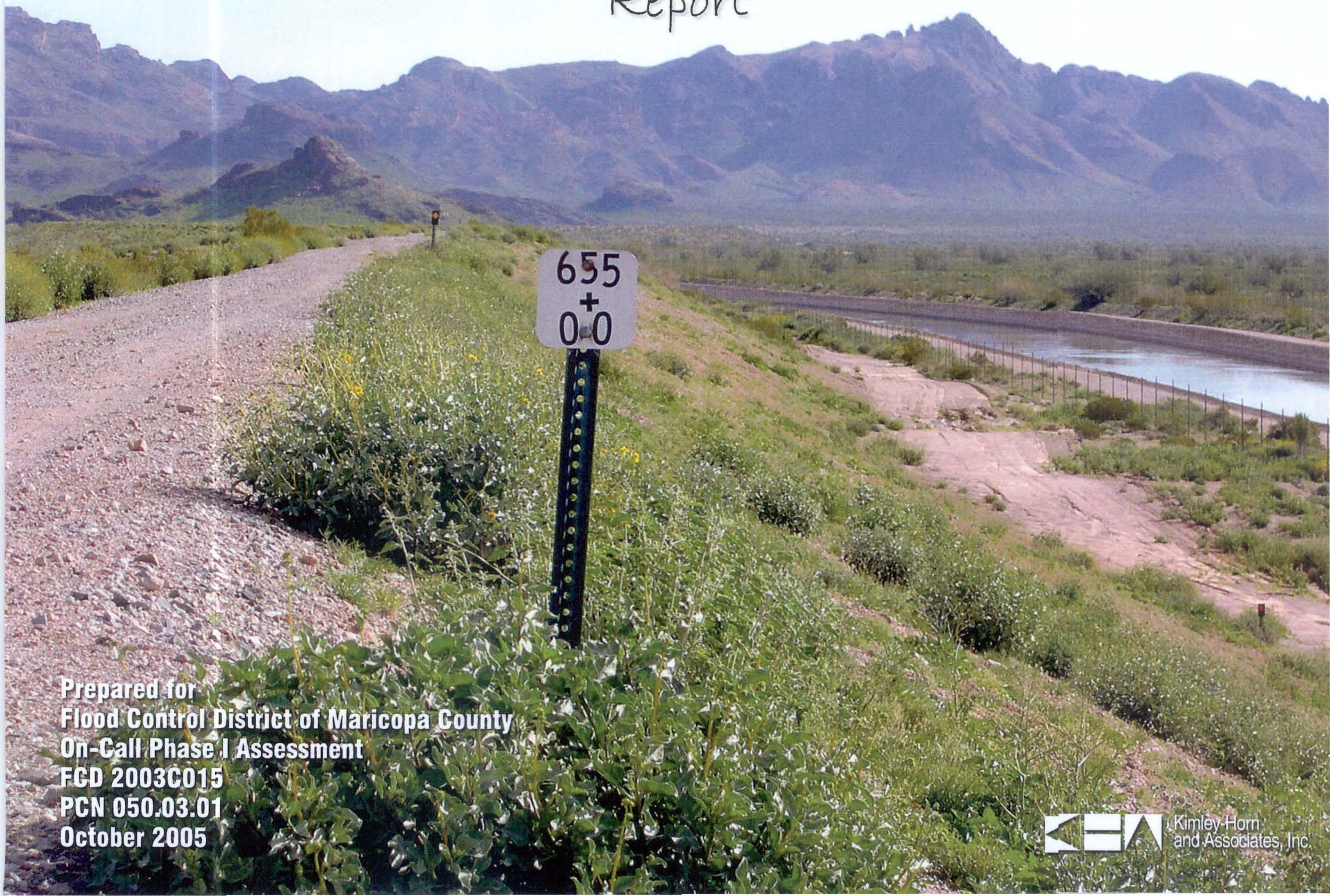





Harquahala
Flood Retarding Structure
Individual Structure Assessment Report



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Kimley-Horn and Associates, Inc.

HARQUAHALA FLOOD RETARDING STRUCTURE INDIVIDUAL STRUCTURE ASSESSMENT REPORT

**FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
FCD 2003C015**

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INDIVIDUAL STRUCTURE ASSESSMENT REPORT
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HARQUAHALA FLOOD RETARDING STRUCTURE

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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 **Harquahala 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 dams, 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 structures. A detailed visual field examination was 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 Harquahala 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

The Harquahala FRS is located in several sections of Township 3 North, Ranges 8-10 West and is about 75 miles west of downtown Phoenix, Arizona. The Harquahala FRS is located upstream of the Central Arizona Project (CAP) canal and about 2 miles upstream of I-10. The project consists of the Harquahala FRS embankment, a 48-inch diameter primary outlet (principal spillway) conduit, two 24-inch diameter auxiliary conduits and a 150-foot wide concrete baffled emergency spillway chute.

The Harquahala FRS reservoir has a capacity of 8,404 acre-feet. A permanent pool is not retained in the reservoir. The Harquahala FRS and reservoir are designed to detain the 100-year floodwater and store the impoundment for a slow release of approximately eight (8) days into the Harquahala Floodway. Reservoir capacity is then restored to detain future stormwater runoff events.

The Harquahala FRS is a 62,058-foot long (11.8 miles) homogeneous earthfill structure with 3:1 upstream slopes, 2:1 downstream slopes, and a crest width of 14 feet. The Harquahala FRS has a 5-foot wide central filter from Stations 452+00 to 717+00. The structure ranges from 1 to 45 feet above the existing ground. 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 dam cutoff trench is 20 feet wide and ranges from 1 to 15 feet below existing ground.

An inclined 5-foot wide central filter and drain was placed in the structure between stations 452+00 to 717+00. The central filter is 3 feet wide and the drain fill portion is 2 feet wide. There are drain outlets approximately every 400 feet between Stations 452+00 and 717+00. Each drain outlet has a cross-section that is 2 feet tall, with a top width of 2 feet, bottom width of 6 feet, and a 6-inch diameter perforated asbestos cement pipe to facilitate internal drainage beyond the toe of the embankment.

The maximum recorded impoundment for Harquahala FRS is 452 acre-feet with a stage of 21.5 feet at the Harquahala FRS ALERT gage in October 2000.

Watershed

The dam was constructed across a number of local drainage washes conveying runoff from the Big Horn and Harquahala Mountains. The total contributing drainage area for the Harquahala FRS is 102.3 square miles. The sediment storage requirement is 414 acre-feet which is the estimated sediment that will be supplied by the watershed over a 50-year period.

Flood Pool

The total floodwater storage capacity of the reservoir is 8,404 acre-feet up to the emergency spillway crest elevation of 1408.4 ft (NGVD29; as-built). This volume includes the sediment capacity of 414 acre-feet. The structure was designed to retain the floodwater runoff from the 100-year storm and discharge it through the principal spillway over a period of approximately eight (8) days. The total surface area of the sediment pool is 123 acres and the retarding pool is 1,231 acres

There are two short inlet channels, one each directing reservoir flow to the New Tank Outlet and Principal Spillway, respectively.

Principal Outlet Works

The primary outlet (principal spillway) is a 48-inch diameter reinforced concrete pressure pipe that conveys the principal spillway flow from the reservoir under the CAP canal to the Harquahala Floodway. The principal spillway is located at Station 1045+08 along the centerline of the Harquahala FRS.

The inlet is an NRCS standard open riser that is about 20 feet in height and includes a trashrack. The crest elevation of the principal spillway is 1387.3 ft (as-built) and the invert elevation of the 48-inch diameter conduit is 1367.3 ft. The outlet structure consists of a concrete chute with a Saint Anthony Falls stilling basin. There is an inlet channel that is designed to facilitate drainage to the principal spillway which is 560 feet long.

Instrumentation at the principal spillway includes a pressure transducer gage and rain gage (ALERT gage installed in 1988 as part the District's flood warning system) and a staff gage.

New Tank Diversion Outlet

The New Tank Diversion Outlet is a gated 24-inch diameter reinforced concrete pipe (RCP) and is located at Station 583+80. The diversion conveys local drainage over the CAP. The outlet has five anti-seepage collars and the conduit is encased in concrete up to the spring line. The invert elevation of the New Tank Outlet is set at 1196.0 ft. The inlet channel is 800 feet long and designed to convey flow to the inlet channel. The final design report indicates that the purpose of the New Tank Outlet is to permit the Bureau of Land Management to detain runoff if New Tank is full, or for any other management reason. The Harquahala FRS Operation and Maintenance Plan prepared by the SCS calls for the gate at New Tank, under normal operation, to be left in the open position.

Drain Outlet

There is a Drain Outlet at Station 746+00. It is also a gated 24-inch diameter RCP with a trashrack on the inlet that conveys local drainage under the CAP. The invert of the conduit is at 1383.86 ft. The 24-inch diameter conduit is also encased in concrete up to the spring line and has eight anti-seepage collars under the Harquahala FRS embankment and two anti-seepage collars under the south (downstream) embankment of the Granite Reef Aqueduct. There is a plunge pool at the outlet. The Drain Outlet was placed at the second lowest natural drainage location along the Harquahala FRS to drain as much of the reservoir as possible. The lowest natural drainage is at the principal spillway. The Drain Outlet was also placed at 746+00 to be a 'substitute principal spillway' because the Granite Reef Aqueduct is routed near the hills and could create a constriction in the flow of floodwater to the principal spillway. The Harquahala FRS Operation and Maintenance Plan prepared by the SCS calls for the gate at Drain Outlet, under normal operation, to be left in the closed position.

Toe Drains and Outlets

There are drain outlets approximately every 400 feet between Stations 452+00 and 717+00. Each drain outlet has a cross-section that is 2 feet tall, with a top width of 2 feet, bottom width of 6 feet, and a 6-inch diameter perforated asbestos cement pipe to facilitate internal drainage beyond the toe of the embankment. There are five additional toe drains placed approximately every 100 feet between Stations 790+00 to 795+00. There are six more toe drains placed approximately every 60 feet between Stations 1040+20 to1044+80 near the primary outlet. The drains are collected into one conduit that is under the CAP and discharges adjacent to the primary outlet stilling basin.

Emergency Spillway

The emergency spillway is a 150-foot wide concrete baffle chute located at Station 939+20. The vertical chute walls are 15 feet tall and drop the emergency spillway flow a total vertical depth of 20 feet. There are seven rows of energy dissipating baffles in the chute. The emergency spillway was designed to pass the PMF with no freeboard (NRCS criteria) and has a discharge capacity is 16,420 cfs. The emergency spillway will discharge into the Granite Reef Aqueduct immediately downstream of the FRS. There has been no emergency spillway discharges to date.

1.2 Hydrologic and Hydraulic Considerations

The NRCS designed the Harquahala FRS to detain the 100-year stormwater runoff volume. No comprehensive hydrologic analysis was found in the design report, but it is assumed the 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 as was the NRCS standard of the time. There was Technical Release TR-48 (DAMS2) output data in the final design report (NRCS 1980).

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. For a Class B structure, the PSH is based on the 100-year precipitation depth (P_{100}).

The TR-48 (DAMS2) output data found in the final design report that related to the principal spillway design included the Freeboard Hydrograph (FBH), Emergency Spillway Hydrograph (ESH), and Principal Spillway Hydrograph (PSH). During the late 1970's, when the structure was designed, the NRCS typically used TR-48 to develop the PSH, ESH, and FBH design storms and flood routing through the reservoir. The TR-48 output identified the principal spillway crest elevation, dam crest elevation, and emergency spillway that were on the as-built plans, so it was assumed this was the TR-48 analysis used for final design of the structure.

The principal spillway crest elevation is 1387.3 ft. Any impoundments below this level were designed to be removed from the reservoir through an 18-inch diameter drawdown gate that is operated from the crest of the structure. In the TR-48 output, the PSH maximum reservoir water surface elevation was 1408.49 ft., the maximum discharge through the principal spillway was 379 cfs, and the drawdown time for a full reservoir was estimated to be 8.8 days.

The estimated 50-year sediment volume was identified as 414 acre-feet in the Watershed Work Plan and was used in the TR-48 analysis. If the 50-year sediment volume were distributed evenly along the entire length of the dam, the top of the sediment pool would be at 1396.2 ft. But, in the supplement to the review of the final design report it is noted that because the dam is 11.5 miles long, the structure would act more like a diversion and the sediment will not be deposited in a level pool at the lowest area of the reservoir.

A significant portion, 80%, of the drainage area and sediment contribution will come from the west half of the dam. The final design report supplement review indicates that the estimated top of sediment pool varies from 1399.0 ft. on the west end of the dam to 1387.3 ft. at the principal outlet, which is the crest elevation of the principal outlet. At the New Tank Outlet, the top of the sediment pool was estimated to be 1396.21 ft. The elevation of the inlet channel for the New Tank Outlet was set at 1396.22 ft. The Drain

Outlet and principal spillway are 3.1 and 8.7 miles away from the New Tank Outlet, so the assumption that there will not be a level sediment pool appears to be valid.

The Drain Outlet invert was placed low to drain as much of the reservoir as possible. The drain was placed at 746+00 to be a 'substitute principal spillway' because the Granite Reef Aqueduct (or CAP) is routed near the hills and could create a constriction in the flow of floodwater to the principal spillway. The design report indicates that the top of the sediment pool at this location would be 10 feet above the invert of the Drain Outlet.

The emergency spillway is a 150-foot wide concrete baffle chute with the centerline of the spillway at Station 939+20 on the dam. The vertical chute walls are 15 feet tall and emergency spillway conveys the discharges through a vertical drop of 20 feet. There are seven rows of energy dissipating baffle blocks in the chute. The emergency spillway was designed to pass the FBH with no freeboard. The emergency spillway crest elevation is at 1408.4 ft (as-built). The dam crest elevation at the emergency spillway is 1419.7 ft., so there is 11.3 feet of freeboard above the emergency spillway.

According to standard NRCS design, the FBH is used to establish the minimum settled elevation of the top of the dam. It is also used to evaluate the structural integrity of the spillway system. For a Class B hazard structure, the FBH is based on a design storm precipitation depth that is derived from a combination of the 100-year precipitation depth, P_{100}, and the probable maximum precipitation (PMP). The FBH is equal to P_{100} + 0.4*(PMP- P_{100}). The Harquahala FRS was identified as a Class B structure by NRCS, but there is some indication by ADWR that the design standard may have been higher than a Class B (ADWR, 1981).

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})}. The Watershed Work Plan and preliminary design review report indicated that the emergency spillway would be located near the east abutment of the structure, but this was changed to its current location during final design. The final design report indicates that the emergency spillway was placed at its current location to take advantage of known rock location, meaning the entire invert of the exit channel is founded on rock.

In routing the FBH and ESH through the reservoir the NRCS assumed that the principal spillway will be operable and would not clog with debris. In the "Final Design Report" the NRCS conducted an analysis to examine the impact of principal spillway inlet clogging on the ESH and the FBH. Under the Freeboard Hydrograph scenario, the maximum reservoir water surface elevation was computed to be 1419.8 ft. (note: the dam crest elevation is 1419.7ft), the maximum reservoir storage would be 28,352 acre-feet, and that the maximum discharge through the emergency spillway would be 16,425 cfs.

In the Final Hydrologic Design Review Memorandum (ADWR, 1981), ADWR reviewed an NRCS hydrology report that could not be located during the data collection phase. In the memorandum, ADWR indicates that TR-20 was used to generate inflow hydrographs for the 6, 24, 36, 48, and 72-hour duration storms and that TR-48 was used for flood routing through the spillways. ADWR also stated that the structure was designed by using an average of the NRCS class 'B' and 'C' precipitation depth requirements for the design storm. The precipitation depth in the design storm for a class 'C' structure is the PMP and, as stated, for class 'B' is $P_{100} + 0.4*(PMP - P_{100})$. The average of these two precipitation design depths is $0.3* P_{100} + 0.7*PMP$.

Spillway Inundation Study. An emergency spillway delineation study was completed for the Harquahala FRS in June 1998 by Dibble and Associates. Spillway inundation routing is provided for the PMF discharge through the emergency spillway of 15,000 cfs, the 2/3 PMF of 10,000 cfs, and 1/3 PMF of 5,000 cfs. The study extends from the emergency spillway to Interstate-10, which is approximately 2.8 miles downstream of the emergency spillway structure. The study was completed using the Corps of Engineers HEC-RAS steady-state one-dimensional analysis program.

The results of the study show that I-10 could be overtopped by approximately 3 feet and 2.3 feet during the 15,000 cfs and 10,000 cfs emergency spillway discharge events, respectively. The study showed that the maximum water surface elevation would be right at the top of the roadway embankment during the 5,000 cfs event.

The study recognized the inherent variability that could be associated with emergency spillway discharges at the Harquahala FRS. Discharge could be conveyed in the CAP canal downstream of the structure and breakout at locations away from the emergency spillway. Failure of the CAP could also potentially increase the maximum discharge being conveyed downstream. Breakout over the CAP could also erode or scour the earth embankment and cause the CAP to fail and increase the amount of discharge being conveyed downstream. With erosion of the downstream channels the split flows identified in the study could change the magnitude, direction, and velocity of discharges. Consequently, the study recommended that a very generous evacuation limits be implemented downstream of the emergency spillway in the emergency action plan.

Dambreak Analysis. A dambreak and dambreak flood routing analysis was completed for the Harquahala FRS by Carter Associates in February 1991 on behalf of the District. The analysis included a full PMP hydrologic assessment using HMR-49 and 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 study found that the Harquahala FRS would not be overtopped during the ½ PMF, so the only dambreak failure mode studied was from internal embankment erosion or piping. The study developed the PMF analysis for the 6-hour and 72-hour duration storm event on the watershed and then routed the ½ PMF flood (inflow design flood) through the reservoir to determine the maximum reservoir water surface elevation and to develop dambreak parameters.

The study indicated that the 72-hour storm was the most critical for the DAMBRK analysis since the 72-hour delivered the largest runoff volume to the reservoir and produced the highest reservoir water surface elevation for the ½ PMF event.

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 and 72-hour ½ PMF events even though a 'sunny day' failure (piping) was the only failure mode evaluated. The dam breach was initiated at the maximum reservoir water surface elevation. Three breach locations were chosen for the structure. One in the east of the structure, one in the middle, and one on the west end. The elevation at which piping was initiated was half the distance between the crest of the dam and bottom of the reservoir.

The dambreak breach parameters and resulting dambreak peak discharges appear to be unrealistic (see **Table 9, Appendix Tables**). The maximum height of the dam is 45 feet and typically, the maximum breach width averages between 3-4 times the height of the structure, or 135-180 feet. The peak discharges also appear to be high given the flood pool volume that is available. Also, the breach occurs during the peak of the ½ PMF inflow 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.

KHA recommends that an updated dambreak analysis and inundation mapping be prepared for the Harquahala 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 sunny day failure (full pool) should be considered without inflow and the ½ PMF should be evaluated with an empty pool as initial conditions.

1.3 Geologic and Geotechnical Considerations

Geologic Setting. The Harquahala 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 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.

Numerous ephemeral stream channels that contain loose, unconsolidated and pervious coarse grained soil cross the Harquahala FRS alignment. According to the NRCS, the presences of these deposits could pose a threat to the foundation unless the cutoff trench interrupts the continuity of these deposits. Where these soils could not be over-excavated and removed, portion of the upstream borrow area were blanketed with compacted fill.

The blanketing was used from Stations 484+00 to 485+00, 511+50 to 513.30, 593+00 to 594+00, and 850+00 to 852+00 (NRCS Sheet 18 of 55; As-Built Drawing, May 13, 1983).

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 Harquahala 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. 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 Harquahala 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 year. This fault may be capable of producing quake with a maximum calculated magnitude of 5.7, producing a maximum calculated peak horizontal acceleration at the Harquahala FRS equal to 4 percent of the gravitational acceleration (g). The recommended peak horizontal acceleration design criteria calculated by AMEC for the Harquahala 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 Harquahala 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 (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).

Study Area Subsidence Historic National Geodetic Survey (NGS) level line data is not available in the vicinity of the Harquahala 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. The data may be used as an indicator of the relative recent land subsidence that may have occurred or is occurring in the project area. The data indicates the change in elevation is greatest along Reach 2 of the Harquahala FRS where there is an apparent thickening of the basin fill sediments beyond the area where the buried bedrock surface drops-off from the edge of the pediment.

According to this data, it appears that some settlement or subsidence has occurred, mainly on the western portion of the dam between monuments A-1 and A-36, from 1984 to 2003. The change in elevation in this area ranges from -0.015 to -0.480 feet. The eastern portion of the dam has not experienced any apparent settlement or subsidence because along this portion of the alignment, bedrock is relatively close to the surface.

Known Earth Fissures in the Project Vicinity There are three earth fissures reported in the Harquahala Valley. The closest fissure to the Harquahala FRS lies approximately 3.4 miles southwest 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.

Another earth fissure was documented in 1961 in a farm field about 4.8 miles south of the Harquahala FRS 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.

The Rogers fissure was discovered in 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 5.9 miles southwest 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 Harquahala FRS.

Foundation Conditions The foundation materials beneath the Harquahala FRS were differentiated in the Geologic Investigation Summary Report (SCS, 1978) into two reaches on the basis of a distinct change in subsurface conditions. The east end of the structure is underlain by coarse-grained gravels while the west end is underlain by finer-grained sands and silts. The change from coarse- to fine-grained materials occurs between Stations 722+00 and 712+00. The eastern end of the alignment was designated Reach 1 (Station 717+00 to Station 1054+20) and the western end, Reach 2 (Station 431+00 to Station 717+00). This distribution of materials, with coarse-grained material along the eastern portion of the alignment and finer-grained material with increasing distance to the west, is typical of alluvial fan deposits.

Reach 1 (EAST)

Sandy silty gravels to silty sandy gravels (GM to GP-GM) predominate throughout Reach 1. These coarse sediments are typical of upper alluvial fan deposits and reflect the location of Reach 1 with respect to the Big Horn Mountains. Although caliche was not found to be significant in most of the borings completed during the geologic investigation for the Harquahala FRS, it is fairly common in this environment and may be locally extensive, though it is not uniformly widespread (SCS, 1978).

The dam alignment crosses the upstream tip of a small rhyolite knoll at Station 938+00. According to the Supplemental Geologic Investigation Report of Emergency Spillway Site, the emergency spillway location was changed from its' original location to the as-built location in order to use the rhyolite bedrock as the spillway foundation.

Reach 2 (WEST)

Reach 2 consists of a heterogeneous mixture of fine-grained materials resulting from alluvial deposition, mudflows and floodplain splays. The subsurface is predominated by silty sand (SM), however thin, stratified layers of silty to clayey sands, sandy to clayey silts, silty to sandy clays, and gravelly sands to silty gravelly sands were observed during the geologic investigation. It was noted by the SCS (1978) that there was no widespread consistency or uniformity either between test pits (or drill holes) or vertically in individual test pits. It was also reported that buried channel sand deposits may potentially be found all along Reach 2 in the top 15 to 20 ft.

During design, it was recognized that the soils in Reach 2 had high collapse potential. In Supplement 1 to the Final Design Report, the laboratory collapse testing results from twenty-seven soil samples from Reach 2 were reported. Percent collapse for these samples was reported to range between 0.7% and

17.3%. Removal and re-compaction of the severely collapse-prone silts, silty sands and low plasticity clays in the upper five to ten ft was required by the designers. Sheet No. 16 in the as-built drawing set indicates that stripping to various depths up to 9 ft was performed prior to construction of the embankment (see page 1-7).

Embankment The design of the embankment explicitly accounted for the differences in foundation conditions in Reach 1 and Reach 2. The Harquahala FRS was constructed as a homogeneous embankment (Zone I; 3H:1V upstream and 2H:1V downstream) with a cutoff trench. An inclined filter/drain (Zone II) was installed in Reach 2 (Station 450+00 to Station 717+00) to mitigate the effects of transverse cracking in the fine-grained embankment materials in this reach (SCS, 1979). The design included toe drains at two locations where the drilling logs indicated channels were present at depths greater than the cutoff trench (SCS, 1980b). The as-built plans indicate that toe drains were installed between Station 790+00 and Station 796+00 and between Station 1040+20 and Station 1044+80. Seepage from the toe drains is collected in a series of 8-in laterals which feed into 10-in collector pipes that ultimately divert the seepage into the CAP canal. In Reach 1, the embankment design relied on the coarse-grained embankment material to resist cracking which by its nature is not prone to piping (SCS, 1980b).

Embankment Construction Difficulties during construction of the filter/drain in Reach 2 resulted in a variable width filter/drain zone that was at times uneven and less than the specified 2 ft wide drain zone, according to construction inspection reports. The Dam Construction Inspection Reports indicated that the contractor had difficulty with filter/drain material placement in Reach 2 and that the filter/drain materials were contaminated with embankment materials. The contractor was not required to perform repair work at the time of construction, suggesting that the contamination was not significant enough to effect the functionality of the filter/drain.

In Reach 1, the dam was constructed with soil containing less than the specified amount of fine-grained soil (> 15% passing the No. 200 sieve) in the upstream section (between Stations 717+00 and 1054+00). The borrow materials available for embankment construction generally contained fewer fines than the specified 15%. According to the CAP Construction Progress Report 50 out of 58 soils tests conducted during embankment construction (at 50% to 60% of construction completed) contained less than 15% fines and 32 of the samples testing contained less than 10% fines. USBR concluded that due to the limited quantity of fines in Reach 1, it was virtually impossible to "blanket" the upstream portion of the dam with materials having the specified fines content. Although it is unclear exactly what was meant by "blanket" it is evident that there are less than 15% fines at some locations on Reach 1.

The concern regarding the lack of fines on the upstream slope was the impact this would have on slope stability. To assess this, permeability testing was performed on soil samples. Permeability test results were used to calculate the time required to fully develop a phreatic line in the dam for comparison to expected impoundment time. The results were verified by in-field constant head permeability tests at three locations in

Reach 1 and it was concluded that the time required to saturate the embankment would be significantly longer than the expected impoundment time, therefore embankment instability due to the lack of fine-grained soil in the upstream section should not be a concern (SCS, 1985).

Compatibility of Zone II Drain Fill as Filter for Zone I An inclined filter/drain system (Zone II) was installed in Reach 2, primarily to protect against potential internal erosion and piping of embankment materials in the event of transverse crack development. The top of the filter/drain is 7 ft below the crest of the embankment at the centerline. The filter/drain is shown on the as-built plans as a 2-foot wide filter upstream of a 3-foot wide drain.

The original design specification band falls within the NRCS permeability criteria. The original design specification band is slightly coarser than the NRCS filtering criteria and may not achieve the recommended filtering limit for the finest base soils. However it appears that at least a portion of the as-built filter was placed within the NRCS criteria. It is possible that some fines from Zone I could penetrate into Zone II under a concentrated leak through a transverse crack, if the Zone II materials were graded on the coarse band in accordance with the specified gradation limits. Considering the variability in gradation of the Zone I materials, and the fact that Zone II meets the criteria except for the finest base soil and coarsest filter possibilities, it is likely that Zone II is providing adequate filter protection. Additional analyses may be done to further evaluate the efficacy of the Zone II filter.

Embankment Settlement The SCS designers recognized the potential for collapsible soils and associated settlement in Reach 2 and performed consolidation testing to evaluate collapse potential during the preliminary design phase. Consolidation testing was conducted under a 2,000 pounds per square foot (psf) load. Review of the preliminary design indicated that collapse potential should be evaluated for actual loads, which were expected to exceed 2,000 psf. Reported collapse potential for these additional tests ranged from 0.6% to 17.3%. The crest elevation in Reach 2 was designed to be one and a half feet higher than in Reach 1 to allow for foundation settlement after construction.

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 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 3 miles downstream of Harquahala FRS and runs parallel 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.

The major significant change under future land use is that the agriculture and vacant lands changes to single family residential. The residential land use changes to completely encompass Harquahala 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, signs of distress in the form of confirmed transverse and longitudinal cracking have been identified on Harquahala FRS.

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 relative to Harquahala FRS. An EAP for Harquahala 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 Harquahala 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 Harquahala FRS.

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

- Failure Due to Transverse Cracks Through Dam that Extend Through Crest Above Filter/Drain in Reach 2 Leading to Internal Erosion and Breach at Location of Crack During Impoundment (Category I; may go to Category II).
- Overtopping During Major Flood Event (Category I, may go to Category II).
- Failure Associated with Piping along the Principal Outlet Leading to Undermining and Breach of Dam at the Location of the Principal Outlet (Category II).
- Failure Due to Piping along the Eastern 24-inch Drain Outlet (Station 746+00) that Leads to Undermining and Breach of Dam during Impoundment (Category II).
- Failure Due to Transverse Cracks through Dam and Filter/Drain in Reach 2 that Leads to Internal Erosion and Breach at Location of Crack during Impoundments. (Category II).
- Potential Adverse Consequences Resulting from Emergency Spillway Discharges during Major Rainfall Events (Category II).

The qualitative risk of the overtopping failure mode ranges from low likelihood, medium consequences (for the PMF event) to medium likelihood, low consequences for higher frequency overtopping storms. The range of risk for this failure mode is dependent on the storm frequency, magnitude, and downstream consequences. The potential failure mode related to transverse cracks through dam that extend through crest above filter/drain in reach 2 leading to internal erosion and breach at location of crack during impoundment has a risk range of medium likelihood and 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 the emergency spillway inundation study be updated. The study should be extended south to Centennial Wash and account for the effects of the CAP canal and the I-10 Interstate embankment and culverts. The study should consider using a dynamic unsteady flow model such as the unsteady flow option in HEC-RAS.

(2) Kimley-Horn recommends that an updated dambreak analysis and inundation mapping be prepared for the Harquahala 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 sunny day failure (full pool) should be considered without inflow and the ½ PMF should be evaluated with an empty pool as initial conditions.

(3) A quantitative risk assessment for the dam 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).

(5) Verify BLM easements have been recorded with County Assessor.

(6) Conduct an updated sediment yield study for the Harquahala FRS watershed.

(7) Potential for Harquahala - Centennial Levee System Alternative. Potential to Install Second Outlet at West End. Since a majority of the drainage area is located at the western reach of the dam and hence most of the inflow from storm events, an opportunity was identified for construction of an additional principal spillway and floodway. The floodway would convey flows from the west principal spillway to Centennial Levee. Centennial Levee, in turn, would direct flows to Centennial Wash. The concept for this alternative could be extended to evaluate the potential for a second emergency spillway located in the western reach of the dam. In this fashion Harquahala FRS would be furnished with two sets of principal and emergency spillways. This concept, at some future time, could segment the dam into two smaller structures. It should be noted that the NRCS in their March 1979 memorandum indicated that the agency looked at an

emergency spillway at the west abutment of the dam. This location was ruled out by the SCS as a hydraulic analysis indicated that emergency spillway flows would cause the Bureau of Reclamation's Tiger Wash Detention Structure to overtop.

(8) Kimley-Horn recommends that a site-specific PMP be conducted for Harquahala FRS. Site specific Probable Maximum Precipitation studies in Arizona have resulted in PMP values lower that those resulting from using the HMR-49 procedures. Harquahala FRS is an 11-mile long dam with significantly different contributing sub-basins from the west end to the east end of the dam. The west reach of the dam contributing drainage area is from alluvial plains and fans. The eastern reach contributing drainage area is from the Big Horn Mountains.

(9) Need PMF Routing. The Probable Maximum Flood or the ½ PMF have not been routed through the impoundment. A review of the project hydrologic and hydraulic records indicate that the dam was designed as an average between the design storm for the Class B and Class C criteria. The March 19, 1981 ADWR memorandum indicates that the Safety of Dams inflow design-flood for the dam should be the ½ PMF. The memo states that the ½ PMF has not been routed through the dam.

(10) Recommend Dynamic Routing to be conducted. Harquahala FRS is an 11-mile long dam that functions as a diversion/levee system. Flows are collected along the dam from many contributing streams. The timing of the hydrographs from these streams will result in a sloping water surface for the reservoir from the west end of the dam toward the east end of the dam. Normal hydrologic routing used in HEC-1 uses the modified Puls routing method that results in a level pool (level pool routing). The use of an unsteady flow model, such as the dynamic capabilities of the HEC-RAS model, will provide the water surface profile/elevation of the inflow design flood (and other storm events) for Harquahala FRS. Knowing the actual water surface profile along the dam will provide the opportunity for determining residual freeboard and the potential for overtopping.

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 in Reach 2 was evaluated for this Phase I Structures Assessment. No information regarding the gradation of the drain materials in the filter/drain were available for review. It is recommended that drain gradation data be obtained, either by review of additional data not available during this Phase I Structures Assessment or by field sampling, and checked for compatibility with the filter materials.

Phase II Documentation of Slope Stability and Seepage Analyses

Under reasonable loading conditions for Harquahala 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 Harquahala FRS.

The original stability analysis does not completely document factors of safety for all the loading conditions required under current NRCS or ADWR criteria.

(1) **End of construction (downstream slope)**: The original factor of safety calculated for this loading condition in Reach 2 (1.23) did not achieve the minimum ADWR criteria of 1.3. Additional analyses, including confirming the shear strength of Reach 1 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 Reach 1 downstream slope are recommended to document the stability of the downstream slope.

   a. **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. However, the original analyses were conducted by assuming the full development of a phreatic surface in the upstream slope. Analyses conducted during Phase I studies for other flood retarding structures in Maricopa County illustrate that a steady state phreatic surface may not develop in dry dams under multiple temporary impoundment events. Therefore, additional analysis of upstream slope stability under rapid drawdown conditions is not necessary.

(2) **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.

(3) **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 is 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.

(4) **Steady state seepage with seismic forces (downstream slope)**: A seismic stability analysis was only documented for Reach 2. To document seismic stability for Reach 1 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. Additional Recommendations from Inspection Report and FEMA 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) Toe Drain Outlets: Toe drain outlets were located in the field between Stations 452+00 to 717+00. The toe drain at Station 467+72 was located and visually inspected at the downstream outlet. Photograph No.6 in the dam safety inspection report provides a photo of the interior of the six-inch perforated asbestos cement pipe. It was noted that the first length of six-inch pipe appears to be offset at the first pipe joint, that the pipe wall has failed, and that the pipe has rotated slightly. It is recommended that this section of pipe be replaced and the drain fill material be replaced around the pipe section. The replacement of the pipe should follow the original detail provided in the as-built plans on Sheet 16. Perforated PVC pipe may be used instead of the asbestos cement pipe. The damaged pipe should be excavated to 2 feet beyond the first pipe joint. The new pipe may be joined to the existing pipe using a MAG standard pipe collar detail. The drain fill materials may be placed around the new pipe section and the excavation trench backfilled with compacted dam embankment materials.

(3) Borrow Area Blankets: The two foot thick borrow area blanket was placed in borrow areas between the following stations: Station 484+00 to 485+00; Station 511+50 to 513+50; Station 593+00 to 594+00; and Station 850+00 to 852+00. It should be noted that the cutoff trench depth was 9-feet between Station 511+50 to 512+50 and was 5-feet elsewhere. It is recommended that the borrow areas that have borrow area blankets be plainly mapped on the as-built plans and that during future regular dam safety inspections inspect the blanket areas to look for signs of illegal excavation or other deleterious factors that may impact the function of the blankets.

(4) Update Emergency Action Plan. Develop an Emergency Action Plan to meet FEMA 64 and ADWR requirements.

(5) Repair the damaged survey monument A28.

(6) Continue active vegetation management program.

(7) Remove sediment and any obstructions in the central filter drain outlet conduits.

(8) Complete a survey to compare the downstream embankment profile to the as-built plans as it appears the toe may have been cut back in several locations.

(9) Conduct a video inspection of central filter drain outlet conduits.

(10) Conduct a video inspection of the toe drain outlet system at the principal spillway.

(11) Add a staff gage and/or ALERT station at the west end of the dam.

(12) Repair toe drain outlet at Station 467+72.

(13) Locate peizometers using as-builts and field markers on dam. Confirm location and mark on set of as-built plans. Abandon peizometers per ADWR groundwater well abandonment guidelines.

(14) 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.

(15) All penetrations without filter diaphragms need more attention. Penetrations through the embankment that do not have a filter diaphragm should be evaluated to determine whether one is needed. Foundation soils in Reach 1 are different from the soils in Reach 2, and the need/effectiveness of a filter diaphragm will differ depending on those conditions.

(16) CAP Canal Elevation Data and Performance Records Need to be Reviewed. The CAP canal and canal embankment provides a measure of mitigation for potential dambreaks or overtopping events. Also the canal has a concrete lining to reduce infiltration losses. CAP canal survey data should be obtained to evaluate elevation changes that may be signs of settlement or regional land subsidence. The CAP canal inspection and maintenance records should also be reviewed to ascertain potential canal lining crack repairs. Cracking of the concrete lining could be an early indicator of regional settlement and land subsidence.

(17) Confirm the operation of the gated outlets (New Tank and Drain Outlet) for normal pool impoundments with the Harquahala FRS Operation and Maintenance Plan prepared by the SCS.

2.0 DESCRIPTION OF DAM

The Harquahala Flood Retarding Structure (FRS) is a structural plan element of the Harquahala Valley Watershed Work Plan for the Harquahala Valley Watershed, Maricopa and Yuma (now La Paz) 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 (District) 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 the additional assistance of the Arizona Department of Water Resources (ADWR), formerly the Arizona Water Commission. 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 in Maricopa and La Paz Counties between the Harquahala, Big Horn and Saddleback Mountains and a broad alluvial plain that drains toward Centennial Wash. The total original watershed area was over 374 square miles (NRCS 1967).

2.1 Purpose of Dam

The Harquahala FRS is one of two flood retarding structures measures designed and constructed under the Watershed Work Plan. The other structure is the Saddleback FRS. The purpose of the flood retarding structures are to provide flood protection and erosion control benefits to over 19,000 acres of farmland in the Harquahala Valley, as well as agricultural infrastructure, the Granite Reef Aqueduct (also known as the Central Arizona Project canal), Interstate-10, county roads, the El Paso Natural Gas Line, the AT&T line, and some residential and commercial properties. The Harquahala FRS was designed to control runoff for the 100-year storm event. (Note: the Granite Reef Aqueduct will be referred to as the Central Arizona Project (CAP) canal for the remainder of the report).

The Harquahala FRS was constructed under the supervision of the NRCS in 1983 and under the local sponsorship of the Flood Control District of Maricopa County and the Wickenburg and Buckeye-Roosevelt Natural Resource Conservation Districts. NRCS produced the design calculations, plans, and contract documents for the Harquahala FRS and Floodway. The Harquahala FRS and Floodway were constructed under the same contract as the adjacent reach of the CAP canal with construction supervision by the United States Bureau of Reclamation.

2.2 Dam Location

The Harquahala FRS is located in several sections of Township 3 North, Ranges 8-10 West and is about 75 miles west of downtown Phoenix, Arizona. The Harquahala FRS is located upstream of the GRA and about 2 miles upstream of I-10. **Figure 1 (Appendix Figures)** shows a location map of Harquahala FRS. The project consists of the Harquahala FRS embankment, a 48-inch diameter primary outlet (principal spillway) conduit, two 24-inch diameter auxiliary conduits and a 150-foot wide concrete baffled emergency spillway chute.

Harquahala FRS collects runoff from the southern and eastern slopes of the Harquahala and Big Horn Mountains and the western slope of Burnt Mountain. The Harquahala FRS primary outlet discharges into the Harquahala Floodway, which empties into the Saddleback FRS reservoir. Flow is then routed through the Saddleback FRS reservoir and into the Saddleback Diversion channel with the ultimate discharge being Centennial Wash.

The Harquahala FRS reservoir has a capacity of 8,404 acre-feet. A permanent pool is not retained in the reservoir. The Harquahala FRS and reservoir are designed to detain the 100-year floodwater and store the impoundment for a slow release of approximately eight (8) days into the Harquahala Floodway. Reservoir capacity is then restored to detain future stormwater runoff events.

2.3 Physical Features

The Harquahala FRS is a 62,058-foot long (11.8 miles) homogeneous earthfill structure with 3:1 upstream slopes, 2:1 downstream slopes, and a crest width of 14 feet. The Harquahala FRS has a 5-foot wide combination central filter/chimney drain from Stations 452+00 to 717+00. The structure ranges from 1 to 45 feet above the existing ground. The volume of fill used for the structure is approximately 4,428,000 cubic yards. The dam crest elevation varies as shown in **Table 1 (Appendix Tables)**. 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 stationing of the dam is based upon stationing of the CAP canal. The dam cutoff trench is 20 feet wide and ranges from 1-15 feet below existing ground.

An inclined 5-foot wide combination central filter and drain was placed in the structure between stations 452+00 to 717+00. The central filter is 3 feet wide and the drain fill portion is 2 feet wide. There are drain outlets approximately every 400 feet between Stations 452+00 and 717+00. Each drain outlet has a cross-section is 2 feet tall, with a top width of 2 feet, bottom width of 6 feet, and a 6-inch diameter perforated asbestos cement pipe to facilitate internal drainage beyond the toe of the embankment.

There are five additional toe drains placed every 100 feet between Stations 790+00 to 795+00. There are six more toe drains placed every 60 feet between Stations 1040+20 to 1044+80 near the primary spillway outlet. This system discharges adjacent to the principal spillway outlet structure.

The maximum recorded impoundment for Harquahala FRS is 452 acre-feet with a stage of 21.5 feet at the Harquahala FRS in October 2000. **Table 2 (Appendix Tables)** provides a summary of the physical data for Harquahala FRS.

2.3.1 Watershed

The dam was constructed across a number of local drainage washes conveying runoff from the Big Horn and Harquahala Mountains. The total contributing drainage area for the Harquahala FRS is 102.3 square miles. The sediment storage requirement is 414 acre-feet which is the estimated sediment that will be supplied by the watershed over a 50-year period. The sediment yield rate computed using the above values is 0.081 acre-feet/mi^2/year

2.3.2 Flood Pool

The total floodwater storage capacity of the reservoir 8,404 acre-feet up to the emergency spillway crest elevation of 1408.4 ft (NGVD29; as-built). This volume includes the sediment capacity of 414 acre-feet. The structure was designed to retain the floodwater runoff from the 100-year storm and discharge it through the primary outlet over a period of approximately eight (8) days. The total surface area of the sediment pool is 123 acres and the retarding pool is 1,231 acres.

There are two short inlet channels, one each directing reservoir flow to the New Tank Outlet and Principal Spillway, respectively.

2.3.3 Principal Outlet Works

The primary outlet (principal spillway) is a 48-inch diameter reinforced concrete pressure pipe that conveys the principal spillway flow from the reservoir under the CAP to the Harquahala Floodway. The principal spillway is located at Station 1045+08 along the centerline of the Harquahala FRS. The Harquahala Floodway crosses I-10 and discharges into the Saddleback FRS Reservoir. The principal spillway has 10 anti-seepage collars that are 9.83 feet tall and 11.5 feet wide under the Harquahala FRS embankment and another 3 anti-seepage collars under the south embankment (downstream side) of the Granite Reef Aqueduct. The 48-inch diameter conduit is encased in concrete up to the spring line.

The inlet is an NRCS standard open riser that is about 20 feet in height and includes a trashrack. The crest elevation of the principal spillway is 1387.3 ft (as-built) and the invert elevation of the 48-inch diameter conduit is 1367.3 ft. The outlet structure consists of a concrete chute with a Saint Anthony Falls stilling basin. The inlet also has an 18-inch diameter drawdown gate to drain the reservoir below elevation 1387.3 ft. The crest elevation of the gate well assembly that covers the 18-inch diameter gate is 1382.3 ft., five (5) feet below the principal spillway crest, and the invert of the 18-inch gate is 1373.3 ft. There is an inlet channel that is designed to facilitate drainage to the principal spillway which is 560 feet long.

Instrumentation at the principal spillway includes a pressure transducer gage and rain gage (ALERT gage installed in 1988 as part the District's flood warning system) and a staff gage.

2.3.4 New Tank Diversion Outlet

The New Tank Diversion Outlet is a gated 24-inch diameter reinforced concrete pipe (RCP) and is located at Station 583+80. The diversion conveys local drainage over the CAP. The portion that crosses the canal is steel pipe. The outlet has five anti-seepage collars and the conduit is encased in concrete up to the spring line. The invert elevation of the New Tank Outlet is set at 1196.0 ft. The inlet channel is 800 feet long and designed to convey flow to the inlet channel. The invert of the inlet channel for the New Tank Outlet is set at 1396.22 ft., which is just above the estimated top of the sediment pool (1396.21 ft). There is also a trashrack incorporated as part of the inlet structure. The outlet structure is a standard impact type basin with a baffle wall. The final design report indicates that the purpose of the New Tank Outlet is to permit the Bureau of Land Management to detain further runoff if New Tank is full, or for any other management reason. According to the Harquahala FRS Operation and Maintenance Plan prepared by the SCS, the New Tank gate is to remain in the open position under normal pool operations.

2.3.5 Drain Outlet

There is a Drain Outlet at Station 746+00. It is also a gated 24-inch diameter RCP with a trashrack on the inlet that conveys local drainage under the CAP. The invert of the conduit is at 1383.86 ft. The 24-inch diameter conduit is also encased in concrete up to the spring line and has eight anti-seepage collars under the Harquahala FRS embankment and two anti-seepage collars under the south (downstream) embankment of the Granite Reef Aqueduct. There is a plunge pool at the outlet. The Drain Outlet was placed at the second lowest natural drainage location along the Harquahala FRS to drain as much of the reservoir as possible. The lowest natural drainage is at the principal spillway. The Drain Outlet was also placed at 746+00 to be a 'substitute principal spillway' because the Granite Reef Aqueduct is routed near the hills and could create a constriction in the flow of floodwater to the principal spillway. According to the Harquahala FRS Operation and Maintenance Plan prepared by the SCS, the Drain Outlet gate is to remain in the closed position under normal pool operations.

2.3.6 Toe Drains and Outlets

There are toe drain outlets approximately every 400 feet between Stations 452+00 and 717+00. Each drain outlet has a cross-section that is 2 feet tall, with a top width of 2 feet, bottom width of 6 feet, and a 6-inch diameter perforated asbestos cement pipe to facilitate internal drainage beyond the toe of the embankment. There are five additional toe drains placed approximately every 100 feet between Stations 790+00 to 795+00. There are six more toe drains placed approximately every 60 feet between Stations 1040+20 to1044+80 near the primary outlet. The drains are collected into one conduit that is under the CAP and discharges adjacent to the primary outlet stilling basin.

2.3.6 Emergency Spillway

The emergency spillway is a 150-foot wide concrete baffle chute located at Station 939+20. The vertical chute walls are 15 feet tall and drop the emergency spillway flow a total vertical depth of 20 feet. There are seven rows of energy dissipating baffles in the chute. The emergency spillway was designed to pass the PMF with no freeboard (NRCS criteria) and has a discharge capacity is 16,420 cfs. The emergency spillway will discharge into the CAP canal immediately downstream of the FRS. There has been no emergency spillway discharges to date.

