filling the erosion holes with sand and allow sand to migrate into filter. Continue to apply sand until further sand will not be accepted. Cover sand at crest with compacted borrow materials from pool area.

   b. Potential permanent repair: remove the embankment from the crest to one foot below the elevation of the centerline filter. Where the filter is not in-place, remove the embankment to one-foot below the design flood maximum elevation. Rebuild the affected embankment sections and drain to the design crest elevation with engineered compacted fill.

   c. Potential permanent repair: Excavate by trenching entire centerline filter. Install geofabric in trench (to encase drainfill) and backfill trench to 1 foot below crest elevation with clean new drainfill material. Cap trench with compacted borrow materials from pool area.

It should be noted that the December 7 1995 ADWR telephone memorandum (ADWR to FCD regarding the crack repair program) stated that the District stated that the NRCS wanted to build a 150 foot long section of the embankment with one foot of soil cover from the top of the drain to the top of the dam test section. This test section differs from the rest of the rehabilitation section in that the remaining rehabilitation section the drainfill material is carried to the top of the dam. The NRCS's idea is to know if the 150 foot long test section will develop any cracks in the future. The January 1, 1996 letter from the District to ADWR states that the crack rehabilitation work was completed. The District's letter states that the special test section was constructed per NRCS guidelines from Station 44+00 to 45+00. The rehabilitation by the District was completed from Stations 45+00 to 52+50. A review of the as-built rehabilitation plans submitted to ADWR indicates that the as-built station limits do not match the District's December letter. The as-built plans indicate that from Station 45+00 to Station 52+50 that the crest was trenched and backfilled with granular material up to one foot to the top of the dam crest.

3.6 Longitudinal Cracks in Pool Area Upstream of Toe of Dam

AMEC Earth & Environmental, Inc. (AMEC) prepared a geotechnical investigation report titled "Geotechnical Investigation Report Saddleback FRS" (AMEC, April 2001) under contract with the District (FCD Contract 99-46) that documents an investigation of longitudinal cracking in the upstream pool area at Saddleback FRS. The object of the AMEC investigation was to "determine the source, extent and implication of longitudinal cracking located upstream of the FRS at two locations".

The history of the longitudinal cracking at the FRS was noted initially in February 2000. The cracking was noted in the ADWR February 2000 inspection report. In October 2000 the District conducted special inspections to observe longitudinal cracks that had formed in the pool area upstream to the toe berm between Stations 53+50 to 56+00 and Stations 260+00 to 260+50. These station limits occur at the FRS in the areas known as Basin 1 (south basin) and Basin 2 (north basin), respectively. The October 2000 District special inspection reports noted that the conditions at the FRS included inclement rainfall weather resulting in runoff from the watershed impounding water within these low lying basins. The District reports noted water flowing into the cracks.

AMEC, in their report, provided a description of the longitudinal cracking at both basin sites. The crack found at Basin 1 has opened up to 20 inches at the ground surface, and probing indicated it extended to a depth of at least 30 inches. The cracking at Basin 2 was not as extensive as observed in Basin 1. At the time of observations, there was no indication of water exiting from the crack on the surface, on the downstream side of the dam, or within several hundred feet of the downstream toe of the dam at either basin site.

The geotechnical investigation conducted by AMEC included an investigation of the cracking and subsurface explorations. The two sites where the cracking had been observed were investigated with perpendicular crossing trenches excavated to depths ranging from 7.0 feet to 8.0 feet using a backhoe. The subsurface exploration of Saddleback FRS site was conducted in December 2000 and January 2001. The investigation included exploration of the geotechnical conditions in and adjacent to impoundment areas along the FRS where cracks were observed. The field exploration included sampling of soils using continuous sampling techniques in borings and hand-sampling in backhoe-dug test pits. The exploration included performing in-situ density tests (nuclear method) along the upstream and downstream faces of the FRS at two locations adjacent to the areas of cracking. AMEC conducted laboratory testing on selected samples including grain size analysis, Atterberg limits, moisture-density relationship (standard Proctor), and consolidation test. The AMEC report includes the results of the exploration and testing. AMEC also conducted a dam stability analysis and determined factor of safety values.

AMEC states that in their opinion the "cracking adjacent to the Saddleback FRS from Stations 52+00 to 57+50 and Stations 260+00 to 270+50 developed as a result of collapse or settlement of the native soils in these areas. Because the areas of settlement were localized, and adjacent areas downstream beneath the structure were not impacted, the differential settlement resulted in horizontal strains that exceeded the threshold of cracking for the soils".

AMEC continues to state that "based on the site topography the two low-lying areas in the impoundment are coincident with the cracking that has formed. This leaves the soils in the immediate area susceptible to ponding and therefore susceptible to the breakdown of weak interparticle calcareous cementation bonds, or soil structure. The breakdown of the soil structure allows consolidation to occur and permit the development of tensile stresses in the soil mass when differential rates of consolidation occur. Cracking due to tensile stresses may form in areas where shallow subsidence has occurred. Once cracking has begun, erosion from water contributes to the development of cracking by furthering the breakdown of physical and chemical bonds between soil particles".

The AMEC reports provides a recommended mitigation measure to in response to the longitudinal cracking. This measure includes the identification of open cracks and "backfilling with sand, such as ASTM C-33 sand. The sand backfill will act to limit seepage and tend to decrease the erosion of the crack walls. The sand should be placed in the features and compacted using a vibratory compactor. Sufficient water should be used to facilitate compaction and to move the sand into any near-surface voids. A native soil layer having a minimum thickness of 6-inches should then be placed above the sand fill.

Finally, the AMEC report, based on the results of the investigation and dam stability analysis, states that the "dam overall appears to be stable". AMEC recommends that a crack monitoring program and repair program be initiated and maintained. They suggest the frequency of monitoring should be at least yearly, with monitoring also being conducted after the occurrence of rainfall events on the order of 1 inch or greater. Kimley-Horn concurs with this inspection and monitoring recommendation.

3.7 Construction History

The Soil Conservation Service (SCS) contracted with a private engineering firm (Sargent, Hauskins, and Beckwith) for all quality control, inspection, and construction supervision for the project. Construction of the dam was by M & B Contracting Corporation.

Quality control was in accordance with specifications, but records of tests were limited in nature. There were no unusual problems (see next paragraph) associated with construction and all work was completed and accepted on June 14, 1982. The as-built plans are dated May 3, 1982.

The FMEA team reviewed construction photographs provided by the NRCS. The photographs provided some insight on the construction of the central filter for Saddleback FRS. The central filter was brought up in lifts along with the dam embankment. It appeared, based on the photographs, that the earth equipment used to place the materials ran the rubber tires over the filter/embankment interface such that the embankment materials were somewhat blended into the filter material by the tires of the trucks (belly dumps). This blending may a contributing factor in the causative nature of the multiple erosion holes appearing on the dam crest surface along centerline.

3.8 Utilities

There are several utilities that were relocated prior to dam construction at Salome Road. Sheet 20 of the as-built plans provided the following note (Note 2) that "all overhead and underground utility relocations have been accomplished by the respective utility companies". A review of the as-built plan sheets 20 and 41 indicate at the time prior to construction the following utilities were relocated: American Telephone & Telegraph Blythe 'A' Cable and 4-inch PVC pipe (noted as empty); Arizona Telephone Cable; Arizona Public Service (APS) overhead 69-kv powerline; and a 1 ¼ inch Arizona Public Service gasline. The APS overhead powerline is located west of the Salome Road crossing of the dm. The as-built plans indicate that the line spans the dam completely and has a 34-foot clearance from the catenary of the line to the top of dam crest elevation. The AT&T cable and the Arizona Telephone cable were relocated under the dam foundation. As-built sheet 41 provides a detail of the relocation plan and section of these two underground utilities. Both utilities pass under the dam cut-off foundation with a clearance of approximately 5-feet. The utilities have cut off collars and have a 6-inch concrete cap over the lines for the full length of bury under the dam and floodway. The APS buried gasline in Salome Road was capped and plugged west of the dam. The remaining abandoned gasline was removed by the contractor, according to Sheet 20.

A buried fiber optic cable was recently constructed across the dam crest at the Salome Road crossing. The cable was crossing was designed by Copperstate Engineering. The cable was buried in trench two feet below grade in the north shoulder of the Salome Road crossing of the dam. The cable consists of six 1-½ inch diameter HDPE pipes. The cable was concrete encased in the limits over the dam. The cable trench was excavated below top of dam elevation but approximately two feet above the central filter (approximate top of central filter is 1191.0 ft). The cable was bored under the Saddleback floodway. The cable is located in a 10-foot wide utility easement. Pivotal Communications owns the cable line. An FCD Right of way use permit was secured for the construction of the fiber optic cable (Permit number 2001P084 dated May 8, 2003).

3.9 Emergency Action Plan

At this time the Flood Control District does not have an individual emergency action plan (EAP) for Saddleback FRS. The District is currently developing EAPs for all of their dams in their inventory. The District has completed several EAPs for their dams (e.g.: Guadalupe FRS, White Tanks FRS No.3) that meet the minimum requirements published in the Federal Emergency Management Agency guidelines FEMA 64 Emergency Action Planning for Dam Owners (FEMA, October 1998). The EAPs provide an EAP flowchart based on percent reservoir impoundment on reservoir filling. The preparation of an individual EAP for Saddleback FRS is tentatively scheduled for the last half of the 2005/2006 fiscal year. Kimley-Horn recommends that the individual EAP for Saddleback FRS also meet or exceed the minimum guidelines for EAPs for jurisdictional dams set forth in the Arizona Department of Water Resources, Office of Water Management. The development of the EAP should be coordinated with the Maricopa County Department of Emergency Management.

EAPs provide downstream inundation mapping for spillway discharges as well as from potential dambreaks. The District has completed both a dambreak study and spillway inundation study for Saddleback FRS. A discussion of these studies was provided above in 3.3.2 and 3.3.3.

The spillway and dambreak inundation mapping for Saddleback FRS is provided in **Figure 2** and **Figure 3** in **Appendix Figures,** respectively. Note that the dambreak mapping does not having shading (that indicates areas of potential inundation) in between the north and south dambreak locations. Since a dambreak could potentially occur anywhere along Saddleback FRS, Kimley-Horn recommends that the inundation map be revised to reflect this possibility.

The Maricopa County Department of Emergency Management has an Emergency Operation Plan (McDEM, 2003) that outlines the procedures and duties of various agencies which are activated in emergency flood situations. Saddleback FRS is included the McDEM Plan in Annex I, Appendix 12. The inundation mapping included in the EOP only includes mapping for a potential dambreak. It does not include downstream inundation mapping for large discharges from the emergency spillway.

The District has prepared a Flood Emergency Response Manual (FERM) (FCD, January 2002) that presents the most current duties for District personnel during significant rainfall events and/or flood emergencies. The FERM indicates that District personnel will be sent to observe the dam during flood emergencies or when weather conditions merit observation. The manual states that the District Operation and Maintenance Division will be notified at an impoundment depth of 6.5 feet. In addition, McDEM would be notified by the District at an impoundment depth of 9.5 feet (3.0 foot difference).

The notification levels from the FERM and the Emergency Operation Plan are presented in the **Table 10** provided in the **Tables Appendix**. The table shows a discrepancy in the notification levels in the two plans. The District should endeavor to correct the notification level for McDEM to be consistent between plans. The time to fill the reservoir at various percent full levels (10, 25, 50, 75, 90, and 100%) should also be evaluated. In this fashion the time to fill from one pool level to the next pool levels may also be determined. The time to fill the Saddleback FRS pool from a percent level to the next will be helpful in decision making in updating response and alerts. For example, the time to fill from 25% full to 50% full would be helpful since the level of pool change is only 3.0 feet at which the trigger to notify McDEM occurs.

4.0 PRELIMINARY FAILURE MODES

Kimley-Horn and Associates, Inc. (KHA) facilitated a Preliminary Failure Modes Identification workshop for Saddleback FRS. The workshop was conducted on January 20, 2005. The overall objective of the workshop was to develop a comprehensive list of potential failure modes for the structure and appurtenances. The purpose of the workshop was to:

- Develop a list of potential failure modes for the structure and appurtenances,
- Identify key issues that require additional review or assessment during the structure assessment or field inspections,
- Discuss/identify field evidence for precursors for potential failure modes, and,
- Provide a baseline for detailed Failure Mode and Effects Analysis.

The workshop was conducted at the offices of Kimley-Horn and Associates, Inc. The following individuals participated in the workshop:

Tom Renckly, P.E.	Flood Control District
Brett Howey, P.E.	Flood Control District
John Chua, P.E.	Natural Resources Conservation Service
Bob Eichinger, P.E., CFM	Kimley-Horn and Associates, Inc.
Dean Durkee, Ph.D, P.E.	Gannett Fleming, Inc.
Ken Euge, R.G.	Geological Consultants, Inc.

The workshop participants identified key issues that would require additional review or assessment during the Structure Assessment and field inspections. A detailed Failure Modes and Effect Analysis (FMEA) will be conducted subsequent to this Preliminary Failure Modes Workshop. The main potential failure modes and items reviewed during the Preliminary Failure Mode Workshop are as follows:

1. **Embankment Overtopping**: The embankment crest is gravel plated and is therefore provided with a measure of erosion protection. The upstream and downstream slopes in the western portion of the dam are not provided with gravel-mulch erosion protection. The upstream and downstream slopes of the eastern portion of the dam are provided with a measure of erosion protection (whether by design or not remains to be assessed). Overtopping of the dam crest embankment could lead to erosion and formation of a breach.

2. **Emergency Spillway Discharges**: This pertains not only to downstream impacts due to failure of one of more components of the dam, but impacts that would result from normal operations at the facility.

3. **Failure of Combined Principal Outlet/Emergency Spillway**: The discharge outlet for Saddleback FRS is a combined principal outlet/emergency spillway that

is a reinforced concrete box culvert (10-ft span by 8-ft rise). The outlet is located at the southern end of the dam and discharges into the Saddleback Diversion Channel. The outlet is not provided with a trash rack.

4. **Piping Involving Foundation and Abutments**: Relates to potential piping erosion of soil materials from the embankment fill into the foundation and/or developing through the foundation under the embankment.

5. **Internal Erosion and Piping through the Embankment**: This failure mode relates to the internal erosion along a transverse crack, or along a penetration through the dam (outlet pipes and utility conduits).

6. **Slope Stability**: This failure mode covers both the upstream and downstream slopes of the embankment.

7. **Failure Mechanisms Associated with Presence of Collapsible Soils in Dam Foundation:** This failure mode relates to the potential for collapse on saturation of meta-stable soils in the dam foundation. Geologic mapping/boring logs/laboratory test data will be reviewed to assess to the extent practical the presence of potentially collapsible materials.

8. **Failure Mechanisms Associated with Earth Fissures**: Previous as well as current investigations by others have identified a strong potential for earth fissures at a number of FCD structures.

9. **Failure Mechanisms Associated with Filter/Drain Pipe**. The filter drain incorporates a drain pipe to collect seepage water. There may be a potential for failure of the drain pipe system by either clogging or structural failure by collapse.

10. **Failure Mechanisms Associated with Longitudinal Crack**. This failure mode relates to internal erosion along a longitudinal crack expressed through the crest of the dam into the central filter.

11. **Failure Mechanisms Associated with Seismic Event**. A seismic event in the vicinity of Saddleback FRS has the potential for exacerbating existing transverse/longitudinal cracks and forms a causative or additive mechanism for central filter collapse.

12. **Other considerations**: This section addresses issues that are not directly related to a failure of the dam or its appurtenant facilities, but which nonetheless may be relevant to the FMEA:
    - Foundation treatment
    - Compaction
    - Use of construction materials (borrow areas)
    - Placement of embankment lifts
    - Filter gradation and outlet drain gradation

A detailed report of the Preliminary Failure Mode Workshop is presented in **Appendix D**.

5.0 Land Ownership and Land Use

This section discusses data on the existing and future land use upstream and downstream of Saddleback FRS. Land use information for Saddleback FRS was collected to allow a qualitative assessment of the consequence of dam failure and/or spillway inundation flood events. The scope of the study required review of 2 miles upstream and downstream of the dam.

5.1 Source of Data

The Flood Control District of Maricopa County provided aerial photography, information regarding dam pools and flood retention structures, and land ownership and land use information. **Figure 5 (Figures Appendix)** provides a map demonstrating land ownership at Saddleback FRS.

5.2 Description of Land Use Categories

The main categories inventoried for land use included residential, commercial, educational facilities; public facilities, active open space, and mixed use (see **Figures 6 and 7** in the **Figures Appendix**). These categories are described briefly below:

- *Residential* land uses include developing residential, large lot residential, estate residential, rural residential, very small lot residential and medium residential.
- *Commercial* land uses include retail establishments, office buildings, hotels, light industrial and warehouses.
- *Agriculture* land use includes farming, grazing, and growing of seasonal crops. Land is typically tilled and laser-leveled for flood irrigation.
- *Public Facilities* include community centers, power sub-stations, libraries, city halls, police/fire stations, and other government facilities).
- *Educational* land uses include public schools, private school and universities.

5.3 Existing Land Use

Existing land uses in the study area generally are characterized as active open space, agriculture, residential, commercial, or as public facilities. This information is depicted on **Figure 6 (Figures Appendix)** and is summarized as follow:

- Interstate 10 is a major road through the project area and contains a large portion of land designated as open space and residential. This road is located approximately 0.5 miles upstream of Saddleback FRS and runs perpendicular to the dam.
- Major agriculture and irrigation canals are located south of Interstate 10.
- There is a power generation station (Allen Generating Station) located at 491st Avenue and Thomas Road.
- No new residential development was recorded near this dam at this time. However, new single family lots are being developed.

5.4 Proposed Land Use

Future land use plans were obtained through the District. The major significant change is that the agriculture and vacant lands are shown as single family residential (see **Figure 7** in **Figures Appendix**). The residential land use change is shown to completely encompass Saddleback FRS. This exhibit illustrates a trend from converting open and vacant space into more intense land use categories.

5.5 Current Property Values

Appendix G provides an inventory of parcels located with approximately two miles of Saddleback FRS and the current full cash value of those properties.

5.6 Population Densities

Appendix G also provides four maps illustrating the change in population densities from the year 2000, to 2010, 2020, and 2030.

5.7 Critical Facilities

Critical facilities exist within approximately a 2-mile radius from the Saddleback FRS. These facilities include the Harquahala Generation Station, the Central Arizona Project canal, and Interstate 10. Only the generation station is located downstream of the dam.

6.0 FIELD INSPECTION

6.1 Previous Inspections

Kimley-Horn reviewed previous field inspection reports for Saddleback FRS from project files at the Flood Control District and Arizona Department of Water Resources. The reports collected from these sources date to July 20, 1982. A total of 30 data sources and inspection reports from July 1982 to November 2004 were reviewed as part of this task. A summary of the more recent inspections from March 1998 to November 2004 are provided in **Appendix E**. These inspection reports were summarized due to the greater detail of recorded observations.

Major findings documented from the above mentioned data sources and field inspection reports (Date of report followed by noted highlights of report) include the following:

- April 7, 1982: Dam construction complete
- June 14, 1982: Soil Conservation Service acceptance of dam.
- July 20, 1982: ADWR letter to District granting temporary permission to store water in the structure.
- March 22, 1983: FCD report no noted signs of distress.
- March 21, 1984: FCD reported "watch cracking on face".
- March 21, 1984: ADWR report stated longitudinal cracks at Station 49+00 through Station 52+00 over dam centerline.
- November 14, 1984: ADWR report noted many cracks. The report stated "the main longitudinal crack at the approximate center runs for about 90% of the embankment length". ADWR, FCD, and the NRCS met to select and inspect a few of the longitudinal and transverse cracks for investigation.
- November 28 and 28, 1984: ADWR report stated a joint inspection was conducted with NRCS and FCD to observe four test trenches on crest of dam. Concluded cracks need to be repaired.
- March 20, 1985: ADWR report stated the centerline longitudinal crack continues to nearly the full length of the dam. The report noted a meeting held on April 17, 1985 between SCS, FCD, and ADWR that it was decided that the repair of the longitudinal crack would be to spread 3-inches to 4-inches thick layer of aggregate material on the crest of the dam.
- April 9, 1986: ADWR report notes that the dam crest was gravel plated with 3-inche to 4-inche thick layer of fine to medium gravel during September 1985. Cracks not observed to due gravel plating. Granted permanent license of approval.
- July 7, 1986: ADWR granted License of Approval dated July 17, 1986.
- March 25, 1993: FCD report noted to monitor the settlement and cracking that exists between Stations 45+00 and 50+00.
- March 25, 1993: ADWR report notes longitudinal cracks were observed between Stations 45+00 to 50+00.

- March 22, 1995: FCD report stated that the settlement area (Stations 45+00 to Station 50+00) appears to be more prominent. FCD to schedule a backhoe for exploration.
- April 3, 1996: FCD report stated that the settlement area (Station 44+00 to 52+00) was recently excavated to depth of 4-ft to 6-ft and additional core drain material placed in trench in accordance to ADWR and NRCS specifications.
- March 25, 1997: FCD report states that the crest area the was restored last year (1996) appears to be stable with no signs of any settlement.
- March 17, 1998: ADWR report noted a series of rodent holes along the centerline crest.
- March 30, 1999: FCD report states that FCD crews have filled in the majority of the holes associated with the cracking.
- Date of current ADWR License April 7, 2000
- February 2, 2000: ADWR report noted longitudinal cracks and erosion holes throughout the centerline crest. The report noted that the NRCS stated previously that "the cracking is not a problem" and "a result of moisture moving through the foundation and the resulting stress transfer causing a crack at the center of the dam in the relatively brittle material three feet above the transition zone". The ADWR report noted the types of repairs done by NRCS and by FCD. The "NRCS appears stable (Station 44+00 to 55+00)" and the FCD area still contains cracking and holes.
- February 2, 2000: FCD report states "holes on dam crest for full length of dam and are located over central filter drain. From Stations 44+00 to 55+00 virtually no groundcover in the area where NRCS repaired central filter/drain in 1996. Reservoir: longitudinal crack Station 53+00 to 56+00 up to 3-inches wide and 3 feet deep aligned along protective berm. Another longitudinal crack on reservoir side at protective berm from Station 260+00 to 260+50.
- January 2003 report. Same comment regarding holes on crest of dam for full length as stated in February 2000 report. Transverse cracks located in many areas on the upstream and downstream slopes throughout length of dam. Longitudinal cracks noted in previous inspections – many not found, however some were noted at downstream toe.
- January 13, 2004 report. Same comment regarding holes on crest of dam for full length as stated in February 2000 report. Many notations of transverse and longitudinal cracks on crest, upstream and downstream slopes.
- November 15, 2004 report. Same comment regarding holes on crest of dam for full length as stated in February 2000 report. Many notations of transverse and longitudinal cracks on crest, upstream and downstream slopes.

6.2 Field Inspection for Structure Assessment

The purpose of the field examination is to provide a systematic visual field technical review in which the structural stability and operational adequacy of the dam project features are reviewed and evaluated to determine if deficiencies exist at the dam and associated project features. The examination was conducted by walking the length of the structure and visually examining the crest, upstream and downstream slopes, upstream and downstream toes, and appurtenant structures. Comments are recorded in an inspection log and photographs taken of pertinent observations. Cracks, holes, and burrows were probed with hand-held 3-foot stainless steel metal rod/probes to examine depth, extent, and resistance to probing. No other intrusive/internal examination method was used during this examination.

The field examination of the structure is accomplished to provide a basis for timely initiation of any corrective measures to be taken where necessary. This examination was conducted on February 2, 2005 by the following technical examination team:

Technical Examination Team

Tom Renckly, P.E.	Structures Branch Manager, Flood Control District of Maricopa County
Brett Howey, P.E.	Dam Safety Engineer, Flood Control District of Maricopa County
Earl Pearcy	Operation and Management, Flood Control District of Maricopa County
John Harrington, P.E.	Engineer, Natural Resources Conservation Service
Robert Eichinger, P.E., CFM	Project Manager, Kimley-Horn and Associates
Ken Euge, R.G.	Principal Geologist, Geological Consultants
Dean Durkee, Ph.D., P.E	Principal Geotechnical Engineer, Gannett-Fleming
David Jensen, P.E.	Engineer, Kimley-Horn and Associates
Kelli Blanchard, E.I.T.	Hydrologist, Kimley-Horn and Associates

Operational Summary

Inspection Frequency: Saddleback Flood Retarding Structure (FRS) is inspected jointly on an annual basis by the Arizona Department of Water Resources (ADWR) and the Flood Control District of Maricopa County (District). The NRCS is invited to participate in annual inspections of Saddleback FRS.

Maximum Water Surface Elevations: The maximum recorded impoundment for Saddleback FRS was on July 7, 1996. The impoundment was recorded at 2.5-feet which is approximately 14.5-feet below the principal spillway crest elevation of 1176.9-feet (as-built; NGVD29 datum).

Emergency Spillway Discharge: There is no separate emergency spillway. The principal spillway is actually a combined principal/emergency spillway.

Distress Observations Corrected or Operation and Maintenance Conducted Since Last Inspection: None were noted. The District has an operation and maintenance program in place in which they continually monitor for rodent activity and vegetation on the dam.

Past Distress Observations Not Yet Corrected: (Maintenance and corrective measures identified in the November 2004 Inspection Report were placed on hold pending completion of the Phase I Individual Structures Assessment)*

- Update emergency action plan (scheduled for fall 2005);
- Fill erosion rills on upstream and downstream slopes with compacted fill, if greater than 12-inches deep;
- Initiate gravel mulch recommendations resulting from the Phase I Structures Assessment;
- Control and repair damage caused by rodent activity;
- Repair longitudinal crack near the upstream toe between Sta. 250+91 to 262+20, 72+14 to 76+34, and Sta. 50+72 to 54+00 utilizing the same procedures utilized during the repair of similar cracking between Station 53+00 and 56+00;
- Restore crest at Salome Road (coordinate with MCDOT);
- Replace toe settlement monument B-25 at Station 250+00

* These measures were taken from the November 2004 Inspection Report.

District Operation and Maintenance Responsibilities: The District maintains operational control of the Saddleback FRS and is responsible for the structural and functional integrity of the FRS and appurtenant features, maintaining the emergency spillway, erosion control of the embankments, and landscaping. The District is responsible for implementation of the emergency action plan. The District conducts quarterly O&M inspections of Saddleback FRS.

Field Examination Results Summary

Embankment Crest: The crest of the dam is gravel plated. All crest settlement monuments located on the crest were located. There are station markers on the dam. The crest is clear of vegetation. The access gates and fences are operational. Longitudinal cracks/transverse cracks, depressions, and many erosion holes were observed on the crest of the dam (see inspection report for specific locations).

Abutments: The left and right abutment contacts appear in satisfactory operational condition. No slides, sign of instability or erosion of the abutment surfaces were observed. Abutment groins were clear of vegetation.

Upstream Slope: There are several small animal burrows scattered on the slope face. There was no evidence of seepage, undermining, settlement or sloughing. Possible transverse cracks were noted at many locations along the dam on the upstream slope. Recommendations for gravel mulching of the upstream slope will be provided during the Phase I Structures Assessment currently being completed by Kimley-Horn.

Downstream Slope: There are several small animal burrows scattered on the slope face. There was no evidence of seepage, undermining, settlement or sloughing. Possible transverse cracks were noted at many locations along the dam on the downstream slope.

Recommendations for gravel mulching of the upstream slope will be provided during the Phase I Structures Assessment currently being completed by Kimley-Horn.

Toe drain outlets are located approximately every 400 feet along the length of the dam. The toe drains extend from the central filter to the downstream toe of the embankment. Two of these toe drains, at Station 248+00 and 204+00, could not be located during the inspection. Many of the toe drain outlet conduits are partially full of sediment at the very downstream end of the conduit. It is not known if the material in the conduits is from the central filter itself or if it is from ponding at the outlets.

Principal Spillway and Reservoir: The approach channel is clear of debris and obstructions. The wing walls of the inlet structure have minor shrinkage and temperature cracks, no repairs required. The interior of the principal spillway conduit was inspected visually. The conduit was clean and there were no apparent signs of seepage.

The discharge outlet structure of the principal spillway was clear of debris. The joints of the outlet structure were straight and appeared tight. There were no signs of seepage.

Vegetation clearing operations were underway for the upstream floodway.

Emergency Spillway: There is no separate emergency spillway.

Irrigation Outlets: The irrigation outlets were located.

Instrumentation: The settlement monuments for Saddleback FRS are located on the downstream side of the dam crest at grade and near the downstream toe. Settlement monuments are marked with sign posts. Settlement monuments located on the crest are noted with an A, settlement monuments located near the toe are noted with a B. Monument B25 at Station 250+00 has been damaged and should be replaced. There are station markers on the Saddleback FRS.

There are no rain or stream gages in the watershed. There is an alert gage at the principal outlet. These instruments help provide an early warning system and should be incorporated into an emergency action plan for the Saddleback FRS.

There is a staff gage at the principal outlet located on the upstream slope. The staff gage is used to indicate the level of water impounded in the reservoir. A pressure transducer is also located at the principal outlet. No staff gage is located anywhere else on the dam embankment. A staff gage is recommended at Solome Road.

6.3 Signs of Distress

Based on the field inspection performed by the Kimley-Horn team, previous inspection reports by ADWR and the District and the results of FMEA for the FRS, major signs of distress in the form of confirmed transverse and longitudinal cracking have been identified at Saddleback FRS. Erosion holes on the dam crest are located over the central filter for the full length of the dam.

6.4 Safety Deficiencies

Based on the field inspection performed by the Kimley-Horn team, previous inspection reports by ADWR and the District and the results of FMEA for the FRS, no safety deficiencies have been identified for Saddleback FRS. An EAP for Saddleback FRS needs to be prepared and developed to meet the minimum guidelines from ADWR and FEMA.

6.5 Conclusions

The overall conclusion of the field examination is that the Saddleback FRS and appurtenant structures are in a satisfactory operational condition.

6.6 Recommendations From Inspection

The following is a list of recommended corrective actions resulting from this field examination:

a. Add watershed instrumentation (stream and rain gages);
b. Develop an Emergency Action Plan to meet FEMA 64 and ADWR requirements;
c. Develop a repair plan for erosion holes on the crest of the dam;
d. Evaluate gravel mulching of embankment slopes;
e. Evaluate effect of Solome Road over crest of dam;
f. Locate the toe drains at Station 204+00 and 248+00;
g. Determine source of material deposited in the toe drain outlets.
h. Due to the length of the structure (5 miles), an additional staff gage should be added. A recommended location would be at the Solome Road crossing of the embankment.
i. Map all cracks on set of as-built plans and profiles as well as aerial photo of dam. Continue to map cracks after all dam safety inspections. Enter GPS coordinate crack location into District HIS system. Monitor, over time, reaches of dam where there has been a noted propensity of cracks.

7.0 FAILURE MODES AND EFFECTS ANALYSIS

7.1 Introduction

Kimley-Horn and Associates, Inc. and the FMEA team conducted a failure modes and effects analysis for Saddleback FRS. The FMEA is a qualitative risk-based procedure that can be usefully applied to any engineered system, especially for those with complex components or component interactions. The FMEA relies on the collective engineering judgment of experience professionals in a workshop setting to describe potential failure modes, the likelihood of that potential failure mode, and the potential consequences resulting from the failure.

The workshop was conducted on March 1, 2005. The workshop participants included:

Tom Renckly, P.E., Flood Control District of Maricopa County, Project Manager,
Brett Howey, P.E., Flood Control District of Maricopa County, Dam Safety Engineer
Bob Eichinger, P.E., CFM, Kimley-Horn and Associates, Inc., Project Manager
David Jensen, P.E. Kimley-Horn and Associates, Inc, Hydrology & Session Recorder
Debora J. Miller, Ph.D., P.E., Gannett Fleming, Inc., FMEA Facilitator
Dean B. Durkee, Ph.D., P.E., Gannett Fleming, Inc., Geotechnical Engineer
Ken Euge, R.G., Geological Consultants, Geology
John Harrington, P.E., Natural Resources Conservation Service
Dan Lawrence, P.E., Flood Control District of Maricopa County, Dam Safety Engineer

The detailed report for the Failure Mode and Effects Analysis Report is provided in **Appendix F**. The FMEA report was reviewed the FMEA team.

The purpose and scope of the FMEA exercise was to:

- Identify potential site-specific failure modes for the dam.
- Discuss qualitatively the likelihood of the occurrence of potential failure modes.
- Determine whether or not, and how, important the potential failure mechanisms are being monitored.
- Examine the potential consequences of failure and the adverse consequences of successful operation during flood loading (e.g. – large spillway releases).
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences.
- Determine what information, investigations or analyses may be needed to resolve uncertainties relative to potential failure modes.

7.2 FEMA Procedure

The FMEA workshop was conducted in the following steps:

- Define the System: This process involves developing a detailed description of the dam system and its components. This is an important step in understanding how the system components operate and relate and how the components or system may fail.

- Define System Potential Failure: Typically, failure of a dam is defined as the uncontrolled release of the reservoir. This definition was modified to include emergency spillway discharges during normal operations of the facility.
- Define Likelihood and Consequence Categories: The likelihood of consequences of potential failure was divided into three broad categories: low, medium, and high.
- Identify Potential Failure Modes: This step involves examining each component in detail to identify the ways in which it might cause a system failure.
- Evaluate Failure Modes: A likelihood and consequence category was assigned to each potential Class I or Class II failure mode.
- Binning: A two-dimensional array/matrix was used to "combine" the likelihood and consequence to obtain the relative risk associated with each potential Class I and Class II failure mode.
- Documentation: The results of the FMEA were documented in a detailed report prepared by Gannett Fleming Inc. and reviewed by the FMEA team. The detailed report is included in **Appendix F**.

7.3 FMEA Results

The FMEA for Saddleback FRS did not identify any potential failure modes with a high likelihood and high consequence. The following Category I and Category II failure modes were assigned a low likelihood of occurrence and a high consequence to a high likelihood and low consequence:

H1. Overtopping During Major Flood Event (Category I).

Failure Mode Description: Saddleback FRS does not have a separate emergency spillway. Flows up to the ½ PMF are routed through the combined principal/emergency spillway to the Saddleback Diversion channel. The maximum water surface elevation for the ½ PMF is 1193.0. The lower portion of the dam crest is 1193.0. It is likely that the PMF would overtop the FRS. A PMF storm event on the watershed may possibly fail the CAP upstream dike increasing the overtopping potential at Saddleback FRS.

S1a. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks under the Filter (Category II).

Failure Mode Description: A transverse crack extends through the embankment and into the foundation soils. In areas where the filter was not extended through the embankment and into the foundation, the crack could conceivably extend beyond the filter in which case this lower "unprotected" section of the embankment is susceptible to internal erosion during impoundment events. Sustained or intermittent flows through the crack below the filter could initiate the process of internal erosion and, over time, could result in widening of the crack and subsequent settlement of filter and embankment material into the void.

As this process continues and the widened crack continues to migrate upward under sustained reservoir head, a breach of the embankment is conceivable.

S1b. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks that extend above the Filter (Category II).

Failure Mode Description: A transverse crack extends into the embankment some nominal depth. The top of the filter terminates at Elev. 1190.0 feet and the crest of the dam varies between Elev. 1193.0 and 1194.0 which results in 3 to 4 feet of "unprotected" embankment above the filter which is susceptible to internal erosion during impoundment events. Sustained or intermittent flows through the crack could initiate the process of internal erosion, and, over time, could result in widening and deepening of the crack. As this process continues and the widened crack continues to migrate downward under sustained reservoir head, a breach of the embankment is conceivable.

S1c. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks that extend through the Filter (Category II).

Failure Mode Description: A transverse crack extends from the crest of the embankment downward and into the embankment and fully through the filter in the transverse direction. During impoundment, flow develops through the transverse crack and initiates the process of internal erosion of upstream embankment material which can then be transported through the crack in the filter. Assuming the crack in the filter is wide enough and not "self-healing" this process could result in widening and deepening of the crack both in the embankment (upstream and downstream sections) and in the filter itself. As this process continues and the widened crack continues to migrate downward under sustained reservoir head, a breach of the embankment is conceivable.

S3. Internal Erosion Leading to a Breach in the Upper Embankment due to the Eroded Character of the Crest (Category II).

Failure Mode Description: During impoundment, water begins to saturate the upstream embankment soils and migrate through the upstream embankment soils and/or transverse cracks on the slope. The erosion holes (closely aligned), and/or longitudinal cracks that have been observed and documented along the centerline of the crest of the embankment, intersect the flow and provide a conduit to initiate internal erosion. There is uncertainty regarding the connectivity and network of existing erosion beyond that which is easily observed at the crest. However, any connectivity from the observed erosion features, particularly toward the downstream section of the embankment, presents the possibility of eventual seepage on the downstream face, internal erosion through the seepage features, and ultimate failure of the dam by breaching during sustained impoundment events.

7.4 FMEA Limitations

It is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

7.5 FMEA Special Study Task 1: Gravel Mulch Erosion Protection Of District Earth Embankment Dams

As part of the FMEA work session for Harquahala and Saddleback FRS, the Kimley-Horn team (Kimley-Horn, Gannett Fleming, and Geological Consultants) and the District personnel discussed an approach to evaluate the use of gravel mulch for erosion protection on embankment slopes of the District's inventory of dams. The form of erosion protection discussed was gravel mulch. The specific task undertaken by the FMEA team was defined in the Work Assignment No. 3 scope of work as follows:

"Recent O&M practice by the District to provide for erosion protection on the slopes of certain Flood Control Dams has been to place gravel mulch on the slopes of dams that have not exhibited transverse cracking. The gravel mulch treatment using a "gravel shooter" has proven to be both efficient and effective in controlling embankment slope erosion while allowing for vegetative growth on the dams. There is a concern by District Dam Safety Engineers that placing gravel mulch on embankment slopes may tend to "mask" certain surface anomalies at transverse cracks such as erosion holes that have been used in the past as an indicator of potential site specific dam safety issues that requires further investigation. Therefore the task of the FMEA team will be to evaluate and provide recommendations to the District on this issue which address both dam safety concerns and the need for erosion protection and erosion repairs at District dams that exhibit both slope erosion and transverse cracking. The FMEA team may find it necessary to make specific recommendations on this issue on a dam by dam basis. The Consultant will be provided a copy of the District's "Recommended Gravel Mulch Priorities" list".

The discussion of erosion protection through gravel mulching centered on several points of discussion as follows:

1. The discussion presented the advantages and disadvantages of gravel mulching the slopes of the dams.

2. The discussion focused on whether or not to gravel mulch embankment dams that exhibit signs of transverse cracks or are known to have transverse cracking (this is the primary District concern at the time of this FMEA special session).
3. Third, design criteria and considerations was presented for gravel mulch.

A summary of this discussion is presented below. A recommendation regarding gravel mulch on District dams is then provided afterwards.

A. Background Information

Many of the District's flood retarding structures were not provided with slope erosion protection during original design or construction. Some structures (e.g. Spook Hill FRS) were hydroseeded after construction to establish a vegetation layer as erosion protection.

Many of the District's structures have experienced transverse and longitudinal cracking since original construction. The NRCS and others over the several past decades have identified several crack-forming mechanisms, including desiccation/shrinkage cracking (especially near the crests of the embankments), differential settlement cracking caused by collapse settlement of moisture-sensitive (metastable) foundation soils under the upstream zone during impoundment events, or by sharp transitions in the foundation profile under these long structures.

Several structures have been rehabilitated by constructing central filters to act as crack stoppers. For other structures, the central filter was installed as part of original construction. However, no all structures have a central filter zone.

The District has recently initiated the placing of gravel mulch on the slopes of several of their structures. Gravel mulch has been placed on Sunset FRS, Casandro Wash Dam, and Sunnycove FRS and placed on a portion of the western end of Harquahala FRS. Gravel mulch was placed on Buckeye FRS No.2 (upstream and downstream slopes) and on the downstream slope of Buckeye FRS No.3 in the spring of 2005. Embankment slopes are typically hydroseeded before placement of gravel mulch. The mulch gradation and application thickness is generally the same for all structures. Maximum particle size is limited to 1½ inches, and thickness parallel to the slopes is between 4 and 6 inches.

B. Advantages and Disadvantages of Gravel Mulch

The primary purpose of applying gravel mulch to the slopes of the District's embankment dams is to provide for erosion protection of the slopes during rainfall events and to repair existing erosion damage on embankment slopes. Gravel mulch, when designed and applied correctly for the dam and slope conditions, can substantially reduce slope erosion through the formation of rills and gullies. The gravel mulch dissipates the rainfall energy impact and distributes rainfall over the surface of the embankment slopes. The gravel mulch also suppresses the impacts of wind erosion effects by armoring the surface. Other potential advantages that may be considered secondary are also listed on **Table A**.

In spite of the evident advantages, however, it has been recognized through the FMEA process that there may also be potential disadvantages of applying gravel mulch. A key consideration is that the cover obscures, or prevents monitoring of cracks on the embankment slopes during inspections. Other potential disadvantages are listed on **Table A**.

Table A. Advantages and Disadvantages of Applying Gravel Mulch

Advantages	Disadvantages
Provides erosion protection	High application costs for long structures
Reduces rodent activity and burrowing	May obscure or cover evidence of incipient crack formation, or changes in existing surface cracks, that would otherwise be observed during routine inspections
Helps retain and stabilize moisture in the embankment soils, minimizing shrinkage cracking	
Works as a mulch in combination with hydroseeding, improving seed survivability and water availability for sustaining plants	
Landscape aesthetics are superior to hydroseeding without mulch (less reflective, darker color)	Maintenance is required; tends to slide down-slope over time.
May provide some level of incidental overtopping protection when applied to downstream slopes	Potential damages from all-terrain vehicles
May provide a filtering effect for transverse embankment cracks	Safety concerns for walking slopes for dam safety inspections
When applied using a gravel shooter, much of the existing vegetation survives	
One-time application and good performance reduces slope erosion O&M costs	

C. Design and Performance Considerations

The purpose of this special study is to provide an analysis and evaluation of the use of gravel mulch to be used by the District in making decisions about future mulch applications. Currently, the District uses a single gradation specification and applies the mulch at a thickness of about 4 to 6 inches using a gravel shooter. There are no rigorous design criteria. However, published guidelines for erosion protection are available. The performance to date of the gravel covers that have been installed has been excellent with regard to erosion protection.

Design considerations for gravel mulch slope protection are inter-related to several performance considerations, as listed on **Table B**.

Table B. Design and Performance Considerations

Design Considerations	Performance Considerations			
	Erosion Protection	Evaporative Barrier	Filter	Aesthetics
Mulch gradation	X	X	X	
Mulch thickness	X	X	X	
Embankment soil characteristics	X	X	X	
Mulch particle angularity	X			
Runoff parameters	X			
Slope inclination	X	X		X
Mulch color				X
Hydroseed	X	X		X

Design for multiple performance goals may need to consider a variety of design parameters. For example, a mulch gradation that maximizes the evaporative barrier effect may not be the same gradation that meets optimum filtering criteria. The procedure governing design should be based first on the primary performance goal, e.g., erosion protection. The "erosion mulch" grading and thickness design could then be evaluated for its effectiveness in providing secondary performance goals such as filtering and as an evaporative barrier. If the design can be modified to enhance those secondary goals (e.g. increase thickness or modify gradation), without compromising the primary design objective, this should then be considered.

D. Risk Reduction through Gravel Mulch

The application of gravel mulch on embankment slopes has varied effects on potential failure modes. Failure modes associated with overtopping and transverse and longitudinal cracking are potentially mitigated or made less likely by application of gravel mulch. Gravel mulch provides some risk reduction for these failure modes because it treats existing erosion damaged areas and prevents formation of new, deep rills and gullies that would be particularly vulnerable locations for breaches caused by overtopping or seepage and erosion through cracks. When such a storm event occurs such that the depth of overtopping of the dam crest is very low, the gravel mulch armor layer may be sufficient to mitigate the impacts of overtopping flows on the downstream slope. The mulch will reduce the formation of rills on the slope and reduce flow energies down the slope.

Another measure of risk reduction may be realized through application of gravel mulch on dams that have exhibited shrinkage cracking. The gradation of the mulch is substantially coarser than the underlying embankment soil gradation, a capillary barrier effect may develop which helps retain and stabilize embankment soil moisture. This should help slow and reduce crack formation over time.

E. Evaluation

Each structure should be evaluated independently for the potential benefits of applying gravel mulch. **Table C** provides a possible checklist that could be used to aid in the assessment of whether or not mulch should be applied, and to prioritize applications among the portfolio of structures.

Table C. Evaluation Checklist

Evaluation Considerations	Yes	No
1. Does the dam exhibit surface erosion (rills/gullies and degree of erosion)		
2. Does the dam have a central filter?		
3. Does the dam exhibit shrinkage cracks?		
4. Does the dam exhibit cracks due to mechanisms other than shrinkage?		
5. Potential failure modes: a. overtopping? b. erosion and breach due to transverse cracks? c. erosion and breach due to inadequate central filter? d. other:		
6. Has dam been remediated for cracks?		
7. Are foundation/embankment conditions particularly conducive to future crack formation? a. known presence of collapsible foundation soils? b. irregular foundation shape or material transitions? c. long dam? d. other?		

F. Suggested Decision Matrix for Gravel Mulch

This section provides a suggested decision matrix for gravel mulching District dams. The District may chose to utilize and adapt this matrix after further evaluation from a Phase II evaluation. The decision to apply gravel mulch to a structure is highly dependent on the degree of erosion occurring on the embankment and whether cracking has been noted at the structure and on the engineering judgment of the extent and degree of cracking. The decision would also be based on the existence of a central filter within the structure. Depending on the degree of cracking, erosion problems, and the existence or non-existence of a central filter may assist in prioritizing gravel much application on embankment dams.

The expression of transverse and longitudinal cracking at District dams typically is noted and observed to be associated with erosion holes. One or more erosion holes of various sizes forms over the crack and provides a visual indicator of a potential crack within the embankment. During dam safety site inspections these erosion hole are probed usually with a 3-foot long steel rod to get an indication of the depth of the erosion hole and a measure of the resistance to probing (which in turn gives an indication if a crack is associated with the holes and the potential width of the crack).

Embankment erosion is experienced at every District dam. The degree of erosion varies from minimal erosion (very small rills extending for short lengths) to heavy gullies (severe gullies of 1 foot in width to 8 to 12 inches in depth or more and spaced fairly close together). Embankment dams with average to severe erosion are repaired by the District (e.g. Buckeye FRS No. 3 and Spook Hill FRS). Dams with minimal embankment erosion are the eastern portion of Harquahala FRS and those dams that already have a gravel mulch applied (such as the Wickenburg structures and the Corps of Engineers sponsored dams).

Many of the District dams (most notably the NRCS sponsored structures) have had a central filter installed after original construction, while others have had central filters installed as part of original construction. The central filters have been installed to either be partially penetrating filters (do not extend to full depth of foundation cutoff or foundation) or fully penetrating filters. The filters were designed and constructed to be "crack stoppers". The filters have been designed to arrest the transverse cracks from fully extending through the dam embankment. The significance of this discussion is that some District dams or portions of the dams have a central filter, some dams do not have or portions of the dam do not have a filter, and then the filters are partially or fully penetrating.

The primary concern at this time regarding the application of gravel mulch to an embankment is focused on those dams that have been noted to have confirmed or highly suspected existence of transverse and longitudinal cracking (e.g. Rittenhouse FRS, Vineyard Road FRS, west end of Harquahala FRS). The gravel mulch will mask or cover the typical method of observation of cracking (e.g.: erosion holes; associated rills and gullies). This would make further observations of the growth of cracking, interpretation of the severity of cracking, and routine maintenance of the embankment more difficult than without gravel mulch.

In relation to potential failure modes as a result of cracking, gravel mulch may not provide a visual means of surface expression of a crack. Dams with cracks and a gravel mulch cover are not observed as readily and potential cracks may go unnoticed. The result is existing cracks may become more severe and the intensity of cracking may increase without surface expressions. The degree and severity of cracking may not be noticed or observed until the crack becomes to such an extent to express through the gravel mulch layer. Cracks may be become larger in extent and degree such that these may make the embankment more conducive to embankment failure and breach during high and longer duration impoundments.

The following matrix is provided as a suggested evaluation subject for a Phase II investigation and to assist in the decision to apply gravel mulch and under what conditions. The table only relates the level of cracking at a dam or a portion of the dam to whether or not a central filter is present and whether that central filter is partially penetrating or fully penetrating.

Table D. Decision Matrix for Gravel Mulch

Presence of Central Filter	No Cracking	Low Cracking	Average Cracking	High Cracking
No Filter	Apply Gravel Mulch	Do Not Apply Gravel Mulch	Do Not Apply Gravel Mulch	Do Not Apply Gravel Mulch
Partially Penetrating Filter	Apply Gravel Mulch	Apply Gravel Mulch	Do Not Apply Gravel Mulch	Do Not Apply Gravel Mulch
Fully Penetrating Filter	Apply Gravel Mulch	Apply Gravel Mulch	Apply Gravel Mulch	Do Not Apply Gravel Mulch

As depicted in the above table, those dams that fall within the white zone would be gravel mulched. Those dams or portions of dams that fall within the shaded zone would not be gravel mulched at this time. This table is open to interpretation and judgment on a dam by dam basis and then on a reach by reach basis on a particular dam. A particular dam that is placed in the white zone may change over time to the shaded zone. The vice-versa is possible as well through a crack repair or dam rehabilitation project such that a dam in the shaded zone will be moved into the white zone. It must be understood that other factors will come into consideration regarding zone placement and cracking may not be the driving factor (e.g., degree of slope erosion for example).

G. Cost

A construction cost estimate was provided to Kimley-Horn by the District based on the District's experience of placing gravel mulch at their dams. The cost per mile of gravel mulch (including material, transportation, placement, and permitting) is approximately $100,000 to $150,000 per mile of dam embankment.

H. Recommendation

Kimley-Horn recommends that gravel mulch slope protection be considered further and carried forward into more detailed Phase II evaluation. The detailed evaluation should include the development of specific technical design criteria for gravel mulch that considers all performance goals (as listed in **Table B**), degree of erosion, application methods, and available sources of materials.

The following data and information should be collected and addressed before a more detailed analysis of gravel mulch slope protection is conducted:

1. Prepare a crack mapping program for all District dams and flood retarding structures. For each dam, a set of as-built plans should record the location of

all noted cracks from previous dam safety site inspections. In this fashion, the areas of each dam where cracking is most pronounced may be monitored and inspected more diligently during future inspections. The areas of the dam where cracks appear most notably may warrant a Phase II investigation.

2. Evaluate geophysical methods to locate and evaluate cracks on dams with gravel mulch covers.
3. Identify a priority list for gravel mulch applications for those dams that require erosion protection (surface erosion is problematic), and that already have central filters. Some District embankment dams only have filters in certain portions of the dam while other portions are without protection. Consider other information about the dam when prioritizing mulch applications, such as the suggested check list provided as **Table C**.
4. Evaluate other methods of slope protection.
5. Evaluate maintenance requirements for gravel mulch protection.
6. Evaluate how to conduct dam safety inspections on dams with gravel mulch slope protection.

8.0 RECOMMENDED STUDIES AND INVESTIGATIONS

The existing available studies, analyses, construction records, and investigations conducted as part of the design and construction of the structure were reviewed by the Kimley-Horn team. Kimley-Horn has developed the following recommendations for further studies and investigations as a result of the data review. In addition, recommendations for further studies and investigations were developed in the Failure Mode and Effect Analysis workshop and the dam safety site inspection for the dam. This section provides a summary of the recommendations.

8.1 Hydrologic and Hydraulic Recommendations

(1) Kimley-Horn recommends that an updated dambreak analysis and inundation mapping be prepared for the Saddleback FRS. New integrated hydraulic models such as HEC-RAS (unsteady flow and dambreak options) could be used to prepare the updated study. The dambreak update should develop reasonable dambreach parameters using published guidelines and the District's dambreach model currently under development. The true "sunny day" failure defined as a full pool with no inflow should be considered as well as a dambreak for the ½ PMF and PMF events using ADWR guidelines for routing through a flood control dam.

(2) Evaluate contributing watershed above CAP and evaluate effects of CAP canal and upstream embankment on flows contributing to Saddleback FRS. Confirm profile of upstream embankment to check if the extra recommended height of dam was constructed as stated in project documents.

(3) A quantitative risk assessment for the facility will require development of stage-frequency and emergency spillway discharge frequency relationships.

(4) Probable Maximum Precipitation. Prepare PMP/PMF using 24-hr and 72-hour durations. Compare routings of these events to PMP 6-hr duration flood to verify that they are less critical (or determine that they are more critical). Evaluate impact of the CAP canal on the PMF routings.

(5) Evaluate the need for a trash rack on the combined principal/emergency spillway.

(6) Input updated stage/storage/discharge rating curves into models. Evaluate impacts on routing IDF.

(7) Dynamic routing is recommended due to length of dam and geometry. Conduct dynamic routing for 100-year, ½ PMF, and PMF.

(8) Update the sediment yield analysis for the watershed. Typical sediment yield studies in Maricopa County have provided yield rates on the order of 0.2 to 0.3 acre-feet per square mile per year. The Watershed Workplan sediment yield for Saddleback FRS is 0.11 acre-feet per square mile per year.

8.2 Geotechnical and Geological Recommendations

A. Phase II Additional Evaluation of Zone II Drain Materials

The compatibility of the embankment materials and the ability of Zone II to adequately act as a filter for Zone I was evaluated for this Phase I Structures Assessment and is discussed in Section 3.4.5.D. No gradation data for the as-built filter materials were available for review, therefore the assessment of filter compatibility was based only on the specified filter gradation with no confirmation that the original filter was built within the specified gradation range. According to ADWR (1995), three filter gradation samples were collected during the dam rehabilitation work (see Section 3.4.5.E) and were within specifications, however these test results were not available for review. It is recommended that filter gradation data be obtained and the compatibility with the filter and embankment materials by confirmed.

B. Phase II Documentation of Slope Stability and Seepage Analyses

Under reasonable loading conditions for Saddleback FRS, it is expected that both upstream and downstream slopes will be stable. However, adequate documentation of slope stability factors of safety for specified loading and design criteria established by appropriate jurisdictional agencies is not available. Additional slope stability analyses are recommended to document the slope stability factors of safety for Saddleback FRS. **Table 5 Appendix B** shows the definitions of various loading conditions and a comparison between the current NRCS design criteria that are outlined in TR-60 (SCS, 1985), and the current criteria as presented in the Arizona Department of Water Resources (ADWR) dam safety rules and regulations for jurisdictional dams.

The original stability analysis does not completely document factors of safety for all the loading conditions required under current NRCS or ADWR criteria. **Table 6 Appendix B** summarizes the results from the original stability analysis and indicates where additional analyses are required.

(1) **End of construction (upstream and downstream slope)**: The original factor of safety calculated for the downstream slope under end of construction loading conditions achieved the minimum ADWR criteria of 1.3 (see **Table 5 Appendix B**). Evaluation of the downstream slope stability under these loading conditions was not performed.

(2) **Rapid drawdown (upstream slope)**: The original stability analysis for this loading condition resulted in calculated factors of safety that are currently acceptable under ADWR rules. Additional analyses are not required.

(3) **Steady state seepage without seismic forces**: The original factor of safety calculated for this loading condition in Reach 2 (1.23) did not achieve the minimum criteria of 1.5 (see **Table 5 Appendix B**). Additional analyses, including confirming the shear strength of embankment soils, either by review of

additional data not available during this Phase I Structures Assessment or by field sampling, and reevaluating the critical failure surface on the downstream slope are recommended to document the stability of the downstream slope for this structure.

(4) **Steady state seepage, partial pool elevation (upstream slope)**: The original analysis did not evaluate upstream slope stability under this loading condition. The ADWR criteria for partial pool conditions is intended for water retention dams, in which a steady state phreatic line may develop for intermediate pool elevations. The factor of safety may be lower for the intermediate pool conditions than the steady state condition under maximum pool. The following analysis could be done to document the minimum partial pool factor of safety, under the scenario that the outlet works are clogged such that the steady state phreatic line develops:

a. Perform seepage analyses under various partial pool elevations to establish the steady state pore pressure distributions within the dam at each pool elevation.
b. Conduct slope stability analyses for each partial pool seepage analysis result, and graph the results as factor of safety versus pool elevation.
c. Report the minimum factor of safety and corresponding pool elevation.

(5) **Steady state seepage with seismic forces (downstream slope)**: No seismic stability analysis was documented for Saddleback FRS. To document seismic stability under current design criteria, a pseudo-static stability analysis is recommended. The analysis should use a peak ground acceleration (PGA) of 0.1g and the ADWR recommendation of a pseudo-static coefficient equal to 60% of the PGA.

8.3 Erosion Holes Along Crest Centerline

Recommendations for Further Investigations and Monitoring

The Kimley-Horn team, as part of this Phase I assessment, is recommending the following actions to ascertain the nature of the erosion hole formation, monitoring of the erosion holes, additional geotechnical investigations for filter compatibility, filter material testing, and a potential repair alternatives to be evaluated further under a Phase II investigation.

1. An erosion hole monitoring program should be developed to note the locations and sizes of the holes along the crest. The program should map the locations on a set of as-built plans, obtain GPS coordinates of the erosion holes, and download the locations into the District GIS system. In this manner, the District may monitor, over time, hole locations and sizes, as well as the sections of the dam where erosion hole formation is most prevalent.

2. A field investigation program should be developed to investigate the compatibility of the filter with the embankment soil, the in-place density of the filter, and the current moisture and stress state of the Holocene soils underlying both the upstream and downstream zones of the dam. This would include:

    a. Drilling, sampling, and in-place density testing (standard penetration tests, or SPTs) of the foundation soils at the upstream and downstream toes of the dam; and drilling, sampling and SPTs of the embankment, filter zone and foundation soils from the crest of the dam,

    b. Test trenching and bulk sampling along the crest; samples should be collected from both the filter and the embankment zones,

    c. Mapping of cracks (transverse and longitudinal) in the test trenches (similar to what was done at Buckeye),

    d. Laboratory testing for grain size characteristics, Atterberg limits, consolidation-collapse potential, and in-place moisture content.

3. Conduct high-resolution ground penetrating radar (GPR) surveys across the embankment crest to locate anomalies that may represent erosion hole, cracks or other voids. Excavate selected anomalies to determine what they represent and their possible relationship to erosion holes. Representative test GPR surveys could be run across areas of known erosion holes or void to define their radar signature. This will assist with the characterization of other anomalies that are not coincident with surface expressions of erosion holes or other voids. The information gathered during the GPR surveys can be integrated into the data base generated in Item 1, above.

4. Using the data gathered during the previous site investigations and dam safety inspections, quantitatively and qualitatively identify the sections of the dam to define zones of high, moderate, low, and no concentrations of erosion holes. With this information, develop a phased mitigation program. The phasing of the mitigation program could be a function of available funding starting with the zone of high concentration.

5. Potential Interim and Permanent Repairs for further evaluation as part of a Phase II investigation may include:

    a. Interim measure: Filling the erosion holes with sand and allow sand to migrate into filter. Continue to apply sand until further sand will not be accepted. Cover sand at crest with compacted borrow materials from pool area.

    b. Potential permanent repair: remove the embankment from the crest to one foot below the elevation of the centerline filter. Where the filter is not in-place, remove the embankment to one-foot below the design flood maximum elevation. Rebuild the affected embankment sections and drain to the design crest elevation with engineered compacted fill.

    c. Potential permanent repair: Excavate by trenching entire centerline filter. Install geofabric in trench (to encase drainfill) and backfill trench to 1 foot below crest elevation with clean new drainfill material. Cap trench with compacted borrow materials from pool area.

8.4 Additional Recommendations from Inspection Report

(1) Provide Additional Means for Flood Warning. Add more gauges in contributing watershed, outside watershed, and stream gauges. Consider use of Doppler radar and satellite imaging.
(2) Conduct a Phase II study to evaluate converting Saddleback FRS and floodway into a true FEMA certified levee system. This scenario would remove the structure from state jurisdictional oversight. The potential of converting this dam and floodway to a levee system is promising.
(3) Develop an Emergency Action Plan to meet FEMA 64 and ADWR requirements;
(4) Evaluate gravel mulching of embankment slopes;
(5) Evaluate effect of Solome Road over crest of dam. It is recommended that the District consider stabilizing Solome Road using concrete asphalt paving for the entire roadway surface over the dam and ¼ mile upstream and downstream from the dam. Paving could be applied to the dam crest at the intersection of the road for approximately 100 feet in either direction.
(6) Locate the toe drains at Station 204+00 and 248+00;
(7) Determine source of material deposited in the toe drain outlets.
(8) Due to the length of the structure (5 miles), an additional staff gage should be added. A recommended location would be at the Solome Road crossing of the embankment.
(9) Map all cracks on set of as-built plans and profiles as well as aerial photo of dam. Continue to map cracks after all dam safety inspections. Monitor, over time, reaches of dam where there has been a noted propensity of cracks.

8.5 Recommendations from FEMA Report

(1) Identify And Quantify The Existence Of Transverse Cracks. Identified transverse cracks should be noted on a set of as-built plans. Over time the District will be provided with a map that indicates the higher frequency of crack occurrence along the dam indicating reaches of the dam that may need a remedial repair.
(2) Determine The Cause Of Crest Holes. Several mechanisms of the formation of the crest erosion holes were discussed by the FMEA team. The team agreed that a likely causative mechanism for erosion hole formation is the downward movement of crest materials in the filter drain as a result of settlement of the filter.
(3) Dynamic Routing Is Recommended Due To Dam Length And Geometry. Unsteady flow modeling will provide better insight in the response of the structure and floodway from large storm events including the IDF and PMF. The dam design was based on the method of level pool routing. Due to the length of the structure, the water surface at one end of the dam may not be the same as at the other end during an impoundment event.
(4) Continue Monitoring Of Drain Outlets Where Silt Has Been And Continues To Be Observed. The inspection team observed silt in several of the downstream

drain outlets. Future dam safety site inspection should document the presence (on not) of silt in every drain outlet. Large amounts of silt could be an indicator of stress within the dam.

(5) Verify Utility Relocations, Add Fiber Optic Line To The Plans, And Locate All Drain Outlets On The Plans And Replace If Not Found. The as-built plans indicate that several utilities were relocated prior to dam construction. A set of as-built plans should be used to record utility crossing of the dam and floodway. A fiber optic cable was recently constructed at the Salome Highway crossing of the dam.

(6) Evaluate Drain Outlet Video Tape. The District conducted a video survey of the drain outlet system located at the principal spillway.

(7) Evaluate Hydrologic Routing With Salome Road Culverts Plugged. This action item may be conducted as a modeling scenario for dynamic routing for large storm events. This approach could check for potential overtopping of the dam crest under the IDF and PMF.

(8) Perform Multi-Frequency Analysis To Determine Incipient Overtopping. A multi-frequency analysis of large storms will provide an indication of the frequency storm to just cause overtopping of the dam.

(9) Locate Utility As-Built Files. The dam construction plans indicate that several utilities were relocated prior to dam construction at Salome Highway. However, the as-built plans for the utility relocations could not be located in District or NRCS files. The locations shown on the dam as-built plans are assumed to be the as-built relocations from the as-built utility plans.

(10) Maintain Annual Surveys In The Area Due To Potential Fissure Risks. Three earth fissures have been documented in the Harquahala and Centennial valleys. It is prudent to continue with dam crest surveys and toe monument surveys to monitor for local land subsidence.

(11) Develop IGA With ADWR On Fissure Monitoring And Include Saddleback FRS In The Study To Develop Baseline INSAR Data. A fissure monitoring program may be developed in conjunction with other state and federal agencies. One agency is ADWR who has state oversight on groundwater use and groundwater pumping in the state. One of ADWR monitoring and interpretive techniques is the use of INSAR data and imagery.

(12) Regular Inspections In The Vicinity Of The Harquahala Floodway Discharge Into The Saddleback FRS And At The Roadside Drainage Next To Courthouse Road Are Recommended. The Phase I dam safety site inspection observed at Courthouse Road that a roadside drainage channel appeared to be migrating toward the right abutment. Monitoring of both abutments and the inflow location from the Harquahala floodway to Saddleback FRS is recommended during dam safety site inspections.

(13) Review Existing Instrumentation (Rainfall And Streamflow Gages) And Recommend Changes And Modifications If Necessary. A non-structural alternatives measure could include evaluation of added rainfall and streamflow instrumentation in the upstream watershed. A staff gage is recommended at the Salome Highway crossing. A stream gage should be considered at the CAP canal overchute that monitors floodwaters released from the CAP embankment.

8.6 Recommendations for Monitoring and Inspection of Longitudinal Cracks in Reservoir Area

(1) Continue to monitor for longitudinal cracks in pool area after major rainfall events and/or impoundments in pool area.
(2) Develop a crack investigation and repair method for longitudinal cracks.
(3) Develop rainfall criteria amount (1 inch or more) to trigger a site inspection of dam and pool area.

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Table 1. Dam Crest Elevations (NGVD29).

Beginning Station	Ending Station	Distance (ft)	Dam Crest Elevation (ft)
3+00	49+00	4,600	1193.0
49+05	72+95	2,390	1193.33
73+00	171+45	9,845	1193.0
171+50	272+70	10,120	1194.0

Note: Table 2 is located on the following page.

Table 3. Irrigation/Vegetation Maintenance Outlets.

FRS Station	Length of Conduit (ft)	No. of Seepage Collars	Seepage Collar Spacing (ft)	Inlet
60+50	104	3	14	Riser Unit 3
103+70	56	1	13	Unit 1
124+10	48	1	10	Unit 1
256+00	64	2	11	Riser Unit 3

Table 2. Saddleback FRS Physical Data Summary.

Item	Unit	Value
Class of Structure (NRCS)		A
Drainage Area (Uncontrolled)	square miles	22.3
Average Curve Number (1-Day AMC II)		83
Elevation-Top of Dam	ft	1193
Elevation-Principal/Emergency Spillway Crest	ft	1176.9
Maximum Height of Dam	ft	20.8
Volume of Fill	yd^3	360,000
Length	ft	5.1 miles
Maximum Bottom Width	ft	111
Top Width	ft	12
Upstream Slope Z:1 ; Downstream Slope Z:1		US = 3 ; DS=2
Total Capacity	acre-feet	3620
Sediment (50-Year)	acre-feet	120
Retarding Pool	acre-feet	3500
Surface Area		
Sediment (50-Year)	acres	61
Retarding Pool	acres	760
Principal Spillway Design		
Rainfall Volume (Areal, 1-Day)	inches	4.15
Rainfall Volume (Areal, 10-Day)	inches	5.67
Runoff Volume (10-Day)	inches	3.27
Capacity	cfs	1,110
Frequency Operation-Emergency Spillway	%	Combined Spillway
Dimensions of Conduit	ft	10' X 8' X 65'
Tailwater Elevation	ft	5.7
Type of Outlet		SAF
Drawdown Time	days	6
Emergency Spillway Design		
Rainfall Volume (ESH, Areal)	inches	3.99
Runoff Volume (ESH)	inches	2.54
Storm Duration	hours	6
Type		Combined Spillway
Maximum Reservoir Water-Surface Elevation	ft	1,189.0
Maximum Outflow from ESH Routing	cfs	1,028
Freeboard Design		
Rainfall Volume (FH, Areal)	inches	6.86
Runoff Volume (FH)	inches	4.9
Storm Duration	hours	6
Maximum Reservoir Water-Surface Elevation	ft	1193
Maximum Outflow from FH Routing	cfs	1,300
Capacity Equivalent		
Sediment Volume	inches	0.098
Retarding Volume	inches	2.19

All elevations based upon the NGVD 1929

Item	NRCS Original Design Criteria	NRCS Current Criteria	ADWR Criteria Eff June 12, 2000	Comment/Remarks
Publications and References for NRCS and ADWR Criteria	1) "Engineering Memorandum 27 - Earth Dams" SCS March 19, 1965 (EM-27) 2) Harquahala Valley Watershed Work Plan SCS January 1967; 3) Supplemental Watershed Work Plan No.1 - Harquahala Valley Watershed. SCS March 1977.	Technical Release No. 60 TR-60. Earth Dams and Reservoirs. Oct. 1985. Amended Jan 1991	Arizona Administrative Code Title 12, Chapter 15 Effective June 12, 2000	
Size			**Intermediate:** Storage capacity 1,000 to but not exceeding 50,000 Acft and height 40 to but not exceeding 100 ft	
Hazard. Classified as a Class A but designed as a Class B.	**Class A.** Structures located in predominately rural or agricultural areas where failure may damage farm buildings, agircultural land, or township and county roads.	**Class A.** Structures located in predominately rural or agricultural areas where failure may damage farm buildings, agircultural land, or township and county roads.	**Significant Hazard Potential.** Failure or improper operation of a dam would be unlikely to result in loss of human life but may cause significant or high economic loss, intangible damage requiring major mitigation, and disruption or impact on lifeline facilities. Property losses would occur in a predominantly rural or agricultural area with a transient population but significant infrastructure.	Significant: Probable loss of human life - none expected Probable Economic, Lifeline, and Intagible Losses - Low to High
Inflow Design Flood (IDF)	One-percent event		Significant, Intermediate. 1/2 PMF	
Total Freeboard (between Emergency Spillway crest and the settled top of the dam crest)			The applicant shall ensure that the total freeboard is the largest of the following: a) The sum of the IDF maximum water depth above the spillway crest plus wave runup. b) The sum of the IDF maximum water depth above the spillway crest plus 3 feet. c) The minimum of 5 feet.	
Residual Freeboard (between maximum IDF water surface elevation to dam crest)		between maximum water surface elevation to dam crest	means the vertical distance between the highest water surface elevation during the IDF and the lowest point at the top of the dam	
Principal Spillway Design Flood	100-year	100-year. A storm duration of not less than 10 days is to be used for sizing the principal spillway. Use NEH-5, TR-29, Design Note 8	N/A	100-year
Principal Spillway Capacity	(a) Discharge through the emergency spillway will not occur (b) Adequate to empty the retarding pool in 10 days or less. Or adequate to empty 80 percent or more of the maximum volume of retarding storage after 10 days. The 10-day is measured starting from the time the maximum water surface elevation is attained during the passage of the principal spillway flood (EM -27 Page E-1 Supplement 6)	(a) Discharge through the emergency spillway will not occur (b) Adequate to empty the retarding pool in 10 days or less. Or adequate to empty 80 percent or more of the maximum volume of retarding storage after 10 days. The 10-day is measured starting from the time the maximum water surface elevation is attained during the passage of the principal spillway flood (c) The minimum diameter of the principal spillway conduit is to be 30 inches.	Low level outlet that is capable of: i) draining the reservoir pool to the sediment pool level ii)significant hazard dams - Outlet works shall be a minimum of 36-inch diameter b. significnat hazard dams: capacity to drain 90% of storage capacity of reservoir within 30 days. c. has diaphram filter or other current practice measure to reduce potential for piping along conduit.	(a) Discharge through the emergency spillway will not occur (b) Adequate to empty the retarding pool in 10 days or less. Or adequate to empty 80 percent or more of the maximum volume of retarding storage after 10 days. The 10-day is measured starting from the time the maximum water surface elevation is attained during the passage of the principal spillway flood
Initial Reservoir Stage for Principal Spillway Hydrograph Routing	Crest elevation of the lowest ungated principal spillway inlet or the anticipated elevation of the sediment storage, whichever is higher	Crest elevation of the lowest ungated principal spillway inlet or the anticipated elevation of the sediment storage, whichever is higher	N/A	Crest elevation of the lowest ungated principal spillway inlet or the anticipated elevation of the sediment storage, whichever is higher
Runoff Volume Estimation Procedures for Principal Spillway Sizing	National Engineering Handbook No 4 Hydrology	Part 630 and NEH 4. Use CN method and AMC II	N/A	
Design Procedures for Principal Spillways	EM -27 Appendix E Principal Spillways	TR 60 Chapt 6 Principal Spillways	for high and significant hazard dams principal spillway shall be 36-inches or greater; all high and significant hazard dams shall have the capacity to evacuate 90% of storage capacity of reservoir within 30 days, excluding reservoir inflows; corrugated metal pipe not acceptable	
PMP Storm Types	NA	General and local. HMR No. 49. the storm duration and distribution that result in the maximum reservoir stage when the hydrograph is routed through the structure should be used.	Both frontal and thunderstorm (tropical) type storms should be studied with due consideration given to tropical storm potential and orographic influences that may greatly increase rainfall. Local Storm duration 6 hour; General Storm duration 72 hour (whichever is greater)	See ADWR guidlelines "PMF Studies for Evaluation of Spillway Adequacy General Guidelines" Revised March 2004. Site-specific PMP studies are acceptable.
Reservoir Stage-Storage Curve for Routing IDF Hydrograph and Stability Design Storm Hydrograph		For Class A Structure 1. emergency spillway hydrograph = P100 2. freeboard hydrograph = P100 + 0.12(PMP-P100) For Class B Structure 1: emergency spillway hydrograph P100 + .12(PMP - P100) 2: freeboard hydrograph = P100 + 0.4(PMP-P100)	The adequacy of the emergency spillway is normally determined by routing the IDF through the reservoir and spillway. Flood routings for spillway capacity determinations will normally be required to begin with reservoir storage at the spillway crest elevation. An infrequent exception is that the reservoir is used exclusively for flood control and would normally be empty.	