3.0 TECHNICAL REVIEW

The purpose of the technical review was twofold. First, the project assessment team reviewed the existing and available engineering records related to the Harquahala FRS and its' construction. Through this review, the project assessment team became familiar with the structure, the history of the structure, and the basis of the original analysis and design, which will assist in the engineering assessment of the structure. Second, to review original design criteria and design guidelines under which the Harquahala FRS was constructed. This 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 Arizona Department of Water Resources (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/repositories, which included the libraries and office files of the District, NRCS, and ADWR-Office of Dam Safety. Kimley-Horn has prepared under separate cover, a data collection report, summarizing the information collected for Harquahala FRS.

This technical document review, along with the field examination, and the failure mode and effects analysis (FMEA), provided the basis for the assessment by evaluating the operational adequacy, structural stability, and current dam safety rules and regulations compliance of the Harquahala FRS.

3.1 Dam Design Criteria

Harquahala FRS was analyzed and designed by the NRCS in the late 1960's though the late 1970's. The basis of the 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 Harquahala FRS was analyzed and designed according to EM-27.

The purpose of the Harquahala FRS was to provide a 100-year level of 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 some residential and commercial properties (NRCS, 1977). The 100-year design event was used to size the principal spillway and reservoir storage volume. The hydrology for the emergency spillway design and freeboard design flood is discussed below in the Hydrology section following NRCS criteria. According to ADWR criteria, the Harquahala FRS Inflow Design Flood (IDF) is the ½ Probable Maximum Flood (PMF). The NRCS designed the structure to convey the freeboard hydrograph, which is generally a certain percentage of the PMF, without overtopping the top of the structure and no freeboard. **Table 3 (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 Harquahala FRS as a Class B structure. Class B structures are defined as those structures located in predominantly rural or agricultural areas where failure may damage isolated homes, major highways, minor railroads, or cause interruption or use of service of relatively important public utilities. The ADWR rules and regulations for jurisdictional dams classify the Harquahala FRS as a significant hazard, intermediate dam. It appears the structure was specified as an intermediate structure based on reservoir capacity.

The downstream population at risk from Harquahala FRS is very minimal and widely scattered throughout potential dambreak inundation areas. Residential development is single lot residences mainly located south of Interstate 10. There are a few scattered mobile-type residences between the dam and Interstate 10. Reclassification to a high hazard dam does not appear to be warranted at this time.

A. Modifications Related Dam Safety

A review of the project records, reports, plans, inspection reports, and other documents indicates that there have been no structural modifications made to the Harquahala FRS since original construction related to dam safety. Active and normal operation and maintenance activities are conducted at the structure. Gravel mulch was placed in November 2003 on both the upstream and downstream slopes at the far west end of the dam.

Crack Investigation: In July 1991 the SCS and the District investigated two longitudinal cracks in the top of the dam. The cracks were discovered by District maintenance crews in early 1991. The SCS prepared an investigation report titled "Engineering Report Harquahala FRS Investigation of Embankment Cracks at Stations 490+00 and 555+00" (SCS, August 1991). The report documents the findings and conclusions of the investigation.

The cracks are located near Stations 490+00 and 555+25 which are within the western three miles of the nearly 12 mile long dam. The District provided a backhoe and water truck to assist in the crack investigation. The District also conducted a settlement survey of the dam crest during June and July of 1991. The survey results indicated that the dam had settled 0.35 feet at the western end of the dam.

The SCS and District excavated four test trenches either adjacent or on top of the cracks (two each at each crack location). Water testing was also conducted to evaluated whether water remained in the cracks or dispersed.

The SCS report concluded that "the cracks are not considered a threat to the structure at this time. The transition zone is intact and should function as designed. The amount of settlement measured so far is not excessive. No action is recommended at this time other than to continue to monitor subsidence through surveys at the normal scheduled intervals and report any new development of cracks for review". The SCS report stated that the District will backfill the excavated material back into place in 6 to 8 inch lifts with a tamper mounted to the backhoe.

Kimley-Horn recommends that the Stations locations of the cracking continue to be monitored during site dam safety inspections.

B. Non-Dam Safety Modifications

In 2003, the District completed a minor modification at the cut slope at the outlet structure of the principal spillway. The downstream CAP embankment around the principal spillway outlet was damaged by unknown parties. Minor amounts of embankment material were excavated from the slope which resulted in a vertical face approximately 4 to 5 feet tall. The District cut back the vertical face to a 1.5:1 slope, the access road around the principal spillway outlet was widened to a minimum of 10 feet, and the excess material was removed.

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 the supplement to the final design report were completed in 1980 (NRCS 1980, 1980a). The NRCS design review report and a supplement to the design review report for the preliminary design were both completed in 1979 (NRCS, 1979, 1979a), as well, and contained data related to the hydrologic analysis and design of the structure. These documents provided insight into the hydrologic design and analysis of the structure since a stand alone hydrologic report could not be identified and the final design report did not cover the entire breadth of hydrologic design. At the least, the location of the emergency spillway was changed to its' current location between the preliminary and final designs. The design review report indicates that the emergency spillway was near the principal spillway, several miles from its current location. A hydrologic review completed by ADWR (ADWR, 1981) was also reviewed for this report.

The Watershed Work Plan (NRCS 1977) identifies the structural elements of the watershed project including the Harquahala FRS, the Saddleback 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 is routed from the Harquahala FRS 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 any of the other structural work plan elements.

The NRCS designed the Harquahala FRS to detain the 100-year stormwater runoff volume. No comprehensive hydrologic analysis was found in the design report, but it is assumed the 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 as was the NRCS standard of the time. There was Technical Release TR-48 (DAMS2) output data in the final design report (NRCS 1980). It appears the design rainfall was determined by using the revised TP-40 map rainfall and ES-1020 sheet 5 of 5. It also appears 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 for the upstream watershed were derived from stream channel hydraulics. Channel cross sections were taken at several locations and velocities computed. Procedures outlined in Chapter 15, NEH-4 were used. These assumptions were made as the rainfall depths, curve numbers, and times of concentration for the Harquahala FRS are similar to other structures in Central Arizona and Maricopa County.

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. For a Class B structure, the PSH is based on the 100-year precipitation depth (P_{100}).

The TR-48 (DAMS2) output data found in the final design report (NRCS, 1980) that related to the principal spillway design included the Freeboard Hydrograph (FBH), Emergency Spillway Hydrograph (ESH), and Principal Spillway Hydrograph (PSH). During the late 1970's, when the structure was designed, the NRCS typically used TR-48 to develop the PSH, ESH, and FBH design storms and flood routing through the reservoir. The TR-48 output identified the principal spillway crest elevation, dam crest elevation, and emergency spillway that were on the as-built plans, so it was assumed this was the TR-48 analysis used for final design of the structure. **Table 4 (Appendix A)** summarizes the TR-48 PSH output data found in the Final Design Report.

The principal spillway crest elevation is 1387.3 ft. Any impoundments below this level were designed to be removed from the reservoir through an 18-inch diameter drawdown gate that is operated from the crest of the structure. In the TR-48 output, the PSH maximum reservoir water surface elevation was 1408.49 ft., the maximum discharge through the principal spillway was 379 cfs, and the drawdown time for a full reservoir was estimated to be 8.8 days.

The estimated 50-year sediment volume was identified as 414 acre-feet in the Watershed Work Plan (sediment rate of 0.081 af/mi^2/yr) and was used in the TR-48 analysis. If the 50-year sediment volume were distributed evenly along the entire length of the dam, the top of the sediment pool would be at 1396.2 ft. But, in the supplement to the review of the final design report (NRCS, 1979a), it is noted that because the dam is 11.5 miles long, the structure would act more like a diversion and the sediment will not be deposited in a level pool at the lowest area of the reservoir (principal spillway and drain outlet). The designers of the structure estimated that there were four separate sub-watershed basins and estimated that the sediment would be deposited separately at the location where each of the four sub-watersheds intersect the FRS; thus four different elevations for the top of the sediment pool.

A significant portion, 80%, of the drainage area and sediment contribution will come from the west half of the dam. The final design report supplement review indicates that the estimated top of sediment pool varies from 1399.0 ft. on the west end of the dam to 1387.3 ft. at the principal outlet, which is the crest elevation of the principal outlet. At the New Tank Outlet, the top of the sediment pool was estimated to be 1396.21 ft. The elevation of the inlet channel for the New Tank Outlet was set at 1396.22 ft. The Drain Outlet and principal spillway are 3.1 and 8.7 miles away from the New Tank Outlet, so the assumption that there will not be a level sediment pool appears to be valid.

The Drain Outlet invert was placed low to drain as much of the reservoir as possible (NRCS, 1980). The drain was placed at 746+00 to be a 'substitute principal spillway' because the CAP is routed near the hills and could create a constriction in the flow of floodwater to the principal spillway. The design report indicates that the top of the sediment pool at this location would be 10 feet above the invert of the Drain Outlet. The design report indicates there was a suspicion that if the 24-inch drain outlet gate were left open, sediment would generally pass through the Outlet conduit and not settle at the Outlet. The designers estimated that sediment removal at the drain outlet would be a regular maintenance requirement for the District, but that very little sediment would need to be removed in most years.

The emergency spillway is a 150-foot wide concrete baffle chute with the centerline of the spillway at Station 939+20 on the dam. The vertical chute walls are 15 feet tall and emergency spillway conveys the discharges through a vertical drop of 20 feet. There are seven rows of energy dissipating baffle blocks in the chute. The emergency spillway was designed to pass the FBH with no freeboard. The emergency spillway crest elevation is at 1408.4 ft (as-built). The dam crest elevation at the emergency spillway is 1419.7 ft., so there is 11.3 feet of freeboard above the emergency spillway.

According to standard NRCS design, the FBH is used to establish the minimum settled elevation of the top of the dam. It is also used to evaluate the structural integrity of the spillway system. For a Class B hazard structure, the FBH is based on a design storm precipitation depth that is derived from a combination of the 100-year precipitation depth, P_{100}, and the probable maximum precipitation (PMP). The FBH is equal to P_{100} + 0.4*(PMP- P_{100}). The Harquahala FRS was identified as a Class B structure by NRCS, but there is some indication by ADWR (ADWR, 1981) that the design standard may have been higher than a Class B, which is discussed below.

The data for the FBH was extracted from the previously mentioned TR-48 output, which indicates that the maximum peak discharge through the emergency spillway during the FBH would be 16,004 cfs and that the maximum water surface elevation in the reservoir would be 1419.64 ft. The maximum discharge through the principal spillway would be 416 cfs. This is based upon a watershed runoff depth of 5.49 inches during the 6-hour storm duration and a curve number of 81. **Table 5 (Appendix Tables)** shows a summary of FBH and ESH data and **Table 6** shows a summary of the reservoir and storage data taken from the TR-48 output.

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})}. The Watershed Work Plan and preliminary design review report indicated that the emergency spillway would be located near the east abutment of the structure, but this was changed to its current location during final design. The final design report indicates that the emergency spillway was placed at its current location to take advantage of known rock location, meaning the entire invert of the exit channel is founded on rock. The emergency spillway will discharge into the Granite Reef Aqueduct immediately downstream of the FRS. The structural details of the emergency spillway design were located in the final design report, but did not include any hydraulic design details. As stated, the maximum outflow from the emergency spillway identified in **Table 5 (Appendix Tables)** is from the TR-48 output. There has been no record of any emergency spillway discharges since the construction of the structure.

In routing the FBH and ESH through the reservoir the NRCS assumed that the principal spillway will be operable and would not clog with debris. In the "Final Design Report" (NRCS, 1980; page 240 of 335) the NRCS conducted an analysis to examine the impact of principal spillway inlet clogging on the ESH and the FBH. Under the Freeboard Hydrograph scenario, the maximum reservoir water surface elevation was computed to be 1419.8 ft. (note: the dam crest elevation is 1419.7ft), the maximum reservoir storage would be 28,352 acre-feet, and that the maximum discharge through the emergency spillway would be 16,425 cfs.

In the Final Hydrologic Design Review Memorandum (ADWR 1981), ADWR reviewed an NRCS hydrology report (that could not be located during the data collection phase of this Assessment). In the memorandum, ADWR indicates that TR-20 was used to generate inflow hydrographs for the 6, 24, 36, 48, and 72-hour duration storms and that TR-48 was used for flood routing through the spillways. ADWR also stated that the structure was designed by using an average of the NRCS class 'B' and 'C' precipitation depth requirements for the design storm. The precipitation depth in the design storm for a class 'C' structure is the PMP and, as stated, for class 'B' is $P_{100} + 0.4*(PMP - P_{100})$. The average of these two precipitation design depths is $0.3* P_{100} + 0.7*PMP$. The ADWR report went on to indicate that the reservoir storage capacity at the crest of the emergency spillway was 8,291 acre-feet, which is below the NRCS design and was 28,018 acre-feet at the dam crest elevation, which is above the NRCS design. **Table 7 (Appendix Tables)** shows a summary of the ADWR Final Hydrologic Design Review.

Note that the 6-hour duration runoff depth of 5.49 inches is the same runoff depth in the TR-48 analysis from the final design report. The NRCS design review report (NRCS, 1979) indicates that the 48-hour duration event was the critical design storm based upon maximum reservoir water surface elevation. This may be true, but **Table 7 (Appendix Tables)** indicates that the emergency spillway can convey the design event peak discharge without overtopping the design crest of the structure during the 6-, 24-, 48-, or 72-hour storm events. ADWR also indicated that NRCS rainfall depths are larger than the expected 1/2 PMP depth (ADWR, 1981).

3.3.2. Spillway Inundation Study. An emergency spillway delineation study was completed for the Harquahala FRS in June 1998 by Dibble and Associates (Dibble, 1998). Spillway inundation routing is provided for the PMF discharge through the emergency spillway of 15,000 cfs, the 2/3 PMF of 10,000 cfs, and 1/3 PMF of 5,000 cfs. The study extends from the emergency spillway to Interstate-10, which is approximately 2.8 miles downstream of the emergency spillway structure. The discharges, mapping, and a Digital Terrain Model used to develop the hydraulic and topographic information for the water-surface profile models were provided to Dibble by the District. The study was completed using the Corps of Engineers HEC-RAS steady-state one-dimensional analysis program. **Figure 2 (Figures Appendix)** illustrates the emergency spillway inundation limits from the Harquahala FRS to I-10.

The study assumed that the CAP had no capacity to convey any emergency spillway discharges and that discharges would not be affected by the canal. The CAP was effectively ignored and emergency spillway discharges would be conveyed downstream of the CAP unimpeded.

A 'primary channel' that the emergency spillway discharges would generally follow downstream was identified in the study. This primary channel has an estimated capacity of 2,000 to 3,000 cfs. The Manning's roughness coefficients, N-values, which were used in the study, were between 0.035 to 0.09, with an average of 0.085 in the main channel and 0.035 in the overbank or floodplain areas. The main channel areas with 0.035 N-values were at the toe of the emergency spillway and downstream at the I-10 box culverts. 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 culverts at I-10 were in good working order and would be fully functional an emergency spillway discharge event.

The results of the study show that I-10 could be overtopped by approximately 3 feet and 2.3 feet during the 15,000 cfs and 10,000 cfs emergency spillway discharge events, respectively. The study showed that the maximum water surface elevation would be right at the top of the roadway embankment during the 5,000 cfs event.

The study recognized the inherent variability that could be associated with emergency spillway discharges at the Harquahala FRS. Discharge could be conveyed in the CAP downstream of the structure and breakout at locations away from the emergency spillway. Failure of the CAP could also potentially increase the maximum discharge being conveyed downstream. Breakout over the CAP could also erode or scour the earth embankment and cause the CAP to fail and increase the amount of discharge being conveyed downstream. With erosion of the downstream channels the split flows identified in the study could change the magnitude, direction, and velocity of discharges. Consequently, the study recommended that very generous evacuation limits be implemented downstream of the emergency spillway in the emergency action plan.

It should also be remembered that the study was completed using a steady-state one-dimensional flow analysis and that an emergency spillway peak discharge peak of 15,000 cfs will likely be attenuated by the time the discharges reaches I-10 2.8 miles downstream of the emergency spillway.

Kimley-Horn recommends that the emergency spillway inundation study be updated. The study should be extended south to Centennial Wash and account to the effects of the CAP canal and the I-10 Interstate embankment and culverts. The study should consider using a dynamic unsteady flow model such as the unsteady flow option in HEC-RAS.

3.3.3. Dambreak Analysis. A dambreak and dambreak flood routing analysis was completed for the Harquahala FRS by Carter & Associates in February 1991 on behalf of the District (Carter & Associates, 1991). The analysis included a full PMP hydrologic assessment using HMR-40 and 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 dam break inundation area estimated during the study.

The study found that the Harquahala FRS would not be overtopped during the ½ PMF, so the only dambreak failure mode studied was from internal embankment erosion or piping. The Corps of Engineers HEC-1 rainfall-runoff modeling program was used to estimate the runoff response of the watershed. The study developed the PMF analysis for the 6-hour and 72-hour duration storm event on the watershed and then routed the ½ PMF flood (inflow design flood) through the reservoir to determine the maximum reservoir water surface elevation and to develop dambreak parameters.

The study indicated that the 72-hour storm was the most critical for the DAMBRK analysis since the 72-hour delivered 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 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.

The dambreak study used all of the physical characteristics of the structure described in the previous paragraphs of this report with a few minor exceptions. The maximum discharge from the emergency spillway was 15,607 cfs in the dambreak study, compared to 16,004 cfs for the FBH. This study also used a maximum principal spillway discharge of 500 cfs, compared to the 416 cfs used with the NRCS FBH. 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 dambreak 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 and 72-hour ½ PMF events even though a 'sunny day' failure (piping) was the only failure mode evaluated. The dam breach was initiated at the maximum reservoir water surface elevation as shown in **Table 8 (Appendix Tables).** Three breach locations were chosen for the structure. One in the east of the structure, one in the middle, and one on the west end. 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 dambreak parameters used in the study.

The dambreak breach parameters and resulting dambreak peak discharge appear to be unrealistic (see **Table 9, Appendix Tables**). The maximum height of the dam is 45 feet and typically, the maximum breach with averages between 3-4 times the height of the structure. The peak discharges also appear to be high given the flood pool volume that is available. Also, the breach occurs during the peak of the ½ PMF event and consequently is not a true 'sunny-day' failure. Sunny Day piping breaches are analyzed when the reservoir water surface elevation is at the emergency spillway crest elevation with no inflow.

KHA recommends that an updated dambreak analysis and inundation mapping be prepared for the Harquahala 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 sunny day failure (full pool) should be considered without inflow and the ½ PMF should be evaluated with an empty pool as initial conditions.

3.4 Geological and Geotechnical Review

This section summarizes the review of the geological and geotechnical aspects of Harquahala 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 Harquahala 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 northeast portion of the Harquahala Valley **(Figure 1 Appendix A)**. 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 Harquahala FRS is the Big Horn Mountains, which run north and northeast of the structure. The mountains are a series of faulted, tilted, Miocene volcanics composed primarily of basalt and rhyolite, along with Laramide-age metamorphics such as granodiorite, schist, and gneiss (Stimac, 1994). The geology of Burnt Mountain includes a variety of Tertiary age volcanic rock types that involved four different types of volcanic activity. The initial activity consisted of volcano-clastic ash flows and agglomerates followed by later sequences of tuff, andesite and basalt flows.

The valley basin fill includes late Tertiary and Quaternary deposits consisting of old alluvium composed of caliche-cemented, unconsolidated to semi-consolidated sand and gravel deposits (ADWR, 2004). The sedimentary sequence with the Harquahala basin 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.

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 may be completely absent in some areas. 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.

The geology along the Harquahala FRS alignment **(Figure 2a & 2b Appendix A)** is dominated by Quaternary age alluvial fan deposits of the Upper Alluvial Unit, which is expected to be very thin, and the Lower Conglomerate Unit. The Middle Alluvial Unit is expected to be very thin or absent along Reach 1 from Stations 717+00 to 1043+00 (Reach 1), the upper alluvial fan deposit composed of very coarse grained sediments including sandy silty gravels and silty sandy gravels. According to the NRCS (1980), caliche was not found to be significant, however, it was found to be fairly common along many of the large washes. The caliche cemented sediment that are probably of Holocene age, were reportedly locally extensive but not uniformly widespread. Volcanic rhyolite bedrock underlies the alluvial fan deposits along this reach with outcrops exposed in rock knoll in the vicinity of Station 938+00 where the dam foundation encounters the volcanic rock. Another nearby volcanic rock outcrop is located a few hundred feet downstream from the emergency spillway.

The surficial geology along Reach 2, Station 443+00 to 717+00, is noticeably different from Reach 1. Thin stratified layers of silty to clayey sands, sandy to clayey silts, silty to sandy clays, and gravelly sands to silty gravelly sands predominate (NRCS, 1980). Silty sand (SM) is the more common soil type encountered along this reach with sequences ranging from about 10 to 25 feet thick at Stations 573+00, 613+00, 653+00, 663+00, 673+00, 683+00, 692+00, and 712+00. Thick sections of silty sand-clayey sand mixtures are found at Stations 593+00, 603+00, and 653+00. Relative loose surface soils up to 8 feet thick are commonly silty sand and silt (SM-ML) soils. Fine grained soils including CL and CL-ML soils are also found locally along the alignment.

Numerous ephemeral stream channels that contain loose, unconsolidated and pervious coarse grained soil cross the Harquahala FRS alignment. According to the NRSC (1980), the presences of these deposits could pose a threat to the foundation unless the cutoff trench interrupts the continuity of these deposits. Where these soils could not be over-excavated and removed, portion of the upstream borrow area were blanketed with compacted fill. The blanketing was used from Stations 484+00 to 485+00, 511+50 to 513.30, 593+00 to 594+00, and 850+00 to 852+00 (NRCS Sheet 18 of 55; As-Built Drawing, May 13, 1983).

The geology of the Emergency Spillway (ES) and Principal Spillway (PS) is similar to the geology found along Reach 1. Volcanic bedrock was encountered in one drill hole DH-210 at a depth of 18.1 feet below a cover of caliche cemented fanglomerate at the ES alignment.

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 Harquahala 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 Harquahala 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 quake with a maximum calculated magnitude of 5.7, producing a maximum calculated peak horizontal acceleration at the Harquahala FRS equal to 4 percent of the gravitational acceleration (g) (AMEC, 2002). The recommended peak horizontal acceleration design criteria calculated by AMEC for the Harquahala 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 Harquahala 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 Harquahala 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

The closest wells to the Harquahala FRS are approximately 1.5 miles north and 2 miles south of the dam. In order to gather sufficient groundwater information, hydrographs for wells within approximately 4 miles of the Harquahala FRS were obtained from the Arizona Department of Water Resources **(Figure 5 Appendix A)**. Eleven hydrographs were obtained, with the oldest dating back to 1952. These hydrographs show an overall decline in groundwater levels of 2 to 200 feet. Four of the wells show an increase in water levels of between 7 and 95 feet but most of the wells show a slow but continuous decline in groundwater 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 Harquahala 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). The data that are plotted in **Figure 6 (Appendix A)** may be used as an indicator of the relative recent land subsidence that may have occurred or is occurring in the project area. As can be seen in **Figure 6 (Appendix A)**, the change in elevation is greatest along Reach 2 of the Harquahala FRS where there is an apparent thickening of the basin fill sediments beyond the area where the buried bedrock surface drops-off from the edge of the pediment (Figure 8).

According to this data, it appears that some settlement or subsidence has occurred, mainly on the western portion of the dam between monuments A-1 and A-36, from 1984 to 2003 **Figure 6 (Appendix A).** The change in elevation in this area ranges from -0.015 to -0.480 feet. The eastern portion of the dam has not experienced any apparent settlement or subsidence because along this portion of the alignment, bedrock is relatively close to the surface.

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 Harquahala FRS vicinity (Oppenheimer, 1980). The Oppenheimer map estimated the depth to 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. No unusual buried bedrock highs were interpreted within the project area from this data.

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). Although these maps do not cover the Harquahala FRS site, Geological Consultants, Inc. has extrapolated the contour lines into the project vicinity. As depicted in Figure 8, a relatively prominent bedrock boundary condition can be deduced that reflects the approximate buried limit of the volcanic rock. It is possible that this boundary between the volcanic bedrock and the basin fill alluvial sediments could be the focus for earth fissure development at or near the Harquahala FRS.

A. Known Earth Fissures in the Project Vicinity

There are three earth fissures reported in the Harquahala Valley. The closest fissure to the Harquahala FRS lies approximately 3.4 miles southwest of the structure in Section 9, Township 2 North, Range 9 West **(Figure 9 Appendix A)**. 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).

Another earth fissure was documented in 1961 in a farm field about 4.8 miles south of the Harquahala FRS 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.

The Rogers fissure was discovered in 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 5.9 miles southwest 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 Harquahala 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 (reference citations are listed at the end of this memorandum):

- Watershed Workplan for the Harquahala Valley Watershed (Flood Control District of Maricopa County, 1967)
- Supplemental Watershed Workplan, Harquahala Valley Watershed (Flood Control District of Maricopa County, 1977)
- Harquahala FRS and Floodway Geological Investigation Summary and Test Results Report (1978)
- Supplemental Geologic Investigation of Emergency Spillway Site (Pedone, 1980)
- Final Design Report and Design Report and Design Calculations, Harquahala Valley WPP, Arizona Harquahala FRS (Soil Conservation Service (SCS), 1980)
- Harquahala Embankment Design (SCS, February 22, 1980)
- Supplement 1 to Final Design Report Dated August 8, 1980 (SCS, 1980)
- Design Review Report, Harquahala Dam and Floodway (U.S. Department of Agriculture Soil Conservation Service, 1979), Supplement No. 1 to Design Review Report, Dated June 8. 1979 (U.S. Department of Agriculture Soil Conservation Service, 1979), Supplement 2, Preliminary Design Report, Harquahala FRS and Floodway, Dated November 6, 1979 (U.S. Department of Agriculture Soil Conservation Service, 1979)
- Harquahala Floodwater Retarding Structure as-built plan set
- Dam Construction Inspection records and portions of CAP Construction Progress Report (United States Department of the Interior, Bureau of Reclamation Lower Colorado Region, 1982)
- Annual dam inspection checklists
- Supplemental package containing permeability testing information
- 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 Harquahala 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 (in) 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 ft (ft) at the mountain fronts to over 5,000 ft 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 ft 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 ft.

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 uppers 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 in 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.

B. Foundation Conditions

The foundation materials beneath the Harquahala FRS were differentiated in the Geologic Investigation Summary Report (SCS, 1978) into two reaches on the basis of a distinct change in subsurface conditions. The east end of the structure is underlain by coarse-grained gravels while the west end is underlain by finer-grained sands and silts. The change from coarse- to fine-grained materials occurs between Stations 722+00 and 712+00. The eastern end of the alignment was designated Reach 1 (Station 717+00 to Station 1054+20) and the western end, Reach 2 (Station 431+00 to Station 717+00). This distribution of materials, with coarse-grained material along the eastern portion of the alignment and finer-grained material with increasing distance to the west, is typical of alluvial fan deposits. It reflects the influence of the dam alignment relative to the Big Horn Mountains to the east.

1. Reach 1 (EAST)

Sandy silty gravels to silty sandy gravels (GM to GP-GM) predominate throughout Reach 1. These coarse sediments are typical of upper alluvial fan deposits and reflect the location of Reach 1 with respect to the Big Horn Mountains. Although caliche was not found to be significant in most of the borings completed during the geologic investigation for the Harquahala FRS, it is fairly common in this environment and may be locally extensive, though it is not uniformly widespread (SCS, 1978).

The dam alignment crosses the upstream tip of a small rhyolite knoll at Station 938+00. According to the Supplemental Geologic Investigation Report of Emergency Spillway Site (Pedone, 1980), the emergency spillway location was changed from its' original location to the as-built location in order to use the rhyolite bedrock as the spillway foundation.

2. Reach 2 (WEST)

Reach 2 consists of a heterogeneous mixture of fine-grained materials resulting from alluvial deposition, mudflows and floodplain splays. The subsurface is predominated by silty sand (SM), however thin, stratified layers of silty to clayey sands, sandy to clayey silts, silty to sandy clays, and gravelly sands to silty gravelly sands were observed during the geologic investigation. It was noted by SCS (1978) that there was no widespread consistency or uniformity either between test pits (or drill holes) or vertically in individual test pits. It was also reported that buried channel sand deposits may potentially be found all along Reach 2 in the top 15 to 20 ft.

During design, it was recognized that the soils in Reach 2 had high collapse potential. In Supplement 1 to the Final Design Report (SCS, 1980b), the laboratory collapse testing results from twenty-seven soil samples from Reach 2 were reported. Percent collapse for these samples was reported to range between 0.7% and 17.3%. Removal and re-compaction of the severely collapse-prone silts, silty sands and low plasticity clays in the upper five to ten ft was required by the designers. Sheet No. 16 in the as-built drawing set indicates that stripping to various depths up to 9 ft was performed prior to construction of the embankment.

C. Embankment

The design of the embankment explicitly accounted for the differences in foundation conditions in Reach 1 and Reach 2. The Harquahala FRS was constructed as a homogeneous embankment (Zone I; 3H:1V upstream and 2H:1V downstream) with a cutoff trench. An inclined filter/drain (Zone II) was installed in Reach 2 (Station 450+00 to Station 717+00) to mitigate the effects of transverse cracking in the fine-grained embankment materials in this reach (SCS, 1979). The design included toe drains at two locations where the drilling logs indicated channels were present at depths greater than the cutoff trench (SCS, 1980b). The as-built plans indicate that toe drains were installed between Station 790+00 and Station 796+00 and between Station 1040+20 and Station 1044+80. Seepage from the toe drains is collected in a series of 8-in laterals which feed into 10-in collector pipes that ultimately divert the seepage into the CAP canal. In Reach 1, the embankment design relied on the coarse-grained embankment material to resist cracking which by its nature is not prone to piping (SCS, 1980b).

The foundation (Reach 1 and 2) was stripped to depths of up to 9 ft and the cutoff trench ranged from 5 ft to 23.5 ft in depth (Sheet No. 16 in the as-built drawing set). To prevent uplift at the downstream toe, a five-foot thick natural blanket was placed at locations where clean sand or gravel channels were present. Typical embankment cross-sections for Reach 1 and Reach 2 are shown as **Figures 1** and **Figure 2 (both in Appendix B)**, respectively.

1. Embankment Materials

The embankment (Zones I and III) and filter materials (Zone II) have the characteristics summarized on **Table 1 Appendix B**, based on the final design report (SCS 1980b) and the design specifications (SCS, 1980c). Design specification for the drain fill were not included in the material reviewed for this Phase I Structures Assessment. In addition, no information on specific borrow areas was included in the Final Design Report.

The Geologic Investigation Summary (SCS, 1978) included laboratory test data for soil samples collected from a total of 10 soil borings and 19 test pits. The following tests were performed on representative samples:

- 58 sieve analyses
- 25 field density tests
- 15 consolidation tests
- 6 Atterberg limit tests
- 6 permeability tests
- 5 direct shear tests
- 4 standard Proctor compaction tests
- 1 triaxial compression test

A summary table of gradation data was included in the Harquahala FRS Embankment Design (SCS 1980a). These data were used as the basis for embankment design and filter gradation calculations. The strength test data (direct shear and triaxial testing) are summarized in **Table 2 Appendix B**. Sample TP-85 was collected from Reach 1 and the remaining samples for which strength testing was conducted were collected from Reach 2.

2. Embankment Construction

Difficulties during construction of the filter/drain in Reach 2 resulted in a variable width filter/drain zone that was at times uneven and less than the specified 2 ft wide drain zone, according to construction inspection reports. The Dam Construction Inspection Reports indicated that the contractor had difficulty with filter/drain material placement in Reach 2 and that the filter/drain materials were contaminated with embankment materials. The contractor was not required to perform repair work at the time of construction, suggesting that the contamination was not significant enough to effect the functionality of the filter/drain.

In Reach 1, the dam was constructed with soil containing less than the specified amount of fine-grained soil (> 15% passing the No. 200 sieve) in the upstream section (between Stations 717+00 and 1054+00). The borrow materials available for embankment construction generally contained fewer fines than the specified 15%. According to the CAP Construction Progress Report (USBR, 1982), 50 out of 58 soils tests conducted during embankment construction (at 50% to 60% of construction completed) contained less than 15% fines and 32 of the samples testing contained less than 10% fines. USBR concluded that due to the limited quantity of fines in Reach 1, it was virtually impossible to "blanket" the upstream portion of the dam with materials having the specified fines content. Although it is unclear exactly what was meant by "blanket" it is evident that there are less than 15% fines at some locations on Reach 1.

The concern regarding the lack of fines on the upstream slope was the impact this would have on slope stability. To assess this, permeability testing was performed on soil samples. Permeability test results were used to calculate the time required to fully develop a phreatic line in the dam for comparison to expected impoundment time. The results were verified by in-field constant head permeability tests at three locations in Reach 1 and it was concluded that the time required to saturate the embankment would be significantly longer than the expected impoundment time, therefore embankment instability due to the lack of fine-grained soil in the upstream section should not be a concern (SCS, 1985).

D. Compatibility of Zone II Drain Fill as Filter for Zone I

An inclined filter/drain system (Zone II) was installed in Reach 2, primarily to protect against potential internal erosion and piping of embankment materials in the event of transverse crack development (SCS, 1979). The top of the filter/drain is 7 ft below the crest of the embankment at the centerline. The filter/drain is shown on the as-built plans as a 2-foot wide filter upstream of a 3-foot wide drain.

The gradation of the filter material (Zone II) 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) to verify its' ability to filter Zone I material. **Figure 3 Appendix B** shows what is believed to be representative gradation curves for the finer materials used in the Zone I "Base Soil" (graphed with solid symbols). These gradation curves were developed using data from the Geological Investigation Summary (SCS, 1978). Reach 2 soil samples TP-75 @ 6.0-6.7' and TP-82 @ 5.5' were selected to represent Base Soil conditions in the Reach 2 embankment. Soil sample TP-75 @ 6.0-6.7' is a clay having the United Soil Classification System (USCS) of CL; TP-82 @ 5.5' is a clayey sand with the USCS classification of SC. Additional soil property data is presented in **Table 2 Appendix B**.

The base soil gradation curves (solid symbols) were adjusted for gravel content, per NRCS guidelines (NRCS, 1994). The adjusted gradation curves are shown on **Figure 3 Appendix B** with open symbols. The NRCS filtering and permeability criteria for the adjusted curves are shown by the solid circles on the 15% passing line. These criteria were used as the basis for developing the NRCS filter band shown on **Figure 3 Appendix B**. Also shown on **Figure 3 Appendix B** is the original design specification filter band and the gradation of three samples from the as-built filter (USBR, 1982). As can be seen in **Figure 3 Appendix B**, the original design specification band falls within the NRCS permeability criteria. The original design specification band is slightly coarser than the NRCS filtering criteria and may not achieve the recommended filtering limit for the finest base soils. However it appears that at least a portion of the as-built filter was placed within the NRCS criteria. It is possible that some fines from Zone I could penetrate into Zone II under a concentrated leak through a transverse crack, if the Zone II materials were graded on the coarse band in accordance with the specified gradation limits. Considering the variability in gradation of the Zone I materials, and the fact that Zone II meets the criteria except for the finest base soil and coarsest filter possibilities, it is likely that Zone II is providing adequate filter protection. Additional analyses may be done to further evaluate the efficacy of the Zone II filter, as outlined under Recommendations.

E. Embankment Settlement

The SCS designers recognized the potential for collapsible soils and associated settlement in Reach 2 and performed consolidation testing to evaluate collapse potential during the preliminary design phase. Consolidation testing was conducted under a 2,000 pounds per square foot (psf) load. Review of the preliminary design (SCS, 1979) indicated that collapse potential should be evaluated for actual loads, which were expected to exceed 2,000 psf. Additional consolidation tests of twenty-seven samples from Reach 2 were conducted under 2 tons per square foot (tsf) load. Reported collapse potential for these additional tests ranged from 0.6% to 17.3% (SCS, 1980c). .The crest elevation in Reach 2 was designed to be one and a half ft higher than in Reach 1 to allow for foundation settlement after construction.

3.4.6. Original Slope Stability Analysis. Table 3 Appendix B summarizes the parameter values used by designers for embankment slope stability analysis (SCS, 1980b). These parameter values were based on a summary graph of soil strength data presented in the Final Design Report (SCS, 1980b). This table was based on soil data that were not available for review during this Phase I Structures Assessment, therefore, the soil strength values summarized in Table 3 (Appendix B) that the designers used for stability analyses differ from strength values summarized in Table 2 Appendix B. Slope stability analysis results were reported for the loading conditions shown on Table 4 Appendix B (SCS, 1980b). No documentation of seismic slope stability was found in the materials reviewed. Stability analyses for Reach 1 were performed using the computer code Univac 1100 Series ECES and stability analyses for Reach 2 were calculated by hand.

The designers assessed the slope stability for end of construction, steady state seepage, and rapid drawdown loading conditions. In addition, the steady state seepage under seismic loading was evaluated for Reach 2.

In Reach 1, downstream slope stability was assessed with the assumption that the phreatic line emerges on the downslope face while in Reach 2, downstream slope stability was assessed for dry slope conditions (assuming the drain intercepts seepage). Although higher factors of safety would be expected for dry slope conditions (Reach 2), higher factors of safety were reported for Reach 1 under all loading conditions evaluated. This is likely the result of the different methods used to assess slope stability (computer code versus hand calculation). The reported factors of safety (SCS, 1980b) were acceptable for all loading conditions.

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 in Reach 2 was evaluated for this Phase I Structures Assessment and is discussed in Section 1.3.3. No information regarding the gradation of the drain materials in the filter/drain were available for review. It is recommended that drain gradation data be obtained, either by review of additional data not available during this Phase I Structures Assessment or by field sampling, and checked for compatibility with the filter materials.

B. Phase II Documentation of Slope Stability and Seepage Analyses

Under reasonable loading conditions for Harquahala 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 Harquahala 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 (downstream slope)**: The original factor of safety calculated for this loading condition in Reach 2 (1.23) did not achieve the minimum ADWR criteria of 1.3 (see **Table 5 Appendix B**). Additional analyses, including confirming the shear strength of Reach 1 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 Reach 1 downstream slope are recommended to document the stability of the downstream slope. (see **page 9, Appendix B**)

a. **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. However, the original analyses were conducted by assuming the full development of a phreatic surface in the upstream slope. Analyses conducted during Phase I studies for other flood retarding structures in Maricopa County (Gannett Fleming, 2004a, 2004b, and 2004c) illustrate that a steady state phreatic surface may not develop in dry dams under multiple temporary impoundment events (Gannett Fleming, 2004a, Gannett Fleming, 2004b, and Gannett Fleming, 2004c). Therefore, additional analysis of upstream slope stability under rapid drawdown conditions is not necessary.

(2) **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.

(3) **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 is 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.

(4) **Steady state seepage with seismic forces (downstream slope)**: A seismic stability analysis was only documented for Reach 2. To document seismic stability for Reach 1 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 Construction History

The Soil Conservation Service (SCS) contracted with the U.S. Bureau of Reclamation for all quality control, inspection, and construction supervision for the project. The Bureau of Reclamation was constructing the Granite Reef Aqueduct of the Central Arizona Project Canal at the same time at the Harquahala FRS was being constructed. Construction of the dam was by MM Sundt Construction Company. Quality control was in accordance with specifications, but records of tests were limited in nature. There were no unusual problems associated with construction and all work was completed and accepted on May 23, 1983. The as-built plans are dated May 23, 1983.

Of note for the purpose of this assessment, the NRCS used two sources of borrow for the construction of Harquahala FRS. These sources include excavation materials from the Central Arizona Project canal and borrow areas from the upstream reservoir pool area. The borrow areas in the upstream pool are depicted on the as-built plans. Sheet 18 of the as-built plans provides a schedule table and typical section of the borrow areas. Included on sheet 18 is a detail for a two-foot thick blanket (reservoir borrow area blanket) that was placed at locations where an exposed clean sand or gravel strata in the borrow area was not intercepted by the dam cutoff trench. The two foot thick borrow area blanket was placed in borrow areas between the following stations: Station 484+00 to 485+00; Station 511+50 to 513+50; Station 593+00 to 594+00; and Station 850+00 to 852+00. It should be noted that the cutoff trench depth was 9-feet between Station 511+50 to 512+50 and was 5-feet elsewhere. It is recommended that the borrow areas that have borrow area blankets be plainly and clearly mapped on the as-built plans and that during future regular dam safety inspections inspect the blanket areas to look for signs of illegal excavation or other deleterious factors that may impact the function of the blankets.

3.6 Utilities

There are no major utilities directly affecting the dam. There is a major overhead powerline owned by Southern California Edison that basically parallels the dam south of the CAP canal. The distance of the powerline from the dam varies from zero to 350 feet. The powerline is also shown on the Harquahala FRS as-built plans. The Central Arizona Project canal is located immediately downstream of the dam. It also parallels the dam for the full length of the dam. The as-built plans show an American Telephone and

Telegraph (AT&T) coaxial cable buried in Salome Highway located off the west end of the dam. It is unknown at this time whether this cable is still active and who the current owner of the facility is. AT&T is no longer in existence.

3.7 Emergency Action Plan

At this time the Flood Control District does not have an individual emergency action plan (EAP) for Harquahala 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 Harquahala FRS is tentatively scheduled for the last half of the 2005/2006 fiscal year. Kimley-Horn recommends that the individual EAP for Harquahala 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 for potential dambreaks. The District has completed both a dambreak study and emergency spillway inundation study for Harquahala FRS. A discussion of these studies was provided above in 3.3.2 and 3.3.3.

The emergency spillway and dambreak inundation mapping for Harquahala 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 west, middle, and east dambreak locations. Since a dambreak could potentially occur anywhere along Harquahala FRS, Kimley-Horn recommends that the inundation map be revised to reflect this possibility.

The Maricopa County Department of Emergency Management has an <u>Emergency Operation Plan</u> (McDEM, 2003) that outlines the procedures and duties of various agencies which are activated in emergency flood situations. Harquahala FRS is included the McDEM Plan in Annex I, Appendix 11. 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 <u>Flood Emergency Response Manual</u> (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 27 feet. In addition, McDEM would be notified at an impoundment depth of 31.5 feet (4.5 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 Harquahala 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 4.5 feet at which the trigger to notify McDEM occurs.

3.8 Sedimentation

The Watershed Work Plan computed the 50-year sediment volume for Harquahala FRS to be 914 acre-feet. It should be noted that the SCS report titled “Preliminary Geologic Investigation of Harquahala FRS Site” (SCS, July 1975) states that the Arizona Water Commissin computed and conducted the sediment yield study of this structure. The 50-year sediment volume corresponds to an annual sediment rate of 0.081 acre-feet/mi^2/year. This annual yield rate appears to be low compared to other sediment yield rates at District dams/watersheds. An average annual sediment yield rate within Maricopa County ranges from 0.2 to 0.3 acre-feet/ mi^2/year. Using a value of 0.3 acre-feet/mi^2/year provides a 50-year sediment volume of 3015 acre-feet. KHA recommends that an updated sediment yield analysis be conducted for Harquahala FRS watershed.

The Watershed Work Plan also developed a 50-year sediment volume for Saddleback FRS. This volume was 120 acre-feet for an average annual sediment yield of 0.11 acre-feet/ mi^2/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 Harquahala FRS but at a future time.

4.0 PRELIMINARY FAILURE MODES

Kimley-Horn and Associates, Inc. (KHA) facilitated a Preliminary Failure Modes Identification workshop for Harquahala 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) was 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 Discharge**: 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. **Central Arizona Project Canal -** The CAP canal is located immediately downstream of the dam. The CAP canal has been constructed in some reaches in

a cut (below existing grade) situation and in other reaches the CAP canal is in a fill (perched) condition.

4. **Failure of Principal Outlet**: The principal outlet for the dam is a reinforced concrete pipe 48-inches in diameter.

5. **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.

6. **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).

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

8. **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.

9. **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.

10. **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.

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

12. **Failure Mechanisms Associated with Emergency Spillway Structural Failure.** A structural failure of the emergency spillway may occur from an inadequate or failure of under-slab drainage system. This may also stem from loss of structural integrity of spillway approach slab, energy dissipater, spillway sidewalls, stilling basin, and/or wing walls.

13. **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 Harquahala FRS. Land use information for Harquahala 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 Harquahala 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 3 miles downstream of Harquahala FRS and runs parallel 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.

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 Harquahala 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 Harquahala 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 a two mile radius of Harquahala FRS. These facilities include the Central Arizona Project canal and Interstate 10. The Harquahala Power Generation Station is located downstream of Harquahala FRS approximately six miles.

6.0 FIELD INSPECTION

6.1 Previous Inspections

Kimley-Horn reviewed previous field inspection reports for Harquahala FRS from project files at the Flood Control District and Arizona Department of Water Resources. The reports collected from these sources date to March 6, 1988. A total of 18 data sources and inspection reports from March 1988 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; also not all reports are summarized) include the following:

- As-built Plans: Dated May 1983.
- November 1987: ADWR letter to FCD that dam is still operating under temporary permission.
- March 1988: FCD report noted that the "gates left in the open position". This reference is referring to the New Tank Outlet and the Drain Outlet. The operation of the gated outlets in contrary to the Harquahala FRS Operation and Maintenance plan prepared by the NRCS. According to the O&M plan the gate for the Drain Outlet is to remain closed and the gate for the New Tank Outlet is to remain open.
- March 1991: FCD report notes sinkholes and cracking at Stations 555+00 and 490+00. Cracking observed on south edge of the crest of the structure.
- April 1991: ADWR report notes field inspection observed longitudinal cracking at Station 555+00 and 490+00 as three manifestations of cracks. ADWR noted that the District will provide a water truck and backhoe for crack investigation. The field investigation is scheduled for Jun 16-17, 1991.
- March 1992: FCD report notes sinkholes excavated by SCS at Stations 555+00 and 490+00. "Found to be not a problem" according to FCD inspection report. (see August 21, 1991 Engineering Report Harquahala FRS Investigation of Embankment Cracks at Stations 490+00 and 555+25" prepared by the SCS).
- April 1994: FCD report noted 3 inches of gravel mulch would be placed on downstream slope for erosion control from Stations 445+00 to Station 587+00 within the next few weeks.
- March 1995: FCD report notes that gravel mulch was installed.
- April 1996: FCD report states that the upstream slope recently repaired and compacted with D-8 dozer from Station 445+00 to Station 510+00.
- March 1997: FCD report states that upstream slope was repaired from Stations 510+00 to 590+00 with D-8 dozer.
- March 30, 1999: FCD report notes highest impoundment on August 12, 1998 of 14.05 feet. Slope erosion and rilling for several miles east of Station 591+75.
- Gravel mulch placed in 1999 from Stations 444+00 to Station 614+50 on the downstream slope.

- Date of current ADWR License April 7, 2000
- January 2000: ADWR report notes the finding of several piezometers (Station 846+92, 1022+60, and 1022+80) as well as longitudinal cracks over central filter drain from Station 491+50 to 493+00. Areas of bulges upstream and downstream slopes.
- January 2001: FCD report notes rodent holes. Longitudinal crack reported at Station 493+87 and transverse crack at 440+45. Erosion on slopes. No changes in bulges.
- January 2003: FCD report notes many possible traverse and longitudinal cracks on downstream slope, crest, and upstream crest.
- January 2004: FCD report indicates many possible traverse and longitudinal cracks on downstream slope, crest, and upstream crest.
- November 2004: FCD inspection report indicates many possible traverse and longitudinal cracks on downstream slope, crest, and upstream crest as well as rodent holes.
- Significant impoundment events have occurred between 1997 through 2003. The highest impoundment of record was 21.5 ft in October 2000.