Item	NRCS Original Design Criteria	NRCS Current Criteria	ADWR Criteria Eff June 12, 2000	Comment/Remarks
Emergency Spillway Capacity	(a) Pass the emergency spillway hydrograph resulting from P100 at the safe velocity (b) Pass the freeboard hydrograph with the water surface elevation at or below the design top of the dam (c) Capacity must not be less than that determined from Figure F-1 on Page F-3 in EM-27	(a) Pass the emergency spillway hydrograph resulting from P100 at the safe velocity (b) Pass the freeboard hydrograph with the water surface elevation at or below the design top of the dam (c) Capacity must not be less than that determined from Figure 7-1 on Page 7-8 in TR-60	i. Ensure that each spillway, in combination with outlets, is able to safely pass the peak discharge flow rate, as calculated on the basis of the inflow design flood. ii. include a control structure to avoid head cutting and lowering o fthe spillway crest for spillways excavated in soils or soft rock.	
Emergency Spillway Crest Elevation	(a) Satisfy the 2500 ac-ft total capacity limit (PL 83-566, NWM 500.20) (b) The discharge through the emergency spillway will not occur during the routing of the principal spillway hydrograph (c) If the 10-day drawdown requirement is not met for principal spillway capacity design, then the crest elevation of the emergency spillway will be raised as noted on Page 6-1, Capacity of Principal Spillway.	(a) Satisfy the 2500 ac-ft total capacity limit (PL 83-566, NWM 500.20) (b) The discharge through the emergency spillway will not occur during the routing of the principal spillway hydrograph (c) If the 10-day drawdown requirement is not met for principal spillway capacity design, then the crest elevation of the emergency spillway will be raised as noted on Page 6-1, Capacity of Principal Spillway.	N/A	(a) Satisfy the 2500 ac-ft total capacity limit (PL 83-566, NWM 500.20) (b) The discharge through the emergency spillway will not occur during the routing of the principal spillway hydrograph (c) If the 10-day drawdown requirement is not met for principal spillway capacity design, then the crest elevation of the emergency spillway will be raised as noted on Page 6-1, Capacity of Principal Spillway.
Initial Reservoir Stage for Emergency Spillway Hydrograph Routing	The highest value from the following elevations: (a) Elevation of the lowest ungated principal spillway inlet (b) The anticipated elevation of the sediment storage (c) The elevation of the water surface associated with significant base flow (d) The pool elevation after 10 days of drawdown from the maximum stage attained when routing the principal spillway hydrograph. (Page 7-2 in TR 60)	The highest value from the following elevations: (a) Elevation of the lowest ungated principal spillway inlet (b) The anticipated elevation of the sediment storage (c) The elevation of the water surface associated with significant base flow (d) The pool elevation after 10 days of drawdown from the maximum stage attained when routing the principal spillway hydrograph. (Page 7-2 in TR 60)	i. Deviations from the normal starting level of routing at the spillway crest elevation must be considered on the basis of risk and reservoir operating procedure, and are evaluated by the Department on a case-by-case basis. ii. See ADWR guidlelines "PMF Studies for Evaluation of Spillway Adequacy General Guidelines" Revised March 2004. Site-specific PMP studies are acceptable.	
Sedimentation	50-year sediment reservoir.	100-year sediment reservoir	N/A	
Dam Breach		See TR-60 for Qmax for depth of water less than 103 feet	Unless waived by the Director, owners of high and significant hazard potential dams shall prepare, maintain, and exercise Emergency Action Plans for immediate defensive action to prevent failure of the dam and minimize threat to downstrem development.	Develop EAP to FEMA 64 guidelines and ADWR requirements.
Special Requirement for Storage	2500 ac-ft (total reservoir capacity = water volume plus the anticipated sediment volume) according to Table 500-2 in Public Law 83-566, National Watershed Manual-Part 500.20. Based on Table 500-2, any amount for construction costs and >4,000 ac-ft of total capacity require a committee on Environment and Public Works of the Senate and committee on Public Works and Transportation of the House of Representatives.	2500 ac-ft (total reservoir capacity = water volume plus the anticipated sediment volume) according to Table 500-2 in Public Law 83-566, National Watershed Manual-Part 500.20. Based on Table 500-2, any amount for construction costs and >4,000 ac-ft of total capacity require a committee on Environment and Public Works of the Senate and committee on Public Works and Transportation of the House of Representatives.	The temporary storage will be evacuated as soon as possible following such periods of flood.(from License)	
Seismic		See NEH-8 and Part 531, 210-v	Design the dam to withstand the maximum credible earthquake (MCE)	AAC R12-15-1216.B.2. Seismic Requirements
Design for Vegetated and Earth Emergency Spillways	N/A	N/A	N/A	
Miscellaneous Design Criteria	Section G. Top width of earth embankments will not be less thatn the value given by the following equation, except for single purpose retarding dams: W = (H+35)/5 where H= max ht of embankment in feet and W = minimum top width of embankment in feet. For single purpose retarding dams, the top width may be in accordance with the table on page G-1. In this case the embankment top width is 11 ft.	Minimum top width is 14 feet.	a. the design …shall include seepage collection and prevent internal erosion or piping due to embankment cracking.. B. the minimum top width of an embankment dam is equal to the structrual height of the dam divided by 5 plus an additional 5 feet. The required minimum top width for any embankment dam is 12 feet. The maximum top width for any embankment dam is 25 feet. c.the applicant shall keep the top of the dam and appurtenant structures accessible by equipment and vehicles for emergency operations and maintenance.	

Table 5. Principal Spillway Hydrograph Summary Data. (NGVD29)

			100-Year/PSH Rainfall				
Weighted CN	T_C [hr]	DA [sq. mi]	1 day [in]	10 day [in]	Sediment Pool Elevation at Outlet [ft]	Orifice Size	Emergency Spillway Crest [ft]
86	1.05 to 1.61	22.3	4.15	5.67	1185.7	10-foot X 8-foot box culvert	1176.9

Table 6. Freeboard and Emergency Spillway Hydrograph Summary Data. (NGVD29)

		Rainfall		ESH		FBH	
Emergency Spillway Crest (ft)	Bottom Width (ft)	ESH (in)	FBH (in)	Peak ES Discharge (cfs)	Maximum WSEL (ft)	Peak ES Discharge (cfs)	Maximum WSEL (ft)
1176.9	8	3.99	6.86	1028	1188.9	1300	1192.8

Table 7. Reservoir and Storage Summary Data. (NGVD29)

Item	Elevation [FT]	Area [AC]	Sum Storage [AF]
Bottom of Pool	1175	0	0
Top of Sediment Pool			
Basin No. 1	1178.3	40	80
Basin No. 2	1185.7	20	30
Crest of Combination Spillway*	1176.9	30	50
PSH Peak Water Surface Elev.	1190.1	760	3620
Dam Crest (w/o camber)	1194.0	1300	7600

*Estimated from Stage-Storage Curve

Table 8. Dambreak Hydrologic Summary

	6-Hour Duration Storm	72-Hour Duration Storm
Drainage Basin Area (mi^2)	76.9*	Same
PMP Rainfall Depth (inches)	15.4	9.5
Curve Numbers	84-87	Same
PMF Reservoir Peak Inflow (cfs)	63,083	23,766
½ PMF Reservoir Peak Inflow (cfs)	31,541	11,883
½ PMF Maximum Reservoir WSEL (ft)	1190.83	1192.07
½ PMF Runoff Volume (Af)	5,483	9,040
Crest of Combination Spillway (ft)	1176.9	Same
Design Crest of Dam w/o camber (ft)	1193.0-1194.0	Same
Maximum Storage Below Dam Crest (Af)	7,500	Same

*Uncontrolled 22.3 square miles and controlled 54.6 square miles

Table 9. Dam Breach Parameters.

	North Breach	South Breach
Initial Reservoir WSEL (ft)		
6-hour duration event	1190.8	Same
72-hour duration event	1192.0	Same
Initial Breach Elevation	1183.5	1176.9
Inflow Peak Discharge (cfs)		
6-hour duration event	6,537	11,844
72-hour duration event	11,835	18,061
Dam Breach Bottom Width (ft)		
6-hour duration event	122	64
72-hour duration event	201	100
Time of Breach Formation (hrs)		
6-hour duration event	5.8	3.4
72-hour duration event	5.4	3.7
72-Hour DAMBRK Peak Outflow Discharge (cfs)	12,300	18,585

Table 10. EOP and FERM Notification Levels

	Emergency Operations Plan (November 2003)	FERM (January 2002)
	Pool Level [ft]	Pool Level [ft]
District Alarm	-	4.0
Notify FCD O&M	-	6.5
Notify McDEM	8.0 ft at the P.O	9.5

Table 11. Saddleback FRS FCD Gage Id 5112, 5113.

STATION DESCRIPTION

LOCATION – The structure is located on the east end of the Harquahala Valley and approximately eight miles west of Tonopah. The dam is downstream of Harquahala FRS and receives inflows from that structure. Access to the structure is from Courthouse Road from either Harquahala Valley Road on the west or Salome Road from the east. Latitude N33 27 55, Longitude W113 04 21. Located in the SE1/4 SW1/4 SW1/4 S34 T2N R8W in the Saddle Mountain 7.5-minute quadrangle.

ESTABLISHMENT – December 16, 1988

DRAINAGE AREA – 29.6 mi^2 not including area from Harquahala FRS

GAGE – The gage is a pressure transducer type instrument. The PT is at elevation 0.30 feet gage height, levels of April 1, 1997, or 1,179.40 feet NAVD 1988.

There are three staff gages at this location. The gages are in five foot intervals. Both the 0 – 5 foot gage and the 5 – 10 foot gage read about 0.04 feet high. The 10 – 15 foot gage reads about 0.55 feet high.

There is no crest gage at this location.

ZERO GAGE HEIGHT – Zero gage height is defined as the inlet invert of the outlet culvert, or 1,179.10 feet NAVD 1988.

HISTORY – No gauging at this site prior to gage installation. In 1991, instrument elevation changed from 0.00 to 0.30 feet gage height. Unsure if this represents an actual movement of the instrument or a change in definition of zero gage height. Datum changed from NGVD 1929 to NAVD 1988 in April 1997.

REFERENCE MARKS –

RM276 is an FCD brass cap in concrete located approximately 250 feet north of the outlet near the bend of the structure. It is not located on the dam but about 50 feet west of it. The BC is stamped with elevation 1,178.89 feet. The FCD 93-51 McLain Harbers surveyed NAVD 1988 elevation is 1,181.05 feet. As surveyed on April 1, 1997 the gage height of the reference is 1.95 feet.

RP1 is the top of the headwall on outlet side north corner near fence post. Elevation 8.57 feet gage height, levels of April 1, 1997.

RP2 is the top of headwall on inlet side just above '+' in Station 16+83 paint. Elevation 9.81 feet gage height, levels of April 1, 1997.

CHANNEL AND CONTROL – The primary outlet of the dam is an ungated 8 foot by 10 foot rectangular box culvert that is 65 feet in length. There is no auxiliary spillway at this location.

PRIMARY / AUXILIARY OUTLET –

The primary outlet is a rectangular box culvert that has dimensions 8 foot high by 10 foot wide. The invert at the inlet has elevation 0.00 feet. The invert at the outlet has elevation –0.30 feet (estimated from 1993 survey). The length of the culvert is 65 feet.

There is no auxiliary spillway at this structure.

The top of dam elevation is about 17.0 feet gage height.

RATING – The current discharge rating is Rating #1 developed by Donaldson using a culvert analysis.

The current capacity rating is Rating #2 developed from DTM data from the FCD93-51 McLain Harbers survey.

DISCHARGE MEASUREMENTS – Direct measurements could be made in the outlet channel downstream from the structure.

POINT OF ZERO FLOW – Flow begins through the outlet at 0.00 feet gage height.

FLOODS / SIGNIFICANT IMPOUNDMENTS –

REGULATION – Some regulation from Harquahala FRS upstream which discharges to Saddleback FRS.

DIVERSIONS – None known

ACCURACY – Good

JUSTIFICATION – Monitor water levels behind Saddleback FRS for public safety.

UPDATE – February 5, 2001

Kimley-Horn
and Associates, Inc.

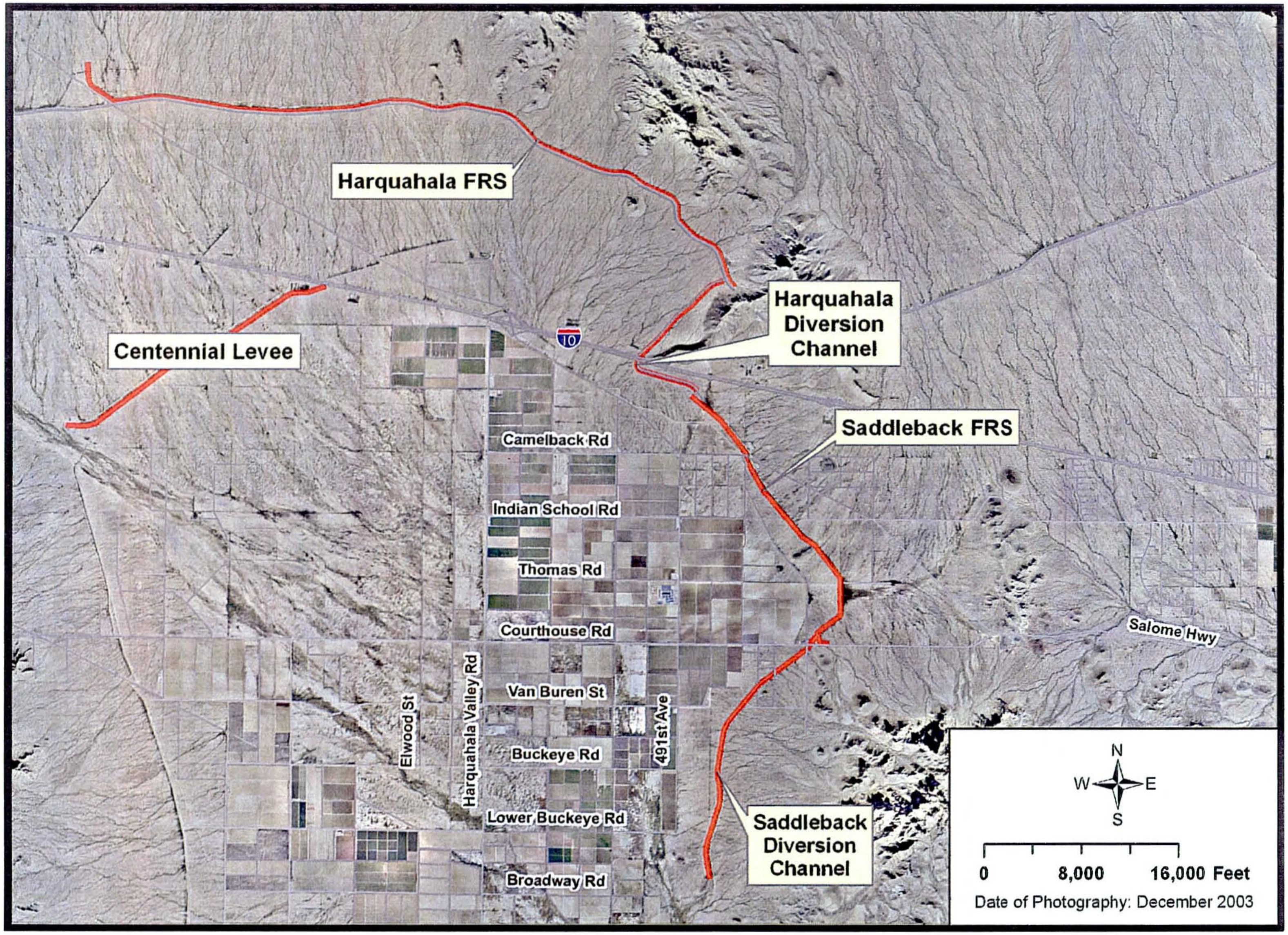

Figure 1. Location Map
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Saddleback FRS

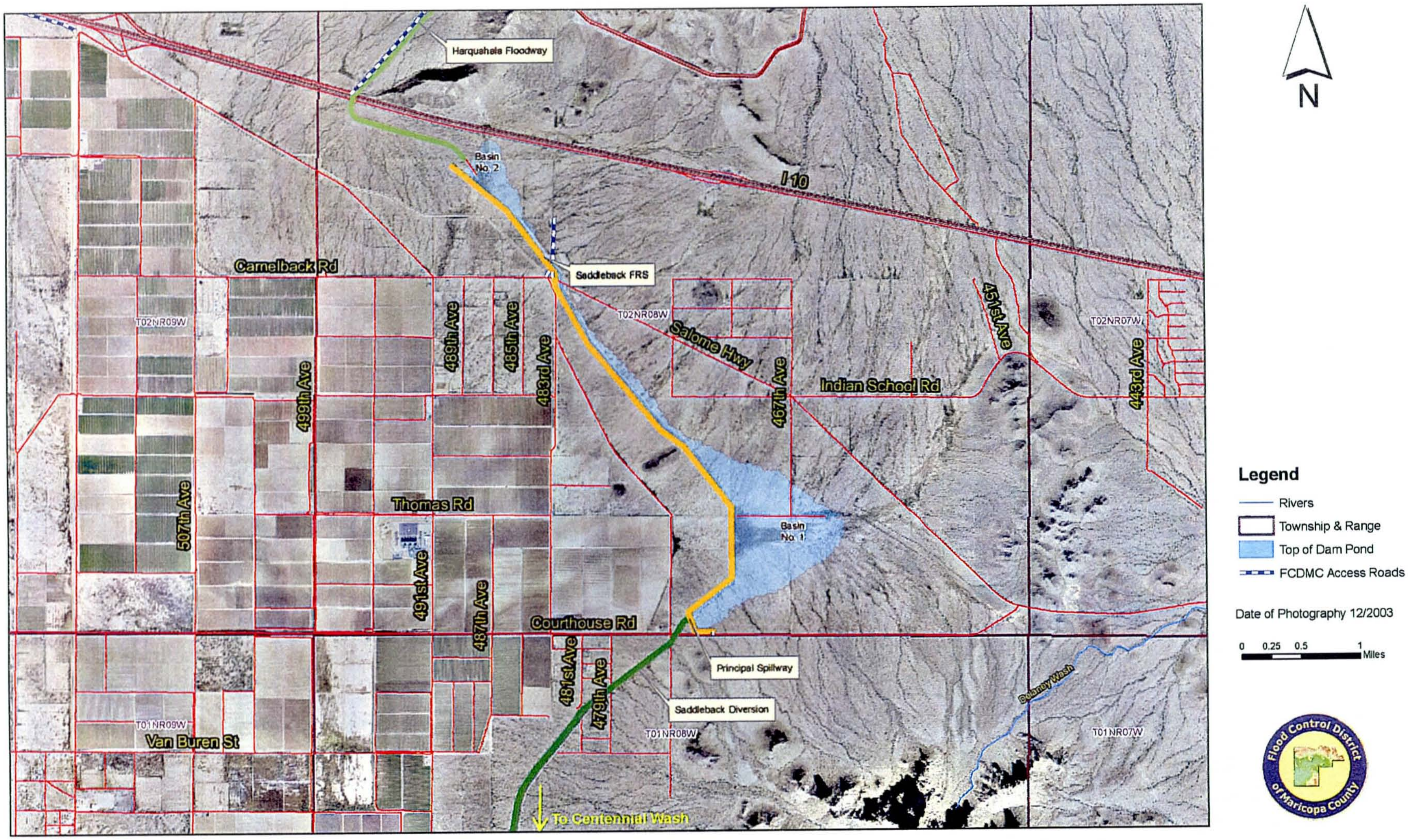

Figure 1A. System Map for Saddleback FRS
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn and Associates, Inc.

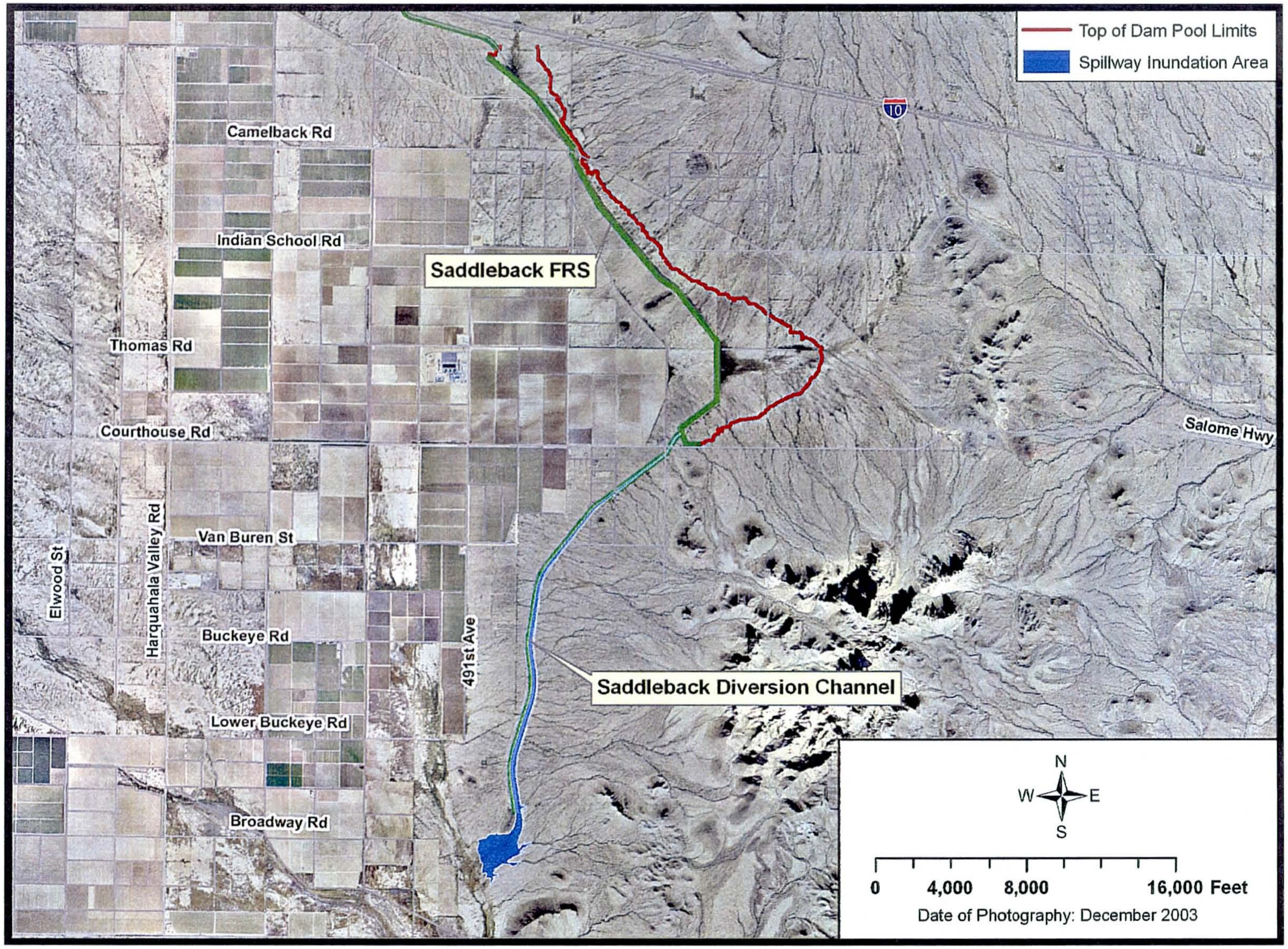

Figure 2. Saddleback FRS Spillway Inundation Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn
and Associates, Inc.

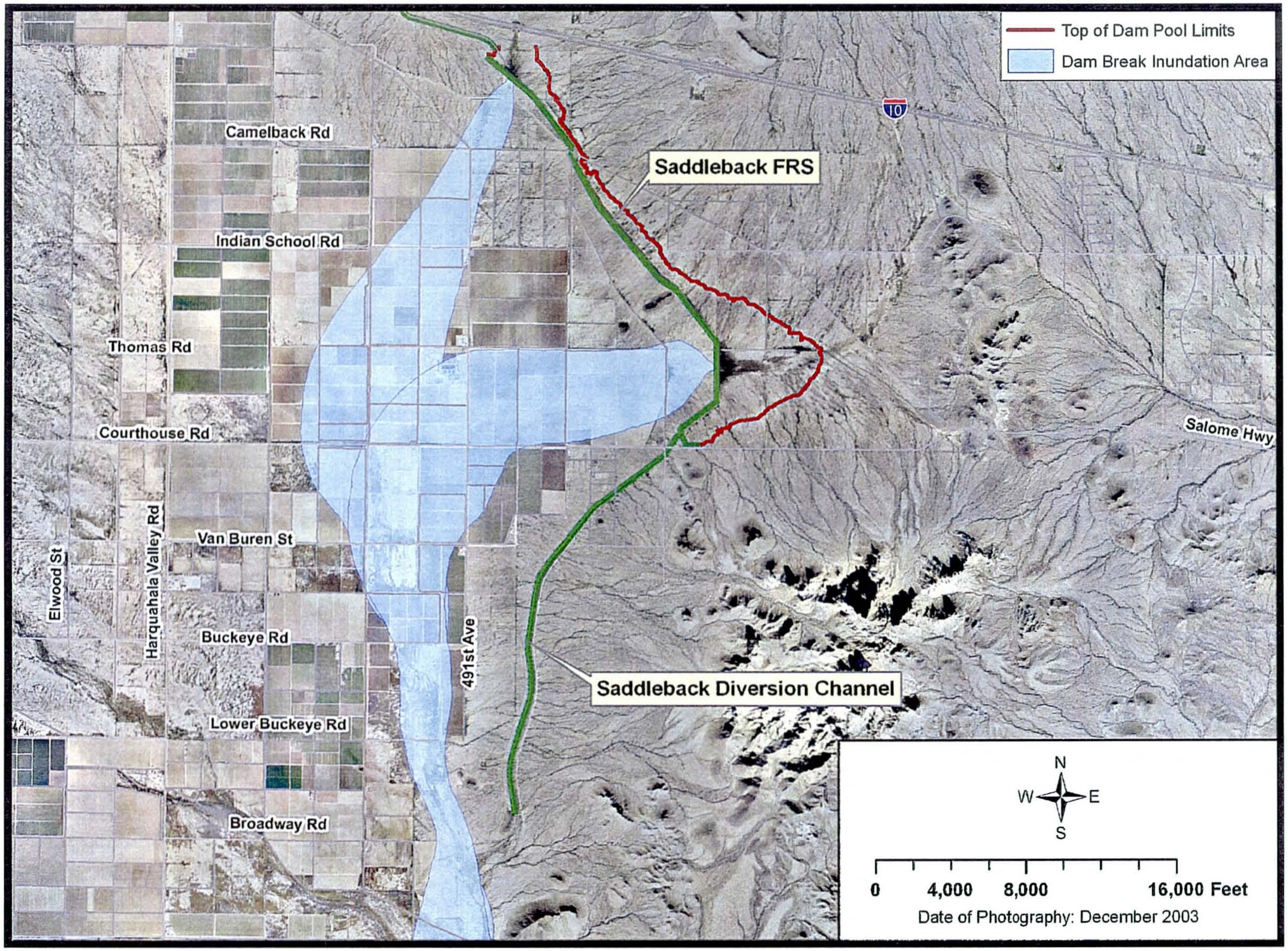

Figure 3. Saddleback FRS Dambreak Inundation Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn
and Associates, Inc.

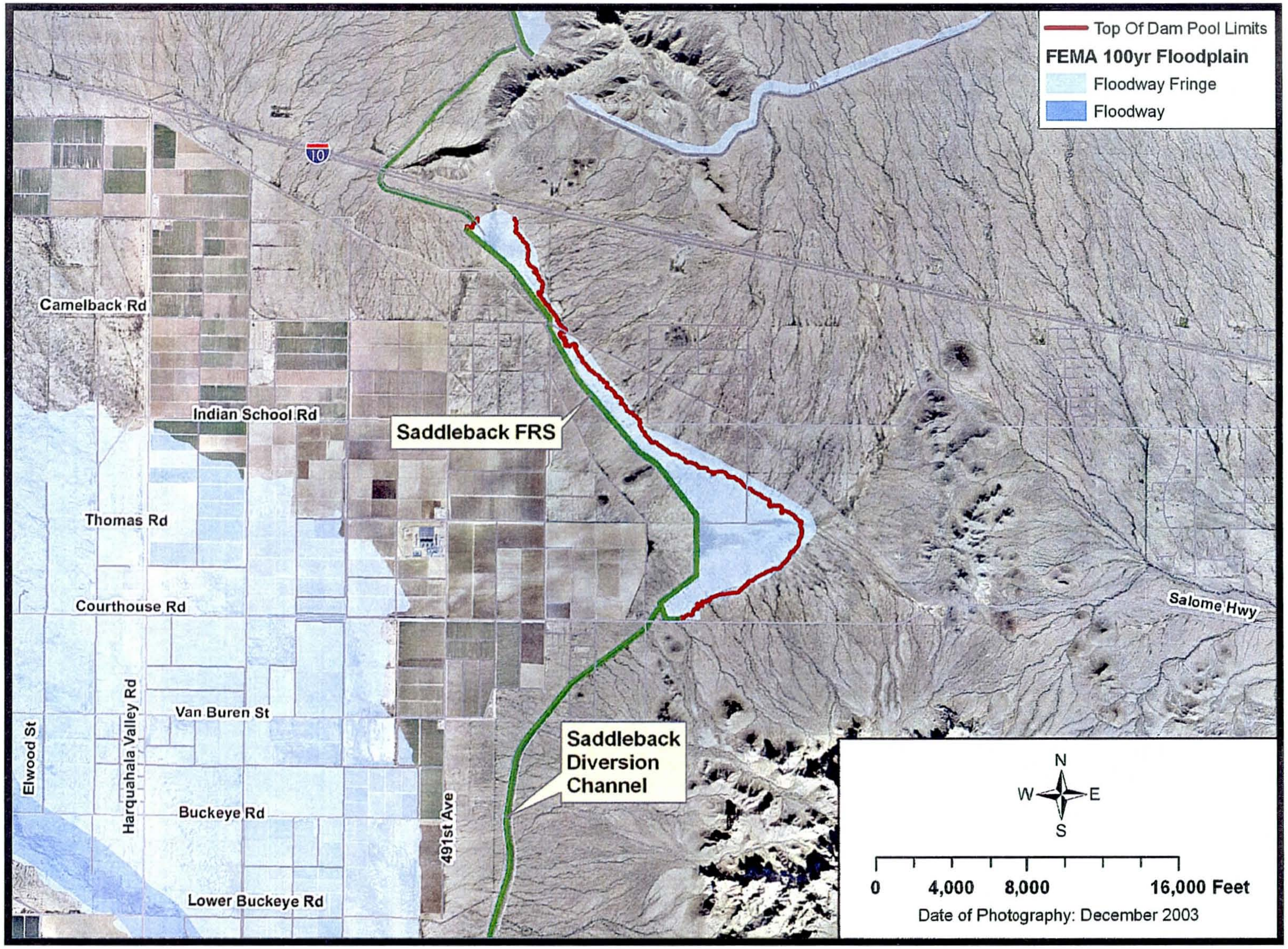

Figure 4. Saddleback FRS Top of Dam Pool Delineation.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

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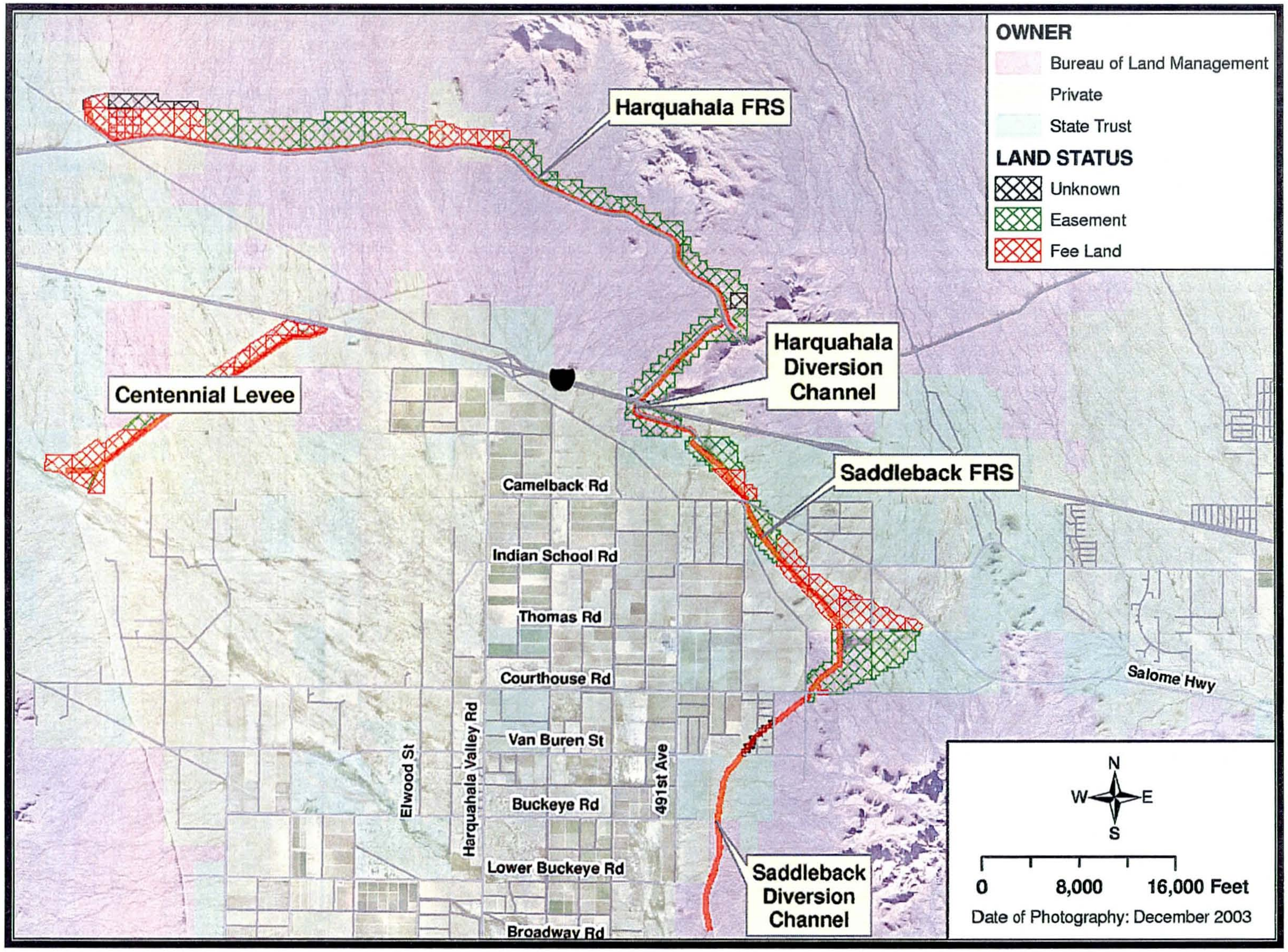

Figure 5. Landownership Map
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

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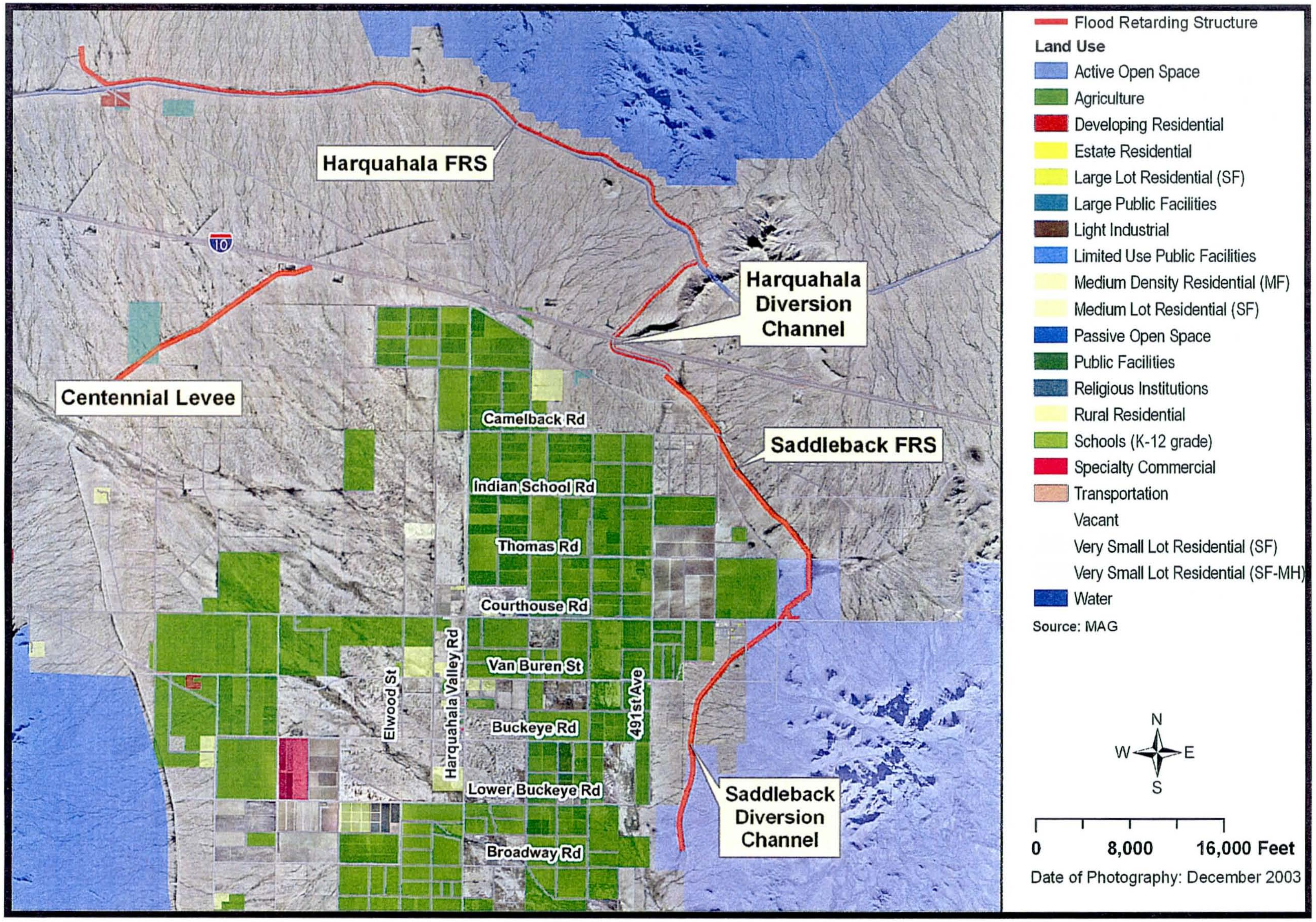

Figure 6. Current Land Use Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

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and Associates, Inc.

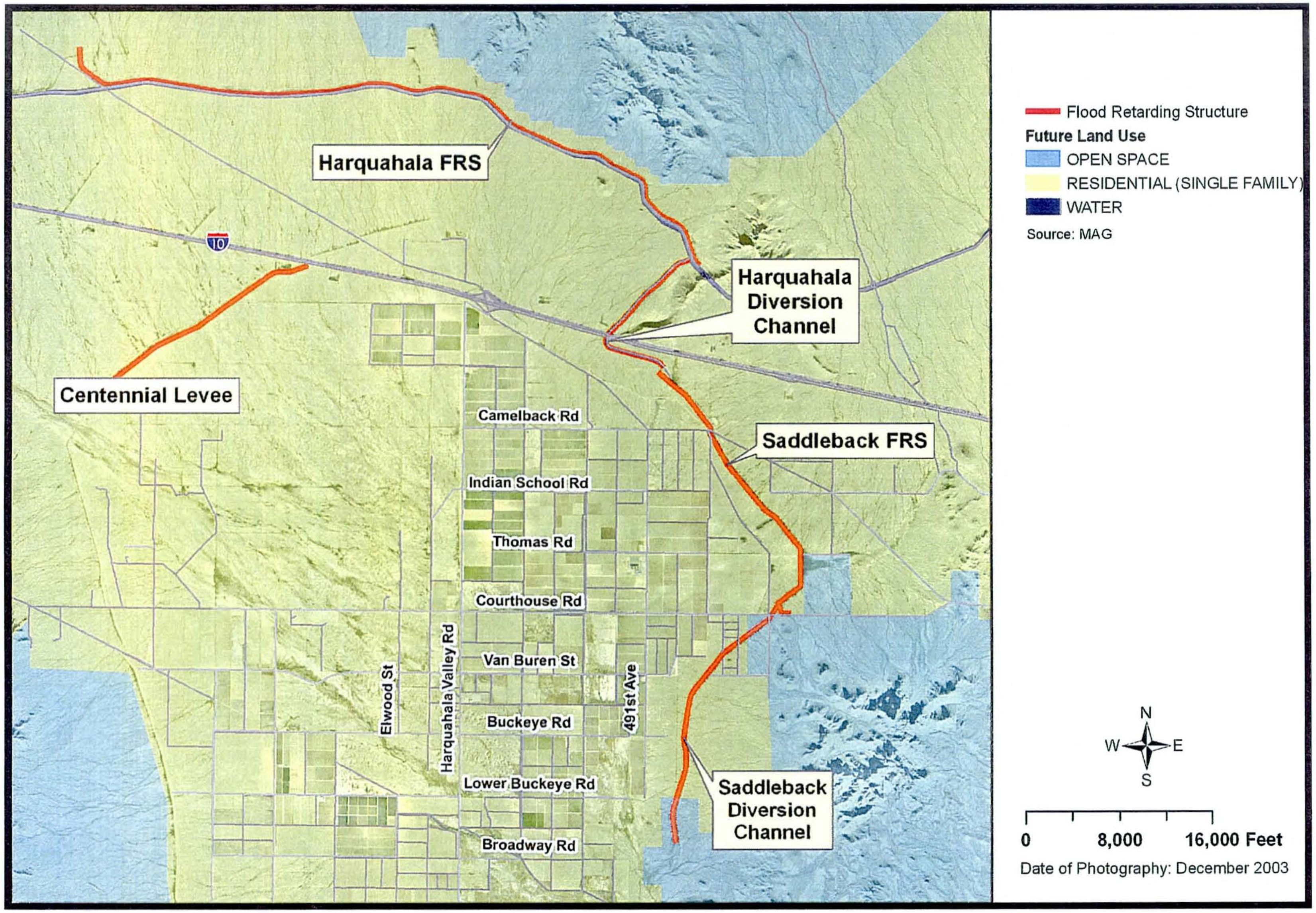

Figure 7. Future Land Use Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

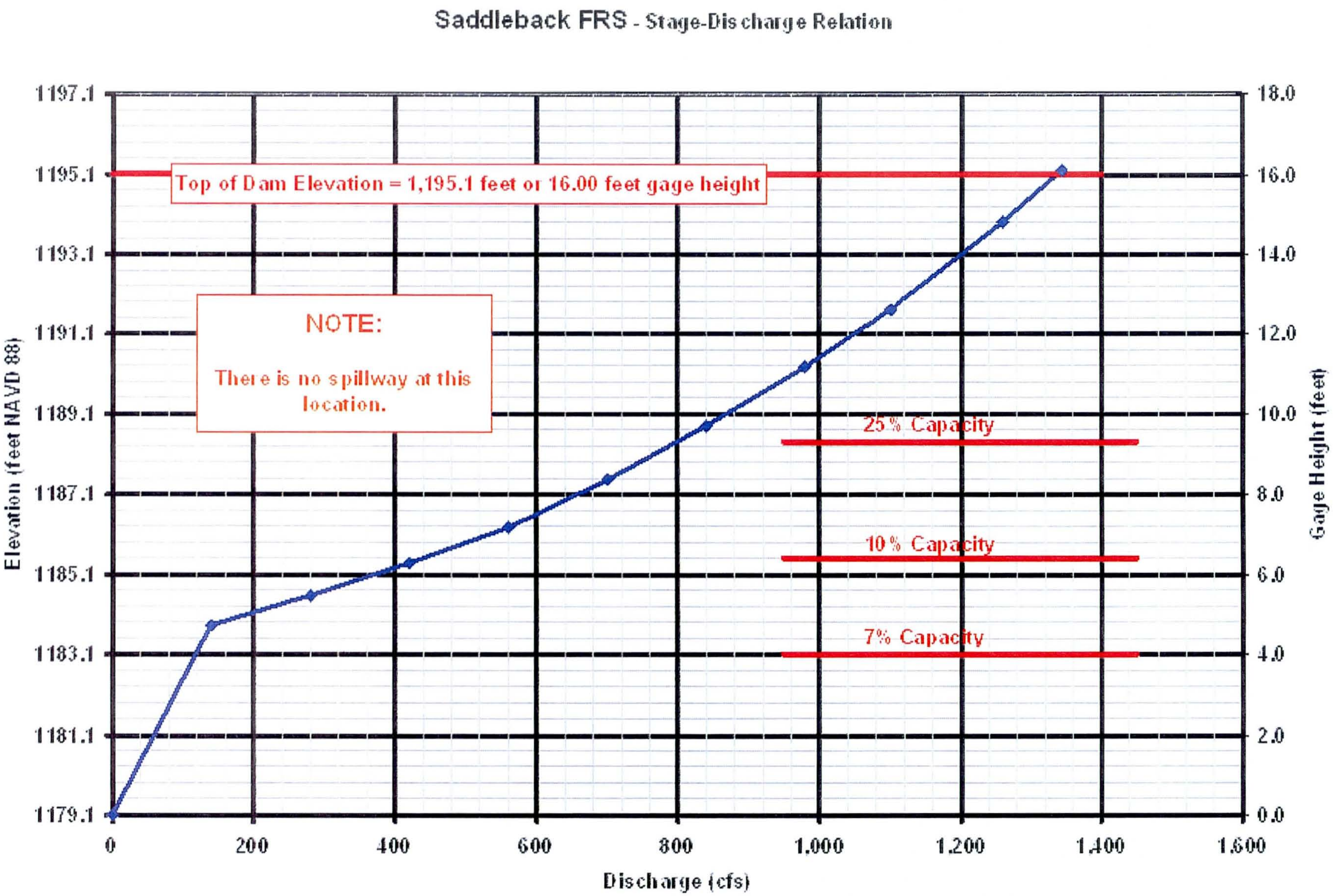

Figure 8. Stage-Discharge Rating Curve. (Source: District Website).

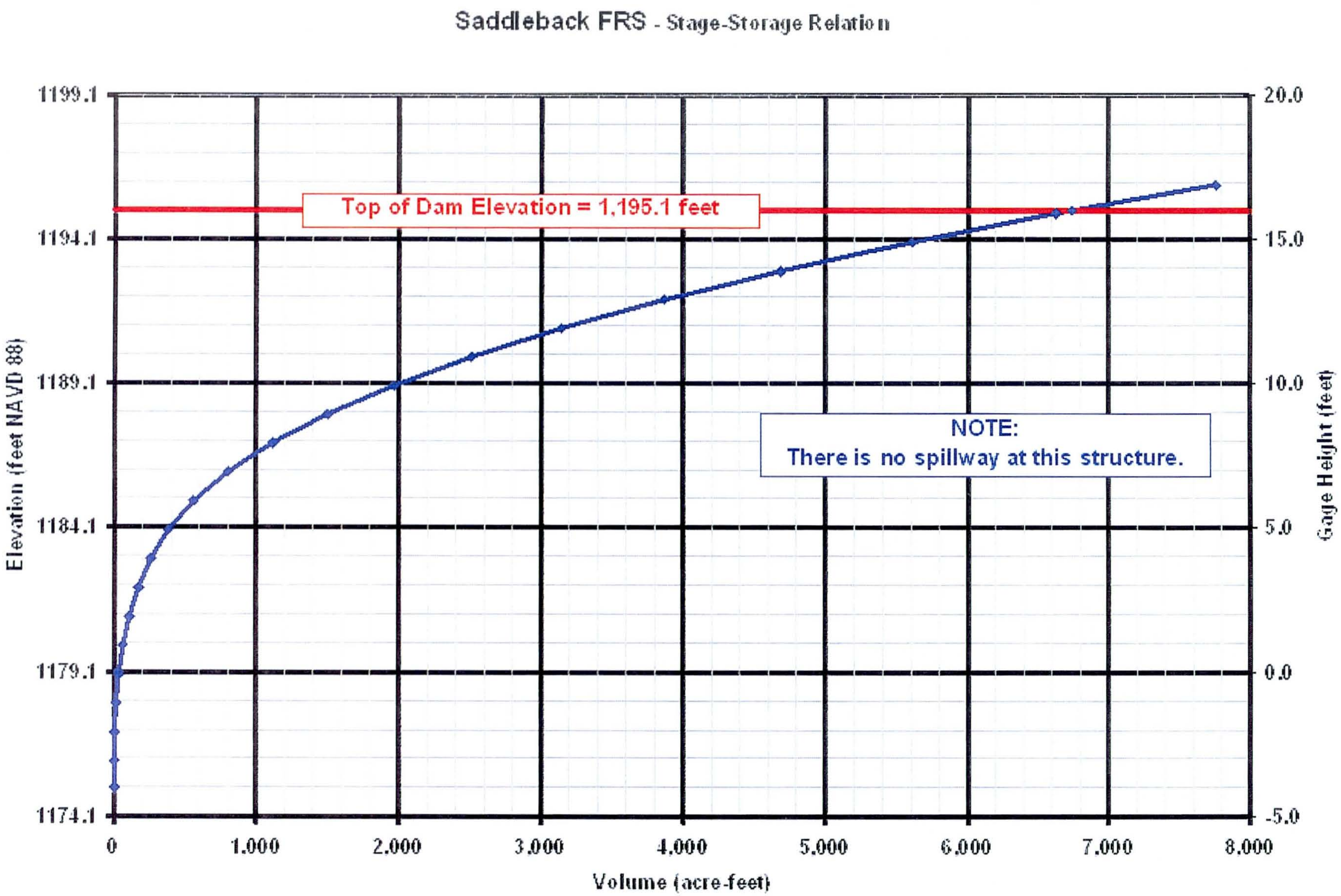

Figure 9. Stage-Storage Rating Curve. (Source: District Website).

Figure 10. Saddleback ALERT Gage Data Webpage

GEOLOGICAL CONSULTANTS INC

KENNETH M. EUGE, R.G.
JASON C. WILLIAMS, R.G.

Memorandum June 3, 2005

To: Mr. Bob Eichinger, P.E
Kimley-Horn and Associates, Inc.

From: Ken Euge, R.G.
Principal Geologist

REGISTERED GEOLOGIST CERTIFICATE NO. 10338 KENNETH M. EUGE Date Signed 11/03/05 U.S.A.

Subject: Geological Input to Structures Assessment Program, Phase 1
Saddleback FRS
Maricopa County, Arizona
FCDMC Contract No. FCD 2003C015
Geological Consultants Project No. 2003-161 (Work Assignment 3)

Geological Consultants, Inc. is pleased to submit the geologic, seismic and ground subsidence information for the Saddleback FRS structures assessment report.

1.0 Geologic Setting

The Saddleback F.R.S. is located within the Sonoran Desert section of the Basin and Range physiographic province. This portion of the Basin and Range is characterized by north and northwest trending mountains that rise abruptly to form broad, elongated, deep, sediment-filled valleys produced by block faulting, tilting and folding.

The structure lies in the east-central portion of the Harquahala Valley. The Harquahala valley is a northwest trending alluvial valley bounded on the north by the Harquahala Mountains, the northeast and east by the Big Horn and Saddle Mountains, the west by the Eagletail and Little Harquahala Mountains, and the south by the Gila Bend Mountains. The most prominent geologic feature near the Saddleback F.R.S. is Saddle Mountain to the south and southeast of the structure and Burnt Mountain to the north and northeast. Saddle Mountain is composed predominately of mid-Tertiary volcanic rocks with underlying Proterozoic crystalline rocks (Ort, 1993). Basalt and basaltic andesite are the most common volcanic rocks, along with a volcanic breccia. Proterozoic crystalline rocks include granodiorite and slaty metavolcanic rocks. Burnt Mountain is a late Tertiary volcanic center of mainly andesitic composition (Stimac, 1994).

The valley basin consists of late Tertiary and Quaternary deposits. The Saddleback F.R.S. lies primarily in Quaternary or Tertiary age old alluvium composed of caliche-cemented, unconsolidated to semi-consolidated sand and gravel deposits (ADWR, 2004). These deposits include a sedimentary sequence that varies in thickness from 0 to more than 5,000 feet and is generally divided into three units, the upper alluvial unit, the middle alluvial unit, and the lower conglomerate unit.

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The upper alluvial unit may range from 0 to greater than 1,300 feet in depth and is composed primarily of late Pliocene to recent deposits. The unit consists of unconsolidated sand and gravel with some interbedding of silt and clay (Bureau of Reclamation, 1976). The middle alluvial unit consists of fine-grained interbedded sand and silty clay overlying a silt and clay layer containing some reworked evaporates, over a layer of primarily evaporates containing minor silt and clay (Bureau of Reclamation, 1976). The middle alluvial unit varies in thickness and because of the proximity of the underlying volcanic bedrock, the middle alluvial unit probably does not underlie the Saddleback FRS. The lower conglomerate unit consists of pebble to cobble size, variably cemented clasts of middle to late Tertiary age (Bureau of Reclamation, 1976). This unit is the primary aquifer in the Harquahala Valley.

According to the SCS (1978), the Saddleback FRS traverses a bajada that consists of primarily of unconsolidated and semi-consolidated (caliche cemented) alluvial fan deposits derived from the Burnt Mountain area and unnamed hills to the east composed of schist and volcanic rock that overlie mixed older alluvial fan deposits and layered volcanic rock. Several ephemeral washes cross the FRS alignment. These channels are incised into the caliche cemented fan deposits and the channels have been subsequently filled with unconsolidated sand and gravel. The surface expression of many of these channels is subdued.

1.1 Dam Centerline Surficial Geology

The following describes the surficial geology along the centerline of the dam. These descriptions are excerpted from the SCS Report of Geologic Investigation for the Saddleback FRS (1978). The descriptions are deduced from the SCS site investigation that included sampling and logging of 20 backhoe pit and 19 drill holes.

Station 0+00 to 33+00: The surficial soils are described a loose to very loose silty gravelly sands to sandy gravels. Desert pavement is moderately well developed suggesting a relatively old and stable surface. The percentage of cobbles in the soils increases with depth. A loose surface horizon is about three feet thick and it is underlain by caliche cemented gravelly sands and sandy gravels with a small percentage of low plasticity fines. Cementation is moderate to strong.

Station 33+00 to 125+00: The soils along this portion of the FRS alignment are very loose to loose consisting of non-plastic silty sands to sandy silts with some fine gravel. The surface soil ranges from about 4 feet to 9 feet thick that are underlain by moderately to strongly cemented sandy silts, silty sand, clayey sand, and sandy clay with varying amounts of fine to coarse gravel. Several buried stream channels containing loose, highly permeable gravelly sands were found along this segment. Some of these loose deposits are interbedded with caliche cemented soil layers while other channels are incised into the cemented older alluvial fan deposits. Desert pavement is lacking along this segment suggesting the surface soils are relative young.

Around Station 115+00 the dam centerline passes between two volcanic rock outcrops. The surface soils in this area are underlain by cemented fanglomerate at a depth of about 6½ feet. Boring B-9 encountered pink porphryritic tuff beneath the fanglomerate. The contact boundary between the tuff and the fanglomerate drops off gradually to greater depths on both sides. At Station 117+14, the contact between the two units is about 13 feet below grade and at Station 112+78 it is about 12 feet below grade.

Station 125+00 to 280+00: Desert pavement is either very will developed or totally lacking along this segment of the alignment. The surface soils consist of loose to very loose sandy clays, silty sands, and clayey sands with varying amount of fine to coarse gravel. The surface soil zone is about 4 feet thick and overlies sandy clay, silty sand and gravelly sand that is well cemented with earthy caliche. Buried channel fill deposits containing loose to firm, permeable gravelly sand and sandy gravel are found locally along this segment.

1.2 Principal Spillway Surficial Geology

Geologic conditions in the Principal Spillway area are described as very loose to loose silty sand to an average depth of about 4 feet. Beneath the surficial soils (that were removed before the structure was constructed), the underlying material consists of consolidated, indurated caliche cemented fanglomerate. The degree of cementation reportedly increases with depth and the material becomes coarser grained grading into a cobble fanglomerate at a depth of 8½ feet. The soils in this zone had very high blow count values from standard penetration testing conducted in this area.

2.0 Seismic Evaluation

In 2002, a Seismic Exposure Evaluation was performed by AMEC Earth & Environmental, Inc. for the Dam Safety Program of the Flood Control District of Maricopa County. According to this report, the Saddleback F.R.S. lies within the Southern Basin and Range Source Zone. A seismicity evaluation conducted for the Arizona Department of Transportation describes this zone as the Sonoran Seismic Source Zone (Figure 3) (Euge, Schell, & Lam, 1992). This source zone appears to have a low level of seismicity and few active or potentially active faults. Within this source zone, the largest historical earthquake was a 1956 magnitude 5.0 event that occurred in the southern portion of the zone (AMEC, 2002).

The closest active fault to the Saddleback F.R.S., Sand Tank Fault, is approximately 83.3 miles southeast of the structure (Figure 3). Sand Tank Fault lies in south-central Maricopa County, east of the town of Gila Bend. Sand Tank Fault is a normal fault with a slip rate of less than 0.02 millimeters per year and a recurrence interval of approximately 100,000 years (AMEC, 2002). This fault may be capable of producing an maximum credible earthquake of magnitude of 5.7 and an associated maximum peak horizontal acceleration at the Saddleback

F.R.S. equal to 4 percent of the gravitational acceleration (g) (AMEC, 2002). The recommended peak horizontal acceleration design criteria calculated by AMEC for the Saddleback F.R.S. is 0.10 g. Figure 4, the Horizontal Acceleration Map (from Euge et al, 1992), shows a 0.03 g horizontal acceleration of bedrock with 90 percent probability of non-exceedance in 50 years in the vicinity of the Saddleback FRS.

3.0 Land Subsidence

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

3.1 Groundwater

The major human-induced factor contributing to subsidence is the large scale pumping and removal of groundwater. Nearly all of the populated southern Arizona basins from Phoenix to Tucson have experienced at least a 100+ foot drop in groundwater level, and an area surrounding the town of Stanfield, Arizona has dropped more than 500 feet (Schumann, 1986).

3.1.1 Groundwater in the Harquahala Groundwater Basin

The Saddleback F.R.S. is located in the Harquahala groundwater basin in west-central Arizona. The lithology of the basin varies widely, but is generally composed of a heterogeneous mixture of clay, silt, sand and gravel (Corkhill, 1998). The alluvium may range from 0 feet deep at the base of the mountains to more than 5000 deep in the center of the basin. The alluvial deposits grade from coarse-grained sand and gravel in the southeast to fine-grained deposits in the center of the basin. Fine-grained clay deposits, over 1000 feet thick, occur in the western part of Township 2 North, Range 9 West (Corkhill, 1998). The fine-grained beds grade toward the west into an alternating sequence of fine-grained and coarse-grained layers from 800 to 850 feet thick, overlying a conglomerate unit.

The main use of groundwater in the Harquahala basin is for agricultural purposes. Prior to 1951, groundwater in the basin flowed from the northwest to southeast. By 1963, three cones of depression had developed in the southeastern part of the basin which, by 1966, had coalesced into one large cone in the center of the valley (ADWR, 2005). By 1986, the basin had experienced a decline in the groundwater level in some areas of as much as 300 to 500 feet (Schumann, 1986).

3.1.2 Groundwater in the Project Vicinity

Hydrographs for 26 wells within approximately 2.5 miles of the Saddleback F.R.S. were obtained from the Arizona Department of Water Resources, with the oldest dating back to 1957 (Appendix A) (Figure 5). These hydrographs show an overall decline in groundwater levels of between 49 and 280 feet. From the early to mid 1980's, the wells in this area have experienced an increase of between 28 and 220 feet, but have not recovered to pre-pumping levels.

3.2 Regional Subsidence

Prior to the utilization of groundwater in south-central Arizona, the water table was higher and hydrogeological conditions were in equilibrium. Water levels within the aquifers were lowered when pumping was initiated and the basin fill sediments were dewatered. In the arid southwest, the water in the aquifer may be removed by pumping faster than it can be naturally replenished causing a net water table decline. As a result, the weight of the soil column is gradually increased as the buoyant effects and aquifer pressures induced by the water acting on the soil column are decreased. This condition causes increased loading stresses to consolidate portions of the thick compressible sediments that result in the lowering (subsidence) of the land surface over a large area.

Land subsidence was first documented in Arizona in 1934 following the releveling of first-order survey lines by the Coast and Geodetic Survey (now the National Geodetic Survey (NGS)). Subsequent leveling by the NGS, the U.S. Geological Survey, the Bureau of Reclamation, and the ADOT has documented substantial land surface subsidence in south-central Arizona including the Salt River Valley, the Queen Creek-Apache Junction area, the Eloy-Casa Grande-Stanfield area, and the Harquahala valley area as overdrafting of the aquifer continues.

Subsidence and earth fissures in urban areas can cause a variety of problems. Structures built across fissures may be damaged, street may crack, flow in gravity water and sewer lines can be reversed, and differential subsidence (although rare) can rupture buried utilities (Arizona Geological Survey, 1987). However, design measures can be implemented to mitigate the effects of land subsidence. Some of these measures can include additional structural reinforcement, over-sized pipes, surface drainage controls, bridging the subsidence feature, and avoidance.

3.2.1 Study Area Subsidence

Historic National Geodetic Survey (NGS) level line data is not available in the vicinity of the Saddleback F.R.S. However, recent historic subsidence-settlement is available from the Flood Control District of Maricopa County using crest and toe monument elevations recorded between 1984 and 2003. A summary of the settlement that has occurred along the dam is shown in Table 1 (FCDMC, 2004) and Figure 6.

According to this data, it appears that negligible settlement or subsidence has occurred across most of the dam and a very minimal amount is recorded on one crest section between A-5 and A-10, A-21 and A-23 and the toe at B-25, from 1983 to 2003 (Figure 6). The change in elevation in this area ranges from -0.001 to -0.300 feet. Overall the entire Saddleback FRS appears to be relatively stable in terms of settlement and subsidence. This is not surprising

Table 1
Change in Elevation 1983 – 2003 (adjusted to 1983 datum)

Crest Marker	Change in Elevation (feet)		Toe Marker	Change in Elevation (feet)
A-1	0.070		B-1	0.162
A-2	0.057		B-2	0.211
A-3	0.022		B-3	0.170
A-4	0.005		B-4	0.144
A-5	-0.013		B-5	0.198
A-6	-0.098		B-6	0.092
A-7	-0.105		B-7	0.050
A-8	-0.035		B-8	0.151
A-9	-0.048		B-9	0.152
A-10	-0.014		B-10	0.084
A-11	0.048		B-11	0.149
A-12	0.036		B-12	0.099
A-13	0.057		B-13	0.258
A-14	0.026		B-14	0.054
A-15	0.017		B-15	0.101
A-16	0.035		B-16	0.136
A-17	0.012		B-17	0.053
A-18	0.011		B-18	0.121
A-19	0.071		B-19	0.161
A-20	0.033		B-20	0.212
A-21	-0.001		B-21	0.080
A-22	-0.010		B-22	0.092
A-23	-0.038		B-23	0.064
A-24	0.046		B-24	0.152
A-25	0.048		B-25	-0.300
A-26	0.082		B-26	0.145
A-27	0.142		B-27	0.150

(Flood Control District of Maricopa County, Dam Safety Program, 2004) when considering the structure is located within the volcanic bedrock pediment area that is overlain by well cemented fanglomerate.

The minimal subsidence recorded from 1983 to 2003, along with the static or increasing groundwater levels in the area would suggest that future land subsidence in the vicinity of the Saddleback F.R.S. would be minimal. This is subject to change if increased pumping of the groundwater caused the water level to decline thereby increasing land subsidence.

3.3 Earth Fissures

Fissures occur in unconsolidated sediments, typically near the margins of alluvial valleys or near the bedrock pediment edge where land water levels have dropped from about 200 feet to 500 feet below land surface (Schumann, 1986).

Fissures are initiated deep underground when tensile stresses exceed the strength of the soils. Tensile stresses induced by the subsidence continue to increase until the ground breaks to form earth fissures. The fissure then propagates upwards to intersect the ground surface. Examples of typical earth fissure characteristics are provided in Figure 7. Early signs of earth fissuring are small, en echelon, hairline cracks and irregular spaced depressions at the surface. As fissures develop the cracks grow in length to create fissures 1 foot to more than 10 feet deep when subject to erosion caused by surface runoff. The fissures often have vegetation growing in them because the ground is commonly moister along the earth fissure. Other physical features associated with fissure are slump-related escarpments from one inch to a few inches in height, as well as a drainage pattern associated with the fissure that does not conform to the areas local drainage pattern.

Field evidence indicates fissures propagate upward and are exposed after overlying sediments are eroded by surface water runoff from rainfall or irrigation (Pewe, 1982). The surface expressions of the fissures are exaggerated because the initial hairline crack is attacked by water to create wide (10 to 20 feet) and deep (more than 15 feet) erosional gullies that often have vegetation growing in them. The fissures are commonly perpendicular to natural drainage channels. The length of the fissure at the ground surface varies, usually less than one mile but one fissure near Picacho is more than 9 miles long. These features are easily recognizable on aerial photographs and in the field except where the ground surface is modified by agricultural activities or urban development.

A regional gravity survey was conducted that included the Saddleback F.R.S. vicinity (Oppenheimer, 1980). The Oppenheimer map estimated the depth to crystalline bedrock under the study area to be from 400 to 600 below ground surface, with the depth to bedrock depth increasing away from the mountain front. The depths to the volcanic bedrock ranges from nil where exposed at the ground surface to probably less than 400 feet below ground surface.

Figure 8 is a modified Bouguer Anomaly map and a modified Structure Contour Map, from the Bureau of Reclamation, Geology and Groundwater Resources Report (1976). As depicted in Figure 8, a relatively prominent bedrock boundary condition can be deduced that reflects the approximate buried limit of the volcanic rock west of the Saddleback FRS. It is possible that this boundary between the volcanic bedrock and the basin fill alluvial sediments could be the focus for earth fissure development; however, the trend of this boundary does not appear to cross the Saddleback FRS alignment. Therefore, it is unlikely that earth fissures could develop that would adversely impact the Saddleback FRS.

3.3.1 Known Earth Fissures in the Project Vicinity

There have been three earth fissures reported in the Harquahala Valley. The closest fissure to the Saddleback F.R.S. lies approximately 2.5 miles west of the structure in Section 36, Township 2 North, Range 9 West (Figure 9). There is no current information on the status of this fissure. An examination of recent aerial photographs of the area did not display any feature that would be indicative of the fissure. This is probably due to the fact that the reported fissure is located in an agricultural area and any surface expression of an earth fissure would be destroyed during agricultural activity.

Another fissure lies approximately 4.7 miles northwest of the structure in Section 9, Township 2 North, Range 9 West. This fissure was first discovered in 1958, visible in an aerial photo. The fissure was examined in 1978 and appeared to have been dormant for many years (Graf, 1980).

The Rogers fissure was discovered in 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 11 miles west of the dam, when it made an abrupt appearance during an unusually heavy rainfall event. The fissure is approximately 4,400 feet long, averages 5 to 15 feet deep and 5 to 10 feet wide, with prominent near vertical side slopes (Photos 1 & 2) (Corkhill, 1998). Development of the surface expression of the Rogers fissure was unusual in that there were no reported precursor features, such as small surface cracks, aligned potholes, linear depressions or linear vegetation, in the area that would have indicated the fissure was present.

In 2001, another earth fissure appeared suddenly, following a heavy rain. This fissure appeared in the West Salt River Valley, west of the Palo Verde Generating Station. This fissure is about 14.4 miles southeast of the Saddleback F.R.S.

Photo 1: View of Rogers earth fissure with gulley headcutting upslope along the fissure alignment.

Photo 2: Well developed fissure gulley along portion of Rogers earth fissure. Note slump blocks in bottom center of view generated from the tabular failure of the over-steepened fissure side slopes.