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 January 31, 2005 and on February 1, 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 (01/31/2005)
Dennis Duffy, P.E., R.G.	Dam Safety Engineer, Flood Control District of Maricopa County (01/31/2005)
Dan Lawrence, P.E.	Dam Safety Engineer, Flood Control District of Maricopa County (02/01/2005)

Earl Pearcy	Operation and Management, Flood Control District of Maricopa County
Robert Eichinger, P.E., CFM	Project Manager, Kimley-Horn and Associates
Ken Euge, P.G.	Principal Geologist, Geological Consultants
Dean Durkey, Ph.D., P.E	Principal Geotechnical Engineer, Gannett-Fleming
Frances, Ackerman, E.I.T., R.G.	Geotechnical Engineer, Gannett-Fleming
David Jensen, P.E.	Engineer, Kimley-Horn and Associates (02/01/2005)
Kelli Blanchard, E.I.T.	Hydrologist, Kimley-Horn and Associates

Several inspection team members were only present for one of the two day inspection as noted.

Operational Summary

Inspection Frequency: Harquahala 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 Harquahala FRS. The District conducts quarterly operational and maintenance inspections.

Maximum Water Surface Elevations: The maximum recorded impoundment for Harquahala FRS was during the October thru November 2000 time period. The impoundment was recorded at 21.47-feet which is approximately 14-feet below the emergency spillway crest elevation of 1408.4 ft (NGVD29 datum).

Emergency Spillway Discharge: Based on District records, there has been no recorded discharges from the emergency spillway at Harquahala FRS. The emergency spillway is a concrete-lined chute spillway with seven rows of baffle block energy dissipaters. The emergency spillway is located between Stations 9+92 and 10+66 along the centerline of the structure. The width of the emergency spillway is 150 feet.

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 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 Phase I Structures Assessment;
- Replace survey monument A28 at Station 470+56;

- Upstream slope has become over steepened due to erosion. Repair with compacted fill to match as-built slopes (see inspection report for station limits);
- Clear silt from central filter drain outlet conduits;
- Level crest (add 0.2-feet) with AB material in vicinity of Station 999+17.

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

District Operation and Maintenance Responsibilities: The District maintains operational control of the Harquahala 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 the preparation and implementation of the individual emergency action plan.

Field Examination Results Summary

Embankment Crest: The crest of the dam is gravel plated. All crest settlement monuments located on the crest were located. Survey monument A28 has been damaged and should be replaced. There are station markers on the dam. Currently station markers are labeled on one side; they should be labeled on both sides of the sign post. 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: There is a distinction between the two abutments due to the foundation. The left abutment is founded and contacted against bed rock. The right abutment is founded on fill. 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 adverse vegetation.

Upstream Slope: Small animal burrows were scattered on the slope face. There was no evidence of seepage, undermining, settlement or sloughing. There are several large erosion gullies located between Stations 450+00 and 579+74 and between Stations 600+70 and 656+00. Recommendations for gravel mulching of the upstream slope will be provided during the Phase I Structures Assessment currently being completed by Kimley-Horn. The upstream slope has become over steepened due to erosion between Stations 638+94 and 639+93. Longitudinal cracks/transverse cracks, depressions, and holes were observed on the slope face (see inspection report for specific locations).

On the east end of the structure, it appears that overburden material was placed on top of the compacted embankment that was not fully compacted. This may have been done to protect the embankment from erosion. Depressions in this material are evident, but do not affect the integrity of the embankment.

There is no low flow channel along the east end of the structure that keeps low flow runoff away from the upstream toe of the embankment. The maintenance follows the natural topography in many areas. Consequently, runoff ponds up against the toe of the embankment at the wash crossings and is eliminated only through seepage or evaporation. As there is no record of collapsible soils, this should not adversely impact the foundation.

The vegetation maintenance program of the District should continue along the structure. Some vegetation is currently over five feet tall.

Downstream Slope: Small animal burrows were scattered on the slope face. There was no evidence of seepage, settlement or sloughing. Slope was prepared for gravel mulch installation between Stations 762+00 and 637+70 but mulch was not installed. Gravel mulch is currently installed between Stations 637+70 and 443+80. Longitudinal cracks/transverse cracks, depressions, and holes were observed on the slope face (see inspection report for specific locations).

Sediment accumulation was noted in many of the central filter outlet drain conduits like in previous investigations. Sediment buildup in the drain outlet conduits should be removed. The toe drain outlet at Station 467+72 was inspected and noted at the pipe is offset at the first joint. The pipe needs to be repaired.

It appears that the toe has been cut back by as much as 18 feet between Stations 646+50-724+50. A survey is recommended to compare the current downstream embankment profile and the as-built plans of the structure.

Principal Spillway and Reservoir: The approach channel is clear of debris and obstructions. The exterior of the inlet structure was clean. The concrete for the inlet structure showed no signs of structural distress. There were several minor shrinkage cracks on the exterior portion of the inlet structure. The trash rack was clear of debris and obstructions. The interior of the principal spillway conduit was inspected visually by deflecting sunlight by mirror into the conduit barrel. The conduit was clean and there were no apparent signs of seepage. It is recommended that the outlet conduit be videotaped for subsequent office review.

The discharge outlet structure of the principal spillway was clear of debris. The joints of the outlet structure were straight and appeared tight. The slope north of the principal spillway outlet structure has recently been repaired. There were no signs of seepage.

The 24-inch diameter Drain Outlet and the 24-inch diameter New Tank Outlet should be video inspected on a regular basis along with the principal spillway conduit. The operation of the gates for these two outlets needs to be confirmed with the Operation and Maintenance Plan.

The toe drain outlet located at the principal spillway outlet structure was located. The service manhole for the toe drain outlet system was located but not inspected. The manhole lid was bolted. The toe drain outlet system should be video inspected to the extent possible.

Emergency Spillway: The emergency spillway is located between stations 9+92 and 10+66 of the dam. The emergency spillway is a 150-foot wide broad crested concrete chute. The spillway is clear of any obstructions and debris. The emergency spillway control structure shows minor cracking, no repairs are required at this time. The joint material has aged and is separating and cracking and should be replaced. The emergency spillway is concrete lined to outfall with baffle block energy dissipaters with no stilling basin. Discharge from the emergency spillways flows into the Granite Reef Aqueduct (GRA) which could cause the GRA to overtop, allowing flow to reach I-10.

Instrumentation: The settlement monuments for Harquahala FRS are located on the 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 A28 at Station 470+56 has been damaged and should be replaced. There are station markers on the Harquahala 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 Harquahala FRS.

There are seven staff gages on the dam at the principal outlet. Two of the staff gages are located on the outlet tower. The other five are separate posts mounted to the upstream slope. Staff gages are used to indicate the level of water impounded in the reservoir. A pressure transducer is also located at the principal outlet.

There is no staff gage or ALERT station at the western end of the dam.

Toe Drain Outlets: Toe drain outlets were located in the field between Stations 452+00 to 717+00 (see field notes in **Appendix E**). The toe drain at Station 467+72 was located and visually inspected at the downstream outlet. Photograph No.6 in the dam safety inspection report provides a photo of the interior of the six-inch perforated asbestos cement pipe. It was noted that the first length of six-inch pipe appears to be offset at the first pipe joint, that the pipe wall has failed, and that the pipe has rotated slightly. It is recommended that this section of pipe be replaced and the drain fill material be replaced around the pipe section. The replacement of the pipe should follow the original detail provided in the as-built plans on Sheet 16. Perforated PVC pipe may be used instead of the asbestos cement pipe. The damaged pipe should be excavated to 2 feet beyond the first pipe joint. The new pipe may be joined to the existing pipe using a MAG standard pipe collar detail. The drain fill materials may be placed around the new pipe section and the excavation trench backfilled with compacted dam embankment materials.

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 relative to Harquahala FRS (see **Appendix E**).

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 relative to Harquahala FRS. An EAP for Harquahala 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 Harquahala FRS and appurtenant structures are in satisfactory operational condition.

6.6 Recommendations from Inspections

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

- Repair the damaged survey monument A28;
- Continue active vegetation management program;
- Remove sediment and any obstructions in the central filter drain outlet conduits;
- Complete a survey to compare the downstream embankment profile to the as-built plans as it appears the toe may have been cut back;
- Add watershed instrumentation (stream gages and rain gages in the upper watershed);
- Develop an Emergency Action Plan to meet FEMA 64 and ADWR requirements
- Conduct a video inspection of central filter drain outlet conduits;
- Conduct a video inspection of the toe drain outlet system at the principal spillway.
- Add a staff gage and/or ALERT station at the west end of the dam.
- Repair toe drain outlet at Station 467+72. See Section 8 – Recommendations.
- Locate piezometers using as-builts and field markers on dam. Confirm location and mark on set of as-built plans. Abandon piezometers per ADWR groundwater well abandonment guidelines.
- 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.
- Confirm the operation of the gated outlets (New Tank and Drain Outlet) for normal pool impoundments with the Harquahala FRS Operation and Maintenance Plan prepared by the SCS.

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 Harquahala 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 2, 2004. The workshop participant included:

Tom Renckly, P.E., Flood Control District of Maricopa County, Project Manager,
Dan Lawrence, P.E., Flood Control District of Maricopa County, Dam Safety Engineer
John Harrington, P.E., Natural Resources Conservation Service, Hydraulics Engineer
Bob Eichinger, P.E., CFM, Kimley-Horn and Associates, Inc., Project Manager
Debbie Miller, P.E., PhD, FMEA Facilitator, Gannett Fleming, Inc.
Frances Ackerman, R.G., E.I.T., Gannett Fleming, Geotechnical Engineer
Ken Euge, R.G., Geological Consultants, Geology
David Jensen P.E., Kimley-Horn and Associates, Inc, Session Recorder
Brett Howey, P.E., Flood Control District of Maricopa County, Dam Safety Engineer.
Dan Lawrence, P.E., Flood Control District of Maricopa County
Dennis Duffy, Ph.D., Flood Control District of Maricopa County

The detailed Failure Mode and Effects Analysis Report is provided in **Appendix F** of this report. 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 documents in a detailed report prepared by Kimley-Horn and reviewed by the FMEA team. The detailed report is included in **Appendix F**.

7.3 FMEA Results

The FMEA for Harquahala 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 medium consequence:

S5a: Failure Due to Transverse Cracks Through Dam that Extend Through Crest Above Filter/Drain in Reach 2 Leading to Internal Erosion and Breach at Location of Crack During Impoundment (Category I; may go to Category II).

Failure Mode Description: Several possible transverse cracks above the filter/drain (in the upper 7 feet of the embankment) in Reach 2 have been identified. In addition, the Reach 2 soils are erosive. During a large flood, flow of impounded water into these cracks could result in continuing internal erosion and enlargement of the cracks through the upper portion of the dam, leading to a breach in the upper embankment that widens and erodes downward ultimately causing a breach of the embankment.

H1: Overtopping During Major Flood Event (Category I, may go to Category II).

Failure Mode Description: The Harquahala FRS is approximately 11 miles long, with the emergency spillway located near the left (eastern) abutment. Sub-basins from the western portion of the watershed are routed eastward by the relatively flat topography to the emergency spillway. In addition, the topography on the upstream side of the structure, and thus the routing of water, is topographically constricted by the presence of the Big Horn Mountains to the west of the Emergency Spillway (Station 839+00). Potentially slow routing of water to the emergency spillway, and/or backup of water behind the topographic constriction, after the occurrence of back-to back storm events in the watershed could lead to overtopping of the dam and subsequent erosion and embankment breach.

S3a. Failure Associated with Piping along the Principal Outlet Leading to Undermining and Breach of Dam at the Location of the Principal Outlet (Category II).

Failure Mode Description: The principal outlet is a 48-inch RCP and has anti-seep collars. Construction issues associated with anti-seep collars (difficulty in compaction around the collars) can lead to areas of preferential flow paths around the collars. The preferential flow can result in piping of embankment materials and undermining, ultimately leading to breaching at the principal outlet location during impoundment events.

S3b. Failure Due to Piping along the Eastern 24-inch Drain Outlet (Station 746+00) that Leads to Undermining and Breach of Dam during Impoundment (Category II).

Failure Mode Description: The drain outlet has anti-seep collars. Construction issues associated with anti-seep collars (difficulty in compaction around the collars) can lead to areas of preferential flow paths around the collars. The preferential flow can result in piping of embankment materials and undermining, ultimately leading to breaching at the outlet location during impoundment events.

S5b. Failure Due to Transverse Cracks through Dam and Filter/Drain in Reach 2 that Leads to Internal Erosion and Breach at Location of Crack during Impoundments. (Category II).

Failure Mode Description: A transverse crack extends from the crest of the embankment downward into the embankment and fully through the filter. During impoundment, flow may develop through the transverse crack and initiate the process of internal erosion of upstream embankment material which can then be transported through the crack and the cracked 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.

H2a. Potential Adverse Consequences Resulting from Emergency Spillway Discharges during Major Rainfall Events (Category II).

Potential Adverse Consequence Description: The Harquahala FRS emergency spillway is a 150 foot wide reinforced concrete baffle block chute spillway located in the eastern reach of the dam (Station 939+20) of the main dam embankment. Normal flood discharges from the spillway are directed into the CAP canal. If discharges are sufficient, the flows will overtop and flow past (overshoot) the canal into several small natural, shallow, ill-defined washes. The flows continue in the washes and are directed toward Interstate 10. This potential "failure mode" does not "fail" the dam or emergency spillway. However, any large appreciable flows from the spillway would likely cause adverse consequences downstream from the dam. Very large flows have the potential for resulting in extensive damage and potential loss of life. This potential adverse consequence was rated as Category II because normal "successful" operation of the emergency spillway can produce discharges that could have significant adverse consequences and the likelihood of occurrence of these adverse consequences is associated with floods of reasonably probable frequency. The floodwaters will pass through the emergency spillway. From that point the water will flow towards and under Interstate 10 and into the agricultural and low density residential housing communities downstream from the dam.

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.

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

<table>
<tr><th>Advantages</th><th>Disadvantages</th></tr>
<tr><td>Provides erosion protection</td><td>High application costs for long structures</td></tr>
<tr><td>Reduces rodent activity and burrowing</td><td rowspan="3">May obscure or cover evidence of incipient crack formation, or changes in existing surface cracks, that would otherwise be observed during routine inspections</td></tr>
<tr><td>Helps retain and stabilize moisture in the embankment soils, minimizing shrinkage cracking</td></tr>
<tr><td>Works as a mulch in combination with hydroseeding, improving seed survivability and water availability for sustaining plants</td></tr>
<tr><td>Landscape aesthetics are superior to hydroseeding without mulch (less reflective, darker color)</td><td>Maintenance is required; tends to slide down-slope over time.</td></tr>
<tr><td>May provide some level of incidental overtopping protection when applied to downstream slopes</td><td>Safety concerns for walking slopes for dam safety inspections</td></tr>
<tr><td>May provide a filtering effect for transverse embankment cracks</td><td></td></tr>
<tr><td>When applied using a gravel shooter, much of the existing vegetation survives</td><td></td></tr>
<tr><td>One-time application and good performance reduces slope erosion O&M costs</td><td></td></tr>
<tr><td>Discourages vehicles on dam slopes</td><td></td></tr>
</table>

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 no erosion, to 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 have had a central filter installed after original construction, while others have had central filters installed as part of original construction. The dams that have had central filters constructed are the NRCS sponsored dams compared to the Corps of Engineers sponsored dams. 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 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 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 the emergency spillway inundation study be updated. The study should be extended south to Centennial Wash and account to the effects of the CAP canal and the I-10 Interstate embankment and culverts. The study should consider using a dynamic unsteady flow model such as the unsteady flow option in HEC-RAS.

(2) Kimley-Horn recommends that an updated dambreak analysis and inundation mapping be prepared for the Harquahala 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 sunny day failure (full pool) should be considered without inflow and the ½ PMF should be evaluated with an empty pool as initial conditions.

(3) A quantitative risk assessment for the dam 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).

(5) Verify BLM easements have been recorded with County Assessor.

(6) Conduct an updated sediment yield study for the Harquahala FRS watershed.

(7) Potential for Harquahala - Centennial Levee System Alternative. Potential to Install Second Outlet at West End. Since a majority of the drainage area is located at the western reach of the dam and hence most of the inflow from storm events, an opportunity was identified for construction of an additional principal spillway and floodway. The floodway would convey flows from the west principal spillway to Centennial Levee. Centennial Levee, in turn, would direct flows to Centennial Wash. The concept for this alternative could be extended to evaluate the potential for a second emergency spillway located in the western reach of the dam. In this fashion Harquahala FRS would be furnished with two sets of principal and emergency spillways. This concept, at some future time, could segment the dam into two smaller structures.

(8) Kimley-Horn recommends that a site-specific PMP analysis be conducted. Site specific Probable Maximum Precipitation studies in Arizona have resulted in PMP values lower that those resulting from using the HMR-49 procedures.

Harquahala FRS is an 11-mile long dam with significantly different contributing sub-basins from the west end to the east end of the dam. The west reach of the dam contributing drainage area is from alluvial plains and fans. The eastern reach contributing drainage area is from the Big Horn Mountains.

(9) Need PMF Routing through the dam. The probable maximum flood or the ½ PMF have not been routed through the impoundment. A review of the project hydrologic and hydraulic records indicates that the dam was designed as an average between the design storm for the Class B and Class C criteria. The March 19, 1981 ADWR memorandum indicates that the Safety of Dams inflow design-flood for the dam should be the ½ PMF. The memo states that the ½ PMF has not been routed through the dam.

(10) Recommend Dynamic Routing be Conducted. Harquahala FRS is an 11-mile long dam that functions as a diversion/levee system. Flows are collected along the dam from many contributing streams. The timing of the hydrographs from these streams will result in a sloping water surface for the reservoir from the west end of the dam toward the east end of the dam. Normal hydrologic routing used in HEC-1 uses the modified Puls routing method that results in a level pool (level pool routing). The use of an unsteady flow model, such as the dynamic capabilities of the HEC-RAS model, will provide the water surface profile/elevation of the inflow design flood (and other storm events) for Harquahala FRS. Knowing the actual water surface profile along the dam will provide the opportunity for determining residual freeboard and the potential for overtopping.

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 in Reach 2 was evaluated for this Phase I Structures Assessment. No information regarding the gradation of the drain materials in the filter/drain were available for review. It is recommended that drain gradation data be obtained, either by review of additional data not available during this Phase I Structures Assessment or by field sampling, and checked for compatibility with the filter materials.

B. Phase II Documentation of Slope Stability and Seepage Analyses

Under reasonable loading conditions for Harquahala 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 Harquahala 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 (downstream slope)**: The original factor of safety calculated for this loading condition in Reach 2 (1.23) did not achieve the minimum ADWR criteria of 1.3 (see **Table 5 Appendix B**). Additional analyses, including confirming the shear strength of Reach 1 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 Reach 1 downstream slope are recommended to document the stability of the downstream slope. (see **page 9, Appendix B**)

   a. **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. However, the original analyses were conducted by assuming the full development of a phreatic surface in the upstream slope. Analyses conducted during Phase I studies for other flood retarding structures in Maricopa County (Gannett Fleming, 2004a, 2004b, and 2004c) illustrate that a steady state phreatic surface may not develop in dry dams under multiple temporary impoundment events (Gannett Fleming, 2004a, Gannett Fleming, 2004b, and Gannett Fleming, 2004c). Therefore, additional analysis of upstream slope stability under rapid drawdown conditions is not necessary.

(2) **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.

(3) **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 is 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.

(4) **Steady state seepage with seismic forces (downstream slope)**: A seismic stability analysis was only documented for Reach 2. To document seismic stability for Reach 1 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 Additional Recommendations from Inspection Report and FMEA 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) Toe Drain Outlets: Toe drain outlets were located in the field between Stations 452+00 to 717+00 (see field notes in **Appendix E**). The toe drain at Station 467+72 was located and visually inspected at the downstream outlet. Photograph No.6 in the dam safety inspection report provides a photo of the interior of the six-inch perforated asbestos cement pipe. It was noted that the first length of six-inch pipe appears to be offset at the first pipe joint, that the pipe wall has failed, and that the pipe has rotated slightly. It is recommended that this section of pipe be replace and the drain fill material be replaced around the pipe section. The replacement of the pipe should follow the original detail provided in the as-built plans on Sheet 16. Perforated PVC pipe may be used instead of the asbestos cement pipe. The damaged pipe should be excavated to 2 feet beyond the first pipe joint. The new pipe may be joined to the existing pipe using a MAG standard pipe collar detail. The drain fill materials may be placed around the new pipe section and the excavation trench backfilled with compacted dam embankment materials.

(3) Borrow Area Blankets**:** The two foot thick borrow area blanket was placed in borrow areas between the following stations: Station 484+00 to 485+00; Station 511+50 to 513+50; Station 593+00 to 594+00; and Station 850+00 to 852+00. It should be noted that the cutoff trench depth was 9-feet between Station 511+50 to 512+50 and was 5-feet elsewhere. It is recommended that the borrow areas that have borrow area blankets be plainly and clearly mapped on the as-built plans and that during future regular dam safety inspections inspect the blanket areas to look for signs of illegal excavation or other deleterious factors that may impact the function of the blankets.

(4) Update Emergency Action Plan. Develop an Emergency Action Plan to meet FEMA 64 and ADWR requirements.

(5) Repair the damaged survey monument A28.

(6) Continue active vegetation management program.
(7) Remove sediment and any obstructions in the central filter drain outlet conduits.
(8) Complete a survey to compare the downstream embankment profile to the as-built plans as it appears the toe may have been cut back.
(9) Conduct a video inspection of central filter drain outlet conduits.
(10) Conduct a video inspection of the toe drain outlet system at the principal spillway.
(11) Add a staff gage and/or ALERT station at the west end of the dam.
(12) Repair toe drain outlet at Station 467+72.
(13) Locate piezometers using as-builts and field markers on dam. Confirm location and mark on set of as-built plans. Abandon piezometers per ADWR groundwater well abandonment guidelines.
(14) 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.
(15) Penetrations without filter diaphragms should be investigated for construction of filter diaphragms. Penetrations through the embankment that do not have a filter diaphragm should be evaluated to determine whether one is needed. Foundation soils in Reach 1 are different from the soils in Reach 2, and the need/effectiveness of a filter diaphragm will differ depending on those conditions.
(16) CAP Canal Elevation Data and Performance Records Need to be Reviewed. The CAP canal and canal embankment provides a measure of mitigation for potential dambreaks or overtopping events. Also the canal has a concrete lining to reduce infiltration losses. CAP canal survey data should be obtained to evaluate elevation changes that may be signs of settlement or regional land subsidence. The CAP canal inspection and maintenance records should also be reviewed to ascertain potential canal lining crack repairs. Cracking of the concrete lining could be an early indicator of regional settlement and land subsidence.
(17) Continue to monitor locations of past noted longitudinal cracks at Stations 490+00 and 555+25 on dam crest.

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Table 1. Dam Crest Elevations (NGVD29).

Beginning Station	Ending Station	Distance (ft)	Top of Dam Design Crest Elevation (ft)
433+50	450+00	1,650	1420.7-1422.2 (0.000907 ft/ft slope)
450+00	530+00	8,000	1422.2
530+00	540+00	1,000	1422.2-1422.1 (0.0001 ft/ft slope)
540+00	710+00	17,000	1422.1
710+00	717+00	700	1422.1-1420.6 (0.00214 ft/ft slope)
717+00	742+00	2,500	1420.6
742+00	752+00	1,000	1420.6-1420.5 (0.0001 ft/ft slope)
752+00	810+00	5,800	1420.5
810+00	910+00	10,000	1420.5-1420.0 (0.00005 ft/ft slope)
910+00	930+00	2,000	1420.0-1419.7 (0.00015 ft/ft slope)
930+00	1054+08	12,408	1419.7
Total Embankment Length (ft)		**62,058**	

Table 2. Harquahala FRS Physical Data Summary.

Item	Unit	Design Value
Class of Structure		B
Drainage Area (Uncontrolled)	square miles	102.3
Average Curve Number (1-Day AMC II)		81
Elevation-Top of Dam	ft	Varies-1419.7 at ES and PO
Elevation-Emergency Spillway Crest	ft	1408.4
Elevation-Principal Spillway Crest	ft	1387.3
Maximum Height of Dam	ft	49.3
Volume of Fill	yd^3	4,428,000
Length	ft	62,058 (11.8 miles)
Maximum Bottom Width	ft	111
Top Width	ft	14
Upstream Slope Z:1 Downstream Slope Z:1		3; 2
Total Capacity	**acre-feet**	**8,404**
Sediment (50-Year)	acre-feet	414
Retarding Pool	acre-feet	7,990
Surface Area		
Sediment (50-Year)	acre	123
Retarding Pool	acre	1,231
Principal Spillway Design		
Runoff Volume (Areal, 1-Day)	Inches	1.6
Runoff Volume (Areal, 10-Day)	Inches	1.76
Capacity	cfs	379
Frequency Operation-Emergency Spillway	%	1
Dimensions of Conduit	ft	48-inch diameter RCP
Tailwater Elevation	ft	6.1
Type of Outlet		SAF
Drawdown Time	days	8.77
Emergency Spillway Design		
Rainfall Volume (ESH, Areal)	Inches	3.49
Runoff Volume (ESH)	Inches	2.52
Storm Duration	hours	6
Type		Concrete Baffled Chute
Bottom Width	ft	150
Velocity of Flow	fps	9.59
Slope of Exit Channel	ft/ft	0.016
Maximum Reservoir Water-Surface Elevation	ft-NGVD 1929	1412.8
Maximum Storage Volume	acre-feet	14,614
Side Slopes Z:1		Vertical
Maximum Outflow from ESH Routing	cfs	4,506
Freeboard Design – 6 hour		
Rainfall Volume (FH, Areal)	Inches	7.43
Runoff Volume (FH)	Inches	5.49
Storm Duration	hours	6
Maximum Reservoir Water-Surface Elevation	ft-NGVD 1929	1419.7
Maximum Storage Volume	acre-feet	27,964
Maximum Outflow from FH Routing	cfs	16420
Peak Reservoir Inflow (cfs)	cfs	71,340
Capacity Equivalent		
Sediment Volume	Inches	0.098
Retarding Volume	Inches	2.19

* Elevation Data is based upon NGVD 29 vertical datum

[2] Harquahala Valley Watershed Work Plan, March 1977

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	**Class B.** Structures located in predominately rural or agricultural areas where failure may damage isolated homes, main highways or minor railroads or cause interruption of use or service or relatively important public utilities.	**Class B.** Structures located in predominately rural or agricultural areas where failure may damage isolated homes, main highways or minor railroads or cause interruption of use or service or relatively important public utilities.	**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.	Freeboard - 11.3 ft
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 PMP Hydrograph and Stability Design Storm Hydrograph		For Class B Structure 1: emergency spillway hydrograph P100 + .12x(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	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.	Additional ADWR criteria: i. include a control structure to avoid head cutting and lowering o fthe spillway crest for spillways excavated in soils or soft rock. Ii. Ensure each spillway, in combination with outlet, is able to safetly pass the peak discharge flow rate, as calculated on the basis of the IDF.
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)	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.	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. Current EAP does not meet 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	NA	NA	NA	
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 14 ft.	Minimum top width is 14 feet.	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.	Meets current ADWR and NRCS criteria

Table 4. TR-48 Principal Spillway Hydrograph Summary Data. (NGVD29)

			100-Year Runoff				
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]
81	5	102.3	1.60	1.76	1387.3	48" diameter	1408.4

Table 5. Freeboard and Emergency Spillway Hydrograph Summary Data. (NGVD29)

		Rainfall Runoff		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)
1408.4	150	2.52	5.49	4,112	1412.8	16,004	1419.64

Table 6. Reservoir and Storage Summary Data. (NGVD29)

Item	Elevation [Ft]	Area [Ac]	Sum Storage [Af}
Bottom of Pool	1370.0	0	0
Top of Sediment Pool	Varies 1399-1387.3	123 *	414 *
Crest of Principal Spillway	1387.3	45.2	11.8
Crest of Emergency Spillway	1408.4	1,231	8,404
Crest of Dam (w/o camber)	1419.7	2,230	27,964

*414 acre-feet of sediment storage would equate to a level pool elevation of 1396.2 and would cover an area of 123 acres

Table 7. ADWR PMP Hydrologic Review Reservoir and Storage Summary Data. (NGVD29)

Storm Duration	6-HR	24-HR	48-HR	72-HR
Storm Precipitation (inches)	7.43	10.16	12.2	13.12
Curve Number	84	84	84	84
Peak Inflow (cfs)	71,340	53,850	53,000	52,430
Runoff (Af)	29,953	42,000	49,400	52,400
Runoff (inches)	5.49	7.73	9.05	9.6
Peak Outflow (cfs)	13,296	15,558	16,098	16,109
Maximum Reservoir WSEL (ft)	1419.3	1419.6	1419.7	1419.7

Table 8. Dambreak Hydrologic Summary

	6-HOUR DURATION STORM	72-HOUR DURATION STORM
Drainage Basin Area (mi^2)	103.3	Same
PMP Rainfall Depth (inches)	15.4	9.5
Curve Numbers	87-89	87-89
PMF Reservoir Peak Inflow (cfs)	162,504	101,520
½ PMF Reservoir Peak Inflow (cfs)	81,252	50,760
½ PMF Maximum Reservoir WSEL (ft)	1415.41	1418.48
½ PMF Runoff Volume (Af)	22,038	38,140
Crest of Principal Spillway	1387.3	45.2
Crest of Emergency Spillway	1408.4	1,231
Crest of Dam (w/o camber)	1419.7	2,230

Table 9. Dam Breach Parameters.

	EAST BREACH	MIDDLE BREACH	WEST BREACH
Initial Reservoir WSEL (ft)			
6-hour duration event	1415.4	Same	Same
72-hour duration event	1418.5	Same	Same
Initial Breach Elevation	1397.8	1404.9	1407.9
Peak Discharge (cfs)			
6-hour duration event	146,437	120,725	71,888
72-hour duration event	217,237	203,186	137,821
Dam Breach Bottom Width (ft)			
6-hour duration event	307	578	563
72-hour duration event	420	788	793
Time of Breach Formation (hrs)			
6-hour duration event	3.2	3.6	4.5
72-hour duration event	3.1	3.1	3.5
72-Hour DAMBRK Peak Discharge (cfs)	185,613	155,853	119,249

* Typical Breach width = 3 to 4 times height of dam (Harquahala FRS would be 135-180 feet)

Table 10. Notification Levels from EOP and FERM.

	Emergency Operations Plan (November 2003)	FERM (January 2002)
	Pool Level [ft]	Pool Level [ft]
District Alarm	-	23.5
Notify FCD O&M	-	27.0 (25% full)
Notify McDEM	8.0 at P.O.	31.5 (50% full)

Table 11. Harquahala FRS FCD Gage Id# 5127 And 5128

STATION DESCRIPTION

LOCATION – The dam is located in western Maricopa County near the Salome Highway exit on Interstate Highway 10. The dam is north of the highway. The gaging is located on the outlet of the dam at the eastern end of the structure. The structure is on the upstream side of the Central Arizona Project canal in the area. Latitude N33 32 54, Longitude W113 05 52. Located in the SE1/4 NW1/4 NE1/4 S05 T2N R8W in the Burnt Mountain 7.5-minute quadrangle.

ESTABLISHMENT – The stage gage was established on March 1, 1994.

DRAINAGE AREA - 102.3 mi^2

GAGE – The gage is a pressure transducer type instrument. The PT is located on the outside of the outlet tower of the principal outlet at elevation 0.38 feet gage height or 1,375.19 feet NAVD 1988.

There are seven staff gages on the dam at the principal outlet. Two are painted on the outlet tower and five are individual staff gage posts mounted to the upstream face of the dam. All staff gages are within 0.07 feet of the indicated reading. Essentially, they all read in gage height.

There are no crest gages at this location.

ZERO GAUGE HEIGHT – Zero gauge height is defined as the zero on the staff gage.

HISTORY – A precipitation gage was installed on September 15, 1993 by the District. A recording level gage was installed on March 1, 1994. The PT diaphragm was surveyed at 0.76 feet gage height on June 17, 1994. On April 1, 1997, the PT was surveyed at 0.38 feet gage height. No known physical change of the PT diaphragm is known. The differences may be due to a combination of subsidence and survey differences between Donaldson's June 1994 survey and Lehman's April 1997 survey and the SCS subsidence survey of 1991, and FCD's McClain-Harbers survey of March 1996. Therefore, the old PT level is used between March 1, 1994 and September 30, 1996. Beginning with Water Year 1997, the PT diaphragm is taken to be 0.38 feet gage height. Since no significant impoundments have been recorded since gage installation, these discrepancies are considered relatively unimportant to the gage record. Elevation data changed from NGVD 1929 to NAVD 1988 in 1997. The relation is as follows. 0.00 gage height = 1,372.94 feet NGVD 1929 = 1,374.81 feet NAVD 1988.

REFERENCE MARKS –

Near the principal outlet

SCS Brass Cap marked 'A61' near station 1040+00. Elevation 46.43 feet gage height, or 1,421.24 feet NAVD 1988.

SCS Brass Cap marked 'A62' near station 1050+00. Elevation 46.70 feet gage height, or 1,421.51 feet NAVD 1988.

Near the Auxiliary Spillway

SCS Brass Cap marked 'A51' near station 940+00. Elevation 46.81 feet gage height, or 1,421.62 feet NAVD 1988.

All references are SCS brass caps for subsidence monitoring that are along the entire length of the dam.

CHANNEL AND CONTROL – The primary outlet for the dam is an outlet tower works located at the east end of the dam. The outlet is a 48-inch diameter pipe culvert. Higher flows occur through an auxiliary spillway.

PRIMARY / AUXILIARY SPILLWAY –

The primary outlet is a 48-inch diameter culvert pipe. Its length is 465 feet. The culvert invert elevation is 1,369.38 feet NAVD 1988 or –5.43 feet gage height. There are two intake points on the tower. The first is an 18-inch diameter orifice with invert elevation of 1,375.38 feet NAVD 1988 or 0.57 feet gage height. The second point is the top of the uncontrolled outlet at the top of the tower that is at elevation 1,389.31 feet NAVD 1988 or 14.50 feet gage height.

The auxiliary spillway crest is at 1,410.51 feet NAVD 1988 or 35.7 feet gage height. The auxiliary spillway is located approximately 2 miles west of the principal outlet. The spillway width is about 150 feet.

The top of the dam elevation is at about 43.0 feet gage height or 1,417.8 feet NAVD 1988.

RATING –

The current discharge rating is Rating #1 computed by T. M. Donaldson in June 1994. The culvert rating was computed by HY8 for the uncontrolled outlet. The auxiliary spillway rating was computed from a weir analysis using C=2.9.

The current capacity rating is Rating #2 developed from DTM by GC Card in April 1997.

DISCHARGE MEASUREMENTS – The primary outlet could be evaluated from the outlet channel downstream of the dam. It would have to be done with the gated outlet at the dam fully open.

POINT OF ZERO FLOW – Flow begins through the outlet at 0.57 feet gage height. Flow begins through the auxiliary spillway at about 35.7 feet gage height.

FLOODS / SIGNIFICANT IMPOUNDMENTS –

REGULATION – The dam is regulation for the natural flows from the mountains to the north.

DIVERSIONS – None known

ACCURACY – Fair to good

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

UPDATE – February 5, 2001

Kimley-Horn
and Associates, Inc.

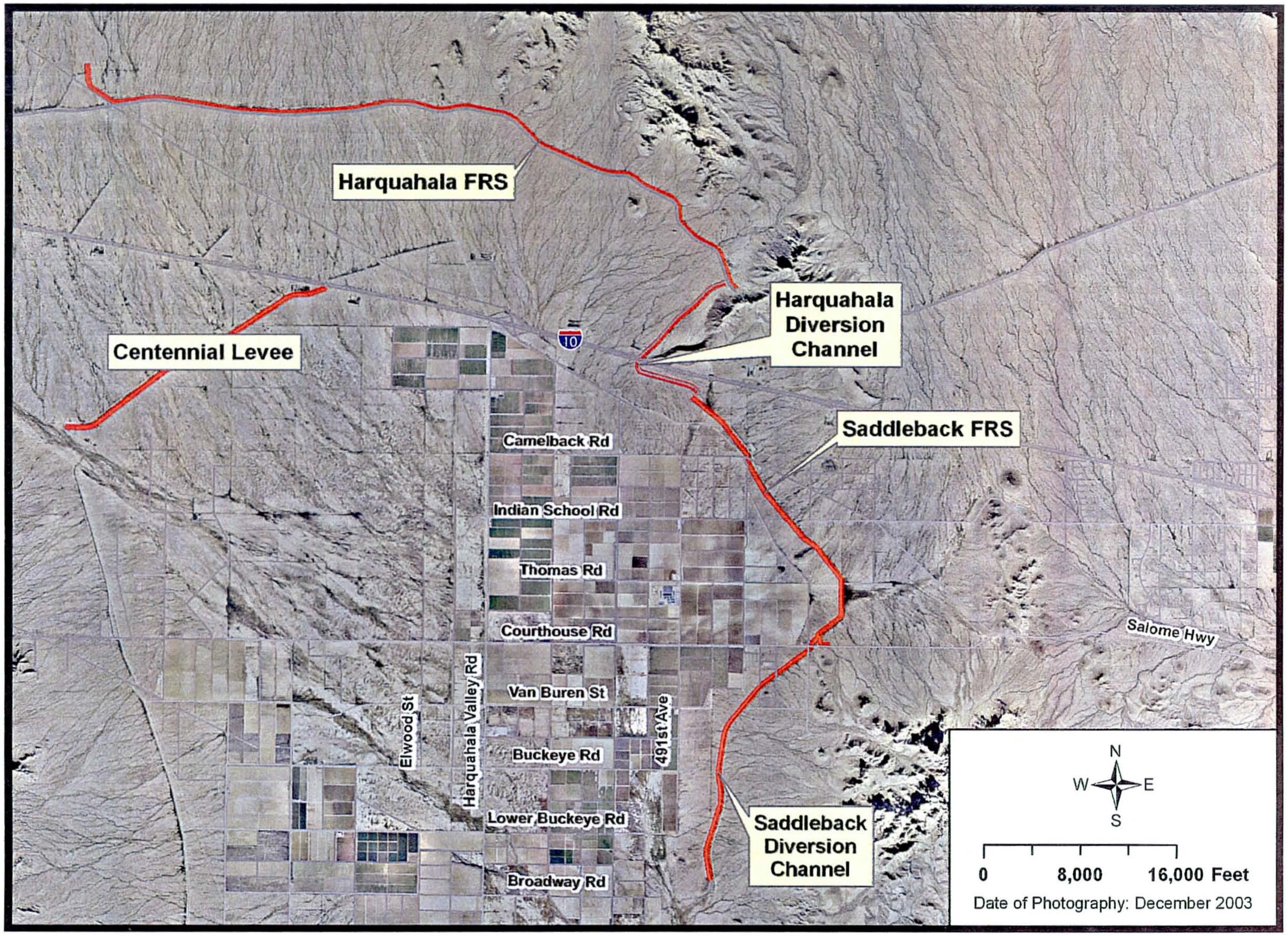

Figure 1. Location Map
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn
and Associates, Inc.

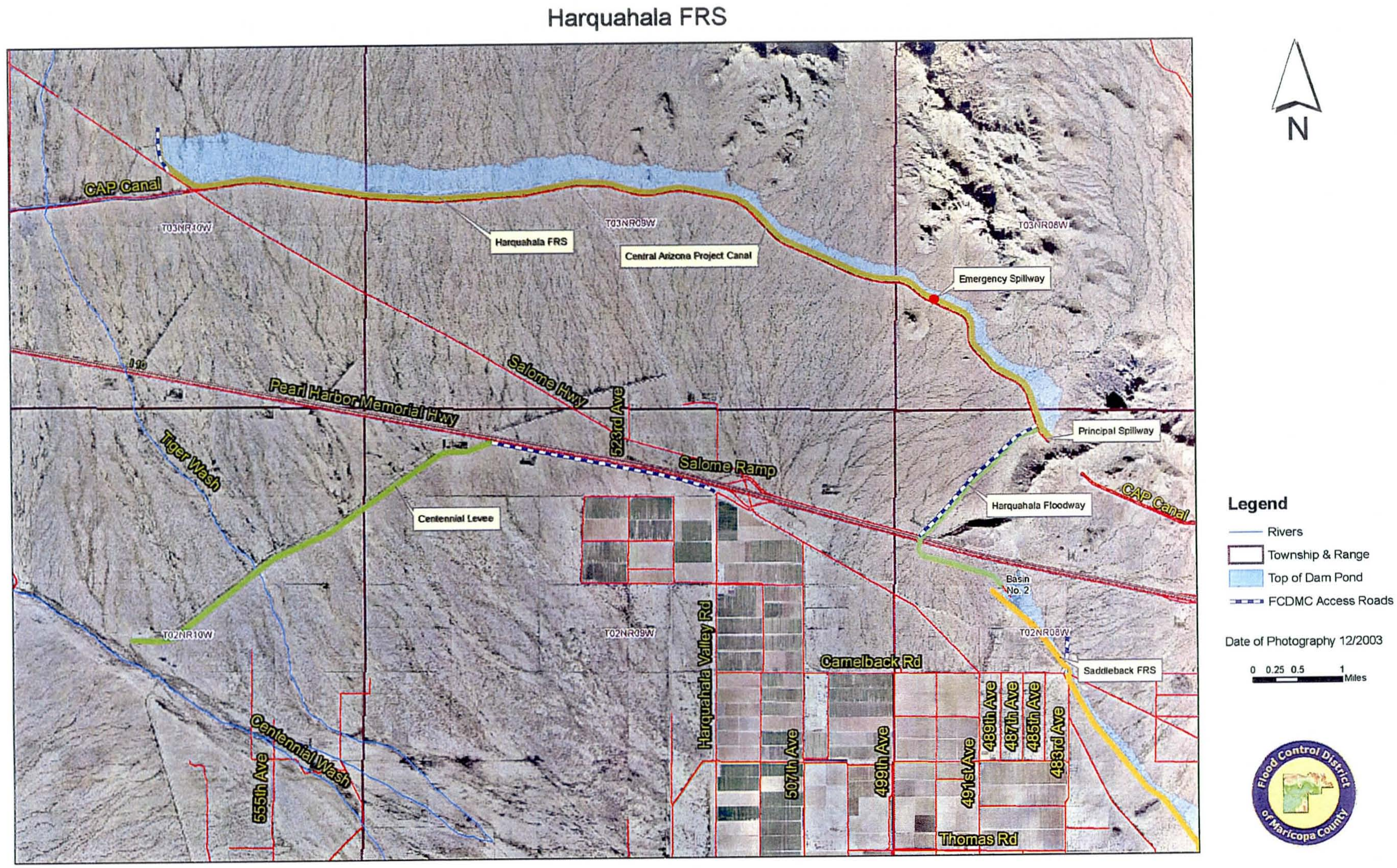

Figure 1A. System Map Harquahala FRS
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn and Associates, Inc.

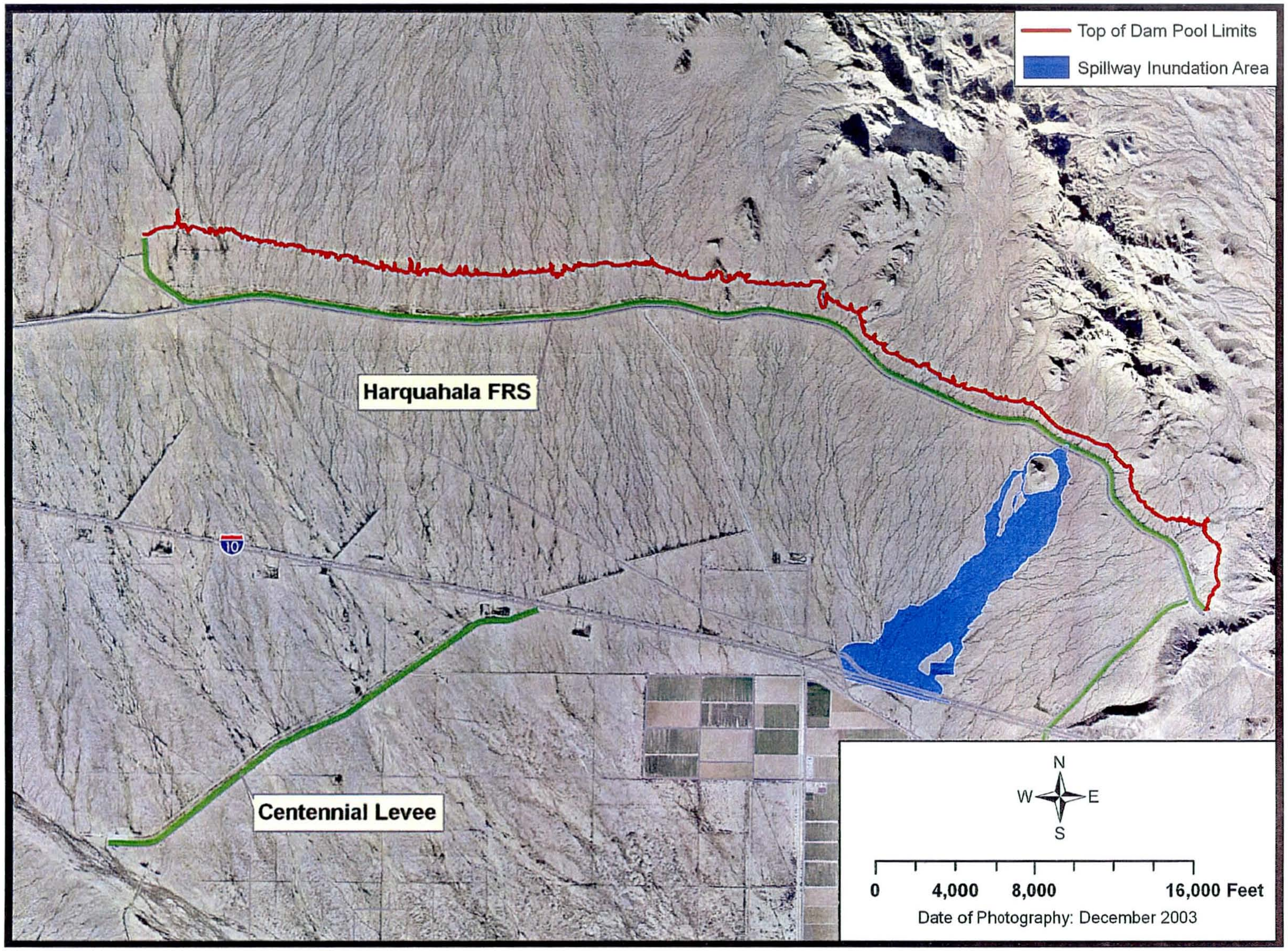

Figure 2. Harquahala FRS Spillway Inundation Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn
and Associates, Inc.

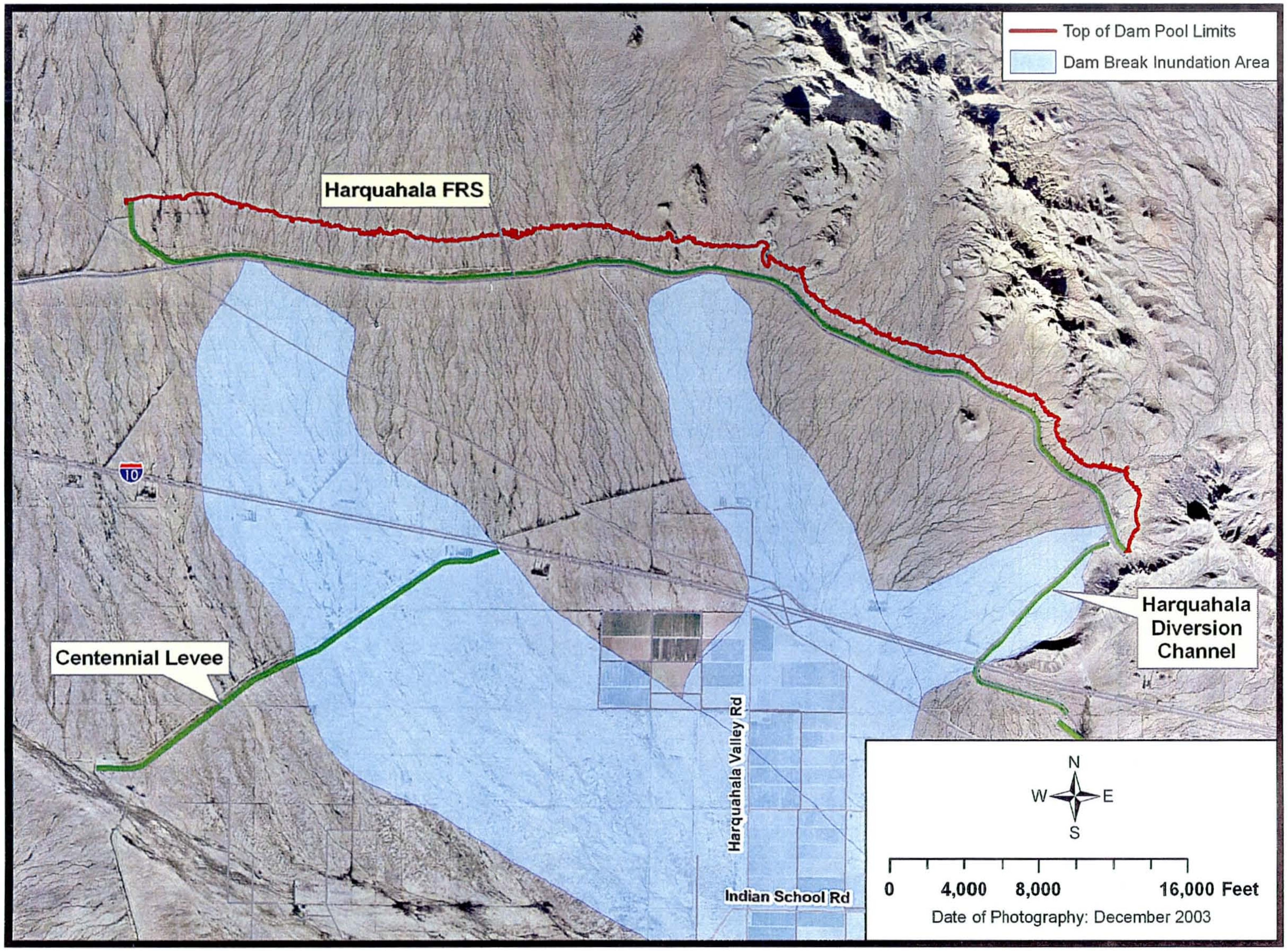

Figure 3. Harquahala FRS Dambreak Inundation Zone Inundation Area Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn
and Associates, Inc.

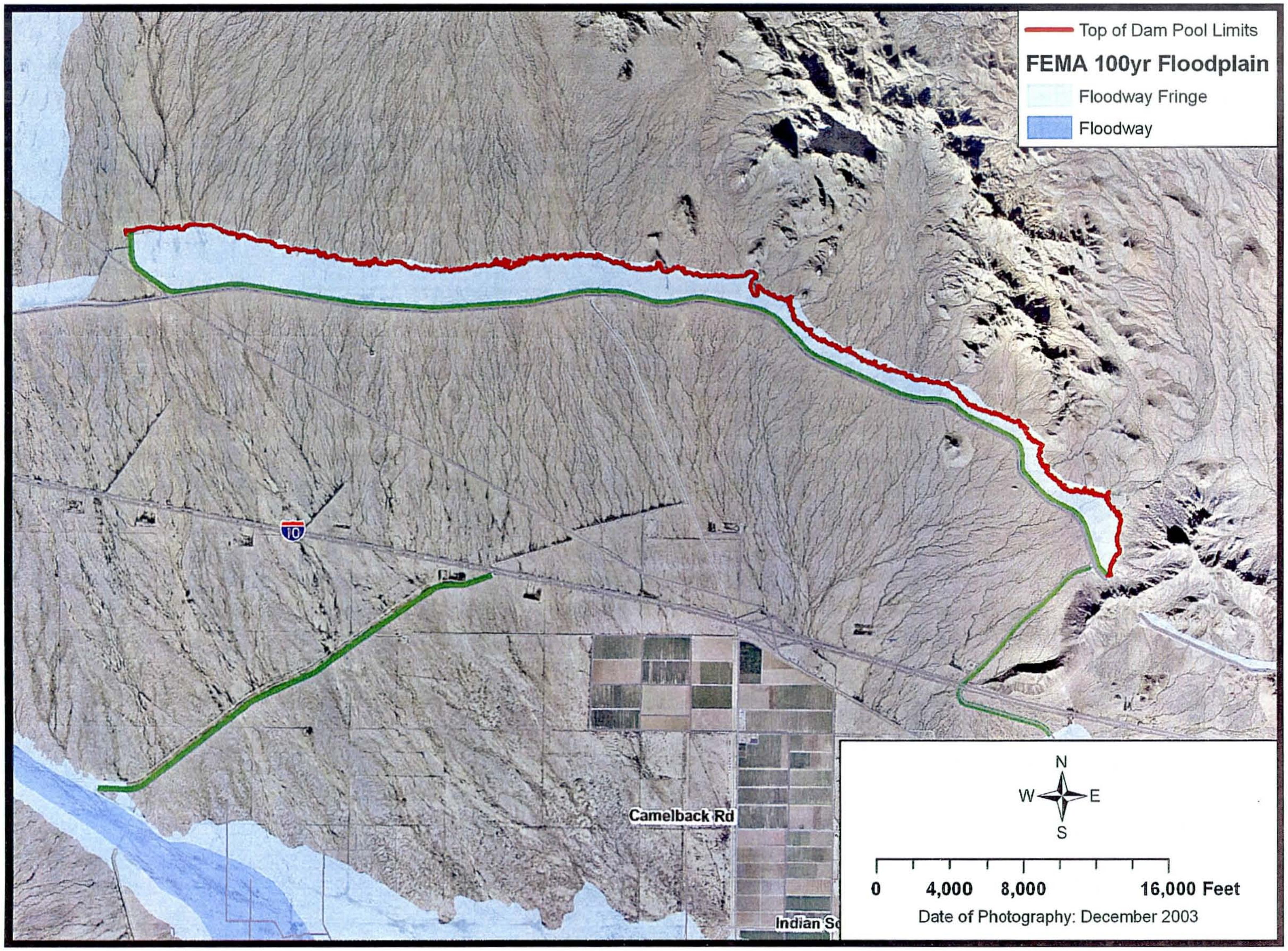

Figure 4. Harquahala FRS Top of Dam Pool Delineation Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn
and Associates, Inc.

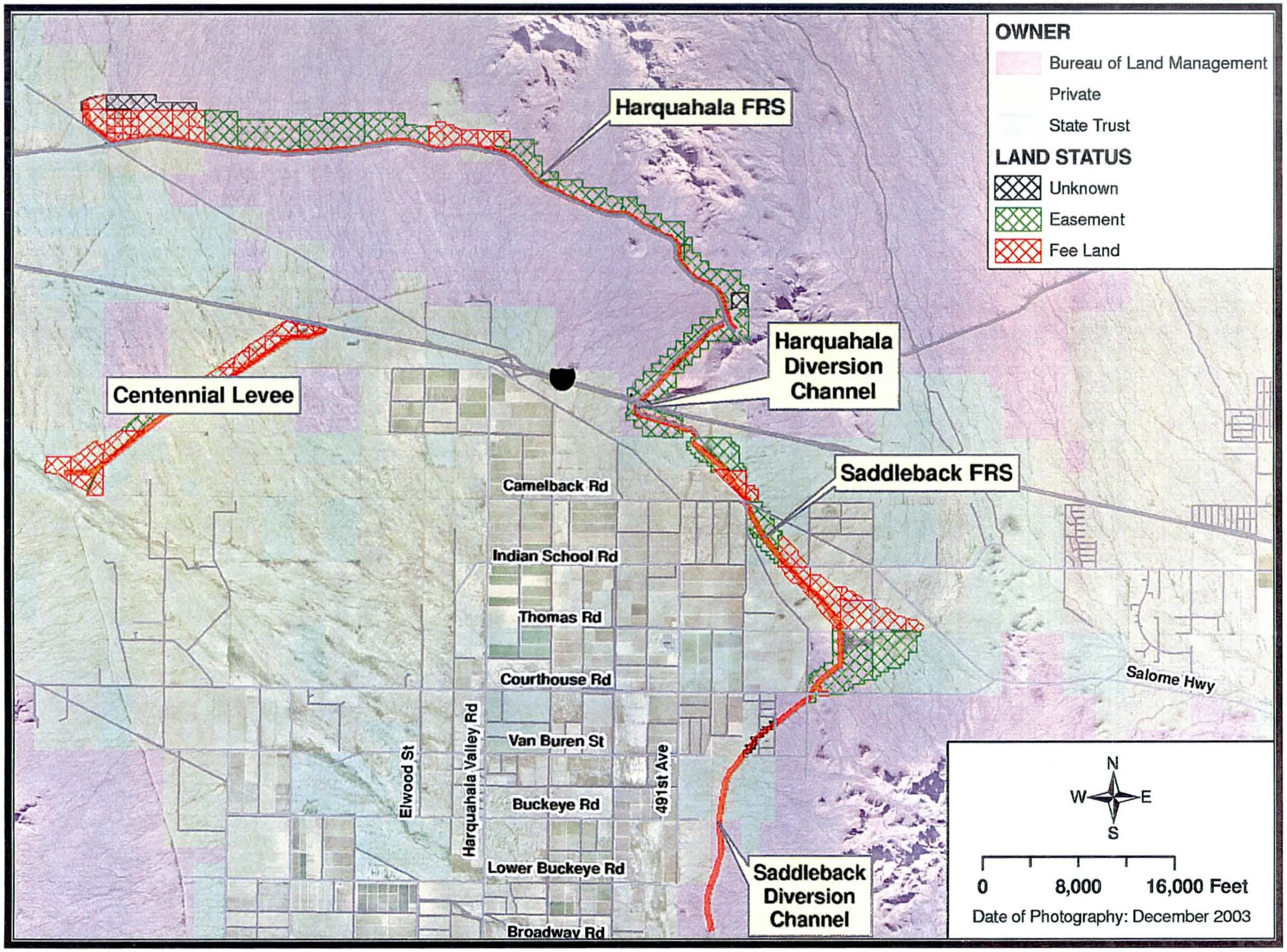

Figure 5. Landownership Map
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn
and Associates, Inc.

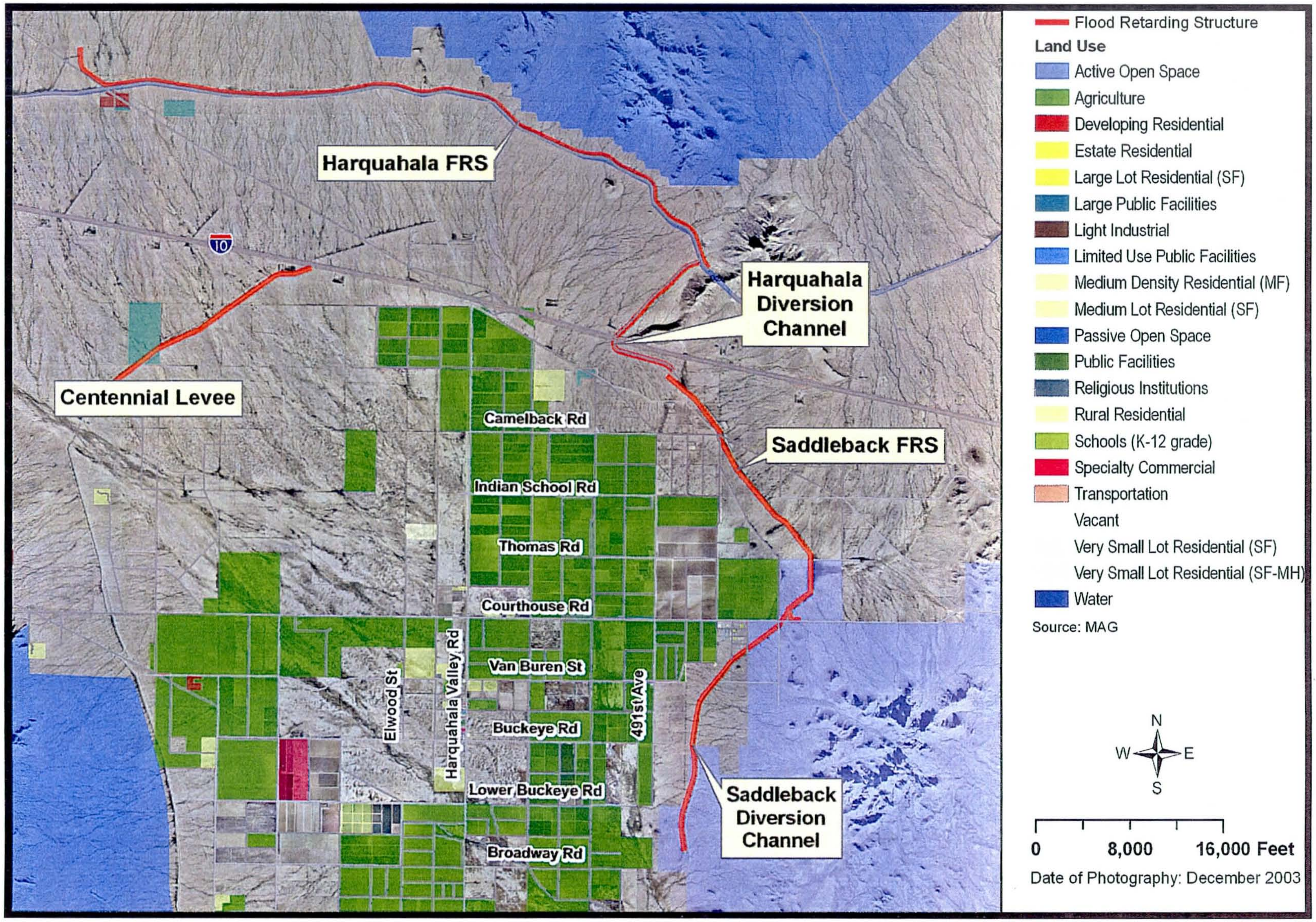

Figure 6. Current Land Use Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

Kimley-Horn and Associates, Inc.

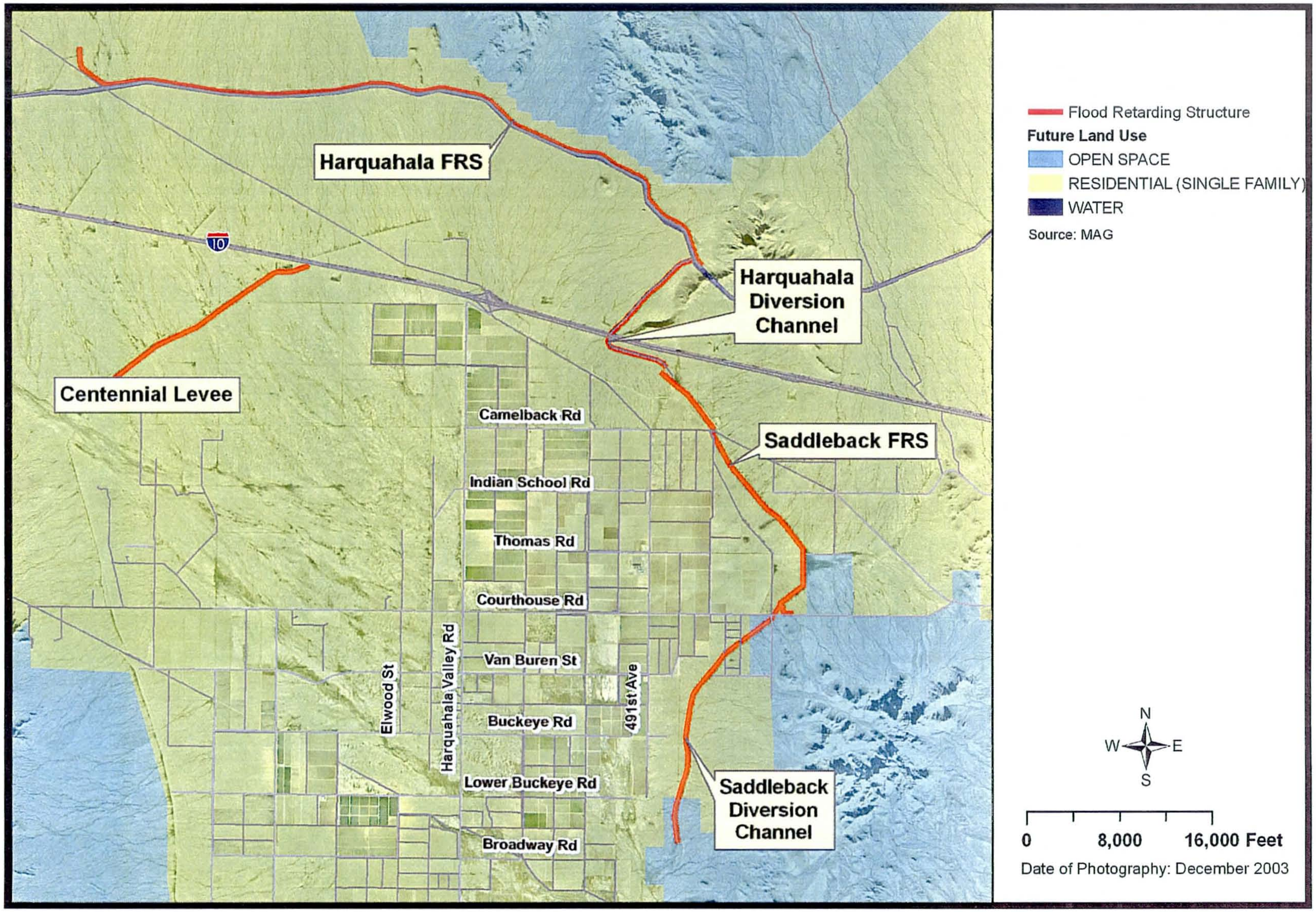

Figure 7. Future Land Use Map.
KHA Project No. 091131010

FCD2003C015
PCN: 05.03.01

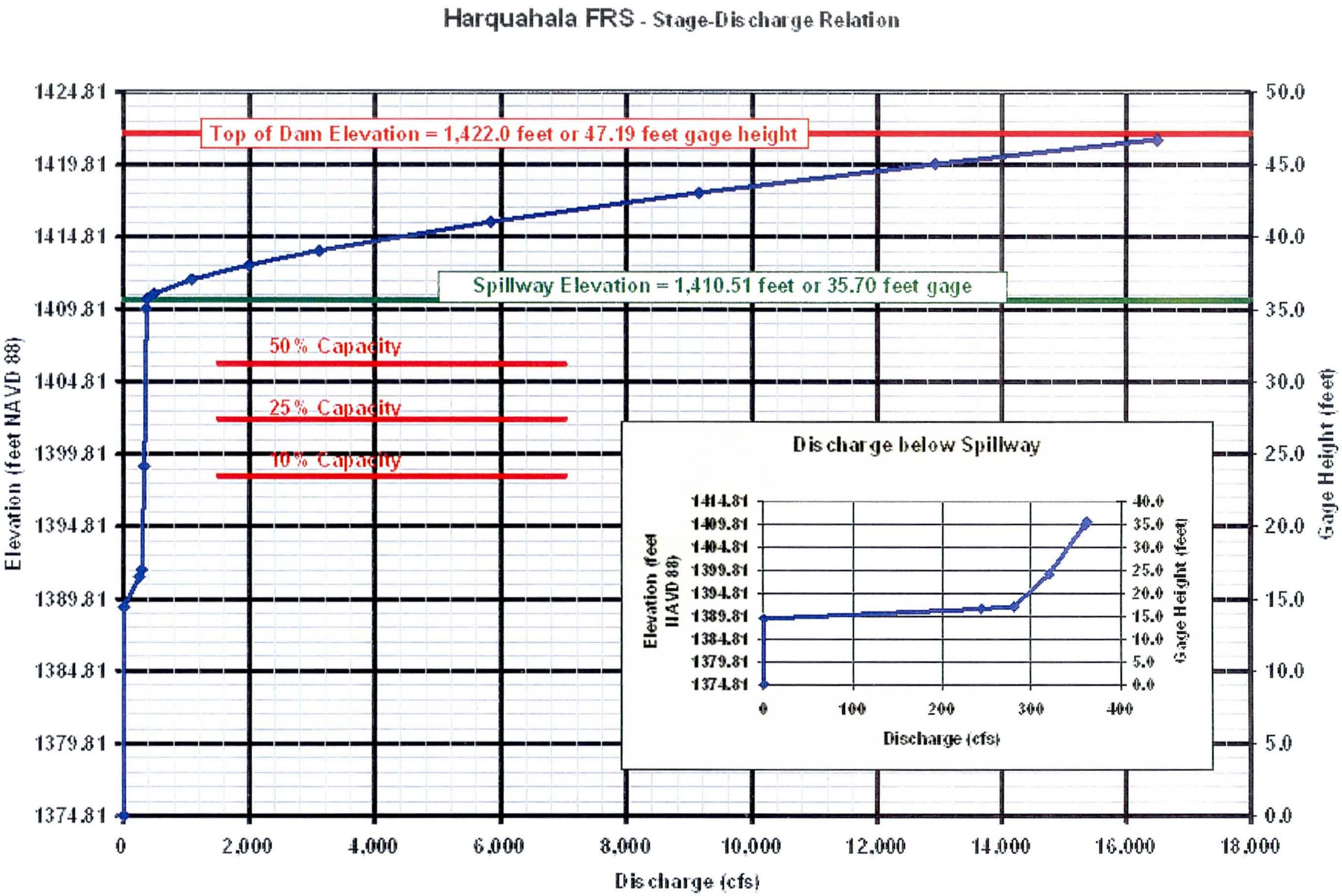

Figure 8. Stage-Discharge Rating Curve. (Source: District Website).

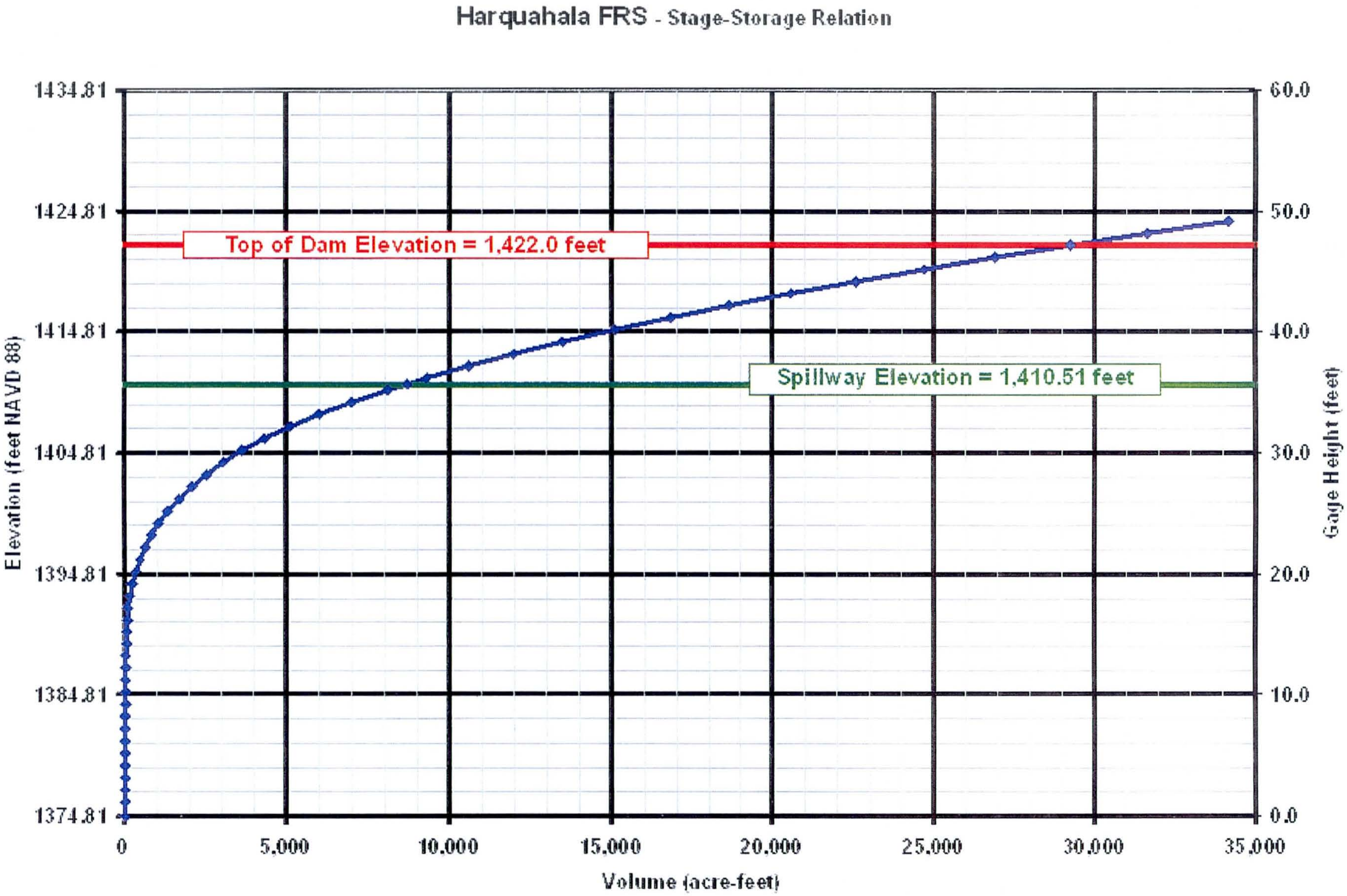

Figure 9. Stage-Storage Rating Curve. (Source: District Website).

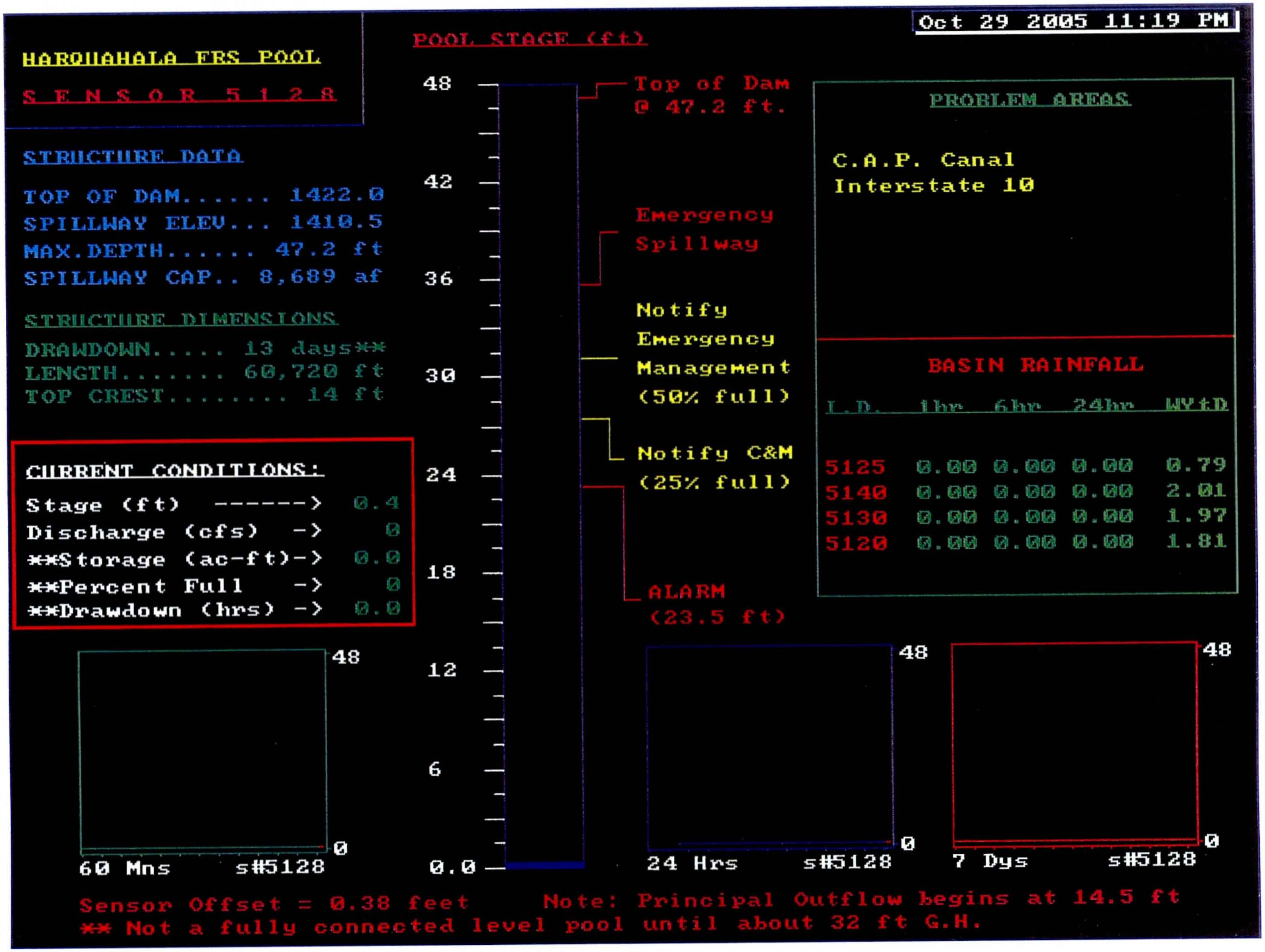

Figure 10. Harquahala FRS ALERT Gage Data Webpage (Source: District Website)

GEOLOGICAL CONSULTANTS INC

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

Memorandum

June 6, 2005

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

From: Ken Euge, R.G.
Principal Geologist

10338
KENNETH M.
EUGE

Subject: Geological Input to Structures Assessment Program, Phase 1
Harquahala 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 Harquahala FRS structures assessment report.

1.0 Geologic Setting

The Harquahala 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 northeast portion of the Harquahala Valley (Figure 1). 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 Harquahala F.R.S. is the Big Horn Mountains, which run north and northeast of the structure. The mountains are a series of faulted, tilted, Miocene volcanics composed primarily of basalt and rhyolite, along with Laramide-age metamorphics such as granodiorite, schist, and gneiss (Stimac, 1994). The geology of Burnt Mountain includes a variety of Tertiary age volcanic rock types that involved four different types of volcanic activity. The initial activity consisted of volcano-clastic ash flows and agglomerates followed by later sequences of tuff, andesite and basalt flows.

The valley basin fill includes late Tertiary and Quaternary deposits consisting of old alluvium composed of caliche-cemented, unconsolidated to semi-consolidated sand and gravel deposits (ADWR, 2004). The sedimentary sequence with the Harquahala basin 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.

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 may be completely absent in some areas. 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.

The geology along the Harquahala FRS alignment (Figure 2a & 2b) is dominated by Quaternary age alluvial fan deposits of the Upper Alluvial Unit, which is expected to be very thin, and the Lower Conglomerate Unit. The Middle Alluvial Unit is expected to be very thin or absent along Reach 1 From Stations 717+00 to 1043+00 (Reach 1), the upper alluvial fan deposit composed of very coarse grained sediments including sandy silty gravels and silty sandy gravels. According to the NRCS (1980), caliche was not found to be significant, however, it was found to be fairly common along many of the large washes. The caliche cemented sediment that are probably of Holocene age, were reportedly locally extensive but not uniformly widespread. Volcanic rhyolite bedrock underlies the alluvial fan deposits along this reach with outcrops exposed in rock knoll in the vicinity of Station 938+00 where the dam foundation encounters the volcanic rock. Another nearby volcanic rock outcrop is located a few hundred feet downstream from the emergency spillway.

The surficial geology along Reach 2, Station 443+00 to 717+00, is noticeably different from Reach 1. Thin stratified layers of silty to clayey sands, sandy to clayey silts, silty to sandy clays, and gravelly sands to silty gravelly sands predominate (NRCS, 1980). Silty sand (SM) is the more common soil type encountered along this reach with sequences ranging from about 10 to 25 feet thick at Stations 573+00, 613+00, 653+00, 663+00, 673+00, 683+00, 692+00, and 712+00. Thick sections of silty sand-clayey sand mixtures are found at Stations 593+00, 603+00, and 653+00. Relative loose surface soils up to 8 feet thick are commonly silty sand and silt (SM-ML) soils. Fine grained soils including CL and CL-ML soils are also found locally along the alignment.

Numerous ephemeral stream channels that contain loose, unconsolidated and pervious coarse grained soil cross the FRS alignment. According to the NRSC (1980), the presences of these deposits could pose a threat to the foundation unless the cutoff trench interrupts the continuity of these deposits. Where these soils could not be over-excavated and removed, portion of the upstream borrow area were blanketed with compacted fill. The blanketing was used from Stations 484+00 to 485+00, 511+50 to 513.30, 593+00 to 594+00, and 850+00 to 852+00 (NRCS Sheet 18 of 55; As-Built Drawing, May 13, 1983).

The geology of the Emergency Spillway (ES) and Principal Spillway (PS) is similar to the geology found along Reach 1. Volcanic bedrock was encountered in one drill hole DH-210 at a depth of 18.1 feet below a cover of caliche cemented fanglomerate at the ES alignment.

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 Harquahala 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 Harquahala 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 quake with a maximum calculated magnitude of 5.7, producing a maximum calculated peak horizontal acceleration at the Harquahala 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 Harquahala 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 Harquahala 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 Harquahala 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

The closest wells to the Harquahala F.R.S. are approximately 1.5 miles north and 2 miles south of the dam. In order to gather sufficient groundwater information, hydrographs for wells within approximately 4 miles of the Harquahala F.R.S. were obtained from the Arizona Department of Water Resources (Appendix A) (Figure 5). Eleven hydrographs were obtained, with the oldest dating back to 1952. These hydrographs show an overall decline in groundwater levels of 2 to 200 feet. Four of the wells show an increase in water levels of between 7 and 95 feet but most of the wells show a slow but continuous decline in groundwater 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 Harquahala 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). The data that are plotted in Figure 6 may be used as an indicator of the relative recent land subsidence that may have occurred or is occurring in the project area. As can be seen in Figure 6, the change in elevation is greatest along Reach 2 of the FRS where there is an apparent thickening of the basin fill sediments beyond the area where the buried bedrock surface drops-off from the edge of the pediment (Figure 8).

According to this data, it appears that some settlement or subsidence has occurred, mainly on the western portion of the dam between monuments A-1 and A-36, from 1984 to 2003 (Figure 6). The change in elevation in this area ranges from -0.015 to -0.480 feet. The eastern portion of the dam has not experienced any apparent settlement or subsidence because along this portion of the alignment, bedrock is relatively close to the surface.

Table 1
Change in Elevation 1984 – 2003 (adjusted to 1984 datum)

Crest Marker	Change in Elevation (feet)		Toe Marker	Change in Elevation (feet)
A-1	-0.229		B-1	-0.228
A-2	-0.293		B-2	-0.241
A-3	-0.317		B-3	-0.042
A-4	-0.315		B-4	-0.248
A-5	-0.480		B-5	-0.031
A-6	-0.389		B-6	-0.240
A-7	-0.303		B-7	-0.159
A-8	-0.243		B-8	0.207
A-9	-0.363		B-9	-0.183
A-10	-0.237		B-10	-0.159
A-11	-0.266		B-11	0.200
A-12	-0.307		B-12	-0.166
A-13	-0.453		B-13	-0.163
A-14	-0.331		B-14	-0.180
A-15	-0.282		B-15	0.820
A-16	-0.268		B-16	-0.148
A-17	-0.283		B-17	-0.051
A-18	-0.265		B-18	-0.038
A-19	-0.215		B-19	-0.016
A-20	-0.227		B-20	-0.041
A-21	-0.165		B-21	0.328
A-22	-0.194		B-22	-0.086
A-23	-0.180		B-23	-0.136
A-24	-0.232		B-24	-0.137
A-25	-0.154		B-25	-0.090
A-26	-0.176		B-26	-0.049
A-27	-0.096		B-27	-0.050
A-28	-0.065		B-28	-0.034
A-29	-0.054		B-29	-0.055
A-30	-0.015		B-30	0.024
A-31	-0.033		B-31	0.021
A-32	0.013		B-32	0.037
A-33	-0.057		B-33	0.088
A-34	-0.026		B-34	0.078
A-35	-0.048		B-35	0.048
A-36	-0.022		B-36	0.055
A-37	0.009		B-37	0.015
A-38	0.003		B-38	0.015
A-39	0.066		B-39	-0.001
A-40	0.047		B-40	0.017
A-41	0.057		B-41	0.048
A-42	0.056		B-42	0.042
A-53	0.066		B-53	0.056
A-44	0.034		B-44	0.074
A-45	0.062		B-45	0.082
A-46	0.060		B-46	0.078
A-47	0.048		B-47	0.040
A-48	0.055		B-48	0.078

A-49	0.070		B-49	0.048
A-50	0.076		B-50	0.056
A-51	0.071		B-51	0.079
A-52	0.082		B-52	0.065
A-53	0.085		B-53	0.042
A-54	0.074		B-54	0.000
A-55	0.042		B-55	0.000
A-56	0.048		B-56	0.021
A-57	0.030		B-57	0.077
A-59	0.030		B-58	0.043
A-59	0.023		B-59	0.018
A-60	0.032		B-60	-0.004
A-61	0.034		B-61	-0.292
A-62	0.038		B-62	0.026

(Flood Control District of Maricopa County, Dam Safety Program, 2004)

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 Harquahala F.R.S. vicinity (Oppenheimer, 1980). The Oppenheimer map estimated the depth to 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. No unusual buried bedrock highs were interpreted within the project area from this data.

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). Although these maps do not cover the Harquahala F.R.S. site, Geological Consultants, Inc. has extrapolated the contour lines into the project vicinity. As depicted in Figure 8, a relatively prominent bedrock boundary condition can be deduced that reflects the approximate buried limit of the volcanic rock. It is possible that this boundary between the volcanic bedrock and the basin fill alluvial sediments could be the focus for earth fissure development at or near the Harquahala FRS.

3.3.1 Known Earth Fissures in the Project Vicinity

There are three earth fissures reported in the Harquahala Valley. The closest fissure to the Harquahala F.R.S. lies approximately 3.4 miles southwest of the structure in Section 9, Township 2 North, Range 9 West (Figure 9). 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).

Another earth fissure was documented in 1961 in a farm field about 4.8 miles south of the Harquahala F.R.S. 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.

The Rogers fissure was discovered in 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 5.9 miles southwest 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 Harquahala 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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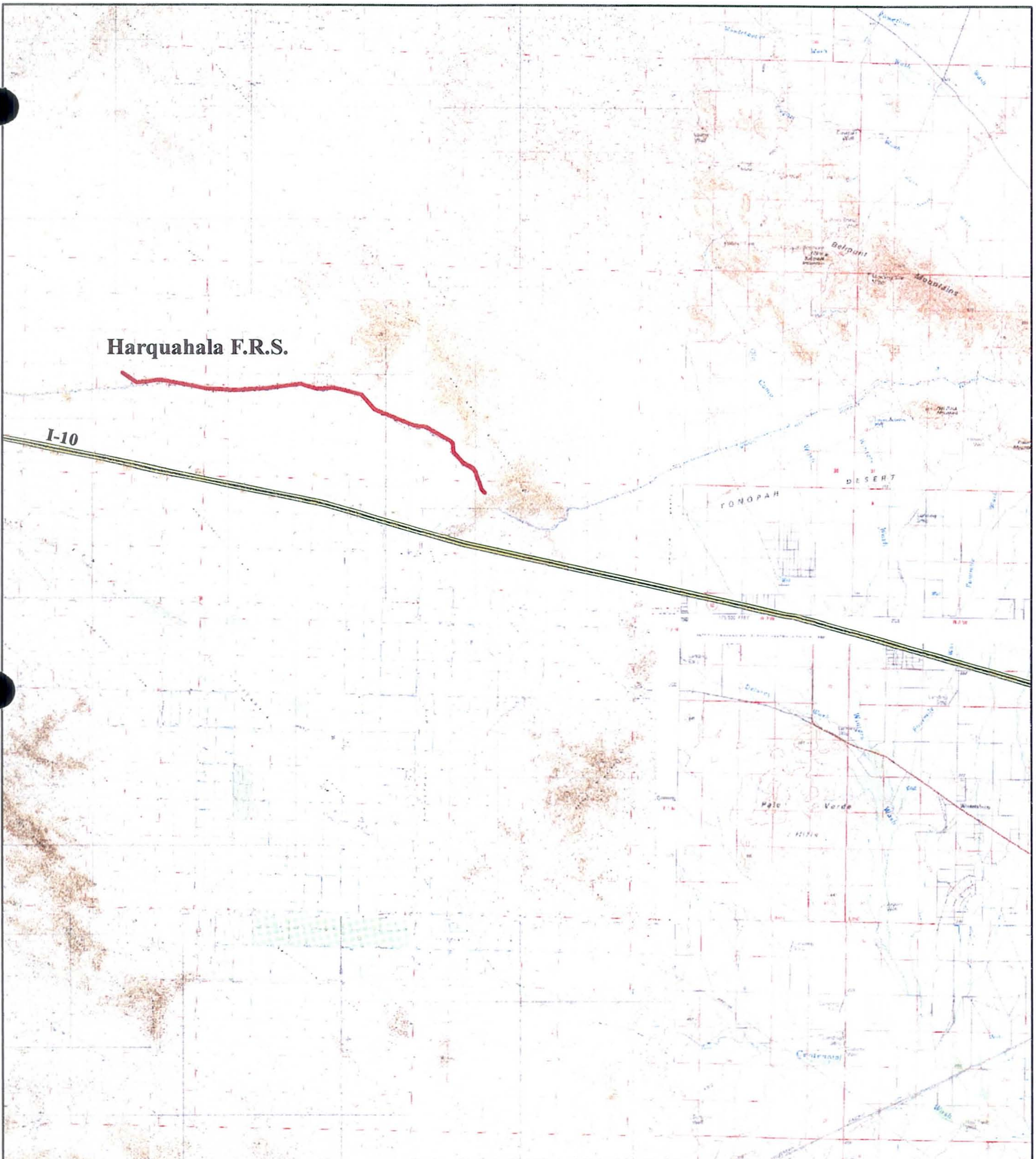
Schumann, H.H. and Genualdi, Robert B.; 1986; Land Subsidence, Earth Fissures, and Water-Level Change in Southern Arizona; Arizona Bureau of Geology and Mineral Technology, Geological Survey Branch, Map 23, 1 sheet.

Soils Conservation Service; 1978; Geological Investigation Summary & Test Results; Harquahala FRS & Floodway, Harquahala Valley Watershed, Maricopa & Yuma Counties, Arizona;

Stimac, James A., Richard, Stephen M., Reynolds, Stephen J., Capps, Richard C., Kortemeier, Curtis P., Grubensky, Michael J., Allen, George B., Gray, Floyd., and Miller, Robert J.; 1994; Big Horn and Belmont Mountains, West-Central Arizona; Arizona Geological Survey Open File Report No., 94-15; 3 sheets, 16 pp, 1:50,000.