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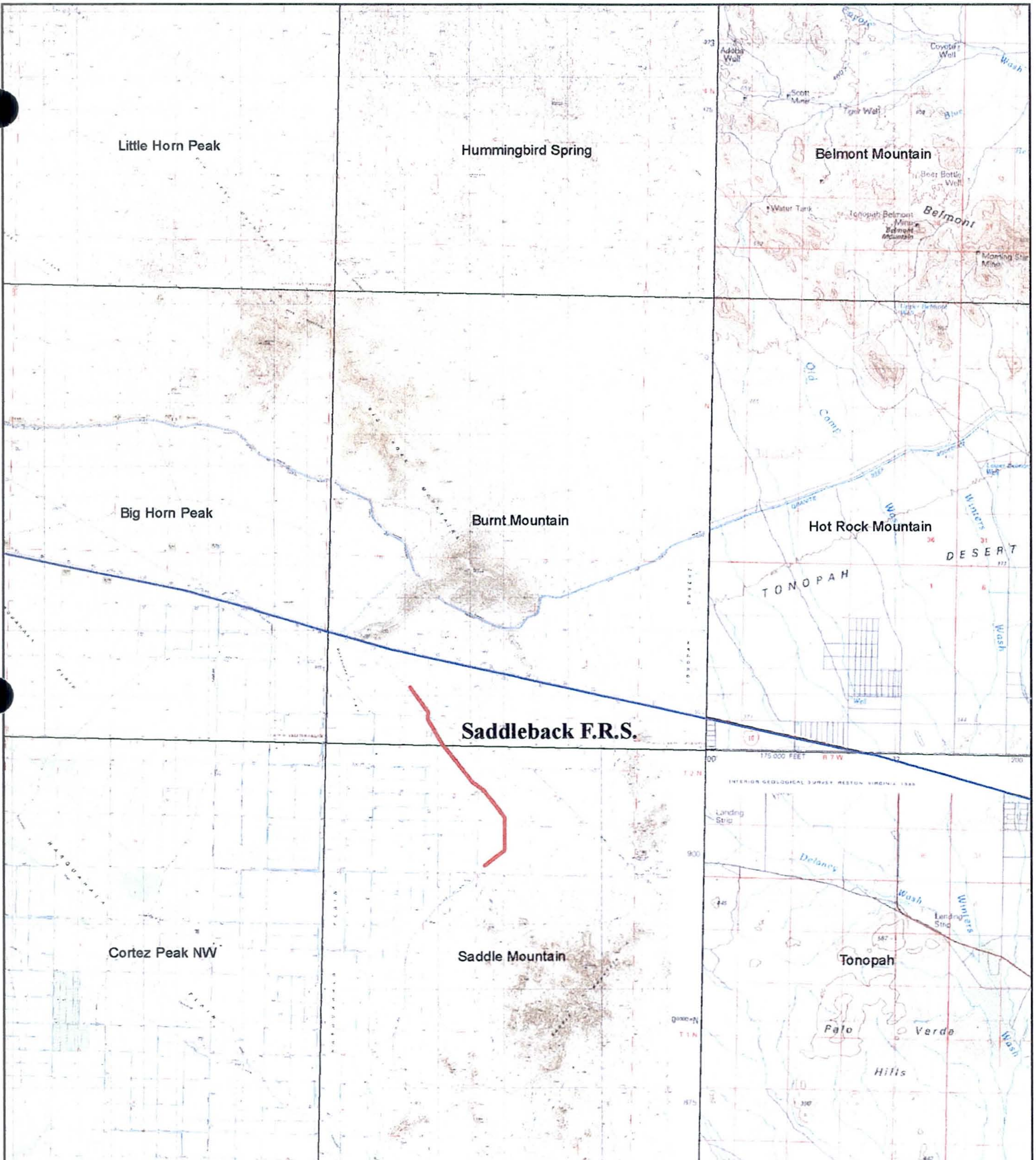

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Saddleback F.R.S.
Location Map
Figure 1

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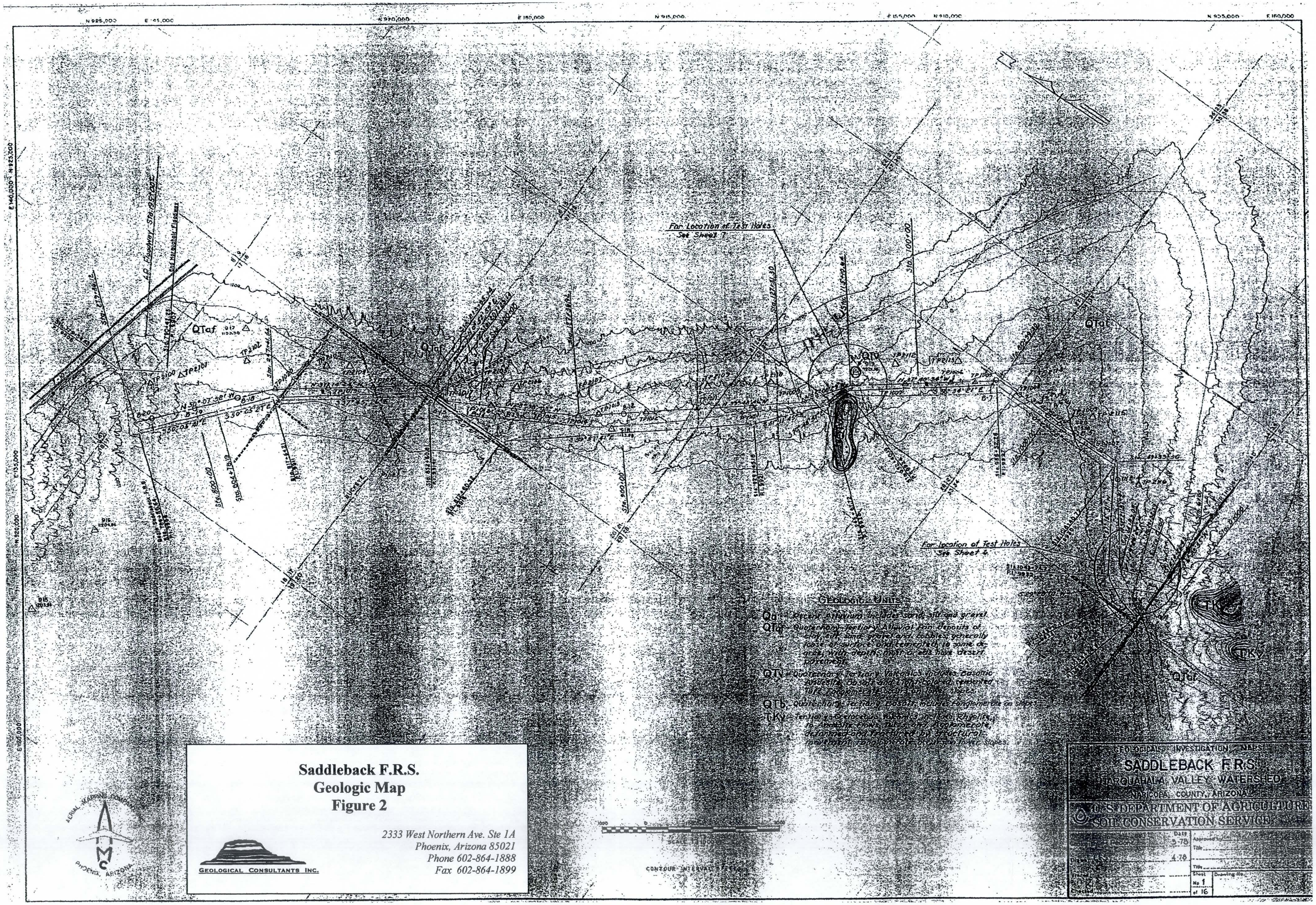

Saddleback F.R.S.
Geologic Map
Figure 2

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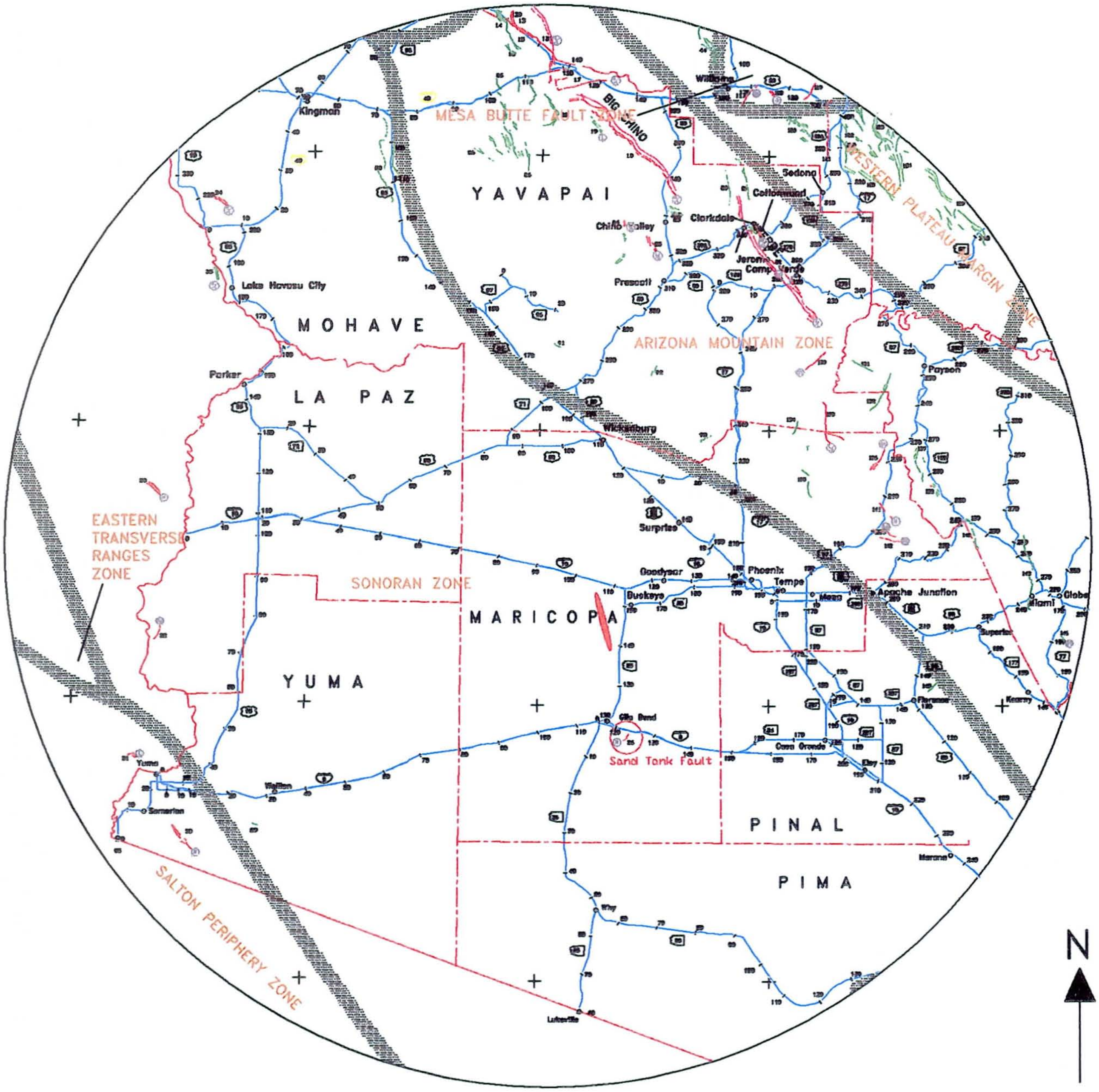


EXPLANATION

Q 90 — Quaternary-age Fault, letters in circle indicate age of most recent displacement according to table; numbers represent fault (identification no. used in referenced report)

Q – Quaternary < 2 my B.P.

Qy – late Quaternary < 0.5 my B.P.

	Approximate Age (million years before present)
h – Late to mid Holocene	< 0.005
H – Early Holocene to Late Pleistocene	0.005– 0.02
L – Late Pleistocene	0.02 – 0.15
M – Mid Pleistocene	0.15 – (0.5–0.7)
E – Early Pleistocene	(0.5–0.7) – 2

45 — Neotectonic Fault of uncertain age, generally of late Pliocene or older Pleistocene age

Seismic source zone boundary

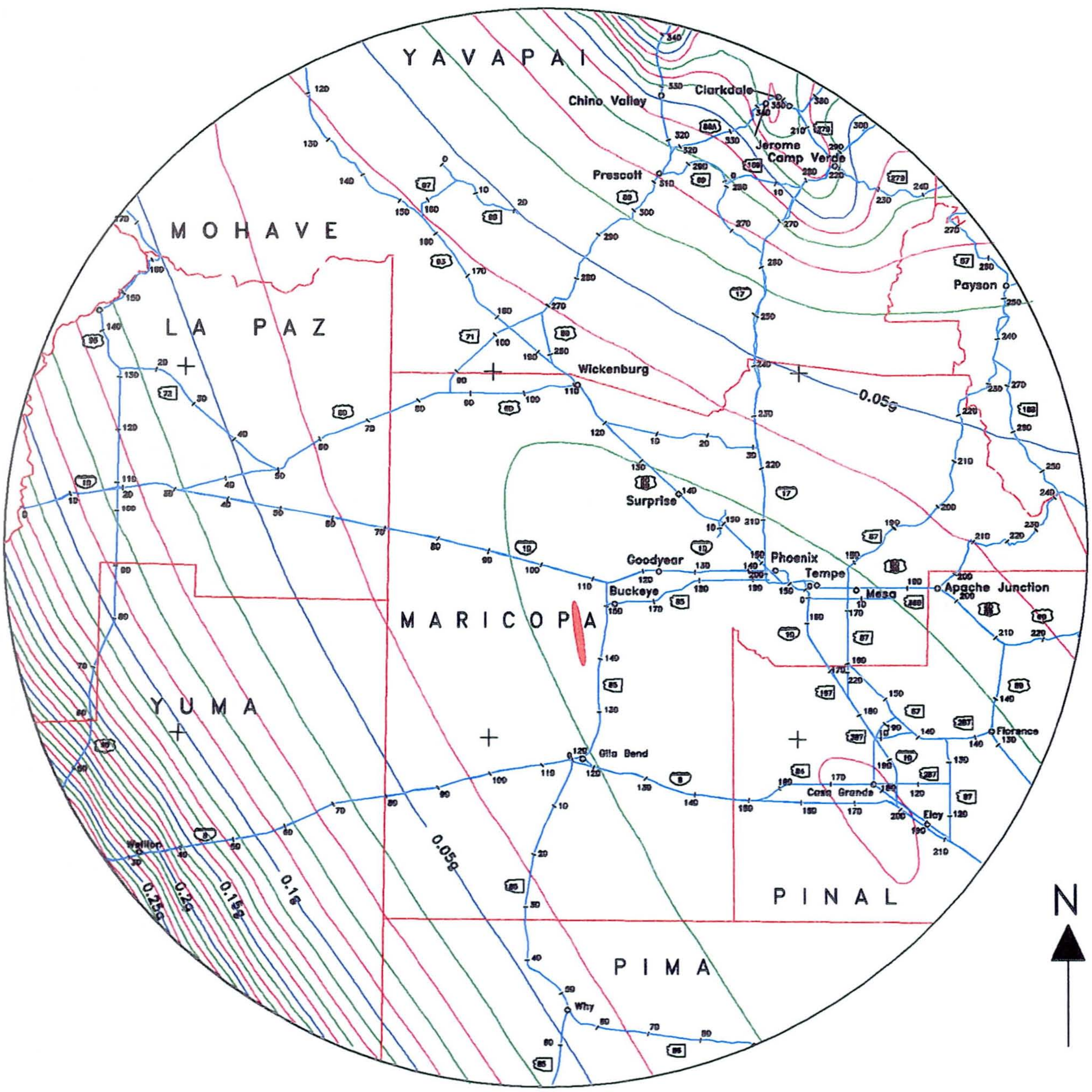
Approximate Area of Harquahala F.R.S.

(modified from: Fault Map of Arizona Area by Bruce A. Schell, Kenneth M. Ezgo, and Ignatius Po Lam, 1993)

Saddleback F.R.S.
Fault Map
Figure 3

GEOLOGICAL CONSULTANTS INC.

2333 West Northern Avenue, Suite 1A
Phoenix, AZ 85021
phone 602-864-1888
fax 602-864-1899



Legend

Approximate location of Saddleback F.R.S.

(modified from: Map of Horizontal Acceleration at Bedrock for Arizona with 90 Percent Probability of Non-Exceedance in 50 Years by Bruce A. Schell, Kenneth M. Euge, and Ignatius Po Lam, 1992)

Saddleback F.R.S.
Horizontal Acceleration Map
Figure 4

Geological Consultants Inc.

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Phoenix, AZ 85021
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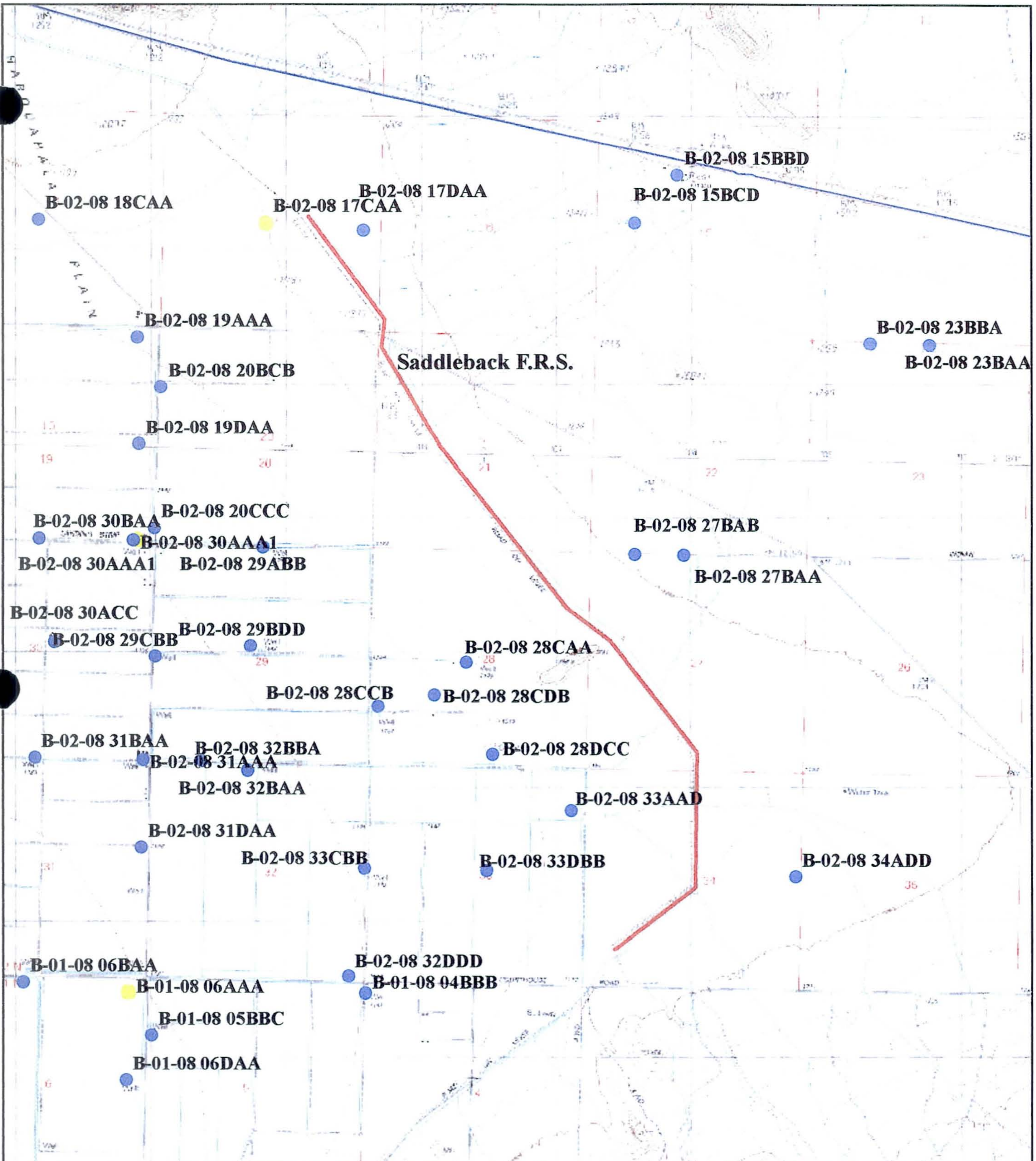
fax 602-864-1899



Legend

- Well
- Index Well

(from ADWR database CD-ROM, 2004)

N

1:40,000

Saddleback F.R.S.
Well Location Map
Figure 5

GEOLOGICAL CONSULTANTS INC.
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Phoenix, AZ 85021
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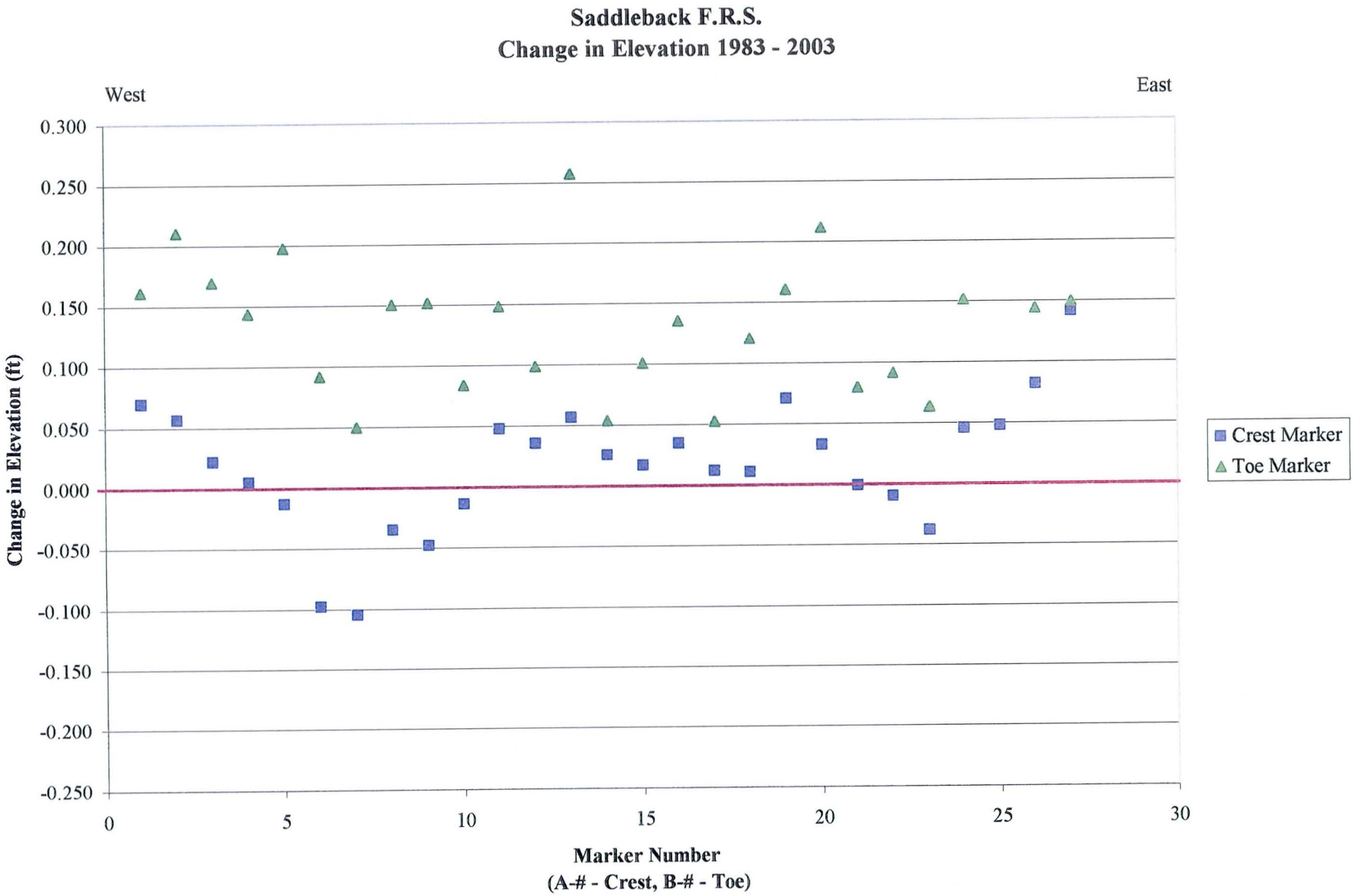
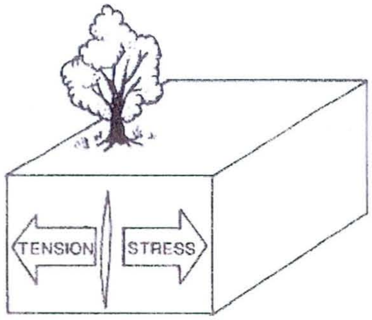
fax 602-864-1899

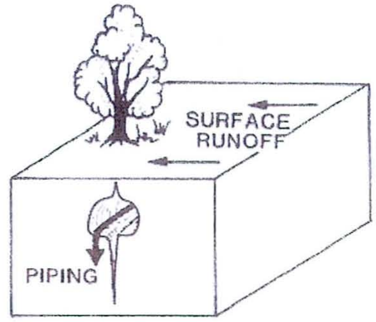


Saddleback F.R.S.
Geological Consultants Project No. 2003-161
Figure 6

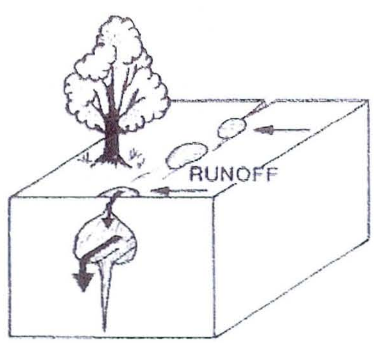
(Data from Flood Control District of Maricopa County, 2004)



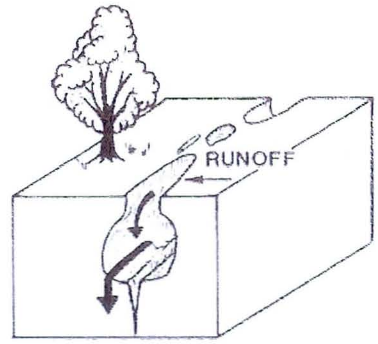
1. Lateral stresses induce tension cracking



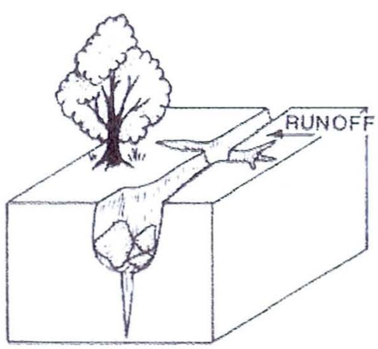
2. Surface runoff and infiltration enlarge crack through subsurface piping



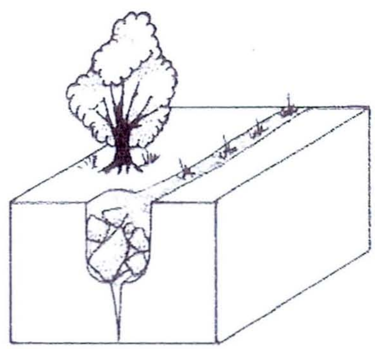
3. As piping continues, fissure begins to appear at surface as series of potholes and small cracks



4. As infiltration and erosion continue, fissure enlarges and completely opens to surface as tunnel roof collapses



5. The entire fissure is opened to the surface and enlargement continues as fissure walls are widened, extensive slumping and side-stream gullying occur



6. Fissure becomes filled with slump and runoff debris and is marked by vegetation lineament and slight surface depression, it may become reactivated upon renewal of tensile stress

Figure from Pewe, 1982

Saddleback F.R.S.
Generalized States of Earth Fissure Development
Figure 7

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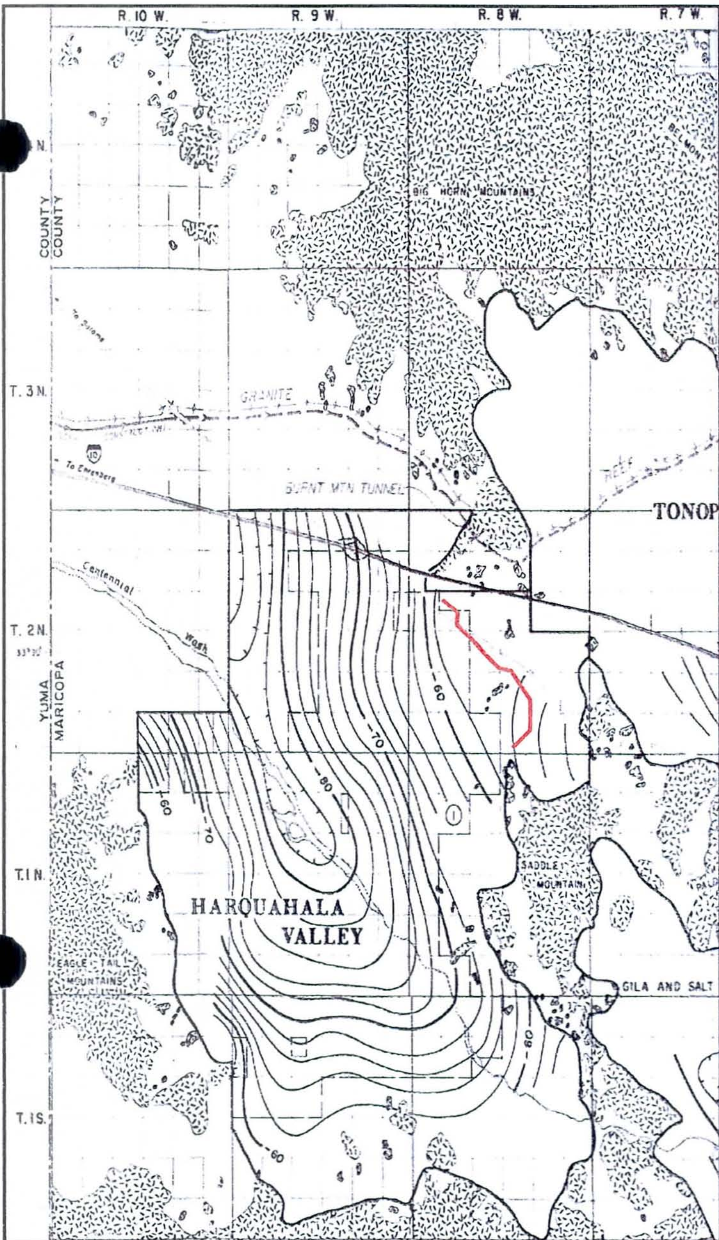
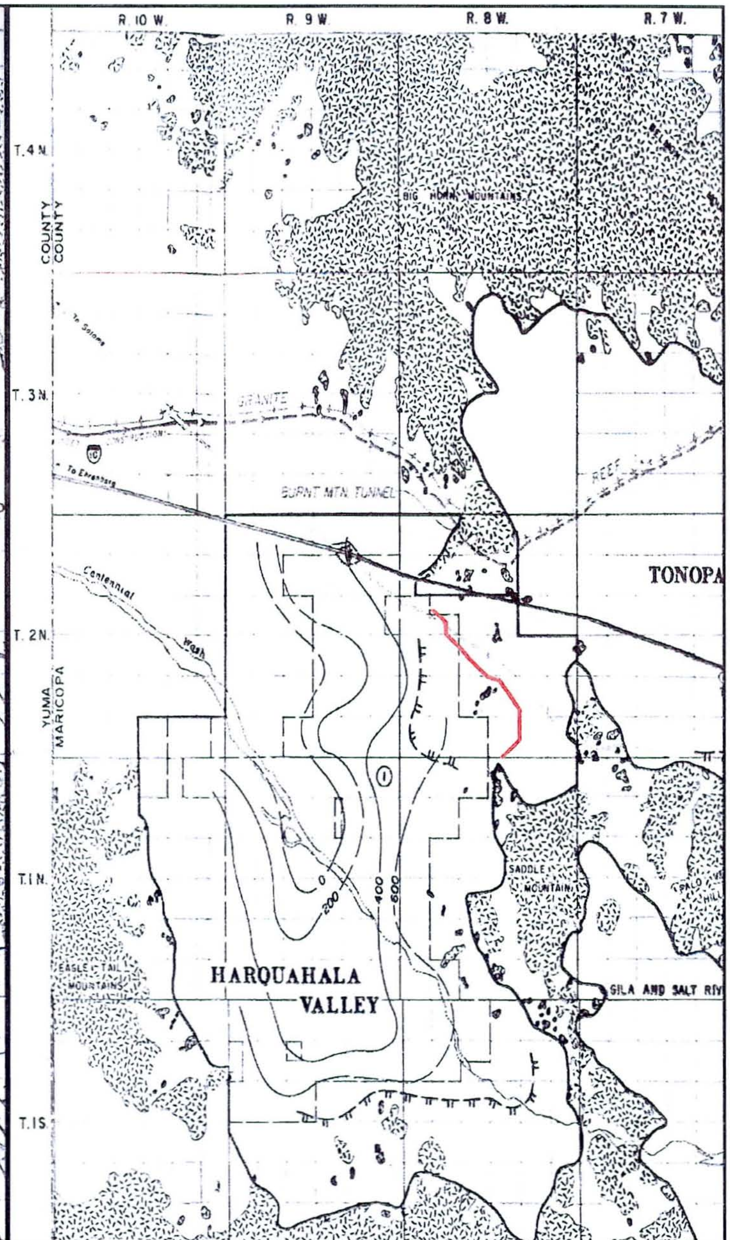
fax 602-864-1899



Bouguer Anomaly Map* showing gravity contours (dashed where inferred), contour interval 2 milligals.



Structure Contour Map* showing generalized structure contours of top of Lower Conglomerate Unit (dashed where inferred) contour interval 200 feet.

Approximate subsurface extent of Quaternary Tertiary volcanic rock adjacent to or interbedded with the Lower Conglomerate Unit. (BOR, 1976)

*Modified from Central Arizona Project Geology and Groundwater Resources Report, Maricopa and Pinal Counties, Arizona; United States Department of the Interior, Bureau of Reclamation, Lower Colorado Region, Volume 1, December 1976.

Saddleback F.R.S.
Bouguer Anomaly and Structure Contours
Figure 8

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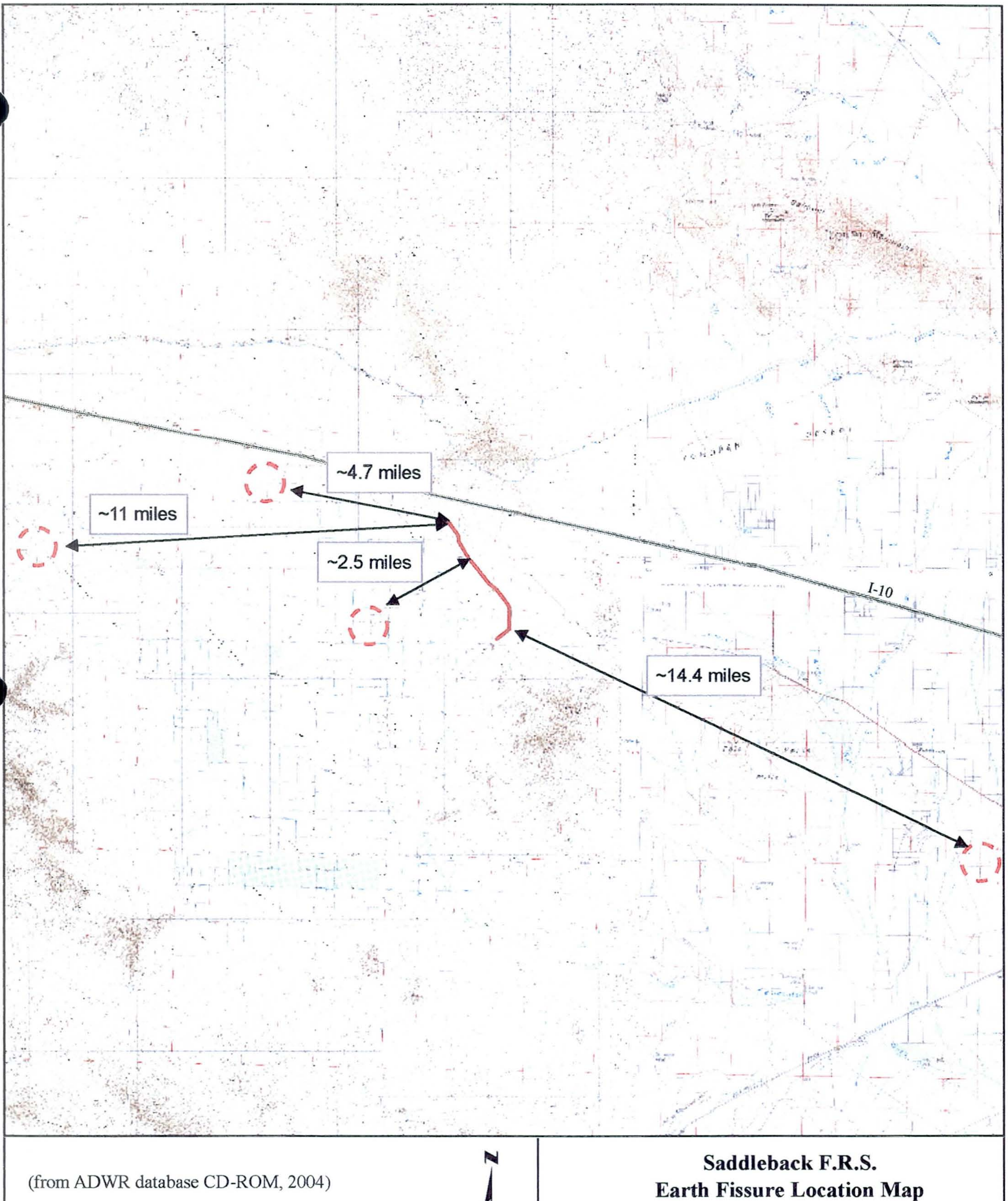
Phoenix, Arizona 85021
Phone 602-864-1888
Fax 602-864-1899



(from ADWR database CD-ROM, 2004)

N

1:246,202

Saddleback F.R.S.
Earth Fissure Location Map
Figure 9

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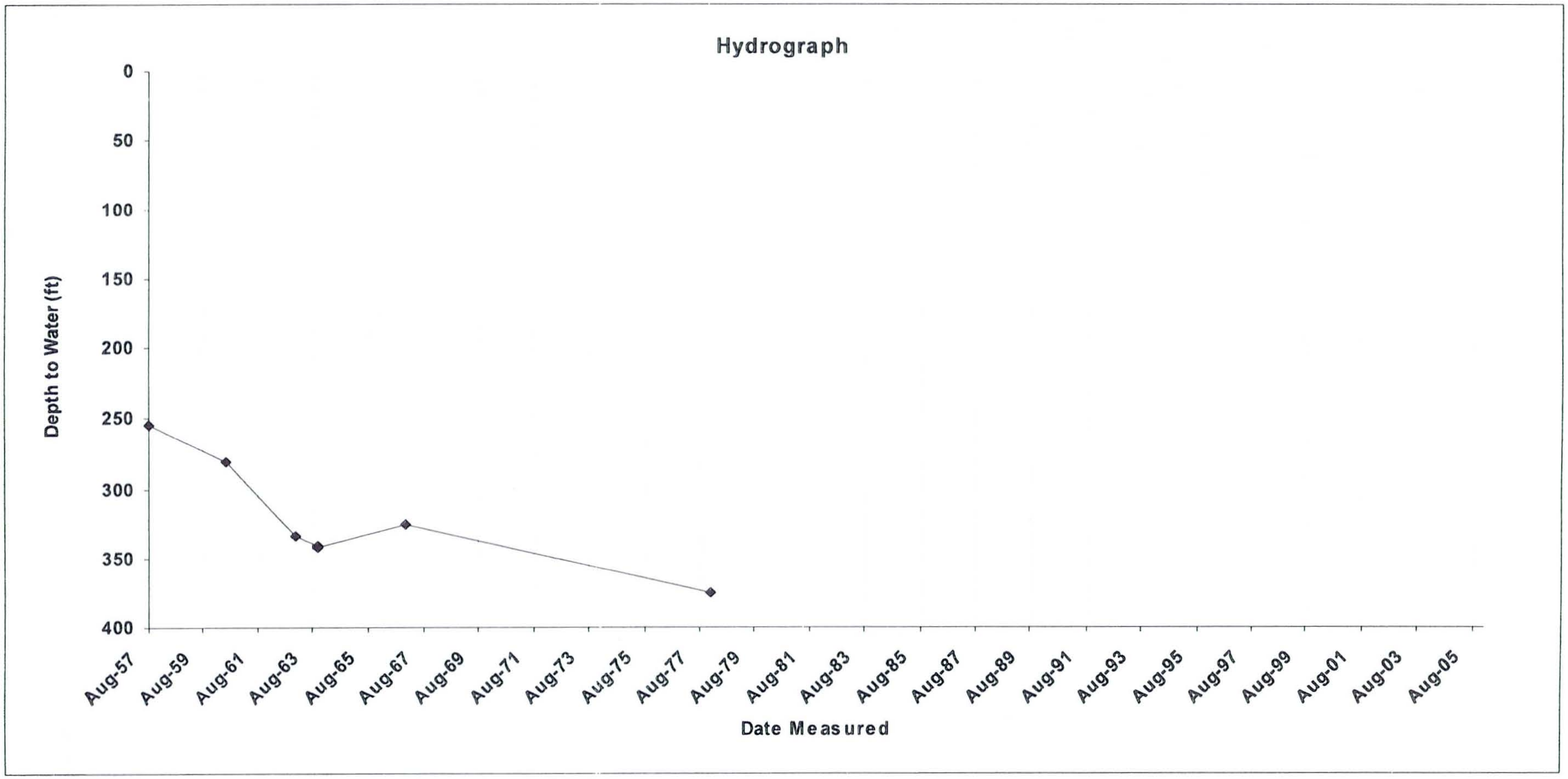
Appendix A

GWSI Well Report and Hydrograph

AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 27BAA	332937113040201	643348	33° 29′ 37″	113° 4′ 2″	IRRIGATION	868	11/1/1963	16	1/18/1978	375.00	829	7

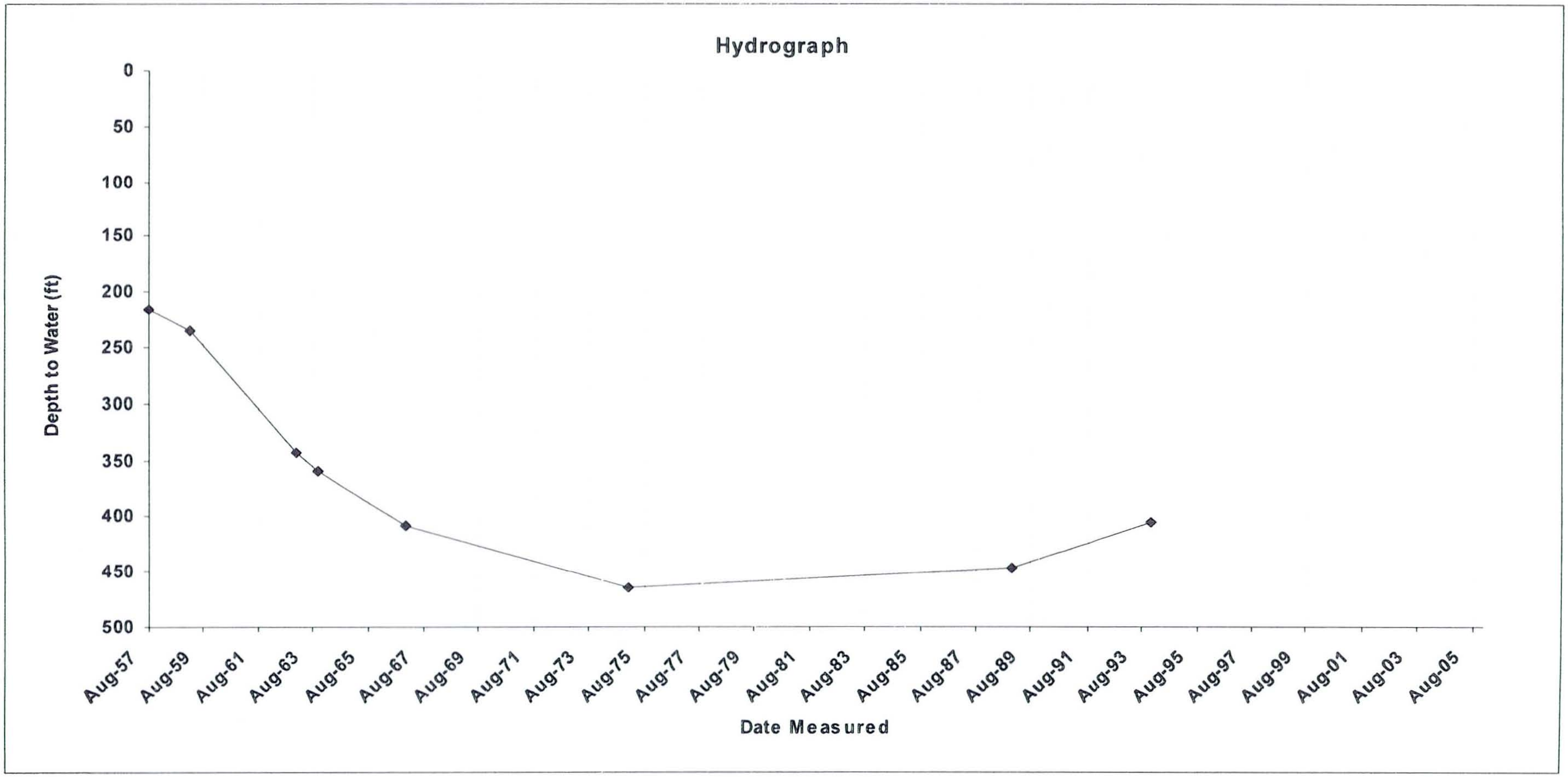


GWSI is ADWR's technical database of well locations, construction data, and water levels.

GWSI Well Report and Hydrograph

AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 19AAA	333027113064001	608459	33° 30′ 27″	113° 6′ 40″	DOMESTIC	672	3/30/1956	20	12/3/1993	404.80	759.2	8



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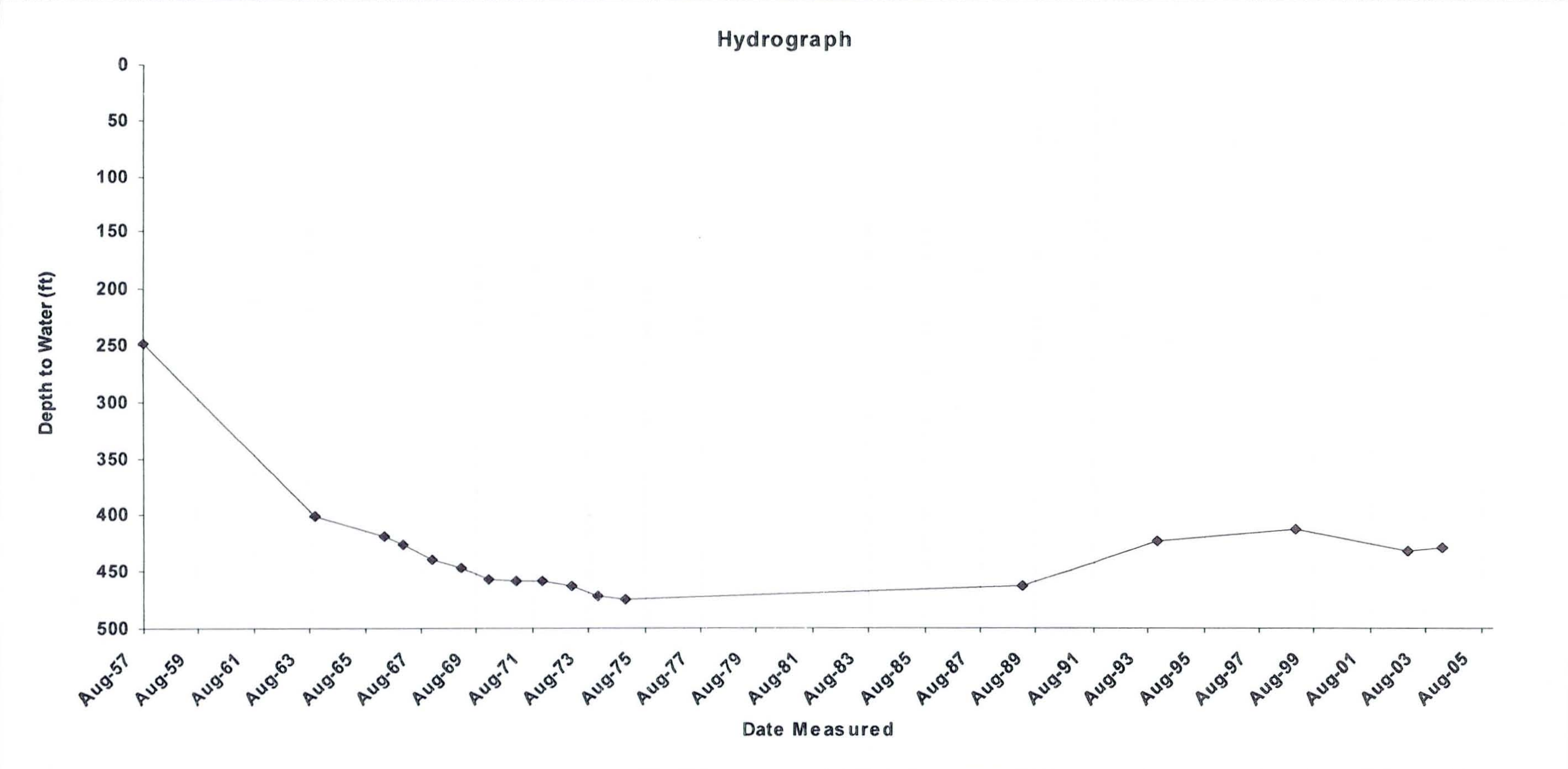
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 18CAA	333055113071101	608006	33° 30′ 55.2″	113° 7′ 8.6″	UNUSED	600	1/1/1956	20	3/18/2004	429.60	756.4	17



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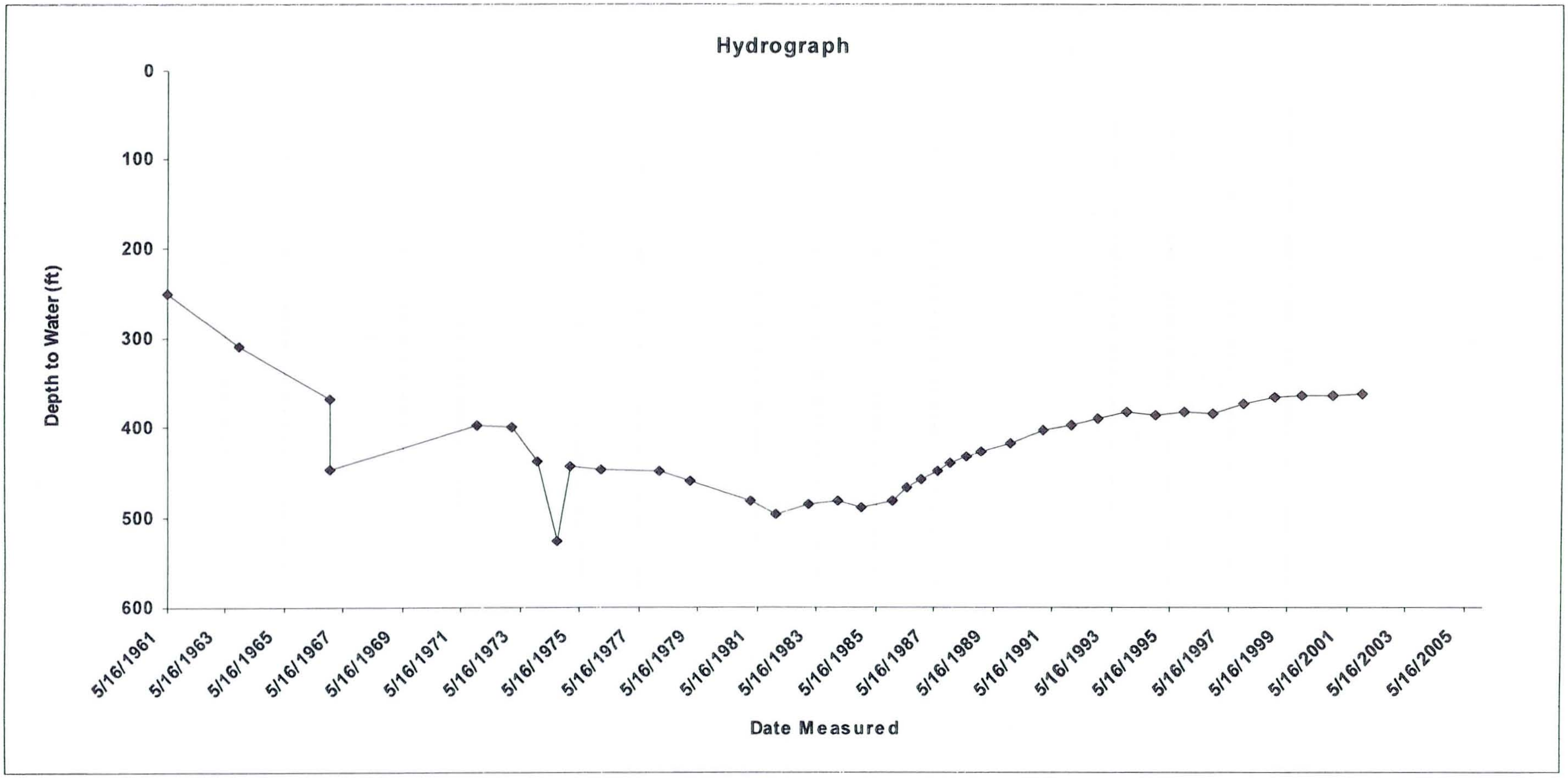
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-01-08 06AAA	332749113063901	612565	33° 27´ 49´´	113° 6´ 39´´	IRRIGATION	759	6/23/1958	20	12/15/2003			38



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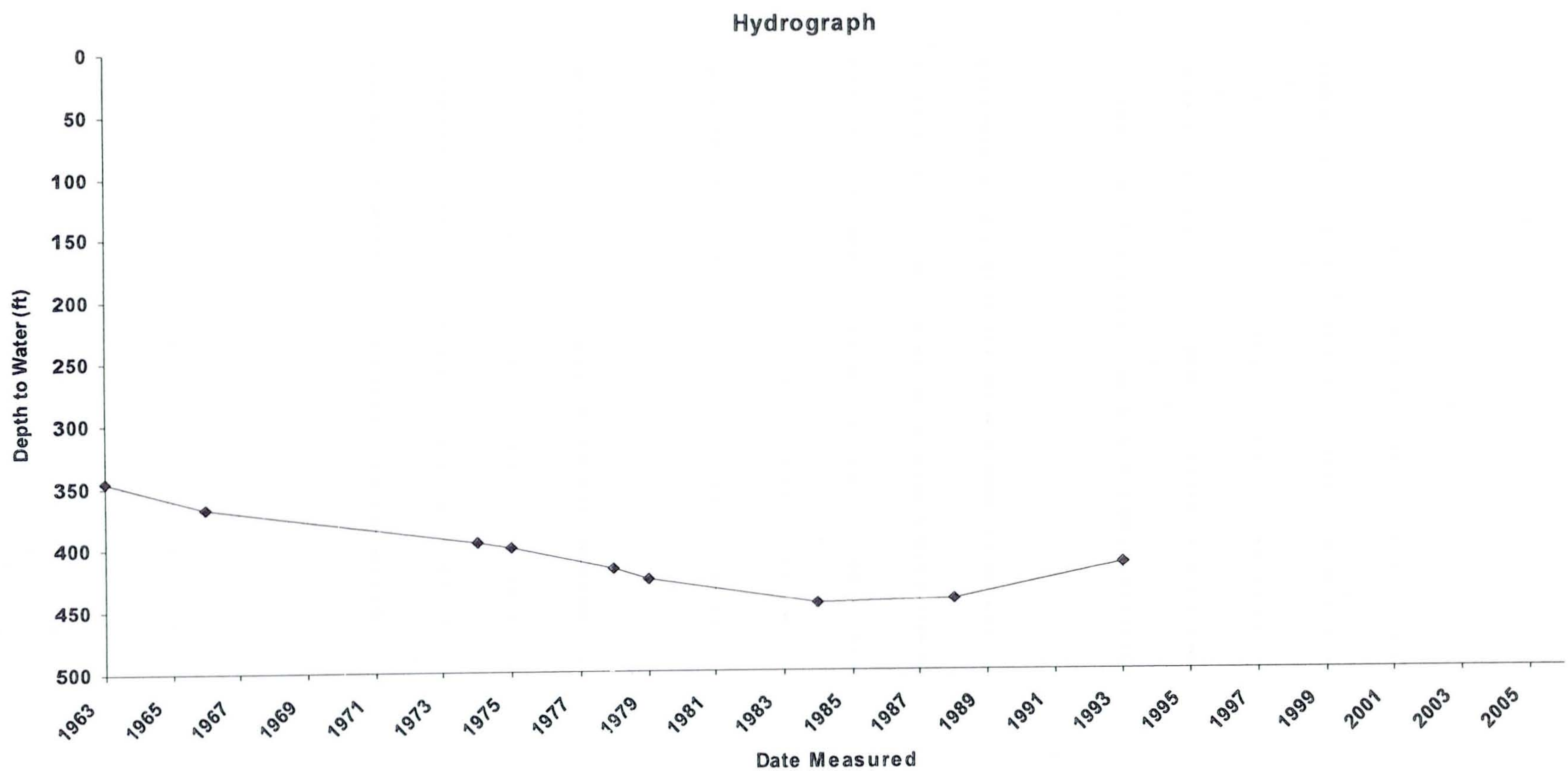
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 33AAD	332835113043301	612561	33° 28′ 35″	113° 4′ 33″	UNUSED	1605	5/19/1959	20	12/4/1998			10



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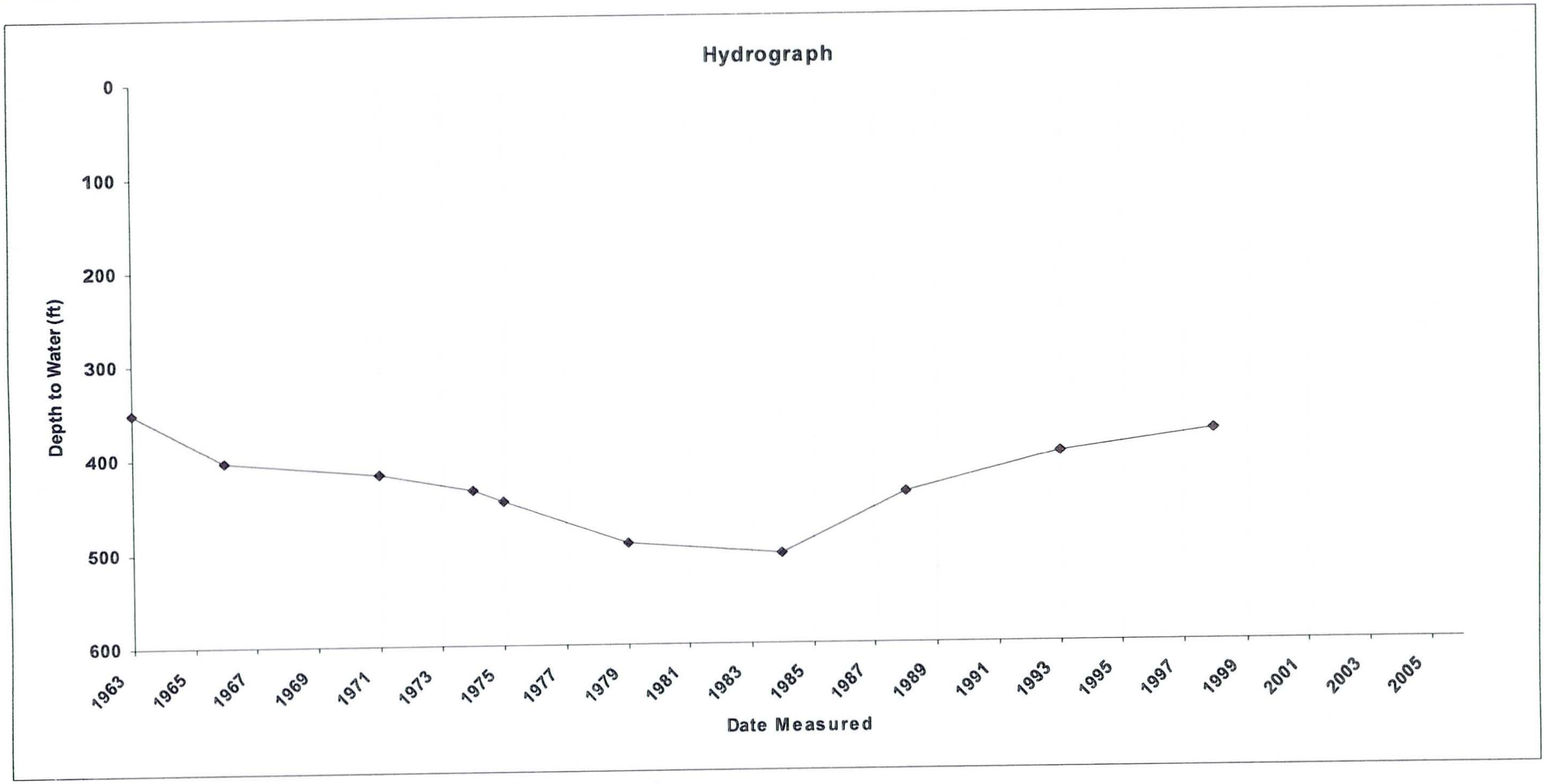
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 33CBB	332820113053201	612559	33° 28′ 20″	113° 5′ 32″	UNUSED				12/3/1998	376.40	767	10



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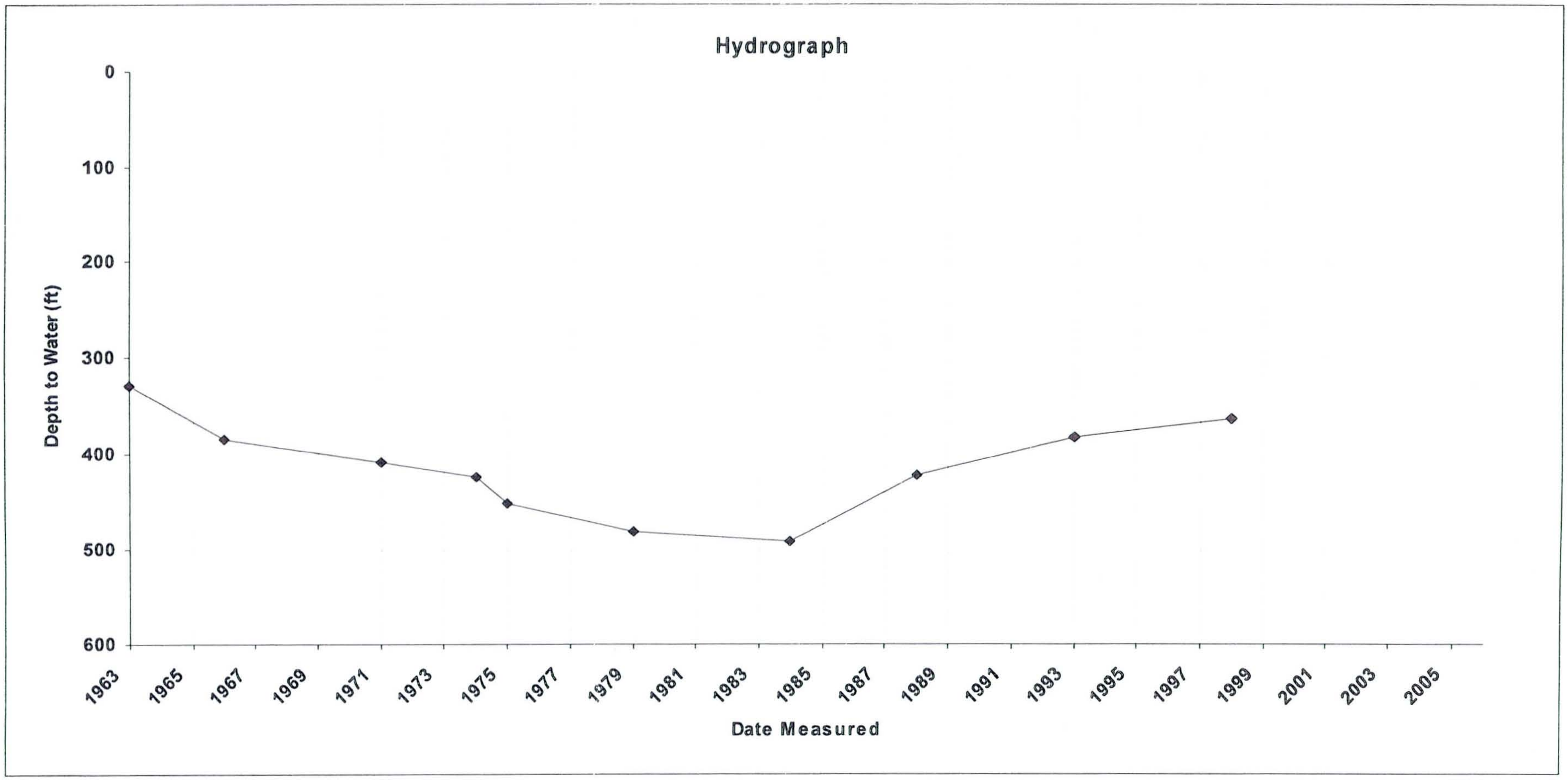
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 31BAA	332845113070701	612555	33° 28′ 45′′	113° 7′ 7′′	IRRIGATION	1200	7/23/1958	20	12/3/1998	364.40	758.6	10



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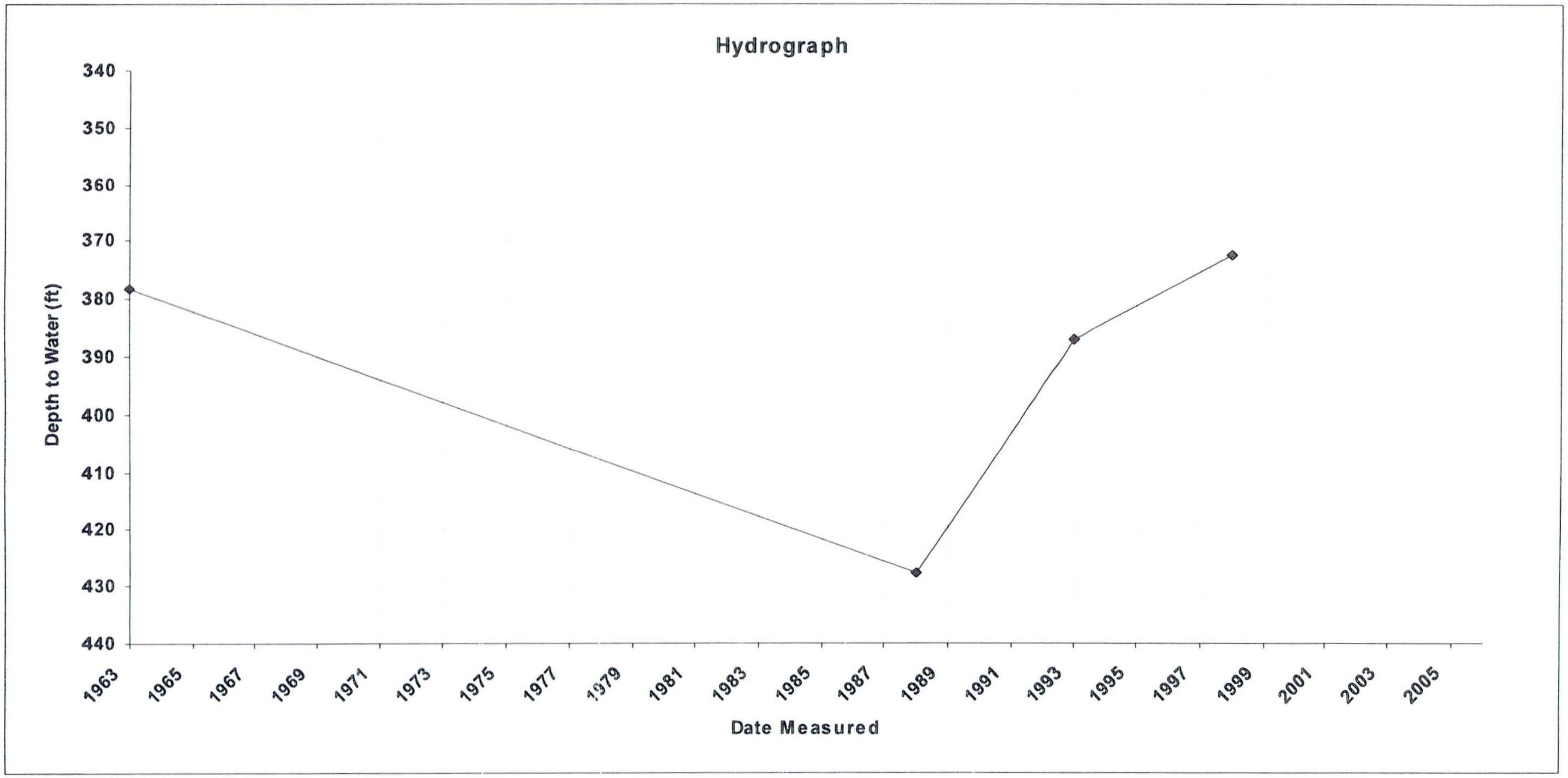
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 30BAA	332938113070701	608456	33° 29′ 38′′	113° 7′ 7′′	UNUSED	1150	1/1/1960	20	12/3/1998	372.60	765	4



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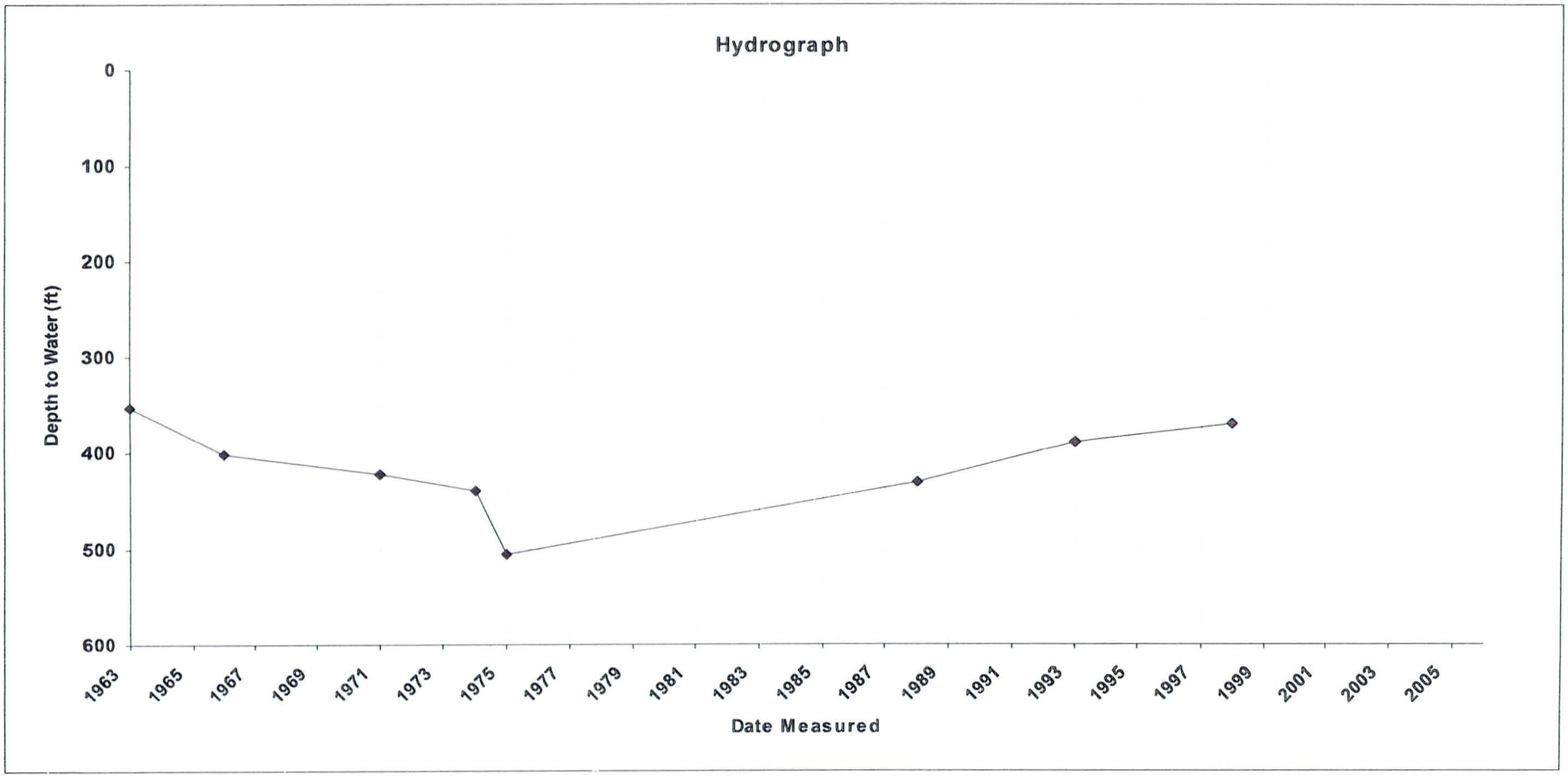
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 29CBB	332910113063301	606836	33° 29′ 10′′	113° 6′ 33′′	UNUSED	900	1/1/1963	20	12/3/1998	371.40	766	8