N

1:240,000

Harquahala F.R.S.
Location Map
Figure 1

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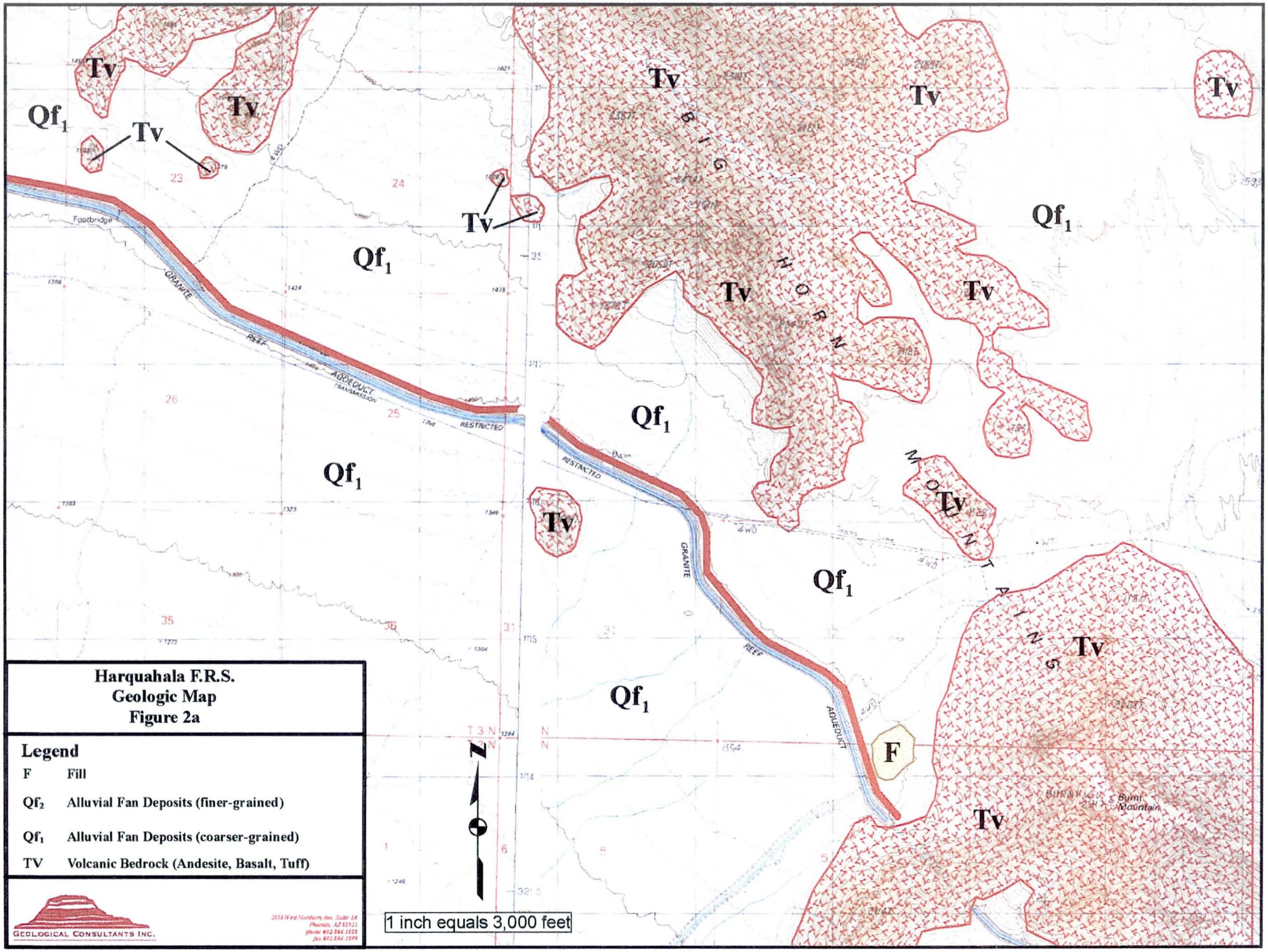
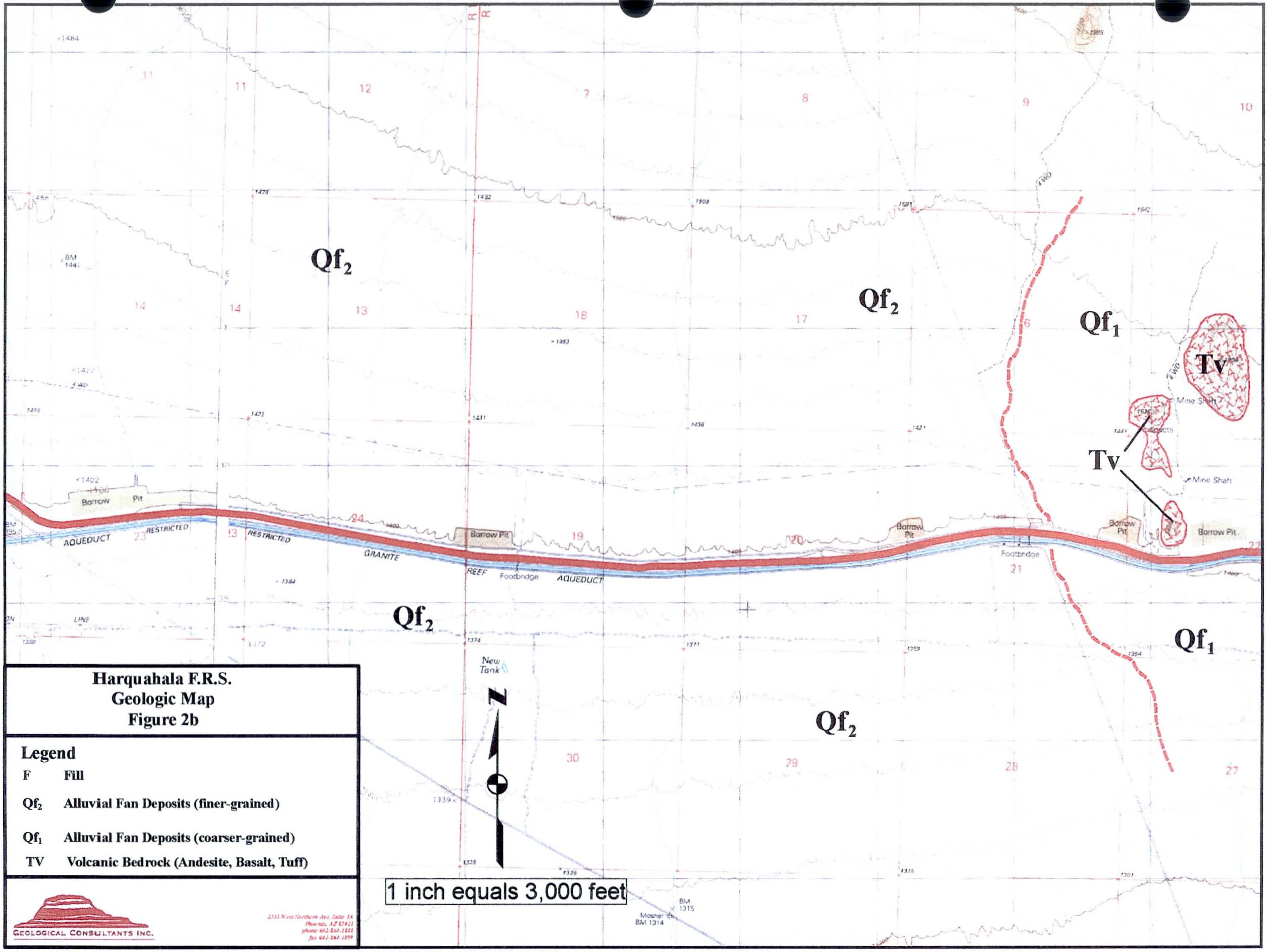
fax 602-864-1899

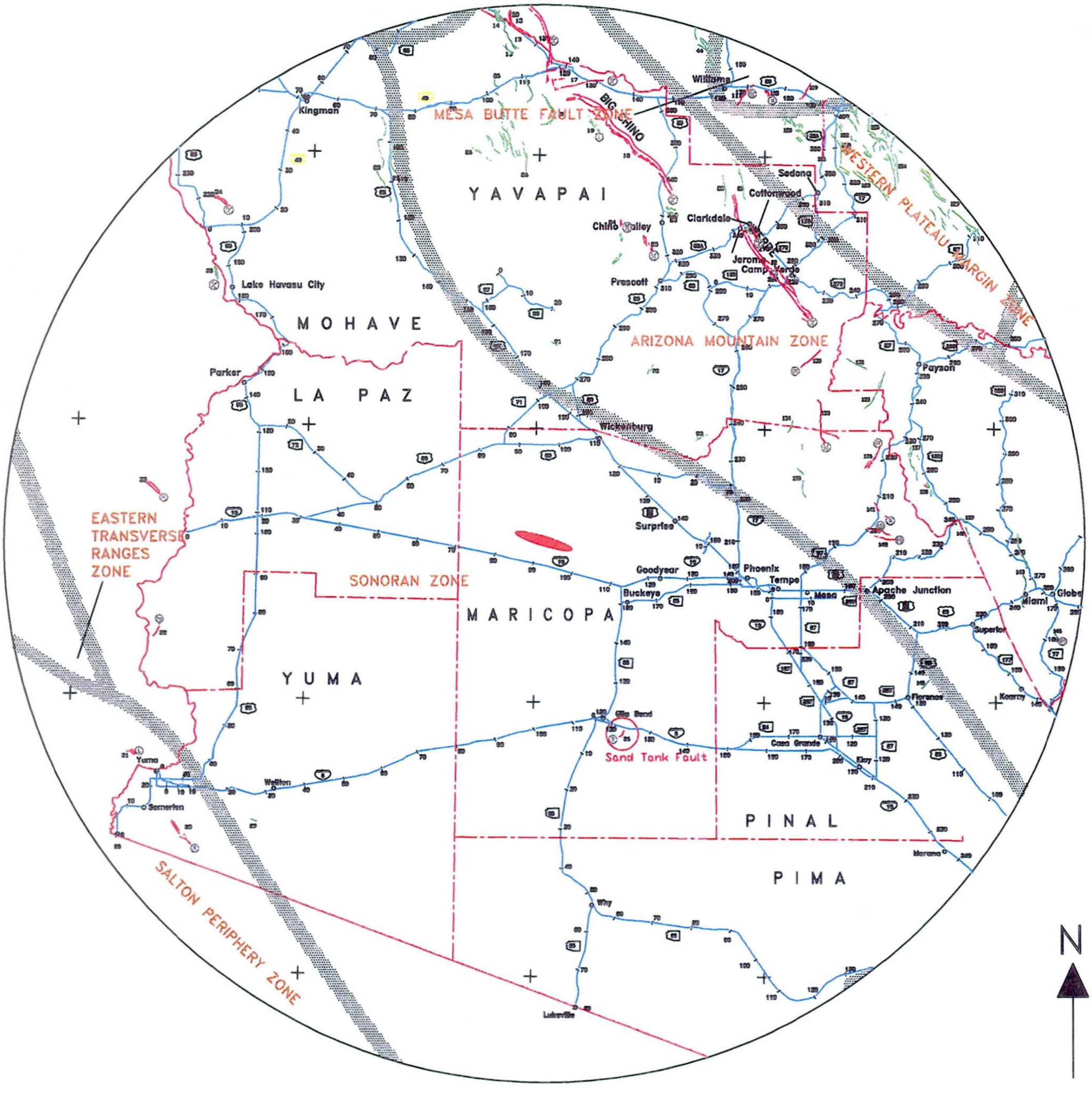

Harquahala F.R.S.
Geologic Map
Figure 2a
Legend
F Fill
Qf_2 Alluvial Fan Deposits (finer-grained)
Qf_1 Alluvial Fan Deposits (coarser-grained)
TV Volcanic Bedrock (Andesite, Basalt, Tuff)
GEOLOGICAL CONSULTANTS INC.
1 inch equals 3,000 feet
N
Tv
Qf_1
F
BIG HORN MOUNTAINS
GRANITE REEF AQUEDUCT
RESTRICTED
Footbridge
Dam
4WD
Burnt Mountain
T 3 N



Harquahala F.R.S.
Geologic Map
Figure 2b

Legend

F	Fill
Qf_2	Alluvial Fan Deposits (finer-grained)
Qf_1	Alluvial Fan Deposits (coarser-grained)
TV	Volcanic Bedrock (Andesite, Basalt, Tuff)



EXPLANATION

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

			Approximate Age (million years before present)
Q - Quaternary < 2 my B.P.	Qy - late Quaternary < 0.5 my B.P.	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. Euge, and Ignatius Po Lam, 1992)

Harquahala F.R.S.
Fault Map
Figure 3

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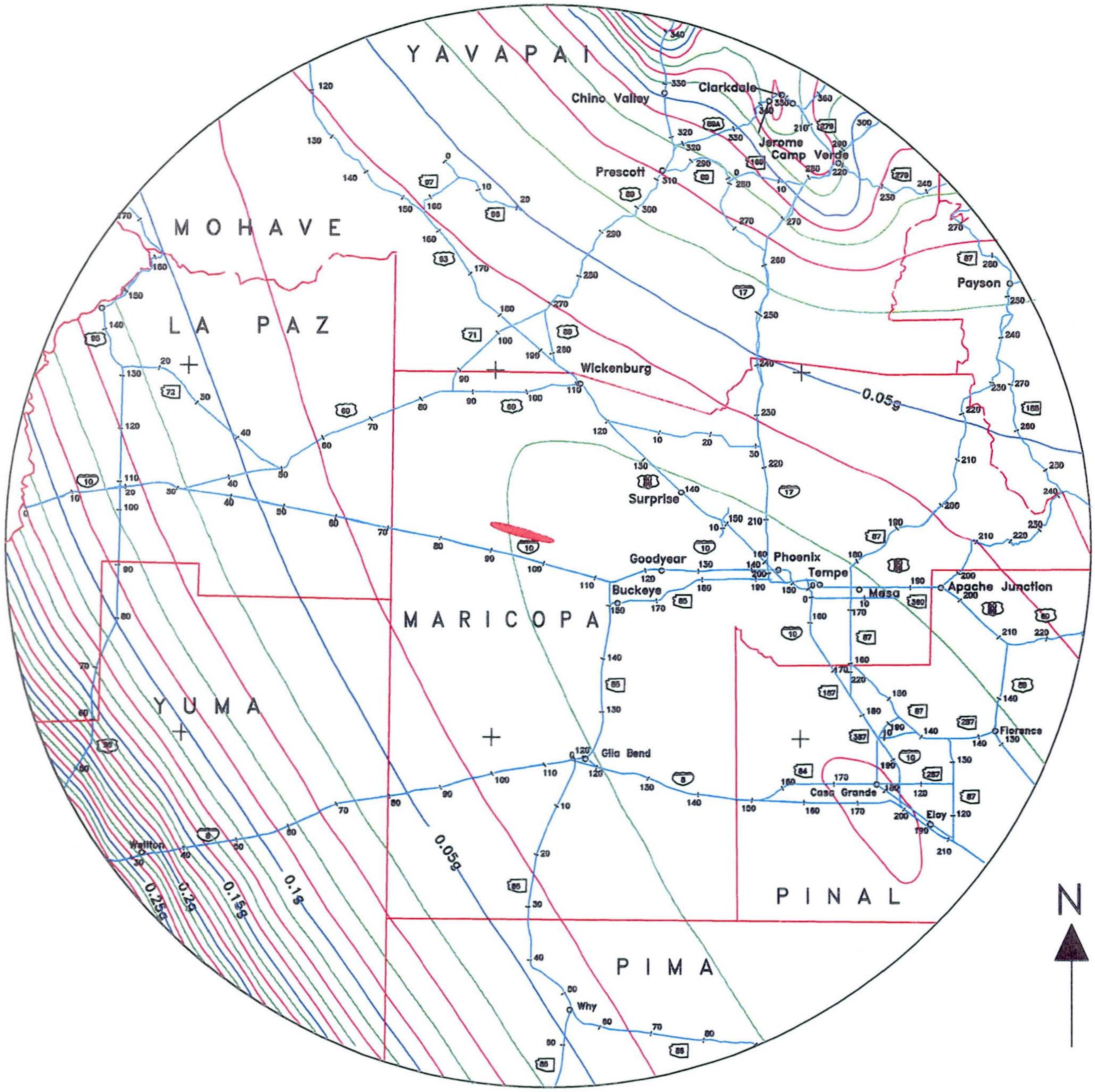
fax 602-864-1899



Legend

Approximate location of Harquahala 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)

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

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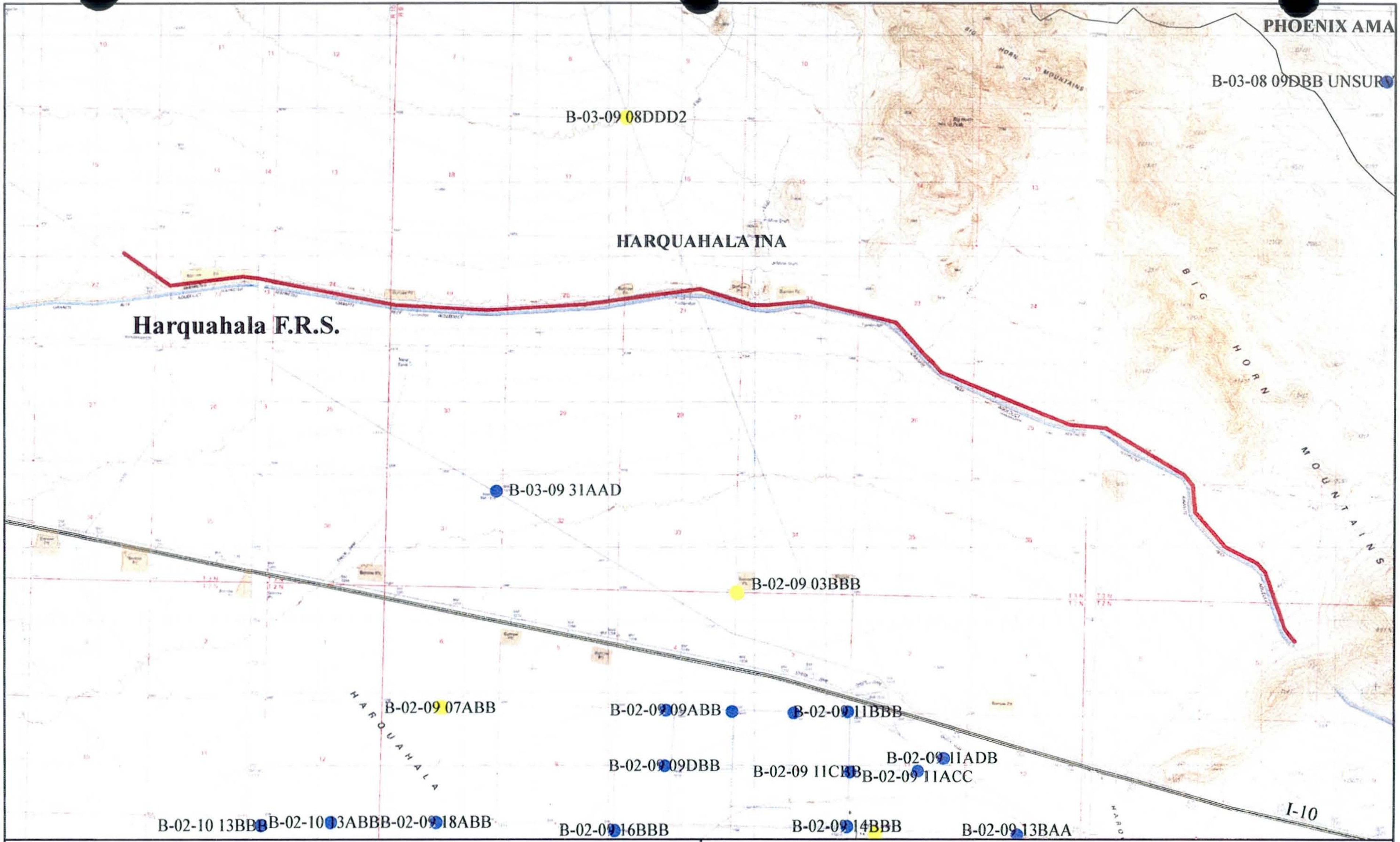
fax 602-864-1899



Legend

Well

Index Well

(from ADWR database CD-ROM, 2004)

Harquahala F.R.S.
Well Locations
Figure 5

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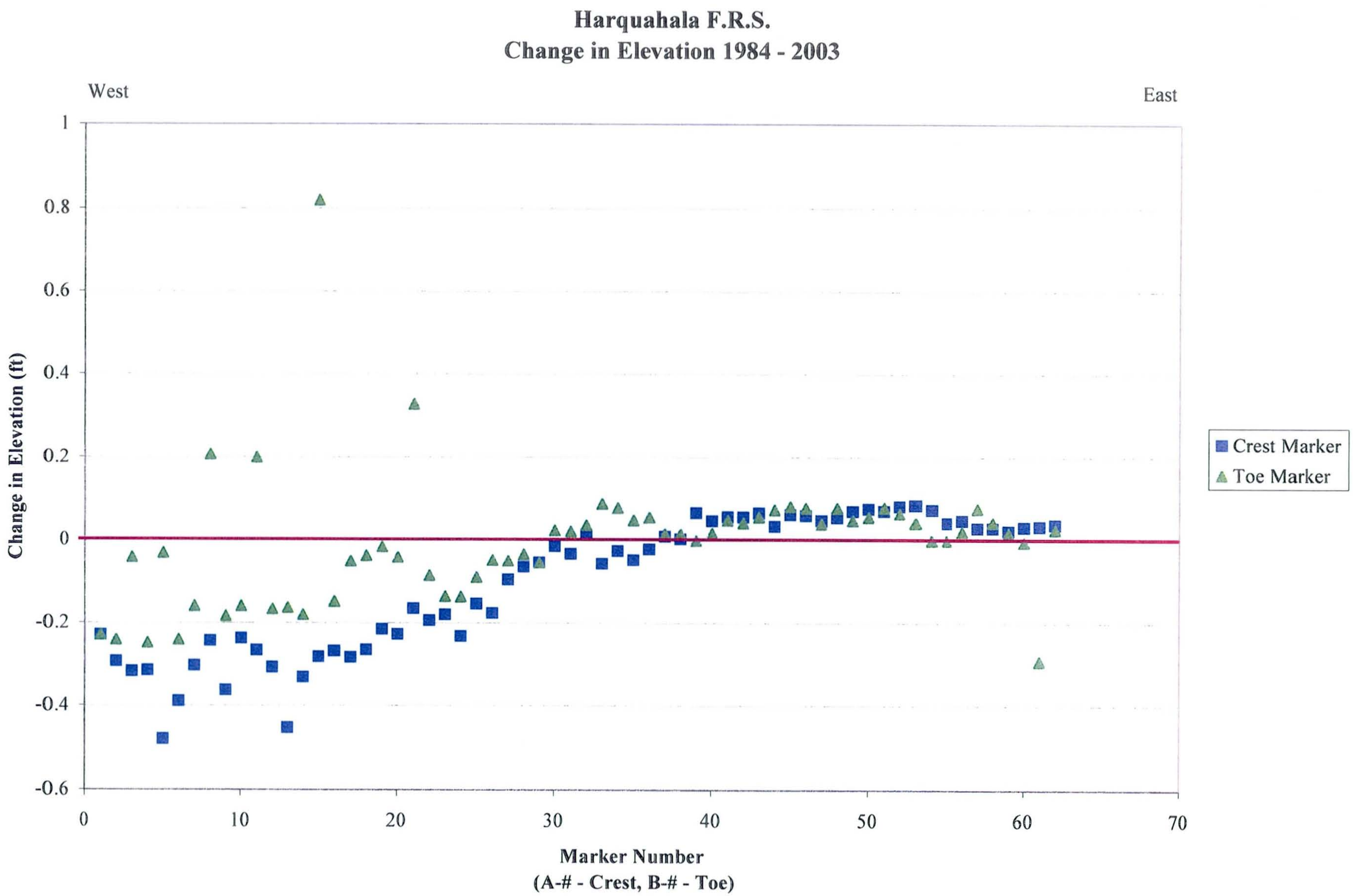
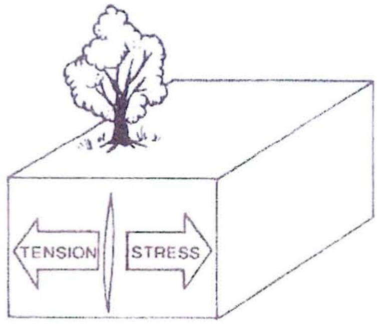
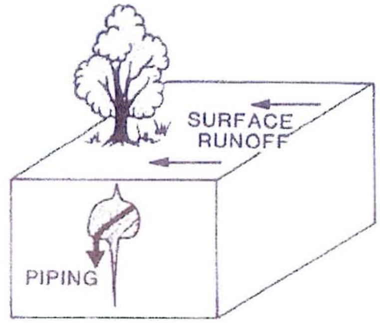


(Data from Flood Control District of Maricopa County, 2004)

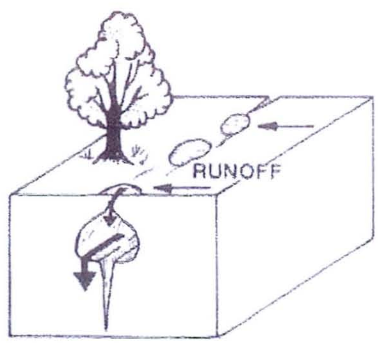
Harquahala F.R.S.
Geological Consultants Project No. 2003-161
Figure 6



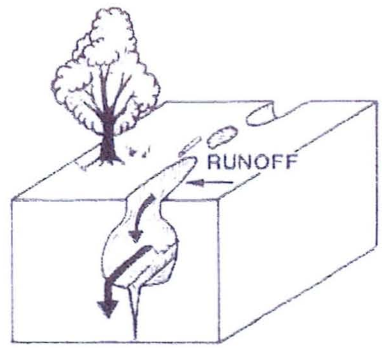
1. Lateral stresses induce tension cracking



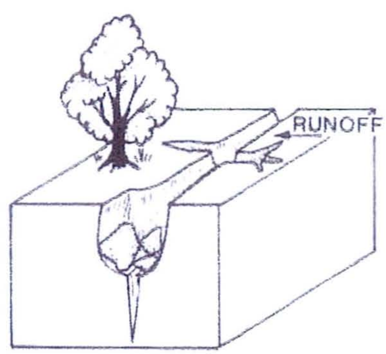
2. Surface runoff and infiltration enlarge crack through subsurface piping



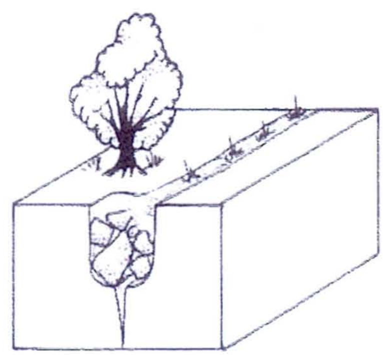
3. As piping continues, fissure begins to appear at surface as series of potholes and small cracks



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

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

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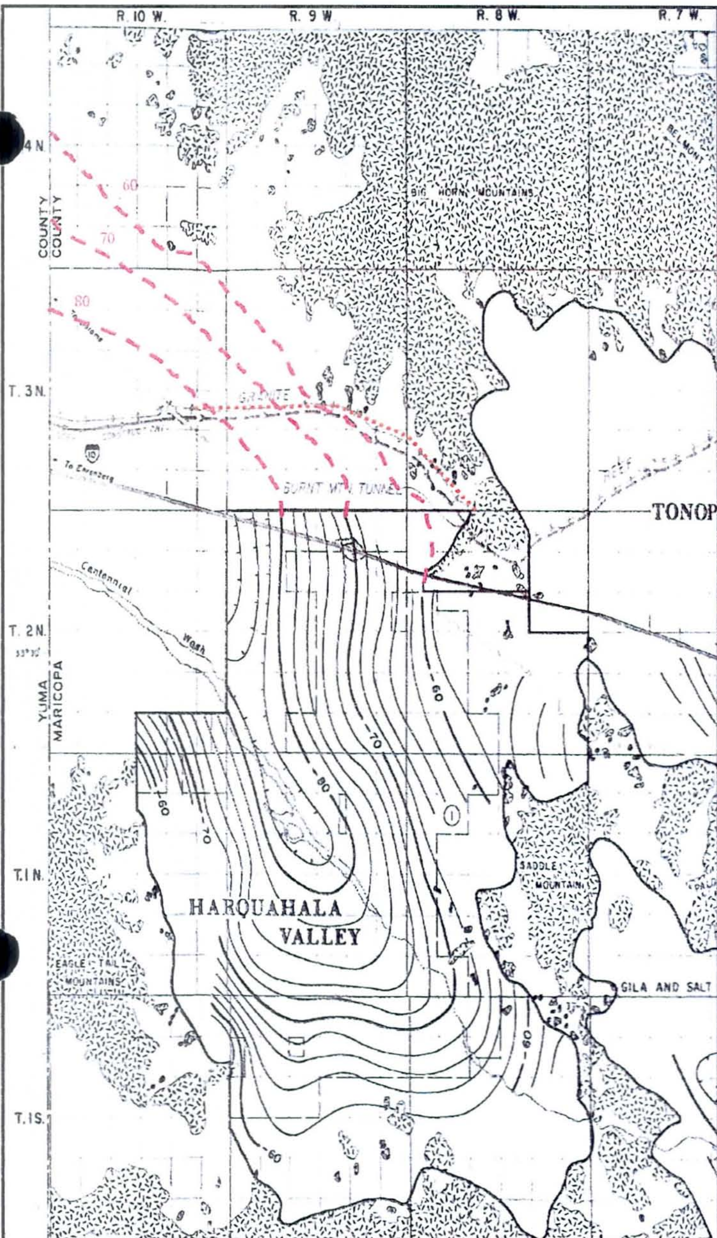
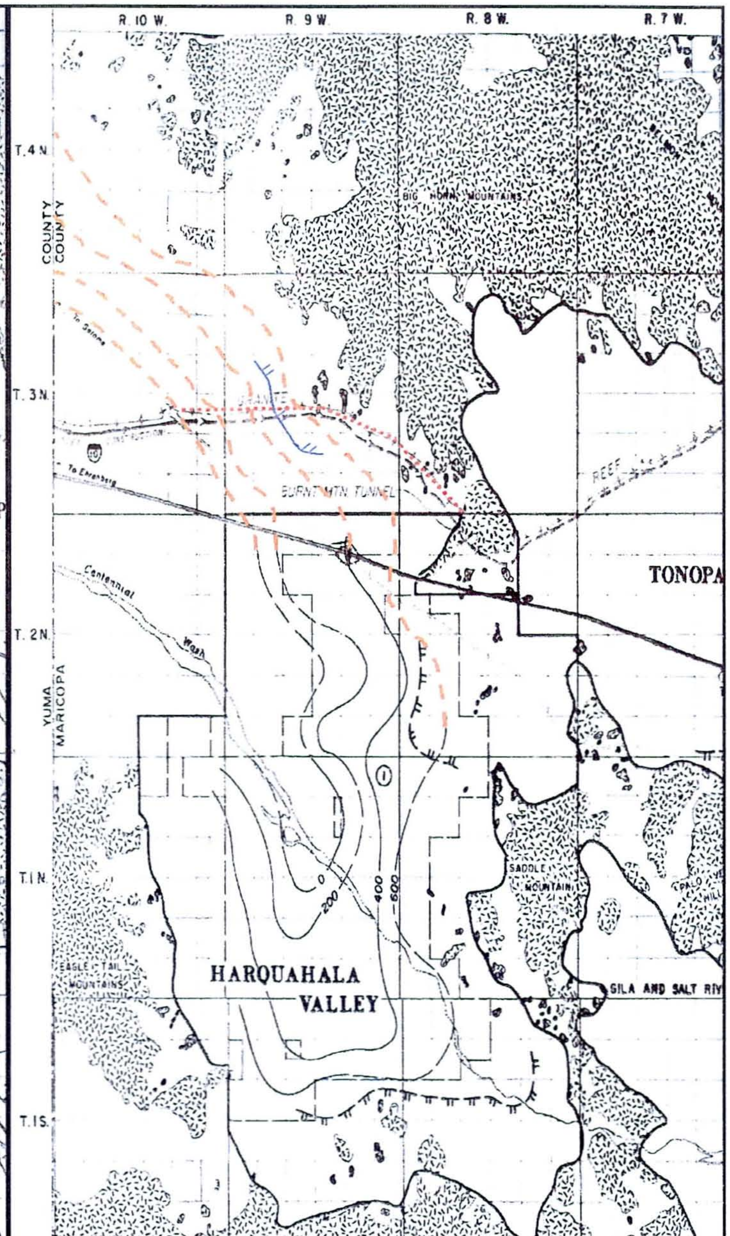


Bouguer Anomaly Map* showing gravity contours (dashed where inferred), contour interval 2 milligals.

Extrapolation of gravity contour lines (GCI, 2005)



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

Extrapolation of structure contour lines, (GCI, 2005)

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

Approximate subsurface extent of Quaternary Tertiary Volcanic rock adjacent to or interbedded with the Lower Conglomerate Unit. (GCI, 2005)

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

*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.

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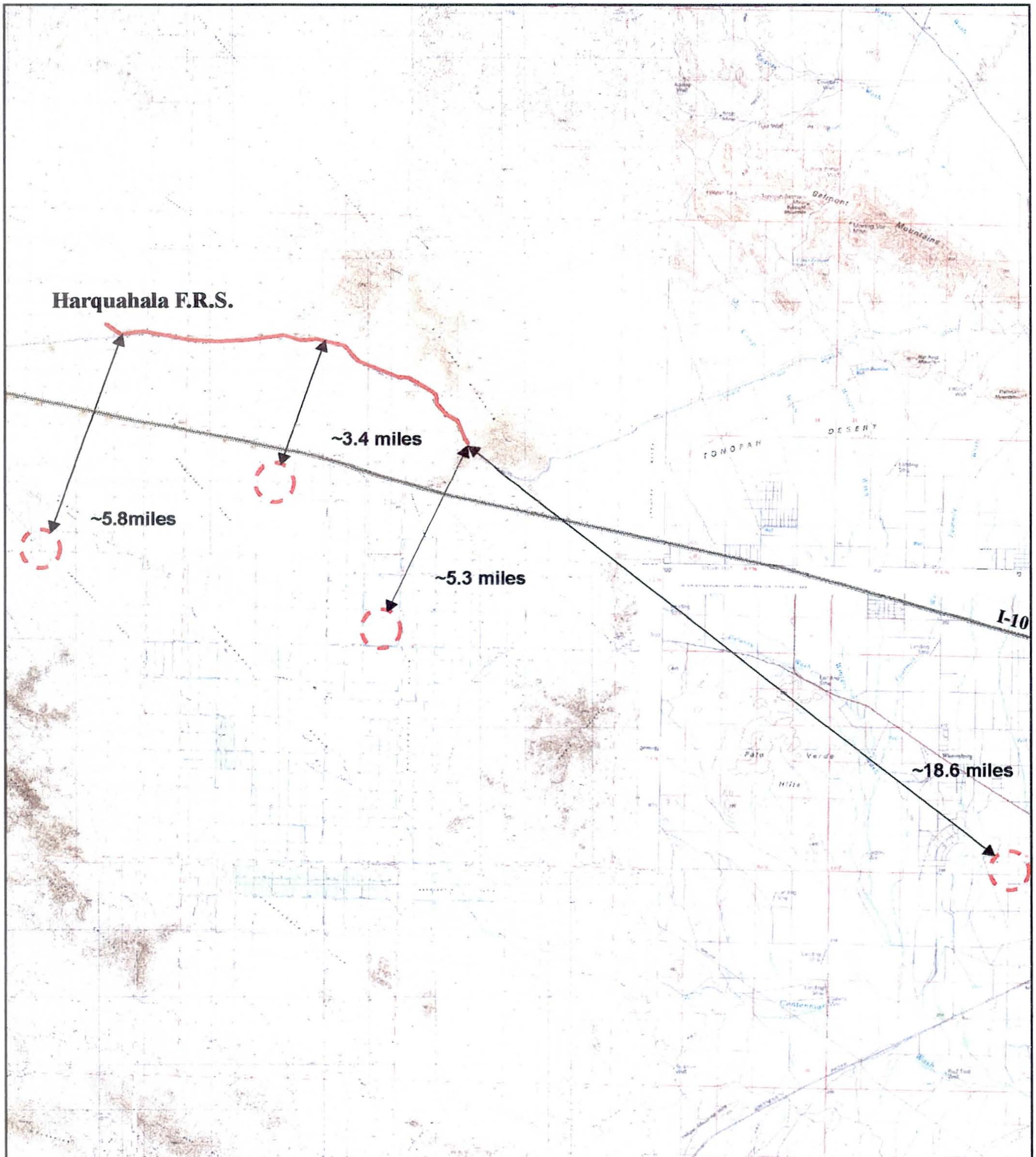

(from ADWR database CD-ROM, 2004)

N

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

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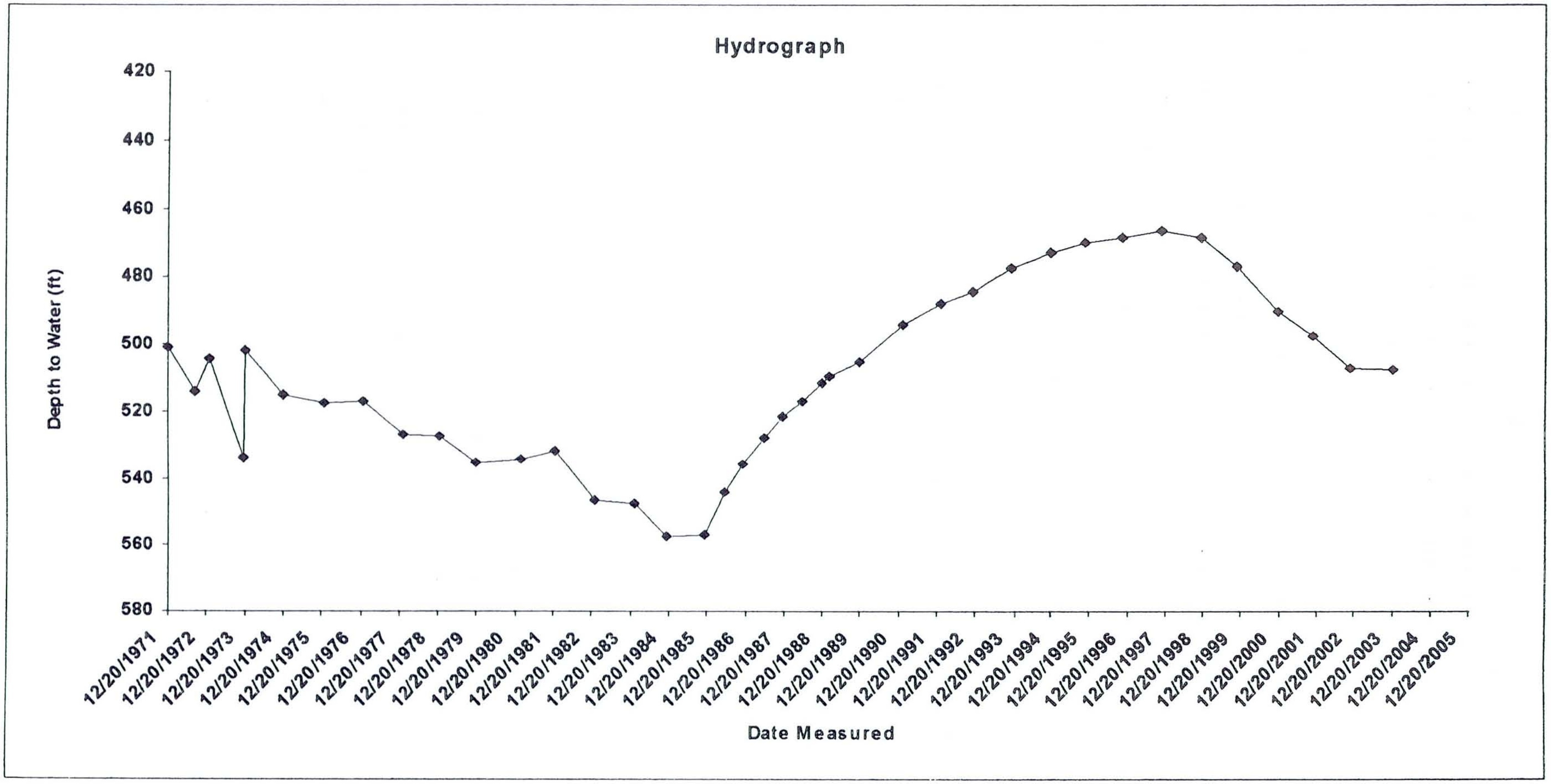
fax 602-864-1899

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-02-09 03BBB	333305113104301		33° 33´ 5.5´´	113° 10´ 41.9´´	UNUSED			18	12/23/2003	507.90	752.1	39

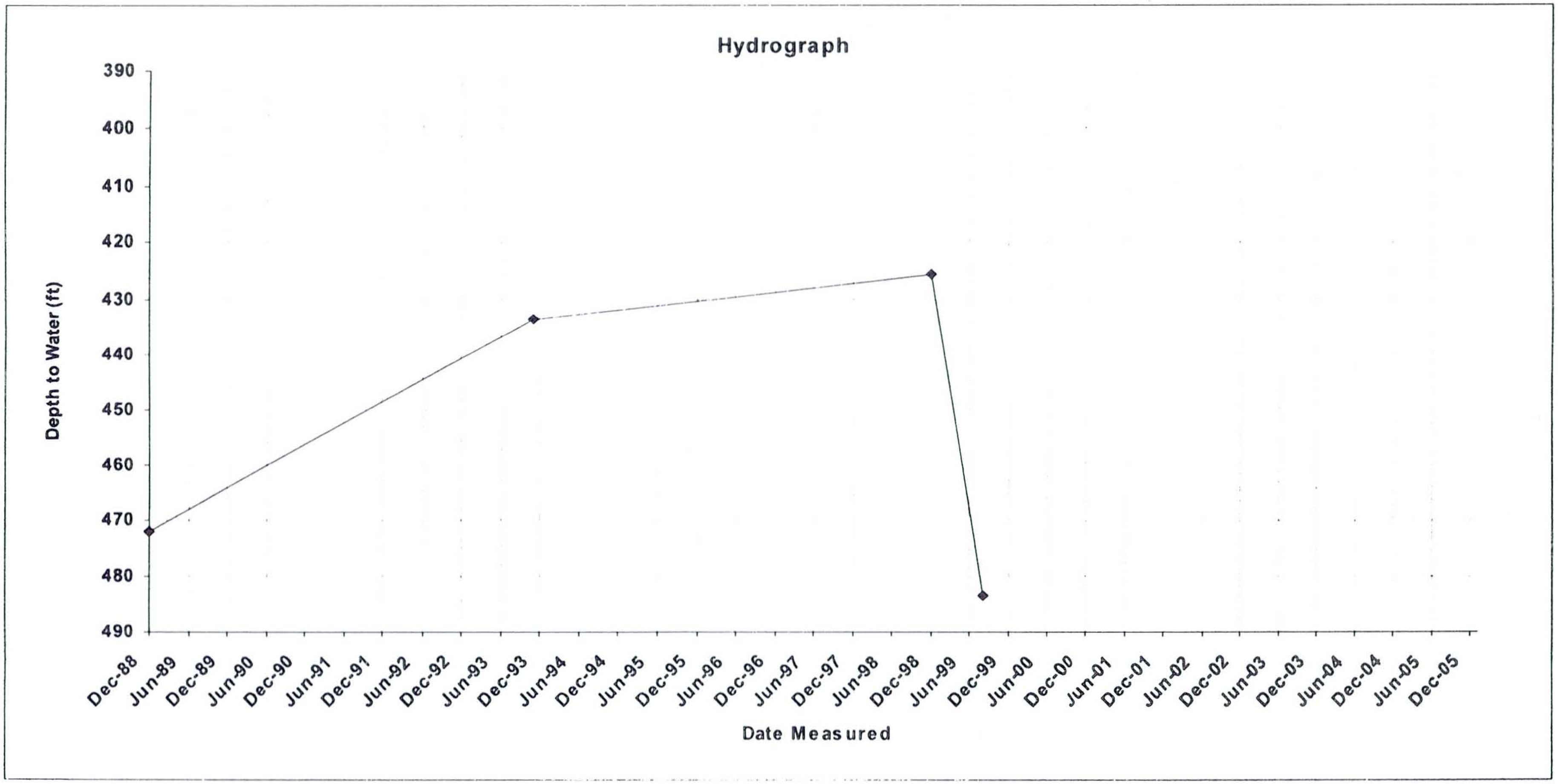


GWSI is ADWR's technical database of well locations, construction data, and water levels.