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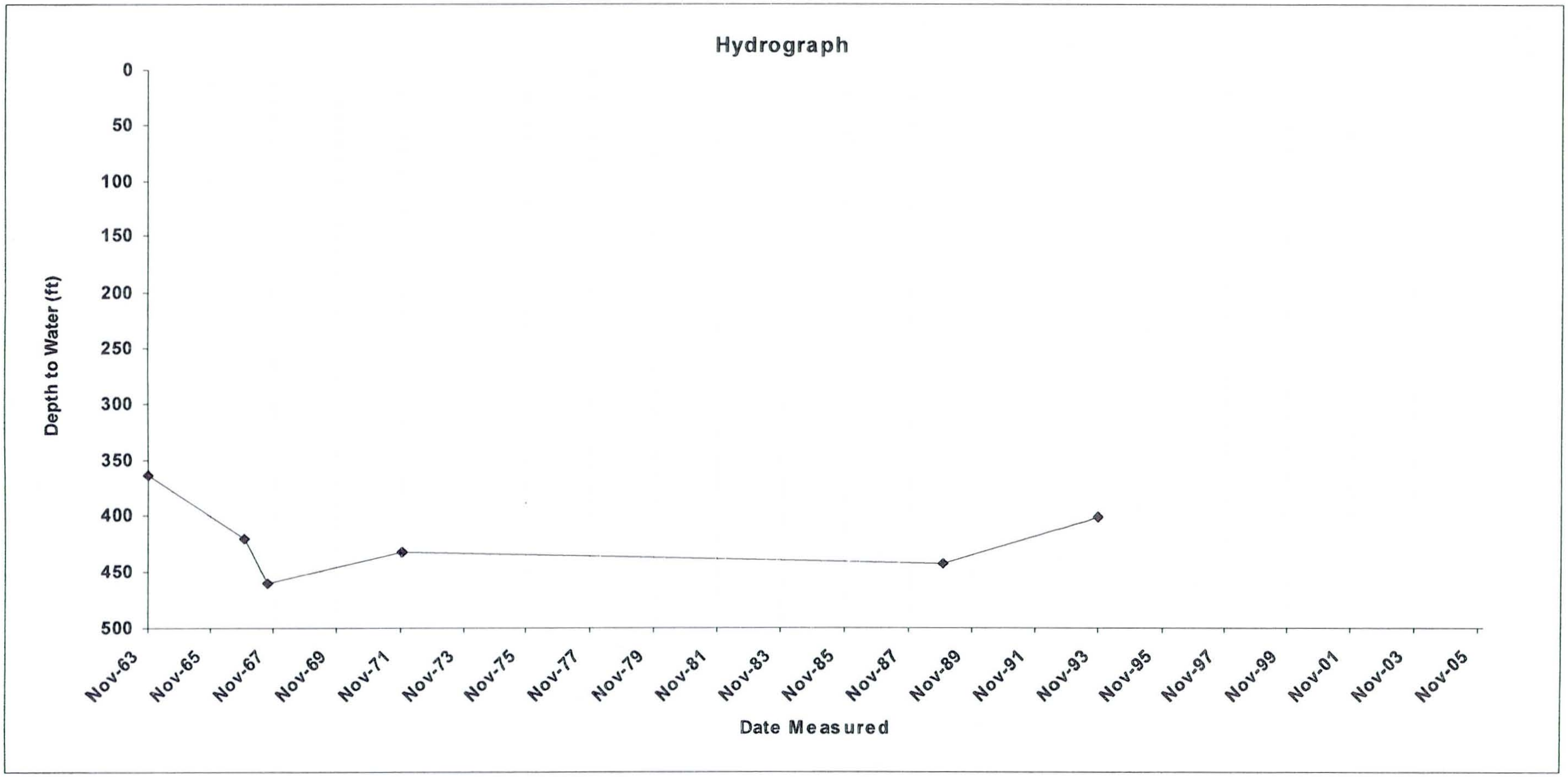
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 29BDD	332913113060601	606839	33° 29′ 13″	113° 6′ 6″	UNUSED	1660	3/1/1959	18	11/23/1993	401.10	746	6



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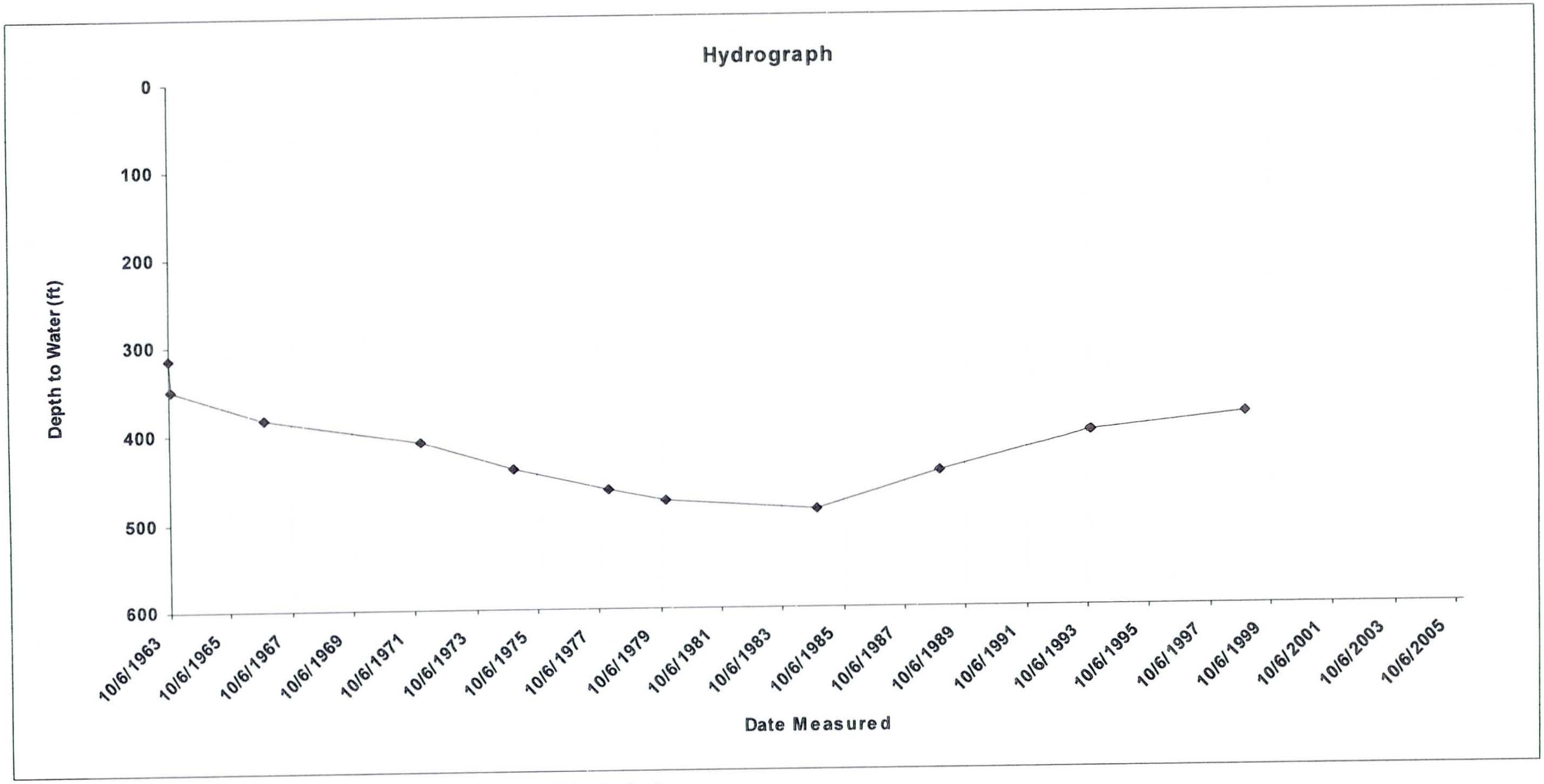
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-01-08 04BBB	332750113053101	802143	33° 27′ 50″	113° 5′ 31″	PUBLIC SUPPLY	1000	5/10/1960	16	12/8/1998	382.50	760	11



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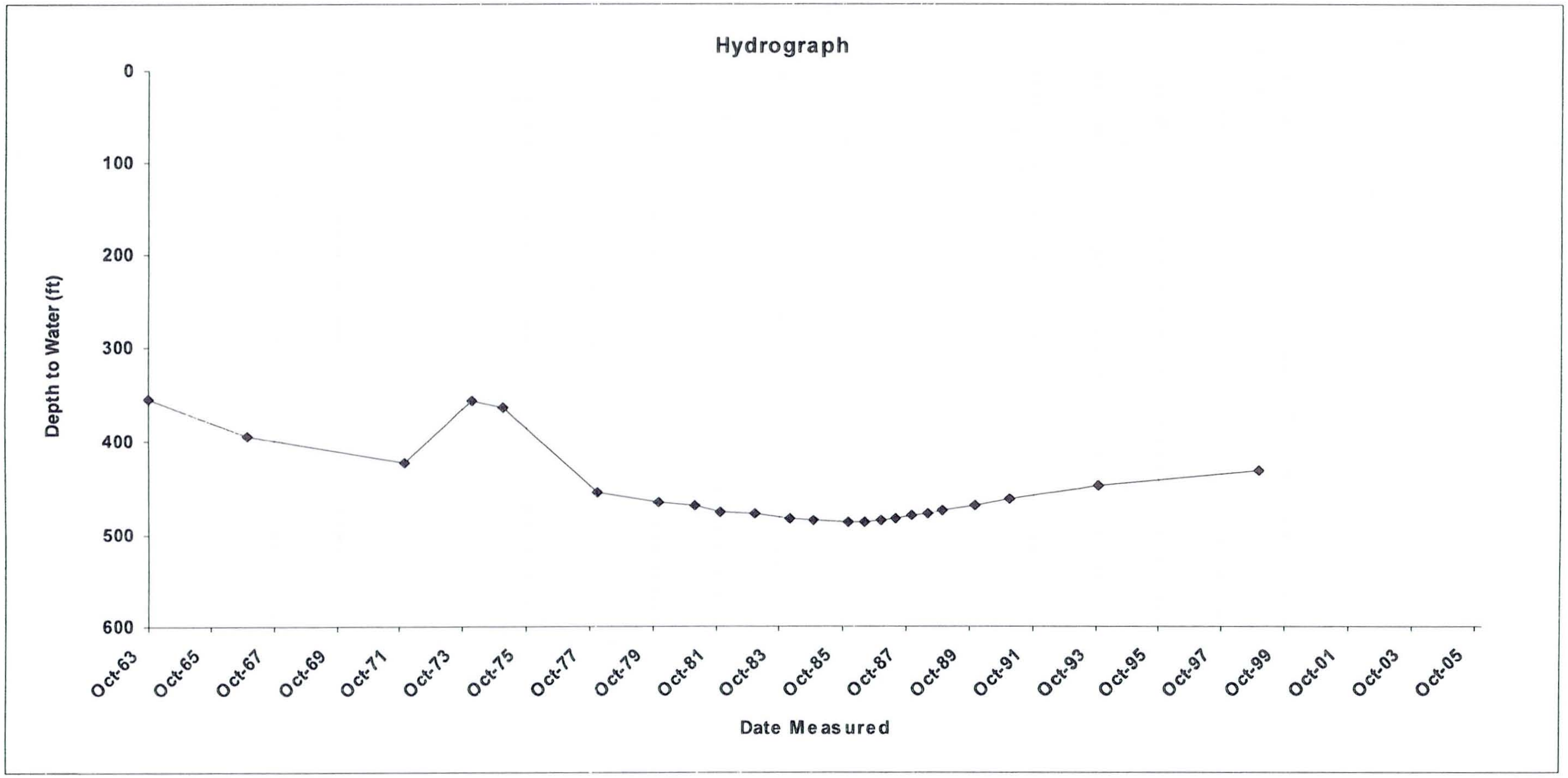
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 27BAB	332937113041601	643347	33° 29′ 37″	113° 4′ 16″	UNUSED	1120	1/1/1963	16	12/4/1998	430.60	773.4	23



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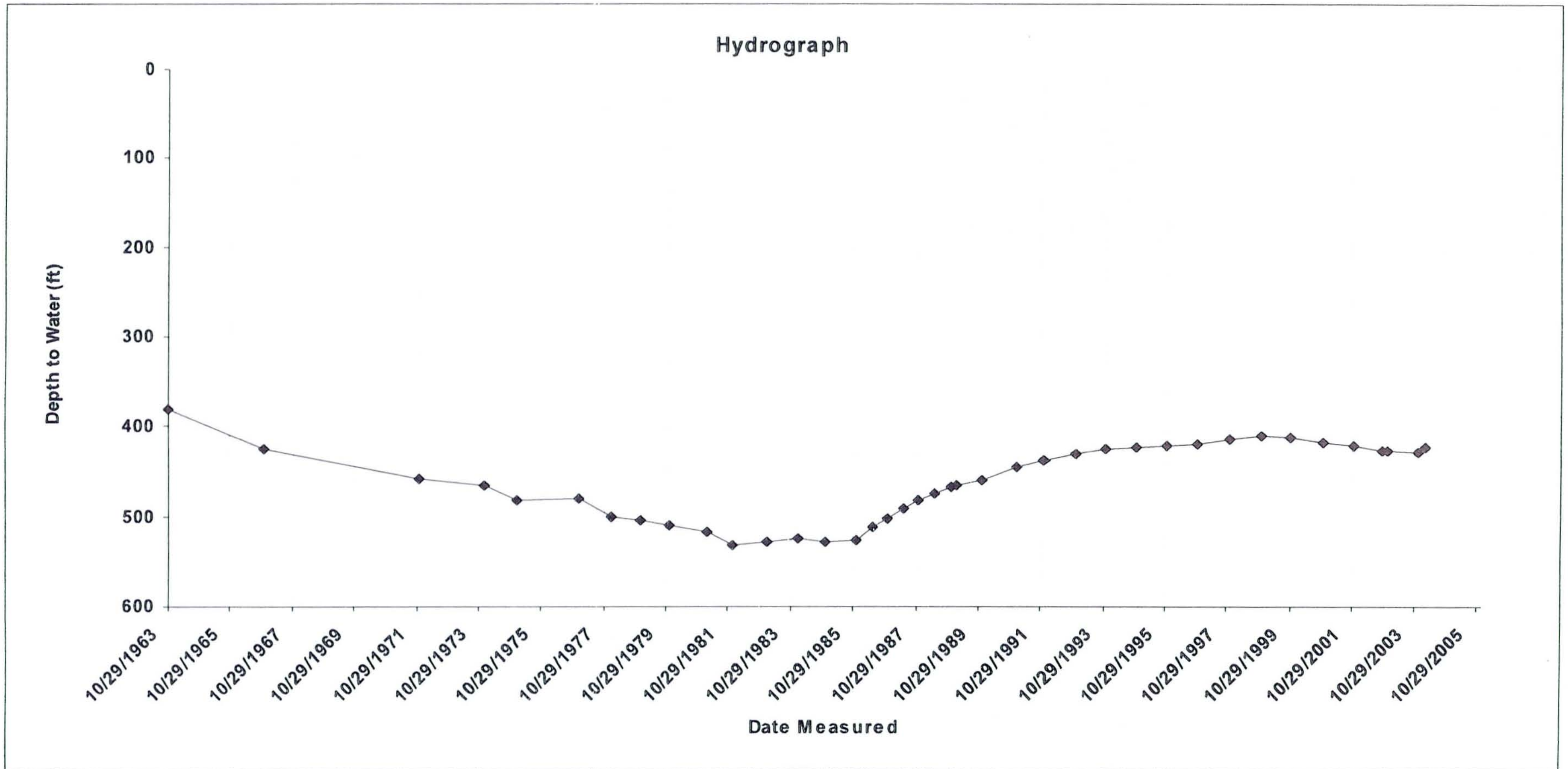
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 17CAA	333057113060701		33° 30′ 54.5′′	113° 6′ 3.5′′	UNUSED	1650	4/15/1960	20	3/18/2004	424.20	755.8	40



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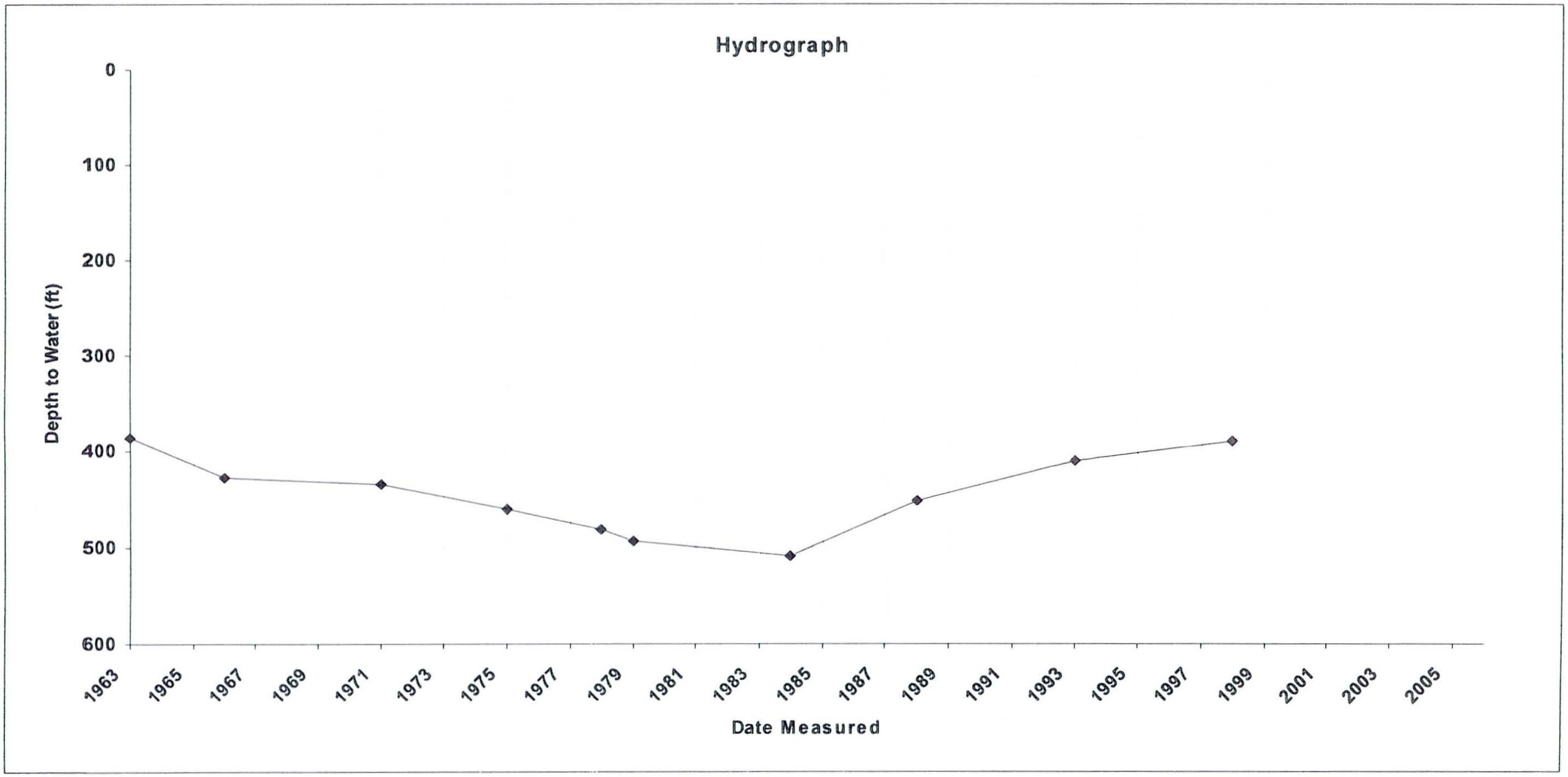
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 29ABB	332937113060301	606841	33° 29′ 37′′	113° 6′ 3′′	UNUSED	1660	1/1/1960	18	12/3/1998	390.10	764.9	10



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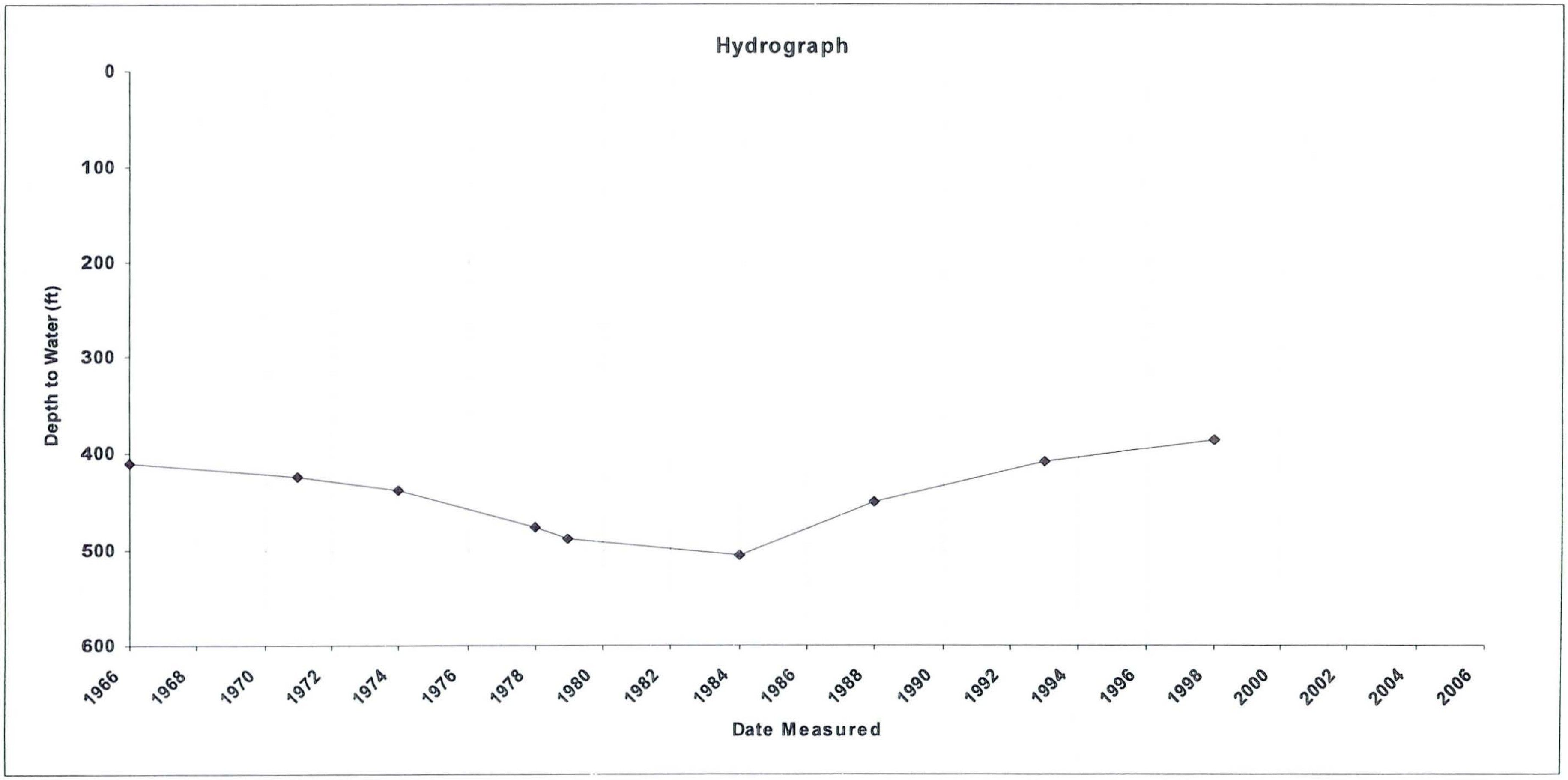
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 28CCB	332859113052901	603489	33° 28′ 59′′	113° 5′ 29′′	UNUSED			17	12/3/1998	385.60	766	9



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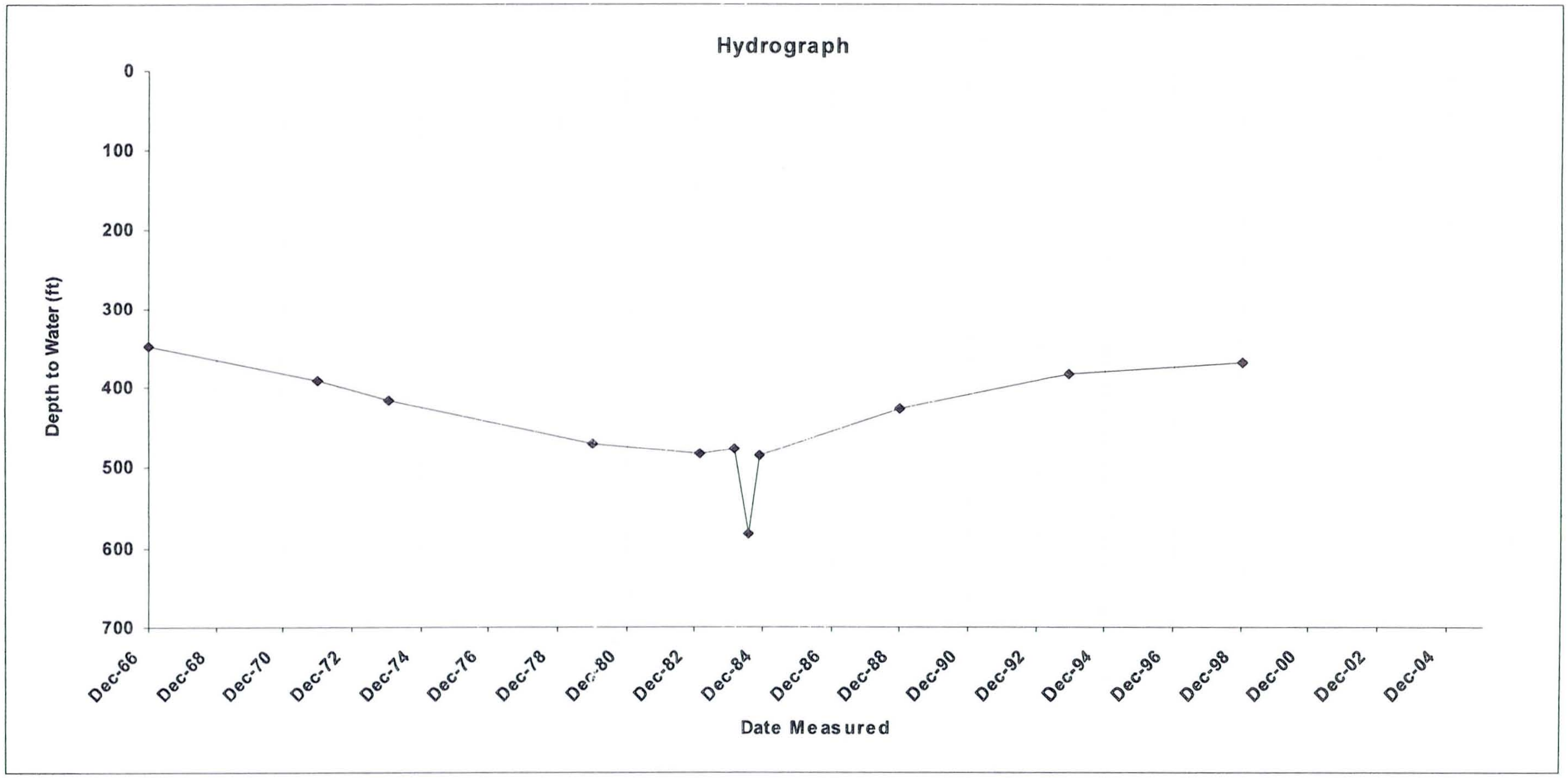
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-01-08 06DAA	332728113063901	612567	33° 27′ 28″	113° 6′ 39″	UNUSED				12/3/1998	367.70	747.3	11



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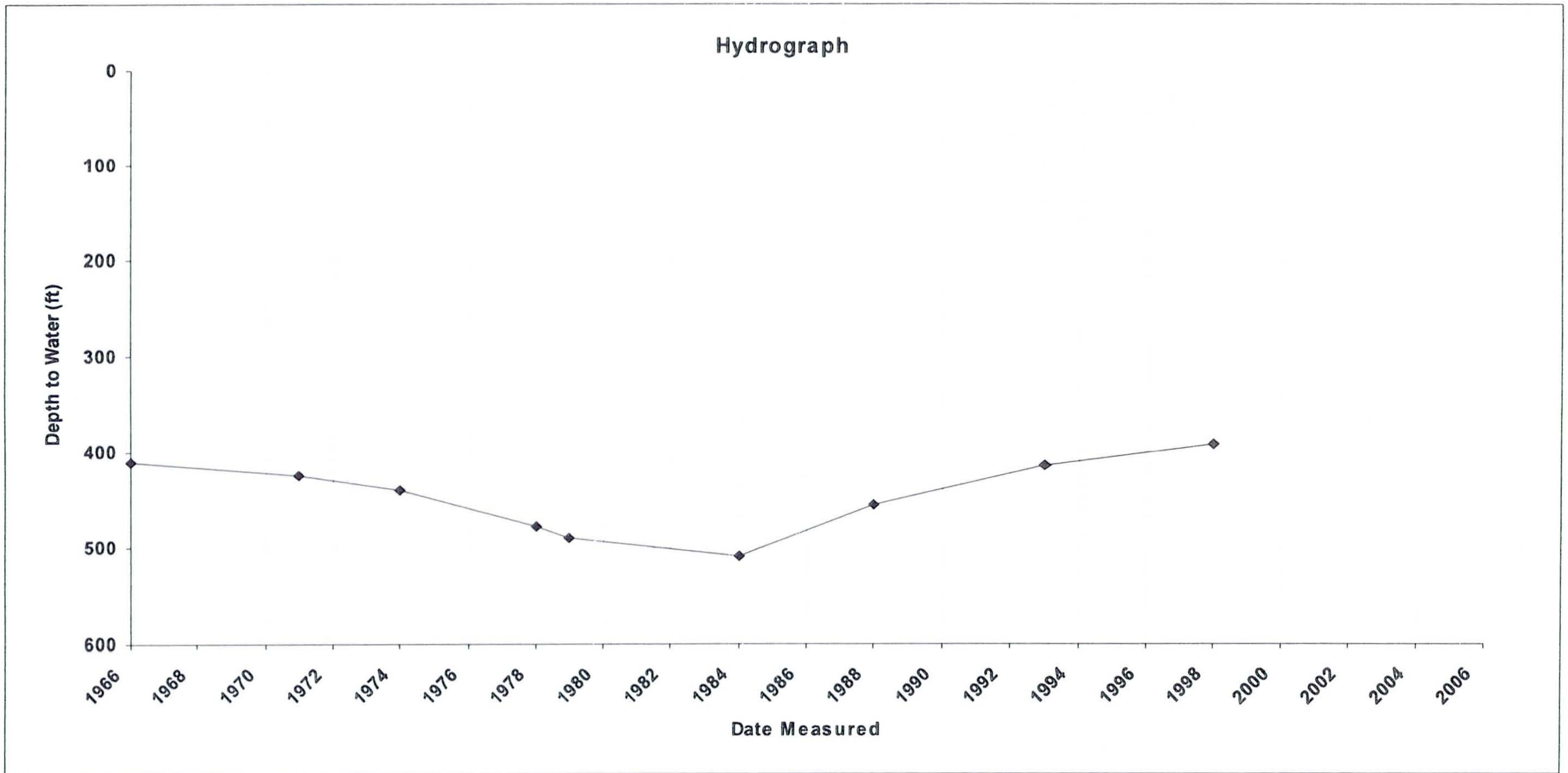
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 28CDB	332902113051301	603488	33° 29′ 2″	113° 5′ 13″	UNUSED	611	1/1/1958	16	12/3/1998	390.90	767.1	9



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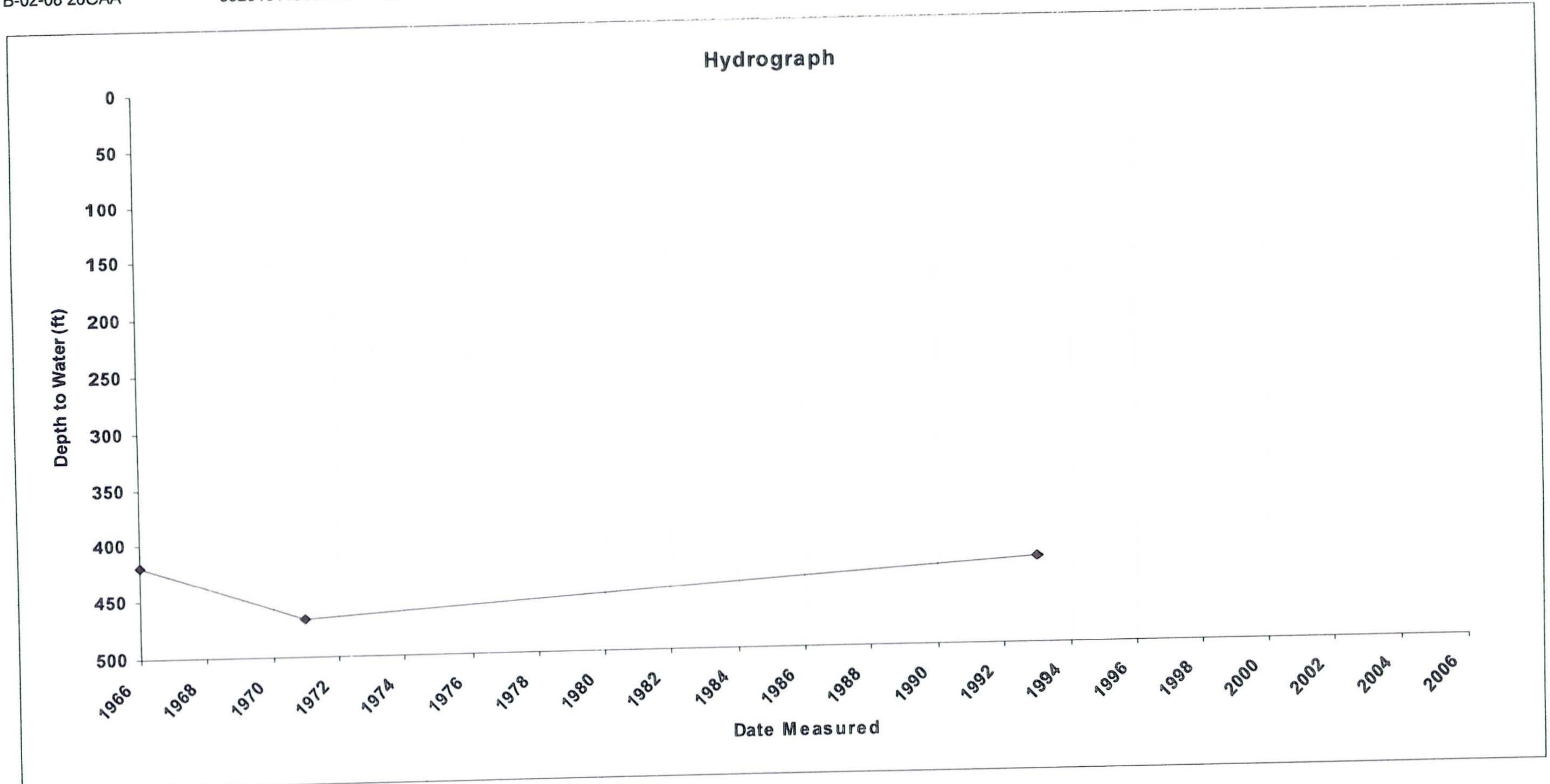
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 28CAA	332910113050401	603487	33° 29′ 10″	113° 5′ 4″	UNUSED	800	1/1/1957	16	12/3/1998			4



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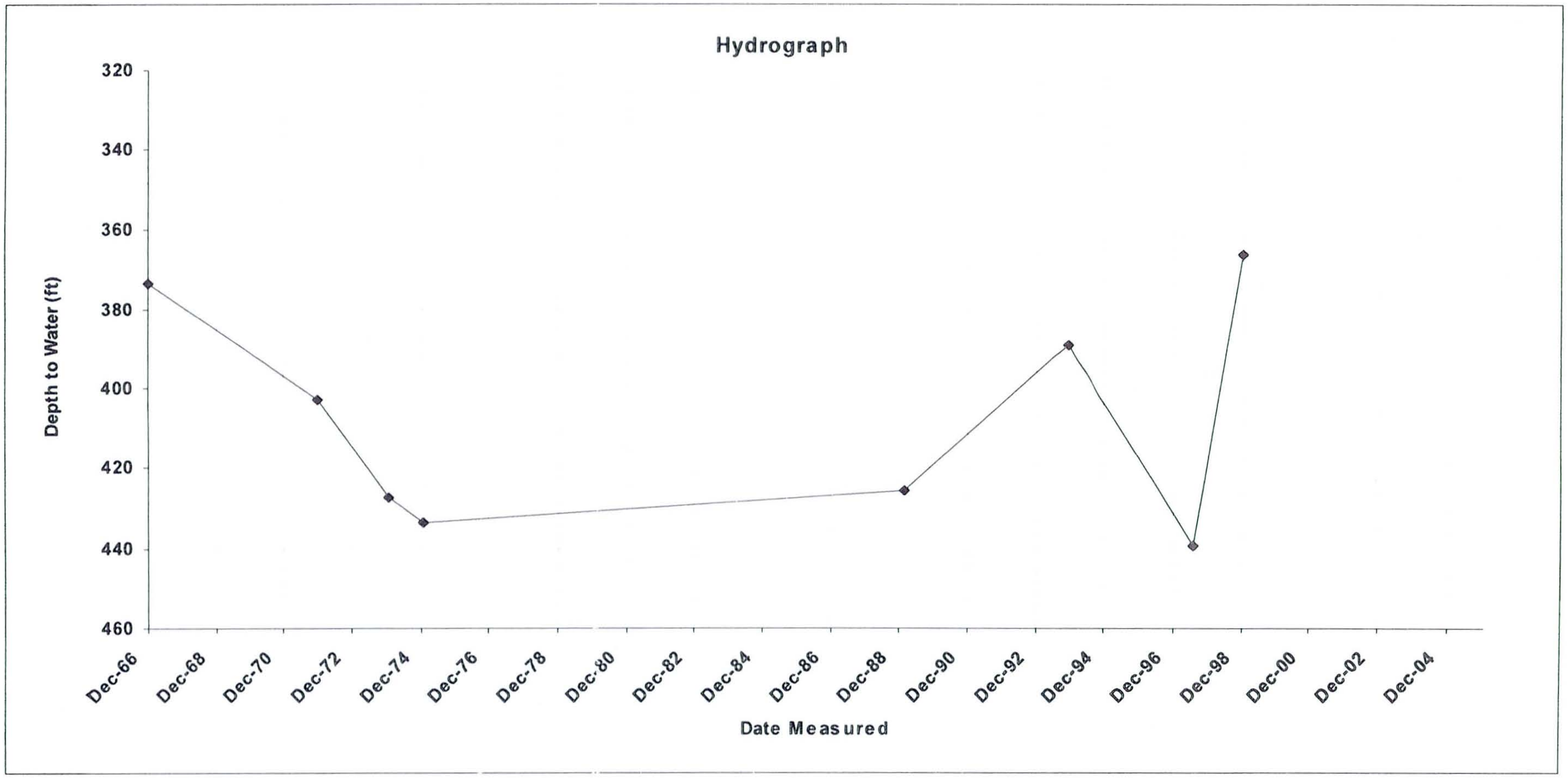
B-02-08 31DAA	332824113063601	612560	33° 28′ 24′′	113° 6′ 36′′	IRRIGATION	1200			12/3/1998	366.40	754	8



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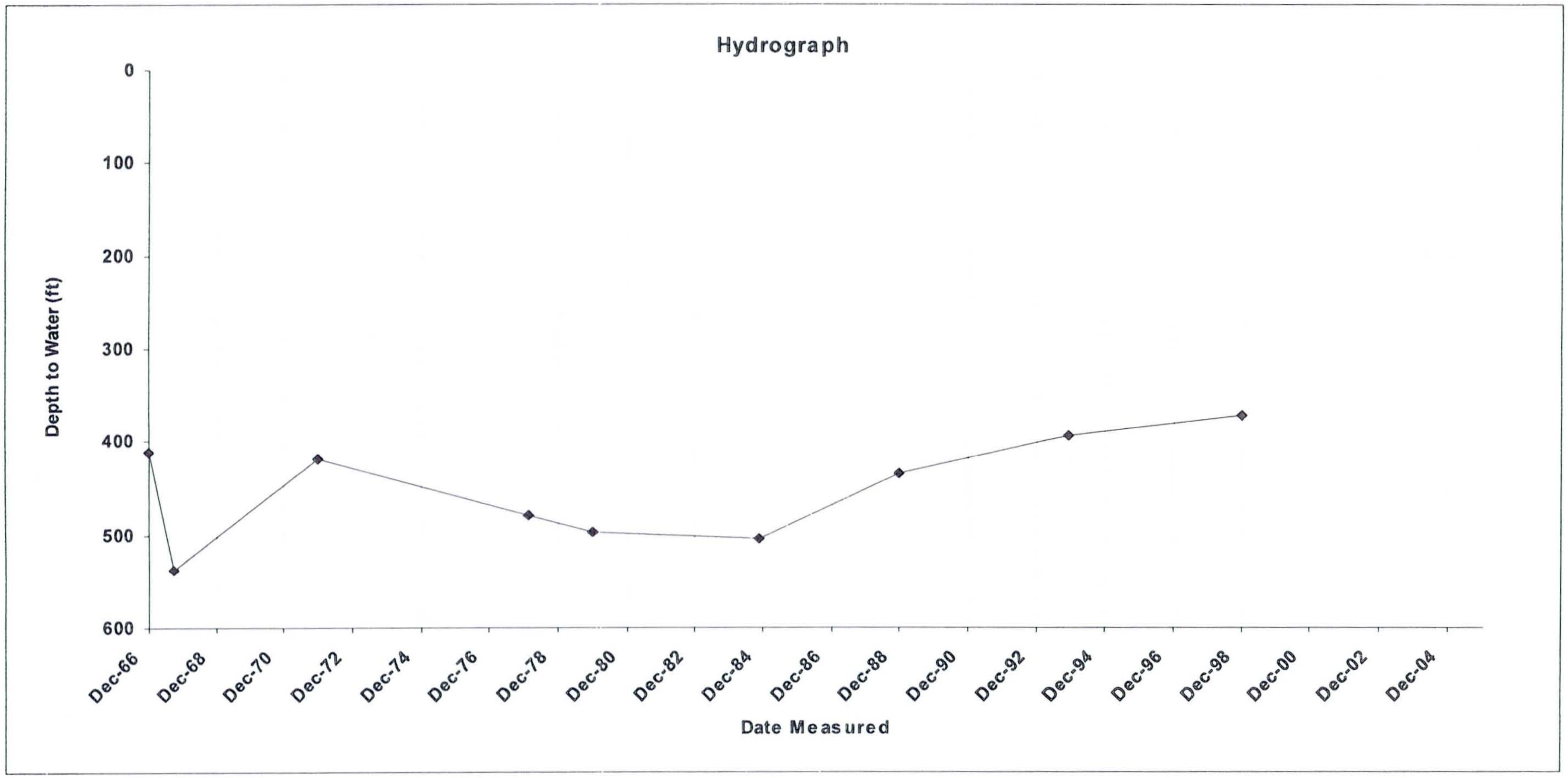
Local ID	*Site ID*	*ADWR Reg. No.*	*Latitude*	*Longitude*	*Water Uses*	*Well Depth*	*Drill Date*	*Case Dia.*	*Latest WL Date*	*Depth to Water*	*WL Alt. above Mean Sea Level*	*Times Meas.*
B-02-08 32BAA	332843113060601	612558	33° 28′ 43″	113° 6′ 6″	UNUSED	1200			12/3/1998	372.70	764	9



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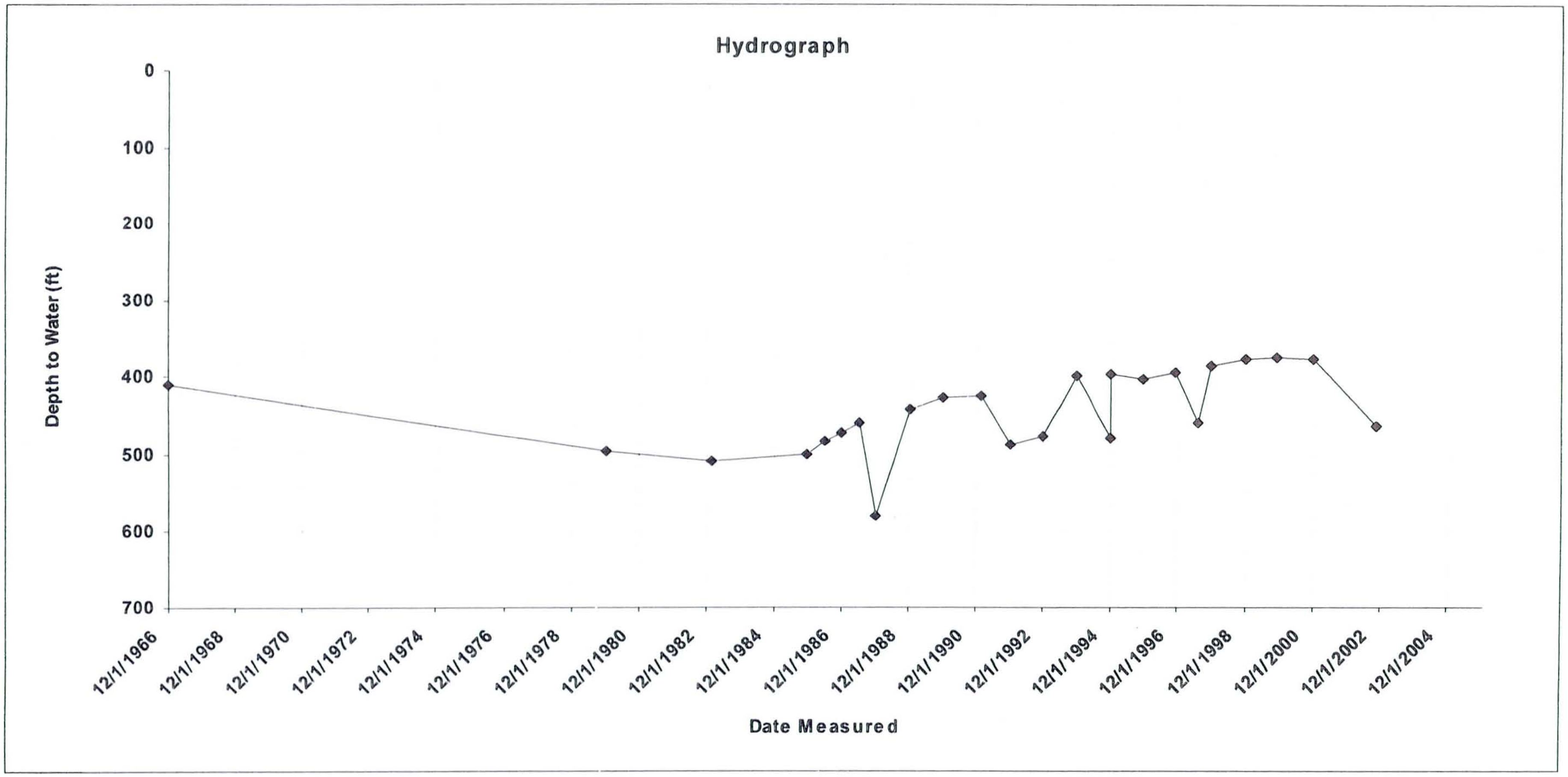
B-02-08 30AAA2	332938113063901	608452	33° 29′ 38′′	113° 6′ 39′′	IRRIGATION	1180	1/1/1960	20	10/30/2002	464.85	679.15	24



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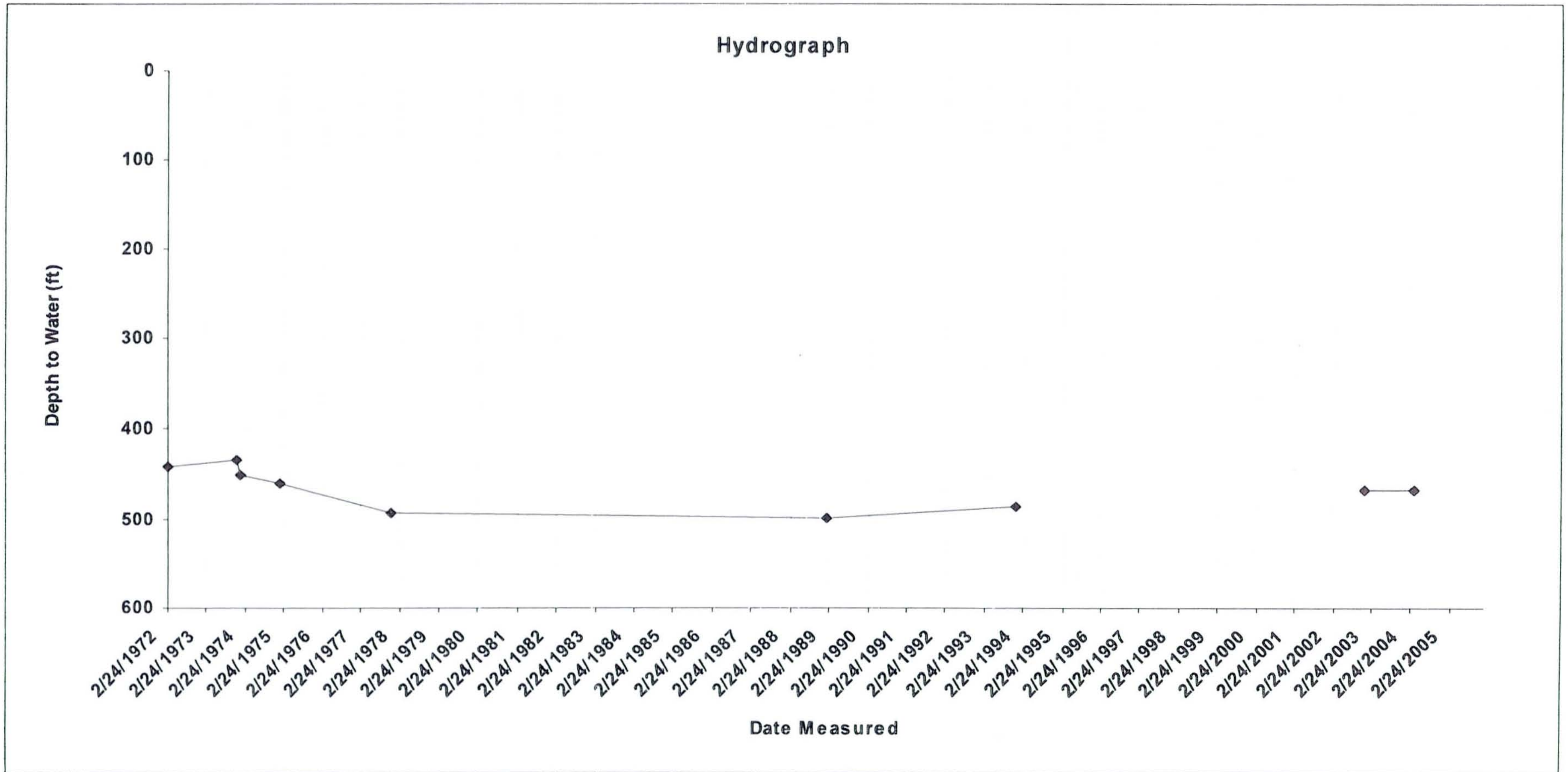
B-02-08 15BCD	333055113041501	636532	33° 30' 57.09''	113° 4' 17.7''	UNUSED	830	1/1/1977	8	3/18/2004	467.40	772.6	10



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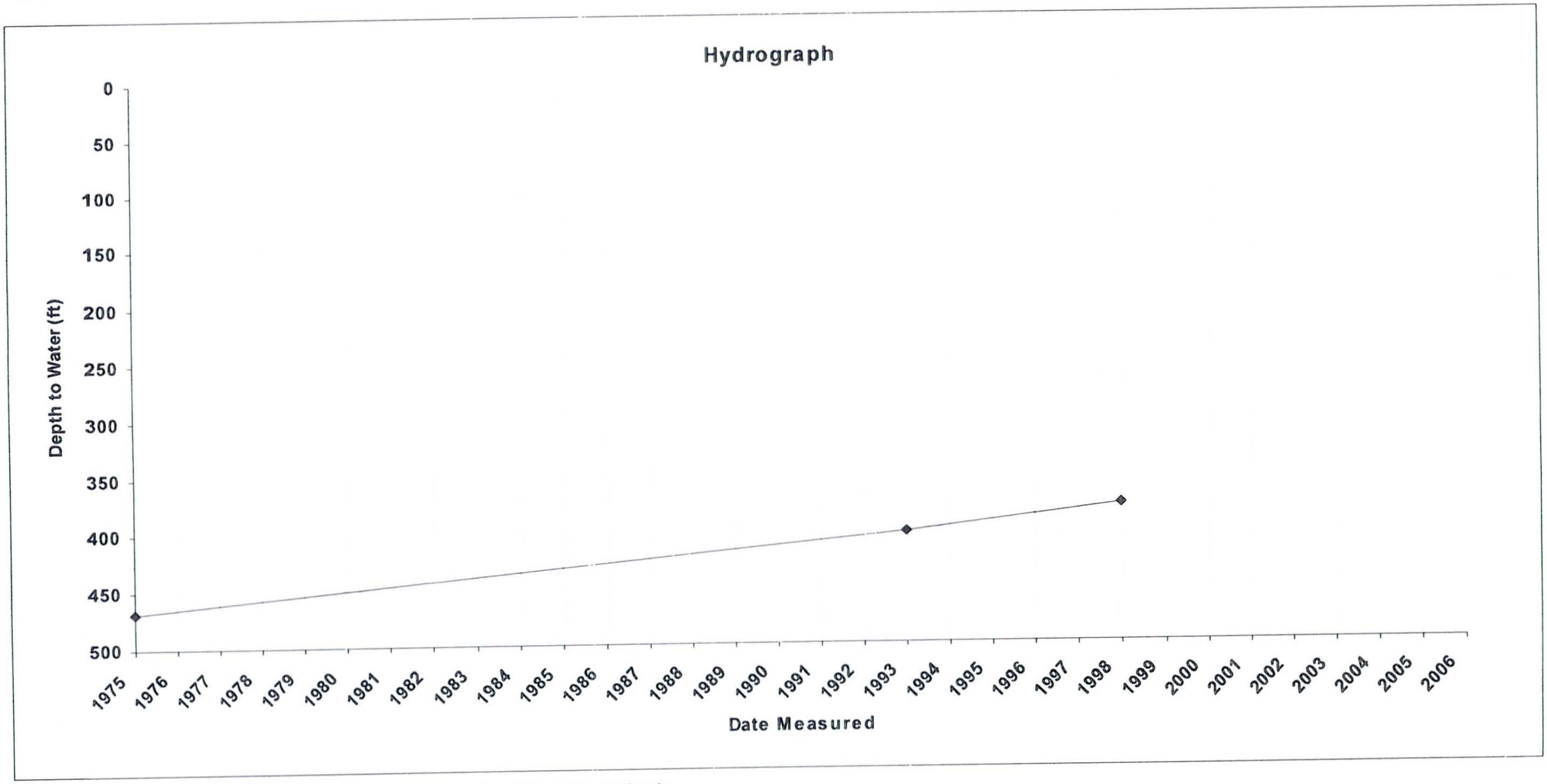
GWSI Well Report and Hydrograph

AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 20CCC	332941113063401		33° 29´ 41´´	113° 6´ 34´´	UNUSED				12/8/1998	379.90	765.1	3



GWSI is ADWR's technical database of well locations, construction data, and water levels.

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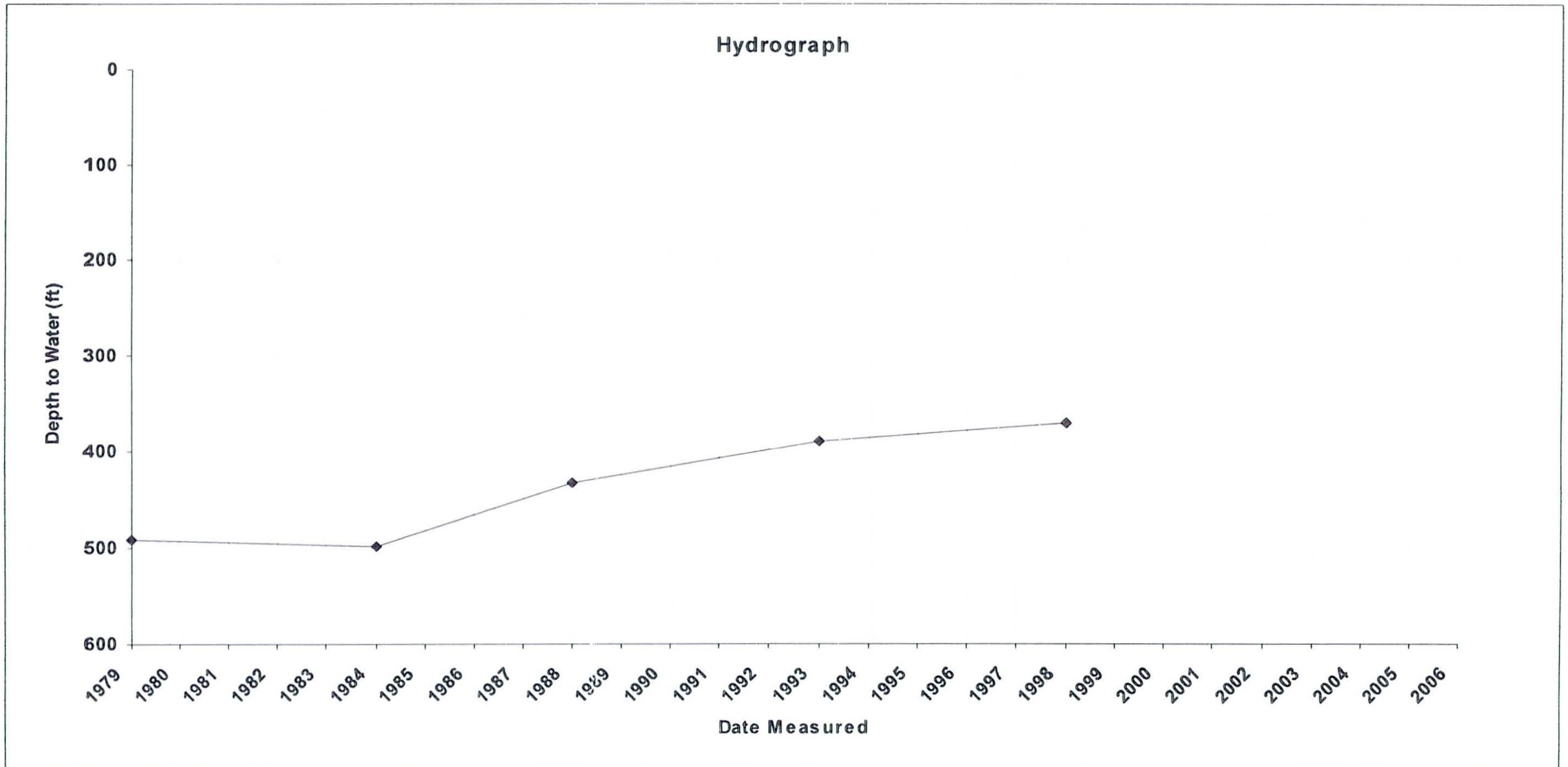
B-02-08 32BBA	332845113061901	612557	33° 28′ 45″	113° 6′ 19″	IRRIGATION	1720	3/1/1961	20	12/3/1998	370.50	763	5



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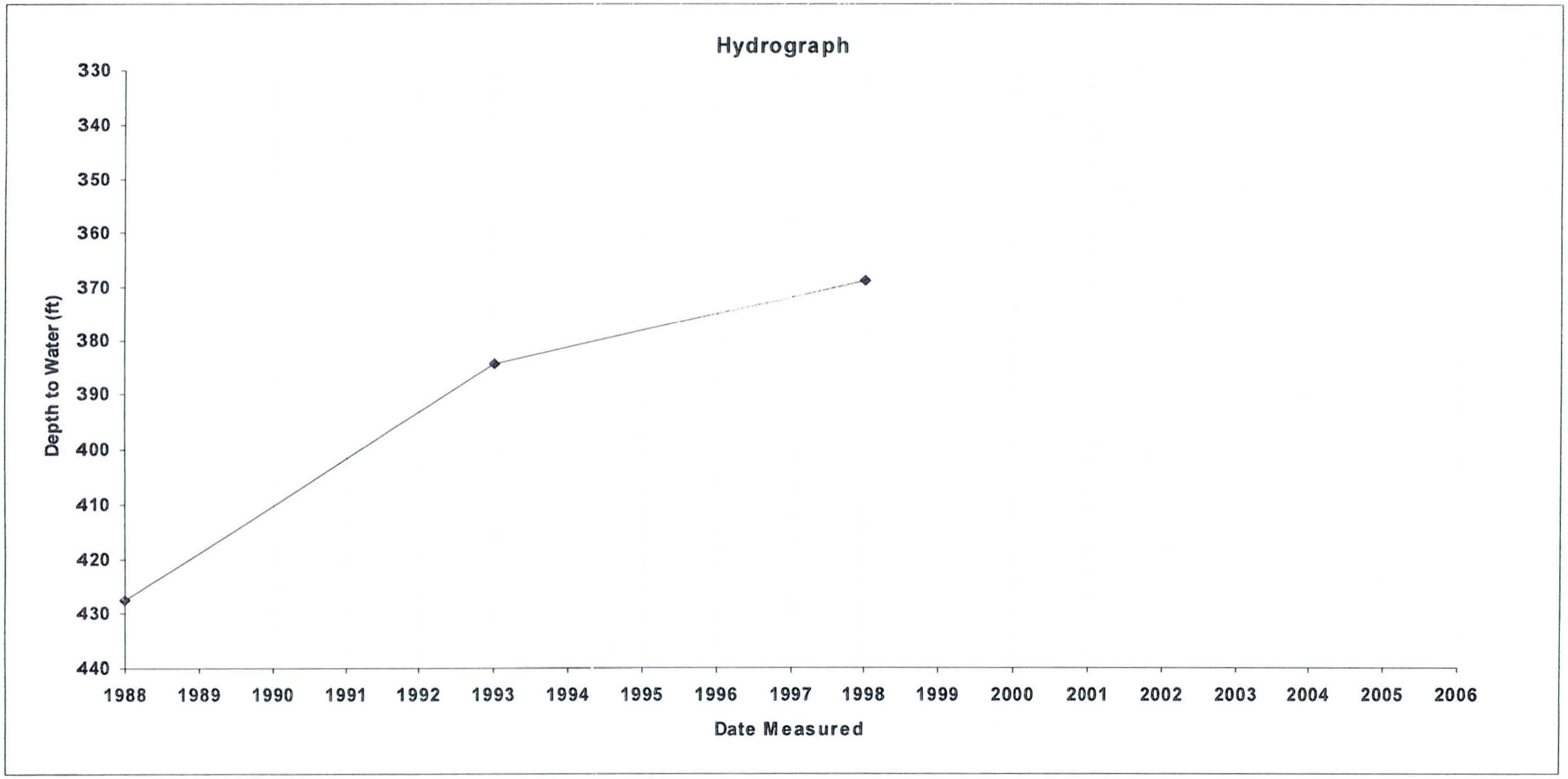
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 30ACC	322913113070201	608453	33° 29′ 13″	113° 7′ 2″	UNUSED	1200			12/3/1998	368.90	762.1	3



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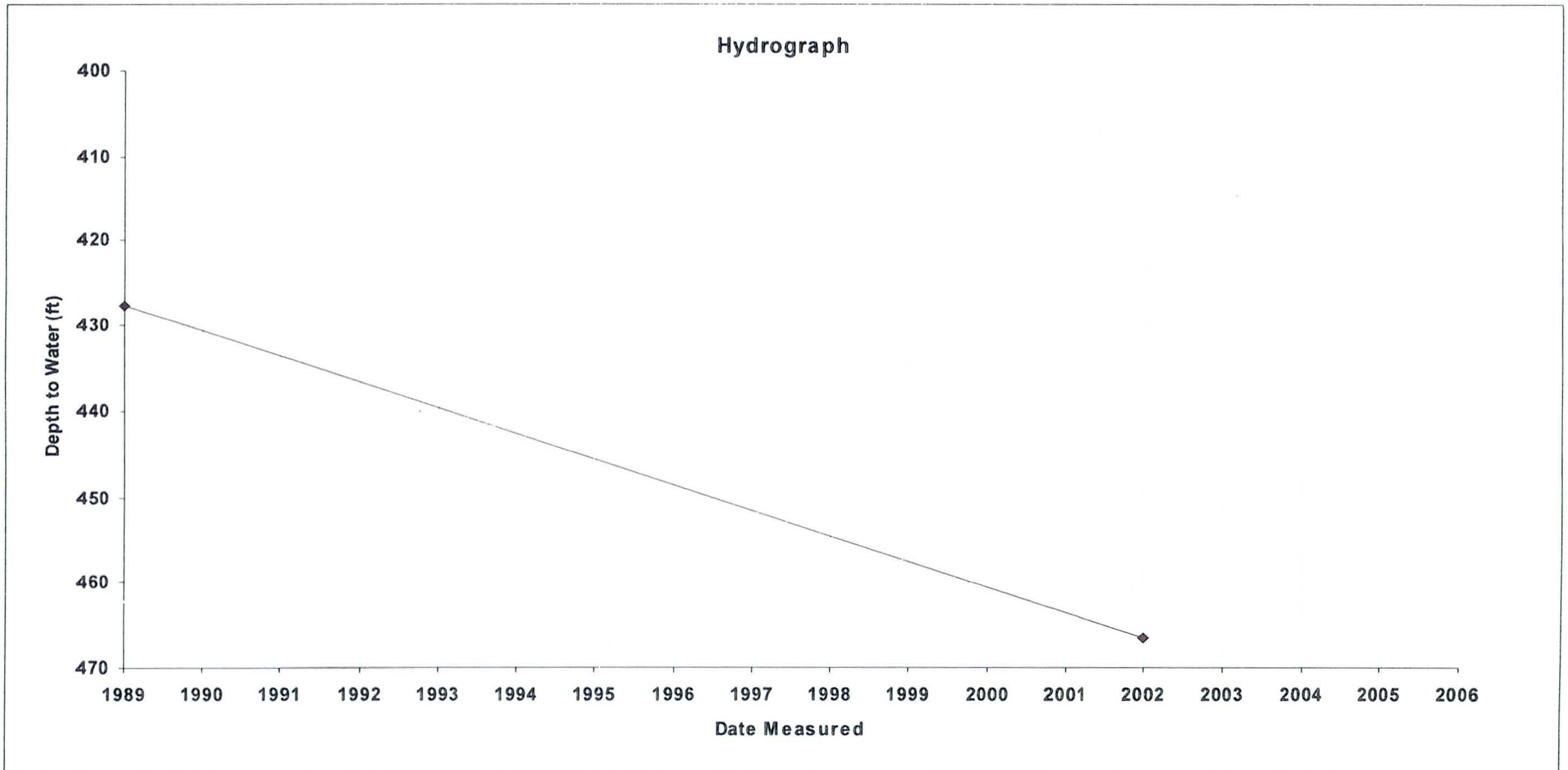
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-08 15BBD	333110113041501	628118	33° 31´ 8.89´´	113° 4´ 5.69´´	PUBLIC SUPPLY	755	2/27/1980	8.62	12/20/2002	466.60	796.4	2



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GEOTECHNICAL MEMORANDUM

SADDLEBACK FLOOD RETARDING STRUCTURE PHASE I STRUCTURES ASSESSMENT

Submitted to:

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Registered Professional Engineer (Civil) CERTIFICATE NO. 35654 DEAN B. DURKEE Date Signed 11-3-05 ARIZONA, U.S.A.

Frank P. Marathe Jr., P.E.
for Frances Ackerman, R.G., E.I.T.
Project Geotechnical Engineering

Dean B. Durkee, Ph.D., P.E.
Project Manager

November 2005

043394

1.0 REVIEW OF PREVIOUS GEOTECHNICAL DOCUMENTATION

A comprehensive review of existing geotechnical reports was performed. The following documents were reviewed (reference citations are listed at the end of this memorandum):

- Watershed Workplan for the Harquahala Valley Watershed (Flood Control District of Maricopa County (FCDMC), 1967)
- Supplemental Watershed Workplan, Harquahala Valley Watershed (FCDMC, 1977)
- Engineering Report, Saddleback Diversion Harquahala Valley Watershed (U.S. Department of Agriculture Soil Conservation Service, 1987)
- Report of Geologic Investigation, Harquahala Valley Watershed, Saddleback Diversion and FRS (SCS, 1978) – including Appendix entitled Harquhala Valley Watershed Earth Crack Investigation by Ronald L. Graner, Geologist, October 10, 1966
- Materials Testing Report section of Geologic Investigation Report, Saddleback FRS and Diversion
- Saddleback Floodwater Retarding Structure, Harquahala Valley Watershed Engineering Documentation, Phase II (PRC Toups Corporation, 1979)
- Structural Stability of Embankments, Appendix to Design Report for the Saddleback FRS (Toups Corporation, 1979)
- Saddleback Floodwater Retarding Structure as-built plan set
- Operation and Maintenance Manual, Saddleback FRS and Diversion (U.S. Department of Agriculture Soil Conservation Service, 1980)
- Plan set (12 sheets) for Saddleback Diversion repair (U.S. Department of Agriculture Soil Conservation Service, 1989)
- A New Earth Fissure Opens in the Harquahala Plain of West-Central Arizona (September 25, 1997) (Arizona Department of Water Resources Hydrology Division and Groundwater Management Support Division, 1998)
- Dam Construction Inspection records
- Annual dam inspection checklists
- Documentation of the rehabilitation of central drain between Stations 44+00 and 52+00
- Special Inspection Reports and documentation regarding longitudinal cracking (Flood Control District of Maricopa County, October through December 2000)
- Geotechnical Investigation Report Saddleback FRS (AMEC Earth and Environmental, 2001)
- Downstream Hazard & Classification Review (Flood Control District of Maricopa County, 2004)

The following sections provide a discussion of findings from that review.

1.1 Regional Setting

Information on the regional setting of the Saddleback FRS was summarized and/or excerpted from FCDMC (1967) and SCS (1978).

The Harquahala Plain overlies a broad elongated alluvium-filled groundwater basin located about 60 miles west of Phoenix, Arizona. The plain is bounded to the north by the Harquahala Mountains, to the west by the Little Harquahala Mountains, to the southwest by the Eagletail Mountains, to the south by the Gila Bend Mountains, to the east by Saddle Mountain, and to the northeast the Big Horn Mountains. The Harquahala Plain and surrounding mountains cover an arid desert area of about 750 square miles. The basin slopes to the southeast at 15 to 20 feet per mile and is principally drained by Centennial Wash, which enters the basin at its northwestern end between the Harquahala and Little Harquahala Mountains, and exits the basin in the southeast corner. Centennial Wash is an ephemeral stream that flows only in response to rainfall events. The average annual precipitation is about 6 inches (in) per year (http://www.water.az.gov/adwr/Content/WaterInfo/OutsideAMAs/LowerColorado/Basins/harquahala.html).

The alluvium of the Harquahala basin is composed of a heterogeneous mixture of clay, silt, sand and gravel. The thickness of the alluvium varies from 0 feet at the mountain fronts to over 5,000 feet in the deepest part of the basin. The alluvial deposits generally grade from coarse sand and gravels in the southeastern portion of the basin to fine-grained deposits in the central portions of the basin. Fine-grained clay deposits exceeding 1,000 feet in thickness occur in the western portion of T2N, R9W. Farther west, near Sections 34-36, T3N, R11W, the fine-grained beds appear to grade into an alternating sequence of fine-grained and coarse-grained layers that overlie a conglomerate beginning at a depth of about 800 feet.