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-02-09 11ACC	333148113090301	611130	33° 31′ 48″	113° 9′ 3″	UNUSED				8/26/1999	483.50	722	4



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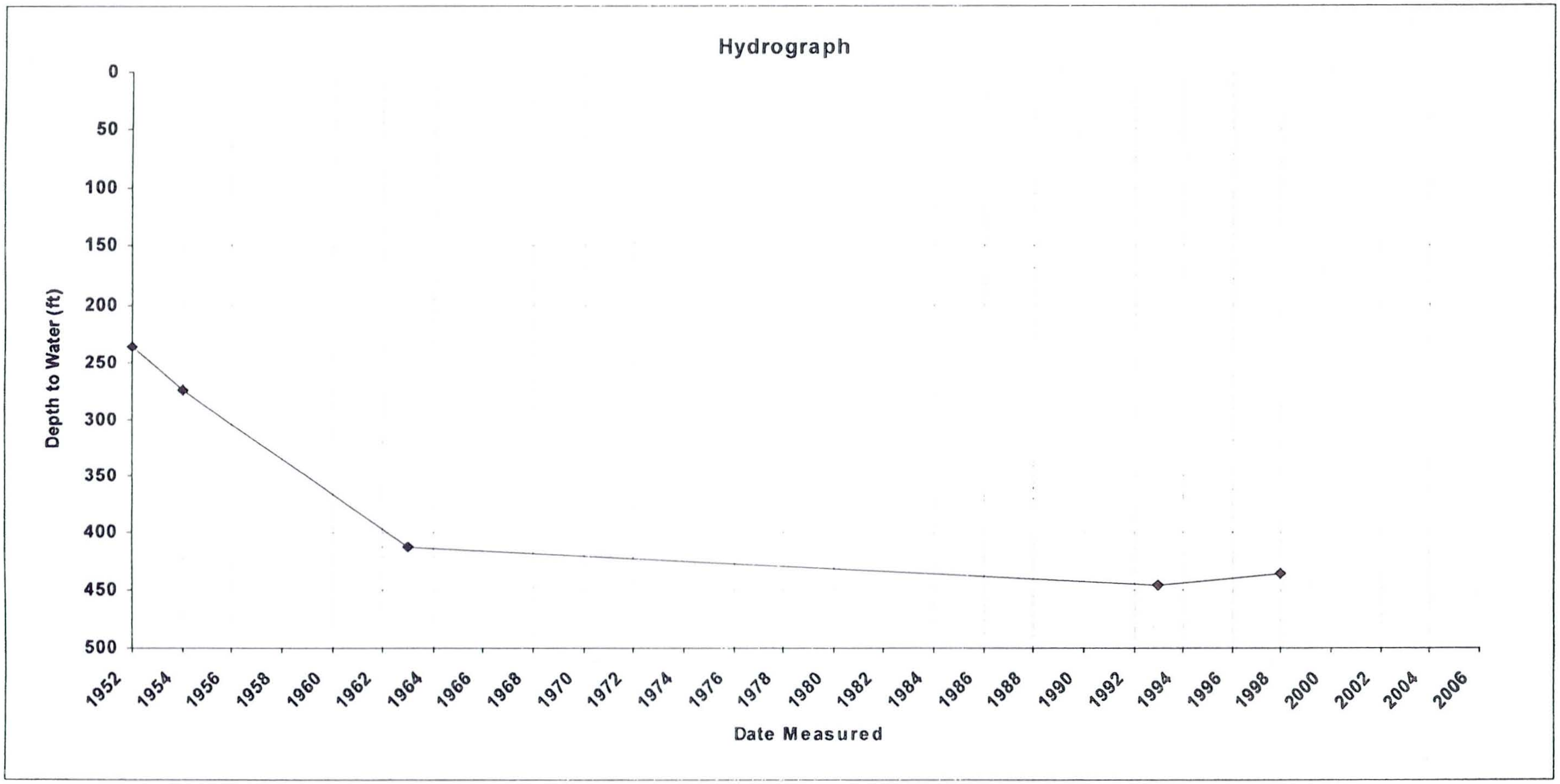
GWSI Well Report and Hydrograph

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-02-09 11BBB	333214113094101	611126	33° 32' 14''	113° 9' 41''	IRRIGATION	1500	1/1/1952	20	11/30/1998	435.20	785	5



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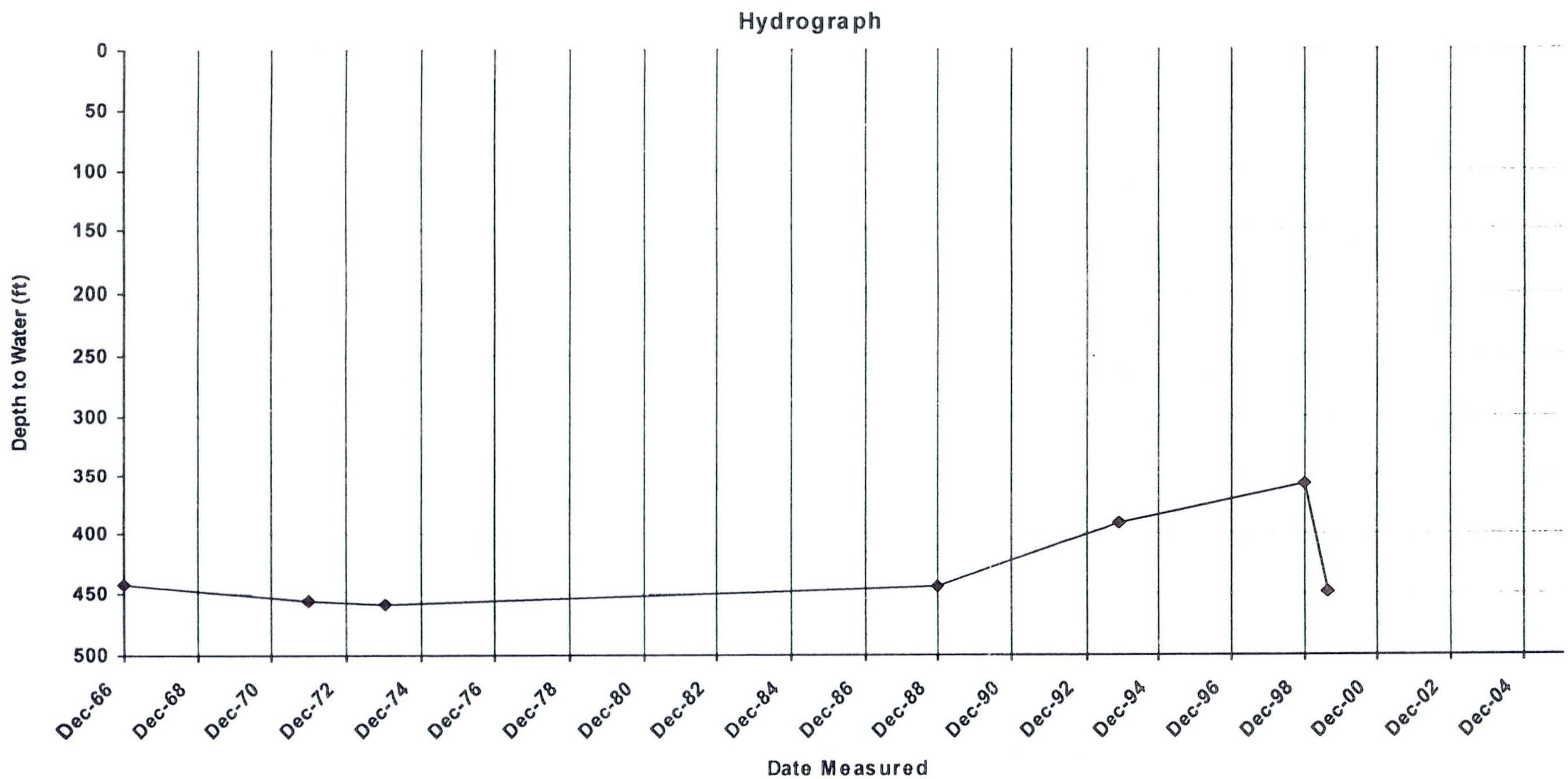
GWSI Well Report and Hydrograph

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-02-09 11CBB	333147113093901	611128	33° 31' 47''	113° 9' 39''	IRRIGATION	1505	5/20/1960	20	8/26/1999	448.60	757	7



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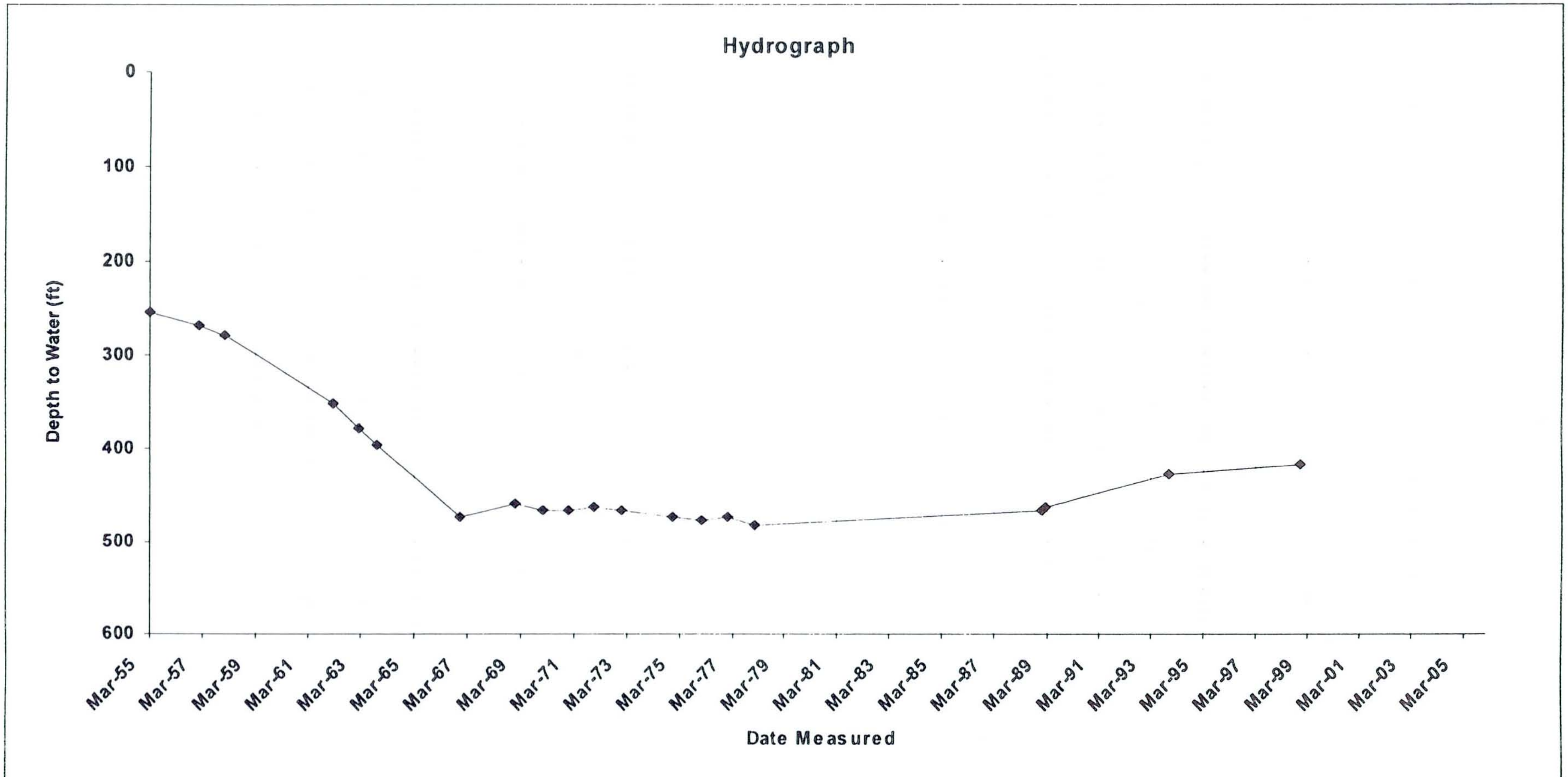
GWSI Well Report and Hydrograph

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-02-09 13BAA	333121113080901	630292	33° 31′ 21″	113° 8′ 9″	UNUSED	603	1/30/1954	18	12/2/1998	416.80	780	20

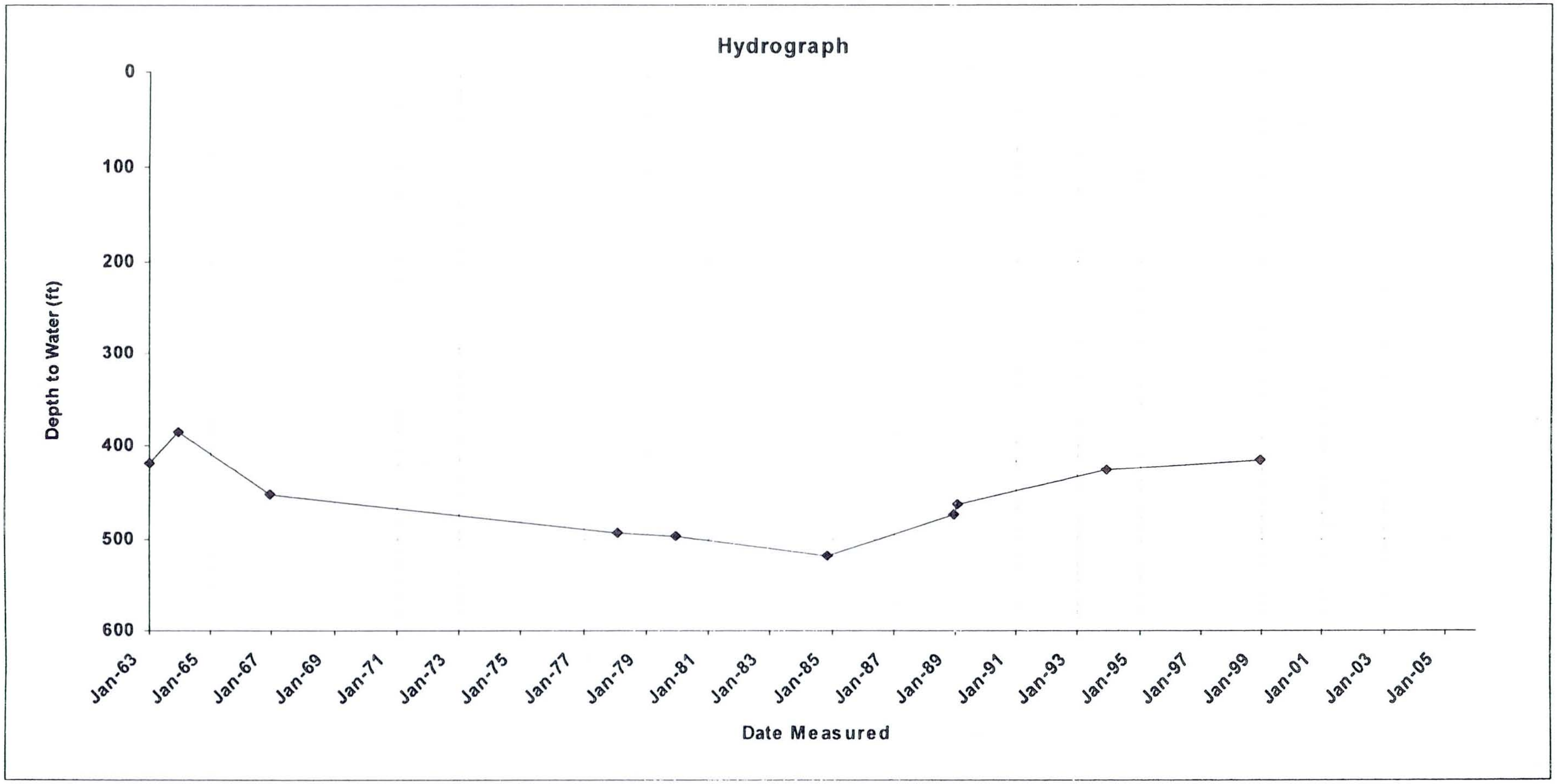


GWSI is ADWR's technical database of well locations, construction data, and water levels.

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-02-09 16BBB	333119113114401	629624	33° 31' 19"	113° 11' 44"	UNUSED	1400		20	12/23/1998	414.80	800.2	10



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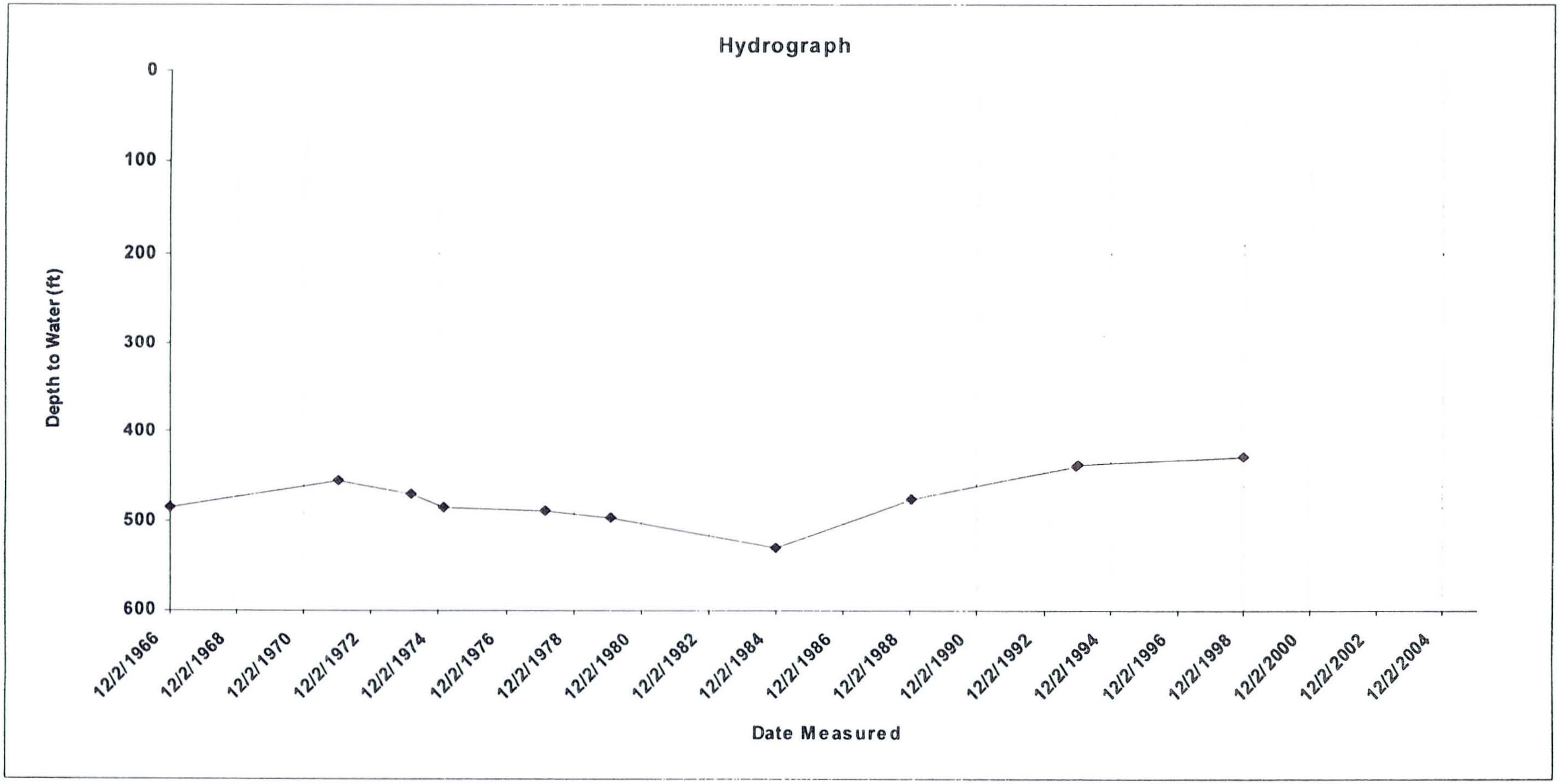
GWSI Well Report and Hydrograph

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-02-09 09DBB	333148113111801	611127	33° 31' 48''	113° 11' 18''	IRRIGATION	1510	5/31/1955	20	12/1/1998	428.90	790	11



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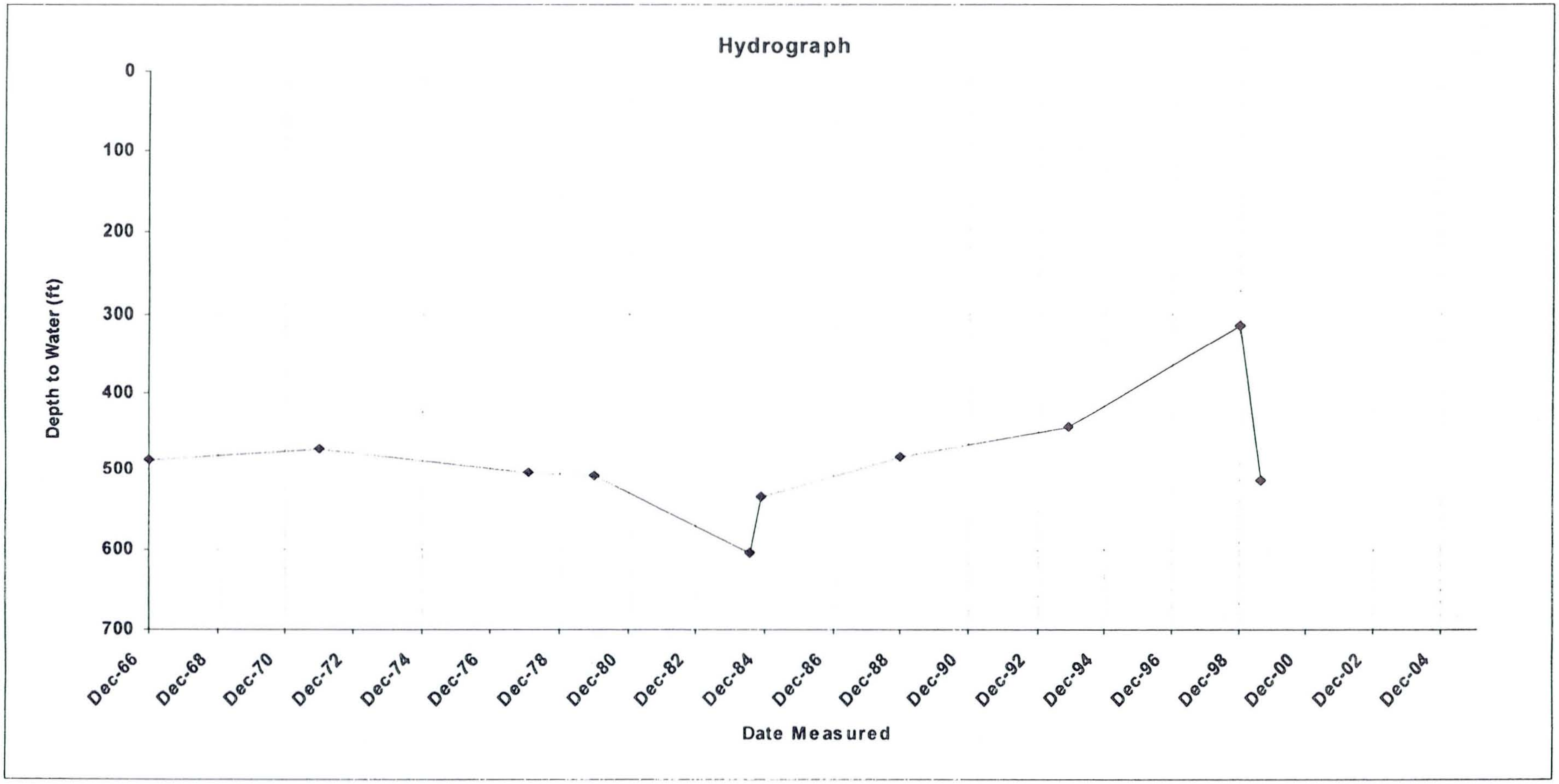
GWSI Well Report and Hydrograph

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-02-09 09ABB	333213113111801	611123	33° 32' 13''	113° 11' 18''	IRRIGATION	1540	7/25/1952	20	8/27/1999	513.00	720	10



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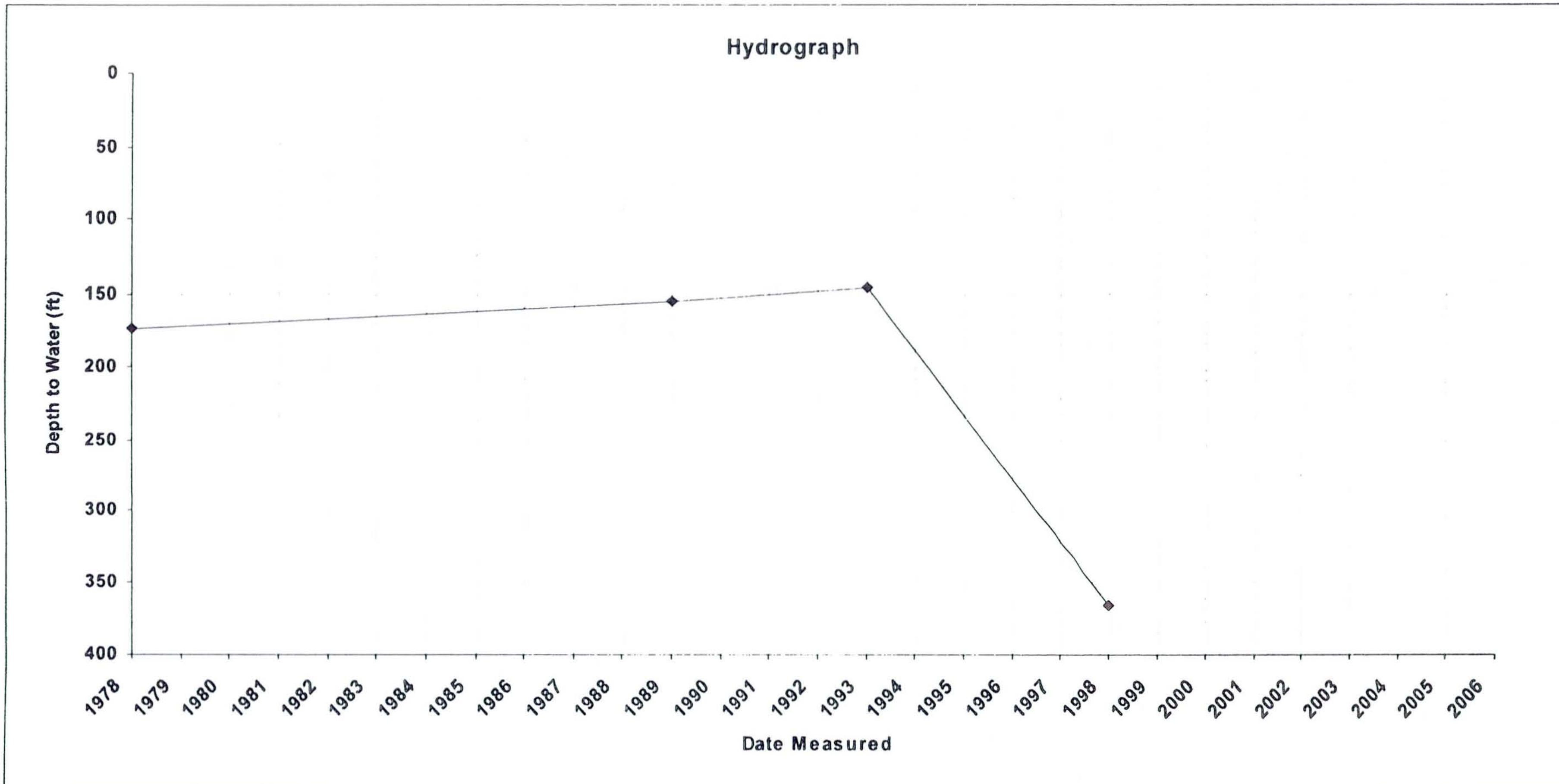
GWSI Well Report and Hydrograph

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-02-09 14BBB	333123113094001	611134	33° 31′ 23″	113° 9′ 40″	DOMESTIC	1530	9/10/1951	20	12/1/1998	366.90	825.1	4



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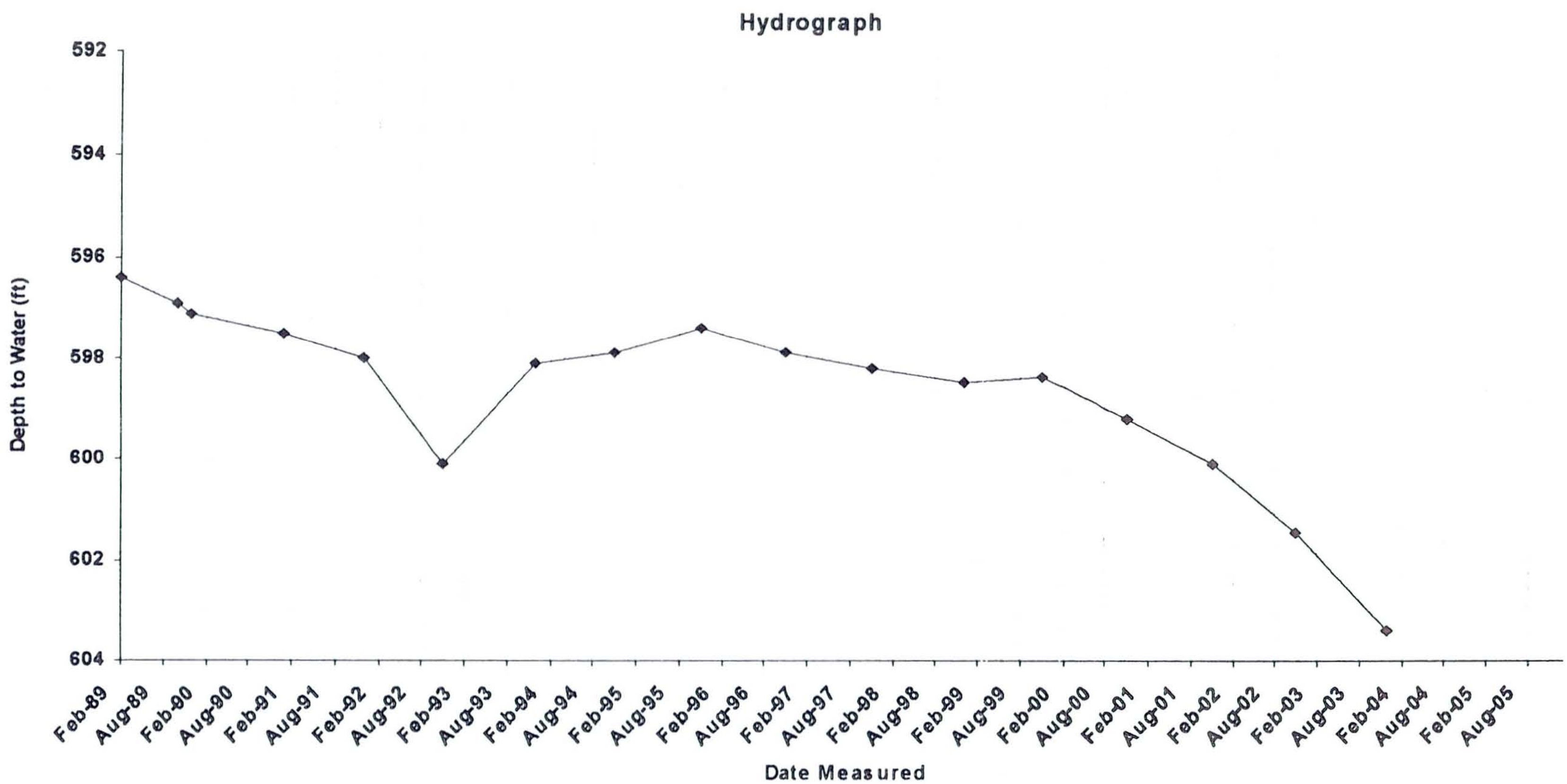
GWSI Well Report and Hydrograph

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-03-09 08DDD2	333637113114701	520439	33° 36' 35.9"	113° 11' 45.7"	UNUSED	765	4/11/1988	5	12/29/2003	603.40	897.6	17



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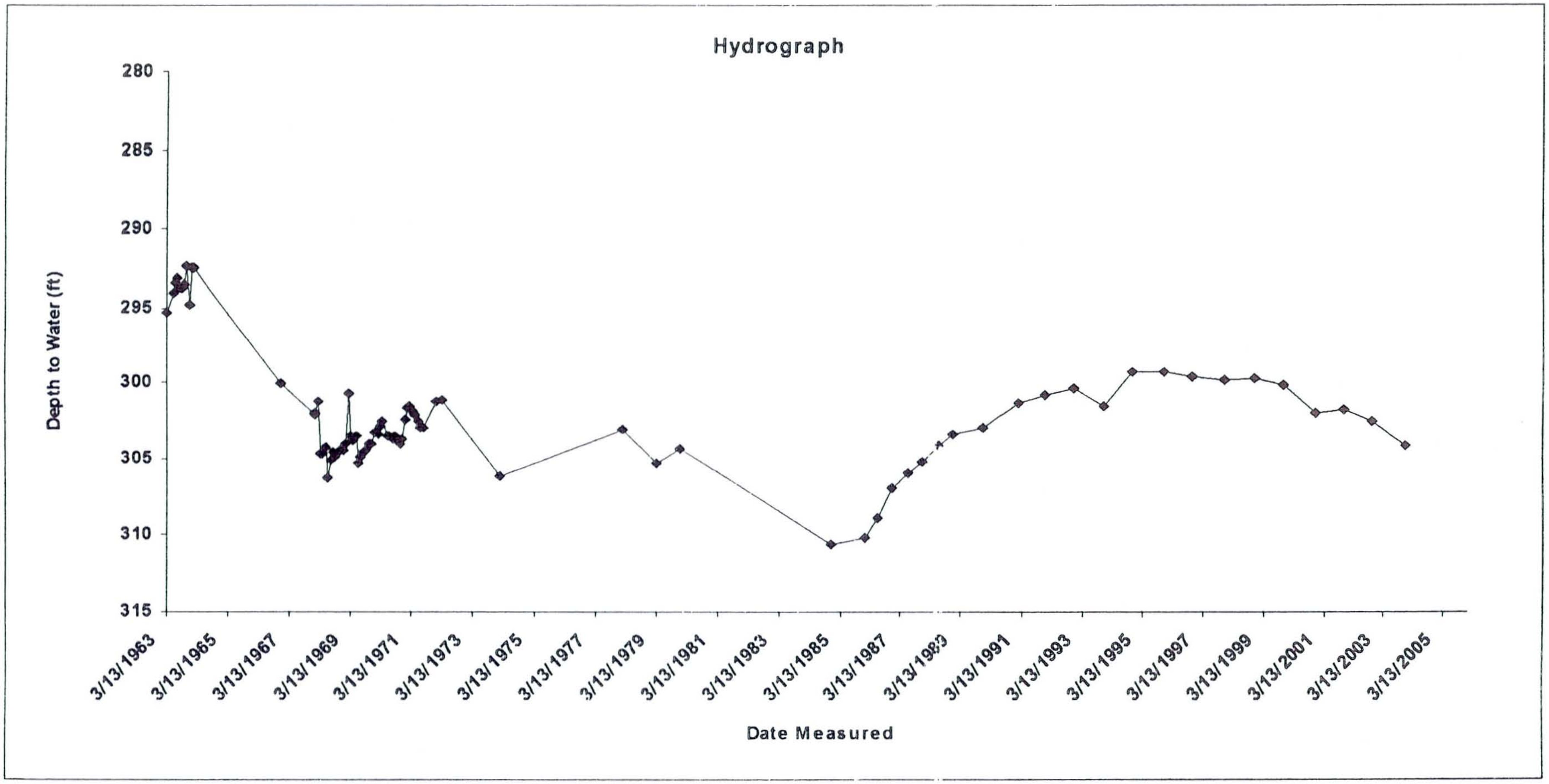
GWSI Well Report and Hydrograph — *AZ Dept of Water Resources*

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-02-09 07ABB	333212113131801	627169	33° 32' 12''	113° 13' 18''	UNUSED	1692	9/28/1952	20	12/15/2003	304.20	955.8	83



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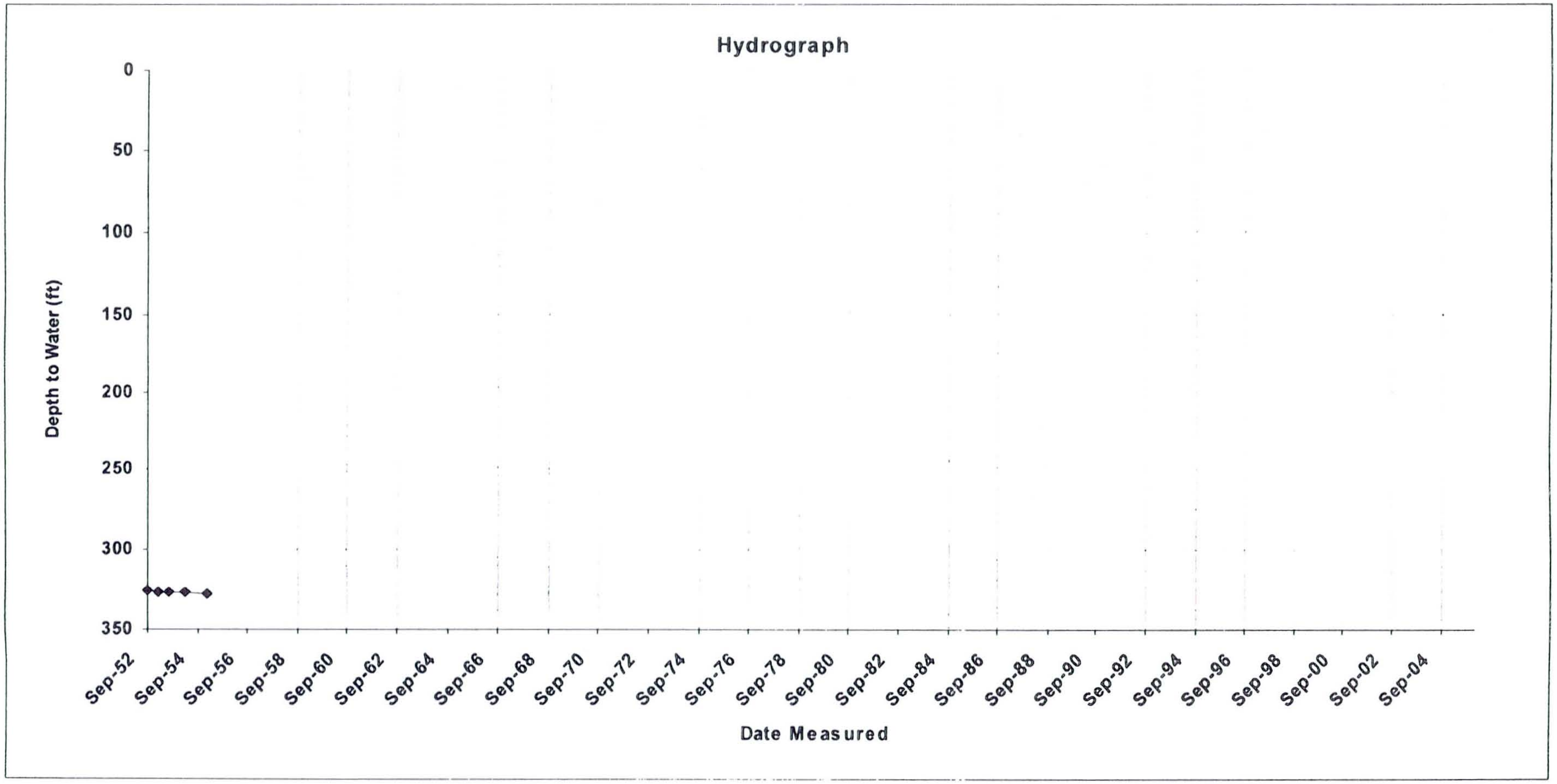
GWSI Well Report and Hydrograph

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-03-09 31AAD	333349113125101		33° 33′ 49′′	113° 12′ 51′′	UNUSED	336	1/1/1935	48	12/1/1966			6



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

GEOTECHNICAL MEMORANDUM

HARQUAHALA FLOOD RETARDING STRUCTURE
PHASE I STRUCTURES ASSESSMENT

Submitted to:

Kimley-Horn and Associates, Inc.

7878 N. 16th Street
Phoenix AZ 85020

Submitted by:

Gannett Fleming

4722 N. 24th Street Suite 250
Phoenix, AZ 85016-4852

Registered Professional Engineer (Civil)
CERTIFICATE NO.
35854
DEAN B.
DURKEE
Date Signed 11-3-05
ARIZONA, U.S.A.

for Frances Ackerman, R.G., E.I.T., P.E.
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, 1967)
- Supplemental Watershed Workplan, Harquahala Valley Watershed (Flood Control District of Maricopa County, 1977)
- Harquahala FRS and Floodway Geological Investigation Summary and Test Results Report (1978)
- Supplemental Geologic Investigation of Emergency Spillway Site (Pedone, 1980)
- Final Design Report and Design Report and Design Calculations, Harquahala Valley WPP, Arizona Harquahala FRS (Soil Conservation Service (SCS), 1980)
- Harquahala Embankment Design (SCS, February 22, 1980)
- Supplement 1 to Final Design Report Dated August 8, 1980 (SCS, 1980)
- Design Review Report, Harquahala Dam and Floodway (U.S. Department of Agriculture Soil Conservation Service, 1979), Supplement No. 1 to Design Review Report, Dated June 8. 1979 (U.S. Department of Agriculture Soil Conservation Service, 1979), Supplement 2, Preliminary Design Report, Harquahala FRS and Floodway, Dated November 6, 1979 (U.S. Department of Agriculture Soil Conservation Service, 1979)
- Harquahala Floodwater Retarding Structure as-built plan set
- Dam Construction Inspection records and portions of CAP Construction Progress Report (United States Department of the Interior, Bureau of Reclamation Lower Colorado Region, 1982)
- Annual dam inspection checklists
- Supplemental package containing permeability testing information
- 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 Harquahala 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 uppers 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.

1.2 Foundation Conditions

The foundation materials beneath the Harquahala FRS were differentiated in the Geologic Investigation Summary Report (SCS, 1978) into two reaches on the basis of a distinct change in subsurface conditions. The east end of the structure, designated as Reach 1, is underlain by coarse-grained gravels while the west end, or Reach 2, is underlain by finer-grained sands and silts. The change from coarse- to fine-grained materials occurs between Stations 722+00 and 712+00. Reach 1 extends from Station 717+00 to Station 1054+20, and Reach 2 extends from Station 431+00 to Station 717+00. This distribution of materials, with coarse-grained material along the eastern portion of the alignment, closer to the mountain front, and finer-grained material with increasing distance to the west, is typical of alluvial fan deposits. It reflects the influence of the dam alignment relative to the Big Horn Mountains to the east.

1.2.1 Reach 1

Sandy silty gravels to silty sandy gravels (GM to GP-GM) predominate throughout Reach 1. These coarse sediments are typical of upper alluvial fan deposits and reflect the proximity of Reach 1 to the Big Horn Mountains. Although caliche was not found to be significant in most of the borings completed during the geologic investigation for the Harquahala FRS, it is fairly common in this environment and may be locally extensive, though it is not uniformly widespread (SCS, 1978).

The dam alignment crosses the upstream tip of a small rhyolite knoll at Station 938+00. According to the Supplemental Geologic Investigation Report of Emergency Spillway Site (Pedone, 1980), the emergency spillway location was changed from its original location to the present location in order to use the rhyolite bedrock as the spillway foundation.

1.2.2 Reach 2

Reach 2 consists of a heterogeneous mixture of fine-grained materials derived from alluvial deposition, mudflows and floodplain splays. The subsurface is predominated by silty sand (SM), however thin, stratified layers of silty to clayey sands, sandy to clayey silts, silty to sandy clays, and gravelly sands to silty gravelly sands were observed during the geologic investigation. It was noted by SCS (1978) that there was no widespread consistency or uniformity either between test pits (or drill holes) or vertically in individual test pits. It was also reported that buried channel sand deposits may potentially be found all along Reach 2 in the top 15 to 20 feet.

During design, it was recognized that the soils in Reach 2 had high collapse potential. In Supplement 1 to the Final Design Report (SCS, 1980b), laboratory collapse testing results were reported for twenty-seven soil samples from Reach 2. Percent collapse for these samples ranged between 0.7% and 17.3%. Removal and recompaction of the collapse-prone silts, silty sands and low plasticity clays in the upper five to ten feet was required by the designers. Sheet No. 16 in the as-built drawing set indicates that stripping to various depths up to 9 feet was performed prior to construction of the embankment.

1.3 Embankment

The design of the embankment explicitly accounted for the differences in foundation conditions in Reach 1 and Reach 2. The Harquahala FRS was constructed as a homogeneous embankment (Zone I), with 3H:1V upstream and 2H:1V downstream slopes, and a cutoff trench. An inclined filter/drain (Zone II) was constructed in Reach 2 (Station 450+00 to Station 717+00) to mitigate the potential effects of transverse cracking in the fine-grained embankment materials in this reach (SCS, 1979). The design included toe drains at two locations where the drilling logs indicated channels were present at depths greater than the cutoff trench (SCS, 1980b). The as-built plans indicate that toe drains were installed between Station 790+00 and Station 796+00 and between Station 1040+20 and Station 1044+80. Seepage from the toe drains is collected in a series of 8-inch laterals which feed into 10-inch collector pipes that ultimately divert the seepage into the CAP canal. In Reach 1, the embankment design relied on the coarse-grained embankment material to resist cracking which by its nature is not prone to piping (SCS, 1980b).

The foundation was stripped to depths of up to 9 feet in Reach 2 and the cutoff trench ranged from 5 feet to 8 feet in depth in Reach 2 and from 5 feet to 23.5 feet in depth in Reach 1 (Sheet No. 16 in the as-built drawing set). To prevent uplift at the downstream toe, a five-foot thick natural blanket was placed at locations where clean sand or gravel channels were present. Typical embankment cross-sections for Reach 1 and Reach 2 are shown as Figures 1 and 2, respectively.

1.3.1 Embankment Materials

The embankment (Zones I and III) and filter materials (Zone II) have the characteristics summarized on Table 1, based on the final design report (SCS 1980b) and the design specifications (SCS, 1980c). Design specifications for the drain fill were not included in the material reviewed for this Phase I Structures Assessment. In addition, no information on specific borrow areas was included in the Final Design Report.

Table 1. Embankment Material Zones – Harquahala FRS

Zone	Description	USCS	Properties	
I	Embankment earth fill, upstream of drain fill – clay, silty and sandy clay, sandy silt, silty sand, silt	CL, CL-ML, SC-CL, SM-ML, ML, SM, ML-CL		
II	Filter	SC, SP-SC, SP-SM, SM-SC, GM, SM	Sieve	% Passing
			1½-in	100
			1-inch	80-100
			¾ - inch	70-90
			⅜ - inch	50-75
			No. 4	35-65
			No. 8	25-60
			No. 30	10-40
			No. 200	0-6.5
III	Embankment earth fill, downstream from drain fill – sand, silty sand, silty and sandy gravel, gravel	SW-SM, GM-GC, SP, SW, GP-GM, GM, SM, GW-GM, GM		

The Geologic Investigation Summary (SCS, 1978) included laboratory test data for soil samples collected from a total of 10 soil borings and 19 test pits. The following tests were performed on representative samples:

- 58 sieve analyses
- 25 field density tests
- 15 consolidation tests
- 6 Atterberg limit tests
- 6 permeability tests
- 5 direct shear tests
- 4 standard Proctor compaction tests
    - 1 triaxial compression test

A summary table of gradation data was included in the Harquahala FRS Embankment Design (SCS 1980a). These data were used as the basis for embankment design and filter gradation calculations. The strength test data (direct shear and triaxial testing) are summarized in Table 2. Sample TP-85 was collected from Reach 1 and the remaining samples for which strength testing was conducted were collected from Reach 2.

Table 2. Summary of Representative Laboratory Test Results from Geologic Investigation

Sample ID and Depth (feet)	Station Location	USCS	% Fines (-#200)	PI (%)	γ_d (pcf)	φ (^{0})	c (ksf)
TP-75 @6.0-6.7	573+00	CL	67	11	100.0	58.5	0.3
TP-77 @4.5-5.0	603+00	SM-ML	54	5			
TP-77 @11.5-12.0	603+00	ML	64	3		40.5	0.14
TP-82 @5.5	663+00	SC	48	9		30	0.15
TP-85 @5.5	861+50					57	0.10

1.3.2 Embankment Construction

Difficulties during construction of the filter/drain in Reach 2 resulted in a variable width filter/drain zone that was at times uneven and less than the specified 2 feet wide drain zone, according to construction inspection reports. The Dam Construction Inspection Reports indicated that the contractor had difficulty with filter/drain material placement in Reach 2 and that the filter/drain materials were contaminated with embankment materials. The contractor was not required to perform repair work at the time of construction, suggesting that the contamination was not significant enough to affect the functionality of the filter/drain.

In Reach 1, the dam was constructed with soil containing less than the specified amount of fine-grained soil (> 15% passing the No. 200 sieve) in the upstream section (between Stations 717+00 and 1054+00). The borrow materials available for embankment construction generally contained fewer fines than the specified 15%. According to the CAP Construction Progress Report (USBR, 1982), 50 out of 58 soils tests conducted during embankment construction (at 50% to 60% of construction completed) contained less than 15% fines and 32 of the samples testing contained less than 10% fines. USBR concluded that due to the limited quantity of fines in Reach 1, it was virtually impossible to "blanket" the upstream portion of the dam with materials having the specified fines content. Although it is unclear exactly what was meant by "blanket" it is evident that there are less than 15% fines at some locations on Reach 1.

The concern regarding the lack of fines on the upstream slope was the impact this would have on slope stability. To assess this, permeability testing was performed on soil samples. Permeability test results were used to calculate the time required to fully develop a phreatic line in the dam for comparison to expected impoundment time. The results were verified by in-field constant head permeability tests at three locations in Reach 1 and it was concluded that the time required to saturate the embankment would be significantly longer than the expected impoundment time, therefore embankment instability due to the lack of fine-grained soil in the upstream section should not be a concern (SCS, 1985).

1.3.3 Compatibility of Zone II Drain Fill as Filter for Zone I

An inclined filter/drain system (Zone II) was installed in Reach 2, primarily to protect against potential internal erosion and piping of embankment materials in the event of transverse crack development (SCS, 1979). The top of the filter/drain is 7 feet below the crest of the embankment at the centerline. The filter/drain is shown on the as-built plans as a 2-foot wide filter upstream from a 3-foot wide drain zone.

The gradation of the filter material (Zone II) 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) to verify its' ability to filter Zone I material. Figure 3 shows what is believed to be representative gradation curves for the finer materials used in the Zone I "Base Soil" (graphed with solid symbols). These gradation curves were developed using data from the Geological Investigation Summary (SCS, 1978). Reach 2 soil samples TP-75 @ 6.0-6.7' and TP-82 @ 5.5' were selected to represent Base Soil conditions in the Reach 2 embankment.

Soil sample TP-75 @ 6.0-6.7' is a clay having the United Soil Classification System (USCS) of CL; TP-82 @ 5.5' is a clayey sand with the USCS classification of SC. Additional soil property data are presented in Table 2.

The base soil gradation curves (solid symbols) were adjusted for gravel content, per NRCS guidelines (NRCS, 1994). The adjusted gradation curves are shown on Figure 3 with open symbols. The NRCS filtering and permeability criteria for the adjusted curves are shown by the solid circles on the 15% passing line. These criteria were used as the basis for developing the NRCS filter band shown on Figure 3. Also shown on Figure 3 is the original design specification filter band and the gradation of three samples from the as-built filter (USBR, 1982).