The area is within the Sonoran Desert Section of the Basin and Range physiographic province. The portion of the Harquahala Mountains included in the watershed area is composed mainly of Precambrian granite gneiss and schist, Paleozoic and Mesozoic shale, quartzite, and limestone, and Laramide granite and related crystalline rocks. The portion of the Big Horn Mountains included in the watershed is made up of Cretaceous andesite and andesitic tuff, Precambrian granite and granite gneiss, and Quarternary basalt with small areas of rhyolite, shale, quartzite, and limestone. The Saddleback Mountains are composed mainly of Precambrian schist, Cretaceous andesite and Quaternary basalt. Gentle alluvial slopes extend basinward from the mountains. Quaternary-Tertiary sand, gravel and conglomerate are present near the mountain fronts with Quaternary clay, silt, sand, and gravel occurring at the lower elevations.

Deep or moderately deep soils are present on the relatively flat-lying (1-5% slope) alluvial plains. Medium or moderately-fine surface soils and subsoils are present on the smoother slopes near the center of the valley. Coarse or moderately-coarse soils are present on the upper fans of washes from the granitic mountains. Along the foot of the mountains, there is usually an area of shallow to moderately deep residual soils. These residual soils often have a medium-textured surface with gravel that is covered with dark desert varnish, and have slightly finer subsoils underlain at 12 to 28 inches by a strongly-cemented lime hardpan. Valley–fill alluvial soils originate in the granite, granite gneiss, schist, limestone, andesite, basalt, and shale rocks of the adjacent mountains. The soils in the plain are slightly to moderately erosive. Because the land surface is relatively flat and a sheet flow runoff condition prevails, erosion is generally not significant. Erosion is active in some of the channels and diversions constructed in and around the cultivated areas where flood flows are concentrated. Generally, the soils have a slow to very slow rate of water transmission and a slow to very slow infiltration rate when thoroughly wetted because of moderately-fine to fine texture or a layer that impedes downward movement of water.

The Saddleback FRS is located in a bajada area that consists primarily of unconsolidated and semi-consolidated (caliche-cemented) regolith which overlies mixed alluvium and intercalated igneous rocks. The regolith is primarily Quaternary-Tertiary alluvial fan deposits derived from the surrounding mountains. The regolith is composed of very loose to very dense semi-consolidated sands, silts, clays and gravels. Calcareous cementation governs the degree of apparent cohesion. Generally, density and cementation increase with depth. The cemented materials consist of gravelly to earth caliches with fairly high in-place densities.

1.2 Foundation Conditions

Surface soils along the centerline of the dam alignment were described in the Report of Geologic Investigation (SCS, 1978) as consisting of very loose to loose, silty or clayey sands and sandy silts and clays with variable amounts of gravel. These loose materials generally extend to depths of between 3 and 5 feet, and up to 9 feet in some locations. The loose materials are underlain by relatively firm and incompressible caliche of cemented sandy gravels, clayey, silty and/or gravelly sands, and sandy silts and clays. Relatively high permeability channel fill deposits of gravelly sands and sandy gravels were reportedly interbedded with, or incised into, the lower permeability caliche at several locations along the dam alignment. There was reportedly no surface expression of these channel fill deposits.

A bedrock high was reported by SCS (1978) in the vicinity of Station 115+00. According to SCS (1978), the loose surface soils were observed to a depth of approximately 6 feet at this location. Approximately 10 feet of cemented fangolmerate was present overlying volcanic bedrock. SCS (1978) concluded that the area of the bedrock high was relatively localized because the depth to the fanglomerate reportedly increased to approximately 12 to 13 feet at Stations 112+78 and 117+14, respectively. The as-built plan set also shows this to be a relatively localized feature. According to the as-built plan set, the bedrock high was encountered near Station 110+00. The reason for the discrepancy between the original Report of Geologic Investigation Report and the construction records is unknown.

The dam design called for removal of the loose surface soils to an average depth of 6 feet, with the final depths to be determined by the engineer after inspection of the materials encountered. The as-built plans indicate that the surface soils were removed to depths of between 3 and 9 feet.

1.3 Embankment

The designers reported that adequate borrow materials were available in the impoundment area upstream from the dam centerline, and that the soils in these locations correlated well with the soils along dam centerline. Abundant fine-grained material was reportedly available for the embankment core and the non-plastic fines, reportedly the most common surface material, were suitable for selective use in the embankment.

The Saddleback FRS was designed as a zoned earth embankment, constructed with impervious (Zone I) fill in the upstream zone, semi-pervious (Zone III) fill in the downstream zone, with an inclined, 5-foot wide chimney filter/drain (Zone II) separating the upstream and downstream zones. The dam has 3H:1V upstream and 2H:1V downstream slopes.

The as-built construction documentation indicates that the chimney filter/drain was actually constructed as a centrally-located vertical drain (Figure 1). The reason for the installation of a vertical drain rather than the inclined drain is unknown. The vertical filter/drain extends 1 foot below original grade into Zone I backfill of the stripping depth zone into the cutoff over most of the alignment. The vertical drain extends to the limits of the foundation excavation at three locations (Station 46+00 to Station 76+00, Station 104+00 to Station 114+00, and Station 238+20 to Station 265+70). The filter/drain was designed to terminate 3 feet below the crest of the embankment; however as-built drawings of the filter/drain rehabilitation performed in 1995 indicate that the depth to the top of the filter/drain is approximately 3.5 feet to 5.5 feet below the dam crest. The vertical filter/drain is connected to a system of drain outlets, comprising 2-feet by 2-feet gravel drains enclosing 6-inch diameter perforated outlet pipes. The drain outlets are installed at 400-feet intervals beginning at Station 8+00. The outfall pipe drains are intended to convey seepage intercepted at the central drain to the downstream toe.

According to the as-built plan set, the bottom of the foundation cutoff trench was grouted using dental grout between Stations 108+00 and 110+00, in the vicinity of the bedrock high described in the previous section. The grouting was presumably performed to fill cracks and surface irregularities and to provide a uniform base for construction of the cutoff trench.

A cut-off trench was installed along the entire dam to depths of up to 11 feet. Protective berms were constructed along two upstream sections at locations that coincide with two of the three vertical filter/drain extension locations (Station 46+00 to Station 76+00, and Station 240+00 to Station 265+00). It is assumed that the berms were incorporated into the design to improve stability during rapid drawdown loading conditions. A typical cross-section of the embankment is shown as Figure 1.

1.3.1 Embankment Materials

The embankment earth fill (Zones I and III) and filter/drain materials (Zone II) have the characteristics summarized on Table 1, based on the project design specifications.

Table 1. Embankment Material Zones – Saddleback FRS

Zone	Description	USCS	Properties	
I	Embankment earth fill, upstream from drain fill – silty gravel, silty sand, sandy silt, clayey sand, sandy or silty clay	GM, SM, ML, SC, CL	Minimum 15% passing No. 200 sieve	
II	Chimney Filter/Drain		Sieve	% Passing
			3-inch	100
			1-inch	80-100
			No. 4	50-75
			No. 40	15-35
			No. 100	5-18
			No. 200	0-3
III	Embankment earth fill, downstream from drain fill – silty gravel, silty sand, sandy silt, clayey sand, sandy or silty clay, sandy gravel, gravelly sand, sandy silt, silty sand	GM, SM, ML, SC, CL, GW or GP, SP, ML, SM	May contain non-plastic materials with less than 15% passing the No. 200 sieve	

The materials used to construct Zones I and III were derived from local borrow sources in the vicinity of the dam. The borrow materials were described by SCS (1978) as consisting of sandy clays, clayey sands, and silty sands with variable amounts of fine gravel. Fines in the borrow materials were reported to be non-plastic silts and slightly plastic clays. Logs of soil borings and test pits and laboratory test results for bulk samples obtained from the impoundment area approximately 500 feet to 800 feet from the dam centerline were provided by SCS (1978). At least twenty-two samples were collected for laboratory analysis from within the impoundment area along the alignment. The results of the laboratory testing reported by SCS (1978) are summarized on Table 2. SCS concluded that there was abundant material available for use in the embankment, however as-built gradation data were not available for review.

Although no Design Report for the Saddleback FRS was available for review during this Phase I Structures Assessment, it is assumed that the filter/drain was designed based on the results of laboratory testing of soils presented above, in a manner similar to the design of nearby flood retarding structures constructed by SCS at a similar time (for example, Harquahala FRS). The compatibility of the Zone II fill as a filter for Zone I/III was assessed and is discussed in the next section.

Table 2. Summary of Representative Laboratory Test Results for Borrow Materials

Borrow Area/ (samples)	USCS	% Fines (-#200)	PI (%)	G_s	γ_d (pcf)	w_{opt} (%)	φ (^{0})	c (tsf)
2101	CL	65	11		93.0	14.0		
2102	CL	59	13		91.6			
2103	CL	52	12	2.671		14.6		
2105	CL	60	19		89.6	14.2		
2106	SM	20	16					
2107	CL	63	17			13.2		
2108	CL	51	12	2.671			16 (undrained)	1.15 (undrained)
2109	CL	63	20		95.9	15.5		
2112 2112.1 2112.2	SC SM-SC SC	46 30 46	11 10 15			11.5		
2113.1 2113.2 2113.3	SM CL CL	44 77 60	NP 14 18			17.6 15.7		
2114	CL	58	17					
2115.1 2115.2	ML CL	62 59	NP 9		93.7			
2116	SM	47	NP			15.0	41 (drained)	0.75 (drained)
2117	SM	43	7		106.5	12.8		
2118 2118.1 2118.2	SM SM GM	27 21 8	NP NP 8					

1.3.2 Filter Compatibility

Zone II is shown on the as-built drawings as a 5-feet wide, vertical chimney drain with the centerline of the drain coincident with the centerline of the dam crest. The most important function for the Zone II materials is to serve as a filter to protect against potential internal erosion and piping of the Zone I materials in the event of transverse crack development.

Because of its critical function as a filter, the Zone II gradation was checked against current filter criteria in accordance with the NRCS, National Engineering Handbook, Chapter 26 "Gradation Design of Sand and Gravel Filters" (NRCS, 1994). Figure 2 shows gradation curves for Zone I "Base Soil" materials (graphed with solid symbols) developed for embankment soil samples collected during a geotechnical investigation of Saddleback FRS in 2001 (AMEC, 2001). The samples for which gradation curves were developed and are shown on Figure 2 (designated B-1 @ 4.5-6.0', B-3 @ 2.0-3.0', and #10 @ 4.5-5.5') were collected in an area where cracking has been documented and rehabilitation work has been performed (see Section 1.3.4), making this location critical with respect to material compatibility.

Soil sample B1 @ 4.5-6.0', a sandy clay having the Unified Soil Classification System (USCS) classification of CL, was collected upstream from the filter/drain. Soil samples B-3 @ 2.0-3.0' (clayey silt, CL-ML) and #10 @ 4.5-5.5' (silty sand, SM) were collected downstream from the filter/drain.

As can be seen in Figure 2, the original design specification band falls within the critical NRCS permeability (minimum D_{15}) and filtering (maximum D_{15}) criteria. As-built filter gradations could not be located for review. However, the original specification band indicates that filter gradations may have been allowed that are not ideal with respect to uniformity; that is, the filter may be too broadly graded. Modern NRCS criteria (green band shown on Figure 2) are intended to result in narrowly-graded filters. The primary purpose of this criterion is to prevent segregation of the filter during placement. The fine range of the design filter band (black lines) does fit all criteria, and it is possible that the in-place filter, if produced on the fine side of the specification band, and properly constructed, does meet all the current criteria for modern filters. In addition, Gannett Fleming verified the internal stability of the specified gradations using a procedure outlined by Kenney and Lau (1985).

However, if the filter materials were produced within the specification band, but on the coarse side, outside of the current criteria, it is possible that the as-built filter may have been too broadly graded, which would have made the materials prone to segregation during placement. If segregation did occur, it is possible that some fines from Zone I could penetrate into Zone II under a concentrated leak through a transverse crack. Additional sampling and analyses of the actual in-place filter could be done to further evaluate the efficacy of the Zone II filter, as outlined under Recommendations.

1.3.3 Dam Rehabilitation

In 1995, rehabilitation of the dam was conducted between Stations 44+00 and 52+00 to remediate longitudinal cracking in the dam crest, above the central filter/drain. During the rehabilitation work, ADWR (1995) noted that longitudinal cracks along the centerline of the dam crest were between ½-inch and ¾-inch wide and extend entirely through the soil cover over the filter/drain and at some locations extend up to 12 inches into the underlying filter/drain. It was also noted that the upper 6 to 12 inches of Zone II drain fill material was contaminated with fine-grained embankment materials.

The rehabilitation consisted of excavating a 2-feet wide trench along the crest of the dam between Stations 44+00 and 52+00, and backfilling and compacting the excavation with filter/drain material up to the crest. The excavation and manual removal of contaminated drain fill materials continued 1 foot into the uncontaminated material.

The gradation of the drain fill material placed during the rehabilitation was designed to be identical to the originally specified filter/drain material. It was reported in ADWR (1995) that laboratory testing confirmed that at least two of the three soil samples tested during the rehabilitation met the design specification for gradation, however, the laboratory results were not available for review during this Phase I Structures Assessment.

1.4 Original Slope Stability Analysis

Slope stability analyses were performed by the designers in general accordance with SCS guidelines (SCS 1985). The stability analyses utilized dry and saturated embankment soil unit weight values of 122 pounds per cubic foot (pcf) and 132 pcf, respectively. Table 3 summarizes the shear strength parameter values used by designers for the embankment slope stability analyses. Direct shear testing (CD testing) provided the drained (effective) shear strength parameters and undrained shear strength parameters were determined from the triaxial testing (CU testing). Laboratory reports indicated that seven of the ten direct shear tests and all three of the triaxial tests were performed on remolded samples. The values shown in Table 3 were adopted by the designers as composite values for the embankment soils based on the results of shear strength data from twelve tests (three CU tests and nine CD tests) of embankment materials (Toups Corporation, 1979).

Table 3. Embankment Shear Strength Parameters Used in Stability Analysis

Shear Strength Parameter	Drained Testing	Undrained Testing
Angle of internal friction	$\phi' = 31^{\circ}$	$\phi' = 16^{0}$
Cohesion	c' = 0.3 ksf	c = 1.0 ksf

The slope stability analysis assumed a circular arc failure surface extending through the toe of the embankment slope would have the minimum factor of safety. Slope stability analyses were conducted for this critical slip surface for loading conditions at the end of construction, during rapid reservoir drawdown and for steady-state conditions. Factors of safety were calculated for each of these loading conditions and are summarized on Table 4.

Table 4. Original Slope Stability Analyses Results

Slope	Conditions	Minimum F.S.	
		Effective Stress Analysis	Total Stress Analysis
2H:1V downstream	End of construction	2.3	NA
2H:1V downstream	Steady-state seepage	2.9	3.3
3H:1V upstream	Rapid drawdown	2.1	3.7

Although recommended by SCS (1985), the designers did not conduct a pseudo-static stability analysis of the downstream slope to assess the embankment slope stability under earthquake loading conditions.

1.4.1 End of construction

Due to the relatively low embankment height and the placement of materials at below optimum moisture content, pore pressures were not included in the end of construction analysis, and the limiting strength values for drained conditions were used in the slope stability analysis (Toups, 1979). The factor of safety computed for the downstream slope on the basis of three trials was 2.3, which is above the allowable minimum value of 1.3 for end of construction loading conditions.

1.4.2 Steady-state seepage condition

Slope stability was assessed for steady-state seepage conditions using both drained (effective stress analysis (ESA)) and undrained (total stress analysis (TSA)) shear strength parameters. Factors of safety of 2.9 and 3.3 were calculated for ESA and TSA, respectively.

1..3 Rapid drawdown condition

For this analysis it was assumed that a full phreatic line could develop on the upstream slope to the elevation of the emergency spillway. Factors of safety were calculated for the upstream slope using both drained and undrained shear strength parameters. The factor of safety calculated using drained strength was 2.1 and the factor of safety calculated using undrained strength was 3.7.

2.0 RECOMMENDATIONS

2.1 Phase II Additional Evaluation of Zone II Drain Materials

The compatibility of the embankment materials and the ability of Zone II to adequately act as a filter for Zone I was evaluated for this Phase I Structures Assessment and is discussed in Section 1.3.2. No gradation data for the as-built filter materials were available for review, therefore the assessment of filter compatibility was based only on the specified filter gradation with no confirmation that the original filter was built within the specified gradation range. According to ADWR (1995), three filter gradation samples were collected during the dam rehabilitation work (see Section 1.3.3) and were within specifications, however these test results were not available for review. It is recommended that in-place filter and drain materials be sampled, and gradation data obtained, to allow a check of the compatibility between the various material zones, and to evaluate the filter for segregation problems.

2.2 Phase II Documentation of Slope Stability and Seepage Analyses

Under reasonable loading conditions for Saddleback FRS, it is expected that both upstream and downstream slopes will be stable. However, adequate documentation of slope stability factors of safety for specified loading and design criteria established by appropriate jurisdictional agencies is not available. Additional slope stability analyses are recommended to document the slope stability factors of safety for Saddleback FRS.

Table 5 shows the definitions of various loading conditions and a comparison between the current NRCS design criteria that are outlined in TR-60 (SCS, 1985), and the current criteria as presented in the Arizona Department of Water Resources (ADWR) dam safety rules and regulations for jurisdictional dams.

Table 5. Slope Stability Design Criteria

Loading Condition	TR-60 (SCS, 1985)	ADWR[1]
End of Construction (upstream and downstream slopes)	1.4	1.3[2]
Rapid Drawdown (upstream slope)	1.2	1.2
Steady seepage without seismic forces, phreatic surface fully developed from full pool (downstream slope)	1.5	1.5

Loading Condition	TR-60 (SCS, 1985)	ADWR[1]
Steady seepage without seismic forces, phreatic surface developed from critical partial pool elevation (upstream slope)	n/a	1.5
Steady seepage with seismic forces, phreatic surface fully developed from full pool (downstream slope)	1.1	n/a[3]

[1] From R-15-1216(B)(1)(c)(i) Table 5, effective June 12, 2000
[2] ADWR specifies FOS = 1.4 for EOC loading for dams > 50 feet high on weak foundations
[3] ADWR specifies pseudo static analysis for embankment dams not subject to liquefaction, and having maximum peak bedrock acceleration < 0.2 g, using a pseudo-static coefficient at least 60% of the maximum peak bedrock acceleration

The original stability analysis does not completely document factors of safety for all the loading conditions required under current NRCS or ADWR criteria. Table 6 summarizes the results from the original stability analysis and indicates where additional analyses are required.

Table 6. Slope Stability Documentation to Date and Additional Analyses Required to Comply with Current Design Criteria

Loading Condition	Minimum Factor of Safety from Original Analysis	Recommendation (see text for discussion)
End of Construction (upstream and downstream slope)	2.3 (for downstream slope)	(1)
Rapid Drawdown (upstream slope)	2.1	(2)
Steady state seepage without seismic forces, phreatic surface fully developed from full pool (downstream slope)	2.9	(3)
Steady state seepage wtihout seismic forces, phreatic surface developed from critical partial pool elevation (upstream slope)	Not evaluated	(4)
Steady state seepage with seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	Not evaluated	(5)

(1) **End of construction (upstream and downstream slope)**: The original factor of safety calculated for the downstream slope under end of construction loading conditions achieved the minimum ADWR criteria of 1.3 (see Table 5). Evaluation of the upstream slope stability under these loading conditions was not performed. However, this loading condition is now irrelevant, since the construction-induced pore pressures have been dissipated, and the embankment has been stable since its construction. Additional analysis for this loading condition is therefore not required.

(2) **Rapid drawdown (upstream slope)**: The original stability analysis for this loading condition resulted in calculated factors of safety that are currently acceptable under ADWR rules. Additional analyses are not required.

(3) **Steady state seepage without seismic forces**: The original factor of safety calculated for this loading condition (FS = 2.9) is significantly higher than the minimum criteria of 1.5 (see Table 5). Additional analyses are not required.

(4) **Steady state seepage, partial pool elevation (upstream slope)**: The original analysis did not evaluate upstream slope stability under this loading condition. The ADWR criteria for partial pool conditions is intended for water retention dams, in which a steady state phreatic line may develop for intermediate pool elevations. The factor of safety may be lower for the intermediate pool conditions than the steady state condition under maximum pool. The following analysis could be done to document the minimum partial pool factor of safety, under the scenario that the outlet works are clogged such that the steady state phreatic line develops:

a. Perform seepage analyses under various partial pool elevations to establish the steady state pore pressure distributions within the dam at each pool elevation.
b. Conduct slope stability analyses for each partial pool seepage analysis result, and graph the results as factor of safety versus pool elevation.
c. Report the minimum factor of safety and corresponding pool elevation.

(5) **Steady state seepage with seismic forces (downstream slope)**: No seismic stability analysis was documented for Saddleback FRS. To document seismic stability under current design criteria, a pseudo-static stability analysis is recommended. The analysis should use a peak ground acceleration (PGA) of 0.1g and the ADWR recommendation of a pseudo-static coefficient equal to 60% of the PGA.

3.0 REFERENCES

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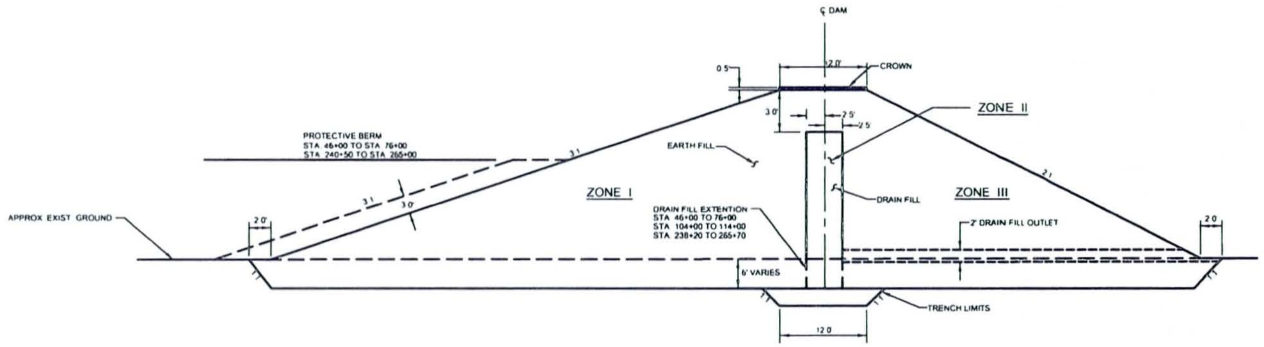
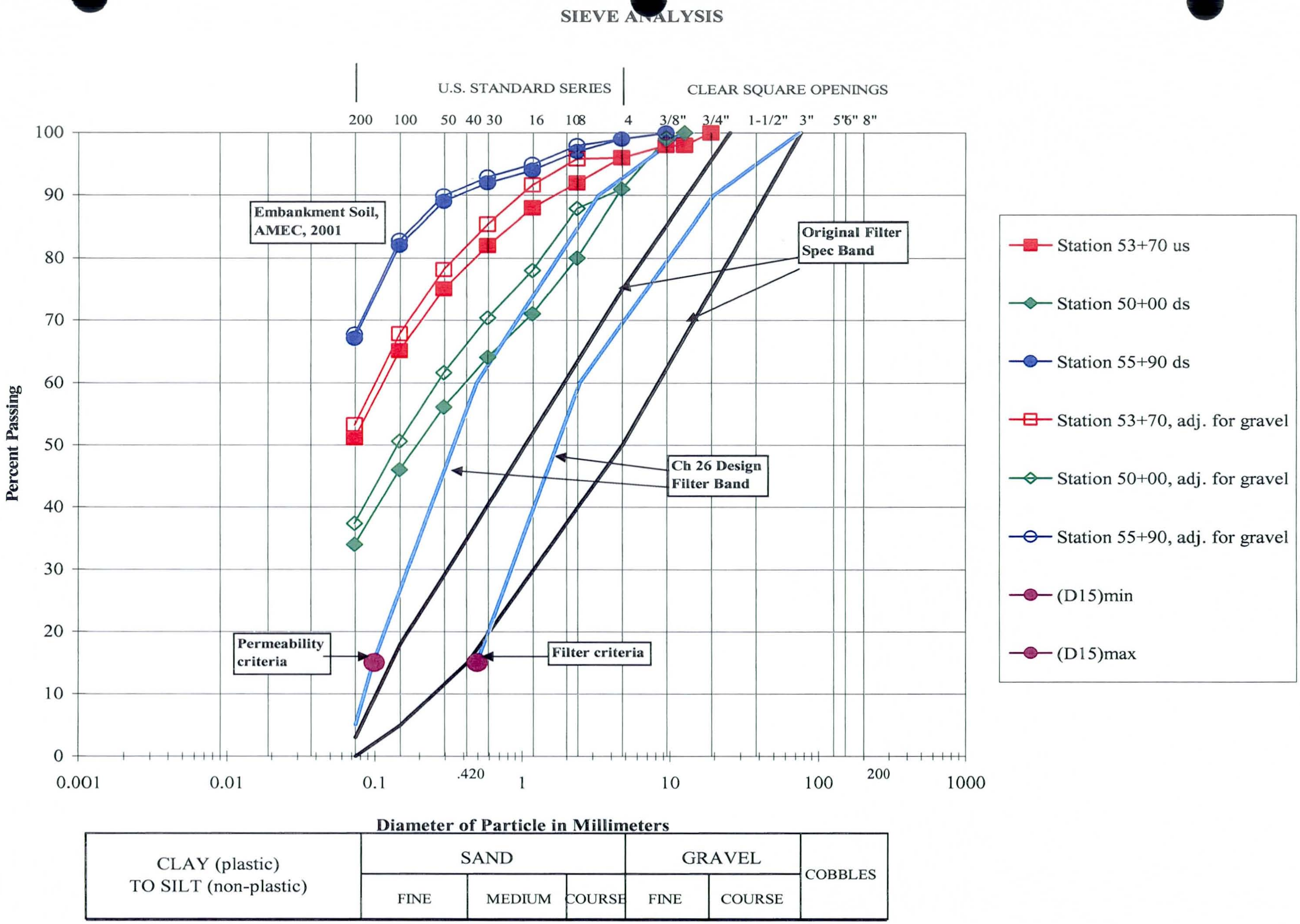


FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
DRAFT GEOTECHNICAL MEMORANDUM

FIGURE 1
TYPICAL EMBANKMENT SECTION
SADDLEBACK FLOOD
RETARDING STRUCTURE

DR	DES	CK	SHEET NO	TOTAL SHEETS	AS BUILT
DATE	DATE	DATE			
APPROXIMATE SCALE 1" = 30'					

K:\43394\harquahala-xsecs.dwg Jun 21, 2005 fstauffer



Gannett Fleming

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
GEOTECHNICAL MEMORANDUM – SADDLEBACK FRS
FILTER CRITERIA

FIGURE 2

Saddleback FRS

Subsidence Survey Data Review

Flood Control District of Maricopa County
Structures Management Branch
Dam Safety Program

11/11/2005

Dam Crest elevation data was not collected prior to year-2002. Previous survey activity was confined to collecting settlement monument elevations which are physically located offset from dam centerline on the downstream edge of the dam crest. **Table 1**, compares the 2003 Crest Elevations taken at the low areas observed in the vicinity of the crest settlement monuments with the Adjusted Design Crest Elevation. The Design Crest elevation is based on elevations taken at the low areas observed in the vicinity of the crest settlement monuments with the Adjusted Design Crest Elevation. The Design Crest elevation is based on 1929 NGVD vertical datum while the 2003 survey is based on NAVD 1988 vertical datum. The Design Crest elevation values must be adjusted for crest elevation comparison with 2003 data. Details of the adjustment calculations are outlined on page 17, **"Reference Marks."**

Figure 1-1 compares the Adjusted Design Crest Elevation and the year-2003 survey data listed in **Table 1.**

Figure 1-2 displays the relative change in Crest Elevation between the Adjusted Design Crest Elevation and the year-2003 survey crest elevations listed in **Table 1.**

Marker	Station	Design Crest Elev	Adj Design Crest Elev	2003 Dam Crest Elev	2003 - Adj Dgn
CRST1	10+00	1193.0	1194.999	1196.128	1.129
CRST2	20+00	1193.0	1194.999	1195.889	0.890
CRST3	30+00	1193.0	1194.999	1195.310	0.311
CRST4	40+00	1193.0	1194.999	1195.695	0.696
CRST5	50+00	1193.33	1195.329	1195.979	0.650
CRST5A	52+50	1193.33	1195.329	1196.119	0.790
CRST5B	55+00	1193.33	1195.329	1196.010	0.681
CRST5C	57+50	1193.33	1195.329	1195.995	0.666
CRST6	60+00	1193.33	1195.329	1195.901	0.572
CRST7	70+00	1193.33	1195.329	1195.219	-0.110
CRST8	80+00	1193.0	1194.999	1195.625	0.626
CRST9	90+00	1193.0	1194.999	1195.614	0.615
CRST10	99+80	1193.0	1194.999	1195.761	0.762
CRST11	110+05	1193.0	1194.999	1195.450	0.451
CRST12	120+00	1193.0	1194.999	1195.727	0.728
CRST13	130+00	1193.0	1194.999	1195.456	0.457

(Fig. 1-1 Plot Data) (Fig. 1-2 Plot Data)

Marker	Station	Design Crest Elev	Adj Design Crest Elev	2003 Dam Crest Elev	2003 - Adj Dgn
CRST14	140+00	1193.0	1194.999	1195.580	0.581
CRST15	150+00	1193.0	1194.999	1195.573	0.574
CRST16	160+00	1193.0	1194.999	1195.539	0.540
CRST17	170+00	1193.0	1194.999	1195.465	0.466
CRST18	180+00	1194.0	1195.999	1196.686	0.687
CRST19	190+00	1194.0	1195.999	1196.940	0.941
CRST20	200+00	1194.0	1195.999	1196.439	0.440
CRST21	210+00	1194.0	1195.999	1196.492	0.493
CRST22	220+00	1194.0	1195.999	1196.357	0.358
CRST23	230+00	1194.0	1195.999	1196.360	0.361
CRST24	240+00	1194.0	1195.999	1196.536	0.537
CRST25	250+00	1194.0	1195.999	1196.266	0.267
CRST25B	257+50	1194.0	1195.999	1196.879	0.880
CRST26	260+00	1194.0	1195.999	1196.174	0.175
CRST26A	262+50	1194.0	1195.999	1196.519	0.520
CRST27	270+00	1194.0	1195.999	1196.230	0.231

(Fig. 1-1 Plot Data) (Fig. 1-2 Plot Data)

Table 1
STA 10+00 to STA 270+00 Dam Crest Elevations and Relative Changes in Elevation

Dam Crest Elevations

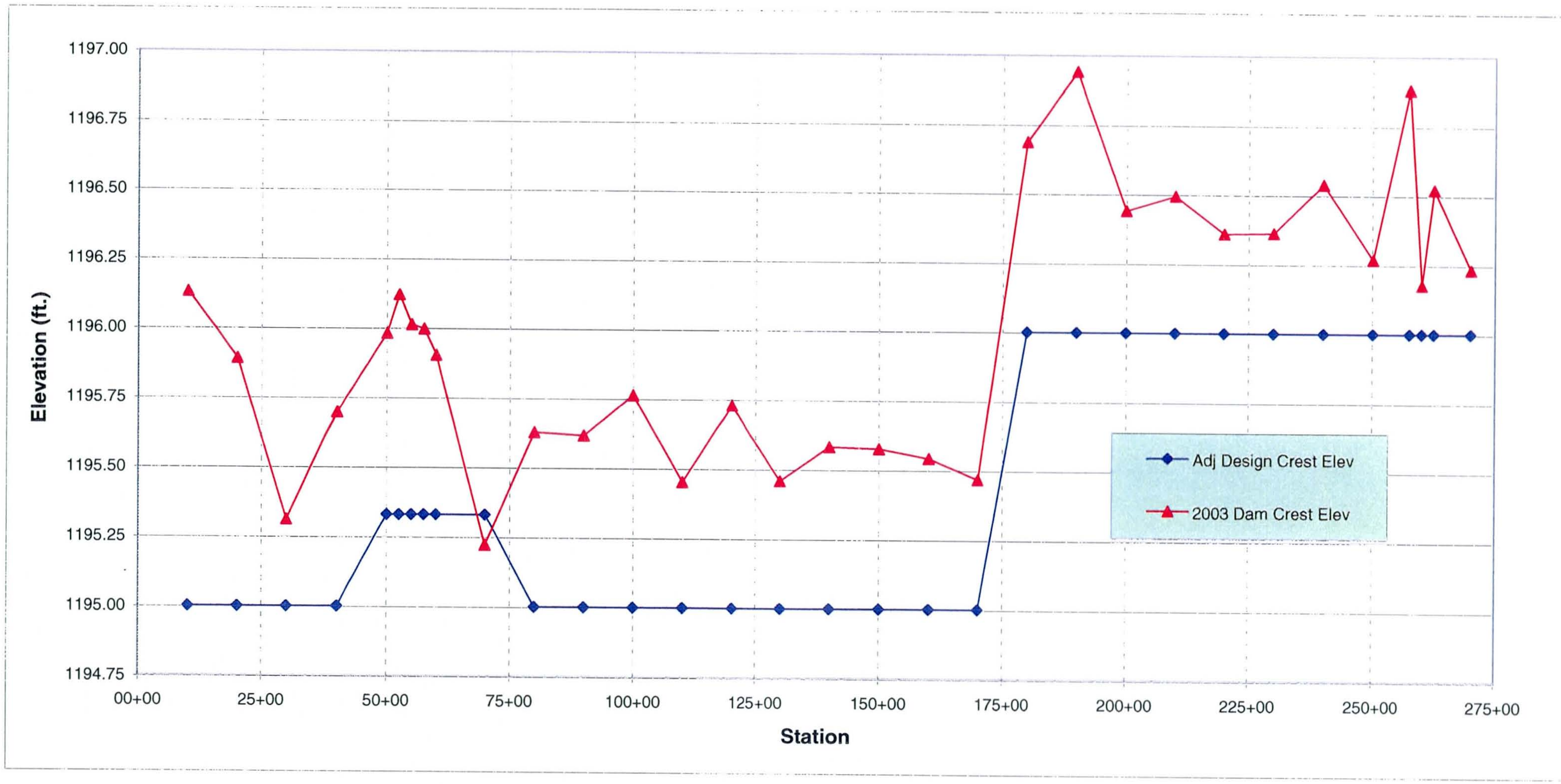


Figure 1-1
Dam Crest Elevation Comparison Chart

Dam Crest Elevations

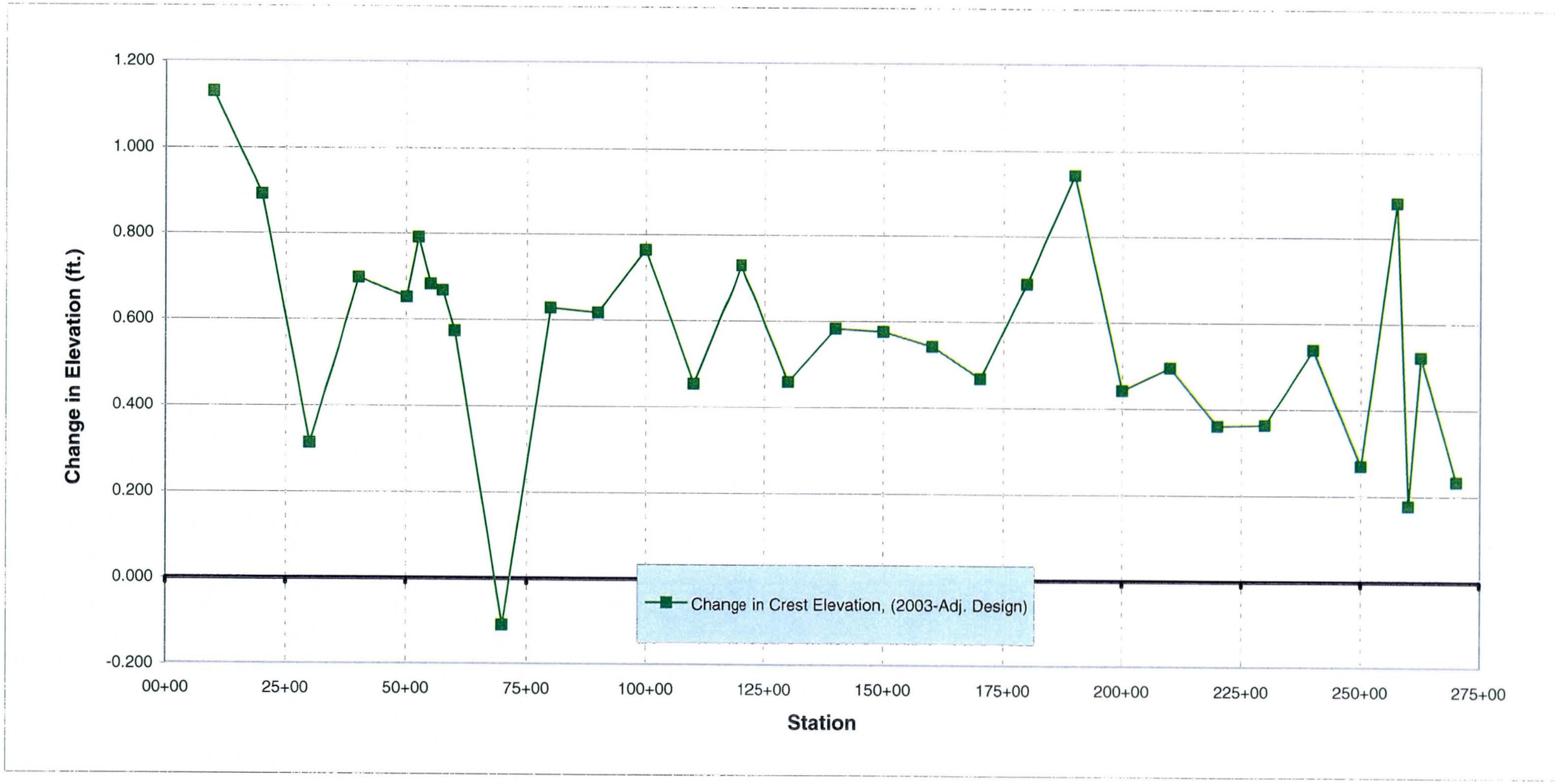


Figure 1-2
Change in Dam Crest Elevation Chart

Settlement Monuments - Crest

Table 2 summarizes elevation data for Crest Settlement Monuments. Elevation survey data collected prior to year-2001 is based on NGVD 1929 vertical datum, and have been adjusted to NAVD 1988 vertical datum. Adjustment details are outlined on page 17, "**Reference Marks**."

Figure 2-1 compares Crest Settlement Monument Elevations for years 1983, 1986, 2001, 2002, and 2003. Elevations for surveys prior to year-2001 are adjusted as noted.

		Crest Monument Survey Data						
Marker	**Station**	**1983**	**Adj. 1983**	**1986**	**Adj. 1986**	**2001**	**2002**	**2003**
A1	10+00	1192.876	1194.875	1192.727	1194.689	1194.804	1194.965	1194.945
A2	20+00	1192.578	1194.577	1192.669	1194.631	1194.479	1194.626	1194.634
A3	30+00	1191.798	1193.797	1191.893	1193.855	1193.707	1193.819	1193.819
A4	40+00	1192.994	1194.993	1193.073	1195.035	1194.881	1195.008	1194.998
A5	50+00	1193.219	1195.218	1193.257	1195.219	1195.107	1195.231	1195.205
A5A	52+50					1195.626	1195.752	1195.700
A5B	55+00					1195.252	1195.338	1195.387
A5C	57+50					1195.339	1195.442	1195.484
A6	60+00	1192.824	1194.823	1192.832	1194.794	1194.586	1194.711	1194.725
A7	70+00	1193.022	1195.021	1193.016	1194.978	1194.818	1194.891	1194.916
A8	80+00	1192.614	1194.613	1192.609	1194.571	1194.453	1194.526	1194.578
A9	90+00	1192.780	1194.779	1192.780	1194.742	1194.603	1194.735	1194.731
A10	99+80	1193.703	1195.702	1193.697	1195.659	1195.533	1195.657	1195.688
A11	110+05	1193.026	1195.025	1193.067	1195.029	1194.961	1195.073	1195.073
A12	120+00	1193.320	1195.319	1193.347	1195.309	1195.274	1195.363	1195.355
A13	130+00	1192.866	1194.865	1192.892	1194.854	1194.800	1194.902	1194.922
			(Fig. 2-1)				(Fig. 2-1 Plot Data)	

Notes: 1) = No data was available for these monuments

2) Construction completed in 1982.

3) See location of crest monuments on page 18, "**Floodplain View**'

Table 2-A

STA 10+00 to STA 130+00 Crest Settlement Monument Elevations

Settlement Monuments - Crest

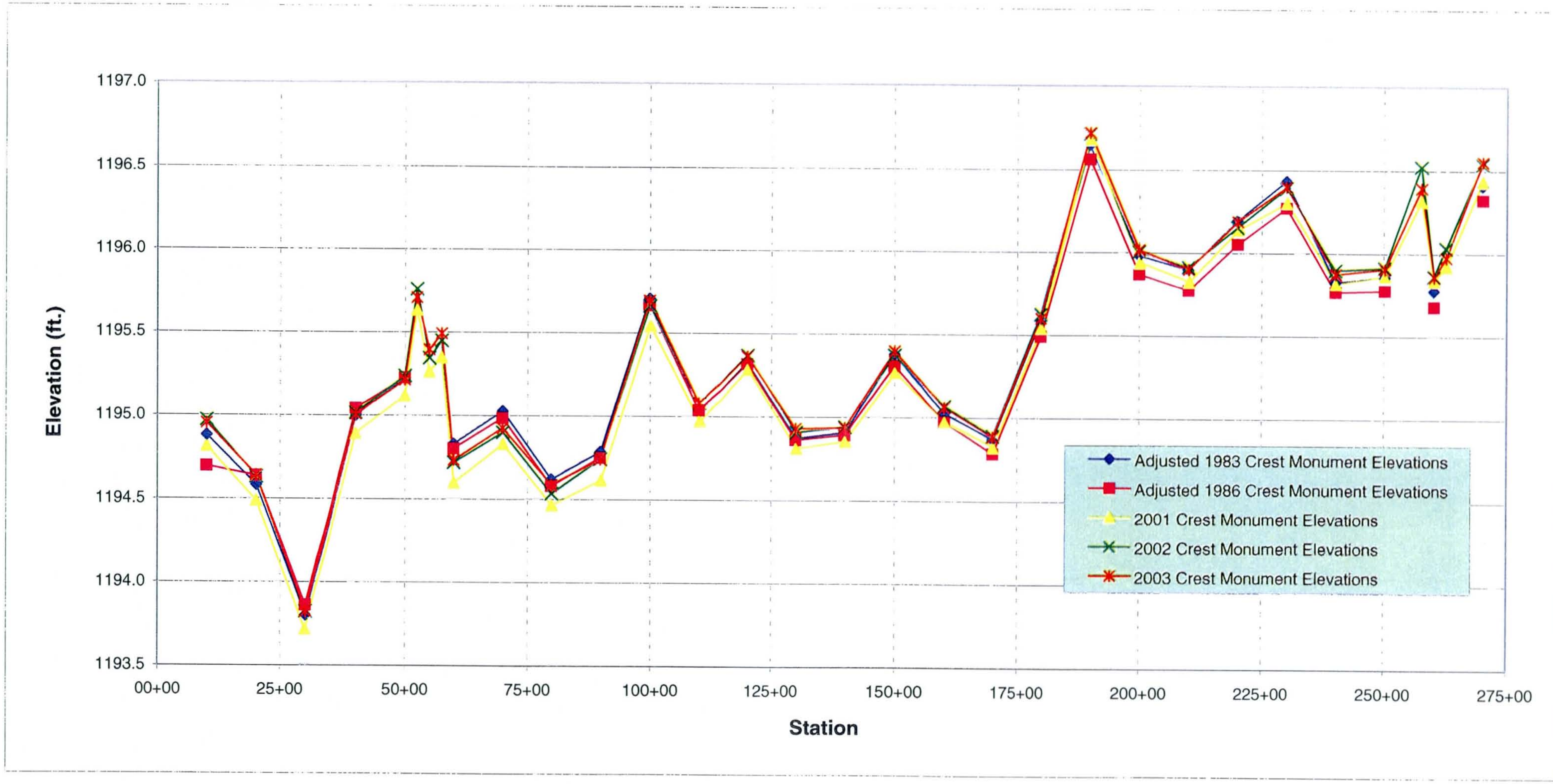
Marker	Station	Crest Monument Survey Data						
		1983	Adj. 1983	1986	Adj. 1986	2001	2002	2003
A14	140+00	1192.903	1194.902	1192.923	1194.885	1194.844	1194.935	1194.928
A15	150+00	1193.375	1195.374	1193.341	1195.303	1195.259	1195.368	1195.391
A16	160+00	1193.020	1195.019	1193.003	1194.965	1194.964	1195.062	1195.054
A17	170+00	1192.865	1194.864	1192.815	1194.777	1194.814	1194.883	1194.876
A18	180+00	1193.582	1195.581	1193.523	1195.485	1195.533	1195.618	1195.592
A19	190+00	1194.647	1196.646	1194.596	1196.558	1196.676	1196.711	1196.717
A20	200+00	1193.979	1195.978	1193.902	1195.864	1195.931	1196.001	1196.011
A21	210+00	1193.895	1195.894	1193.805	1195.767	1195.816	1195.909	1195.893
A22	220+00	1194.193	1196.192	1194.085	1196.047	1196.128	1196.149	1196.182
A23	230+00	1194.431	1196.430	1194.303	1196.265	1196.294	1196.385	1196.392
A24	240+00	1193.823	1195.822	1193.798	1195.760	1195.808	1195.894	1195.868
A25	250+00	1193.853	1195.852	1193.806	1195.768	1195.855	1195.910	1195.900
A25B	257+50					1196.309	1196.519	1196.386
A26	260+00	1193.771	1195.770	1193.709	1195.671	1195.826	1195.858	1195.852
A26A	262+50					1195.913	1196.028	1195.968
A27	270+00	1194.406	1196.405	1194.354	1196.316	1196.426	1196.536	1196.547
			(Fig. 2-1)			(Fig. 2-1 Plot Data)		

Table 2-B
STA 140+00 to STA 270+00 Crest Settlement Monument Elevations

SADDLEBACK FRS

Subsidence Survey Data Review

Settlement Monuments - Crest



<u>**Figure 2-1**</u>

Crest Settlement Monuments' Elevation Chart

Settlement Monuments - Crest

Table 3 below summarizes the Crest Monument Elevations adjusted to 1988 NAVD for the years 1983, 1986, 2001, 2002, and 2003, and calculates the elevation change from the 1983 initial survey as the baseline. **Figure 3-1** illustrates the relative change in Settlement monuments as calculated in **Table 3**.

Marker	Station	Crest Monument Survey Data				
		Adj 83	Adj 86	2001	2002	2003
A1	10+00	1194.875	1194.689	1194.804	1194.965	1194.945
A2	20+00	1194.577	1194.631	1194.479	1194.626	1194.634
A3	30+00	1193.797	1193.855	1193.707	1193.819	1193.819
A4	40+00	1194.993	1195.035	1194.881	1195.008	1194.998
A5	50+00	1195.218	1195.219	1195.107	1195.231	1195.205
A6	60+00	1194.823	1194.794	1194.586	1194.711	1194.725
A7	70+00	1195.021	1194.978	1194.818	1194.891	1194.916
A8	80+00	1194.613	1194.571	1194.453	1194.526	1194.578
A9	90+00	1194.779	1194.742	1194.603	1194.735	1194.731
A10	99+80	1195.702	1195.659	1195.533	1195.657	1195.688
A11	110+05	1195.025	1195.029	1194.961	1195.073	1195.073
A12	120+00	1195.319	1195.309	1195.274	1195.363	1195.355
A13	130+00	1194.865	1194.854	1194.800	1194.902	1194.922
A14	140+00	1194.902	1194.885	1194.844	1194.935	1194.928
A15	150+00	1195.374	1195.303	1195.259	1195.368	1195.391
A16	160+00	1195.019	1194.965	1194.964	1195.062	1195.054
A17	170+00	1194.864	1194.777	1194.814	1194.883	1194.876
A18	180+00	1195.581	1195.485	1195.533	1195.618	1195.592
A19	190+00	1196.646	1196.558	1196.676	1196.711	1196.717
A20	200+00	1195.978	1195.864	1195.931	1196.001	1196.011
A21	210+00	1195.894	1195.767	1195.816	1195.909	1195.893
A22	220+00	1196.192	1196.047	1196.128	1196.149	1196.182
A23	230+00	1196.430	1196.265	1196.294	1196.385	1196.392
A24	240+00	1195.822	1195.760	1195.808	1195.894	1195.868
A25	250+00	1195.852	1195.768	1195.855	1195.910	1195.900
A26	260+00	1195.770	1195.671	1195.826	1195.858	1195.852
A27	270+00	1196.405	1196.316	1196.426	1196.536	1196.547

Adj 86 - Adj 83	2001 - Adj 83	2002 - Adj 83	2003 - Adj 83
-0.186	-0.071	0.090	0.070
0.054	-0.098	0.049	0.057
0.058	-0.090	0.022	0.022
0.042	-0.112	0.015	0.005
0.001	-0.111	0.013	-0.013
-0.029	-0.237	-0.112	-0.098
-0.043	-0.203	-0.130	-0.105
-0.042	-0.160	-0.087	-0.035
-0.037	-0.176	-0.044	-0.048
-0.043	-0.169	-0.045	-0.014
0.004	-0.064	0.048	0.048
-0.010	-0.045	0.044	0.036
-0.011	-0.065	0.037	0.057
-0.017	-0.058	0.033	0.026
-0.071	-0.115	-0.006	0.017
-0.054	-0.055	0.043	0.035
-0.087	-0.050	0.019	0.012
-0.096	-0.048	0.037	0.011
-0.088	0.030	0.065	0.071
-0.114	-0.047	0.023	0.033
-0.127	-0.078	0.015	-0.001
-0.145	-0.064	-0.043	-0.010
-0.165	-0.136	-0.045	-0.038
-0.062	-0.014	0.072	0.046
-0.084	0.003	0.058	0.048
-0.099	0.056	0.088	0.082
-0.089	0.021	0.131	0.142

(Fig. 3-1 Plot Data)

Table 3

Crest Monument Elevation Change from Initial 1983 Survey Data as Baseline

Settlement Monuments - Crest

Figure 3-1
Relative Change in Crest Monument Elevation Chart, 1983 Survey Data Baseline Elevation Reference

Settlement Monuments - Toe

Table 4 below summarizes the Toe Monument Elevations adjusted to 1988 NAVD for the years 1983, 1986, 2001, 2002, and 2003, and calculates the elevation change from the 1983 initial survey as the baseline. **Figure 4-1** illustrates the relative change in Settlement monuments as calculated in **Table 4**.

Marker	Station	Toe Monument Survey Data						
		1983	Adj 83	1986	Adj 86	2001	2002	2003
B1	10+00	1185.953	1187.952	1186.044	1188.006	1187.949	1188.098	1188.114
B2	20+00	1179.673	1181.672	1179.781	1181.743	1181.712	1181.849	1181.883
B3	30+00	1181.133	1183.132	1181.252	1183.214	1183.202	1183.281	1183.302
B4	40+00	1181.365	1183.364	1181.482	1183.444	1183.331	1183.504	1183.508
B5	50+00	1177.253	1179.252	1177.379	1179.341	1179.291	1179.404	1179.450
B6	60+00	1173.820	1175.819	1173.907	1175.869	1175.830	1175.904	1175.911
B7	70+00	1175.994	1177.993	1176.044	1178.006	1177.960	1178.023	1178.043
B8	80+00	1179.008	1181.007	1179.086	1181.048	1181.067	1181.137	1181.158
B9	90+00	1180.747	1182.746	1180.826	1182.788	1182.794	1182.891	1182.898
B10	99+80	1184.245	1186.244	1184.271	1186.233	1186.235	1186.365	1186.328
B11	110+05	1185.782	1187.781	1185.839	1187.801	1187.798	1187.917	1187.930
B12	120+00	1186.592	1188.591	1186.622	1188.584	1188.606	1188.735	1188.690
B13	130+00	1185.577	1187.576	1185.590	1187.552	1187.764	1187.813	1187.834
B14	140+00	1185.120	1187.119	1185.118	1187.080	1187.142	1187.206	1187.173
B15	150+00	1185.474	1187.473	1185.443	1187.405	1187.469	1187.604	1187.574
B16	160+00	1185.552	1187.551	1185.526	1187.488	1187.540	1187.672	1187.687
B17	170+00	1186.243	1188.242	1186.146	1188.108	1188.209	1188.250	1188.295
B18	180+00	1186.849	1188.848	1186.787	1188.749	1188.940	1188.921	1188.969
B19	190+00	1190.891	1192.890	1190.514	1192.476	1192.992	1193.022	1193.051
B20	200+00	1187.657	1189.656	1187.647	1189.609	1189.749	1189.876	1189.868
B21	210+00	1190.592	1192.591	1190.514	1192.476	1192.605	1192.677	1192.671
B22	220+00	1189.433	1191.432	1189.359	1191.321	1191.452	1191.542	1191.524
B23	230+00	1186.800	1188.799	1186.732	1188.694	1188.789	1188.996	1188.883
B24	240+00	1185.880	1187.879	1185.847	1187.809	1187.968	1188.051	1188.031
B25	250+00	1185.337	1187.336	1185.287	1187.249	1186.893	1187.040	1187.036
B26	260+00	1183.295	1185.294	1183.239	1185.201	1185.354	1185.463	1185.439
B27	270+00	1189.645	1191.644	1189.605	1191.567	1191.714	1191.875	1191.794

Adj 86 - Adj 83	2001 - Adj 83	2002 - Adj 83	2003 - Adj 83
0.054	-0.003	0.146	0.162
0.071	0.040	-0.003	0.211
0.082	0.070	0.149	0.170
0.080	-0.033	0.070	0.144
0.089	0.039	0.152	0.198
0.050	0.011	0.039	0.092
0.013	-0.033	0.030	0.050
0.041	0.060	-0.033	0.151
0.042	0.048	0.145	0.152
-0.011	-0.009	0.048	0.084
0.020	0.017	0.136	0.149
-0.007	0.015	0.017	0.099
-0.024	0.188	0.237	0.258
-0.039	0.023	0.188	0.054
-0.068	-0.004	0.131	0.101
-0.063	-0.011	-0.004	0.136
-0.134	-0.033	0.008	0.053
-0.099	0.092	-0.033	0.121
-0.414	0.102	0.132	0.161
-0.047	0.093	0.102	0.212
-0.115	0.014	0.086	0.080
-0.111	0.020	0.014	0.092
-0.105	-0.010	0.197	0.084
-0.070	0.089	-0.010	0.152
-0.087	-0.443	-0.296	-0.300
-0.093	0.060	-0.443	0.145
-0.077	0.070	0.231	0.150

(Fig. 4-1 Plot Data)

<u>Table 4</u>

Toe Monument Elevation Change from Initial 1984 Survey Data as Baseline

Settlement Monuments - Toe

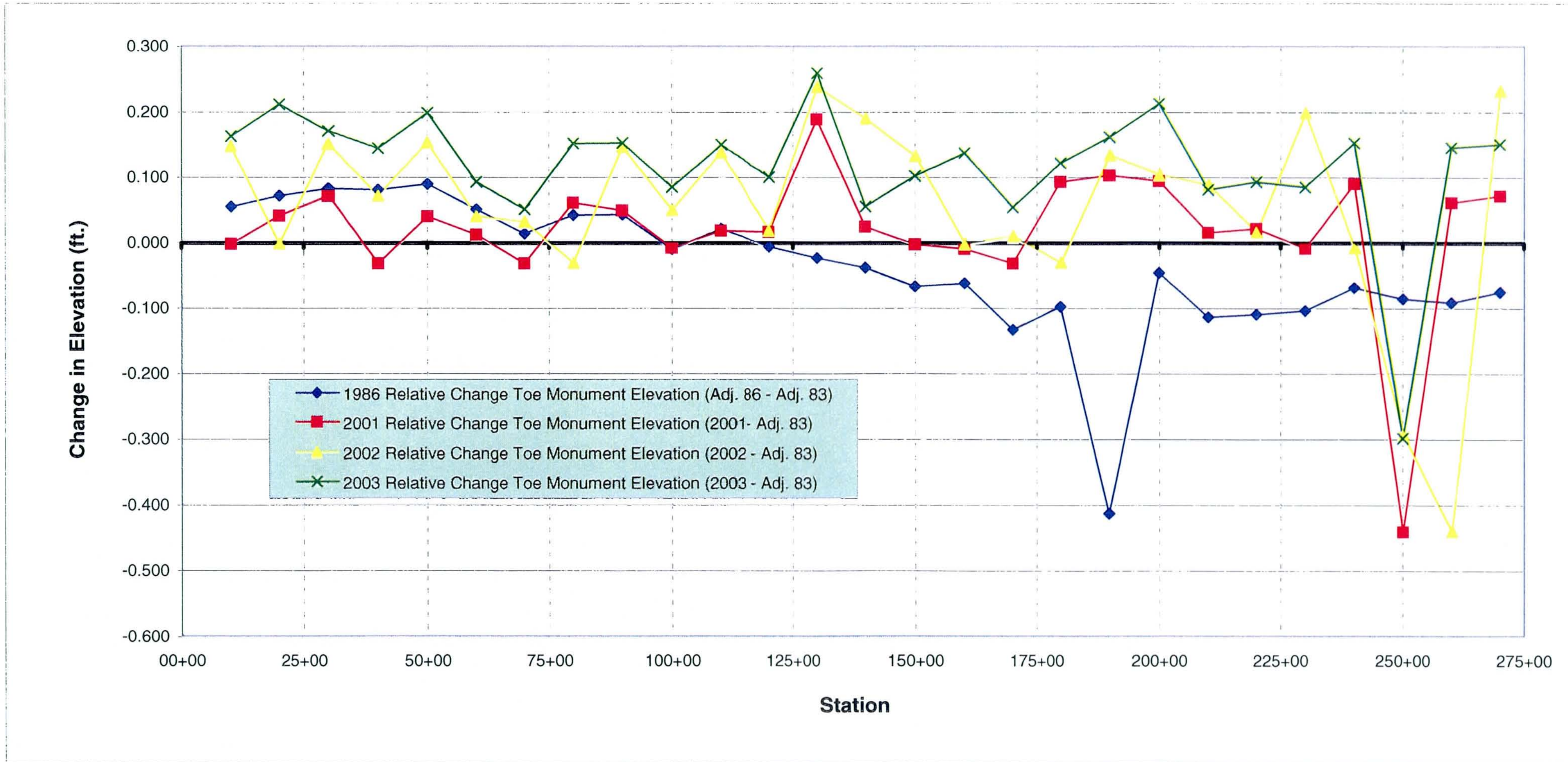


Figure 4-1
Relative Change in Toe Monument Elevation Chart, 1983 Survey Data Baseline Elevation Reference

Settlement Monuments - Crest to Toe

Table 5 below displays the elevation difference between each settlement monument pair by subtracting the toe settlement monument from its corresponding crest settlement monument.
Figure 5-1 plots the elevation difference between crest and corresponding toe monuments at each station.

Monument Pair	Station	Elevation Difference: Crest Mon. to Toe Mon. Adj 83	Adj 86	2001	2002	2003
A1-B1	10+00	6.923	6.683	6.855	6.867	6.831
A2-B2	20+00	12.905	12.888	12.767	12.777	12.751
A3-B3	30+00	10.665	10.641	10.505	10.538	10.517
A4-B4	40+00	11.629	11.591	11.550	11.504	11.490
A5-B5	50+00	15.966	15.878	15.816	15.827	15.755
A6-B6	60+00	19.004	18.925	18.756	18.807	18.814
A7-B7	70+00	17.028	16.972	16.858	16.868	16.873
A8-B8	80+00	13.606	13.523	13.386	13.389	13.420
A9-B9	90+00	12.033	11.954	11.809	11.844	11.833
A10-B10	99+80	9.458	9.426	9.298	9.292	9.360
A11-B11	110+05	7.244	7.228	7.163	7.156	7.143
A12-B12	120+00	6.728	6.725	6.668	6.628	6.665
A13-B13	130+00	7.289	7.302	7.036	7.089	7.088
A14-B14	140+00	7.783	7.805	7.702	7.729	7.755

(Fig. 5-1 Plot Data)

Monument Pair	Station	Elevation Difference: Crest Mon. to Toe Mon. Adj 83	Adj 86	2001	2002	2003
A15-B15	150+00	7.901	7.898	7.790	7.764	7.817
A16-B16	160+00	7.468	7.477	7.424	7.390	7.367
A17-B17	170+00	6.622	6.669	6.605	6.633	6.581
A18-B18	180+00	6.733	6.736	6.593	6.697	6.623
A19-B19	190+00	3.756	4.082	3.684	3.689	3.666
A20-B20	200+00	6.322	6.255	6.182	6.125	6.143
A21-B21	210+00	3.303	3.291	3.211	3.232	3.222
A22-B22	220+00	4.760	4.726	4.676	4.607	4.658
A23-B23	230+00	7.631	7.571	7.505	7.389	7.509
A24-B24	240+00	7.943	7.951	7.840	7.843	7.837
A25-B25	250+00	8.516	8.519	8.962	8.870	8.864
A26-B26	260+00	10.476	10.470	10.472	10.395	10.413
A27-B27	270+00	4.761	4.749	4.712	4.661	4.753

(Fig. 5-1 Plot Data)

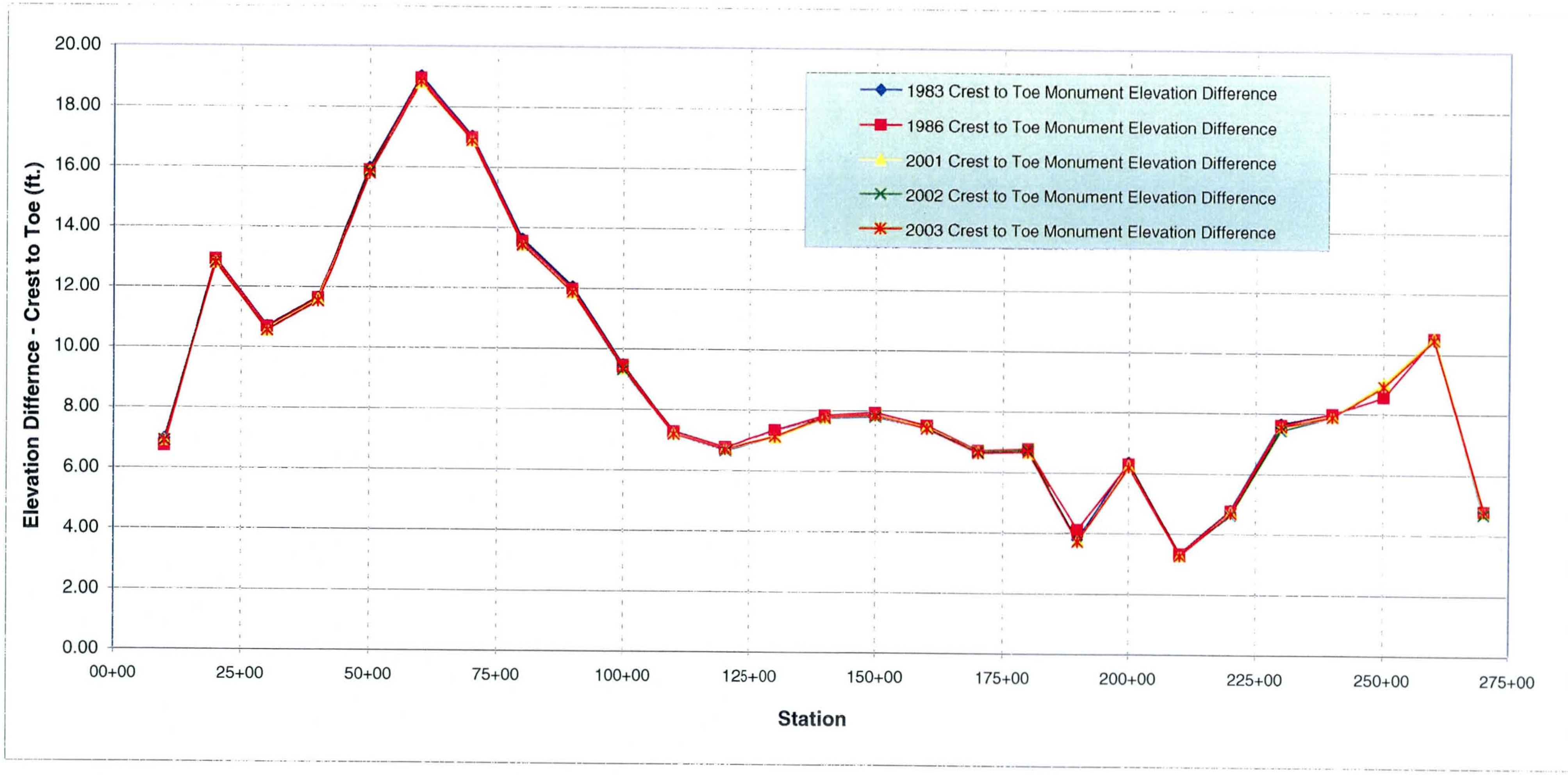
<u>Table 5</u>

Elevation Difference Between Crest Monument and Corresponding Toe Monument

SADDLEBACK FRS

Subsidence Survey Data Review

Settlement Monuments - Crest to Toe



Settlement Monuments - Crest to Toe

Table 6 summarizes the calculation of the relative change in differential elevation between the crest settlement monuments and their corresponding toe settlement monument. The results of this calculation can be used to determine if the associated crest/toe settlement monument pairs are moving closer together, and if so, by how much.
Figure 6-1 plots the relative change in differential elevation with the 1983 survey data used as a baseline.

Monument Pair	Station	Adj 86 - Adj 83	2001 - Adj 83	2002 - Adj 83	2003 - Adj 83
A1-B1	10+00	-0.240	-0.068	-0.056	-0.092
A2-B2	20+00	-0.017	-0.138	-0.128	-0.154
A3-B3	30+00	-0.024	-0.160	-0.127	-0.148
A4-B4	40+00	-0.038	-0.079	-0.125	-0.139
A5-B5	50+00	-0.088	-0.150	-0.139	-0.211
A6-B6	60+00	-0.079	-0.248	-0.197	-0.190
A7-B7	70+00	-0.056	-0.170	-0.160	-0.155
A8-B8	80+00	-0.083	-0.220	-0.217	-0.186
A9-B9	90+00	-0.079	-0.224	-0.189	-0.200
A10-B10	99+80	-0.032	-0.160	-0.166	-0.098
A11-B11	110+05	-0.016	-0.081	-0.088	-0.101
A12-B12	120+00	-0.003	-0.060	-0.100	-0.063
A13-B13	130+00	0.013	-0.253	-0.200	-0.201
A14-B14	140+00	0.022	-0.081	-0.054	-0.028

(Fig. 6-1 plot data)

Monument Pair	Station	Adj 86 - Adj 83	2001 - Adj 83	2002 - Adj 83	2003 - Adj 83
A15-B15	150+00	-0.003	-0.111	-0.137	-0.084
A16-B16	160+00	0.009	-0.044	-0.078	-0.101
A17-B17	170+00	0.047	-0.017	0.011	-0.041
A18-B18	180+00	0.003	-0.140	-0.036	-0.110
A19-B19	190+00	0.326	-0.072	-0.067	-0.090
A20-B20	200+00	-0.067	-0.140	-0.197	-0.179
A21-B21	210+00	-0.012	-0.092	-0.071	-0.081
A22-B22	220+00	-0.034	-0.084	-0.153	-0.102
A23-B23	230+00	-0.060	-0.126	-0.242	-0.122
A24-B24	240+00	0.008	-0.103	-0.100	-0.106
A25-B25	250+00	0.003	0.446	0.354	0.348
A26-B26	260+00	-0.006	-0.004	-0.081	-0.063
A27-B27	270+00	-0.012	-0.049	-0.100	-0.008

(Fig. 6-1 plot data)

Table 6
Relative Change in Differential Elevation Between Crest Monument and Corresponding Toe Monument, 1983 Survey Data as Baseline

Settlement Monuments - Crest to Toe

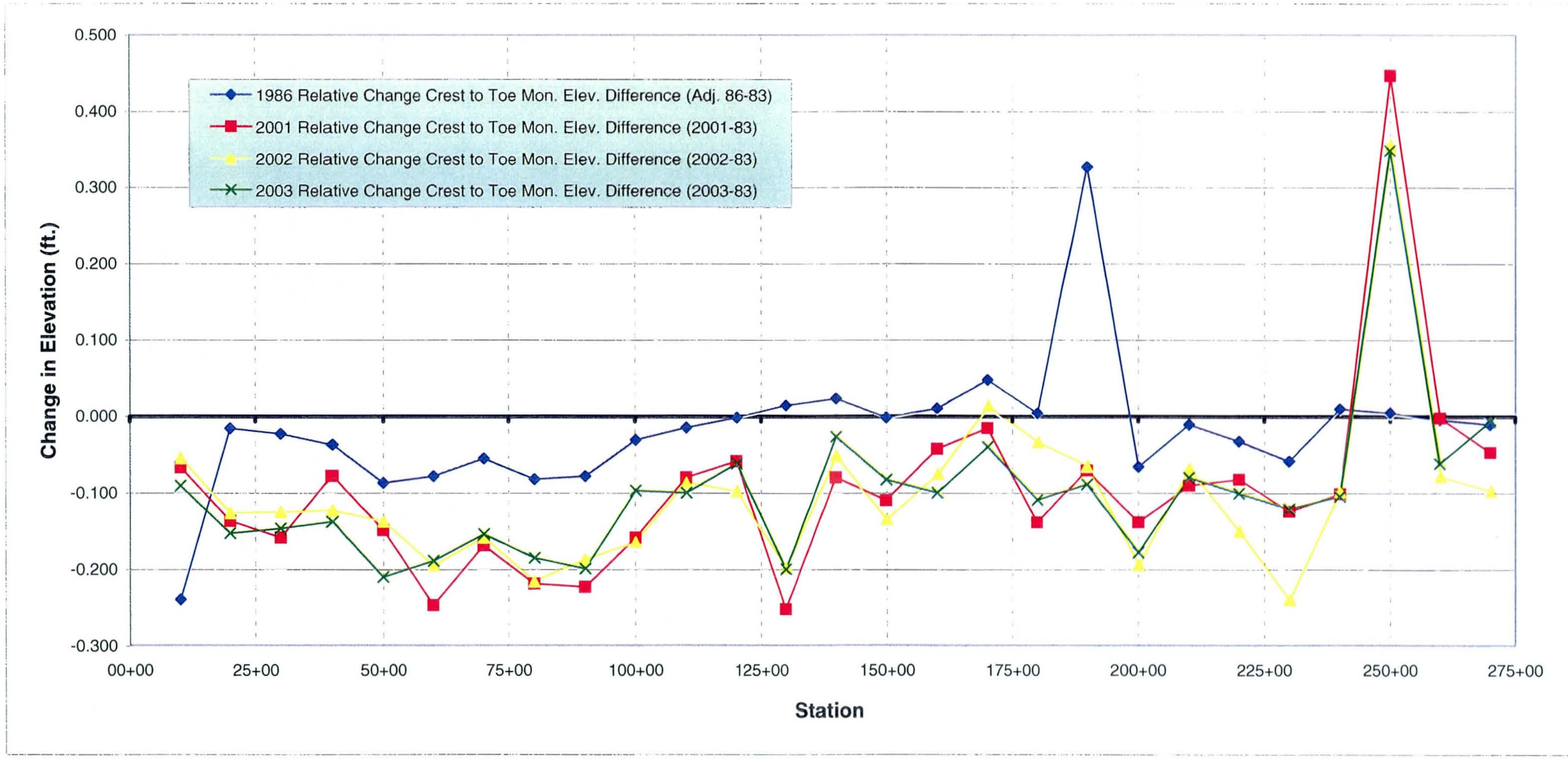


Figure 6-1
Relative Change in Elevation Difference Between Associated Crest and Toe Settlement Monument Pairs

Spillway & Miscellaneous Elevations

Table 7 below displays various Outlets, Principal Spillway, and Emergency Spillway elevations for 2002 and 2003, and calculates the elevation change from the As-Built Design Drawings as the baseline.

Marker	Station	2001	2002	2003	Description
10000	256+00	1190.809	1191.006	1190.914	CCX * 256+00 UPSTREAM CHISELED X
10001	256+00	1185.922	1186.153	1186.065	CCX * 256+00 OUTLET - CHISELED X
10002	124+00	1191.648	1191.882	1191.794	CCX * 124+00 INLET - CHISELED X
10003	124+00	1188.979	1189.083	1189.117	CCX * 124+00 OUTLET - CHISELED X
10004	103+70	1189.641	1189.737	1189.787	CCX * 103+70 INLET - CHISELED X
10005	103+70	1186.828	1186.975	1186.947	CCX * 103+70 OUTLET - CHISELED X
10006	60+50	1182.959	1183.056	1183.111	CCX * 60+50 INLET - CHISELED X
10007	60+50	1175.872	1175.973	1176.014	CCX * 60+50 OUTLET - CHISELED X
10008	15+83	1187.338	1187.497	1187.523	CCX * MAIN SPILLWAY OUTLET N. C-8.73INV15+
10009	15+83	1187.398	1187.549	1187.529	CCX * MAIN SPILLWAY OUTLET S. C-8.83INV15+
10010	15+83	1187.584	1187.708	1187.768	CCX * MAIN SPILLWAY INLET N. - CHISELED X
10011	15+83	1187.550	1187.754	1187.724	CCX * MAIN SPILLWAY INLET S. C-9.83INV15+8
10012	50+00	1183.470	1183.596	1183.607	CCF * 3"BC FCDMC NO ID
10013	55+00	1181.461	1181.569	1181.593	CCF * 3"BC FCDMC NO ID
10014	50+00	1174.257	1174.465	1174.492	CCF * 3"BC FCDMC NO ID
10015	55+00	1172.040	1172.184	1172.196	CCF * 3"BC FCDMC NO ID

<u>**Table 7**</u>
Miscellaneous Points

Settlement Summary

Table 8 below summarize the settlement that has occurred at Saddleback FRS from 1983 to 2003.

Crest Marker	Station	2003 - Adj 83
A1	10+00	0.070
A2	20+00	0.057
A3	30+00	0.022
A4	40+00	0.005
A5	50+00	-0.013
A6	60+00	-0.098
A7	70+00	-0.105
A8	80+00	-0.035
A9	90+00	-0.048
A10	99+80	-0.014
A11	110+05	0.048
A12	120+00	0.036
A13	130+00	0.057
A14	140+00	0.026

Crest Marker	Station	2003 - Adj 83
A15	150+00	0.017
A16	160+00	0.035
A17	170+00	0.012
A18	180+00	0.011
A19	190+00	0.071
A20	200+00	0.033
A21	210+00	-0.001
A22	220+00	-0.010
A23	230+00	-0.038
A24	240+00	0.046
A25	250+00	0.048
A26	260+00	0.082
A27	270+00	0.142

Toe Marker	Station	2003 - Adj 83
B1	10+00	0.162
B2	20+00	0.211
B3	30+00	0.170
B4	40+00	0.144
B5	50+00	0.198
B6	60+00	0.092
B7	70+00	0.050
B8	80+00	0.151
B9	90+00	0.152
B10	99+80	0.084
B11	110+05	0.149
B12	120+00	0.099
B13	130+00	0.258
B14	140+00	0.054

Toe Marker	Station	2003 - Adj 83
B15	150+00	0.101
B16	160+00	0.136
B17	170+00	0.053
B18	180+00	0.121
B19	190+00	0.161
B20	200+00	0.212
B21	210+00	0.080
B22	220+00	0.092
B23	230+00	0.084
B24	240+00	0.152
B25	250+00	-0.300
B26	260+00	0.145
B27	270+00	0.150

Table 8
Settlement Summary of Crest and Toe Monuments

Based on the Datum Shift, the elevations at the Benchmarks equal the NGVD 1929 elevations plus the datum shifts shown in **Table 9.** The highlighted elevation values in the "Adjusted" columns of the tables reflect this calculation.

Marker	Description	1983 (NGVD 29)	1986 (NGVD 29)	2001 (NAVD 88)	2002 (NAVD 88)	2003 (NAVD 88)	2001-1983	2002-1983	2003-1983	2001-1986	2002-1986	2003-1986
R-1	CCF * 2.5"BC FCDMC R-1	1251.881	1251.881	1253.720	1253.853	1253.885	1.839	1.972	2.004	1.839	1.972	2.004
R-2	CCF * 3"BC FCDMC R-2	1199.987	1200.060	1201.995	1202.072	1202.072	2.008	2.085	2.085	1.935	2.012	2.012
R-476	CCF * 3"BC NGSVCM R476 1981			1201.995	1236.336	1236.345						
Z-262	CCF * 3"BC USCGSBM Z262 1947			1215.614	1215.710	1215.692						
C-13	CCF * 3"BC USCGSBM C 13 1927			1191.714	1190.629	1190.607						
4DP1	GDACS CONTROL			1210.963	1210.963	1210.963						
EBM1	CCF * 3"BC FCDMC SADDLE EBM1 2002 BEDROCK				1279.944	1279.921						
EBM2	CCF * 3"BC FCDMC SADDLE EBM2 2002 BEDROCK				1233.102	1233.107						
							Average 1983 shift =	1.999		Average 1986 shift =	1.962	

Notes: Average datum shift 2001, 2002, and 2003 to 1983 = 1.999' (Value used to adjust 1983 elevations to 1988 Datum)
Average datum shift 2001, 2002, and 2003 to 1986 = 1.962' (Value used to adjust 1986 elevations to 1988 Datum)

Table 9
Summary of Reference Marks

Floodplain View

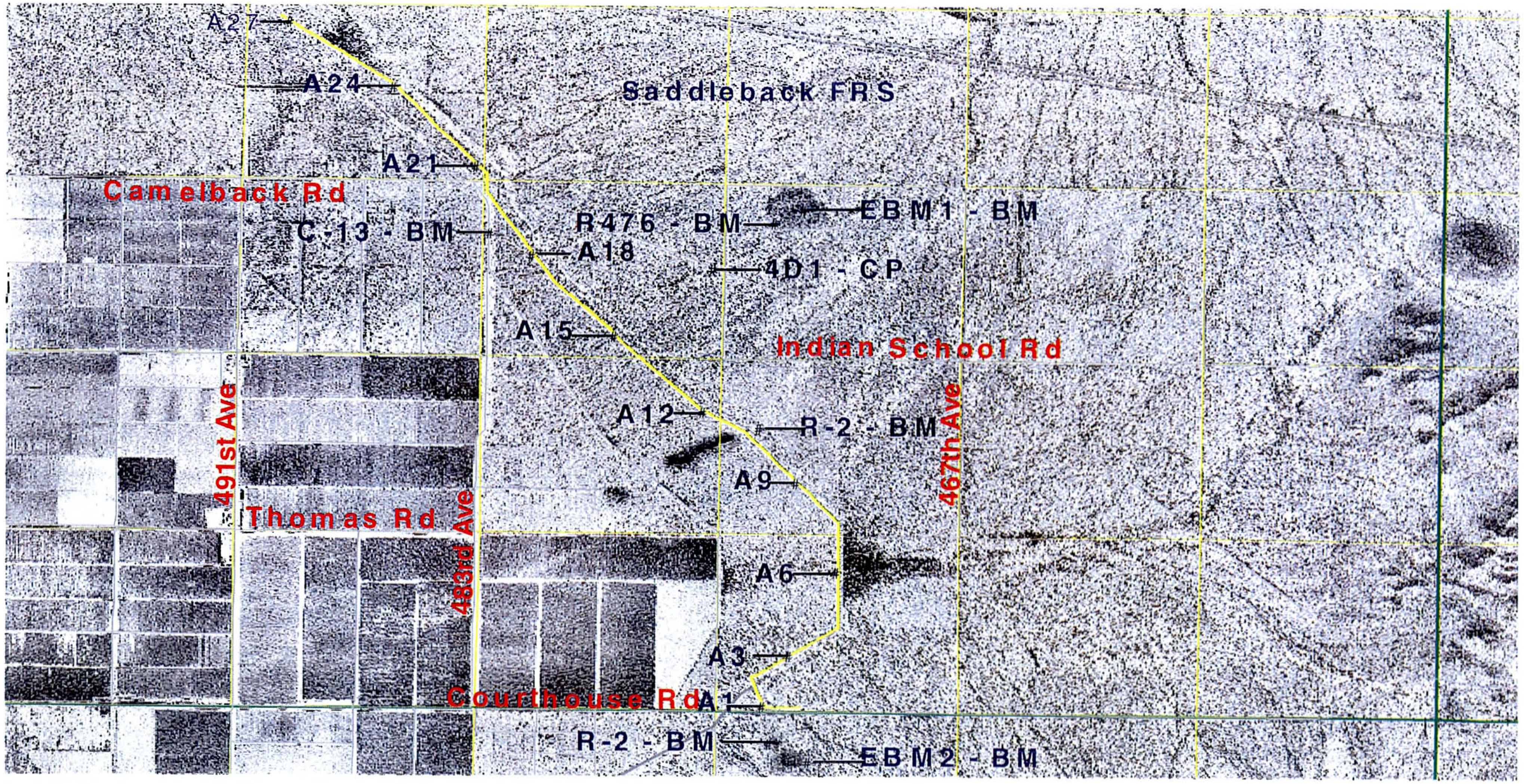


Figure 8 - Saddleback FRS Dam

PRELIMINARY FAILURE MODES IDENTIFICATION REPORT SADDLEBACK FLOOD RETARDING STRUCTURE FLOOD CONTROL DISTRICT OF MARICOPA COUNTY, ARIZONA JANUARY 20, 2005

1.0 Introduction

Kimley-Horn and Associates, Inc. (KHA) prepared this report to document discussions related to the Preliminary Failure Modes Identification workshop for Saddleback FRS conducted on January 20, 2005. The purpose of the workshop was to:

- Develop a list of potential failure modes for the structure and appurtenances,
- Identify key issues that require additional review or assessment during the structure assessment or field inspections,
- Discuss/identify field evidence for precursors for potential failure modes, and,
- Provide a baseline for detailed Failure Mode and Effects Analysis.

The workshop was conducted at the offices of Kimley-Horn and Associates, Inc. The following individuals participated in the workshop:

Tom Renckly, P.E.	Flood Control District
Brett Howey, P.E.	Flood Control District
John Chua, P.E.	Natural Resources Conservation Service
Bob Eichinger, P.E., CFM	Kimley-Horn and Associates, Inc.
Dean Durkee, Ph.D, P.E.	Gannett Fleming, Inc.
Ken Euge, R.G.	Geological Consultants, Inc.

2.0 Facility Descriptions

Saddleback FRS is an earthfill dam that has a crest length of 24,270 ft, a crest width of 12 ft, upstream slope of 3:1 and downstream slope of 2:1 and has a maximum height of 21.0 ft. There is no emergency spillway. The principal outlet consists of a single barrel reinforced concrete box culvert (10-ft span by 8-ft rise) constructed through the dam near the left abutment. The structure and impoundment was designed not to have a permanent storage pool. A central filter with 6-inch drain outlets were constructed with the dam embankment.

3.0 Summary of Inspection Reports

Flood Control District inspection reports dating from 1998 to 2004 were collected and reviewed. The January 2003 through November 2004 inspection reports document the surface expression of potential longitudinal and transverse cracks on the centerline crest and the downstream and upstream slopes of the dam. Noteable and sizeable erosion holes are found along the centerline of the crest which appears to be associated with the central filter (supposition that this is related to settlement or collapse of central filter?).

A record of impoundment prepared by the District includes both dates and depths of impoundments for the period from 1989 to 2004. The maximum gage depth of impoundment was 2.5-ft in 1996. The crest is gravel plated. The inspection reports document erosion rills and gullies of various sizes along both the upstream and downstream slopes.

4.0 Preliminary Failure Modes

The potential failure modes have been categorized into the following categories for the purposes of the workshop: hydrologic/hydraulic (flood related), geotechnical/geological (static), geological, structural, and other considerations.

A. Hydrologic/Hydraulic Potential Failure Modes

1. **Embankment Overtopping**: The embankment crest is gravel plated and is therefore provided with a measure of erosion protection. The upstream and downstream slopes of the dam are not provided with gravel-mulch erosion protection. Overtopping of the dam crest embankment could lead to erosion and formation of a breach. In assessing the probability of occurrence of this failure mode, the following items should be reviewed:

    a. Review and document the freeboard available when routing the Inflow Design Flood (IDF) through the emergency spillway. The IDF for the dam is currently the ½ PMF. Check full PMF.
    b. Qualitatively assess the impact of regional subsidence on the dam crest elevation. Locate the most recent crest survey data.
    c. Review and document the initial reservoir conditions for each of the spillway routings.
    d. Perform a preliminary assessment to evaluate if dynamic routing of the inflow hydrograph would impact the freeboard. Apply conservative assumptions as needed. Compare "dynamic routing" approach versus "kinematic routing" or "modified-Puls" approach.
    e. Review and document the most current estimate of reservoir stage capacity.
    f. Review the available estimates of the Probable Maximum Precipitation (PMP). Identify the differences between each of the estimates. In particular, what factors causes a duration (6-hour or 72-hour) to become more critical?
    g. Check erosion protection on slopes. Compare areas that are protected with gravel mulch with those areas that are not protected. Should gravel mulch be placed on embankment slopes that are experiencing and showing transverse cracking?
    h. Are high capacity groundwater wells having localized effect?
    i. Need to check routing of IDF. (duration, depth of flow, time of impoundment)

   j. Check inundation areas and limits downstream of dam.
   k. Loss of freeboard – embankment settlement, foundation collapse, regional subsidence. Review settlement surveys.
   l. Salome Road constructed over and cut through embankment. Restore embankment height and crest at this location.

2. **Failure of Principal Outlet**: The principal outlet for the dam is a reinforced concrete box culvert (10-ft span by 8-ft rise). The following items require review:
   a. Review available information to assess the structural adequacy of the principal outlet.
   b. Qualitatively assess the potential for piping around the principal outlet.
   c. Inspect approach to the principal outlet to assess and document grouted rip-rap condition.
   d. Review available geotechnical information to assess is the principal outlet is underlain by collapsible soils.
   e. Seepage collars around principal spillway.
   f. Does drain fill wrap fully around outlet?
   g. Is seepage out of joint possible?
   h. Conduct video survey of vegetative outlets and drain outlets.
   i. Clogging of principal outlet with vegetation (tree limbs, logs) during runoff event. The outlet does not have a trash rack.

B. Geotechnical/Geological Failure Modes.

1. **Piping Involving Foundation and Abutments**: Relates to potential piping erosion of soil materials from the embankment fill into the foundation and/or developing through the foundation under the embankment. The following items need to be reviewed to assess this failure mechanism.
   a. **Geotechnical/Geometric Profile**. Review the geotechnical profile along the embankment and the construction details of the cutoff trench(s), if any.
      i. Look for sharp transitions in foundation material types, foundation stripping/excavation (e.g. to remove zones of soft or collapsible materials), dramatic changes in bedrock depth, etc. – conditions that could lead to differential settlement and transverse cracking
   b. **Buried Gravel Channels**. Review the surficial geology/soil at the site to assess whether permeable gravel channels are present.
      i. Consider potential pathways for preferential seepage and erosion under the dam embankment.
      ii. Check filter compatibility between embankment fill and foundation soils (potential for downward piping into any openwork gravels/alluvial deposits?)
   c. **Cutoff Trenches**. Review the design and construction details of cutoff trenches to assess the potential for a defects/design flaws in the cutoff that could lead to seepage and erosion.
      i. Cutoff trenches of limited width (top of core trench not as wide as base of core zone) - potential for differential settlements that result

in cracking of core material or cracking at interface between core zone and adjacent shell zones

ii. Cutoff trenches of limited depth/or no core trench - potential for concentrated seepage along base of dam/core trench
iii. Cutoff trench (near dam centerline) only extends for limited width (differential movement that results in cracking of core material or cracking at interface between various zones of material)

d. **Erosivity of Foundation Soils**. For dams with or without core trenches – consider erosivity of foundation soils and potential for concentrated exit gradients at unprotected toe(s) of dam(s) (under seepage during impoundment events).
e. **Potential for earth fissures** extending under dam?
f. **Downstream runoff erosion**. Review and assess if discharge from natural drainages adversely impacts the downstream face or toe of the embankment.

2. **Internal Erosion along Transverse Crack**: This failure mode relates to internal erosion along a transverse crack, or along a penetration through the dam (outlet pipes, drain fill pipes, or vegetative outlets). The following are critical items that will be reviewed and assessed prior to the FMEA:
    a. **Transverse Cracking**. Information related to identifying potential for transverse crack formation through embankment fill. Transverse cracking has been reported at Saddleback and case histories on other District dams warrant the evaluation of potential failure modes related to embankment internal erosion.
        i. Potential for desiccation shrinkage cracking of clayey fill materials (review soil PI's and fines content, depth of non-clayey cover protecting clayey materials, etc).
        ii. Potential for differential settlement-induced cracking (transitions at cutoff trenches, collapsible soils in foundation, variability of foundation in longitudinal direction, etc.)
        iii. Discuss inability to view/inspect for transverse cracking due to rock mulch slope protection.
    b. **Internal Filters**. Review and assess to the extent practical the level of protection against concentrated leak piping provided by internal filters. This review should also evaluate the potential for a defect through the central filter.
    c. Check for gradation data on filter/drain and core material zones.
    d. Check and see if it wraps fully around piping along outlet conduit.
    e. Review internal stability of central chimney/filter drain materials
    f. **Penetrations through Dam**. Review drawings and information to evaluate vulnerability to piping along penetrations through dam (outlet conduits/utilities).
        i. Consider outlet pipe construction methods (seepage collars, cradles, pipe bedding, etc). For example, if seepage collars were installed around principal spillway, we know that poor compaction

around seepage collars has lead to piping erosion in numerous case histories.

ii. Were filter diaphragms installed, or does internal zoning around pipe meet requirements for filter diaphragms?

iii. Review utility plans

g. **Internal zoning geometry**. Review construction details for internal zoning. Look for core/shell zones that do not extend to dam crest – if only extend to emergency spillway crest elevation – possibility of seepage "overtopping" core zone leading to erosion/loss of dam crest.

h. Review the characteristics of case history of FCD embankment cracking (width, spacing, depth).

i. **Partially penetrating central filters**. Review the central filter configuration in light of maximum crack depths to evaluate the potential for piping under a partially-penetrating center filter.

j. Evaluate if **animal burrows** can serve as seepage conduits across the entire width of the embankment.

3. **Slope Stability**: This failure mode covers both the upstream and downstream slopes of the embankment. The following items require review prior to the FMEA:
   a. General static and seismic stability of the upstream and downstream slopes of the dam.
   b. Rapid drawdown instability.
   c. Review the configuration of the central filter and assess to the extent practical, if a full head of water within the central filter could destabilize the downstream face of the dam.
   d. Erosional stability of dam crest under wave action.

4. **Failure Mechanisms Associated with Presence of Collapsible Soils in Dam Foundation:** This failure mode relates to the potential for collapse on saturation of meta-stable soils in the dam foundation. Geologic mapping/boring logs/laboratory test data will be reviewed to assess to the extent practical the presence of potentially collapsible materials. If these soils are suspected to be present the following needs to be considered:
   a. Potential for loss of freeboard/overtopping in zones of limited width where collapsible soils are present
   b. Differential settlement leading to formation of transverse cracks in embankment fill/foundation.
   c. Slope instability caused by loss of support/oversteepening of either upstream or downstream slopes.

5. **Failure Mechanisms Associated with Earth Fissures**: Previous as well as current investigations by others have identified a strong potential for earth fissures at a number of FCD structures. The following issues need to be reviewed as part of the FMEA:

   a. Review current investigations to evaluate the potential for earth fissures in the vicinity of the dam.
   b. Review the geotechnical properties of the soils to assess the potential for "pipe" or "tunnel" formation through the embankment/foundation along an earth fissure.
   c. Cracking of the embankment due to one or more earth fissures. This could result in some of the failure mechanisms related to seepage and erosion piping through the embankment.
   d. Review geotechnical data to assess the stability of the upstream slope under rapid drawdown conditions. The failure mechanism is similar to that discussed above, with the exception that seepage along a fissure through the foundation could result in loss of support due to erosion of the (as opposed to collapsible) soils.

6. **Failure Mechanisms Associated with Filter/drain pipe**. The filter drain incorporates a drain pipe to collect seepage water. The may be a potential for failure of the drain pipe system by either clogging or structural failure by collapse. The following issues need to be reviewed as part of the FMEA:
   a. Review design and construction records for drain pipe and drain pipe openings versus filter material size.
   b. Review pipe strength specifications versus loading.
   c. Need to verify what type of soil surrounds the chimney.

7. **Internal Erosion along Longitudinal Crack**: This failure mode relates to internal erosion along a longitudinal crack expressed through the crest of the dam into the central filter. The following are critical items that will be reviewed and assessed prior to the FMEA:
   a. **Longitudinal Cracking**. Information related to identifying potential for internal erosion with longitudinal crack formation along centerline crest. Longitudinal cracking has been reported at Saddleback and case histories on other District dams warrant the evaluation of potential failure modes related to embankment internal erosion.
      i. Potential for desiccation shrinkage cracking of fill materials (review soil PI's and fines content, depth of non-clayey cover protecting clayey materials, etc).
      ii. Potential for differential settlement-induced cracking (transitions at cutoff trenches, collapsible soils in foundation, variability of foundation in longitudinal direction, etc.)
      iii. Discuss inability to view/inspect for cracking due to rock mulch slope protection.
   b. **Internal Filters**. Review and assess to the extent practical the level of protection against concentrated leak piping provided by internal filters. This review should also evaluate the potential for a defect through the central filter.
   c. Check for gradation data on filter/drain and core material zones.
   d. Check and see if it wraps fully around piping along outlet conduit.

e. Review internal stability of central /filter drain materials

C. Geological Failure Modes

1. **Failure Mechanisms Associated with Seismic Event.**
   a. What is potential for liquefaction?
   b. Seismic event potential for exacerbating existing transverse/longitudinal cracks.
   c. Causative or additive mechanism for central filter collapse.

D. Other Considerations:

This section addresses issues that are not directly related to a failure of the dam or its appurtenant facilities, but which nonetheless may be relevant to the FMEA:

a. Foundation treatment
b. Compaction
c. Use of construction materials (borrow areas)
d. Placement of embankment lifts
e. Filter gradation and outlet drain gradation

5.0 Closure

The aim of the workshop on January 20, 2005 was to identify and develop a list of failure modes for Saddleback FRS. In addition, the participants also identified key issues that require additional review or assessment during the Individual Structures Assessment and the Field Inspections. A detailed Failure Modes and Effects Analysis (FMEA) was beyond the scope of the workshop. The FMEA for the dam is scheduled as a future task of this work assignment (February 28 through March 4, 2005). The list of items to be reviewed as presented is intended to provide guidance to the risk assessment team, and does not represent a comprehensive list of documents and information items that need to be reviewed in advance of the formal FMEA.

ON-CALL PHASE I ASSESSMENT

SADDLEBACK FLOOD RETARDING STRUCTURE

FIELD INSPECTION REPORT

Purpose

The purpose of the field examination is to provide a systematic visual field technical review in which the structural stability and operational adequacy of the dam project features are reviewed and evaluated to determine if deficiencies exist at the dam and associated project features. The examination was conducted by walking the length of the structure and visually examining the crest, upstream and downstream slopes, upstream and downstream toes, and appurtenant structures. Comments are recorded in an inspection log and photographs taken of pertinent observations. Cracks, holes, and burrows were probed with hand-held 3-foot stainless steel metal rod/probes to examine depth, extent, and resistance to probing. No other intrusive/internal examination method was used during this examination.

The field examination of the structure is accomplished to provide a basis for timely initiation of any corrective measures to be taken where necessary. This examination was conducted on February 2, 2005 by the following technical examination team:

Technical Examination Team

Tom Renckly, P.E.	Structures Branch Manager, Flood Control District of Maricopa County
Brett Howey, P.E.	Dam Safety Engineer, Flood Control District of Maricopa County
Earl Pearcy	Operation and Management, Flood Control District of Maricopa County
John Harrington, P.E.	Engineer, Natural Resources Conservation Service
Robert Eichinger, P.E., CFM	Project Manager, Kimley-Horn and Associates
Ken Euge, R.G.	Principal Geologist, Geological Consultants
Dean Durkee, Ph.D., P.E	Principal Geotechnical Engineer, Gannett-Fleming
David Jensen, P.E.	Engineer, Kimley-Horn and Associates
Kelli Blanchard, E.I.T.	Hydrologist, Kimley-Horn and Associates

Operational Summary

Inspection Frequency: Saddleback Flood Retarding Structure (FRS) is inspected jointly on an annual basis by the Arizona Department of Water Resources (ADWR) and the Flood Control District of Maricopa County (District). The NRCS is invited to participate in annual inspections of Saddleback FRS.

Maximum Water Surface Elevations: The maximum recorded impoundment for Saddleback FRS was on July 7, 1996. The impoundment was recorded at 2.5-feet.

Emergency Spillway Discharge: There is no separate emergency spillway.

Distress Observations Corrected or Operation and Maintenance Conducted Since Last Inspection: None were noted. The District has an operation and maintenance program in place in which they continually monitor for rodent activity and vegetation on the dam.

Past Distress Observations Not Yet Corrected: (Maintenance and corrective measures identified in the November 2004 Inspection Report were placed on hold pending completion of the Phase I Individual Structures Assessment)*

- Update emergency action plan (scheduled for fall 2005);
- Fill erosion rills on upstream and downstream slopes with compacted fill, if greater than 12-inches deep;
- Initiate gravel mulch recommendations resulting from the Phase I Structures Assessment;
- Control and repair damage caused by rodent activity;
- Repair longitudinal crack near the upstream toe between Sta. 250+91 to 262+20, 72+14 to 76+34, and Sta. 50+72 to 54+00 utilizing the same procedures utilized during the repair of similar cracking between Station 53+00 and 56+00;
- Restore crest at Salome Road (coordinate with MCDOT);
- Replace toe settlement monument B-25 at Station 250+00

* These measures were taken from the November 2004 Inspection Report.

District Operation and Maintenance Responsibilities: The District maintains operational control of the Saddleback FRS and is responsible for the structural and functional integrity of the FRS and appurtenant features, maintaining the emergency spillway, erosion control of the embankments, and landscaping. The District is responsible for implementation of the emergency action plan. Operation and Maintenance inspections of the dam are conducted on a quarterly basis.

Field Examination Results Summary

Embankment Crest: The crest of the dam is gravel plated. All crest settlement monuments located on the crest were located. There are station markers on the dam. The crest is clear of vegetation. The access gates and fences are operational. Longitudinal cracks/transverse cracks, depressions, and erosion holes were observed on the crest of the dam (see inspection report for specific locations).

Abutments: The left and right abutment contacts appear in satisfactory operational condition. No slides, sign of instability or erosion of the abutment surfaces were observed. Abutment groins were clear of vegetation.

Upstream Slope: There are several small animal burrows scattered on the slope face. There was no evidence of seepage, undermining, settlement or sloughing.

Recommendations for gravel mulching of the upstream slope will be provided during the Phase I Structures Assessment currently being completed by Kimley-Horn.

Downstream Slope: There are several small animal burrows scattered on the slope face. There was no evidence of seepage, undermining, settlement or sloughing. Recommendations for gravel mulching of the upstream slope will be provided during the Phase I Structures Assessment currently being completed by Kimley-Horn.

Toe drain outlets are located approximately every 400 feet along the length of the dam. The toe drains extend from the central filter to the downstream toe of the embankment. Two of these toe drains, at Station 248+00 and 204+00, could not be located during the inspection. Many of the toe drain outlet conduits are partially full of sediment at the very downstream end of the conduit. It is not known if the material in the conduits is from the central filter itself or if it is from ponding at the outlets.

Principal Spillway and Reservoir: The approach channel is clear of debris and obstructions. The wing walls of the inlet structure have minor shrinkage and temperature cracks, no repairs required. The interior of the principal spillway conduit was inspected visually. The conduit was clean and there were no apparent signs of seepage.

The discharge outlet structure of the principal spillway was clear of debris. The joints of the outlet structure were straight and appeared tight. There were no signs of seepage.

Vegetation clearing operations were underway for the upstream floodway.

The principal outlet for Saddleback FRS is a combination principal/emergency spillway.

Irrigation Outlet: The irrigation outlets were located and observed to be clear of debris at the inlet and outlets.

Instrumentation: The settlement monuments for Saddleback FRS are located on the downstream side of the dam crest at grade and near the downstream toe. Settlement monuments are marked with sign posts. Settlement monuments located on the crest are noted with an A, settlement monuments located near the toe are noted with a B. Monument B25 at Station 250+00 has been damaged and should be replaced. There are station markers on the Saddleback FRS.

There are no rain or stream gages in the watershed. There is an alert gage at the principal outlet. These instruments help provide an early warning system and should be incorporated into an emergency action plan for the Saddleback FRS.

There is a staff gage at the principal outlet located on the upstream slope. The staff gage is used to indicate the level of water impounded in the reservoir. A pressure transducer is also located at the principal outlet. No staff gage is located any where else on the dam embankment.

Conclusions

The overall conclusion of the field examination is that the Saddleback FRS and appurtenant structures are in satisfactory operational condition.

Recommendations from Inspection

The following is a list of recommended corrective actions resulting from this field examination:

a. Add watershed instrumentation (stream and rain gages);
b. Develop an Emergency Action Plan to meet FEMA 64 and ADWR requirements;
c. Develop a repair plan for erosion holes on the crest of the dam;
d. Evaluate gravel mulching of embankment slopes;
e. Evaluate effect of Solome Road over crest of dam;
f. Locate the toe drains at Station 204+00 and 248+00;
g. Determine source of material deposited in the toe drain outlets.
h. Due to the length of the structure (5 miles), an additional staff gage should be added. A recommended location would be at the Solome Road crossing of the embankment.

Next Annual Inspection

The next annual inspection is scheduled for January 2006.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY - DAM SAFETY
EMBANKMENT DAM INSPECTION CHECKLIST / REPORT

Each item of the checklist should be completed. Repair is required when obvious problems are observed. Monitoring is recommended if there is a potential for a problem to occur in the future. Investigation is necessary if the reason for the observed problem is not obvious.

A brief description should be made of any noted irregularities, needed maintenance, or problems. Abbreviations and short descriptions are recommended. Additional sheet(s) may be used for any items not listed and additional comments.

ADWR NO.: **07.52** FCDMC NO.: **331**	DAM NAME: **Saddleback FRS**	TYPE: **Earthfill**
CONTACTS: **Brett Howey, Flood Control District (506-1501)** **ADWR (Invited – declined attendance)** **NRCS – John Harrington**	REPORT DATE: **February 18, 2005 and June 8, 2005**	
INSPECTED BY: **Bob Eichinger (Kimley-Horn), David Jensen (Kimley-Horn), Ken Euge (Geological Consultants), Dean Durkee (Gannett-Fleming), Tom Renckly (FCD), Brett Howey (FCD), Earl Pearcy (FCD O&M), Kelli Blanchard (Kimley-Horn), John Harrington (NRCS)**	DATE: **February 2, 2005**	
REVIEWED BY: **Bob Eichinger, P.E., CFM**	DATE: February 18, 2005	PAGE **1of 11**
SPILLWAY DESIGN CREST ELEVATION: **1178.89 ft. (1988 NAVD) (no emergency spillway)**	HAZARD CLASS: **Significant**	SIZE: **Intermediate**
INFLOW DESIGN FLOOD: **½ PMF**	SPILLWAY CREST WIDTH: **10 ft.**	ADWR DAM HEIGHT: **21 ft.**
DAM CREST LENGTH: **24,270 ft.**	DAM CREST WIDTH: **12 ft.** Note: Crest elevation data from November 2004 Inspection Report	CREST ELEV.: **Varies (datum NGVD29)** **Sta. 3+00 to 49+00: 1193.0** **Sta. 49+05 to 72+95: 1193.33** **Sta. 73+00 to 171+37: 1193.0** **Sta. 171+62 to 272+70: 1194.0**
CURRENT RESERVOIR LEVEL: **Empty**	TOTAL DESIGN FREEBOARD: **16.1 ft.**	PHOTOS: **Yes**

Item / Comments	NOT APPLIC.	NO	YES	MONITOR	REPAIR	INVESTIGATE
1. CREST A 5 ft. wide central filter/drain installed during dam construction that extends to the bottom of the foundation cutoff trench from Station 46+00 to 76+00, 104+00 to 114+00, and 238+20 to 265+70.						
a. Settlements, slides, depressions? • **Numerous indications of longitudinal cracking along the centerline of the dam (see table at the end of the report). Cracking is suspected to be a result of settlement of the central filter/drain. Sinkholes in the crest have become of such size that repairs are warranted.** • **Grading of the Salome Road crossing has significantly reduced (≈ 1.5 ft.) the available freeboard at this location.**			✓	✓	✓	✓
b. Misalignment?		✓				
c. Longitudinal/Transverse cracking? **Longitudinal cracking, in line with the central filter/drain is intermittent from the spillway to the right abutment (see table at the end of the report). Cracking is suspected to be a result of settlement of the central filter/drain (see comment 1.a.).**			✓	✓	✓	✓
d. Animal burrows? **Scattered throughout the full length of the dam along the upstream and downstream shoulders. Continue rodent control following standard procedures.**			✓		✓	
e. Adverse Vegetation?		✓				
f. Erosion?		✓				
2. UPSTREAM SLOPE **3H : 1V with protective berms at the upstream toe from Station 46+00 to 76+00 and 240+00 to 265+00.**						
a. Erosion? **Scattered rilling throughout. Repair rills and or gullies greater than 12-inches deep.**			✓		✓	
b. Inadequate ground cover?		✓				
c. Adverse vegetation?		✓				
d. Longitudinal/Transverse cracking? **See table at the end of the report for a listing of possible transverse and longitudinal cracks Monitor). Longitudinal cracking observed near the upstream toe between Sta. 250+91 to 262+20, 72+14 to 76+34, and Sta. 50+72 to 54+00 (Repair utilizing the same procedures utilized during the repair of similar cracking between Station 53+00 and 56+00).**			✓	✓	✓	
e. Inadequate riprap? **There is no riprap on the upstream slope. See comment 2.a. regarding the installation of gravel mulch.**		✓				
f. Stone deterioration?	✓					
g. Settlements, slides, depressions, bulges?		✓				

Item	Comments	N/A	NO	YES	MON	REP	INV
Animal burrows? **Scattered throughout the full length of the dam. Continue rodent control following standard procedures.**				✓		✓	
3. DOWNSTREAM SLOPE **2H : 1V with drain outlets for the central filter/drain located at 400 ft. intervals starting at Station 8+00. Some toe drain outlets have sediment in the invert of the conduit.**							
a. Erosion? **Scattered rilling throughout. Repair rills and or gullies greater than 12-inches deep (see comment 2.a.)**				✓		✓	
b. Inadequate ground cover?			✓				
c. Adverse vegetation?			✓				
d. Longitudinal/Transverse cracking? **See table at the end of the report for a listing of possible transverse cracks.**				✓	✓		
e. Animal burrows? **Scattered throughout the full length of the dam. Continue rodent control following standard procedures.**				✓		✓	
f. Settlements, slides, depressions, bulges?			✓				
g. Soft spots or boggy areas?			✓				
h. Movement at or beyond toe?			✓				
4. DRAINAGE-SEEPAGE CONTROL **Central filter/drain that extends to the bottom of the foundation cutoff trench from Station 46+00 to 76+00, 104+00 to 114+00, and 238+20 to 265+70 with drain outlets located at 400-foot intervals starting at Station 8+00. The drain outlets consist of 6-inch diameter perforated pipe with drain rock.**							
a. Internal drains flowing? **Reservoir empty**			✓				
b. Boils at or beyond toe?			✓				
c. Seepage at or beyond toe? **Reservoir empty**			✓				
Does seepage contain fines?		✓					
5. ABUTMENT CONTACTS							
a. Erosion?			✓				
b. Differential movement?			✓				
c. Cracks?			✓				
d. Settlements, slides, depressions, bulges?			✓				
e. Seepage?			✓				
f. Animal burrows?			✓				
6. IRRIGATION OUTLET - INLET STRUCTURES **Four irrigation outlets at Station 60+50, 103+70, 124+10, and 256+00. The irrigation outlets located at Station 60+50 and 256+00 are gated while the irrigation outlets at Station 103+70 and 124+10 are ungated.**							
a. Seepage into structure? **Reservoir empty - No indication of seepage**			✓				
b. Debris or obstructions?			✓				
c. If concrete, do surfaces show:							
1. Spalling or Scaling?			✓				
2. Cracking? **Shrinkage and temperature cracks (minor non-structural). No repairs required.**				✓	✓		
3. Erosion?			✓				
Exposed reinforcement?			✓				
d. If metal, do surfaces show:							
1. Corrosion?		✓					

SADDLEBACK FRS INSPECTION REPORT	PAGE 3 of 11	ADWR NO.: 07.52 FCDMC NO.: 331	N/A	NO	YES	MON	REP	INV
INSPECTED BY: **Kimley-Horn Team**		DATE: **February 2, 2005**						
Item	Comments							

Item	N/A	NO	YES	MON	REP	INV
2. Protective coating deficient?	✓					
3. Misalignment or spilt seams?	✓					
e. Do the joints show:						
1. Displacement or offset?	✓					
2. Loss of joint material?	✓					
3. Leakage?	✓					
f. Are the trash racks:						
1. Broken or bent?		✓				
2. Corroded or rusted?		✓				
3. Obstructed?		✓				
g. Irrigation Outlet Gate(s):						
1. Broken or bent?		✓				
2. Corroded or rusted?		✓				
3. Leaking? **Reservoir empty - No indication of seepage**		✓				
4. Not seated properly?		✓				
5. Not operational?		✓				
6. Not periodically maintained?		✓				
7. Date last operated? **Operated quarterly. Based on the proximity of transverse cracks, the irrigation outlets with gates are to remain closed until after the Phase I Assessment has been completed.**						
7. IRRIGATION OUTLET CONDUITS: **All irrigation outlet conduits are constructed of 12-inch diameter steel pipe with cutoff collars. Based on video inspections completed in 2003, preliminary indications are no problems. A review and report of the 2003 video inspection is pending.** **(NOTE: June 7 2005: Video inspections have been performed and the video is being reviewed by the District).**						
a. Seepage into structure? **Reservoir empty - No indication of seepage**		✓				
b. Debris or obstructions?		✓				
c. If concrete, do surfaces show:						
1. Spalling or Scaling?	✓					
2. Cracking?	✓					
3. Erosion?	✓					
d. If metal, do surfaces show:						
1. Corrosion?						
2. Protective coating deficient?						
3. Misalignment or spilt seams?						
e. Do the joints show:						
1. Displacement or offset?						
2. Loss of joint material?						

Item / Comments	N/A	NO	YES	MON	REP	INV
3. Leakage?						
8. PRINCIPAL SPILLWAY – APPROACH CHANNEL						
a. Eroding or backcutting?		✓				
b. Sloughing?		✓				
c. Restricted by vegetation?		✓				
d. Obstructed with debris?		✓				
e. Silted in?		✓				
9. PRINCIPAL SPILLWAY - INLET STRUCTURE **Concrete box culvert type of construction (10 ft. wide by 8 ft. high) – Crest elevation 1179.0 (NAVD 1988). Does not include a trash rack and is ungated.**						
a. Seepage into structure? **Reservoir empty – no indications of seepage**		✓				
b. Debris or obstructions?		✓				
c. If concrete, do surfaces show:						
1. Spalling or Scaling?		✓				
2. Cracking? **Shrinkage and temperature cracks (minor non-structural) on the wing walls. No repairs required.**			✓	✓		
3. Erosion?		✓				
4. Exposed reinforcement?		✓				
d. If metal, do surfaces show:						
1. Corrosion?	✓					
2. Protective coating deficient?	✓					
3. Misalignment or spilt seams?	✓					
e. Do the joints show:						
1. Displacement or offset?		✓				
2. Loss of joint material?		✓				
3. Leakage?		✓				
f. Are the trash racks:						
1. Broken or bent?	✓					
2. Corroded or rusted?	✓					
3. Obstructed?	✓					
g. Principal Spillway Gate(s):						
1. Broken or bent?	✓					
2. Corroded or rusted?	✓					
3. Leaking?	✓					
4. Not seated properly?	✓					
5. Not operational?	✓					
6. Not periodically maintained?	✓					

Item / Comments	N/A	NO	YES	MON	REP	INV
7. Date last operated? **N/A**						
10. PRINCIPAL SPILLWAY CONDUIT: Concrete box culvert type of construction (10 ft. wide by 8 ft. high).						
a. Seepage into structure? **Reservoir empty – no indications of seepage**		✓				
b. Debris or obstructions?		✓				
c. If concrete, do surfaces show:						
1. Spalling or Scaling?		✓				
2. Cracking? **Shrinkage and temperature cracks (minor non-structural). No repairs required.**			✓	✓		
3. Erosion?		✓				
4. Exposed reinforcement?		✓				
d. If metal, do surfaces show:						
1. Corrosion?	✓					
2. Protective coating deficient?	✓					
3. Misalignment or spilt seams?	✓					
e. Do the joints show:						
1. Displacement or offset?		✓				
2. Loss of joint material?		✓				
3. Leakage? **Reservoir empty – no indications of seepage**		✓				
11. PRINCIPAL SPILLWAY CHUTE						
a. Seepage into chute?		✓				
b. Debris present?		✓				
c. If concrete, do surfaces show:						
1. Spalling or scaling?		✓				
2. Cracking?		✓				
3. Erosion?		✓				
4. Exposed reinforcement?		✓				
5. Other?		✓				
12. PRINCIPAL SPILLWAY - STILLING BASIN/POOL						
a. If concrete, do surfaces show:						
1. Spalling or Scaling?		✓				
2. Cracking? **Shrinkage and temperature cracks (minor non-structural) on the wing walls and baffle blocks. No repairs required.**			✓	✓		
3. Erosion?		✓				
4. Exposed reinforcement?		✓				
b. If concrete, do joints show:						
1. Displacement?		✓				
2. Loss of joint material?		✓				

Item / Comments	N/A	NO	YES	MON	REP	INV
b. Leakage?		✓				
c. Do the energy dissipaters show:						
1. Signs of deterioration?		✓				
2. Covered with debris?		✓				
3. Signs of inadequacy?		✓				
4. Other?		✓				
13. PRINCIPAL SPILLWAY – OUTLET CHANNEL **Grouted riprap lined channel**						
a. Eroding or backcutting?		✓				
b. Sloughing?		✓				
c. Obstructed?		✓				
d. Poorly riprapped?		✓				
e. Tailwater elevation and flow condition:						
14. EMERGENCY SPILLWAY-APPROACH CHANNEL **None Present**						
15. EMERGENCY SPILLWAY-CONTROL STRUCTURE **None Present**						
16. EMERGENCY SPILLWAY – CHANNEL **None Present**						
17. EMERGENCY SPILLWAY – OUTLET CHANNEL **None Present**						
18. RESERVOIR						
a. High water marks?		✓				
b. Erosion/Slides into pool area? **Did not walk reservoir rim**						
c. Sediment accumulation? **Unknown – sediment survey would be required**		?				
d. Floating debris present?		✓				
e. Depressions, sinkholes or vortices?		✓				
f. Low ridges/saddles allowing overflow?		✓				
g. Structures below dam crest elevation?		✓				
19. INSTRUMENTATION						
a. List type(s) of instrumentation: **Reservoir gage at principal spillway inlet and settlement survey monuments.**						
b. Any repair or replacement required? **Toe settlement monument B-25 appears to have been damaged.**			✓		✓	
c. Last monitoring report: **Surveys completed in 2002 and 2003. See additional comments.**						
20. CONDITION SUMMARY / EAP / MAINTENANCE RECOMMENDATIONS / NEXT INSPECTION						
a. Any safety deficiencies?		✓				
b. Safe storage level on License: **Principal spillway crest elevation 1176.9 ft. (NGVD 1929)**						
c. Date of current ADWR License: **April 7, 2000**						
d. Any ADWR Actions Outstanding? **Update EAP to meet the requirements of A.A.C. R12-15-1221(A) – currently the EAP update is scheduled for fall 2005.**			✓		✓	
e. Recorded size: **Significant** Should size be revised? **No**		✓				

Item	N/A	NO	YES	MON	REP	INV
f. Recorded downstream hazard: **Significant** Should hazard be revised? **The Phase I Structures Assessment is scheduled to be completed November 2004 to early 2005. A task in the Phase I includes the review and assessment of the current and expected 10-yr future downstream hazard potential. Recommendations for changes in the downstream hazard classification will be provided.**						✓
g. Date of last Emergency Action Plan revision: **2002** Should EAP be revised? **Needs to meet A.A.C. R12-15-1221(A) – see detail in 20.d.**			✓		✓	
h. Normal inspection frequency: Should inspection frequency be revised? **Annually**		✓				

i. Maintenance Recommendations:
(1) **Fill erosion rills upstream and downstream if over 12-inches deep**
(2) **Initiate gravel mulch recommendations resulting from the Phase I Structures Assessment**
(3) **Control rodent activity and repair damage due to rodent activity**
(4) **Repair longitudinal crack near the upstream toe between Sta. 250+91 to 262+20, 72+14 to 76+34, and Sta. 50+72 to 54+00 utilizing the same procedures utilized during the repair of similar cracking between Station 53+00 and 56+00**
(5) **Restore crest at Salome Road (coordinate with MCDOT)**
(6) **Replace toe settlement monument B-25 at Station 250+00.**

Item	N/A	NO	YES	MON	REP	INV
j. Is supplemental inspection required?		✓				

k. Recommended date for next inspection: **January 2006**

l. Status of Structure Assessment Program: **Contract issued and scheduled to begin December 2004.**

ADDITIONAL COMMENTS:

(1) Specific Location Details of Significant Conditions

Station	Location	Type	1st Date Report	2005 FY vs. 1st Report	2005 FY Inspection Results
General	Crest	Holes			There are holes located on the dam crest along the full length of the dam. The holes are located over the filter drain and typically are a series of holes in close proximity, holes vary in diameter from ½- to 12-inches and a probe can be inserted to its handle or to a depth of 34-feet.
9+00	U/S slope	Holes	2005		Small erosion holes around vegetation on the upstream slope. Appears to be preferred area for burrowing.
10+00 to 25+00	Floodway	Protection	2005		Between Sta. 10+00 and 25+00 the floodway appears to provide for significant protection against impoundment. The floodway is approximately 5 feet deep and 40 feet wide at this location and located a considerable distance from the toe of the dam.
15+25 to 15+65	Crest	Holes.	2002	Change	Series of holes along dam centerline found (max. 6-inch diameter). A 3-inch diameter hole was observed at Station 15+48 in 2003 and 2004. Another 3-inch diameter hole probed 24-inch was observed at Sta. 15+25 in 2004. The hole at Sta. 15+25 was observed in 2005 to be 6-inch in diameter and 24-inches deep.
16+00	D/S slope	Outlet Drain Pipe	2005		Fine sediment inside pipe, small sediment pile about 10-feet back in pipe. Was this pipe replaced at one time? Material of pipe looks like HDPE; pipe at 9+00 was ACP.
16+45	Crest	Hole	2004	No Change	2-inch diameter hole probed 18-inch.
17+08	Crest	Hole	2004	No Change	Two 2-inch diameter hole probed 18-inch.
17+70	Crest	Long.	2004	No Change	3-inch wide by 18-inch long probed 24-inch.
17+79	Crest	Hole	2004	No Change	1-inch diameter hole probed 24-inch
19+00	Crest	Hole	2005		Several holes in a 15-foot linear alignment
19+00	U/S slope	Erosion gully	2005		Minor erosion gully on upstream slope/groin area.
19+65	Crest	Long.	2004	Did not find	Series of 5 small holes probed 12-inch
20+82	Crest	Long.	2004	No Change	Series of 2 small holes probed 24-inch
[illegible]+00	U/S floodway	Holes	2005		Series of holes aligned perpendicular to dam on upstream bench between floodway and dam.
31+00	D/S slope	Trans.	2005		Series of holes aligned perpendicular to dam on upstream bench between floodway and dam

[illegible]+50	Crest	Hole	2005		Two large erosion holes: a. 2-feet diameter with 3-inch diameter pipe hole b. 18-inch diameter hole with 6-inch diameter pipe hole - probed 3-feet. (Photo 9)
59+00	Crest	Hole	2005		Big hole with Brett standing in hole. (Photo 7)
51+79	U/S & Crest	Hole	2001	Change	Possible transverse crack. Probed 24-inch in 2004. Only probed 34-inch in 2003.
52+45	U/S slope	Trans.	2003	Change	Transverse crack at upstream toe to about 5-feet up slope. Probed 34-inch in 2003 but only probed 6-inch in 2004.
53+40	U/S slope	Trans.	2003	Did not find	Transverse crack at upstream toe to about 10-feet up slope. Probed 34-inch in 2003.
53+86	U/S slope	Trans.	2003	Did not find	Did not observe a transverse crack only rodent holes
50+72 to 56+00	U/S slope	Long.	2000	Did not find	Depression and cracking at upstream toe. Cracking, first identified in 2000 was repaired by between 53+00 and 56+00 by backfilling with ASTM C33 sand. Additional intermittent cracking ¼ to ½-inch wide was observed between Sta. 50+72 and 54+00
54+83	U/S slope	Trans.	2003	No Did not find	Transverse crack at upstream toe to about 10-feet up slope. Probed 34-inch.
57+15 to 65+00	Crest	Long.	2004	Change	Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch. An 18-inch diameter hole was observed at Sta. 57+15 (probed 30-inch below the depression of the hole. 12-inch diameter holes were observed at Sta. 57+37 and 58+46. Series of small holes observed at Sta. 58+80. Six 12-inch diameter depressions observed at Sta. 62+31
57+35	D/S slope	Holes	2004		Possible transverse crack with small holes observed 5-feet up from the toe and near the crest.
57+72	U/S slope	Trans.	2003		Transverse crack at upstream toe and continues to within 15-feet of dam crest. Probed 34-inch.
60+00	D/S toe	Monu.	2005		At station 60+00 the furthest downstream monument has been undercut by grading and requires resetting. (Photo 6)
60+50	Crest	Trans.	2005	Change	Small (1/8-inch wide) crack observed over the vegetative outlet at this location. Observed 3-inch diameter hole during 2004 inspection. Outlet was closed. Hole has increased to 8-inch x 6-inch. (Photo 4)
60+50 to 65+00	D/S slope	Slope change	2005		Downstream Slope from Stations 60+50 to 65+00: Noticeable change in downstream slope face: more rilling, pronounced gullies, and change in vegetation on slopes. No creosote around which was evident at lower stations. (Photo 14)
64+67	U/S slope	Trans.	2003	Change	Transverse crack at upstream toe berm and at dam crest. No indication of crack along mid-height of slope. Appears to be a collapsed rodent burrow.
64+90	U/S slope	Long			Crack or erosion at the toe of the upstream slope, approximately 20 feet long, 6 inches wide, inspection probe inserted approximately 2 feet . (Photo 12)
67+00	U/S slope	Long	2005		Crack or erosion at the toe of the upstream slope, approximately 20 feet long, 1 to 2 inches wide, inspection probe inserted approximately 1 foot.
67+50 to 70+00	Crest	Long.	2004	No change	Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch. 6-inch by 12-inch holes observed at Sta. 67+91 and 68+12. The hole at 68+12 was probed 4 feet.
71+00	U/S slope	Long	2005		Crack or erosion at the toe of the upstream slope, approximately 20-feet long, 1- to 2-inches wide.
72+14 to [illegible]+76	Crest	Long.	2004		Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch. 6-inch by 12-inch depression observed at Sta. 76+34. 12-inch diameter depression observed at Sta. 78+25. Two 1-inch diameter holes at 80+56. 18-inch diameter depression 6-inch deep observed at Sta. 80+93. 1-inch diameter hole at Sta. 83+28. 5-inch diameter hole at Sta. 84+12 probed 24-inch. 1-inch diameter hole at Sta. 84+76 probed 12-inch.

[illegible]36	U/S slope	Trans.	2002	Did not find	Possible transverse crack on upstream slope about 10-feet up from toe. Probed 24-inch.
75+51	D/S side Crest	Long.	2002	No change	Longitudinal crack on downstream side of crest about 3-feet from crest. Crack about 45-feet long and probe inserted 24-inch.
82+94	U/S slope	Trans.	2004		Possible transverse crack observed 5-feet up from the toe.
87+66	Crest	Hole	2004		1-inch diameter hole probed 12-inch.
90+86	Crest	Hole	2004		1-inch diameter hole probed 12-inch.
94+54	U/S slope	Trans.	2002	Did not find	Possible transverse crack on upstream slope from toe to mid-height.
96+17	U/S slope	Trans.	2003	No change	Possible transverse crack about min-height on upstream slope. Probe inserted about 12-inch in a series of four holes in line.
97+50	U/S slope	Hole	2005		Erosion hole on the upstream slope approximately 2 feet long, and 3 to 6 inches wide, inspection probe inserted 1 foot.
98+30	U/S slope	Trans.	2002	Change	Transverse crack on upstream slope from about 5-feet above toe to mid-height of dam. Probed 30-inch.
104+00	Crest	Hole	2005		Root zone hole 2 ½ feet from upstream crest.
104+39	U/S slope	Trans.	2004		Possible transverse crack indicated by hole midway up the slope probed 24-inch.
106+81	Crest	Long.	2004		Two 5-inch diameter holes about 3-inch apart in the centerline of the crest probed 18-inch.
109+73 to 117+35	Crest	Long.	2004		Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch. 6-inch diameter depression 18-inch deep at Sta. 109+73
119+90	Crest	Trans.	2002	Did not find	Transverse crack on dam crest from upstream side to centerline.
130+09	U/S slope	Trans.	2002	Did not find	Possible transverse crack on upstream slope at mid-height.
140+36	Crest	Hole	2004		4-inch diameter depression probed 12-inch.
[illegible]66 to 146+22	Crest	Long.	2004	Change	Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch.
150+61 to 160+96	Crest	Long.	2004		Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch. Three 6-inch diameter holes over a 3-feet length at Sta. 150+84. Depression 1-foot wide and 12-inch long probed 12-inch at Sta. 154+07. Two 4-inch diameter depressions at 155+13
156+34 to 160+54	U/S slope	Trans.	2004		Possible transverse cracks.
165+90	U/S slope	Slope Depress ion	2005		Depression at upstream edge of crest, 1½-feet in diameter & 0.3-feet deep. Hole located just below downstream shoulder approximately 1.5-feet wide and 4-inches deep.
169+72	Crest	Hole	2004	No change	3-inch diameter depression in the centerline of the crest. (Noted in as Station 169+75 in 2004 Inspection Report)
175+40	Crest	Hole	2002	No change	Hole in dam crest over filter-drain. Hole probed 12-inch.
197+75	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope near the crest.
198+63	U/S slope	Trans.	2003		Possible transverse crack in US embankment slope; probed13-inches
204+00	U/S slope	Trans.	2003	Did not find	Just south of south gate on Salome toad found a transverse crack on upstream slope that extends to dam crest. Did not find in 02/02/05 inspection nor in the 11/15/04 inspection.
204+00	D/S Slope	Toe Drain	2005	Did not find	Could not locate toe drain at Station 204+00. This location is just south of the Salome Highway Road crossing.
Solome [illegible]d at [illegible]S		Road	2005		Road cuts into crest of dam. Recommend rebuild to correct dam height and hard surfacing (asphalt or concrete) road pavement at FRS location to prevent "over grading" by others of FRS. (Photo 10)

[illegible]+13 to 256+52	Crest	Long.	2004		Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch. Two depressions (one 6-inch diameter and one 12-inch diameter) at Sta. 212+17. 5-inch diameter hole 4-inhes deep; could not probe at 215+21 6-inch diameter hole probed 24-inch at Sta. 218+57. 6-inch diameter hole with visible longitudinal crack in the sidewall at Sta. 227+04. 6-inch diameter depression at Sta. 227+23. 12-inch diameter depression at Sta. 227+66. 6-inch diameter depressions at Sta. 235+52, 247+76, and 248+95. Numerous depressions in gravel road layer (applied to mitigate safety issues to personnel and vehicle traffic) between Sta. 250+00 and 255+00 where previous holes had been reported.
218+57	Crest	Hole	2005		10-inch diameter hole, 6-inches deep on centerline of crest probed to 1-foot depth. (Photo 8)
220+00 to 225+00	Crest	Holes/ Depressions	2005		Various holes and depressions on crest at and near centerline (occurs throughout length of dam).
227+50	Crest		2005		8-inch diameter depression approximately 2-inches deep at CL
227+59	Crest		2005		12-inch diameter depression approximately 2-inches deep at CL
234+24	Crest	Holes	2005		Several small burro holes near crest
238+77	Crest	Hole	2005		2-inch diameter hole on the crest probed 8-inch.
244+00	D/S Slope	Toe Drain	2005		Toe drain completely fill with sediment
245+00-250+00	Crest	Holes	2005		Various holes and depressions on crest at or near CL
246+63	Crest	Hole	2005		1-inch diameter hole on the crest probed 20-inch.
[illegible]+00	Crest	Hole	2005		6- and 1-inch diameter hole on the crest probed 24-inch.
[illegible]+00	D/S Slope	Toe Drain	2005		Could not locate toe drain at this location
250+47-250+83	Crest	Holes	2005		Various holes and depressions on crest at or near CL
251+80-252+39	U/S slope	Holes	2005		Heavy rodent activity on upstream slope.
254+06	U/S slope	Long.	2005		Approximate end of longitudinal crack feature at US toe of embankment that started at station 255+52. Noted small burrow holes 2-3 feet below crest.
253+29	Crest		2005		6-inch diameter depression approximately 2-inches deep
254+21	Crest	Hole			1-inch diameter hole on the crest probed 12-inches.
254+97	Crest		2005		16-inche wide-1-inch deep depression on crest
255+52	U/S slope	Long.	2005		Longitudinal crack along downstream toe of low embankment section; probed 18 to 20 inches, 3 to 4 inches wide; crack discontinuous.
256+00	D/S slope	Trans.	2003	Did not find	Possible transverse crack on downstream slope where irrigation outlet located. Crack extends from downstream slope to crest centerline. Did not see crack on upstream slope. (Photo 3)
256+42	Crest	Hole	2005		2-inch diameter hole on the crest probed 18-inch.
257+02	Crest	Hole	2005		6-inch diameter hole on the crest probed 18-inch.
258+02 to 257+47					Longitudinal crack along downstream toe of low embankment section; series of 8 aligned erosion pipes.
255+00 to 260+00	U/S slope	Holes	2005		Heavy rodent activity on upstream slope.
[illegible]7+02	Crest Centerline	Hole			
260+00	Crest	Trans.	2000	Did not find	Crack first reported in 2000 but have not found crack in subsequent inspections, including 2003. Noted some rodent holes on upstream side near crest.

[illegible]+92	U/S slope	Long	2005		Longitudinal crack along downstream toe of low embankment section; ½-inch. to 10 inches in maximum diameter; shows evidence of recent inflow. (Photo 11)
General Observation					The creosote shrubs typically have holes around the root zone, likely associated with burrowing animals.

Notes:
An exhaustive effort was not made during the 2005 FY inspection to locate previously identified crack features. It is accepted that cracks exist and have been well documented in prior inspections and investigations. If previously identified features were located they are noted, new features were also documented and described.

(2) Review of 2002 & 2003 Subsidence Survey*

A review of the 2003 subsidence survey data (attached) indicates a low spot (-0.11 ft.) in the crest around Sta. 70+00. Repairs at this location are not recommended at this time. Station 70+00 lies within a stretch of the dam that is overbuilt by more than 0.25-ft. Survey points B19 shows an inconsistent (spike) reading in the 1986 data set with respect to more recent survey data recordings. The 1986 data should be considered unreliable for this particular survey point. Survey point B25 appears to have been damaged which is supported by the inconsistent (spike) data shown in the 2001, 2002, and 2003 survey data sets. Stability of survey point B25 should be assessed and repairs made as necessary. A new baseline for point B25 should then be developed.

The next subsidence survey is scheduled for FY 2005-2006.
***From November 2004 Inspection Report**

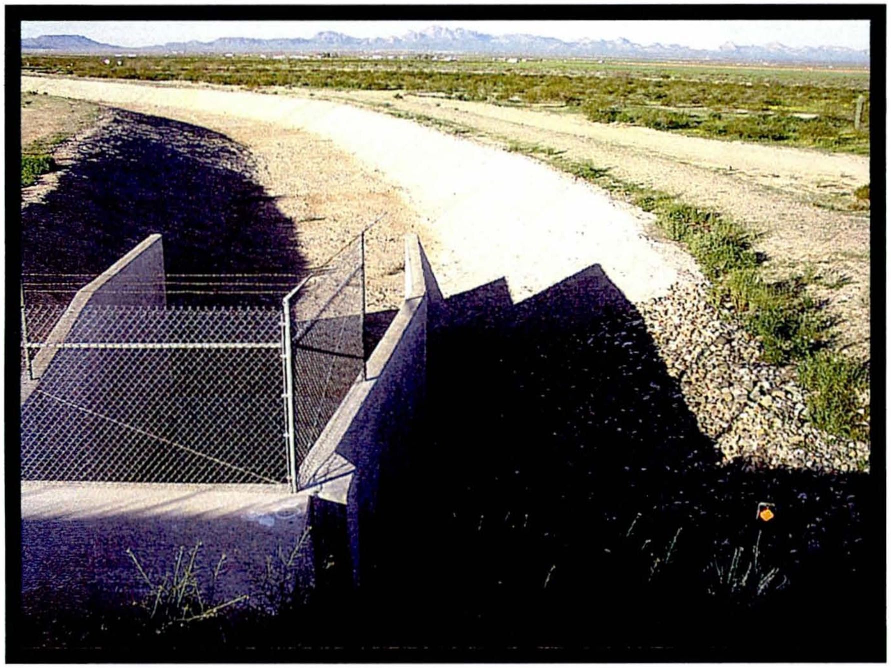


Photo 1 – Principal Outlet Floodway: Saddleback diversion channel.

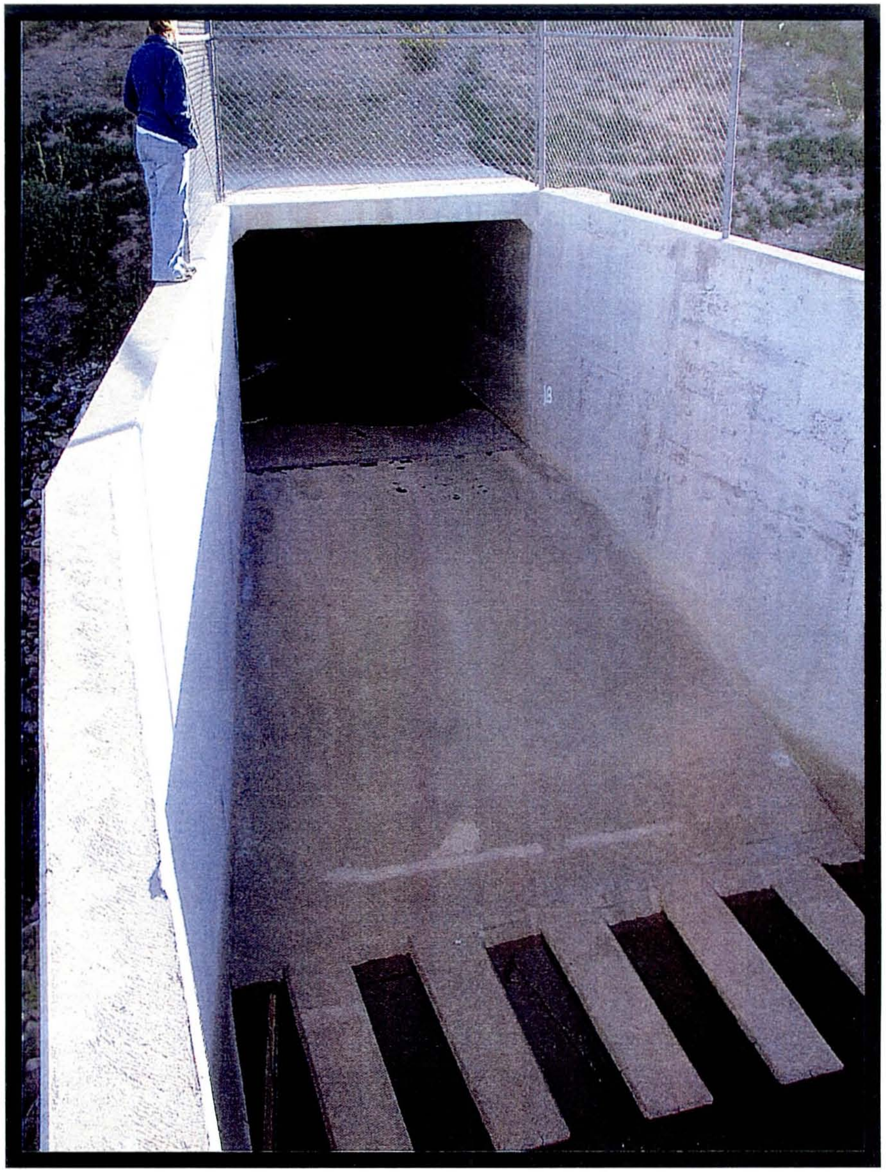


Photo 2 – Saddleback FRS Principal Outlet and Energy Dissipator.

Photo 3 – Typical Toe Drain Outlet Pipe.

Photo 4 – Downstream Slope: Typical view of downstream embankment slope looking northwest.

Photo 5 – Crest Centerline at Station 57+50:
A. 2-feet diameter erosion hole with 3-inch diameter pipe hole.
B. 18-inch diameter erosion hole with 6-inch diameter pipe hole probed 3-feet.

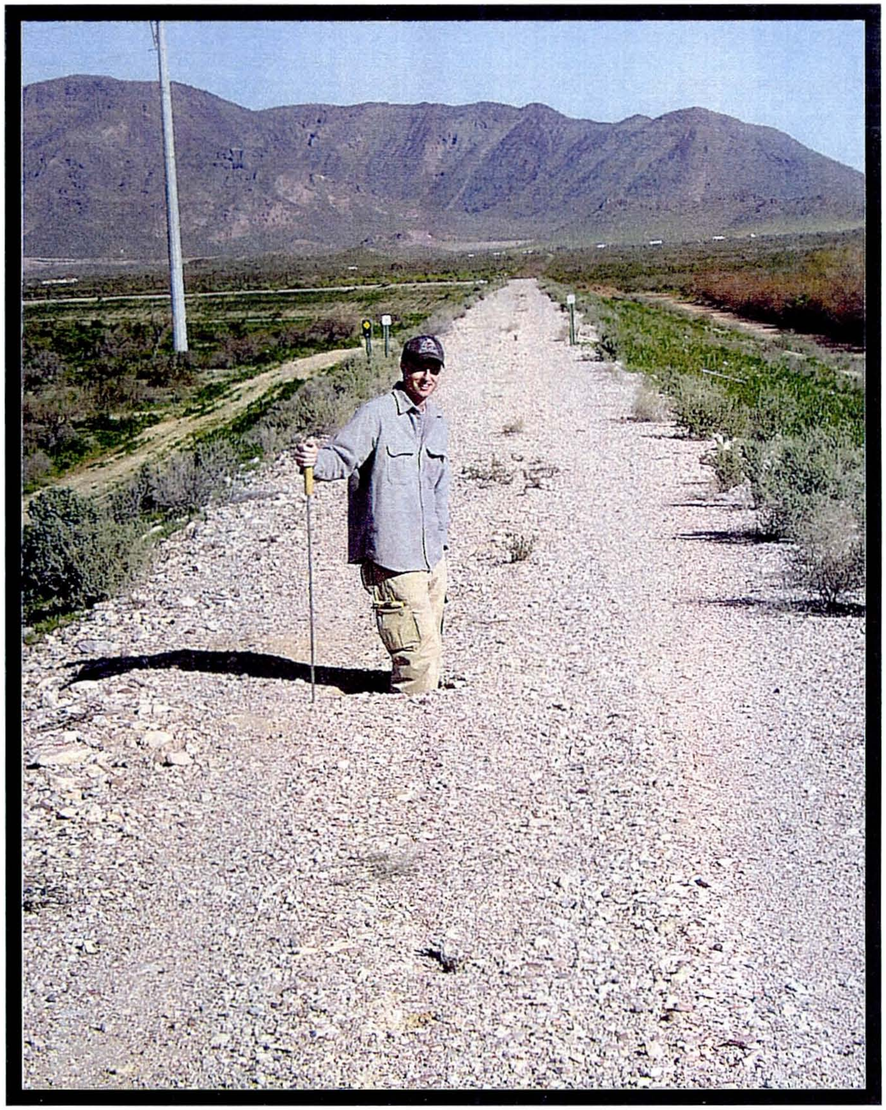


Photo 6 – Crest at Station 59+00: Large erosion hole.

Photo 7 – Downstream Monument: At station 60+00 the furthest downstream monument has been undercut by grading and requires resetting.

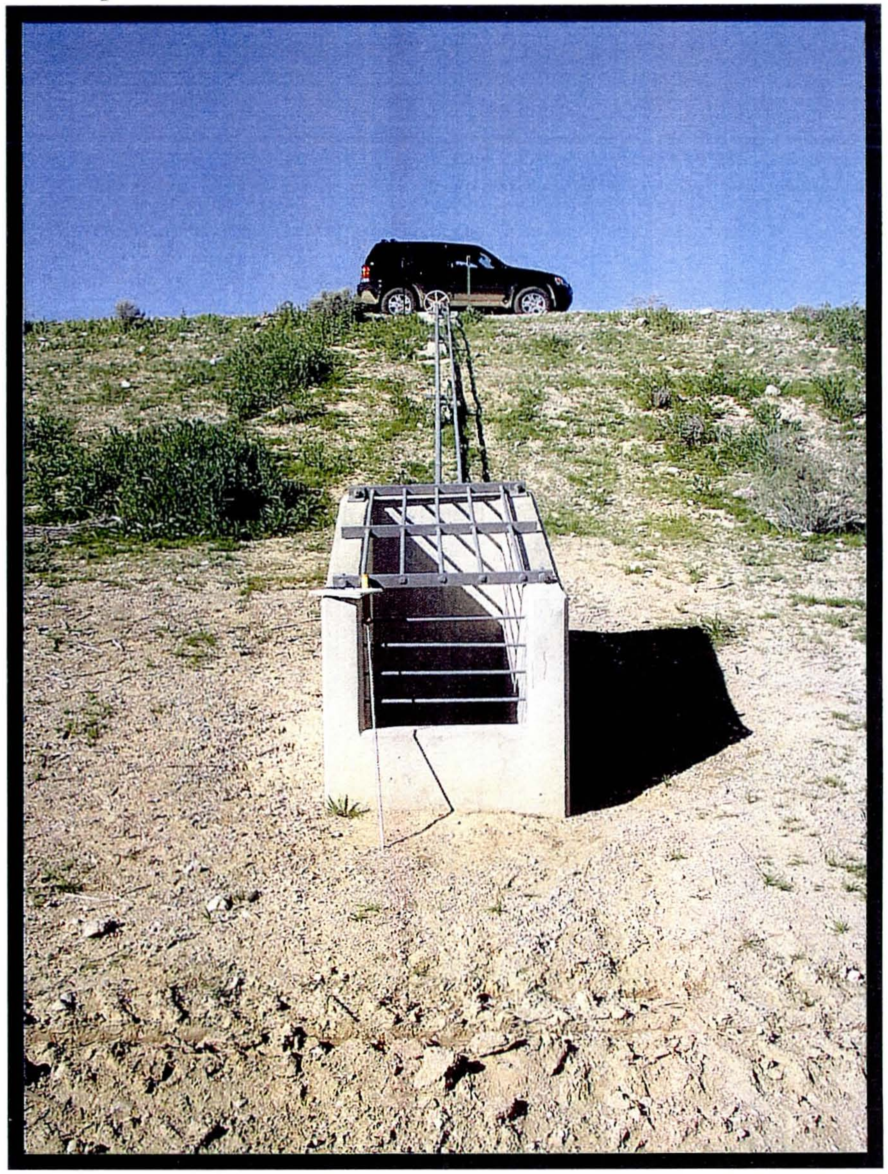


Photo 8 – Vegetation Outlet at Station 60+50: Box was clean with the valve closed. Two holes were located on either side of the outlet.