As can be seen in Figure 3, the original design specification band falls within the NRCS permeability (minimum D_{15}) criteria. The original design specification band is slightly coarser than the NRCS filtering (maximum D_{15}) criteria and may not achieve the recommended filtering limit for the finest base soils. However it appears that at least a portion of the as-built filter (represented by the red curves on Figure 3) was placed within the NRCS D_{15} range criteria. The as-built gradations and original specification band indicate that the filter is not ideal with respect to uniformity; that is, the filter is too broadly graded. Modern NRCS criteria (green band) are intended to result in narrowly-graded filters. The primary purpose of these criteria is to prevent segregation of the filter during placement. In addition, because the coarse side of the original specification band has an adverse (convex) shape that could indicate the filter gradation is internally unstable, Gannett Fleming checked the internal stability of the as-built gradations (red curves) using a procedure outlined by Kenney and Lau (1985). This procedure indicates that the filter is potentially internally unstable and could lose the fine fraction of the filter material during seepage flows.

Because the as-built filter may have segregated during placement due to the overly broad gradation, or may not meet filtering criteria on the D_{15} size, it is possible that some fines from Zone I could penetrate into Zone II under a concentrated leak through a transverse crack. In addition, the potential internal instability of the filter could result in fines being washed out of the filter during impoundment events. 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.

1.3.4 Embankment Settlement

The SCS designers recognized the potential for collapsible soils and associated settlement in Reach 2 and performed consolidation testing to evaluate collapse potential during the preliminary design phase. Consolidation testing was conducted under a 2,000 pounds per square foot (psf) load. Review of the preliminary design (SCS, 1979) indicated that collapse potential should be evaluated for actual loads, which were expected to exceed 2,000 psf. Additional consolidation tests of twenty-seven samples from Reach 2 were conducted under 2 tons per square foot (tsf) load. Reported collapse potential for these additional tests ranged from 0.6% to 17.3% (SCS, 1980c). The crest elevation in Reach 2 was designed to be one and a half feet higher than in Reach 1 to allow for foundation settlement after construction.

1.3 Original Slope Stability Analysis

Table 3 summarizes the parameter values used by designers for embankment slope stability analysis (SCS, 1980b). These parameter values were based on a summary graph of soil strength data presented in the Final Design Report (SCS, 1980b). This graph was based on soil data that were not available for review during this Phase I Structures Assessment, therefore, the soil strength values summarized in Table 3 that the designers used for stability analyses differ from strength values summarized in Table 2. Slope stability analysis results were reported for the loading conditions shown on Table 4 (SCS, 1980b). No documentation of seismic slope stability was found in the materials reviewed. Stability analyses for Reach 1 were performed using the computer code Univac 1100 Series ECES and stability analyses for Reach 2 were calculated by hand.

Table 3. Embankment Soil Properties Used in Stability Analysis

Property	Reach 1 Zones I and III	Reach 2 Zones I and III
Dry unit weight (γ_d) (pcf)	112.6 pcf	122.6 pcf
Moist unit weight ($\square_m$)	125.6 pcf	127.7 pcf
Saturated unit weight (γ_{sat}) (pcf)	133.1 pcf	130.3 pcf
Angle of internal friction (φ)	23°	31.5^{0}
Cohesion (c)	0.75 ksf	0 ksf
Effective angle of internal friction (φ')	39^{0}	
Effective cohesion (c)	0.1 ksf	

The designers assessed the slope stability for end of construction, steady state seepage, and rapid drawdown loading conditions. In addition, the steady state seepage under seismic loading was evaluated for Reach 2.

Table 4. Original Slope Stability Analyses Results

Slope	Conditions	Minimum F.S.	
		Reach 1	Reach 2
2H:1V downstream	End of construction	2.0	1.2
2H:1V downstream	Steady-state seepage	2.0	1.2*
3H:1V upstream	Rapid drawdown	2.7	1.8
2H:1V downstream	Steady-state seepage under seismic load		1.1

*Factor of safety is for "infinite slope" failure mode. More critical, deep-seated failure modes will have a higher factor of safety.

In Reach 1, downstream slope stability was assessed with the assumption that the phreatic line emerges on the downslope face, while in Reach 2, downstream slope stability was assessed for dry slope conditions (assuming the drain intercepts seepage). The low factor of safety for the Reach 2 dry slope condition, is due to the assumption of zero cohesion (c=0) and an infinite slope (shallow slope raveling) failure mode. Gannett Fleming anticipates that analyses for more significant, deep-seated failure surfaces would show significantly higher factors of safety for the same strength assumptions.

Higher factors of safety were reported for Reach 1 compared to Reach 2 under all loading conditions evaluated. This is likely the result of the different assumptions used for shear strength, and the different methods used to assess slope stability (computer code versus hand calculations). The reported factors of safety (SCS, 1980b) were acceptable for all loading conditions, except the downstream, steady seepage and end-of-construction conditions for Reach 2. However, as explained previously, the low factor or safety for steady seepage is reported for an infinite slope failure assumption, and does not reflect the factor of safety for significant slope instability that would result in loss of freeboard or failure of the embankment. Also, the end-of-construction factor of safety is now a moot point, since the embankments have been stable over their lifetime, and any excess pore pressures caused by construction loading should be completely dissipated.

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 in Reach 2 was evaluated for this Phase I Structures Assessment and is discussed in Section 1.3.3. No information regarding the gradation of the drain materials in the filter/drain were 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 Harquahala 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 Harquahala 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.

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 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 w/o seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	1.5	1.5
Steady seepage w/ phreatic surface developed from critical partial pool elevation (upstream slope)	n/a	1.5
Steady seepage w/seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (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.

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)
	Reach 1	Reach 2	
End of Construction (downstream slope)	2.01	1.23	(1)
Rapid Drawdown (upstream slope)	2.71	1.84	(2)
Steady state seepage w/o seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	2.02	1.23	(3)
Steady state seepage w/ phreatic surface developed from critical partial pool elevation (upstream slope)	Not evaluated	Not evaluated	(4)
Steady state seepage w/seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	Not evaluated	1.1	(5)

(1) End of construction (downstream slope): The original factor of safety calculated for this loading condition in Reach 2 (1.23) did not achieve the minimum ADWR criteria of 1.3 (see Table 5). However, as discussed in Section 1.3, this loading condition is now irrelevant, since the construction-induced pore pressures have been dissipated, and the embankments have remained stable since their construction. Additional analyses for this loading condition are 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. However, the original analyses were conducted by assuming the full

development of a phreatic surface in the upstream slope. Analyses conducted during Phase I studies for other flood retarding structures in Maricopa County illustrate that a steady state phreatic surface may not develop in dry dams under multiple temporary impoundment events (Gannett Fleming, 2004a, 2004b, and 2004c). Therefore, additional analysis of upstream slope stability under rapid drawdown conditions is not considered necessary.

(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). However, as discussed in Section 1.3, this is due to the analysis approach (infinite slope assumptions), and does not reflect an appropriate factor of safety against a significant failure mode. Additional analyses of deeper, more significant failure surfaces are recommended, using reasonable shear strength assumptions for Reach 1 embankment soils, 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 is 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): A seismic stability analysis was only documented for Reach 2. To document seismic stability for Reach 1 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.

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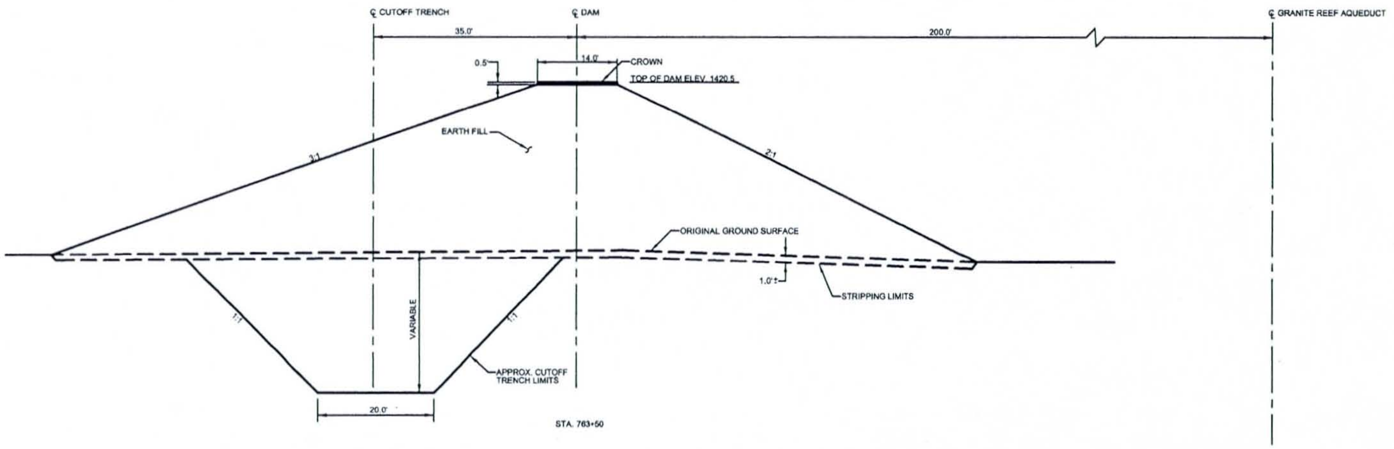

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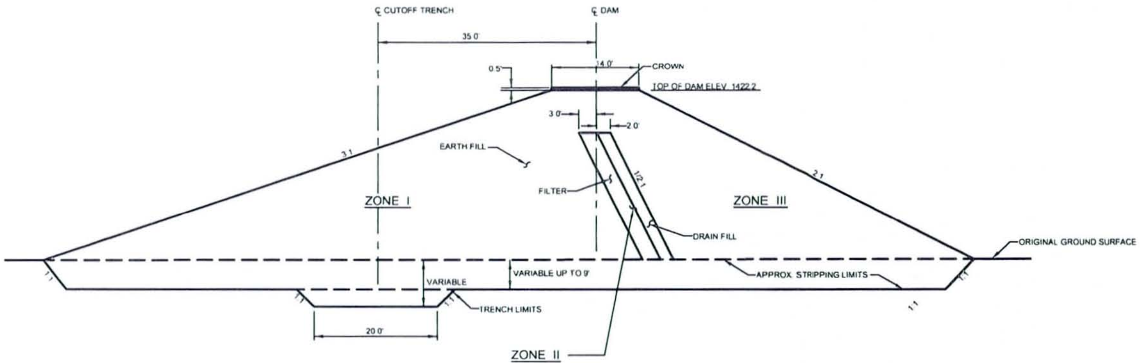
FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
DRAFT GEOTECHNICAL MEMORANDUM

FIGURE 1
TYPICAL EMBANKMENT SECTION - REACH 1
HARQUAHALA FLOOD
RETARDING STRUCTURE

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



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FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
DRAFT GEOTECHNICAL MEMORANDUM

FIGURE 2
TYPICAL EMBANKMENT SECTION - REACH 2
HARQUAHALA FLOOD
RETARDING STRUCTURE

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

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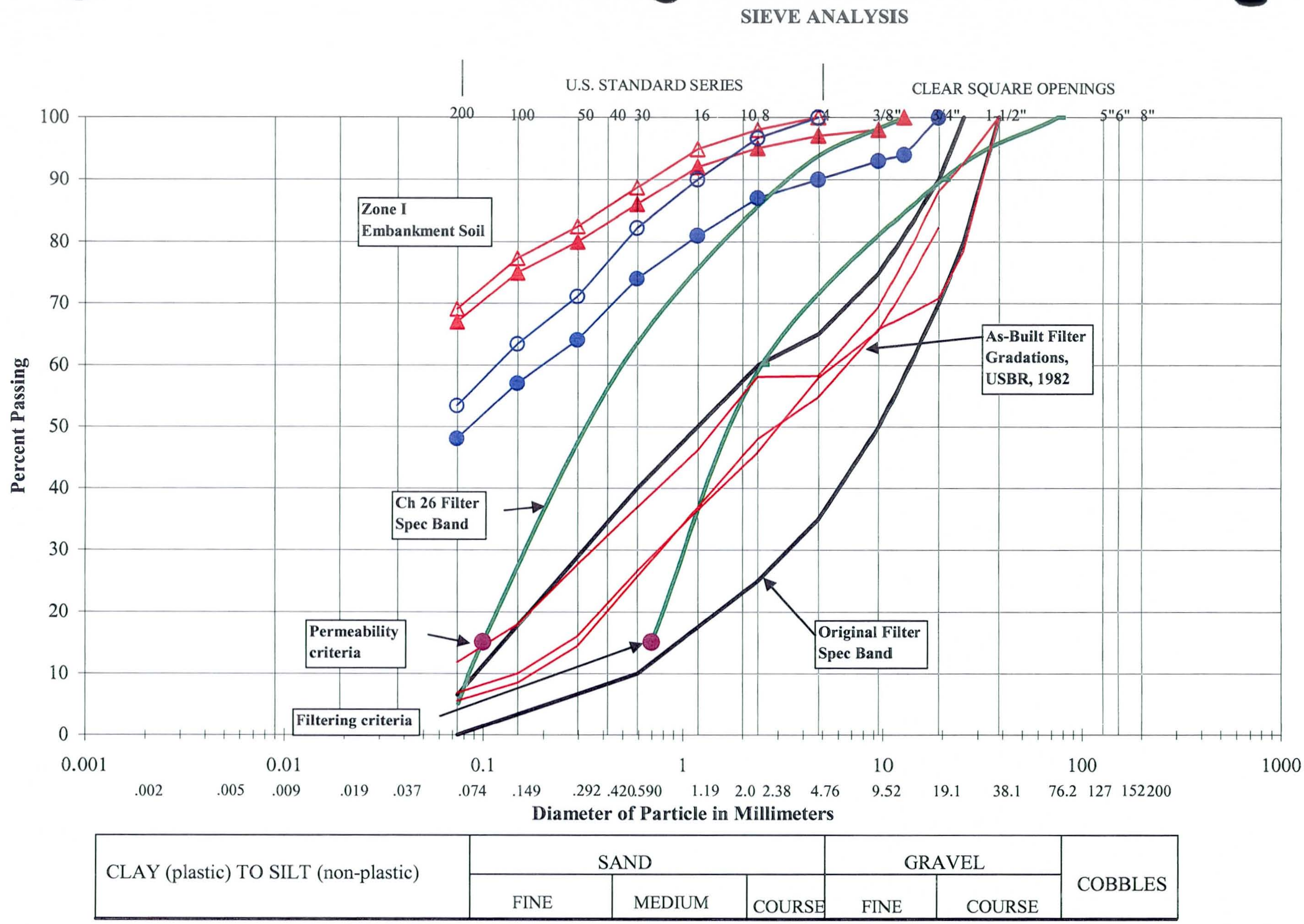

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FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
GEOTECHNICAL MEMORANDUM – HARQUAHALA FRS
FILTER CRITERIA

FIGURE 3

Harquahala Flood Retarding Structure

Subsidence Survey Data Review

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

11/14/2005

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 19, **"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.**

Station	Design Crest Elev	Adj Design Crest Elev	2003 Dam Crest Elev	2003 - Adj Dgn
439+78	1421.70	1423.579	1423.598	0.019
449+80	1422.18	1424.059	1424.392	0.333
460+60	1422.20	1424.079	1424.559	0.480
471+23	1422.20	1424.079	1424.662	0.583
480+20	1422.20	1424.079	1424.260	0.181
489+68	1422.20	1424.079	1424.858	0.779
500+01	1422.20	1424.079	1424.843	0.764
509+85	1422.20	1424.079	1424.807	0.728
517+23	1422.20	1424.079	1424.612	0.533
527+39	1422.20	1424.079	1424.262	0.183
537+92	1422.12	1423.999	1424.593	0.594
547+83	1422.10	1423.979	1424.631	0.652
557+87	1422.10	1423.979	1424.460	0.481
567+98	1422.10	1423.979	1424.301	0.322
578+22	1422.10	1423.979	1424.907	0.928
588+65	1422.10	1423.979	1424.691	0.712
598+17	1422.10	1423.979	1424.582	0.603
607+79	1422.10	1423.979	1424.925	0.946
617+41	1422.10	1423.979	1424.979	1.000
627+05	1422.10	1423.979	1425.007	1.028
637+32	1422.10	1423.979	1424.502	0.523
		(Fig. 1-1	Plot Data)	(Fig. 1-2)

Station	Design Crest Elev	Adj Design Crest Elev	2003 Dam Crest Elev	2003 - Adj Dgn
647+92	1422.10	1423.979	1424.763	0.784
658+10	1422.10	1423.979	1424.637	0.658
668+18	1422.10	1423.979	1424.657	0.678
678+36	1422.10	1423.979	1424.903	0.924
688+38	1422.10	1423.979	1424.736	0.757
698+14	1422.10	1423.979	1424.843	0.864
708+63	1422.10	1423.979	1425.080	1.101
718+45	1420.60	1422.479	1423.742	1.263
729+59	1420.60	1422.479	1422.733	0.254
737+32	1420.60	1422.479	1422.912	0.433
749+84	1420.53	1422.409	1423.025	0.616
759+18	1420.50	1422.379	1422.822	0.443
766+15	1420.50	1422.379	1422.827	0.448
776+99	1420.50	1422.379	1422.826	0.447
777+06	1420.50	1422.379	1423.384	1.005
786+45	1420.50	1422.379	1423.217	0.838
808+45	1420.50	1422.379	1423.242	0.863
818+27	1420.46	1422.339	1423.273	0.934
827+23	1420.40	1422.279	1423.237	0.958
837+87	1420.35	1422.229	1422.686	0.457
847+73	1420.30	1422.179	1422.959	0.780
		(Fig. 1-1	Plot Data)	(Fig. 1-2)

Station	Design Crest Elev	Adj Design Crest Elev	2003 Dam Crest Elev	2003 - Adj Dgn
857+54	1420.25	1422.129	1422.593	0.464
867+86	1420.20	1422.079	1422.773	0.694
877+92	1420.15	1422.029	1422.736	0.707
887+64	1420.10	1421.979	1422.720	0.741
897+33	1420.05	1421.929	1422.676	0.747
907+89	1420.00	1421.879	1422.868	0.989
917+96	1419.85	1421.729	1422.608	0.879
928+83	1419.70	1421.579	1422.581	1.002
930+19	1419.70	1421.579	1422.419	0.840
948+66	1419.70	1421.579	1422.242	0.663
958+35	1419.70	1421.579	1422.235	0.656
970+87	1419.70	1421.579	1422.123	0.544
979+47	1419.70	1421.579	1421.845	0.266
988+60	1419.70	1421.579	1422.366	0.787
999+17	1419.70	1421.579	1421.423	-0.156
1010+09	1419.70	1421.579	1422.101	0.522
1020+11	1419.70	1421.579	1422.330	0.751
1030+98	1419.70	1421.579	1422.044	0.465
1040+19	1419.70	1421.579	1421.791	0.212
1049+91	1419.70	1421.579	1421.974	0.395
		(Fig. 1-1	Plot Data)	(Fig. 1-2)

Table 1

STA 439+78 to STA1049+91 Dam Crest Elevations and Relative Changes in Elevation

Dam Crest Elevations

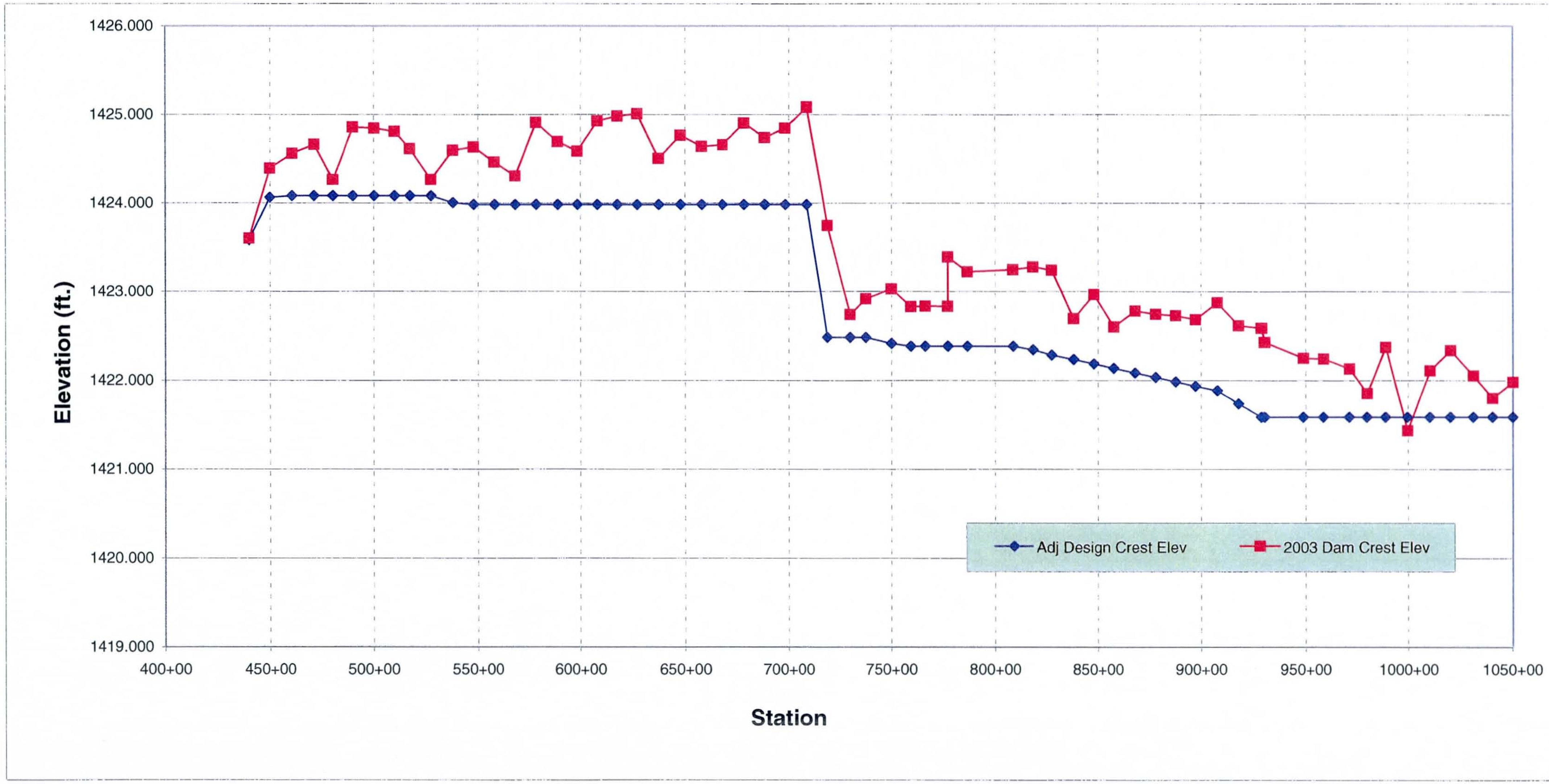


Figure 1-1
Dam Crest Elevation Comparison Chart

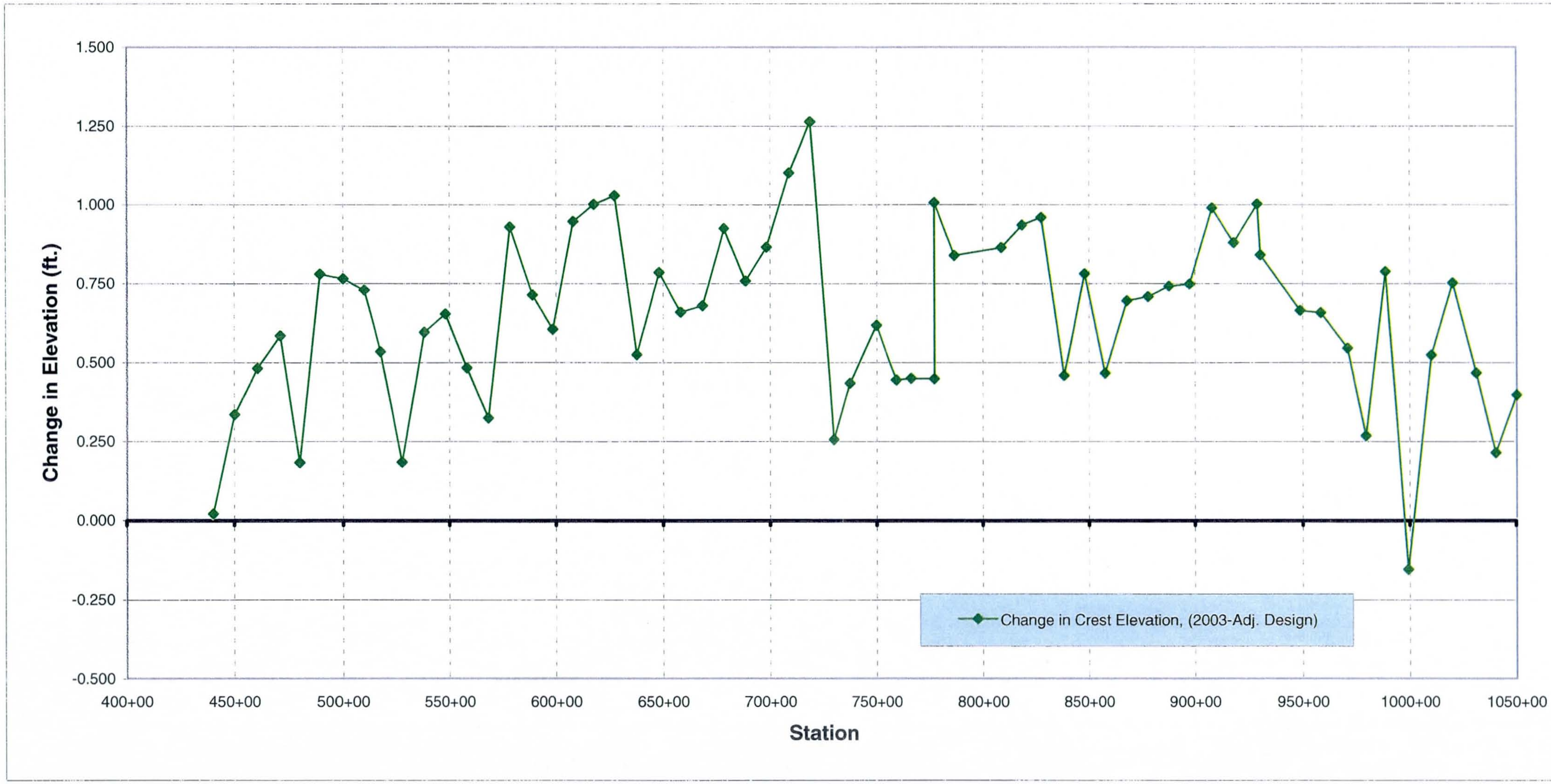


Figure 1-2
Change in Dam Crest Elevation Chart

Table 2 summarizes elevation data for Crest Settlement Monuments. Elevation survey data collected prior to year-2002 is based on NGVD 1929 or 1983 vertical datum, and have been adjusted to NAVD 1988 vertical datum. Adjustment details are outlined on page 19, "**Reference Marks.**"

Figure 2-1 compares Crest Settlement Monument Elevations for years 1984, 1986, 1991, 2002, and 2003. Elevations for surveys prior to year-2002 are adjusted as noted.

Note: See location of crest monuments on page 21, "**Floodplain View**'

		Crest Monument Survey Data							
Marker	**Station**	**1984**	**Adj 84**	**1986**	**Adj 86**	**1991**	**Adj 91**	**2002**	**2003**
A1	439+78	1421.645	1423.524	1421.223	1423.298	1421.373	1423.448	1423.314	1423.295
A2	449+80	1422.442	1424.321	1421.983	1424.058	1422.135	1424.210	1424.012	1424.028
A3	460+60	1423.005	1424.884	1422.529	1424.604	1422.704	1424.779	1424.528	1424.567
A4	471+23	1422.915	1424.794	1422.483	1424.558	1422.624	1424.699	1424.567	1424.479
A5	480+20	1422.791	1424.670	1422.340	1424.415	1422.476	1424.551	1424.281	1424.190
A6	490+20	1423.331	1425.210	1422.903	1424.978	1423.038	1425.113	1425.000	1424.821
A7	500+20	1422.600	1424.479	1422.169	1424.244	1422.317	1424.392	1424.249	1424.176
A8	510+00	1422.643	1424.522	1422.209	1424.284	1422.329	1424.404	1424.314	1424.279
A9	517+42	1422.969	1424.848	1422.482	1424.557	1422.584	1424.659	1424.603	1424.485
A10	527+58	1422.662	1424.541	1422.182	1424.257	1422.290	1424.365	1424.262	1424.304
A11	538+27	1422.479	1424.358	1422.011	1424.086	1422.170	1424.245	1424.131	1424.092
A12	547+93	1422.776	1424.655	1422.308	1424.383	1422.467	1424.542	1424.370	1424.348
A13	548+13	1422.949	1424.828	1422.368	1424.443	1422.493	1424.568	1424.491	1424.375
A14	567+97	1422.625	1424.504	1422.185	1424.260	1422.338	1424.413	1424.242	1424.173
A15	578+65	1422.905	1424.784	1422.446	1424.521	1422.621	1424.696	1424.557	1424.502
A16	589+28	1422.786	1424.665	1422.363	1424.438	1422.518	1424.593	1424.567	1424.397
A17	597+16	1422.655	1424.534	1422.256	1424.331	1422.388	1424.463	1424.308	1424.251
A18	607+37	1422.815	1424.694	1422.425	1424.500	1422.566	1424.641	1424.541	1424.429
A19	617+67	1422.899	1424.778	1422.526	1424.601	1422.664	1424.739	1424.570	1424.563
A20	627+50	1423.059	1424.938	1422.676	1424.751	1422.804	1424.879	1424.702	1424.711
A21	637+77	1422.566	1424.445	1422.222	1424.297	1422.331	1424.406	1424.229	1424.280
A22	647+92	1422.574	1424.453	1422.229	1424.304	1422.361	1424.436	1424.265	1424.259
A23	657+97	1422.603	1424.482	1422.286	1424.361	1422.386	1424.461	1424.334	1424.302
A24	668+08	1422.776	1424.655	1422.479	1424.554	1422.548	1424.623	1424.501	1424.423
A25	678+32	1422.619	1424.498	1422.332	1424.407	1422.422	1424.497	1424.436	1424.344
A26	688+40	1422.529	1424.408	1422.245	1424.320	1422.341	1424.416	1424.249	1424.232
A27	698+42	1422.646	1424.525	1422.358	1424.433	1422.484	1424.559	1424.478	1424.429
A28	708+44	1422.837	1424.716	1422.554	1424.629	1422.689	1424.764	1424.583	1424.651
A29	718+47	1421.573	1423.452	1421.295	1423.370	1421.434	1423.509	1423.448	1423.398
			(Fig. 2-1)		(Fig. 2-1)		(Fig. 2-1 Plot Data)		

*Monuments A20 and A21 were destroyed in 2002, so these are assumed values. Appears to have been reset by 2003.

Table 2

STA 439+78 to STA 718+47 Crest Settlement Monument Elevations

Marker	Station	Crest Monument Survey Data							
		1984	Adj 84	1986	Adj 86	1991	Adj 91	2002	2003
A30	729+58	1420.336	1422.215	1420.075	1422.150	1420.197	1422.272	1422.149	1422.200
A31	737+36	1420.149	1422.028	1419.875	1421.950	1420.000	1422.075	1422.024	1421.995
A32	749+52	1421.035	1422.914	1420.781	1422.856	1420.892	1422.967	1423.015	1422.927
A33	758+41	1420.887	1422.766	1420.630	1422.705	1420.732	1422.807	1422.700	1422.709
A34	766+14	1420.503	1422.382	1420.245	1422.320	1420.353	1422.428	1422.362	1422.356
A35	777+05	1421.141	1423.020	1420.872	1422.947	1420.989	1423.064	1422.972	1422.972
A36	786+49	1421.015	1422.894	1420.759	1422.834	1420.868	1422.943	1422.851	1422.872
A37	796+99	1420.519	1422.398	1420.259	1422.334	1420.373	1422.448	1422.405	1422.407
A38	808+46	1421.179	1423.058	1420.947	1423.022	1421.029	1423.104	1423.094	1423.091
A39	818+25	1420.828	1422.707	1420.649	1422.724	1420.714	1422.789	1422.759	1422.773
A40	827+22	1421.264	1423.143	1421.097	1423.172	1421.162	1423.237	1423.192	1423.190
A41	837+91	1420.266	1422.145	1420.119	1422.194	1420.164	1422.239	1422.247	1422.202
A42	847+76	1420.774	1422.653	1420.612	1422.687	1420.677	1422.752	1422.776	1422.709
A43	857+54	1420.509	1422.388	1420.348	1422.423	1420.408	1422.483	1422.536	1422.454
A44	867+87	1420.582	1422.461	1420.417	1422.492	1420.475	1422.550	1422.552	1422.495
A45	877+95	1420.462	1422.341	1420.318	1422.393	1420.360	1422.435	1422.467	1422.403
A46	887+65	1420.499	1422.378	1420.356	1422.431	1420.386	1422.461	1422.454	1422.438
A47	897+33	1420.311	1422.190	1420.163	1422.238	1420.187	1422.262	1422.238	1422.238
A48	907+89	1420.506	1422.385	1420.348	1422.423	1420.380	1422.455	1422.444	1422.440
A49	917+96	1420.076	1421.955	1419.950	1422.025	1419.967	1422.042	1422.018	1422.025
A50	928+83	1420.098	1421.977	1419.987	1422.062	1419.992	1422.067	1422.044	1422.053
A51	940+19	1419.796	1421.675	1419.680	1421.755	1419.697	1421.772	1421.801	1421.746
A52	948+59	1419.502	1421.381	1419.378	1421.453	1419.394	1421.469	1421.430	1421.463
A53	958+35	1419.593	1421.472	1419.483	1421.558	1419.509	1421.584	1421.572	1421.557
A54	970+80	1419.943	1421.822	1419.855	1421.930	1419.864	1421.939	1421.890	1421.896
A55	979+49	1419.549	1421.428	1419.435	1421.510	1419.444	1421.519	1421.473	1421.470
A56	988+55	1420.010	1421.889	1419.897	1421.972	1419.914	1421.989	1421.965	1421.937
A57	999+13	1418.688	1420.567	1418.577	1420.652	1418.592	1420.667	1420.610	1420.597
A58	1010+08	1419.880	1421.759	1419.759	1421.834	1419.778	1421.853	1421.781	1421.789
A59	1020+11	1419.971	1421.850	1419.839	1421.914	1419.868	1421.943	1421.896	1421.873
A60	1030+98	1420.029	1421.908	1419.907	1421.982	1419.908	1421.983	1421.959	1421.940
A61	1040+15	1419.401	1421.280	1419.295	1421.370	1419.281	1421.356	1421.322	1421.314
A62	1049+90	1419.669	1421.548	1419.560	1421.635	1419.564	1421.639	1421.591	1421.586
			(Fig. 2-1)		(Fig. 2-1)		(Fig. 2-1 Plot Data)		

Table 2 (Continued)

STA 729+58 to STA 1049+91 Crest Settlement Monument Elevations

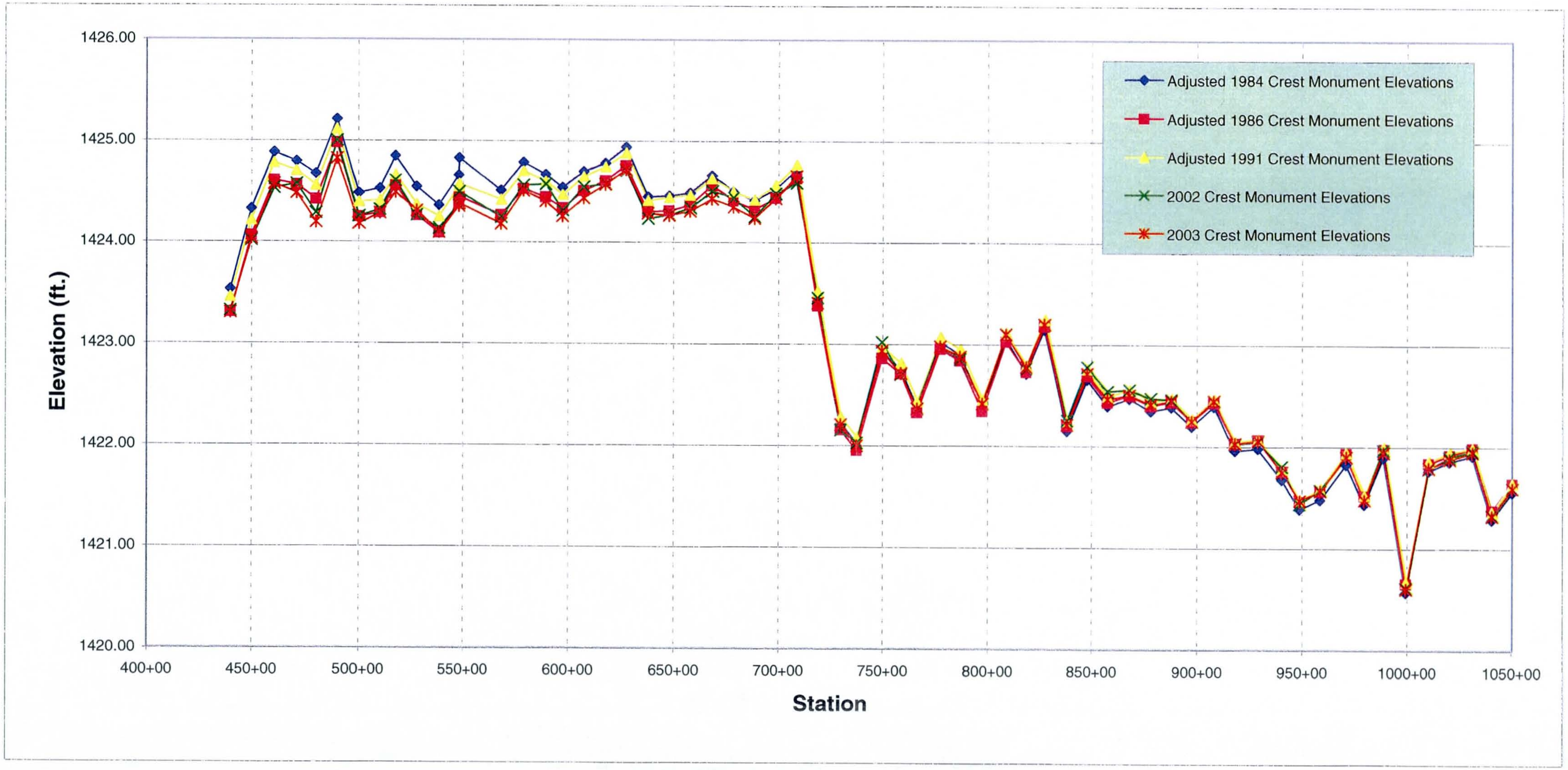


Figure 2-1
Crest Settlement Monuments' Elevation Chart

Table 3 below summarizes the Crest Monument Elevations adjusted to 1988 NAVD for the years 1984, 1986, 1991, 2002, and 2003, and calculates the elevation change from the 1984 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 84	Adj 86	Adj 91	2002	2003
A1	439+78	1423.524	1423.298	1423.448	1423.314	1423.295
A2	449+80	1424.321	1424.058	1424.210	1424.012	1424.028
A3	460+60	1424.884	1424.604	1424.779	1424.528	1424.567
A4	471+23	1424.794	1424.558	1424.699	1424.567	1424.479
A5	480+20	1424.670	1424.415	1424.551	1424.281	1424.190
A6	490+20	1425.210	1424.978	1425.113	1425.000	1424.821
A7	500+20	1424.479	1424.244	1424.392	1424.249	1424.176
A8	510+00	1424.522	1424.284	1424.404	1424.314	1424.279
A9	517+42	1424.848	1424.557	1424.659	1424.603	1424.485
A10	527+58	1424.541	1424.257	1424.365	1424.262	1424.304
A11	538+27	1424.358	1424.086	1424.245	1424.131	1424.092
A12	547+93	1424.655	1424.383	1424.542	1424.370	1424.348
A13	548+13	1424.828	1424.443	1424.568	1424.491	1424.375
A14	567+97	1424.504	1424.260	1424.413	1424.242	1424.173
A15	578+65	1424.784	1424.521	1424.696	1424.557	1424.502
A16	589+28	1424.665	1424.438	1424.593	1424.567	1424.397
A17	597+16	1424.534	1424.331	1424.463	1424.308	1424.251
A18	607+37	1424.694	1424.500	1424.641	1424.541	1424.429
A19	617+67	1424.778	1424.601	1424.739	1424.570	1424.563
A20	627+50	1424.938	1424.751	1424.879	1424.702	1424.711
A21	637+77	1424.445	1424.297	1424.406	1424.229	1424.280
A22	647+92	1424.453	1424.304	1424.436	1424.265	1424.259
A23	657+97	1424.482	1424.361	1424.461	1424.334	1424.302
A24	668+08	1424.655	1424.554	1424.623	1424.501	1424.423
A25	678+32	1424.498	1424.407	1424.497	1424.436	1424.344
A26	688+40	1424.408	1424.320	1424.416	1424.249	1424.232
A27	698+42	1424.525	1424.433	1424.559	1424.478	1424.429
A28	708+44	1424.716	1424.629	1424.764	1424.583	1424.651
A29	718+47	1423.452	1423.370	1423.509	1423.448	1423.398

Adj 86 - Adj 84	Adj 91 - Adj 84	2002 - Adj 84	2003 - Adj 84
-0.226	-0.076	-0.210	-0.229
-0.263	-0.111	-0.309	-0.293
-0.280	-0.105	-0.356	-0.317
-0.236	-0.095	-0.227	-0.315
-0.255	-0.119	-0.389	-0.480
-0.232	-0.097	-0.210	-0.389
-0.235	-0.087	-0.230	-0.303
-0.238	-0.118	-0.208	-0.243
-0.291	-0.189	-0.245	-0.363
-0.284	-0.176	-0.279	-0.237
-0.272	-0.113	-0.227	-0.266
-0.272	-0.113	-0.285	-0.307
-0.385	-0.260	-0.337	-0.453
-0.244	-0.091	-0.262	-0.331
-0.263	-0.088	-0.227	-0.282
-0.227	-0.072	-0.098	-0.268
-0.203	-0.071	-0.226	-0.283
-0.194	-0.053	-0.153	-0.265
-0.177	-0.039	-0.208	-0.215
-0.187	-0.059	-0.236	-0.227
-0.148	-0.039	-0.216	-0.165
-0.149	-0.017	-0.188	-0.194
-0.121	-0.021	-0.148	-0.180
-0.101	-0.032	-0.154	-0.232
-0.091	-0.001	-0.062	-0.154
-0.088	0.008	-0.159	-0.176
-0.092	0.034	-0.047	-0.096
-0.087	0.048	-0.133	-0.065
-0.082	0.057	-0.004	-0.054

(Fig. 3-1 Plot Data)

Table 3

STA 439+78 to STA 718+47

Crest Monument Elevation Change from Initial 1984 Survey Data as Baseline

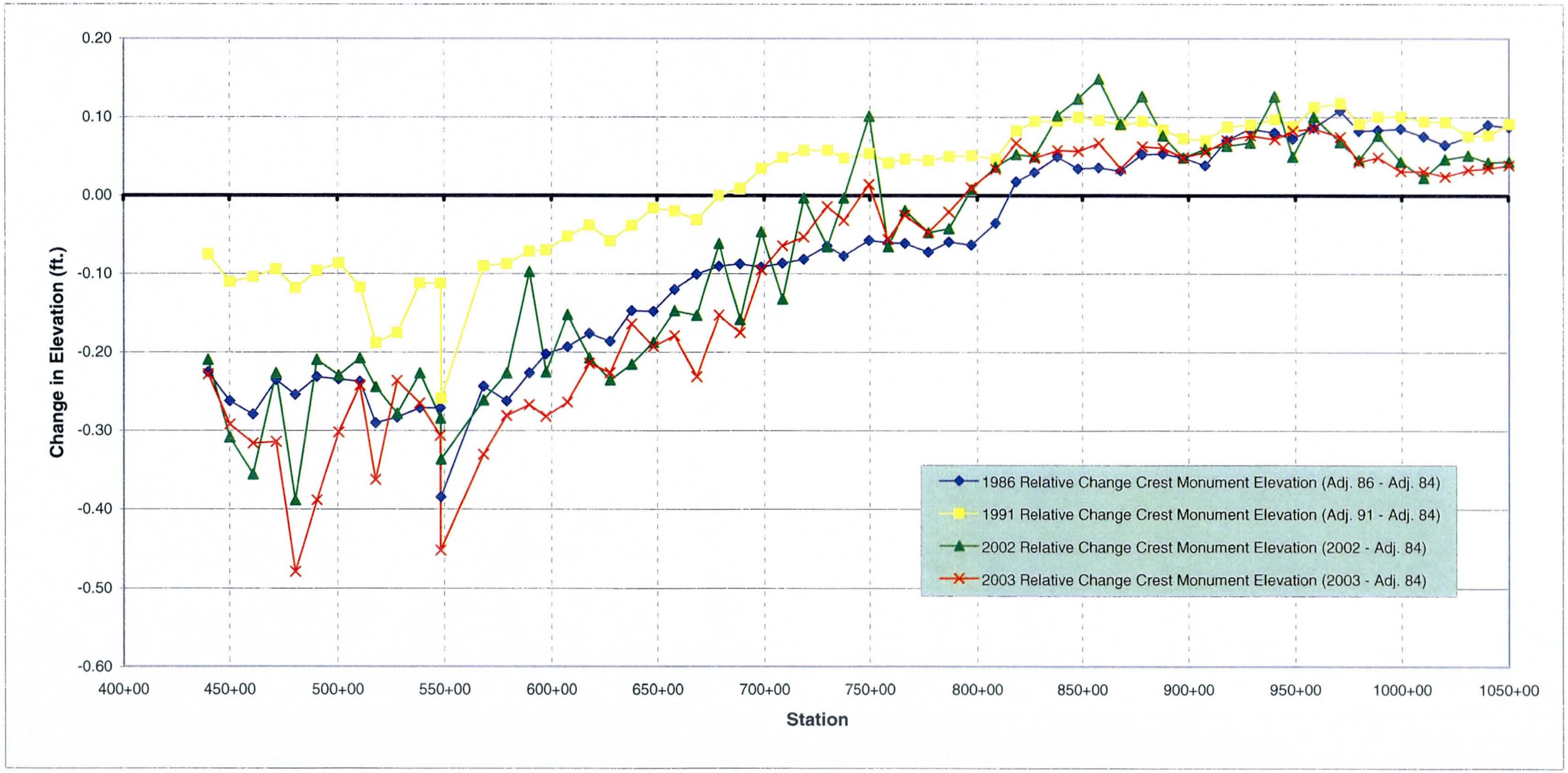
Marker	Station	Adj 84	Adj 86	Adj 91	2002	2003	Adj 86 - Adj 84	Adj 91 - Adj 84	2002 - Adj 84	2003 - Adj 84
		Crest Monument Survey Data								
A30	729+58	1422.215	1422.150	1422.272	1422.149	1422.200	-0.065	0.057	-0.066	-0.015
A31	737+36	1422.028	1421.950	1422.075	1422.024	1421.995	-0.078	0.047	-0.004	-0.033
A32	749+52	1422.914	1422.856	1422.967	1423.015	1422.927	-0.058	0.053	0.101	0.013
A33	758+41	1422.766	1422.705	1422.807	1422.700	1422.709	-0.061	0.041	-0.066	-0.057
A34	766+14	1422.382	1422.320	1422.428	1422.362	1422.356	-0.062	0.046	-0.020	-0.026
A35	777+05	1423.020	1422.947	1423.064	1422.972	1422.972	-0.073	0.044	-0.048	-0.048
A36	786+49	1422.894	1422.834	1422.943	1422.851	1422.872	-0.060	0.049	-0.043	-0.022
A37	796+99	1422.398	1422.334	1422.448	1422.405	1422.407	-0.064	0.050	0.007	0.009
A38	808+46	1423.058	1423.022	1423.104	1423.094	1423.091	-0.036	0.046	0.036	0.033
A39	818+25	1422.707	1422.724	1422.789	1422.759	1422.773	0.017	0.082	0.052	0.066
A40	827+22	1423.143	1423.172	1423.237	1423.192	1423.190	0.029	0.094	0.049	0.047
A41	837+91	1422.145	1422.194	1422.239	1422.247	1422.202	0.049	0.094	0.102	0.057
A42	847+76	1422.653	1422.687	1422.752	1422.776	1422.709	0.034	0.099	0.123	0.056
A43	857+54	1422.388	1422.423	1422.483	1422.536	1422.454	0.035	0.095	0.148	0.066
A44	867+87	1422.461	1422.492	1422.550	1422.552	1422.495	0.031	0.089	0.091	0.034
A45	877+95	1422.341	1422.393	1422.435	1422.467	1422.403	0.052	0.094	0.126	0.062
A46	887+65	1422.378	1422.431	1422.461	1422.454	1422.438	0.053	0.083	0.076	0.060
A47	897+33	1422.190	1422.238	1422.262	1422.238	1422.238	0.048	0.072	0.048	0.048
A48	907+89	1422.385	1422.423	1422.455	1422.444	1422.440	0.038	0.070	0.059	0.055
A49	917+96	1421.955	1422.025	1422.042	1422.018	1422.025	0.070	0.087	0.063	0.070
A50	928+83	1421.977	1422.062	1422.067	1422.044	1422.053	0.085	0.090	0.067	0.076
A51	940+19	1421.675	1421.755	1421.772	1421.801	1421.746	0.080	0.097	0.126	0.071
A52	948+59	1421.381	1421.453	1421.469	1421.430	1421.463	0.072	0.088	0.049	0.082
A53	958+35	1421.472	1421.558	1421.584	1421.572	1421.557	0.086	0.112	0.100	0.085
A54	970+80	1421.822	1421.930	1421.939	1421.890	1421.896	0.108	0.117	0.068	0.074
A55	979+49	1421.428	1421.510	1421.519	1421.473	1421.470	0.082	0.091	0.045	0.042
A56	988+55	1421.889	1421.972	1421.989	1421.965	1421.937	0.083	0.100	0.076	0.048
A57	999+13	1420.567	1420.652	1420.667	1420.610	1420.597	0.085	0.100	0.043	0.030
A58	1010+08	1421.759	1421.834	1421.853	1421.781	1421.789	0.075	0.094	0.022	0.030
A59	1020+11	1421.850	1421.914	1421.943	1421.896	1421.873	0.064	0.093	0.046	0.023
A60	1030+98	1421.908	1421.982	1421.983	1421.959	1421.940	0.074	0.075	0.051	0.032
A61	1040+15	1421.280	1421.370	1421.356	1421.322	1421.314	0.090	0.076	0.042	0.034
A62	1049+90	1421.548	1421.635	1421.639	1421.591	1421.586	0.087	0.091	0.043	0.038

(Fig. 3-1 Plot Data)

Table 3 (Continued)

STA 729+58 to 1049+90

Crest Monument Elevation Change from Initial 1984 Survey Data as Baseline



<u>**Figure 3-1**</u>

Relative Change in Crest Monument Elevation Chart, 1984 Survey Data Baseline Elevation Reference

Table 4 below summarizes the Toe Monument Elevations adjusted to 1988 NAVD for the years 1984, 1986, 2002, and 2003, and calculates the elevation change from the 1984 initial survey as the baseline. **Figure 4-1** illustrates the relative change in Settlement monuments as calculated in **Table 4**.