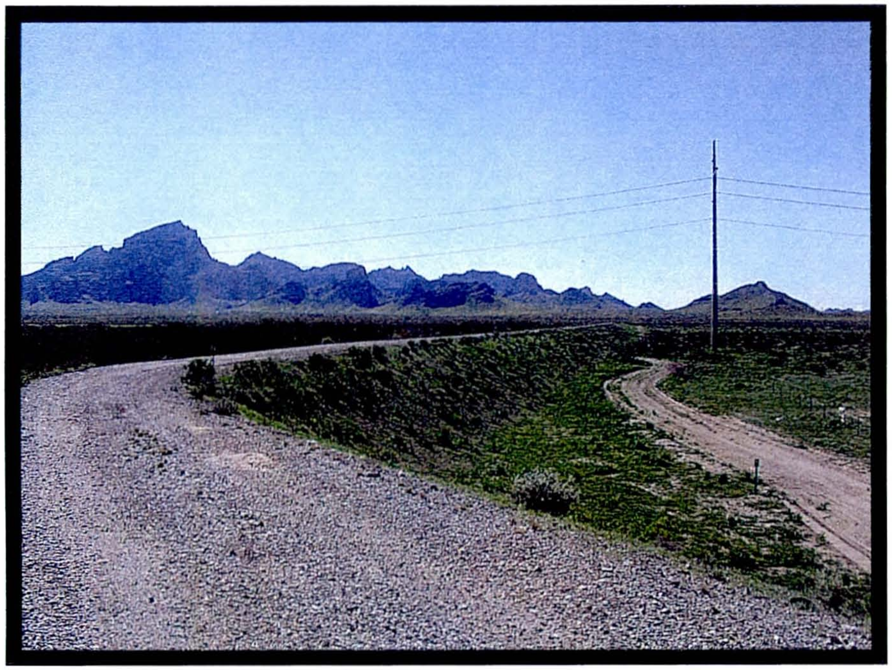


Photo 9 – Downstream Slope from Stations 60+50 to 65+00: Noticeable change in downstream slope face: more rilling, pronounced gullies, and change in vegetation on slopes. No creosote around which was evident at lower stations.

Photo 10 – Upstream Slope at Station 64+90: Crack or erosion at the toe of the upstream slope, approximately 20-feet long, 6-inches wide, inspection probe inserted approximately 2-feet.

Photo 11 – Erosion rills that is possibly a transverse crack at Station 197+75: Old reports indicate a possible transverse crack at this location.

Photo 12 – Upstream Slope at Solome Road Crossing (Station 203+00): View of upstream slope looking southwest.

Photo 13 – Crest at Station 218+57: 10-inch diameter Hole, 6-inches deep on crest at centerline probed to 1-foot depth.

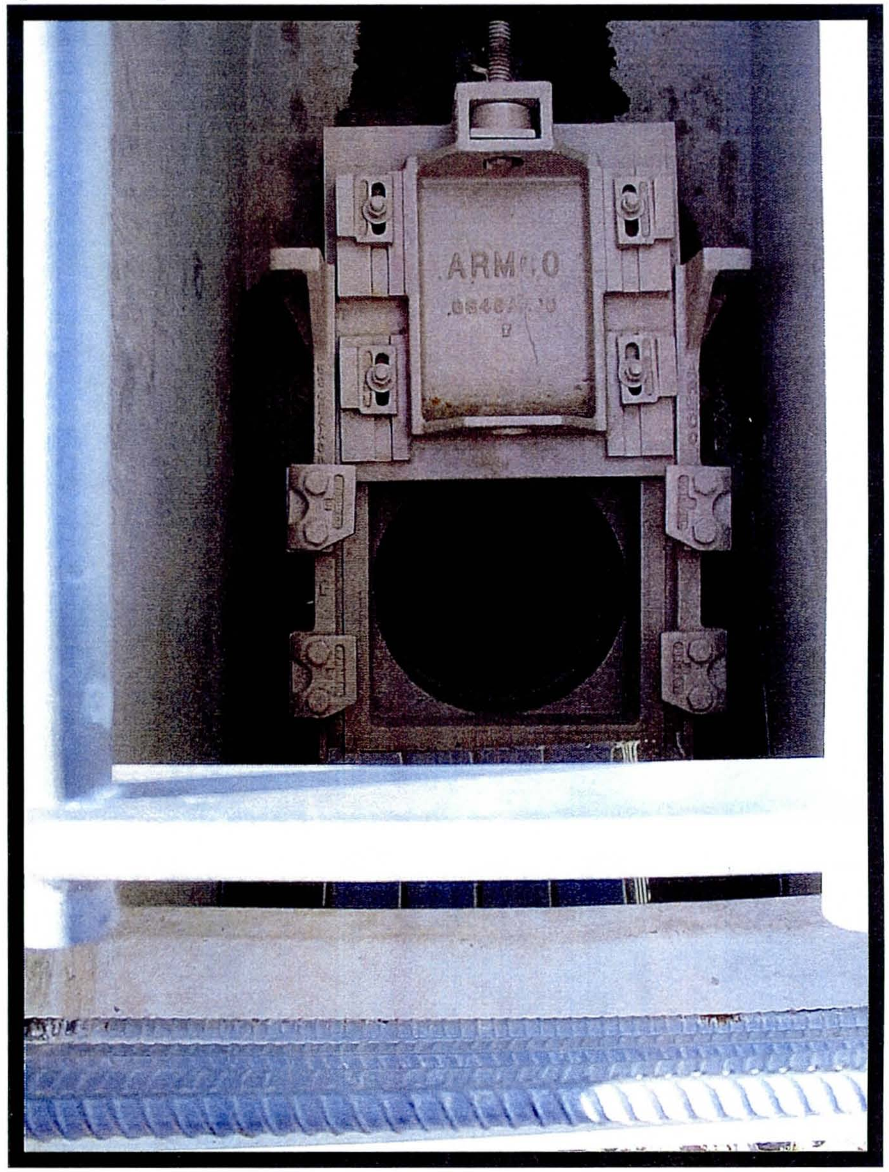


Photo 14 – Vegetation Gate at Station 256+00 : Vegetation gate with the outlet gate open.

Photo 15 – Upstream Slope at Station 261+92: Longitudinal crack along downstream toe of low embankment section; ½-inch to 10 inches in maximum diameter; shows evidence of recent inflow.

March 17,1998 Inspection Conducted by K.M. Hussain Reviewed by Jon Benoist (ADWR)
Contacts: Ernie Hamer (FCDMC) and Chuck Smith (FCDMC)

Inspection Notes					
Station	Location	Type	Date First Reported	1998 FY vs. First Report	1998 FY Inspection Results
75+00	Crest	Hole	1998	No Comment	Series of rodent holes along the centerline of the dam crest, need to be repaired by filling the hole with sand and bentonite slurry.
250+00 to 255+00	Crest	Hole	1998	No Comment	Series of rodent holes along the centerline of the dam crest, need to be repaired by filling the hole with sand and bentonite slurry.

March 30, 1999 Inspection Conducted by Tom Renckly (FCDMC), Chuck Smith (FCDMC), Ernie Hamer (FCDMC) and Carlos Rivera (FCDMC)

Inspection Notes					
Station	Location	Type	Date First Reported	1999 FY vs. First Report	1999 FY Inspection Results
44+00 to 52+00	Crest	Longitudinal	1996	No change	The longitudinal cracking on the crest that had been restored a few years ago still appears stable with no signs of any more settlement.
65+00 to 70+00	D/S slope	Holes	1999	No Comment	On the downstream slope between Sta. 65+00 and 70+00, many large holes were noticed that appear to be dug by a large animal and a few more were noticed intermittently further upstream to the north.

February 2, 2000 Inspection Conducted by Michael Greenslade (FCDMC) Reviewed by Jon Benoist (ADWR)
Contacts: Tom Renckly (FCDMC) and Noller Hebert (NRCS)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2000 FY vs. First Report**	**2000 FY Inspection Results**
General	Crest	Holes	(No Date)	No Comment	There are holes located on the dam crest along the full length of the dam. The holes are located over the filter drain and typically are a series of holes in close proximity, holes vary in diameter from ½- to 6-inch and a probe can be inserted to a maximum depth of about 24-feet.
(No Station)	D/S slope	Transverse	2000	No Comment	A transverse crack was observed over the drain outlet located right of the principal spillway.
44+00 to 55+00	D/S slope	Ground Cover	2000	No Comment	There is virtually no ground cover in the area where NRCS repaired the central filter/drain in 1996. It appears that excavated material was spread over the downstream slope.
64+00 to 65+00	U/S slope	Longitudinal	2000	No Comment	Longitudinal crack, probed to a depth of about 3-feet, was observed in the protective berm between Sta. 64+00 and Sta. 65+00.
66+50	U/S slope	Transverse	2000	No Comment	Transverse crack observed at Sta. 66+50 in the protective berm.
53+50 to 56+00	Reservoir	Longitudinal	2000	No Comment	Longitudinal cracks up to 3-inch wide and 3-feet deep were observed in the upstream maintenance road adjacent to the protective berms.
260+00 to 260+50	Reservoir	Longitudinal	2000	No Comment	Longitudinal cracks up to 3-inch wide and 3-feet deep were observed in the upstream maintenance road adjacent to the protective berms.

January 14-15, 2003 Inspection Conducted by Larry Lambert (FCDMC)
Contacts: Noller Hebert (NRCS) and Brett Howey (ADWR)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2003 FY vs. First Report**	**2003 FY Inspection Results**
General	Crest	Holes		No Comment	There are holes located on the dam crest along the full length of the dam. The holes are located over the filter drain and typically are a series of holes in close proximity, holes vary in diameter from ½- to 6-inch and a probe can be inserted to a maximum depth of about 24-feet.
12+00 to 15+00	Crest	Longitudinal	1999	Did not find	Longitudinal crack along centerline of dam was reported in various locations beginning at Sta. 12+96 to Sta. 14+55 (these cracks could not be found in 2002 or 2003).
15+25 to 15+65	Crest	Longitudinal	2002	Add. Hole on crest found	Series of holes along dam centerline found (max. 3-inch diameter). Another 3-inch diameter hole was observed at Sta. 15+50 in 2003.
15+90	Crest and possible D/S side	Transverse	2002	No change	Transverse crack in line or close to alignment of right side or principal spillway box outlet. Crack is wider towards downstream side of crest and becomes hairline about 5-feet down from crest. Station location in 2003 was 16+09. The original location was noted as 15+90 (see comment below).
16+09				No Comment	Transverse crack in line or close to alignment of right side or principal spillway box outlet. Crack is wider towards downstream side of crest and becomes hairline about 5-feet down from crest. Station location in 2003 was 16+09. The original location was possibly in error. There are not two separate cracks at 15+90 and 16+09.
29+21	Crest	Transverse	2002	Did not find	Transverse crack on crest from shoulder to centerline where intersects longitudinal crack.
37+32	U/S Slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope about mid-height. Probe inserted about 12-inch, crack also runs parallel along slope.
48+79	Crest	Transverse	2002	Did not find	Possible transverse crack at crest towards the upstream slope.
49+10	U/S Slope	Transverse	2002	Did not find	Transverse crack runs from upstream slope about mid-height across crest to downstream slope about one-third down from crest.
51+79	U/S & Crest	Transverse	2001	No change	Inserted probe to handle, no change in 2003.
52+45	U/S slope	Transverse	2003	No Comment	Transverse crack at upstream toe to about 5-feet up slope. Probe inserted to handle (34-inch).

January 14-15, 2003 Inspection Conducted by Larry Lambert (FCDMC)
Contacts: Noller Hebert (NRCS) and Brett Howey (ADWR)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2003 FY vs. First Report**	**2003 FY Inspection Results**
53+40	U/S slope	Transverse	2003	No Comment	Transverse crack at upstream toe to about 10-feet up slope. Probe inserted to handle (34-inch).
53+86	U/S slope	Transverse	2003	No Comment	Transverse crack at upstream toe to about 10-feet up slope. Probe inserted to handle (34-inch).
54+00	U/S slope	Longitudinal	(No date)	No change	Depression at upstream toe. Longitudinal crack was open and could insert probe to handle (34-inch). Crack runs approximately 140-feet.
54+83	U/S slope	Transverse	2003	No Comment	Transverse crack at upstream toe to about 10-feet up slope, probed 34-inch.
57+72	U/S slope	Transverse	2003	No Comment	Transverse crack at upstream toe and continues to within 15-feet of dam crest, probed 34-inch.
64+67	U/S slope	Transverse	2003	No Comment	Transverse crack at upstream toe berm and at dam crest. No indication of crack along mid-height of slope.
66+50	U/S slope	Transverse	2001	Did not find	Transverse crack on bench at upstream toe.
71+15	U/S slope	Longitudinal	2001	Did not find	Did not find longitudinal crack in 2002 or 2003.
71+62	U/S slope	Transverse	2001	Did not find	Possible transverse crack reported about 12-feet up from toe. Did not find in 2002.
72+14 to 76+34	D/S toe	Longitudinal	2002	No Comment	Longitudinal crack varies from hairline to 1/8-inch wide at downstream toe. Probe inserted at various locations along crack 12- to 18-inch. Could only insert probe about 6-inch in adjacent ground several feet away from crack.
73+36	U/S slope	Transverse	2002	No change	Possible transverse crack on upstream slope about 10-feet up from toe, probed 24-inch.
75+51	D/S side Crest	Longitudinal	2002	No change	Longitudinal crack on downstream side of crest about 3-feet from crest. Crack about 45-feet long and probe inserted 24-inch.
81+46	D/S slope	Transverse	2002	Did not find	Possible transverse crack on downstream slope from toe to about mid-height, probe inserted to 18-inch.
82+00	D/S toe	Longitudinal	2022	Did not find	Possible longitudinal crack on downstream slope about 4-feet away from toe towards the maintenance yard.

January 14-15, 2003 Inspection Conducted by Larry Lambert (FCDMC)
Contacts: Noller Hebert (NRCS) and Brett Howey (ADWR)

Inspection Notes					
Station	Location	Type	Date First Reported	2003 FY vs. First Report	2003 FY Inspection Results
94+54	U/S slope	Transverse	2002	No change	Possible transverse crack on upstream slope from toe to mid-height.
94+84	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope from mid-height to dam crest.
95+28	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope from toe to mid-height.
95+76	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope from toe to mid-height.
96+20	U/S slope	Transverse	2003	No Comment	Possible transverse crack about mid-height on upstream slope. Probe inserted about 12-inch in a series of four holes in line.
96+30	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope from toe to mid-height.
97+52	Crest	Transverse	2002	Did not find	Transverse crack on dam crest from downstream side to centerline.
98+45	U/S slope	Transverse	2002	Did not find	Transverse crack on upstream slope from about 5-feet above toe to mid-height of dam, probe inserted to 24-inch.
116+65	Crest	Transverse	2002	Did not find	Transverse crack on dam crest from upstream side to centerline.
119+90	Crest	Transverse	2002	No change	Transverse crack on dam crest from upstream side to centerline.
130+09	U/S slope	Transverse	2002	No change	Possible transverse crack on upstream slope at mid-height.
160+54	U/S slope	Transverse	2002	No change	Possible transverse crack on upstream slope at mid-height.
175+55	Crest	Hole	2002	No change	Hole in dam crest over filter-drain. Hole 6-inch diameter and 18-inch deep.
197+80	D/S slope	Transverse	2002	No change	Possible transverse crack on downstream slope from about mid-height to crest.

January 14-15, 2003 Inspection Conducted by Larry Lambert (FCDMC)
Contacts: Noller Hebert (NRCS) and Brett Howey (ADWR)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2003 FY vs. First Report**	**2003 FY Inspection Results**
204+00	U/S slope	Transverse	2003	No Comment	Just south of south gate on Salome Road found a transverse crack on upstream slope that extends to dam crest.
215+27	Crest	Transverse	2001	Did not find	Transverse crack on crest and did not find in 2003.
218+54	Crest& D/S slope	Transverse	2001	Did not find	Did not find crack in either 2002 or 2003.
256+00	D/S slope	Transverse	2003	No Comment	Possible transverse crack on downstream slope where irrigation outlet located. Crack extends from downstream slope to crest centerline. Did not see crack on upstream slope.
257+40	Crest	Hole	2003	No Comment	The last location along dam crest where a hole over the filter drain was noted.
250+91 to 261+75	U/S toe	Longitudinal	2003	No Comment	Longitudinal crack at upstream toe. Exact location varies from slightly up on slope to toe. Crack is not one continuous crack but may skip 10- to 15-feet lengths between these two stations.
260+00	Crest	Transverse	2000	No change	Crack first reported in 2000 but have not found crack in subsequent inspections, including 2003.

January 13, 2004 Inspection Conducted by Michael Greenslade, PE (FCDMC) and reviewed by Brett Howey, P.E.
Contacts: John Chua (NRCS) and ADWR

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2004 FY vs. First Report**	**2004 FY Inspection Results**
General	Crest	Holes		No Comment	There are holes located on the dam crest along the full length of the dam. The holes are located over the filter drain and typically are a series of holes in close proximity, holes vary in diameter from ½ - to 12-inch and a probe can be inserted to its handle or to a depth of 34-inch.
15+25 to 15+65	Crest	Longitudinal	2002	Change	Series of holes along dam centerline found (max. 3-inch diameter). A 3-inch diameter hole was observed at Station 15+48 in 2003 and 2004. Another 3-inch diameter hole probed 24-inch was observed at Sta. 15+25 in 2004.
16+09	Crest and D/S slope	Transverse	2002	Did not find	Transverse crack in line or close to alignment of right side or principal spillway box outlet. Crack is wider towards downstream side of crest and becomes hairline about 5-feet down from crest. Station location in 2003 was 16+09. The original location was possibly in error. There are not two separate cracks at 15+90 and 16+09.
16+45	Crest	Hole	2004	No Comment	2-inch diameter hole probed 18-inch.
17+08	Crest	Hole	2004	No Comment	Two 2-inch diameter holes probed 18-inch.
17+70	Crest	Longitudinal	2004	No Comment	Longitudinal crack 3-inch wide by 18-inch long, probed 24-inch.
17+79	Crest	Hole	2004	No Comment	1-inch diameter hole probed 24-inch.
19+65	Crest	Longitudinal	2004	No Comment	Series of 5 small holes probed 12-inch.
20+82	Crest	Longitudinal	2004	No Comment	Series of 2 small holes probed 24-inch.
51+79	U/S & Crest	Hole	2001	Change	Possible transverse crack. Probed 24-inch in 2004. Probed 34-inch in 2003.
52+45	U/S slope	Transverse	2003	Change	Transverse crack at upstream toe to about 5-feet up slope. Probed 34-inch in 2003 but only probed 6-inch in 2004.
53+40	U/S slope	Transverse	2003	No change	Transverse crack at upstream toe to about 10-feet up slope. Probed 34-inch in 2003.

January 13, 2004 Inspection Conducted by Michael Greenslade, PE (FCDMC) and reviewed by Brett Howey, P.E.
Contacts: John Chua (NRCS) and ADWR

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2004 FY vs. First Report**	**2004 FY Inspection Results**
53+86	U/S slope	Transverse	2003	Change	Did not observe a transverse crack only rodent holes.
50+72 to 56+00	U/S slope	Longitudinal	2000	Change	Depression and cracking at upstream toe. Cracking, first identified in 2000 was repaired between 53+00 and 56+00 by backfilling with ASTM C33 sand. Additional intermittent cracking ¼- to ½-inch wide was observed between Sta. 50+72 and 54+00.
54+83	U/S slope	Transverse	2003	Did not find	Transverse crack at upstream toe to about 10-feet up slope, probed 34-inch.
57+15 to 65+00	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. An 18-inch diameter hole was observed at Sta. 57+15 (probed 30-inch below the depression of the hole). 12-inch diameter holes were observed at Sta. 57+37 and 58+46. Series of small holes observed at Sta. 58+80. Six 12-inch diameter depressions observed at Sta. 62+31.
57+35	D/S slope	Holes	2004	No Comment	Possible transverse crack with small holes observed 5-feet up from the toe and near the crest.
57+72	U/S slope	Transverse	2003	No Comment	Transverse crack at upstream toe and continues to within 15-feet of dam crest, probed 34-inch.
60+50	Crest	Transverse	2004	Change	Small (1/8-inch wide) crack observed over the vegetative outlet at this location. Observed 3-inch diameter hole during 2004 inspection. Ordered the outlet closed as a precaution.
64+67	U/S slope	Transverse	2003	Did not find	Transverse crack at upstream toe berm and at dam crest. No indication of crack along mid-height of slope.
67+50 to 70+00	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. 6- by 12-inch holes were observed at Sta. 67+91 and 68+12. The hole at 68+12 was probed 4-feet.
72+14 to 84+76	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. 6- by 12-inch depression observed at Sta. 76+34. 12-inch diameter depression observed at Sta. 78+25. Two 1-inch diameter holes at 80+56. 18-inch diameter depression 6-inch deep observed at Sta. 80+93. 1-inch diameter hole at Sta. 83+28. 5-inch diameter hole at Sta. 84+12 probed 24-inch. 1-inch diameter hole at Sta. 84+76 probed 12-inch.

January 13, 2004 Inspection Conducted by Michael Greenslade, PE (FCDMC) and reviewed by Brett Howey, P.E.
Contacts: John Chua (NRCS) and ADWR

Inspection Notes					
Station	Location	Type	Date First Reported	2004 FY vs. First Report	2004 FY Inspection Results
73+36	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope about 10-feet up from toe, probed 24-inch.
75+51	D/S side Crest	Longitudinal	2002	No change	Longitudinal crack on downstream side of crest about 3-feet from crest. Crack about 45-feet long and probe inserted 24-inch.
82+94	U/S slope	Transverse	2004	No Comment	Possible transverse crack observed 5-feet up from the toe.
87+66	Crest	Hole	2004	No Comment	1-inch diameter hole probed 12-inch.
90+86	Crest	Hole	2004	No Comment	1-inch diameter hole probed 12-inch.
94+54	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope from toe to mid-height.
96+17	U/S slope	Transverse	2003	No change	Possible transverse crack about mid-height on upstream slope. Probe inserted about 12-inch in a series of four holes in line.
98+30	U/S slope	Transverse	2002	Change	Transverse crack on upstream slope from about 5-feet above toe to mid-height of dam. Probed 30-inch.
104+39	U/S slope	Transverse	2004	No Comment	Possible transverse crack indicated by hole midway up the slope probed 24-inch.
106+81	Crest	Longitudinal	2004	No Comment	Two 5-inch diameter holes about 3-inch apart in the centerline of the crest probed 18-inch.
109+73 to 117+35	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch. 6-inch diameter depression 18-inch\ deep at Sta. 109+73.
119+90	Crest	Transverse	2002	Did not find	Transverse crack on dam crest from upstream side to centerline.
130+09	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope at mid-height.
140+36	Crest	Hole	2004	No Comment	4-inch diameter depression probed 12-inch.

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Contacts: John Chua (NRCS) and ADWR

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2004 FY vs. First Report**	**2004 FY Inspection Results**
142+66 to 146+22	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch.
150+61 to 160+96	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. Three 6-inch diameter holes over a 3-feet length at Sta. 150+84. Depression 1-feet wide and 12-inch long probed 12-inch at Sta. 154+07. Two 4-inch diameter depressions at 155+13.
156+34	U/S slope	Transverse	2004	No Comment	Possible transverse crack.
156+44	U/S slope	Transverse	2004	No Comment	Possible transverse crack.
160+54	U/S slope	Transverse	2002	No change	Possible transverse crack on upstream slope at mid-height. Hole probed 12-inch.
169+75	Crest	Hole	2004	No Comment	3-inch diameter depression in the centerline of the crest.
175+55	Crest	Hole	2002	No change	Hole in dam crest over filter-drain. Hole probed 12-inch.
175+90	D/S slope	Transverse	2004	No Comment	Possible transverse crack (series of small holes).
179+60 to 185+20	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12-inch to 24-inch. 3-inch and 1-inch diameter depressions at 185+20 probed 6-inch.
197+75	D/S slope	Transverse	2002	No change	Possible transverse crack on downstream slope near the crest.
204+00	U/S slope	Transverse	2003	Did not find	Just south of south gate on Salome Road found a transverse crack on upstream slope that extends to dam crest.

January 13, 2004 Inspection Conducted by Michael Greenslade, PE (FCDMC) and reviewed by Brett Howey, P.E.
Contacts: John Chua (NRCS) and ADWR

Inspection Notes

Station	Location	Type	Date First Reported	2004 FY vs. First Report	2004 FY Inspection Results
210+13 to 256+52	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. Two depressions (one 6-inch diameter and one 12-inch diameter) at Sta. 212+17. 6-inch diameter hole probed 24-inch at Sta. 218+57. 6-inch diameter hole with visible longitudinal crack in the sidewall at Sta. 227+04. 6-inch diameter depression at Sta. 227+23. 12-inch diameter depression at Sta. 227+66. 6-inch diameter depressions at Sta. 235+52, 247+76, and 248+95. Numerous depressions in gravel road layer (applied to mitigate safety issues to personnel and vehicle traffic) between Sta. 250+00 and 255+00 where previous holes had been reported.
215+27	Crest	Transverse	2001	Change	Transverse crack on crest and did not find in 2003. 3-inch diameter hole observed in 2004 on the crest probed 12-inch and possible transverse crack on the upstream slope also observed.
256+00	D/S slope	Transverse	2003	No change	Possible transverse crack on downstream slope where irrigation outlet located. Crack extends from downstream slope to crest centerline. Did not see crack on upstream slope.
257+10	Crest	Hole	2003	No Comment	6-feet diameter hole on the crest probed 24-inch.
250+91 to 262+20	U/S toe	Longitudinal	2003	No Change	Longitudinal crack at upstream toe. Exact location varies from slightly up on slope to toe. Crack is not one continuous crack but may skip 10- to 15-feet lengths between these two stations. Increased in length from Sta. 261+75 to 262+20 between 2003 and 2004 inspections.
260+00	Crest	Transverse	2000	Did not find	Crack first reported in 2000 but have not found crack in subsequent inspections, including 2003.

Nov. 15, 2004 Inspection Conducted by Brett Howey, P.E. (FCDMC) and Reviewed by Michael Greenslade, P.E. (FCDMC)
Contacts: Michael Johnson, Ph.D., P.E. (ADWR) and Idle Chavez, P.E. (NRCS)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2005 FY vs. First Report**	**2005 FY Inspection Results**
General	Crest	Holes	(not noted)	No Comment	There are holes located on the dam crest along the full length of the dam. The holes are located over the filter drain and typically are a series of holes inch close proximity, holes vary inch diameter from ½-inch to 12-inch and a probe can be inserted to its handle or to a depth of 34-feet.
15+25 to 15+65	Crest	Longitudinal	2002	No Change	Series of holes along dam centerline found (max. 3-inch diameter). A 3-inch diameter hole was observed at Station 15+48 in 2003 and 2004. Another 3-inch diameter hole probed 24-inch was observed at Sta. 15+25 in 2004.
16+09	Crest and D/S slope	Transverse	2002	Did not find	Transverse crack in line or close to alignment of right side or principal spillway box outlet. Crack is wider towards downstream side of crest and becomes hairline about 5-feet down from crest. Station location in 2003 was 16+09. The original location was possibly in error. There are not two separate cracks at 15+90 and 16+09. The original location was noted as 15+90 in 2002.
16+45	Crest	Hole	2004	No Change	2-inch diameter hole probed 18-inch.
17+08	Crest	Hole	2004	No Change	Two 2-inch diameter hole probed 18-inch.
17+70	Crest	Longitudinal	2004	No Change	3-inch wide by 18-inch long probed 24-inch.
17+79	Crest	Hole	2004	No Change	1-inch diameter hole probed 24-inch.
19+65	Crest	Longitudinal	2004	Did not find	Series of 5 small holes probed 12-inch.
20+82	Crest	Longitudinal	2004	No Change	Series of 2 small holes probed 24-inch.
51+79	U/S & Crest	Hole	2001	Change	Possible transverse crack. Probed 24-inch in 2004. Only probed 34-inch in 2003.
52+45	U/S slope	Transverse	2003	Change	Transverse crack at upstream toe to about 5-feet up slope. Probed 34-inch in 2003 but only probed 6-inch in 2004.
53+40	U/S slope	Transverse	2003	Did not find	Transverse crack at upstream toe to about 10-feet up slope. Probed 34-inch in 2003.

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Contacts: Michael Johnson, Ph.D., P.E. (ADWR) and Idle Chavez, P.E. (NRCS)

Inspection Notes					
Station	Location	Type	Date First Reported	2005 FY vs. First Report	2005 FY Inspection Results
53+86	U/S slope	Transverse	2003	Did not find	Did not observe a transverse crack only rodent holes.
50+72 to 56+00	U/S slope	Longitudinal	2000	Did not find	Depression and cracking at upstream toe. Cracking, first identified in 2000 was repaired between 53+00 and 56+00 by backfilling with ASTM C33 sand. Additional intermittent cracking ¼- to ½-inch wide was observed between Sta. 50+72 and 54+00.
54+83	U/S slope	Transverse	2003	Did not find	Transverse crack at upstream toe to about 10-feet up slope. Probed 34-inch.
57+15 to 65+00	Crest	Longitudinal	2004	Change	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. An 18-inch diameter hole was observed at Sta. 57+15 (probed 30-inch below the depression of the hole). 12-inch diameter holes were observed at Sta. 57+37 and 58+46. Series of small holes observed at Sta. 58+80. Six 12-inch diameter depressions observed at Sta. 62+31.
57+35	D/S slope	Holes	2004	No Comment	Possible transverse crack with small holes observed 5-feet up from the toe and near the crest.
57+72	U/S slope	Transverse	2003	No Comment	Transverse crack at upstream toe and continues to within 15-feet of dam crest. Probed 34-inch.
60+50	Crest	Transverse	2004	Change	Small (1/8-inch wide) crack observed over the vegetative outlet at this location. Observed 3-inch diameter hole during 2004 inspection. Ordered the outlet closed as a precaution. Hole has increased to 8- x 6-inch.
64+67	Crest	Transverse	2003	Change	Transverse crack at upstream toe berm and at dam crest. No indication of crack along mid-height of slope. Appears to be a collapsed rodent burrow.
67+50 to 70+00	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. 6- by 12-inch holes were observed at Sta. 67+91 and 68+12. The hole at 68+12 was probed 4-feet.

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Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2005 FY vs. First Report**	**2005 FY Inspection Results**
72+14 to 84+76	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. 6- by 12-inch depression observed at Sta. 76+34. 12-inch diameter depression observed at Sta. 78+25. Two 1-inch diameter holes at 80+56. 18-inch diameter depression 6-inch deep observed at Sta. 80+93. 1-inch diameter hole at Sta. 83+28. 5-inch diameter hole at Sta. 84+12 probed 24-inch. 1-inch diameter hole at Sta. 84+76 probed 12-inch.
73+36	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope about 10-feet up from toe. Probed 24-inch.
75+51	D/S side Crest	Longitudinal	2002	No change	Longitudinal crack on downstream side of crest about 3-feet from crest. Crack about 45-feet long and probe inserted 24-inch.
82+94	U/S slope	Transverse	2004	No Comment	Possible transverse crack observed 5-feet up from the toe.
87+66	Crest	Hole	2004	No Comment	1-inch diameter hole probed 12-inch.
90+86	Crest	Hole	2004	No Comment	1-inch diameter hole probed 12-inch.
94+54	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope from toe to mid-height.
96+17	U/S slope	Transverse	2003	No change	Possible transverse crack about mid-height on upstream slope. Probe inserted about 12-inch in a series of four holes in line.
98+30	U/S slope	Transverse	2002	Change	Transverse crack on upstream slope from about 5-feet above toe to mid-height of dam. Probed 30-inch.
104+39	U/S slope	Transverse	2004	No Comment	Possible transverse crack indicated by hole midway up the slope probed 24-inch.
106+81	Crest	Longitudinal	2004	No Comment	Two 5-inch diameter holes about 3-inch apart in the centerline of the crest probed 18-inch.
109+73 to 117+35	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. 6-inch diameter depression 18-inch deep at Sta. 109+73.

Nov. 15, 2004 Inspection Conducted by Brett Howey, P.E. (FCDMC) and Reviewed by Michael Greenslade, P.E. (FCDMC)
Contacts: Michael Johnson, Ph.D., P.E. (ADWR) and Idle Chavez, P.E. (NRCS)

Inspection Notes					
Station	Location	Type	Date First Reported	2005 FY vs. First Report	2005 FY Inspection Results
119+90	Crest	Transverse	2002	Did not find	Transverse crack on dam crest from upstream side to centerline.
130+09	U/S slope	Transverse	2002	Did not find	Possible transverse crack on upstream slope at mid-height.
140+36	Crest	Hole	2004	No Comment	4-inch diameter depression probed 12-inch.
142+66 to 146+22	Crest	Longitudinal	2004	Change	Random holes along the centerline of the crest with some in series typically ranging from 1-inch to 2-inch in diameter and probed 12- to 24-inch.
150+61 to 160+96	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. Three 6-inch diameter holes over a 3-feet length at Sta. 150+84. Depression 1-feet wide and 12-inch long probed 12-inch at Sta. 154+07. Two 4-inch diameter depressions at 155+13.
156+34	U/S slope	Transverse	2004	No Comment	Possible transverse crack.
156+44	U/S slope	Transverse	2004	No Comment	Possible transverse crack.
160+54	U/S slope	Transverse	2002	No change	Possible transverse crack on upstream slope at mid-height. Hole probed 12-inch.
169+75	Crest	Hole	2004	No Comment	3-inch diameter depression in the centerline of the crest.
175+55	Crest	Hole	2002	Change	Hole in dam crest over filter-drain. Hole probed 12-inch.
175+90	D/S slope	Transverse	2004	No Comment	Possible transverse crack (series of small holes).
179+60 to 185+20	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. 3- and 1-inch diameter depressions at 185+20 probed 6-inch.

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Contacts: Michael Johnson, Ph.D., P.E. (ADWR) and Idle Chavez, P.E. (NRCS)

Inspection Notes					
Station	Location	Type	Date First Reported	2005 FY vs. First Report	2005 FY Inspection Results
197+75	D/S slope	Transverse	2002	No change	Possible transverse crack on downstream slope near the crest.
204+00	U/S slope	Transverse	2003	Did not find	Just south of south gate on Salome Road found a transverse crack on upstream slope that extends to dam crest.
210+13 to 256+52	Crest	Longitudinal	2004	No Comment	Random holes along the centerline of the crest with some in series typically ranging from 1- to 2-inch in diameter and probed 12- to 24-inch. Two depressions (one 6-inch diameter and one 12-inch diameter) at Sta. 212+17. 6-inch diameter hole probed 24-inch at Sta. 218+57. 6-inch diameter hole with visible longitudinal crack in the sidewall at Sta. 227+04. 6-inch diameter depression at Sta. 227+23. 12-inch diameter depression at Sta. 227+66. 6-inch diameter depressions at Sta. 235+52, 247+76, and 248+95. Numerous depressions in gravel road layer (applied to mitigate safety issues to personnel and vehicle traffic) between Sta. 250+00 and 255+00 where previous holes had been reported.
215+27	Crest	Transverse	2001	Did not find	Transverse crack on crest and did not find in 2003. 3-inch diameter hole observed in 2004 on the crest probed 12-inch and possible transverse crack on the upstream slope also observed.
256+00	D/S slope	Transverse	2003	No change	Possible transverse crack on downstream slope where irrigation outlet located. Crack extends from downstream slope to crest centerline. Did not see crack on upstream slope.
257+10	Crest	Hole	2003	No Comment	6-feet diameter hole on the crest probed 24-inch.
250+91 to 262+20	U/S toe	Longitudinal	2003	No Change	Longitudinal crack at upstream toe. Exact location varies from slightly up on slope to toe. Crack is not one continuous crack but may skip 10- to 15-feet lengths between these two stations. Increased in length from Sta. 261+75 to 262+20 between 2003 and 2004 inspections.
260+00	Crest	Transverse	2000	Did not find	Crack first reported in 2000 but have not found crack in subsequent inspections, including 2003.

FINAL

FAILURE MODE AND EFFECTS ANALYSIS
FOR
SADDLEBACK FLOOD RETARDING STRUCTURE
MARICOPA COUNTY, ARIZONA

Submitted to:

Kimley-Horn and Associates, Inc.
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Submitted by:

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Registered Professional Engineer (Civil) CERTIFICATE NO. 35554 DEAN B. DURKEE Date Signed 11-3-05 ARIZONA, U.S.A.

Registered Professional Engineer (Civil) CERTIFICATE NO. 38508 DEBORA J. MILLER Date Signed 11/15/05 ARIZONA, U.S.A.

Dean B. Durkee, Ph.D., P.E.
Project Manager

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FMEA Facilitator

November 2005

043394

1.0 INTRODUCTION

General Description

The Saddleback Flood Retarding Structure (FRS) is located in the Harquahala Valley of Maricopa County, approximately 10 miles west of Tonopah, Arizona and immediately west of Saddleback Mountain. Saddleback FRS consists of a 21-feet high, 4.6-mile long homogeneous earth embankment dam, with a vertical central filter/drain and six-inch drain outlets, on 400-feet intervals. The principal outlet is a 10-foot wide by 8-foot high reinforced concrete box culvert located near the left abutment. There are four, 12-inch diameter vegetative maintenance outlets located at Station 60+50 (gated), Station 103+70 (ungated), Station 124+10 (ungated), and Station 256+00 (gated). The FRS was designed not to have a permanent storage pool.

Saddleback FRS was designed with an NRCS hazard classification of "b" but is designated as an "a". The current ADWR hazard potential classification is significant and size of the dam is intermediate. The reservoir surface area behind the dam is 760 acres with an impoundment capacity of 3620 acre-feet at the principal outlet elevation. Design of Saddleback FRS was completed by the Natural Resources Conservation Service (NRCS), formerly Soil Conservation Service (SCS). Construction of the FRS and appurtenant structures was completed in 1982 by M&B Contracting Corporation.

The FRS has performed satisfactorily to date and has experienced impoundments of various depths since construction. The maximum gage depth of impoundment was 2.5 feet recorded in 1996.

Dam Data

- Dam type: Homogeneous Compacted Earthfill
- Dam height: 21 feet
- Dam length: 24,270 feet
- Dam crest: width: 12 feet
- Dam crest elevation: 1193.00 feet (Sta. 3+00 to 49+00); 1193.33 (Sta. 49+05 to 72+95); 1193.00 (Sta. 73+00 to Sta. 171+45); and 1194.00 (Sta. 171+50 to Sta. 272+70). One foot of material was added to the crest elevation as a driving surface in 1995.
- Spillways: Principal - 10-foot wide by 8-foot high reinforced concrete box culvert, inlet invert elevation of 1176.9 feet. (There is no emergency spillway)
- Freeboard: 1.5 feet on the 100-year event; 0 feet on the PMF
- Reservoir Surface: 760 acres
- Storage: 3620 acre-feet
- Hazard Classification: Significant

Hydrology Data (elevations in NGVD 1929 datum)

- Probable Maximum Precipitation = 15.66 inches (Reidel & Hansen: original design method); 15.4 inches (HMR 49 – 72 hour); 10.1 inches (HMR 49 - 6-hour)
- 100-year 24-hour = 4.03 inches and 100-year 10-day = 5.59 inches
- PMF Inflow Estimate: 6-hour PMF 63,083 cfs; 72-hour PMF 23,766 cfs (Carter & Burgess Dambreak report)
- ½ PMF Inflow Estimate: 6-hour 31,541 cfs; 72-hour 11,883 (Carter & Burgess Dambreak report)

- ½ PMF Outflow Estimate: 6-hour 1175 cfs at 1190.83 ft (1.2 ft freeboard); 72-hour 1250 cfs at 1192.07 feet (0.93 ft freeboard) (Carter & Burgess Dambreak report)
- Principal Spillway capacity: 1120 cfs
- Reservoir Volume: 3620 acre feet; flood retarding volume: 3500 acre feet; sediment volume (50 year): 120 acre feet
- Drawdown flood pool: less than 6 days

Purpose and Scope

In general, the purpose of the Failure Mode and Effects Analysis (FMEA) exercise was to:

- Identify potential site-specific failure modes for the dam.
- Discuss qualitatively the likelihood of the occurrence of failure modes.
- Determine whether or not, and how, important failure mechanisms are being monitored.
- Examine the potential consequences of failure and the adverse consequences of successful operation during flood loading (e.g. – large spillway releases).
- Identify possible risk reduction actions that may be taken to reduce the likelihood of failure or to mitigate adverse consequences.
- Determine what information, investigations or analyses may be needed to resolve uncertainties related to potential failure modes.

In this phase, the FMEA team only examined the general nature of the "consequences" for the failure modes identified, and where appropriate, estimated how these may be different than previously anticipated. Greater detail on the estimate of the magnitude of the "consequences" for each significant failure mode may be addressed in the quantitative portion (risk analysis part) of the risk assessment for the dam at some future time.

Team Members

Tom Renckly, P.E., Flood Control District of Maricopa County, Project Manager,
Brett Howey, P.E., Flood Control District of Maricopa County, Dam Safety Engineer
Bob Eichinger, P.E., CFM, Kimley-Horn and Associates, Inc., Project Manager
David Jensen, P.E. Kimley-Horn and Associates, Inc, Hydrology & Session Recorder
Debora J. Miller, Ph.D., P.E., Gannett Fleming, Inc. FMEA Facilitator
Dean B. Durkee, Ph.D., P.E., Gannett Fleming, Inc. Geotechnical
Ken Euge, R.G., Geological Consultants, Geology
John Harrington, P.E., Natural Resources Conservation Service

2.0 Major Findings And Understandings Gained

The following is a summary of the major findings and understandings for Saddleback FRS as a result of the Failure Mode and Effects Analysis (FMEA). Saddleback FRS is one of two dams, in addition to a levee, located in relative proximity of each other in the Harquahala Valley of Arizona (the other two structures are Harquahala FRS and Centennial Levee).

The major findings and understandings given below are organized as follows. First the important geotechnical, geologic, design, construction, and performance differences or unique aspects related to the potential for failure mode development of Saddleback FRS are listed. Findings related to failure modes or adverse consequences for overtopping and spillway discharge are given next, followed by findings related to consequences, and general findings that are informational and/or generally similar in nature. Finally, a list of potential action items (risk reduction and investigations) is provided based on the team's recommendations.

Key Findings/Differences Related To Failure Mode Development – "Static Loading Failures – Seepage Erosion – Fissuring – Foundation Erosion –Etc."

1) The Dam was Designed and Constructed Considering Previous Experience Relating to Settlement and Cracking. The original design included a central filter/drain system, cutoff, and stripping/foundation preparation in anticipation of problems experienced previously in the arid southwest. The approach showed an understanding of the state-of-practice and the types of problems that would be encountered.

2) Stability Analyses Exceed TR-60 Requirements. Slope stability factors of safety were calculated for all loading conditions, except earthquake loading, required by SCS. The calculated factors of safety were above the minimums set by SCS.

3) The Filter Zone Extends to the Bottom of the Over-Excavation in Some Areas and There is a Protective Berm in These Areas. The as-built plans indicate that the filter zone extends to the limit of foundation excavation at three locations (Station 46+00 to Station 76+00, Station 104+00 to Station 114+00, and Station 238+20 to Station 265+70). In addition, stability berms were constructed along two upstream sections at locations that coincide with two of the three filter/drain extension locations (Station 46+00 to Station 76+00 and from Station 240+00 to Station 265+00).

4) Surface Expression of Potential Transverse Cracks is Less than on other District Structures. Inspection of several flood retarding structures in Maricopa County have been performed by the FMEA team. During inspection of the Saddleback FRS, the team did not observe as much surface expression of cracking as has been observed on other similar structures.

5) The Cause of Longitudinal Cracks and Voids on the Crest is not well Understood. The inspection team and FMEA team identified a number of possible causes for the longitudinal cracks observed on the crest of the dam, including settlement of the upstream section of the embankment due to collapsible soils in the foundation, wetted upon impoundment; settlement of the drain itself due to improper or poor compaction at installation; suffusion, or internal migration of fines through broadly graded cohesionless drain rock; and/or piping of fine-grained embankment soils by percolating water during precipitation events.

6) Additional Seismic Design Analysis is not Required. The slope stability analyses documentation indicated that additional analyses to assess stability under earthquake loading conditions was not necessary. It was reported that an acceptable factor of safety under earthquake loading would be achieved for a slope having a static factor of safety greater than 1.5 using the recommended horizontal force of 0.1 times the weight of the failure slice. Because the calculated static factor of safety was greater than 1.5, the designers did not perform a seismic slope stability analysis. In addition, the low height of the dam and low seismicity in the area results in a low impact/effect on the dam.

7) Longitudinal Cracks have been Observed, Investigated, and Repaired within the Basin along the Upstream Toe of the FRS. AMEC Earth & Environmental investigated longitudinal cracking between Sta. 33+00 and Sta. 125+00 (Embankment Stationing). They concluded that the observed longitudinal cracks were due to collapse settlement within the basin near-surface soils. The horizontal strains developed due to collapse settlement, and exceeded

the threshold for soils of this type, resulting in cracking. AMEC further recommended mitigation of existing cracks by backfilling with ASTM C-33 sand, compacted with a vibratory compactor. The intent is for the sand to limit seepage and decrease internal erosion in the cracks. The embankment itself was considered to be stable.

8) Specifications to keep Drain Fill 1 foot above Embankment Fill were not followed during Construction. There are photos of construction available and those were reviewed by the FMEA Team. In general the photos indicated that drain fill was not placed in accordance with the specifications. This likely resulted in contamination of and varying width of the drain fill.

9) Southern Half of Structure located on Shallow Soil, Northern Half on Deeper Unconsolidated Soil.

10) Potential for Land Subsidence and Earth Fissuring have been Documented Regionally. There has been no evidence in the survey data, to date, to suggest that subsidence is a problem. Given the alignment of the dam, the foundation conditions, and the recorded occurrence of subsidence and earth fissuring in the area, it is likely that the northern half of the dam would be more susceptible to potential earth fissuring.

11) The 1996 Repair Report Indicates there is as much as 5 feet of Cover above the Drain Fill. As-built drawings included in the January 16, 1996 letter show as much as 5 feet of embankment material above the drain. This material was removed and replaced with granular material up to 1 foot below the crest between Sta. 45+00 and 52+00.

12) Potential Filter Defects were identified. Based on observations from the construction photos, it is possible that defects relating to filter compatibility exist between the embankment material and the drain fill, and this may be a possible cause of longitudinal cracking observed on the crest.

13) There are Anti-Seepage Collars on all Embankment Penetrations. According to the as-built plan sets, the principle spillway and other outlets were constructed with anti-seep collars. Performance issues have been associated with anti-seep collars, notably difficulty in compaction around the collars resulting in preferential flow paths around the collars and the pipes. As such, anti-seepage collars are no longer generally accepted in standard practice.

14) A Formal Crack Investigation has not been Performed. AMEC Earth & Environmental conducted an investigation of longitudinal cracking that was observed between Sta. 30+00 and Sta. 103+00 and between Sta. 250+00 and Sta. 275+00. That investigation focused on cracking within the basin and did not include embankment cracking, in particular the suspected longitudinal cracking associated with observed "jug" holes along the crest on the south end of the embankment. An embankment wide comprehensive crack investigation should be performed.

15) Numerous Cross Channels were Over-Excavated to Prepare the Foundation.

Key Findings/Differences Related To Failure Mode Development – "Flooding – Overtopping – Spillway Discharges – Etc."

16) Crest Elevation Varies. The dam crest elevation varies as follow; 1193.00 feet (Sta. 3+00 to 49+00); 1193.33 (Sta. 49+05 to 72+95); 1193.00 (Sta. 73+00 to Sta. 171+45); and 1194.00

(Sta. 171+50 to Sta. 272+70). The dam crest elevation varies by design. The dam crest elevation is 1194.0 feet (as-built plans) from the right end of the dam through the Solomon Road crossing. The dam crest elevation is 1193.33 feet (as-built plans) at the large wash coming into Basin 1 which is 0.33 feet above the maximum water surface elevation shown on the as-built plans. This section of the dam may be higher to provide freeboard at the wash location. The uneven dam crest profile allows for determination of the location of where potential overtopping may occur first. These are the sections of the dam where the crest elevation is 1193.0 feet. In addition, the dam crest is higher than design; one foot of material was added to the crest elevation as a driving surface in 1995.

17) Hydrologic Assumptions Related to Interstate-10 (I-10) and the Granite Reef Aqueduct (GRA : also known as the Central Arizona Project canal) are not Well Understood and Require Re-Evaluation with Emphasis on Contributing Drainage Areas. The Harquahala Valley is protected by a "series" of NRCS designed dams and levees (Harquahala FRS, Saddleback FRS, and Centennial Levee). The embankment dike located on the upstream side of the Central Arizona Canal and the canal both provide a measure of flood protection to the Harquahala Valley as well as the Saddleback FRS. The original SCS hydrologic analyses, for Saddleback FRS indicates that the analysis accounted for the CAP embankment and canal floodwater overchutes. The analysis was unclear however in regards to the actual contributing drainage area upstream of the CAP canal that contributes floodwaters to Saddleback FRS. Kimley-Horn recommends that the contributing drainage area upstream of the CAP canal be re-evaluate and verified. Interstate 10 is located north of Saddleback FRS. The roadway embankment is a hydrologic control such that stormwater from the upstream contributing area are ponded against the upstream side of the embankment. The hydrologic relationship of the CAP embankment, CAP canal (available freeboard), and the I-10 embankment and cross culverts should be evaluated in a future District study.

18) The Design of the Saddleback FRS Recommended that the United States Bureau of Reclamation Raise the Granite Reef Aqueduct Dike (CAP Canal Dike) by 1 Foot to Accommodate the Freeboard Hydrograph. The original hydrologic analysis conducted by the NRCS for Saddleback investigated the freeboard hydrology for a Class B dam. The analysis included the watershed area upstream of the CAP canal dike. The analysis indicated that additional flood protection benefits could be afforded to the Saddleback FRS project if the CAP canal flood protection dike was raised by 1-foot.

19) No Overtopping for During the ½ PMF Event with Granite Reef Aqueduct Assumptions. The original hydrologic analysis and the hydrology analysis conducted for the dambreak study indicates that Saddleback FRS will not be overtopped for the ½ PMF event. The dambreak study did not conduct an overtopping analysis for the PMF event. An overtopping analysis for the PMF event using the dambreak hydrology model could be accomplished with minimal effort.

20) The 100-yr Design Storm Water Surface Elevation is within 4 Feet of the Dam Crest. The original NRCS hydrology analysis routed the 100-year storm event through the dam and reservoir. The results indicate that the maximum water surface elevation for the 100-year storm event is 1189.0 ft (NGVD29) which is four feet below dam crest elevation.

21) The Full PMF Has Not Been Routed Through the Reservoir. The NRCS designed Saddleback FRS as a Class "b" dam. The freeboard hydrograph for a Class "b" dam is a function of the 100-year storm event and the PMF event. The Class "b" freeboard

hydrograph does not represent the PMF event. The Carter & Burgess dambreak study computed the HMR-49 PMP and developed a HEC-1 hydrology model of determine the PMF from the PMP. However, the Carter & Burgess study did not route the PMF through the dam. The ½ PMF was routed through the reservoir. See Item 19 above.

22) The District has Land Rights above the Top of Dam. The as-built plans for Saddleback FRS indicate that the District has land rights above the top of dam and maximum pool elevation. This may allow for a dam raise in the future if needed.

23) Bedrock Outcrop at Station 110+00 may lead to Differential Settlement. A bedrock highpoint at Station 110+00 is noted in the as-built plans as a fanglomerate material._The alignment of the dam at this location is founded on an infilled saddle between two exposed bedrock formations. The profile of the bedrock under this portion of the dam falls off fairly quickly. This location is an area of the dam to monitor for differential settlement on either side of the bedrock high and perhaps an area to monitor for potential transverse cracking. The as-built plans indicate that the bedrock was treated with dental grout to provide a uniform bearing surface and fill in cracks and fissures within the bedrock.

24) No Failure Modes were Identified at the Principal Outlet. The principal outlet is a large, massive reinforced concrete box culvert. Drain fill material was provided as a filter diaphragm around the culvert as well as a piped drain system. The principal outlet is founded on native material. As a result of the massive concrete structure, foundation on native material, and drain fill, no failure mode was identified for the outlet.

25) There is Only One Outlet to Drawdown the Reservoir. Saddleback FRS was designed and constructed with the principal outlet as the only primary facility to evacuate the flood storage pool. There is no emergency spillway for this dam. The freeboard hydrograph is contained within the reservoir; therefore, the designers did not include an emergency spillway. There are four vegetative maintenance outlets at the dam. These could be used to assist in evacuating a large flood pool.

26) Dambreak Parameters need to be Re-evaluated. The failure mode modeled in the Carter & Burgess dambreak study was piping failure under the "sunny-day" scenario. The failure mode chosen as the inflow design flood (IDF), the ½ PMF, does not overtop the dam. The dam breach parameters used in the study appear to be too conservative. The time to failure and final breach bottom width are 5.4 hours and 198 feet, respectively for the 72-hr PMF for the north breach location. The time to breach, based on case history, is on the order of 20 to 40 minutes.

27) Principal Outlet does not have a Trash Rack. The box culvert opening is large – a single barrel 8-ft rise by 10-ft span which will pass rather large debris, shrubs, and trees. A review of watershed vegetation, however, includes mature trees such as mesquite, palo verde, and ironwood. The need for a trash rack should be evaluated.

28) District has updated the Stage-Storage Discharge Curve for Saddleback FRS. The new rating curve was prepared using new survey/topographic contour mapping for the dam. The new rating curves should be input into the hydrologic models to evaluate the impact on routing of the design and inflow design floods.

29) Hydrologic Routing for the Dambreak Study was Started with an Empty Reservoir. This is consistent with ADWR guidelines for dry flood control dams for routing the inflow design flood.

Consequence Evaluation

30) No Visible Warning to a Dambreak. There are no conventional emergency spillway discharges, and the principal outlet discharges away from the dambreak inundation area. The classical dam, reservoir, and spillway configuration is such that the principal outlet is located within the dam embankment and discharges into a downstream channel from the dam. Saddleback FRS has only one spillway, the principal spillway located at the left abutment. The spillway discharges into the Saddleback Diversion Channel which routes flows to Centennial Wash. The diversion channel is not located in the downstream inundation area of the dambreak. Under the classic example, flows in the downstream channel would indicate to the population downstream of the dam that the dam is operating. With Saddleback, no such indication is provided unless the District or McDEM provides information that the dam is operating.

31) EAP not updated to Current ADWR Standards. The current emergency action plan does not meet current ADWR standards or FEMA 64 guidelines. The District however is in the process of updating all EAPs for all of their dams.

32) The Dam is a Class "a" but was Designed to Class "b" Standards. Saddleback FRS was classified by NRCS as a Class "a" dam but was designed to Class "b" standards as a consideration for downstream future development.

33) There is No Clear Evidence the Dam has gone to High Hazard. The downstream inundation area is primarily cultivated agriculture. A power generation facility is also located downstream. There are none to a few habitable structures.

Hydrology is based on Existing Land Use. The hydrologic evaluation for the upstream contributing watershed was based on land use existing at the time of analysis. This approach appears to be valid since upstream development at the time of this report has been miminal. However, given the explosive growth in the west valley, a future conditions land use hydrologic model may be warranted.

General Findings

34) Evidence suggests the dam is performing better than previously constructed structures.

35) Only one Category I Potential Failure Mode was identified.

36) Three Category II Failure Modes were identified.

37) The principal outlet flowed at 750 cfs in 1984, resulting in 6 feet of head (25% of impoundment capacity).

38) Dam rehabilitation is a viable alternative.

39) Structure could easily be converted to a levee floodway system.

40) Final design Documentation is missing.

41) WPP originally designed dam as a diversion. A Design Supplement was issued to change the design to a dam.

42) FMEA recommendations are important to the District Dam Safety Program.

Action Items – Risk Reduction Measures or Investigations

1) Identify and Quantify the Existence of Transverse Cracks.
2) Determine the Cause of Crest Holes.
3) Dynamic Routing is recommended due to Dam Length and Geometry.
4) Continue Monitoring of Drain Outlets where Silt has been and continues to be Observed.
5) Verify Utility Relocations, Add Fiber Optic Line to the Plans, and Locate all Drain Outlets on the Plans and Replace if not found.
6) Evaluate Drain Outlet Video Tape.
7) Evaluate Hydrologic Routing with Salome Road Culverts Plugged.
8) Perform Multi-Frequency Analysis to Determine Incipient Overtopping.
9) Locate utility as-built files.
10) Review AMEC report on longitudinal cracking. Evaluate failure mode conclusions and monitoring recommendations and comment as necessary.
11) Maintain annual surveys in the area due to potential fissure risks.
12) Develop IGA with ADWR on fissure monitoring and include Saddleback FRS in the study to develop baseline InSAR data.
13) Regular inspections in the vicinity of the Harquahala Floodway discharge into the Saddleback FRS and at the roadside drainage next to Courthouse Road are recommended.
14) Review existing instrumentation (rainfall and streamflow gages) and recommend changes and modifications if necessary.

3.0 POTENTIAL FAILURE MODES

Potential failure modes identified by the FMEA team are presented below. The failure modes were placed into one of four categories as follows.

Category I – Highlighted Potential Failure Modes: Those potential failure modes of greatest significance considering need for awareness, potential for occurrence, magnitude of consequence and likelihood of adverse response (physical possibility is evident, fundamental flaw or weakness is identified and conditions and events leading to failure seemed reasonable and credible) are highlighted.

Category II – Potential Failure Modes Considered but not Highlighted: These are judged to be of lesser significance and likelihood. Note that even though these potential failure modes are considered less significant than Category I they are all also described and included with reasons for and against the occurrence of the potential failure mode. The reason for the lesser significance is noted and summarized in the documentation report or notes.

Category III – More Information or Analyses are Needed in order to Classify: These potential failure modes to some degree lacked information to allow a confident judgment of significance and thus a dam safety investigative action or analyses can be recommended. Because action is required before resolution the need for this action may also be highlighted.

Category IV – Potential Failure Mode Ruled Out: Potential failure modes may be ruled out because the physical possibility does not exist, information came to light which eliminated the concern that had generated the development of the potential failure mode, or the potential failure mode is clearly so remote as to be non-credible or not reasonable to postulate.

For each of the potential failure modes identified, a failure mode description is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur are listed following the failure mode description. In addition, any identified potential actions for risk reduction for each potential failure are then provided.

CATEGORY I – HIGHLIGHTED POTENTIAL FAILURE MODES

H1. Overtopping During Major Flood Event (Category I).

Failure Mode Description: Saddleback FRS does not have an emergency spillway. Flows up to the ½ PMF are routed through the principal outlet to the Saddleback Diversion. The maximum water surface elevation for the ½ PMF is 1193.0. The top of dam is 1193.0. It is likely that the PMF would overtop the FRS. A PMF storm event on the watershed may possibly fail the CAP upstream dike increasing the overtopping potential at Saddleback FRS.

Adverse Factors:

(1) Earthen embankment can not withstand significant depths of overtopping flow.
(2) Failure of the CAP canal dike during the PMF would contribute additional reservoir inflow.
(3) Downstream slope does not have erosion protection.
(4) Large watershed may result in broad hydrographs and long overtopping periods.
(5) Crest is not level and potential ground subsidence could lower the crest.
(6) There is currently a power plant located downstream and there is potential for more downstream development in the future.

(7) There is vegetation in the storage basins and in the low flow channel.
(8) Salome Road may produce a backwater effect.
(9) Principal outlet is susceptible (however minor) to clogging with debris, (no trash rack).
(10) No emergency spillway.
(11) Presence of voids in the crest could exacerbate or accelerate erosion during overtopping.
(12) Multiple storms could lead to overtopping.

Positive Factors:
(1) ½-PMF does not overtop the dam.
(2) Presence of the CAP embankment dike provides significant protection.
(3) Dam crest was plated with gravel in 1996.
(4) CAP can control flows at the Burnt Mountain tunnel.
(5) The I-10 embankment will attenuate flows.
(6) Downstream failure impacts are low.
(7) Available freeboard in the CAP canal can provide some storage capacity if upstream embankment of CAP embankment was overtopped or breached.
(8) Dam height is low over the majority of the length (limited breach depth) 80% of length.
(9) Principal outlet discharges into the Saddleback Diversion which is five miles long and which discharges into Centennial Wash.
(10) A breach on the northern section of the dam could be partially contained in the existing irrigation canal.
(11) The gravel surface on the downstream slope could improve stability during overtopping.
(12) The PMF is an extremely rare event.

Potential Actions for Risk Reduction (Potential Failure Mode H1):
(1) Check survey data to verify the elevation of the FRS at the Salome Road crossing.
(2) Plot actual crest elevation in addition to monument elevations.
(3) Conduct a more thorough subsidence study
(4) Confirm base flow contributions from Harquahala FRS and CAP over-chutes for 100-year and PMF events.
(5) Review drainage area assumptions in previous analyses, including:
  - Inflow from Harquahala dambreak analysis,
  - Impacts of I-10
(6) Evaluate CAP canal embankment for storms greater than 100-year.
(7) Level crest along entire length of FRS, particularly at the Salome Road crossing.
(8) Perform dynamic flood routing for the ½ PMF and PMF events.
(9) Consider modifying the FRS to convert it to a floodway/levee system and or segmenting the FRS.
(10) Evaluate the hydraulics of the principal outlet (tail water depth effects).
(11) Perform a structural analysis of the principal outlet.
(12) Verify that the CAP canal embankment was raised 1 foot for freeboard hydrograph.
(13) Verify Hazard Classification of the FRS. There is significant potential for a higher hazard classification and the need for an emergency spillway.

Other Considerations (Potential Failure Mode H1):

(1) There is significant potential for a High hazard classification in the future and the need for an emergency spillway.

CATEGORY II – POTENTIAL FAILURE MODES CONSIDERED BUT NOT HIGHLIGHTED

S1a. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks under the Filter (Category II).

Failure Mode Description: A transverse crack extends through the embankment and into the foundation soils. In areas where the filter was not extended through the embankment and into the foundation, the crack could conceivably extend beyond the filter in which case this lower "unprotected" section of the embankment is susceptible to internal erosion during impoundment events. Sustained or intermittent flows through the crack below the filter could initiate the process of internal erosion and, over time, could result in widening of the crack and subsequent settlement of filter and embankment material into the void. As this process continues and the widened crack continues to migrate upward under sustained reservoir head, a breach of the embankment is conceivable.

Adverse Factors:

(1) Transverse cracks have been identified in previous inspection reports outside of the area where the filter is fully penetrating to the foundation.
(2) The extent (frequency and depth) of cracking has not been determined.
(3) The potential for differential settlement upon saturation of weekly cemented soils adjacent to the conglomerate bedrock outcrop (Sta. 104+00 to Sta. 114+00) has not been tested.
(4) The embankment could be in a dynamic drying mode.
(5) Future subsidence is possible.

Positive Factors:

(1) The dam was constructed with a central filter (crack stopper) which could collapse into a developing erosion void.
(2) Compared to other local (District) dams, this structure is relatively free of cracks.
(3) The dam is founded on a firm base of over-excavated and compacted soil.
(4) The highest incidence of potential transverse cracking is within the area where the filter was extended into the foundation.
(5) There has not been widespread evidence of cracking/erosion holes documented outside of the areas where the filter was extended into the foundation.
(6) There is no evidence of longitudinal cracks within the embankment slopes.
(7) The embankment soils may not be particularly erodible, but variable (plasticity concentration).
(8) The dam has a relatively short impoundment time.
(9) Transverse cracks deep in the section are less likely.
(10) Dam has impounded water to Elev. 1185 (6 feet above the P.O.).
(11) Through-going transverse cracks have not been observed/verified.
(12) Saddleback FRS is a low-head structure.

S1b. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks that extend above the Filter (Category II).

Failure Mode Description: A transverse crack extends into the embankment some nominal depth. The top of the filter terminates at Elev. 1190.0 feet and the crest of the dam varies between Elev. 1193.0 and 1194.0 which results in 3 to 4 feet of "unprotected" embankment above the filter which is susceptible to internal erosion during impoundment events. Sustained or intermittent flows through the crack could initiate the process of internal erosion, and, over time, could result in widening and deepening of the crack. As this process continues and the widened crack continues to migrate downward under sustained reservoir head, a breach of the embankment is conceivable.

Adverse Factors:

(1) There is potential for more cracks and wider cracks near the top of the embankment where the tensile strain is the greatest.
(2) The top of the embankment is the narrowest section of the dam.
(3) The embankment is in the poorest condition in the upper few feet as evidenced by observed longitudinal cracks/erosion holes.
(4) The top of the filter is likely to clog or a filter cake will develop (if designed properly) as the crack erodes.
(5) Longitudinal cracking at the crest provided connectivity, exacerbating the problem.
(6) The structure has not been tested by a significant impoundment event of sufficient height and duration.
(7) Potential for transverse cracks deep in the section implies that cracks extending to the surface are more likely.
(8) Potential transverse cracks have been identified in previous inspection reports.
(9) The extent (frequency and depth) of cracking has not been determined.
(10) The potential for differential settlement upon saturation of weekly cemented soils adjacent to the conglomerate bedrock outcrop (Sta. 104+00 to Sta. 114+00) has not been tested.
(11) The embankment could be in a dynamic drying mode.
(12) Future subsidence is possible.

Positive Factors:

(1) There is a short impoundment time between the top of the dam and the top of the filter (less than 2 days).
(2) The only time the reservoir level is higher than the top of the filter elevation storm event is above a 100-year return frequency.
(3) The drain will convey seepage from cracks.
(4) Mitigation of cracking and the filter can be done simultaneously.
(5) Compared to other local (District) dams, this structure is relatively free of cracks.
(6) The dam is founded on a firm base of over-excavated and compacted soil.
(7) There is not evidence of widespread cracking or erosion holes in this embankment.
(8) There is no evidence of longitudinal cracks within the embankment slopes.
(9) The embankment soils may not be particularly erodible, but variable (plasticity concentration).
(10) Through-going transverse cracks have not been observed/verified.
(11) Saddleback FRS is a low-head structure and under the conditions of this failure mode there is very low head and gradient (3 to 5 feet).

S1c. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks that extend through the Filter (Category II).

Failure Mode Description: A transverse crack extends from the crest of the embankment downward and into the embankment and fully through the filter in the transverse direction. During impoundment, flow develops through the transverse crack and initiates the process of internal erosion of upstream embankment material which can then be transported through the crack in the filter. Assuming the crack in the filter is wide enough and not "self-healing" this process could result in widening and deepening of the crack both in the embankment (upstream and downstream sections) and in the filter itself. As this process continues and the widened crack continues to migrate downward under sustained reservoir head, a breach of the embankment is conceivable.

Adverse Factors:

(1) The full 5-foot width of filter may not be effective due to contamination along the edges upon placement during construction (evidenced in the construction photos).

(2) Filter compatibility for the entire 4.6-mile dam is in question.

(3) The potential exists for cementation to develop within the filter due to downward movement water and subsequent upward movement of water vapor through cracks and erosion holes observed on the dam crest. This is exacerbated by the possible presence of $CaCO_3$ in the pore water.

(4) Water flowing through voids and cracks can transport fines into the filter which subsequently makes the filter more susceptible to shrinkage cracking.

Positive Factors:

(1) The dam was constructed with a 5-foot filter.

(2) Drain outlets were placed every 400 feet.

(3) This potential failure mode implies that the transverse crack in the embankment is aligned with the crack in the filter.

(4) The construction method minimized segregation of the filter material during placement.

(5) The compaction effort on the broad (5 feet wide) filter minimizes the potential for arching.

(6) The structure has a relatively short impoundment time.

(7) The character and extent of transverse cracking has not been identified.

(8) The filter was constructed continuously (though at variable depth and thickness) along the entire length of the dam.

(9) A full height filter was constructed in previously identified potential problem areas.

Potential Actions for Risk Reduction (Potential Failure Modes S1a-c):

(1) Check the filter compatibility between filter and the embankment.

(2) Conduct a Phase II crack investigation.

(3) Conduct studies to develop a better understanding of cracking mechanisms on the crest.

(4) Consider adding gravel mulch to reduce drying and provide additional overtopping protection.

Other Considerations (Potential Failure Modes S1a-c):

(1) The FMEA team listed these failure modes are Category II with a strong recommendation to investigate them further.

(2) The FMEA team believes these failure modes will not go to a Category IV.

S3. Internal Erosion Leading to a Breach in the Upper Embankment due to the Eroded Character of the Crest (Category II).

Failure Mode Description: During impoundment, water begins to saturate the upstream embankment soils and migrate through the upstream embankment soils and/or transverse cracks on the slope. The erosion holes (closely aligned), and/or longitudinal cracks that have been observed and documented along the centerline of the crest of the embankment, intersect the flow and provide a conduit to initiate internal erosion. There is uncertainty regarding the connectivity and network of existing erosion beyond that which is easily observed at the crest. However, any connectivity from the observed erosion features, particularly toward the downstream section of the embankment, presents the possibility of eventual seepage on the downstream face, internal erosion through the seepage features, and ultimate failure of the dam by breaching during sustained impoundment events.

Adverse Factors:
(1) There is evidence of large voids/longitudinal cracking in the crest over substantial length of the dam.
(2) If the void formation is due to a lack of filter compatibility between the filter and the embankment soil a major flood event will accelerate the process.
(3) The potential for transverse cracks exacerbates this failure mode.
(4) There is a higher concentration of erosion holes in the areas where the filter was extended the full depth.

Positive Factors:
(1) Most voids are present only on/near the centerline and may not be present upstream and downstream from the filter.
(2) The erosion voids/longitudinal cracks have only been observed on the dam crest and not on the slopes.
(3) A large storm event (100-yearr or larger) is necessary to initiate this type of failure mode.

Other Considerations (Relating to Failure Mode S3):
This potential failure mode was identified based on observed erosion features that appear to be associated with a defect of the vertical central filter and/or adjacent embankment zones. Downward seepage during precipitation events has led to erosion of crest material into voids that are present under the central portion of the dam crest. This process has resulted in a zone of porous, weakened material in the upper 3 to 4 feet of the crest. The FMEA team generally agreed that without further investigation the real cause of the observed distress could not be determined. A short brainstorming session identified a number of possible causes and mechanisms for the observed erosion holes along the crest. These included:

- Suffusion of embankment materials into the filter due to non-compatible filter criteria.
- Settlement of the filter.
- Settlement of the embankment adjacent to the filter.
- Rotation of the upstream zone due to collapse of alluvial sediments, particularly in the area of buried paleo-channel areas where tension cracks have been observed on the upstream slope. The rotation of the upstream zone is manifested as a tensile separation between the upstream embankment and vertical filter/drain zones.
- Settlement of the upstream fill due to saturation on inundation.

CATEGORY III – MORE INFORMATION OR ANALYSES ARE NEEDED IN ORDER TO CLASSIFY

S2a. Potential Failure due to Internal Erosion Associated with the Principal Outlet and Irrigation Penetrations through the Dam (Category III).

Failure Mode Description: The principal spillway is a concrete box structure, and the irrigation penetrations are steel pipes. Both types of penetrations have relatively smooth sides adjacent to the compacted backfill soil. Potential failure modes associated with penetrations through dams are typically initiated at the smooth conduit surface-soil interfaces where compaction problems are prevalent. During impoundment events, flow is initiated along the interfaces, resulting in internal erosion which progresses back from the downstream end of the pipes toward the upstream end of the penetrating structures. As soil is eroded, a large void can develop that can cause settlement and cracking of the embankment or in the worst case a loss of the majority of the impounded water through a tunnel breach.

Adverse Factors:
(1) The principal outlet and irrigation outlets have anti-seep collars which have been shown to inhibit compaction around the outlet pipe.
(2) Longitudinal cracking on the dam may result in opening of pipe joints.
(3) Irrigation outlets are on fill so there is greater risk of settlement and joint separation.
(4) There is potential for filter defects in the vicinity of the principal outlet (erosion holes have been observed on the crest of the dam).

Positive Factors:
(1) The filter encompasses the principal outlet and the irrigation outlets.
(2) The principal outlet can be visually inspected.
(3) The irrigation outlets have been video inspected.
(4) The shape of the principal outlet (rectangular) is more conducive to good compaction adjacent to side walls, compared to under the haunches of a pipe.
(5) The irrigation outlets are encased in concrete.
(6) Saddleback FRS is a low dam and the foundation is firm, resulting is less risk of lateral spreading under differential settlement.
(7) The principal outlet is founded on firm native cemented soil.
(8) Irrigation outlets are encased in a filter on the downstream section of the dam.

Potential Actions for Risk Reduction (Potential Failure Mode S2a):
(1) Review the video inspections of the principal outlet and irrigation outlets.

CATEGORY IV – POTENTIAL FAILURE MODE RULED OUT

S2B. Potential Failure due to Internal Erosion Associated with the Utility Penetrations through the Dam (Category IV).

Failure Mode Description: The utility penetrations are of a varying nature (telephone, fiber optic lines, etc.). These penetrations are typically placed in a utility box or trench below the foundation of the dam. Poor compaction of backfill soil is commonly the cause of problems stemming from internal erosion during impoundment events. The potential failure mode associated with these types of structures is initiated in the backfill or adjacent to the utility box. During impoundment of water, flow is initiated along the interface resulting in internal erosion which progresses back from the downstream end of the pipe toward the upstream end of the penetrating structure. As soil is eroded a large enough void can develop that can cause settlement and cracking of the embankment or in the worst case a loss of the majority of the impounded water through a breach.

Adverse Factors:
(1) Anti-seep collars on telephone lines likely hampered compaction of backfill.
(2) The level of quality assurance (QA) is uncertain since the utilities were installed by others.
(3) There is a limited zone of backfill under the dam cutoff at utility crossings however anti-seep collars would require enlarging the excavation.

Positive Factors:
(1) AT&T and Arizona Telephone are below the cutoff, 5 feet below the foundation.
(2) AT&T and Arizona Telephone are encased in concrete.
(3) The fiber optic line is shallow, constructed to ADWR/FCDMC standards.
(4) The utility crossings at Saddleback FRS are dry utilities.

Other Considerations (Relating to Failure Mode S2b):
(1) List all penetrations in the ISA Report (size, location, type)
(2) The Arizona Telephone lines were constructed prior to the dam being constructed at Salome Crossing.
(3) The abandoned gas line was likely removed during construction of the cutoff.
(4) The fiber optic was constructed through the dam in 1999 according to ADWR standards, likely above the filter.

H2. Structural Failure of the Principal Outlet (Category IV).

Failure Mode Description: The principal outlet is a large reinforced concrete box culvert construction to modern standards. The culvert is founded on native material. Construction of the culvert included a filter diaphragm and a drain pipe system. The box culvert is relatively short in length. In order for the box culvert to fail based on a structural failure or collapse there would have to be a failure of the concrete and/or reinforcing steel. The load on the box culvert, either from the earth embankment or flood flows would overstress the culvert and cause a structural failure.

Adverse Factors:
(1) The principal outlet structure has never been tested at full reservoir head.

Positive Factors:
(1) The structure appears to be in good condition, no concrete deterioration or cracking.
(2) The FRS is a low head structure.
(3) The Saint Anthony Falls structure is a proven design and has been tested.
(4) The filter encompasses the structure.
(5) O&M inspections are done regularly.
(6) The downstream outlet and approach channels have erosion protection (shotcrete) which is in good condition.

Other Considerations (Relating to Failure Mode H2):
(1) Failure of the Harquahala floodway impinging on the Saddleback embankment. The local drainage pattern is parallel to the structure. Breach of the Harquahala floodway

could potentially scour the upstream and or downstream toe of the dam locally at the right abutment.

(2) At the left abutment, the roadside drainage channel could scour locally.

(3) Potential scour in the floodway channel upstream from the dam would encroach on the upstream toe and slope. The 3:1 upstream slope is a mitigating factor. The channel has a large capacity and inflow to the channel from the contributing drainage is disturbed. The outflow Q is constrained by the outlet structure.

Other Considerations: These issues were discussed by the FMEA team but a potential failure mode was not identified for evaluation (descriptions of adverse and positive factors were not developed).

(1) **Longitudinal Cracking at the upstream toe:** No failure modes associated with longitudinal cracking along the upstream toe were identified by the investigation conducted by AMEC.

(2) **There is no Evidence of Earth Fissures in the area of the FRS:** Earth fissures have been observed west of the FRS and with the possibility of future development it is possible that earth fissures will develop in response to increased groundwater withdrawal and associated ground subsidence.

(3) **Slope Stability:** Static and rapid drawdown stability analysis are documented and are adequate. There is no failure mode associated with slope stability. The FRS is in a very low seismicity zone and there is no risk of seismic slope stability failure.

4.0 LIKELIHOOD AND CONSEQUENCE CATEGORIES

The likelihood of occurrence of each identified failure mode has been assigned to one of three categories according to the FMEA team professional judgment. This adopts a subjective, degree-of-belief approach to the expression of uncertainty, as opposed to relative-frequency statistics of observed occurrences. These likelihood judgments express degrees of uncertainty but are not quantified in the probability matrix. They recognize simply that the occurrence of some failure modes is believed to be more likely than others for this particular dam. This relative measure of likelihood is contained in the categories defined in Table 1.

Table 1. Likelihood Categories

Category	Description
High	Highest likelihood of occurrence
Medium	Intermediate likelihood of occurrence
Low	Lowest likelihood of occurrence

In assigning likelihoods during the FMEA workshop, failure modes representative of the most likely and the least likely categories were evaluated.

Consequence categories follow along similar lines as likelihood categories in reflecting the relative severity of failure effects specific to the dam. The actual magnitude of the downstream consequences depends on such factors as economic losses, population at risk, and the effectiveness of the warning and evacuation. These were not evaluated directly by the FMEA team. This relative measure of consequence is contained in the categories defined in Table 2.

Table 2. Consequence Categories

Category	Description
High	Highest inundation effects.
Medium	Intermediate inundation effects.
Low	Lowest inundation effects.

5.0 FAILURE MODE AND EFFECTS TABLE

Construction of the Failure Mode and Effects Table (Table 3) summarizes the failure modes identified and evaluated in the FMEA workshop by the workshop FMEA team. The columns contain the following elements from left to right:

- Failure Mode – identifies the primary failure mechanism
- Initiating Condition – condition(s) giving rise to initiation of the failure mode/sequence
- Effects – distinguishes dam breach and spillway discharge failure types
- Likelihood – likelihood category form Table 1
- Consequences – consequence category from Table 2
- Information Needs – summary of important additional information that could support or modify the failure mode assessment provided
- Existing Risk Reduction Factors – conditions or measures in place that have acted to reduce likelihood and/or consequences assigned
- Potential Risk Reduction Measures – action, studies, or features that might reduce the assigned likelihood and/or consequences
- Comments – supplemental remarks

Table 3. Summary of Failure Mode and Effects Analysis - Saddleback FRS
Maricopa County, Arizona

FAILURE MODE	INITIATING CONDITION	EFFECT	LIKELIHOOD	CONSEQUENCES	INFORMATION NEEDS	EXISTING RISK REDUCTION FACTORS	POTENTIAL RISK REDUCTION FACTORS	COMMENTS
H1. Overtopping During Major Flood Event (Category I).	Reservoir inflow for events larger than the ½ Probable Maximum Flood	Downstream inundation impacts (low)	Low to High (PMF to ½ PMF)	Low to Medium (½ PMF to PMF)	Dynamic reservoir routing. Check Crest Elevation Evaluate effect of a defined upstream low-flow channel. Check impacts of I-10 and Harquahala Dambreak.	Existing CAP embankment dike. Dam crest is gravel plated. I-10 embankment will attenuate flows. Low height structure, limited breach depth.	Review drainage area include effects of I-10 and Harquahala dambreak. Modify FRS to a floodway/levee. Verify Hazard Classification (consider emergency spillway).	
S1a. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks under the Filter (Category II).	Reservoir impoundment event, with an existing crack extending below the filter.	Internal erosion leading to a breach, downstream inundation	Low	Low	Extent and frequency of cracking. Collapse potential of weakly cemented soil. Future subsidence potential.	Dam has a central filter/drain. Structure is relatively free of cracks. Embankment soils are not particularly erodible.	Check filter compatibility. Phase II crack investigation. Gravel mulch on the slopes.	
S1b. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks that extend above the Filter (Category II).	Reservoir impoundment event, with an existing crack extending above the filter.	Internal erosion leading to a breach, downstream inundation	Low	Medium	Extent and frequency of cracking. Collapse potential of weakly cemented soil. Future subsidence potential.	Dam has a central filter/drain. Structure is relatively free of cracks. Embankment soils are not particularly erodible.	Check filter compatibility. Phase II crack investigation. Gravel mulch on the slopes. Extend filter to the surface.	
S1c. Failure Due to Internal Erosion of Embankment Soils through Transverse Cracks that extend through the Filter (Category II).	Reservoir impoundment event, with an existing crack extending through a the filter	Internal erosion leading to a breach, downstream inundation	Low	Low/Medium	Filter investigation (for contamination). Filter compatibility check. Degree of cementation.	Dam has a central filter/drain. Structure is relatively free of cracks. Embankment soils are not particularly erodible.	Check filter compatibility. Phase II crack investigation. Determine crack mechanisms on crest.	
S3. Internal Erosion Leading to a Breach in the Upper Embankment due to the Eroded Character of the Crest (Category II).	Reservoir impoundment event, with existing embankment erosion holes.	Internal erosion or piping leading to a breach, downstream inundation.	Medium	Low	Mechanism for observed erosion features on the crest. Extent of connectivity of erosion holes with other cracking.	Dam has a central filter/drain. Structure is relatively free of cracks. Embankment soils are not particularly erodible.	Determine the cause/mechanism for observed erosion features on the crest. Phase II crack investigation.	

Table 3. Summary of Failure Mode and Effects Analysis - Saddleback FRS
Maricopa County, Arizona

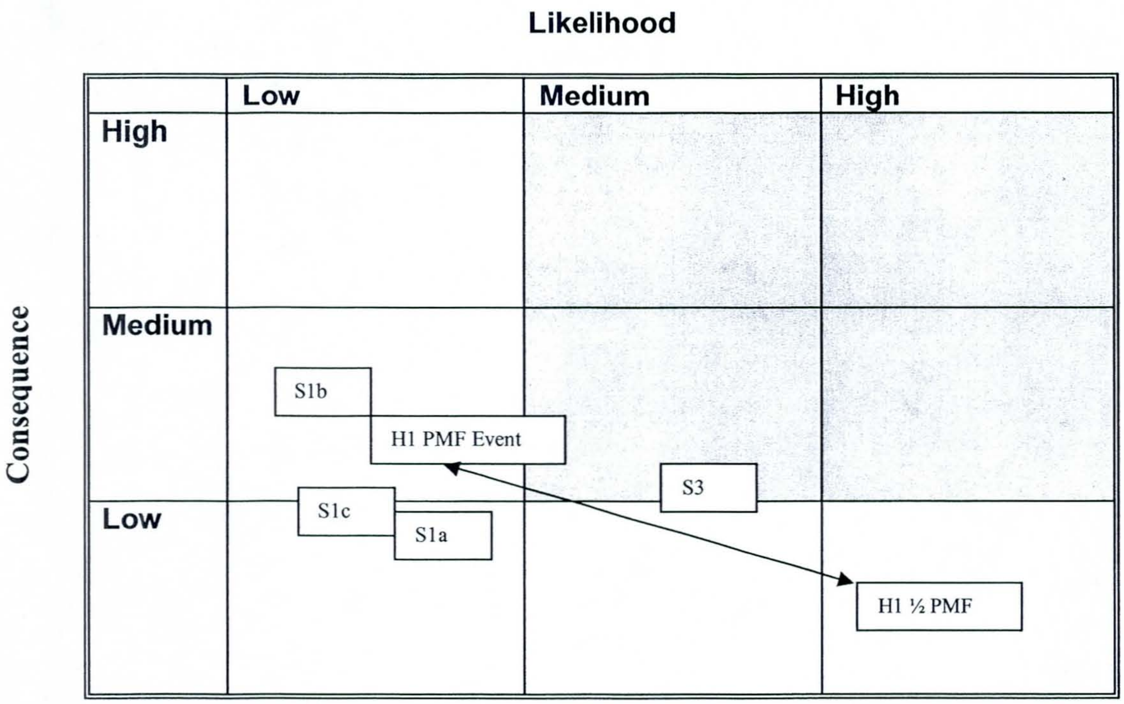
Gannett Fleming

FAILURE MODE	INITIATING CONDITION	EFFECT	LIKELIHOOD	CONSEQUENCES	INFORMATION NEEDS	EXISTING RISK REDUCTION FACTORS	POTENTIAL RISK REDUCTION FACTORS	COMMENTS
S2a. Potential Failure due to Internal Erosion Associated with the Principal Outlet and Irrigation Penetrations through the Dam (Category III).	Reservoir impoundment and flow initiated in voids near a penetration.	Preferred flow paths are initiated and eventually initiates piping of fine grained embankments soils.	Not determined.	Not determined	Better understanding of the central filter's ability to act as a filter diaphragm. Understanding of the cause/mechanism for observed erosion features on the crest.	Dam has a central filter/drain. Structure is relatively free of cracks. Embankment soils are not particularly erodible.	Check filter compatibility. Phase II crack investigation.	
S2B. Potential Failure due to Internal Erosion Associated with the Utility Penetrations through the Dam (Category IV).	Reservoir impoundment and flow initiated in voids near a penetration	Preferred flow paths are initiated and eventually initiates piping of fine grained embankments soils.	Not determined	Not determined				
H2. Structural Failure of the Principal Outlet (Category IV).	Failure of structural concrete in the Principal Outlet, initiated by overstressing.	Failure of the box culvert followed by an impoundment event that results in uncontrolled release and possible breach.						

6.0 FAILURE MODE BINNING

While the FMEA table contains the likelihood and consequence attributes of risk, it does not portray risk as such. Binning extends the FMEA to the final step of separating failure modes into rank-ordered groupings according to their respective relative risks. It is convenient to bin failure modes into a two-dimensional array as shown in Table 4, where each failure mode falls into a discrete region of risk space according to its particular likelihood and consequence attributes.

Table 4. Failure Mode Binning for Saddleback FRS
(Numbers refer to failure mode identification numbers in Table 3 and shaded region represents comparatively greater risk)



In the format of Table 4, risk increases to the upper right of the array and decreases to the lower left. Thus the shaded region of Table 4 contains any failure modes of generally greater risk. Note that Table 4 indicates a portion of Failure Modes H1 (overtopping during a major flood event) and S3 (internal erosion leading to a breach in the upper embankment due to the eroded character of the crest) are within the shaded region. Failure Mode 1 is depicted as ranging from low likelihood, medium consequences (for the PMF event) to high likelihood, lower consequences. The determination of Failure Mode H1 falling within this block of the shaded region is dependent on the storm frequency, magnitude, and downstream consequences. Failure Mode S3 is depicted as medium likelihood and low to medium consequence.

7.0 Summary And Conclusions

Saddleback FRS was constructed pursuant to a relatively modern dam design. Construction appears to have been without any particular issues, although it is apparent from the construction photos that some contamination of the filter occurred. It is also apparent from as-built drawings that the filter was not extended to the bottom of the over-excavation along the entire length of the dam. The dam has performed normally and satisfactorily for 23 years. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that there is a dam safety deficiency at the site. Rather, it identifies physically possible conditions or processes (generally with a remote but still credible chance of occurrence) that persons associated with owning, inspecting, analyzing and operating the dam should be aware. Some of the potential failure modes are highlighted (or prioritized) for attention of the dam owners and operators. They are highlighted because the specific conditions at the dam and appurtenant structures are such that these failure modes are physically possible and are considered the most realistic and most credible potential failure modes definable at the site.

One Category I potential failure modes was identified by the FMEA team. The Category I failure mode is related to overtopping during a major flood event (excess of the ½ PMF). The principal outlet provides good protection and immediate opportunity for discharge before, during, and after an overtopping event. However, there are residential structures downstream from the structure and there is no defined downstream channel. The number of people and structures at risk in the flood path is uncertain, but is likely to increase with anticipated future development.

Structure Assessment Program, Phase I
Hoque & Associates, Inc. Work Assignment No. 3
Harquahala FRS and Saddleback FRS

DATA REVIEW

Hoque and Associates, Inc. (HA) collected and compiled information required under Sections 2.2, 2.3, and 2.4 of the contract Scope of Work. The following paragraphs describe the activities performed and information obtained.

UTILITIES RESEARCH

HA conducted a utility search for each of the two dams. The search was limited to areas located within 1/8 mile upstream and downstream of each dam embankment. Information related to utility locations was gathered from utility companies provided as-built drawings and data provided to HA by Kimley-Horn. The following table includes all utilities found within 1/8 mile and or vicinity of the dams. All utility locations were input to and identified on the AutoCAD base maps provided by Kimley-Horn.

Sl. No.	Utility Name	Description
1.	Southern California Edison Power Line	Runs at south and approximately parallel to Harquahala FRS. Distance of the Power Line from Harquahala FRS varies from 0 to 350 feet.
2.	Central Arizona Project	Canal runs immediately south of Harquahala FRS.
3.	Southwest Gas	In the vicinity of Saddleback FRS
4.	Arizona Public Service (APS) Overhead Electric Lines	In the vicinity of Saddleback FRS
5.	American Telephone & Telegraph	Coaxial cable buried in Salome Highway at west and of dam

PROPOSED RESIDENTIAL AND COMMERCIAL DEVELOPMENTS

HA collected information on proposed residential and commercial developments as well as proposed infrastructure in the vicinity (two mile radius) of the dams from the Maricopa County Assessor's Office (County Assessor's) website and the State Land Department.

Based on information contained in the Land Use Maps provided by the State Land Department and maps available through the County Assessor's website, total land within two miles upstream and two miles downstream of Saddleback FRS is 29,920 acres. 20 percent (6,030 acres) of this land is owned by the State Trust, 35 percent (10,450 acres) is owned by US Bureau of Land Management (USBLM), and 45 percent (13,470 acres) is owned by private parties.

Total land within two miles upstream and two miles downstream of Harquahala FRS is 41,390 acres. 10 percent (4,128 acres) of this land is owned by the State Trust, 81 percent (33,453 acres) is owned by US Bureau of Land Management (USBLM), and 9 percent (3,808 acres) is owned by private parties.

Based on the information provided there is no proposed infrastructure other than the existing Interstate 10 Freeway and its Right of Way within a two mile radius of the dams.

ADJACENT AREA PROPERTIES

HA collected data related to current properties, critical facilities, and present and projected populations within the prescribed distances/radii from each dam as required under our Scope of Work. HA compiled related information in spreadsheets that are contained in Appendix A – Data Review Tables. A description of information obtained under each category is presented separately below.

Current Properties

HA collected information on current properties located within a distance of two miles upstream and downstream of each dam from maps available through the Maricopa County Assessor's website. Properties located within two miles were researched and listed.

Based on our research, a total of 561 properties were located within the prescribed area of Saddleback FRS. The Current Properties information obtained for Saddleback FRS is presented in Tables 1 – List of Current Properties within Two Miles of Saddleback FRS. The Current Properties Listing table includes the Assessor's Parcel Number, value and condition for each property listed.

Based on our research, a total of 118 properties were located within the prescribed area of Harquahala FRS. The Current Properties information obtained for Harquahala FRS is presented in Tables 2 – List of Current Properties within Two Miles of Saddleback FRS. The Current Properties Listing table includes the Assessor's Parcel Number, value and condition for each property listed.

Various lands within the vicinity of the dams do not have information available such as lands owned by State Trust. The properties without an assessor parcel number and or information available are not included in Table 1 and 2.

Critical Facilities

HA collected information on Critical Facilities located within 2 miles upstream and downstream of each dam from maps provided by the State Land Department, communicating with respective agencies, and browsing several websites including the Maricopa County Assessor's website.

Based on our research, several Critical Facilities currently exists within a 2-mile radius of the Saddleback FRS and Harquahala FRS. These include the CAP canal, Interstate 10, and the Harquahala generating station.

Present and Projected Populations

HA collected information on current and projected populations in the areas adjacent to the dams (up to two miles upstream and downstream of the dams) from the Maricopa Association of Governments (MAG) website. Data obtained through the MAG website is presented as follows.

Year	Persons / sq mi
2000	0 - 50
2010	0 - 50
2020	0 - 50
2030	0 - 50

Based on the MAG information, it appears that present and projected future populations do not differ and significant growth is not expected. HA compiled population density maps that are contained in Appendix B – Population Density Maps

REFERENCES

1. As-builts provided by Kimley-Horn
2. Maricopa County Assessor's Website – (http://www.maricopa.gov/assessor/gisPortal/gis_portal.asp
3. Maricopa Association of Governments Website - (http://www.mag.maricopa.gov)
4. Utility company furnished utility as-built drawings.
5. Land Use Maps provided by the State Land Department

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
1	506-30-010	$104,000	$83,512	NO	-
2	506-30-012B	$37,000	$26,875	NO	-
3	506-30-012C	$38,500	$35,312	NO	-
4	506-30-011	$70,000	$70,000	NO	-
5	506-30-013-A	$53,000	$53,000	NO	-
6	506-30-013-B	$48,500	$48,500	NO	-
7	506-30-021-A	$298,217	$298,217	NO	-
8	506-30-024-A	$153,600	$153,600	NO	-
9	506-30-016-C	$4,800	$4,800	NO	-
10	506-30-016-A	$299,139	$299,139	NO	-
11	506-30-016-D	$38,971	$35,420	NO	-
12	506-30-007-C	$908	$908	NO	-
13	506-30-007B	$70,100	$70,100	NO	-
14	506-30-007-A	$9,600	$9,600	NO	-
15	506-30-004	$38,400	$38,400	NO	-
16	506-30-006	$38,400	$38,400	NO	-
17	506-30-008	$12,288	$12,288	NO	-
18	506-30-002-B	$6,000	$4,744	NO	-
19	506-30-002-D	$51,000	$51,000	NO	-
20	506-30-002-L	$31,500	$26,767	NO	-
21	506-30-002-P	$38,500	$32,504	NO	-
22	506-30-002-Q	$31,500	$26,767	NO	-
23	506-30-002-M	$29,000	$24,492	NO	-
24	506-30-002-K	$25,500	$21,624	NO	-
25	506-30-002-T	$43,000	$30,599	NO	-
26	506-30-002-S	$47,000	$33,586	NO	-
27	506-30-015-J	$41,000	$24,977	NO	-
28	506-30-015-G	$21,000	$17,210	NO	-
29	506-30-015-E	$113,553	$65,171	NO	-
30	506-30-015-H	$80,000	$71,276	NO	-
31	506-30-015-F	$29,000	$19,007	NO	-
32	506-62-008	$7,500	$6,283	NO	-
33	506-62-082-B	$6,000	$5,502	NO	-
34	506-62-082-A	$19,500	$9,741	NO	-
35	506-62-083	$21,000	$11,320	NO	-
36	506-62-088	$22,000	$11,975	NO	-
37	506-62-089	$21,500	$11,632	NO	-
38	506-62-090	$21,500	$11,632	NO	-
39	506-62-091	$22,000	$11,975	NO	-
40	506-62-096	$21,500	$11,621	NO	-
41	506-62-097	$20,500	$11,067	NO	-
42	506-62-098	$20,500	$11,071	NO	-
43	506-62-099	$21,000	$11,394	NO	-
44	506-62-104	$22,000	$11,978	NO	-
45	506-62-105	$21,500	$116,350	NO	-
46	506-62-106	$21,500	$11,635	NO	-
47	506-62-107	$4,700	$4,700	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS					
Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
48	506-62-112	$21,000	$11,321	NO	-
49	50-62-113	$20,500	$11,102	NO	-
50	506-62-111	$21,000	$11,320	NO	-
51	506-62-110	$20,500	$11,071	NO	-
52	506-62-109	$4,700	$4,700	NO	-
53	506-62-108	$4,700	$4,700	NO	-
54	506-62-103	$22,000	$11,977	NO	-
55	506-62-102	$4,700	$4,700	NO	-
56	506-62-101	$21,000	$11,372	NO	-
57	506-62-100	$21,000	$11,393	NO	-
58	506-62-095	$21,500	$11,622	NO	-
59	506-62-094	$21,000	$11,370	NO	-
60	506-62-093	$21,500	$11,721	NO	-
61	506-62-092	$22,000	$11,975	NO	-
62	506-62-086-A	$17,594	$10,991	NO	-
63	506-62-086-B	$15,500	$7,982	NO	-
64	506-62-087	$22,000	$11,975	NO	-
65	506-62-084	$21,000	$11,320	NO	-
66	506-62-085	$20,500	$11,067	NO	-
67	506-62-057	$20,500	$11,069	NO	-
68	506-62-058	$21,500	$11,539	NO	-
69	506-62-059	$22,000	$11,757	NO	-
70	506-62-060	$21,500	$11,506	NO	-
71	506-62-061	$22,000	$11,953	NO	-
72	506-62-062	$22,000	$11,975	NO	-
73	506-62-064	$21,500	$11,756	NO	-
74	506-62-065	$21,000	$11,370	NO	-
75	506-62-066	$21,000	$11,370	NO	-
76	506-62-067	$21,500	$11,622	NO	-
77	506-62-072	$22,000	$11,976	NO	-
78	506-62-073	$22,000	$11,976	NO	-
79	506-62-075	$22,000	$11,976	NO	-
80	506-62-074	$22,000	$11,976	NO	-
81	506-62-081	$20,500	$11,070	NO	-
82	506-62-080	$21,000	$11,320	NO	-
83	506-62-078-D	$10,500	$8,610	NO	-
84	506-62-0788-B	$10,500	$8,610	NO	-
85	506-62-078-C	$10,500	$8,610	NO	-
86	506-62-078-A	$10,500	$8,610	NO	-
87	506-62-079	$21,000	$11,320	NO	-
88	506-62-076	$22,000	$11,976	NO	-
89	506-62-077	$21,500	$11,633	NO	-
90	506-62-070	$21,500	$11,633	NO	-
91	506-62-071	$22,000	$11,976	NO	-
92	506-62-068	$21,500	$11,622	NO	-
93	506-62-069	$20,500	$11,067	NO	-
94	506-62-063	$21,000	$11,320	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
95	506-62-053	$2,000	$2,000	NO	-
96	506-62-054	$2,000	$2,000	NO	-
97	506-62-055	$2,000	$2,000	NO	-
98	506-62-056	$2,000	$2,000	NO	-
99	506-62-049	$2,000	$2,000	NO	-
100	506-62-050	$2,000	$2,000	NO	-
101	506-62-051	$2,000	$2,000	NO	-
102	506-62-052	$2,000	$2,000	NO	-
103	506-62-041	$6,500	$5,473	NO	-
104	506-62-042	$6,500	$5,473	NO	-
105	506-62-043	$6,500	$5,473	NO	-
106	506-62-044	$6,500	$5,473	NO	-
107	506-62-045	$6,500	$5,473	NO	-
108	506-62-046	$6,500	$5,473	NO	-
109	506-62-047	$6,500	$5,473	NO	-
110	506-62-048	$6,500	$5,473	NO	-
111	506-62-033	$6,500	$5,473	NO	-
112	506-62-034	$6,500	$5,473	NO	-
113	506-62-036	$6,500	$5,473	NO	-
114	506-62-037	$6,500	$5,473	NO	-
115	506-62-038	$6,500	$5,473	NO	-
116	506-62-039	$6,500	$5,473	NO	-
117	506-62-040	$2,000	$2,000	NO	-
118	506-62-025	$7,500	$6,283	NO	-
119	506-62-026	$7,500	$6,283	NO	-
120	506-62-027	$7,500	$6,283	NO	-
121	506-62-028	$7,500	$6,283	NO	-
122	506-62-029	$7,500	$6,283	NO	-
123	506-62-030	$7,500	$6,283	NO	-
124	506-62-031	$7,500	$6,283	NO	-
125	506-62-032	$7,500	$6,283	NO	-
126	506-62-017	$7,500	$6,283	NO	-
127	506-62-018	$7,500	$6,283	NO	-
128	506-62-019	$7,500	$6,283	NO	-
129	506-62-020	$7,500	$6,283	NO	-
130	506-62-021	$7,500	$6,283	NO	-
131	506-62-022	$7,500	$6,283	NO	-
132	506-62-023	$7,500	$6,283	NO	-
133	506-62-024	$7,500	$6,283	NO	-
134	506-62-009B	$7,500	$6,283	NO	-
135	506-62-010-B	$7,500	$6,283	NO	-
136	506-62-012-A	$500	$500	NO	-
137	506-62-011-B	$7,500	$6,283	NO	-
138	506-62-012-B	$7,500	$5,900	NO	-
139	506-62-013	$7,500	$6,283	NO	-
140	506-62-014	$7,500	$6,283	NO	-
141	506-62-015	$7,500	$6,283	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
142	506-62-016	$7,500	$6,283	NO	-
143	506-62-002	$7,000	$5,733	NO	-
144	506-62-003	$7,000	$5,733	NO	-
145	506-62-004	$7,000	$5,733	NO	-
146	506-62-005-C	$500	$500	NO	-
147	506-62-005-B	$7,000	$5,806	NO	-
148	506-62-006-B	$500	$500	NO	-
149	506-62-006-A	$7,500	$6,181	NO	-
150	506-62-007-A	$4,000	$6,126	NO	-
151	506-62-007-B	$7,000	$5,583	NO	-
152	506-30-009-C	$58,500	$41,535	NO	-
153	506-30-009-D	$59,500	$42,245	NO	-
154	506-30-009-B	$54,000	$54,000	NO	-
155	506-31-012-A	$49,500	$49,500	NO	-
156	506-31-012-B	$12,000	$9,272	NO	-
157	506-31-013	$69,500	$68,970	NO	-
158	506-31-014-F	$15,000	$13,258	NO	-
159	506-31-014-G	$44,000	$44,000	NO	-
160	506-31-014-E	$41,500	$38,068	NO	-
161	506-31-014-C	$73,000	$71,995	NO	-
162	506-31-014-D	$15,000	$13,258	NO	-
163	506-31-011-B	$43,000	$43,000	NO	-
164	506-31-020	$13,000	$11,523	NO	-
165	506-31-021	$13,000	$11,523	NO	-
166	506-31-022	$13,000	$11,523	NO	-
167	506-31-023	$13,000	$11,523	NO	-
168	506-31-025	$13,000	$11,523	NO	-
169	506-31-019-A	$13,000	$11,523	NO	-
170	506-31-024	$13,000	$11,523	NO	-
171	506-31-019-C	$9,500	$8,350	NO	-
172	506-31-019-D	$9,500	$8,350	NO	-
173	506-31-026	$13,000	$11,523	NO	-
174	506-31-017	$13,000	$11,523	NO	-
175	506-31-027	$13,000	$11,523	NO	-
176	506-31-018	$13,000	$11,523	NO	-
177	506-31-016	$13,000	$11,523	NO	-
178	506-31-016-A	$12,500	$11,523	NO	-
179	506-31-028	$18,000	$15,488	NO	-
180	506-31-049	$18,000	$15,968	NO	-
181	506-31-050	$18,000	$15,968	NO	-
182	506-31-056	$18,000	$15,968	NO	-
183	506-31-055	$18,000	$15,968	NO	-
184	506-31-054-E	$11,000	$9,020	NO	-
185	506-31-054-C	$11,000	$9,020	NO	-
186	506-31-054-D	$11,000	$9,020	NO	-
187	506-31-054-F	$11,000	$9,020	NO	-
188	506-31-051	$18,000	$15,968	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
189	506-31-052-A	$14,500	$13,178	NO	-
190	506-31-052-B	$13,000	$11,523	NO	-
191	506-31-053	$18,000	$15,968	NO	-
192	506-31-044-B	$13,000	$11,523	NO	-
193	506-31-043-A	$13,000	$11,523	NO	-
194	506-31-043-E	$36,947	$36,947	NO	-
195	506-31-045	$18,000	$15,968	NO	-
196	506-31-046	$18,500	$15,517	NO	-
197	506-31-042-A	$13,000	$11,523	NO	-
198	506-31-042-A	$13,000	$11,523	NO	-
199	506-31-042-B	$13,000	$11,523	NO	-
200	506-31-041-B	$13,000	$11,260	NO	-
201	506-31-041-A	$13,000	$11,260	NO	-
202	506-31-048	$18,500	$15,213	NO	-
203	506-31-039	$13,000	$11,523	NO	-
204	506-31-040	$13,000	$11,523	NO	-
205	506-31-029	$18,500	$15,852	NO	-
206	506-31-030-A	$13,500	$11,628	NO	-
207	506-31-030-B	$13,500	$11,628	NO	-
208	506-31-038	$18,500	$16,317	NO	-
209	506-31-037	$13,500	$11,569	NO	-
210	506-31-031	$13,500	$11,628	NO	-
211	506-31-032	$13,500	$11,628	NO	-
212	506-31-036	$13,500	$11,569	NO	-
213	506-31-035	$13,500	$11,569	NO	-
214	506-31-030-C	$13,500	$11,628	NO	-
215	506-31-033	$13,500	$11,331	NO	-
216	506-31-034	$13,500	$11,331	NO	-
217	506-31-003-B	$49,500	$49,500	NO	-
218	506-31-003-C	$49,000	$49,000	NO	-
219	506-31-003-D	$35,500	$32,759	NO	-
220	506-31-003-F	$1	$1	NO	-
221	506-31-003-G	$1	$1	NO	-
222	506-31-003-H	$1	$1	NO	-
223	506-31-010-B	$3,900	$3,198	NO	-
224	506-31-010-C	$7,900	$6,478	NO	-
225	506-31-010-D	$5,912	$4,848	NO	-
226	506-31-010-E	$5,917	$4,852	NO	-
227	506-31-010-F	$4,946	$4,056	NO	-
228	506-31-008-B	$33,000	$28,168	NO	-
229	506-31-007-F	$58,500	$58,500	NO	-
230	506-31-007-G	$41,500	$41,069	NO	-
231	506-31-061	$37,000	$26,313	NO	-
232	506-31-060	$40,500	$28,406	NO	-
233	506-31-058-C	$8,380	$5,991	NO	-
234	506-31-058-B	$20,605	$14,731	NO	-
235	506-31-058-A	$7,515	$5,373	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
236	506-31-006-B	$70,000	$70,000	NO	-
237	506-31-006-A	$69,500	$69,500	NO	-
238	506-30-026	$133,500	$118,949	NO	-
239	506-30-022	$152,002	$128,862	NO	-
240	506-30-023	$152,002	$128,862	NO	-
241	506-29-003-D	$8,500	$7,041	NO	-
242	506-29-003-U	$19,500	$14,898	NO	-
243	506-29-003-N	$16,000	$11,709	NO	-
244	506-29-003-M	$16,000	$11,709	NO	-
245	506-29-003-V	$16,000	$14,256	NO	-
246	506-29-003-W	$16,000	$14,256	NO	-
247	506-29-003-T	$16,000	$11,709	NO	-
248	506-29-003-S	$17,000	$12,146	NO	-
249	506-29-003-L	$71,653	$71,653	NO	-
250	506-29-003-R	$15,000	$10,004	NO	-
251	506-29-003-Q	$14,000	$10,429	NO	-
252	506-29-001	$75,254	$75,254	NO	-
253	506-29-022-A	$119,889	$101,638	NO	-
254	506-29-022-B	$29,808	$25,270	NO	-
255	506-29-002	$28,800	$28,800	NO	-
256	506-29-003-J	$16,000	$12,565	NO	-
257	506-29-003-H	$16,000	$12,565	NO	-
258	506-29-003-A	$40,500	$36,592	NO	-
259	506-58-194	$8,000	$8,000	NO	-
260	506-58-195	$6,000	$6,000	NO	-
261	506-58-196	$6,000	$5,474	NO	-
262	506-58-197	$8,000	$8,000	NO	-
263	506-58-198	$6,000	$6,000	NO	-
264	506-58-188	$6,000	$6,000	NO	-
265	506-58-189	$6,000	$5,474	NO	-
266	506-58-190	$8,000	$8,000	NO	-
267	506-58-191	$8,000	$8,000	NO	-
268	506-58-192	$6,000	$6,000	NO	-
269	506-58-193	$6,000	$6,000	NO	-
270	506-58-180	$6,000	$6,000	NO	-
271	506-58-181	$6,000	$6,000	NO	-
272	506-58-182	$6,000	$6,000	NO	-
273	506-58-183	$6,000	$6,000	NO	-
274	506-58-184	$6,000	$6,000	NO	-
275	506-58-185	$6,000	$6,000	NO	-
276	506-58-186	$6,000	$6,000	NO	-
277	506-58-187	$6,000	$6,000	NO	-
278	506-58-175	$8,000	$8,000	NO	-
279	506-58-176	$8,000	$8,000	NO	-
280	506-58-177	$8,000	$8,000	NO	-
281	506-58-178	$6,000	$6,000	NO	-
282	506-58-179	$6,000	$5,500	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
283	506-58-172	$8,000	$7,917	NO	-
284	506-58-173	$7,500	$7,500	NO	-
285	506-58-022-C	$8,500	$7,162	NO	-
286	506-58/-174-A	$8,000	$8,000	NO	-
287	506-58-174-B	$2,500	$2,054	NO	-
288	506-58-167	$6,000	$4,313	NO	-
289	506-58-168	$6,000	$4,313	NO	-
290	506-58-169-A	$8,000	$8,000	NO	-
291	506-58-169-B	$1,500	$1,500	NO	-
292	506-58-170	$8,000	$8,000	NO	-
293	506-58-171	$8,000	$8,000	NO	-
294	506-58-161	$6,000	$6,000	NO	-
295	506-58-162	$6,000	$6,000	NO	-
296	506-58-163-A	$7,500	$7,500	NO	-
297	506-58-163-B	$4,000	$3,421	NO	-
298	506-58-164	$6,000	$4,313	NO	-
299	506-58-165	$6,000	$4,313	NO	-
300	506-58-166	$8,500	$7,288	NO	-
301	506-58-154-D	$2,500	$2,355	NO	-
302	506-58-154-A	$6,000	$6,000	NO	-
303	506-58-154-C	$5,500	$5,500	NO	-
304	506-58-155	$6,000	$4,313	NO	-
305	506-58-156	$6,000	$4,313	NO	-
306	506-58-157	$6,000	$4,313	NO	-
307	506-58-158	$6,000	$4,313	NO	-
308	506-58-159	$6,000	$4,313	NO	-
309	506-58-160-A	$5,500	$5,500	NO	-
310	506-58-160-B	$3,000	$2,431	NO	-
311	506-58-146	$6,000	$6,000	NO	-
312	506-58-147	$6,000	$6,000	NO	-
313	506-58-148	$6,000	$6,000	NO	-
314	506-58-149-A	$2,000	$1,678	NO	-
315	506-58-149-B	$5,500	$5,500	NO	-
316	506-58-150	$6,000	$6,000	NO	-
317	506-58-151	$6,000	$6,000	NO	-
318	506-58-152	$6,000	$6,000	NO	-
319	506-58-153	$6,000	$6,000	NO	-
320	506-58-138	$6,000	$5,662	NO	-
321	506-58-139	$6,000	$5,662	NO	-
322	506-58-140	$6,000	$6,000	NO	-
323	506-58-141	$6,000	$6,000	NO	-
324	506-58-142	$6,000	$6,000	NO	-
325	506-58-143	$6,000	$6,000	NO	-
326	506-58-144	$6,000	$6,000	NO	-
327	506-58-145	$6,000	$6,000	NO	-
328	506-58-133	$6,000	$6,000	NO	-
329	506-58-134	$6,000	$5,493	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
330	506-58-135	$6,000	$6,000	NO	-
331	506-58-136	$6,000	$6,000	NO	-
332	506-58-137-A	$6,000	$6,000	NO	-
333	506-58-137-B	$6,000	$6,000	NO	-
334	506-58-025-A	$6,000	$5,605	NO	-
335	506-58-025-B	$6,000	$5,605	NO	-
336	506-58-017-B	$36,500	$32,966	NO	-
337	506-58-017-A	$36,500	$32,966	NO	-
338	506-58-005-J	$11,500	$11,500	NO	-
339	506-58-020-A	$6,000	$5,448	NO	-
340	506-58-020-B	$6,000	$5,605	NO	-
341	506-58-013-B	$6,000	$5,605	NO	-
342	506-58-013-A	$6,000	$5,605	NO	-
343	506-58-005-N	$6,000	$4,920	NO	-
344	506-58-005-M	$6,000	$4,920	NO	-
345	506-58-005-Q	$6,000	$4,920	NO	-
346	506-58-005-P	$6,000	$4,920	NO	-
347	506-58-016-A	$6,000	$4,920	NO	-
348	506-58-016-B	$6,000	$5,865	NO	-
349	506-58-009-B	$43,000	$42,522	YES	-
350	506-58-009-A	$43,000	$42,522	YES	-
351	506-58-005-R	$8,000	$6,560	NO	-
352	506-58-005-L	$8,000	$8,000	NO	-
353	506-58-012-A	$6,000	$5,605	NO	-
354	506-58-012-B	$6,000	$5,605	NO	-
355	506-58-005-G	$6,000	$5,170	NO	-
356	506-58-006	$54,500	$54,500	NO	-
357	506-58-005-D	$8,000	$8,000	NO	-
358	506-58-005-E	$6,000	$5,170	NO	-
359	506-58-005-F	$6,000	$6,505	NO	-
360	506-58-021-A	$45,190	$24,095	NO	-
361	506-58-132-L	$7,000	$7,000	NO	-
362	506-58-132-K	$6,000	$6,000	NO	-
363	506-58-132-J	$6,000	$6,000	NO	-
364	506-58-132-H	$8,000	$8,000	NO	-
365	506-58-132-G	$6,000	$6,000	NO	-
366	506-58-132-F	$4,500	$4,500	NO	-
367	506-58-206	$7,500	$7,128	NO	-
368	506-58-205	$7,500	$7,128	NO	-
369	506-58-204-B	$7,500	$7,128	NO	-
370	506-58-204-A	$7,500	$7,500	NO	-
371	506-58-203	$6,000	$5,474	NO	-
372	506-58-202	$6,000	$6,000	NO	-
373	506-58-201	$6,000	$6,000	NO	-
374	506-58-200	$6,000	$6,000	NO	-
375	506-58-212	$6,000	$6,000	NO	-
376	506-58-211	$6,000	$6,000	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS					
Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
377	506-58-210	$6,000	$6,000	NO	-
378	506-58-209	$6,000	$6,000	NO	-
379	506-58-208	$10,500	$10,500	NO	-
380	506-58-207	$7,500	$7,128	NO	-
381	506-58-220	$6,000	$5,500	NO	-
382	506-58-219	$6,000	$5,740	NO	-
383	506-58-218	$6,000	$5,740	NO	-
384	506-58-217	$6,000	$5,740	NO	-
385	506-58-216	$6,000	$5,740	NO	-
386	506-58-215	$6,000	$5,740	NO	-
387	506-58-214	$6,000	$6,000	NO	-
388	506-58-213	$6,000	$6,000	NO	-
389	506-58-227-A	$5,500	$4,950	NO	-
390	506-58-227-B	$1,500	$1,500	NO	-
391	506-58-226	$6,000	$5,740	NO	-
392	506-58-225	$6,000	$5,740	NO	-
393	506-58-224	$6,000	$5,740	NO	-
394	506-58-223-B	$6,000	$5,740	NO	-
395	506-58-223-A	$5,500	$5,500	NO	-
396	506-58-222	$6,000	$6,000	NO	-
397	506-58-221	$6,000	$6,000	NO	-
398	506-58-234-B	$6,000	$6,000	NO	-
399	506-58-224-A	$6,000	$6,000	NO	-
400	506-58-233	$6,000	$5,099	NO	-
401	506-58-232	$6,000	$5,674	NO	-
402	506-58-231	$6,000	$5,740	NO	-
403	506-58-232	$6,000	$5,740	NO	-
404	506-58-229	$6,000	$5,740	NO	-
405	506-58-228	$6,000	$5,740	NO	-
406	506-58-241	$6,000	$5,740	NO	-
407	506-58-240	$6,000	$5,740	NO	-
408	506-58-239-B	$1,500	$1,500	NO	-
409	506-58-239-A	$5,500	$5,349	NO	-
410	506-58-238	$6,000	$5,474	NO	-
411	506-58-237	$8,000	$8,000	NO	-
412	506-58-236	$6,000	$5,474	NO	-
413	506-58-235	$6,000	$6,000	NO	-
414	506-58-246	$6,000	$5,474	NO	-
415	506-58-245	$10,000	$10,000	NO	-
416	506-58-244	$8,000	$8,000	NO	-
417	506-58-243	$6,000	$5,674	NO	-
418	506-58-242-A	$5,500	$5,500	NO	-
419	506-58-242-B	$2,500	$2,500	NO	-
420	506-58-252	$8,000	$8,000	NO	-
421	506-58-251	$6,000	$6,000	NO	-
422	506-58-520	$6,000	$6,000	NO	-
423	506-58-249	$6,000	$6,000	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
424	506-58-248	$6,000	$6,000	NO	-
425	506-58-247	$8,000	$8,000	NO	-
426	506-58-259	$6,000	$6,000	NO	-
427	506-58-258	$6,000	$6,000	NO	-
428	506-58-257	$6,000	$6,000	NO	-
429	506-58-256	$6,000	$6,000	NO	-
430	506-58-255	$6,000	$6,000	NO	-
431	506-58-254	$6,000	$6,000	NO	-
432	506-58-253	$8,000	$8,000	NO	-
433	506-58-266	$6,000	$6,000	NO	-
434	506-58-265	$6,000	$6,000	NO	-
435	506-58-564	$8,000	$8,000	NO	-
436	506-58-263	$6,000	$6,000	NO	-
437	506-58-262	$6,000	$6,000	NO	-
438	506-58-261	$6,000	$5,474	NO	-
439	506-58-260	$6,000	$5,720	NO	-
440	506-58-267	$8,000	$8,000	NO	-
441	506-58-113-A	$5,801	$4,325	NO	-
442	506-58-113-D	$4,000	$4,000	NO	-
443	506-58-113-C	$500	$500	NO	-
444	506--58-113-E	$4,000	$4,000	NO	-
445	506-58-113-B	$500	$500	NO	-
446	506-58-268	$6,000	$5,474	NO	-
447	506-58-269	$6,000	$5,474	NO	-
448	506-58-112-B	$1,000	$1,000	NO	-
449	506-58-112-E	$10,000	$8,030	NO	-
450	506-58-112-D	$6,329	$4,299	NO	-
451	506-58-112-C	$1,000	$1,000	NO	-
452	506-58-274	$6,000	$6,000	NO	-
453	506-58-270	$6,000	$6,000	NO	-
454	506-58-111-A	$1,000	$1,000	NO	-
455	506-58-111-C	$5,500	$5,500	NO	-
456	506-58-111-B	$4,000	$4,000	NO	-
457	506-58-273	$500	$500	NO	-
458	506-58-110-D	$1,500	$1,500	NO	-
459	506-58-110-C	$1,500	$1,500	NO	-
460	506-58-110-B	$5,500	$5,500	NO	-
461	506-58-110-A	$1,000	$1,000	NO	-
462	506-58-114-F	$4,000	$4,000	NO	-
463	506-58-114-E	$7,683	$7,683	NO	-
464	506-58-114-D	$8,753	$8,753	NO	-
465	506-58-114-C	$13,003	$13,003	NO	-
466	506-58-114-B	$9,282	$6,453	NO	-
467	506-58-114-A	$4,000	$4,000	NO	-
468	506-58-121-D	$4,000	$2,971	NO	-
469	506-58-121-B	$6,000	$6,000	NO	-
470	506-58-121-C	$4,000	$2,971	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
471	506-58-115-B	$1,000	$1,000	NO	-
472	506-58-115-A	$4,000	$4,000	NO	-
473	506-58-115-B	$1,000	$1,000	NO	-
474	506-58-115-D	$2,500	$2,500	NO	-
475	506-58-115-F	$500	$500	NO	-
476	506-58-115-E	$500	$500	NO	-
477	506-58-115-C	$2,500	$2,500	NO	-
478	506-58-120-B	$5,500	$5,500	NO	-
479	506-58-120-A	$5,500	$5,254	NO	-
480	506-58-116-E	$12,000	$9,636	NO	-
481	506-58-116-A	$4,000	$4,000	NO	-
482	506-58-116-F	$10,000	$8,030	NO	-
483	506-58-116-C	$500	$500	NO	-
484	506-58-116-B	$500	$500	NO	-
485	506-58-119-C	$3,000	$2,532	NO	-
486	506-58-119-B	$3,000	$2,532	NO	-
487	506-58-119-D	$5,500	$4,890	NO	-
488	506-58-119-A	$12,000	$3,797	NO	-
489	506-58-117-E	$4,000	$4,000	NO	-
490	506-58-117-C	$1,500	$1,500	NO	-
491	506-58-117-B	$2,000	$2,000	YES	-
492	506-58-117-A	$1,000	$1,000	YES	-
493	506-58-118-D	$4,500	$4,015	NO	-
494	506-58-118-C	$4,000	$4,015	NO	-
495	506-58-118-B	$4,000	$4,015	NO	-
496	506-58-118-A	$4,000	$4,000	NO	-
497	506-58-122-B	$4,000	$3,630	NO	-
498	506-58-122-D	$3,000	$2,838	NO	-
499	506-58-122-C	$3,000	$2,609	NO	-
500	506-58-122-G	$15,127	$15,127	NO	-
501	506-58-123-E	$3,000	$2,224	NO	-
502	506-58-123-D	$10,245	$10,245	NO	-
503	506-58-123-C	$13,666	$10,799	NO	-
504	506-58-123-B	$5,500	$5,500	NO	-
505	506-58-124	$8,500	$8,500	NO	-
506	506-58-125	$21,268	$20,840	NO	-
507	506-58-132-E	$6,000	$6,000	NO	-
508	506-58-132-D	$6,000	$6,000	NO	-
509	506-58-132-C	$6,000	$6,000	NO	-
510	506-58-132-B	$6,000	$6,000	NO	-
511	506-58-131	$2,500	$2,500	NO	-
512	506-58-130	$2,500	$2,500	NO	-
513	506-58-129	$2,500	$2,500	NO	-
514	506-58-128	$4,500	$4,500	NO	-
515	506-58-127	$4,500	$4,500	NO	-
516	506-58-126	$14,391	$10,826	NO	-
517	506-58-274	$19,000	$14,938	NO	-

Table 1 List of Current Properties within Two Miles of Saddleback FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
518	506-29-067	$107,000	$107,000	NO	-
519	506-29-066	$81,500	$81,500	NO	-
520	506-30-225	$31,000	$26,957	NO	-
521	506-30-001-A	$68,000	$68,000	NO	-
522	506-30-001-B	$61,000	$61,000	NO	-
523	506-18-022-R	$21,780	$15,954	NO	-
524	506-18-022-S	$21,780	$15,954	NO	-
525	506-18-022-G	$27,500	$21,514	NO	-
526	506-18-022-F	$26,500	$20,893	NO	-
527	506-18-022-E	$26,500	$20,893	NO	-
528	506-18-930	$25,000	$11,946	NO	-
529	506-18-022-T	$69,000	$47,296	NO	-
530	506-18-022-U	$189,000	$125,255	NO	-
531	506-18-022-P	$148,500	$82,892	NO	-
532	506-18-022-Q	$12,000	$12,000	NO	-
533	506-18-022-B	$19,000	$14,681	NO	-
534	506-18-022-A	$19,000	$14,681	NO	-
535	506-18-023	$916,261	$278,649	NO	-
536	506-18-020	$38,493	$31,450	NO	-
537	506-18-019-B	$28,000	$22,074	NO	-
538	506-18-019-C	$28,000	$22,074	NO	-
539	506-18-019-A	$37,000	$30,872	NO	-
540	506-18-022-M	$26,500	$21,068	NO	-
541	506-18-022-J	$26,500	$18,748	NO	-
542	506-18-022-K	$26,500	$17,775	NO	-
543	506-18-022-L	$26,500	$20,893	NO	-
544	506-18-022-N	$38,500	$31,413	NO	-
545	506-18-024-B	$28,000	$24,224	NO	-
546	506-18-024-A	$28,000	$24,224	NO	-
547	506-18-016-D	$39,500	$23,680	NO	-
548	506-18-018-B	$19,000	$14,799	NO	-
549	506-18-018-A	$61,500	$59,022	NO	-
550	506-18-016-B	$63,500	$38,145	NO	-
551	506-18-017-D	$13,366	$10,960	NO	-
552	506-18-017-E	$1,351	$1,108	NO	-
553	506-18-016-C	$56,000	$33,510	NO	-
554	506-31-002	$70,000	$70,000	NO	-
555	506-30-005	$37,286	$37,286	NO	-
556	506-30-006	$38,400	$38,400	NO	-
557	506-30-008	$76,800	$76,800	NO	-
558	506-30-007-B	$70,100	$70,100	NO	-
559	506-30-007-A	$9,600	$9,600	NO	-
560	506-30-004	$38,400	$38,400	NO	-
561	506-30-003	$37,286	$37,286	NO	-

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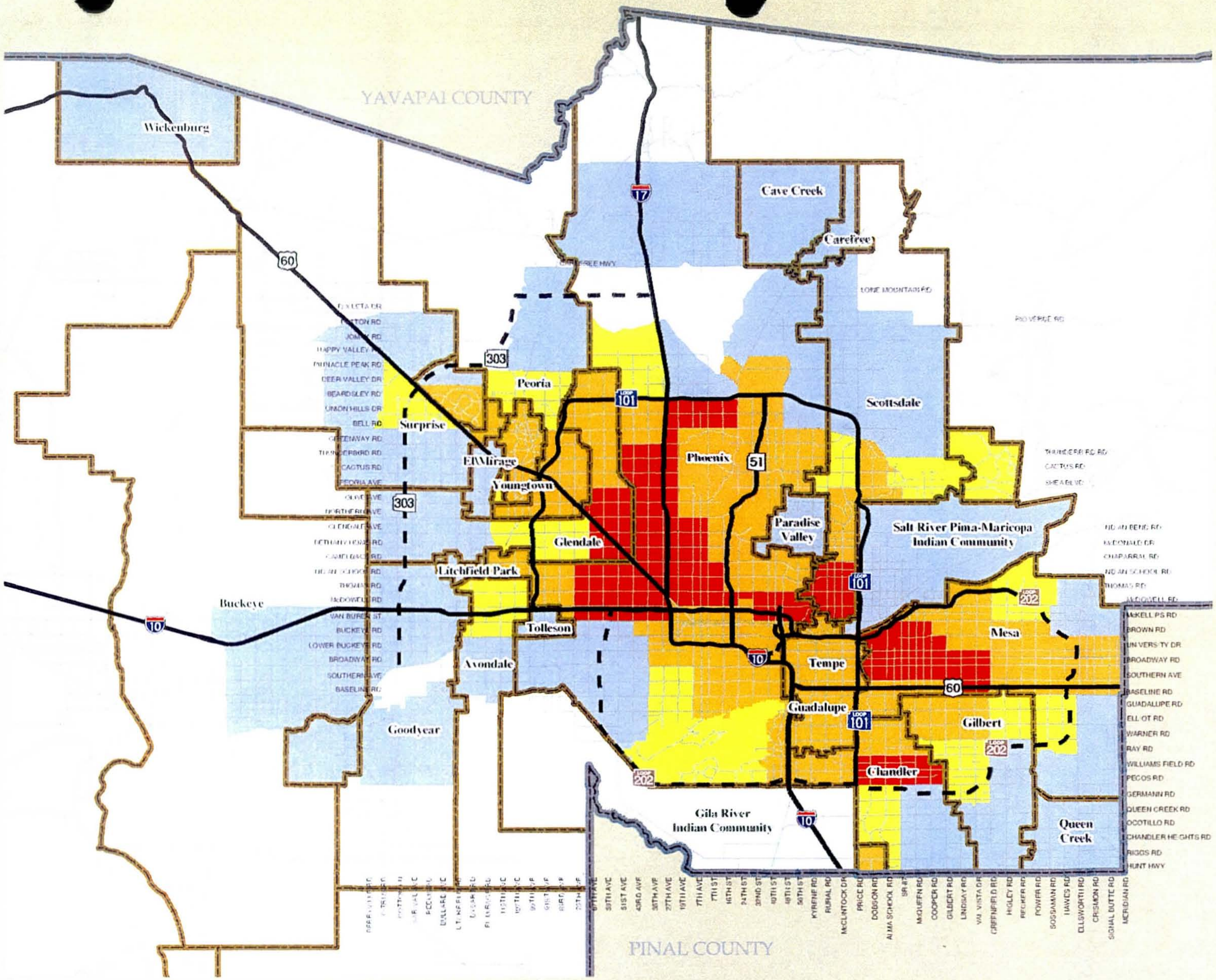
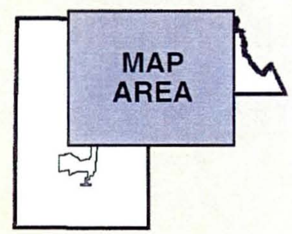
2000 Population Density for Interim Socioeconomic Projections by RAZ*



People Per Square Mile
(Maricopa County Average = 336)

- 0 - 50
- 50 - 1000
- 1000 - 2000
- 2000 - 5000
- More than 5000
- County Boundary
- MPA Boundaries
- Existing Freeways/Expressways
- Planned Freeways/Expressways
- Major Roads



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0 5 10 15 Miles

*Based on interim projections by Municipal Planning Area (MPA) and Regional Analysis Zone (RAZ) for 2010, 2020, 2025 and 2030 accepted by MAG Regional Council on June 25, 2003.

Prepared by Maricopa Association of Governments, July 2003

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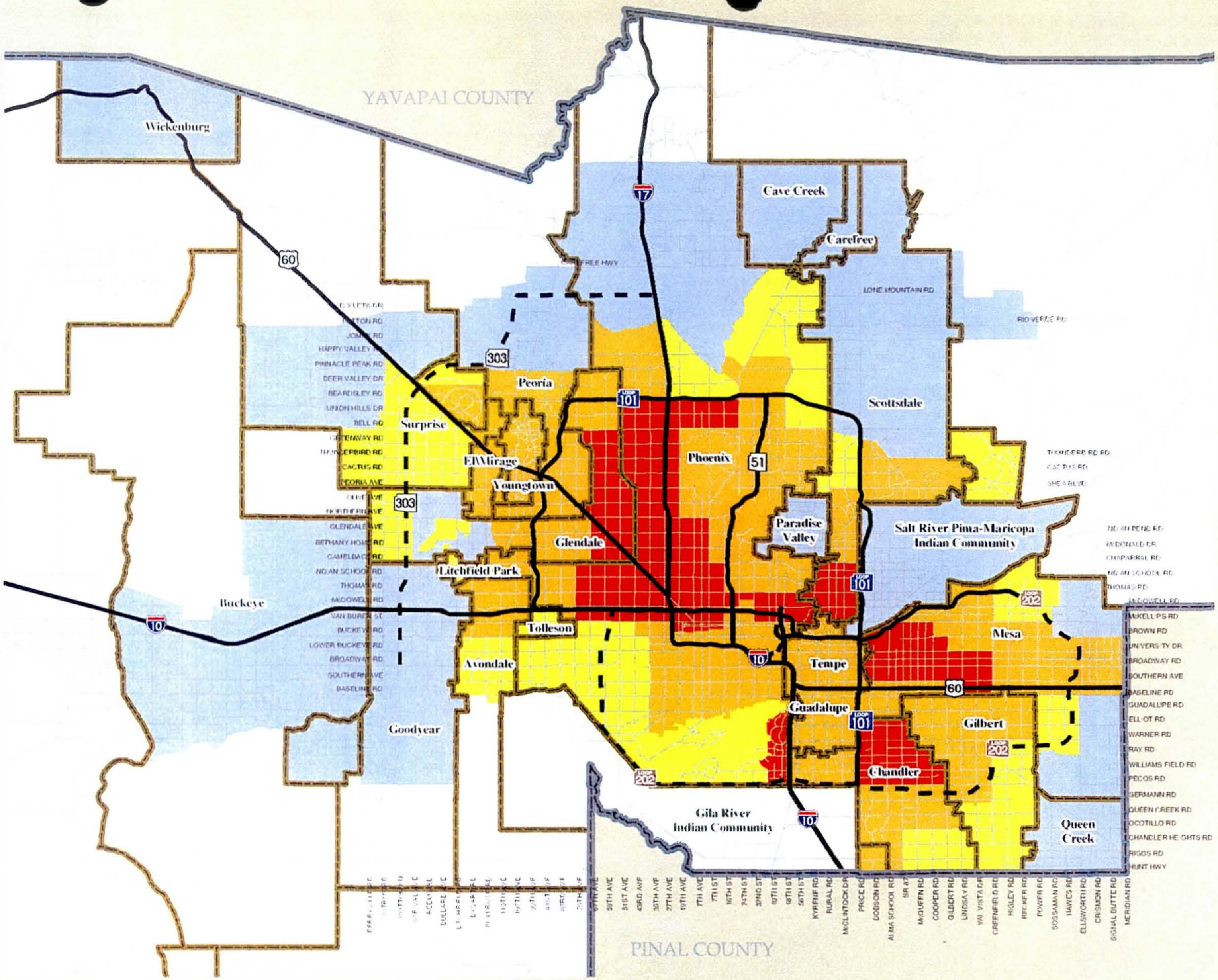
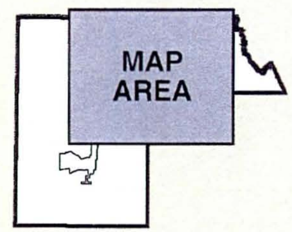
2010 Population Density for Interim Socioeconomic Projections by RAZ*



People Per Square Mile
(Maricopa County Average = 448)

- 0 - 50
- 50 - 1000
- 1000 - 2000
- 2000 - 5000
- More than 5000
- County Boundary
- MPA Boundaries
- Existing Freeways/Expressways
- Planned Freeways/Expressways
- Major Roads



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0 5 10 15 Miles

*Based on Interim projections by Municipal Planning Area (MPA) and Regional Analysis Zone (RAZ) for 2010, 2020, 2025 and 2030 accepted by MAG Regional Council on June 25, 2003.

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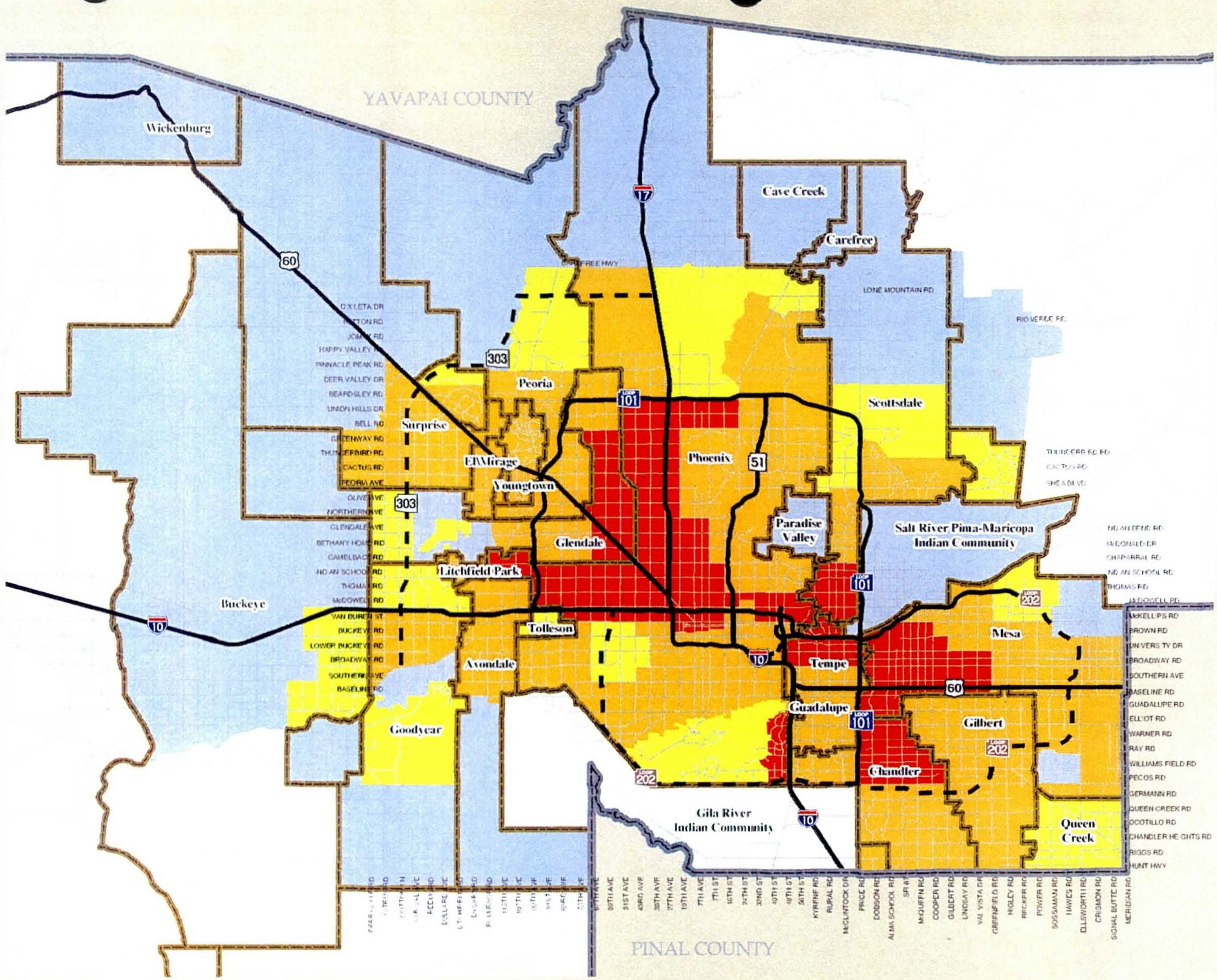
2020 Population Density for Interim Socioeconomic Projections by RAZ*



People Per Square Mile
(Maricopa County Average = 560)

- 0 - 50
- 50 - 1000
- 1000 - 2000
- 2000 - 5000
- More than 5000
- County Boundary
- MPA Boundaries
- Existing Freeways/Expressways
- Planned Freeways/Expressways
- Major Roads

MAP AREA

Miles: 0, 5, 10, 15

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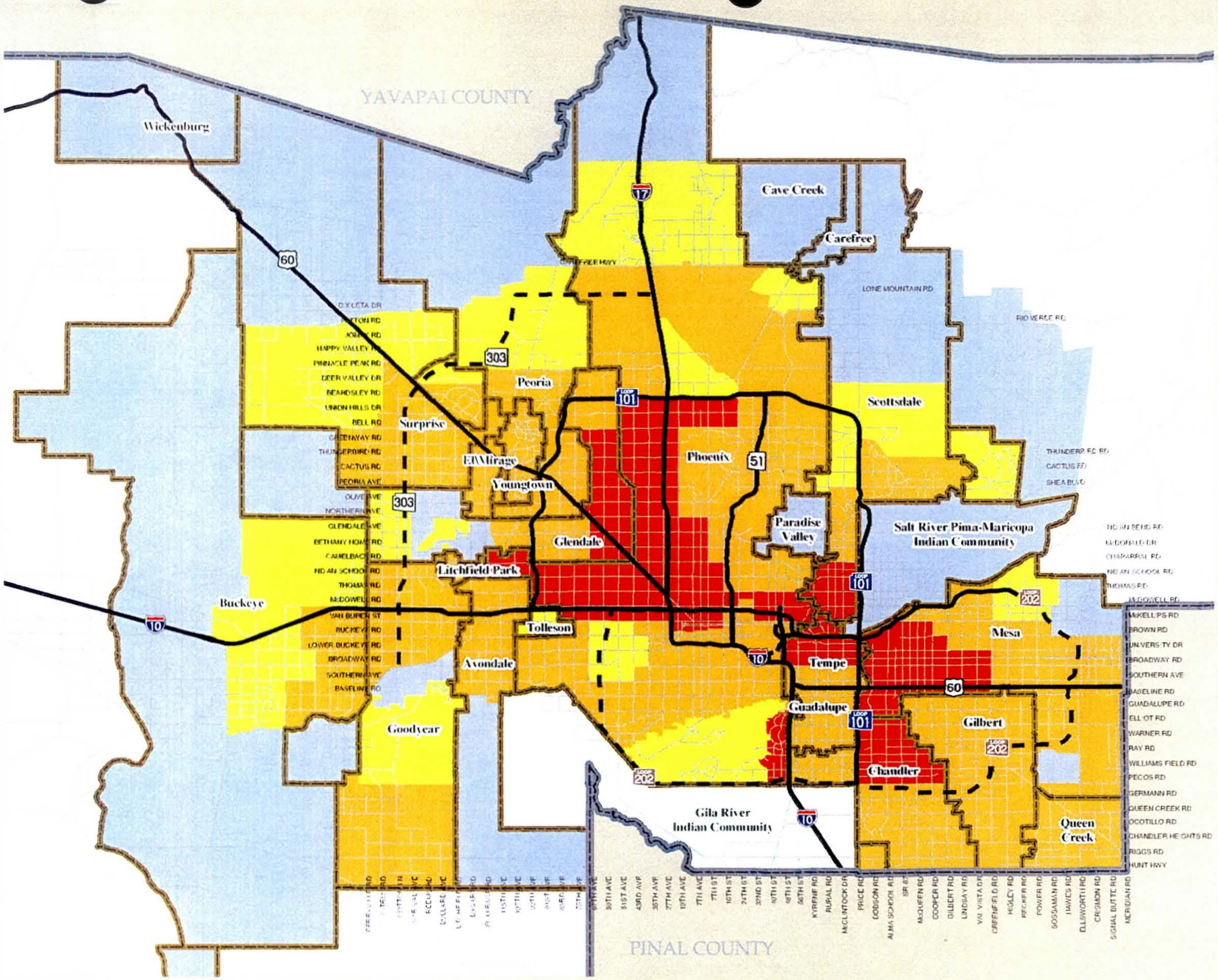
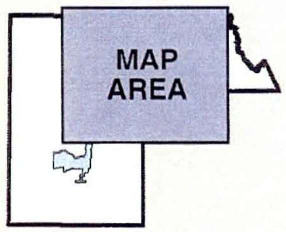
2030 Population Density for Interim Socioeconomic Projections by RAZ*



People Per Square Mile
(Maricopa County Average = 665)

- 0 - 50
- 50 - 1000
- 1000 - 2000
- 2000 - 5000
- More than 5000
- County Boundary
- MPA Boundaries
- Existing Freeways/Expressways
- Planned Freeways/Expressways
- Major Roads



0 5 10 15 Miles

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*Based on interim projections by Municipal Planning Area (MPA) and Regional Analysis Zone (RAZ) for 2010, 2020, 2025 and 2030 accepted by MAG Regional Council on June 25, 2003.

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