NOTE: There was no data reported for the toe monuments in 1991.

Marker	Toe Monument Survey Data									
	Station	1984	Adj 84	1986	Adj 86	2002	2003	Adj 86 - Adj 84	2002 - Adj 84	2003 - Adj 84
B1	439+78	1417.738	1419.617	1417.329	1419.404	1419.396	1419.389	-0.213	-0.221	-0.228
B2	449+80	1413.495	1415.374	1413.096	1415.171	1415.187	1415.133	-0.203	-0.187	-0.241
B3	460+00	1404.015	1405.894	1403.569	1405.644	1405.896	1405.852	-0.250	0.002	-0.042
B4	471+23	1398.822	1400.701	1398.436	1400.511	1400.554	1400.453	-0.190	-0.147	-0.248
B5	480+20	1396.678	1398.557	1396.326	1398.401	1398.573	1398.526	-0.156	0.016	-0.031
B6	490+20	1395.051	1396.930	1394.686	1396.761	1396.719	1396.690	-0.169	-0.211	-0.240
B7	500+20	1395.921	1397.800	1395.569	1397.644	1397.605	1397.641	-0.156	-0.195	-0.159
B8	510+00	1395.834	1397.713	1395.452	1397.527	1397.999	1397.920	-0.186	0.286	0.207
B9	517+42	1395.724	1397.603	1395.343	1397.418	1397.490	1397.420	-0.185	-0.113	-0.183
B10	527+58	1395.714	1397.593	1395.293	1397.368	1397.464	1397.434	-0.225	-0.129	-0.159
B11	538+27	1396.631	1398.510	1396.216	1398.291	1398.724	1398.710	-0.219	0.214	0.200
B12	547+93	1396.281	1398.160	1395.879	1397.954	1398.041	1397.994	-0.206	-0.119	-0.166
B13	558+13	1396.104	1397.983	1395.729	1397.804	1397.822	1397.820	-0.179	-0.161	-0.163
B14	567+97	1395.704	1397.583	1395.315	1397.390	1397.448	1397.403	-0.193	-0.135	-0.180
B15	578+65	1395.114	1396.993	1394.735	1396.810	1397.831	1397.813	-0.183	0.838	0.820
B16	588+17	1395.697	1397.576	1395.321	1397.396		1397.428	-0.180		-0.148
B17	597+16	1394.670	1396.549	1394.398	1396.473	1396.490	1396.498	-0.076	-0.059	-0.051
B18	607+37	1395.060	1396.939	1394.781	1396.856	1396.939	1396.901	-0.083	0.000	-0.038
B19	617+67	1397.234	1399.113	1396.951	1399.026	1399.154	1399.097	-0.087	0.041	-0.016
B20	627+50	1395.110	1396.989	1394.808	1396.883		1396.948	-0.106		-0.041
B21	638+11	1395.030	1396.909	1394.741	1396.816		1397.237	-0.093		0.328
B22	647+92	1394.694	1396.573	1394.335	1396.410	1396.535	1396.487	-0.163	-0.038	-0.086
B23	658+12	1394.187	1396.066	1393.835	1395.910		1395.930	-0.156		-0.136
B24	668+38	1391.014	1392.893	1390.645	1392.720		1392.756	-0.173		-0.137
B25	678+32	1394.490	1396.369	1394.178	1396.253	1396.260	1396.279	-0.116	-0.109	-0.090
B26	688+40	1396.670	1398.549	1396.372	1398.447	1398.517	1398.500	-0.102	-0.032	-0.049
B27	698+42	1396.771	1398.650	1396.465	1398.540	1398.524	1398.600	-0.110	-0.126	-0.050
B28	708+44	1396.281	1398.160	1395.958	1398.033	1398.061	1398.126	-0.127	-0.099	-0.034
B29	718+47	1397.162	1399.041	1396.848	1398.923	1398.960	1398.986	-0.118	-0.081	-0.055

Note: *Means no readings were obtained

(Fig. 4-1 Plot Data)

<u>**Table 4**</u>

STA 439+78 to STA 718+47

Toe Monument Elevation Change from Initial 1984 Survey Data as Baseline

Marker	Station	Toe Monument Survey Data 1984	Adj 84	1986	Adj 86	2002	2003	Adj 86 - Adj 84	2002 - Adj 84	2003 - Adj 84
B30	729+58	1391.828	1393.707	1391.542	1393.617	1393.727	1393.731	-0.090	0.020	0.024
B31	737+36	1389.333	1391.212	1389.049	1391.124	1391.207	1391.233	-0.088	-0.005	0.021
B32	749+52	1386.358	1388.237	1386.114	1388.189	1388.264	1388.274	-0.048	0.027	0.037
B33	758+41	1388.571	1390.450	1388.334	1390.409	1390.509	1390.538	-0.041	0.059	0.088
B34	766+14	1392.114	1393.993	1391.874	1393.949	1394.035	1394.071	-0.044	0.042	0.078
B35	777+05	1397.228	1399.107	1396.987	1399.062	1399.068	1399.155	-0.045	-0.039	0.048
B36	786+49	1393.940	1395.819	1393.692	1395.767	1395.827	1395.874	-0.052	0.008	0.055
B37	796+99	1395.121	1397.000	1394.870	1396.945	1397.008	1397.015	-0.055	0.008	0.015
B38	808+46	1396.150	1398.029	1395.912	1397.987	1398.035	1398.044	-0.042	0.006	0.015
B39	818+25	1400.893	1402.772	1400.641	1402.716	1402.720	1402.771	-0.056	-0.052	-0.001
B40	827+35	1400.014	1401.893	1399.751	1401.826	1401.916	1401.910	-0.067	0.023	0.017
B41	837+91	1397.068	1398.947	1396.816	1398.891	1398.973	1398.995	-0.056	0.026	0.048
B42	847+76	1397.678	1399.557	1397.414	1399.489	1399.636	1399.599	-0.068	0.079	0.042
B43	857+54	1397.261	1399.140	1396.994	1399.069	1399.222	1399.196	-0.071	0.082	0.056
B44	867+87	1399.023	1400.902	1398.753	1400.828	1401.017	1400.976	-0.074	0.115	0.074
B45	877+63	1395.735	1397.614	1395.446	1397.521	1397.651	1397.696	-0.093	0.037	0.082
B46	887+65	1396.688	1398.567	1396.391	1398.466	1398.589	1398.645	-0.101	0.022	0.078
B47	897+49	1394.666	1396.545	1394.383	1396.458	1396.627	1396.585	-0.087	0.082	0.040
B48	907+91	1394.838	1396.717	1394.550	1396.625	1396.775	1396.795	-0.092	0.058	0.078
B49	917+99	1397.314	1399.193	1397.025	1399.100	1399.334	1399.241	-0.093	0.141	0.048
B50	928+83	1397.737	1399.616	1397.428	1399.503	1399.623	1399.672	-0.113	0.007	0.056
B51	940+19	1396.522	1398.401	1396.216	1398.291	1398.445	1398.480	-0.110	0.044	0.079
B52	948+66	1394.400	1396.279	1394.106	1396.181	1396.335	1396.344	-0.098	0.056	0.065
B53	958+35	1395.842	1397.721	1395.561	1397.636	1397.782	1397.763	-0.085	0.061	0.042
B54	970+80			1395.267	1397.342	1397.484	1397.491			
B55	980+00			1393.278	1395.353	1395.505	1395.483			
B56	988+60	1395.240	1397.119	1394.955	1397.030	1397.188	1397.140	-0.089	0.069	0.021
B57	1000+00	1394.178	1396.057	1393.874	1395.949	1396.099	1396.134	-0.108	0.042	0.077
B58	1010+00	1394.801	1396.680	1394.485	1396.560	1396.713	1396.723	-0.120	0.033	0.043
B59	1020+00	1394.666	1396.545	1394.323	1396.398	1396.572	1396.563	-0.147	0.027	0.018
B60	1030+00	1392.225	1394.104	1391.863	1393.938	1394.127	1394.100	-0.166	0.023	-0.004
B61	1040+00	1386.155	1388.034	1385.793	1387.868	1387.785	1387.742	-0.166	-0.249	-0.292
B62	1050+00	1388.535	1390.414	1388.289	1390.364	1390.413	1390.440	-0.050	-0.001	0.026

(Fig. 4-1) (Fig. 4-1 Plot Data) (Fig. 4-1 Plot Data)

Note: *Means no readings were obtained

Table 4 (Continued)
STA 729+58 to 1050+00
Crest Monument Elevation Change from Initial 1984 Survey Data as Baseline

Settlement Monuments - Toe

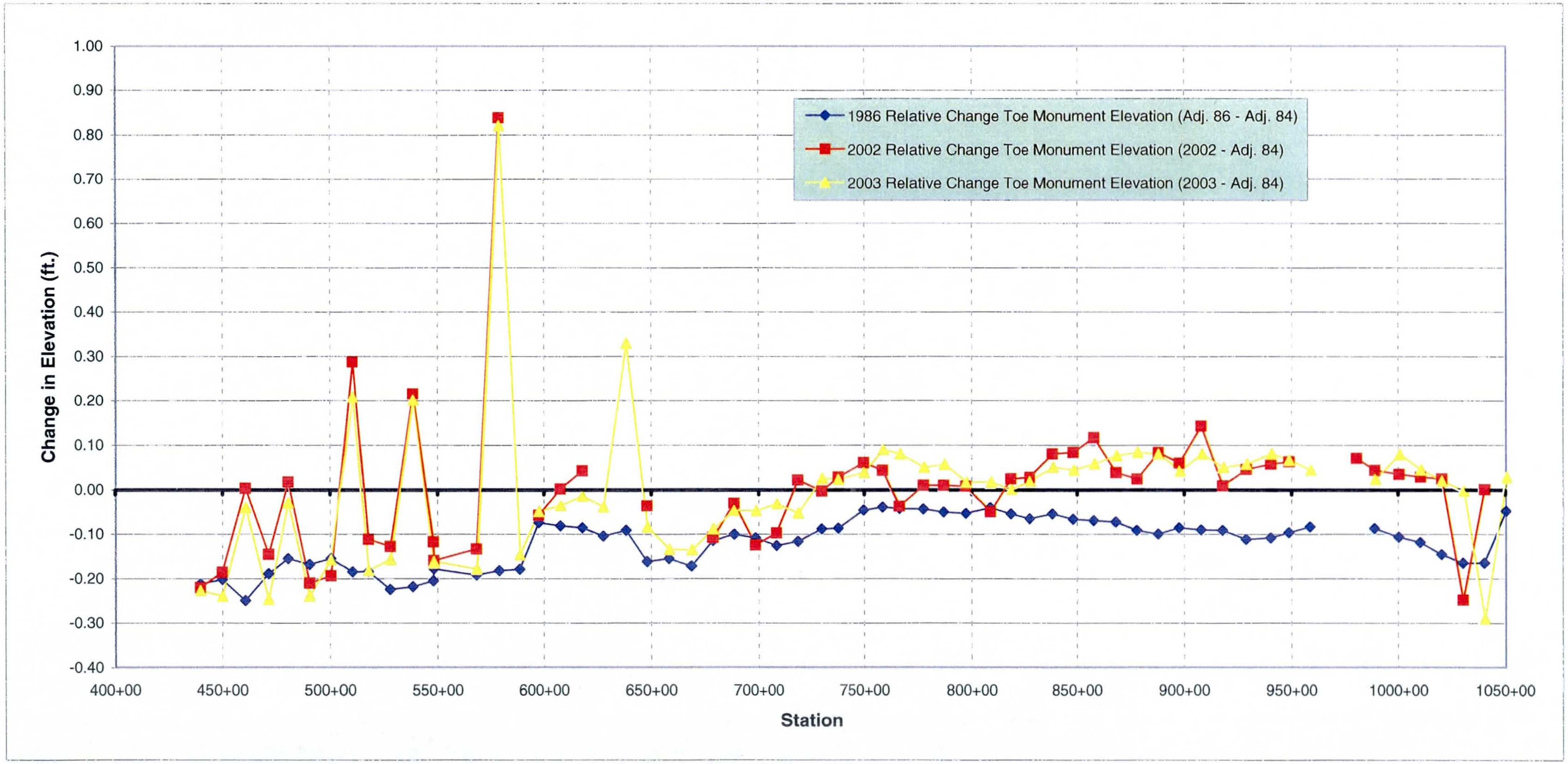


Figure 4-1

Relative Change in Toe Monument Elevation Chart, 1984 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 84	Adj 86	2002	2003
A1-B1	439+78	3.907	3.894	3.918	3.906
A2-B2	449+80	8.947	8.887	8.825	8.895
A3-B3	460+60	18.990	18.960	18.632	18.715
A4-B4	471+23	24.093	24.047	24.013	24.026
A5-B5	480+20	26.113	26.014	25.708	25.664
A6-B6	490+20	28.280	28.217	28.281	28.131
A7-B7	500+20	26.679	26.600	26.644	26.535
A8-B8	510+00	26.809	26.757	26.315	26.359
A9-B9	517+42	27.245	27.139	27.113	27.065
A10-B10	527+58	26.948	26.889	26.798	26.870
A11-B11	538+27	25.848	25.795	25.407	25.382
A12-B12	547+93	26.495	26.429	26.329	26.354
A13-B13	548+13	26.845	26.639	26.669	26.555
A14-B14	567+97	26.921	26.870	26.794	26.770
A15-B15	578+65	27.791	27.711	26.726	26.689
A16-B16	589+28	27.089	27.042		26.969
A17-B17	597+16	27.985	27.858	27.818	27.753
A18-B18	607+37	27.755	27.644	27.602	27.528
A19-B19	617+67	25.665	25.575	25.416	25.466
A20-B20	627+50	27.949	27.868		27.763
A21-B21	637+77	27.536	27.481		27.043
A22-B22	647+92	27.880	27.894	27.730	27.772
A23-B23	657+97	28.416	28.451		28.372
A24-B24	668+08	31.762	31.834		31.667
A25-B25	678+32	28.129	28.154	28.176	28.065
A26-B26	688+40	25.859	25.873	25.732	25.732
A27-B27	698+42	25.875	25.893	25.954	25.829
A28-B28	708+44	26.556	26.596	26.522	26.525
A29-B29	718+47	24.411	24.447	24.488	24.412
A30-B30	729+58	28.508	28.533	28.422	28.469
A31-B31	737+36	30.816	30.826	30.817	30.762

(Fig. 5-1 Plot Data)

Monument Pair	Station	Elevation Difference: Crest Mon. to Toe Mon. Adj 84	Adj 86	2002	2003
A32-B32	749+52	34.677	34.725	34.751	34.653
A33-B33	758+41	32.316	32.357	32.191	32.171
A34-B34	766+14	28.389	28.433	28.327	28.285
A35-B35	777+05	23.913	23.958	23.904	23.817
A36-B36	786+49	27.075	27.127	27.024	26.998
A37-B37	796+99	25.398	25.453	25.397	25.392
A38-B38	808+46	25.029	25.071	25.059	25.047
A39-B39	818+25	19.935	19.991	20.039	20.002
A40-B40	827+22	21.250	21.317	21.276	21.280
A41-B41	837+91	23.198	23.254	23.274	23.207
A42-B42	847+76	23.096	23.164	23.140	23.110
A43-B43	857+54	23.248	23.319	23.314	23.258
A44-B44	867+87	21.559	21.633	21.535	21.519
A45-B45	877+95	24.727	24.820	24.816	24.707
A46-B46	887+65	23.811	23.912	23.865	23.793
A47-B47	897+33	25.645	25.732	25.611	25.653
A48-B48	907+89	25.668	25.760	25.669	25.645
A49-B49	917+96	22.762	22.855	22.684	22.784
A50-B50	928+83	22.361	22.474	22.421	22.381
A51-B51	940+19	23.274	23.384	23.356	23.266
A52-B52	948+59	25.102	25.200	25.095	25.119
A53-B53	958+35	23.751	23.836	23.790	23.794
A54-B54	970+80		24.480	24.406	24.405
A55-B55	979+49		26.075	25.968	25.987
A56-B56	988+55	24.770	24.859	24.777	24.797
A57-B57	999+13	24.510	24.618	24.511	24.463
A58-B58	1010+08	25.079	25.199	25.068	25.066
A59-B59	1020+11	25.305	25.452	25.324	25.310
A60-B60	1030+98	27.804	27.970	27.832	27.840
A61-B61	1040+15	33.246	33.412	33.537	33.572
A62-B62	1049+90	31.134	31.184	31.178	31.146

(Fig. 5-1 Plot Data)

Table 5

Elevation Difference Between Crest Monument and Corresponding Toe Monument

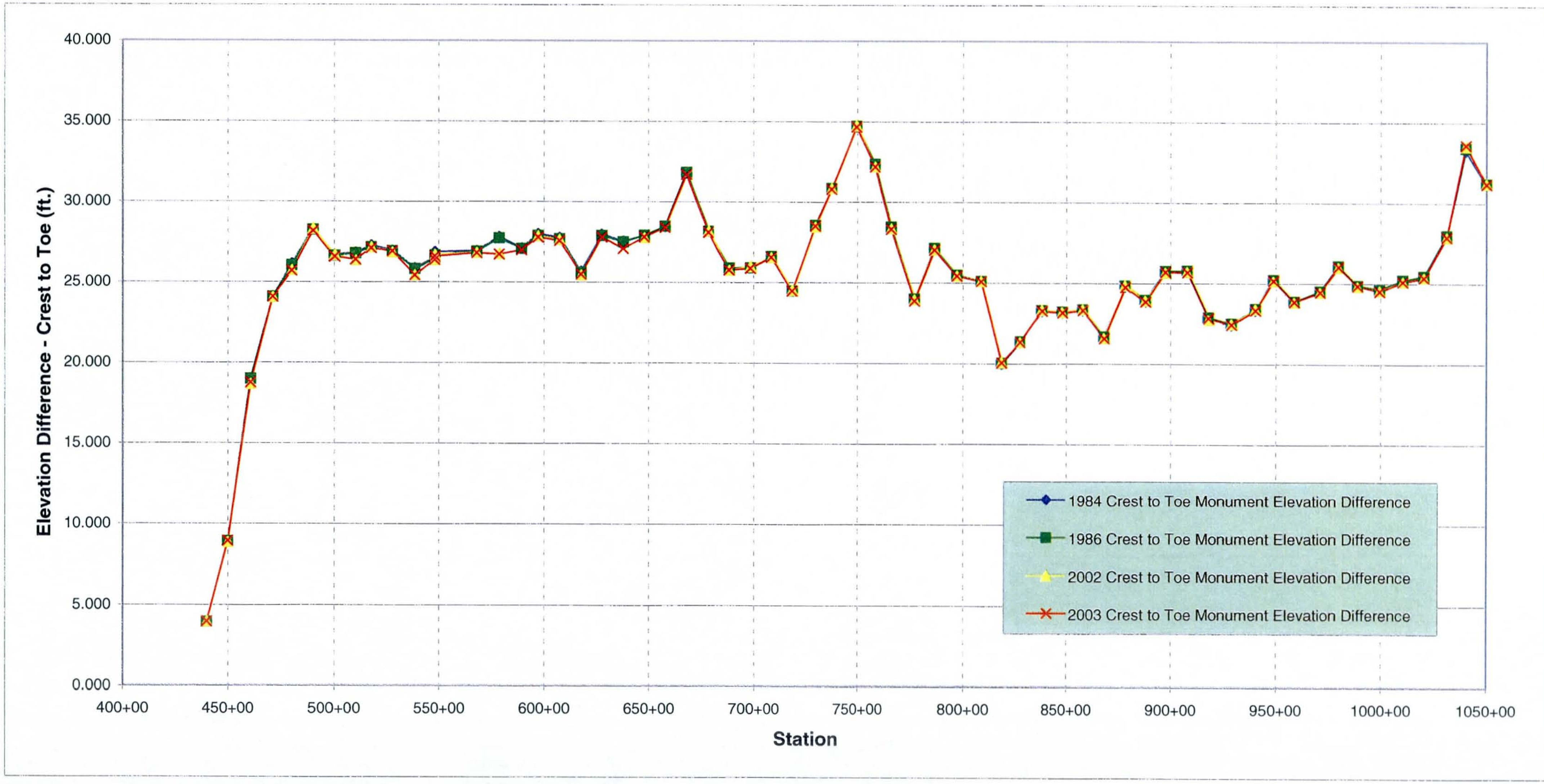


Figure 5-1

Elevation Difference Between Crest Settlement Monument and Corresponding Toe Settlement Monument

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 1984 survey data used as a baseline.

Monument Pair	Station	Adj 86 - Adj 84	2002 - Adj 84	2003 - Adj 84
A1-B1	439+78	-0.013	0.011	-0.001
A2-B2	449+80	-0.060	-0.122	-0.052
A3-B3	460+60	-0.030	-0.358	-0.275
A4-B4	471+23	-0.046	-0.080	-0.067
A5-B5	480+20	-0.099	-0.405	-0.449
A6-B6	490+20	-0.063	0.001	-0.149
A7-B7	500+20	-0.079	-0.035	-0.144
A8-B8	510+00	-0.052	-0.494	-0.450
A9-B9	517+42	-0.106	-0.132	-0.180
A10-B10	527+58	-0.059	-0.150	-0.078
A11-B11	538+27	-0.053	-0.441	-0.466
A12-B12	547+93	-0.066	-0.166	-0.141
A13-B13	548+13	-0.206	-0.176	-0.290
A14-B14	567+97	-0.051	-0.127	-0.151
A15-B15	578+65	-0.080	-1.065	-1.102
A16-B16	589+28	-0.047		-0.120
A17-B17	597+16	-0.127	-0.167	-0.232
A18-B18	607+37	-0.111	-0.153	-0.227
A19-B19	617+67	-0.090	-0.249	-0.199
A20-B20	627+50	-0.081		-0.186
A21-B21	637+77	-0.055		-0.493

(Fig. 6-1 plot data)

Monument Pair	Station	Adj 86 - Adj 84	2002 - Adj 84	2003 - Adj 84
A22-B22	647+92	0.014	-0.150	-0.108
A23-B23	657+97	0.035		-0.044
A24-B24	668+08	0.072		-0.095
A25-B25	678+32	0.025	0.047	-0.064
A26-B26	688+40	0.014	-0.127	-0.127
A27-B27	698+42	0.018	0.079	-0.046
A28-B28	708+44	0.040	-0.034	-0.031
A29-B29	718+47	0.036	0.077	0.001
A30-B30	729+58	0.025	-0.086	-0.039
A31-B31	737+36	0.010	0.001	-0.054
A32-B32	749+52	0.048	0.074	-0.024
A33-B33	758+41	0.041	-0.125	-0.145
A34-B34	766+14	0.044	-0.062	-0.104
A35-B35	777+05	0.045	-0.009	-0.096
A36-B36	786+49	0.052	-0.051	-0.077
A37-B37	796+99	0.055	-0.001	-0.006
A38-B38	808+46	0.042	0.030	0.018
A39-B39	818+25	0.056	0.104	0.067
A40-B40	827+22	0.067	0.026	0.030
A41-B41	837+91	0.056	0.076	0.009
A42-B42	847+76	0.068	0.044	0.014

(Fig. 6-1 plot data)

Monument Pair	Station	Adj 86 - Adj 84	2002 - Adj 84	2003 - Adj 84
A43-B43	857+54	0.071	0.066	0.010
A44-B44	867+87	0.074	-0.024	-0.040
A45-B45	877+95	0.093	0.089	-0.020
A46-B46	887+65	0.101	0.054	-0.018
A47-B47	897+33	0.087	-0.034	0.008
A48-B48	907+89	0.092	0.001	-0.023
A49-B49	917+96	0.093	-0.078	0.022
A50-B50	928+83	0.113	0.060	0.020
A51-B51	940+19	0.110	0.082	-0.008
A52-B52	948+59	0.098	-0.007	0.017
A53-B53	958+35	0.085	0.039	0.043
A54-B54	970+80			
A55-B55	979+49			
A56-B56	988+55	0.089	0.007	0.027
A57-B57	999+13	0.108	0.001	-0.047
A58-B58	1010+08	0.120	-0.011	-0.013
A59-B59	1020+11	0.147	0.019	0.005
A60-B60	1030+98	0.166	0.028	0.036
A61-B61	1040+15	0.166	0.291	0.326
A62-B62	1049+90	0.050	0.044	0.012

(Fig. 6-1 plot data)

Table 6

Relative Change in Differential Elevation Between Crest Monument and Corresponding Toe Monument, 1987 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

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	Design Drawing	Adj Design	2002	2002 - Adj Design	2003	2003 - Adj Design	Description
2000	583+80	1394.090	1395.969	1396.362	0.393	1396.576	0.607	Gated Outlet Upstream Invert Elevation
2003	583+80	1393.310	1395.189	1395.486	0.297	1395.406	0.217	Gated Outlet Downstream Headwall Elevation
2007	746+00	N/A		1383.976		1384.004		Gated Outlet Upstream Invert Elevation
2012	746+00	N/A		1364.416		1364.441		Gated Outlet Downstream Top of Pipe Elevation
2013	1045+08	1387.300	1389.179	1389.452	0.273	1389.443	0.264	Principal Spillway Upstream Inlet Elevation
2015	1045+80	1357.560	1359.439	1359.629	0.190	1359.729	0.290	Principal Spillway Downstream Top of Wall Elevation
2017	1045+80	1344.530	1346.409	1346.768	0.359	1346.706	0.297	Principal Spillway Downstream Sill Elevation
2009	939+41	1408.400	1410.279	1410.725	0.446	1410.647	0.368	Emergency Spillway Left Side Elevation
2010	938+70	1408.400	1410.279	1410.696	0.417	1410.773	0.494	Emergency Spillway Center Elevation
2011	938+01	1408.400	1410.279	1410.696	0.417	1410.680	0.401	Emergency Spillway Right Side Elevation

NOTE: As-constructed elevation data has not been found. Baseline elevation data was obtained from the "as-built" drawings.

<u>Table 7</u>
Miscellaneous Points

Table 8A, 8B, & 8C below summarize the settlement that has occurred at Harquahala FRS from 1984 to 2003.

Crest Marker	2003 - Adj 84
A-1	-0.229
A-2	-0.293
A-3	-0.317
A-4	-0.315
A-5	-0.480
A-6	-0.389
A-7	-0.303
A-8	-0.243
A-9	-0.363
A-10	-0.237
A-11	-0.266
A-12	-0.307
A-13	-0.453
A-14	-0.331
A-15	-0.282
A-16	-0.268
A-17	-0.283
A-18	-0.265
A-19	-0.215
A-20	-0.227
A-21	-0.165

Crest Marker	2003 - Adj 84
A-22	-0.194
A-23	-0.180
A-24	-0.232
A-25	-0.154
A-26	-0.176
A-27	-0.096
A-28	-0.065
A-29	-0.054
A-30	-0.015
A-31	-0.033
A-32	0.013
A-33	-0.057
A-34	-0.026
A-35	-0.048
A-36	-0.022
A-37	0.009
A-38	0.033
A-39	0.066
A-40	0.047
A-41	0.057
A-42	0.056

Crest Marker	2003 - Adj 84
A-43	0.066
A-44	0.034
A-45	0.062
A-46	0.060
A-47	0.048
A-48	0.055
A-49	0.070
A-50	0.076
A-51	0.071
A-52	0.082
A-53	0.085
A-54	0.074
A-55	0.042
A-56	0.048
A-57	0.030
A-58	0.030
A-59	0.023
A-60	0.032
A-61	0.034
A-62	0.038

Toe Marker	2003 - Adj 84
B-1	-0.228
B-2	-0.241
B-3	-0.042
B-4	-0.248
B-5	-0.031
B-6	-0.240
B-7	-0.159
B-8	0.207
B-9	-0.183
B-10	-0.159
B-11	0.200
B-12	-0.166
B-13	-0.163
B-14	-0.180
B-15	0.820
B-16	-0.148
B-17	-0.051
B-18	-0.038
B-19	-0.016
B-20	-0.041
B-21	0.328

Toe Marker	2003 - Adj 84
B-22	-0.086
B-23	-0.136
B-24	-0.137
B-25	-0.090
B-26	-0.049
B-27	-0.050
B-28	-0.034
B-29	-0.055
B-30	0.024
B-31	0.021
B-32	0.037
B-33	0.088
B-34	0.078
B-35	0.048
B-36	0.055
B-37	0.015
B-38	0.015
B-39	-0.001
B-40	0.017
B-41	0.048
B-42	0.042

Toe Marker	2003 - Adj 84
B-43	0.056
B-44	0.074
B-45	0.082
B-46	0.078
B-47	0.040
B-48	0.078
B-49	0.048
B-50	0.056
B-51	0.079
B-52	0.065
B-53	0.042
B-54	0.000
B-55	0.000
B-56	0.021
B-57	0.077
B-58	0.043
B-59	0.018
B-60	-0.004
B-61	-0.292
B-62	0.026

Table 8-A
Settlement Summary of Crest and Toe Monuments

Marker	2003 - Adj As-Built	Description
2013	0.264	Principal Spillway Upstream Inlet Elevation
2015	0.290	Principal Spillway Downstream Top of Wall Elevation

Table 8-B
Settlement Summary of Principal Spillway

Marker	2003 - Adj As-Built	Description
2009	0.368	Emergency Spillway Left Side Elevation
2010	0.494	Emergency Spillway Center Elevation
2011	0.401	Emergency Spillway Right Side Elevation

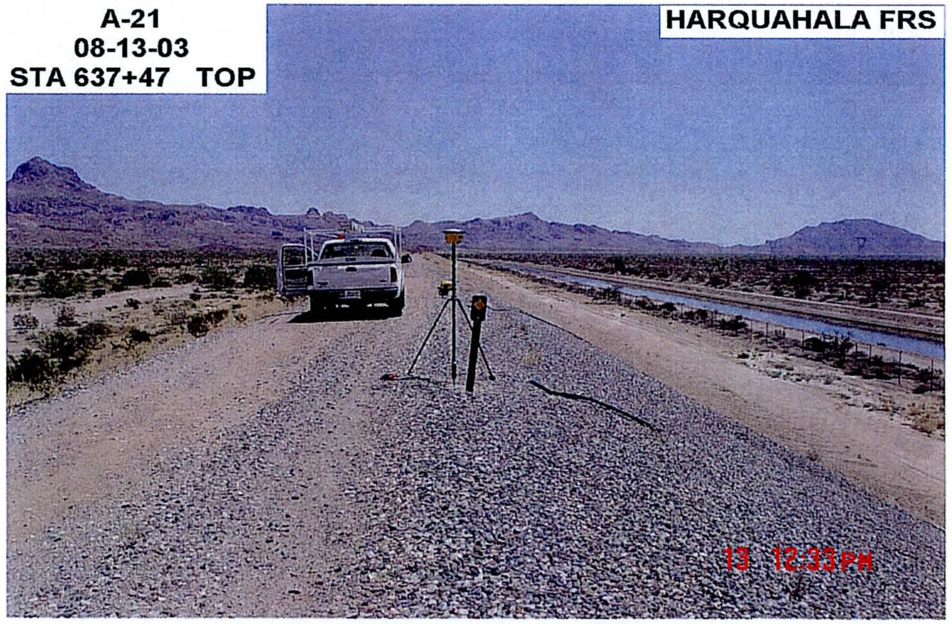
Table 8-C
Settlement Summary of Emergency Spillway Crest

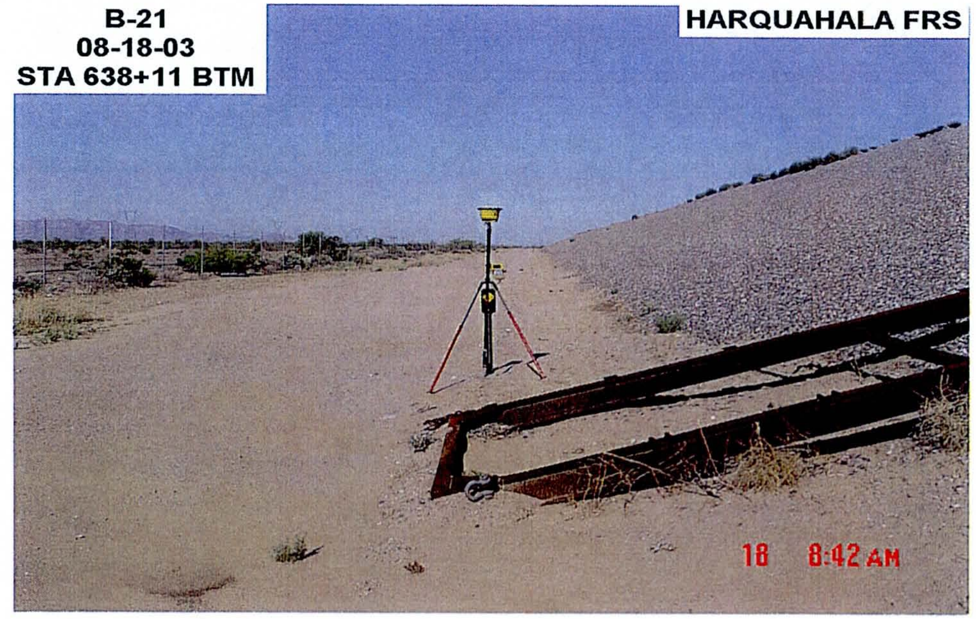
Based on the Datum Shift, the elevations at the Benchmarks equal the NGVD 1929 elevations plus the datum shifts shown in **Table 9.** The highlighted elevation values in the "Adjusted" columns of the tables reflect this calculation.

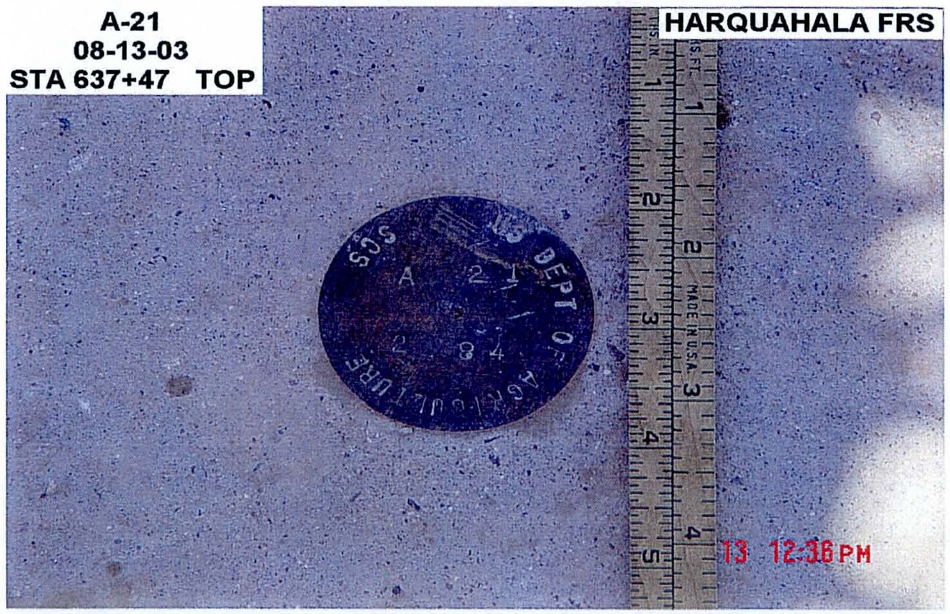
Marker	Description	1984 (NGVD29)	1986 (NGVD29)	1991 (NGVD29)	2002 (NAVD88)	2003 (NAVD88)	2002-1984 Dat. Shift	2003-1984 Dat. Shift	2002-1986 Dat. Shift	2003-1986 Dat. Shift
BURNT	2" Brass Cap - USGS	1442.092	1441.992	1441.992	1443.999	1444.009	1.9070	1.917	2.007	2.017
GREGG	2" Brass Cap - USGS	1433.652	1433.362	1433.362	1435.492	1435.506	1.8400	1.854	2.130	2.144
EBM1	3" Brass Cap - FCDMC	N/A	N/A	N/A	1621.644	1621.644				
EBM2	3" Brass Cap - FCDMC	N/A	N/A	N/A	1613.625	1613.613				
EBM3	3" Brass Cap - FCDMC	N/A	N/A	N/A	1618.888	1618.877				
4DP1	GDACS CONTROL	N/A	N/A	N/A	1210.965	1210.963				
4DR2	GDACS CONTROL	N/A	N/A	N/A	1170.561	1170.562				
4EQ2	GDACS CONTROL	N/A	N/A	N/A	1245.072	1245.073				
4ES2	GDACS CONTROL	N/A	N/A	N/A	1286.788	1286.788				
4FR2	GDACS CONTROL	N/A	N/A	N/A	1394.485	1394.485				
						Average 1984 shift =	**1.879**			
								Average '86 and '91 shift =	**2.075**	

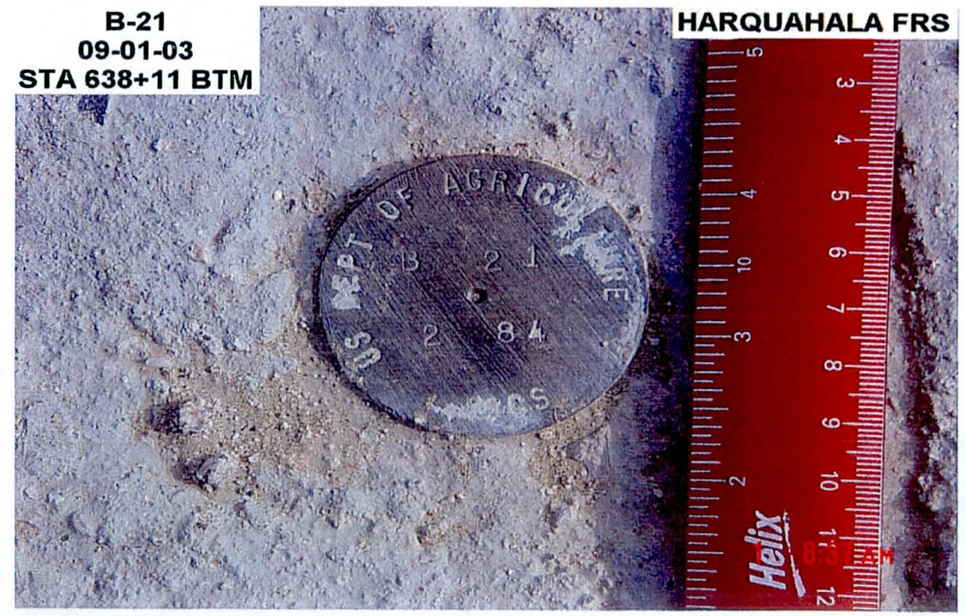
Notes: Average datum shift 2002 and 2003 to 1984 = 1.879' (Value used to adjust 1984 elevations to 1988 Datum)
Average datum shift 2002 and 2003 to 1986 and 1991 = 2.075' (Value used to adjust 1986 & 1991 elevations to 1988 Datum)

Table 9
Summary of Reference Marks









<u>Figure 8-1</u> - 2003 Photos of Crest A-21 and Toe B-21 Subsidence Caps

Floodplain View

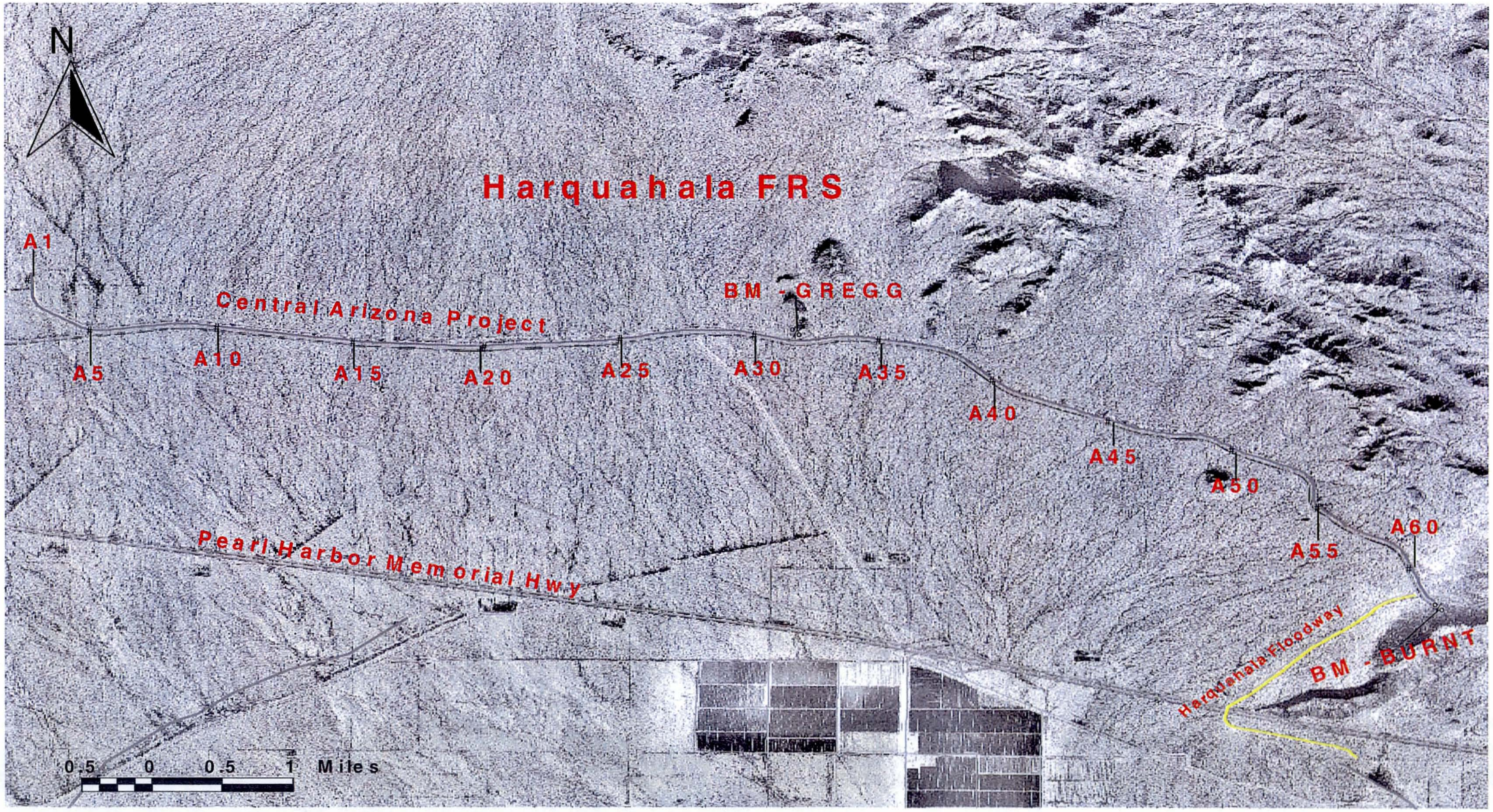


Figure 8-2 - Harquahala FRS Dam

PRELIMINARY FAILURE MODES IDENTIFICATION REPORT
HARQUAHALA 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 Harquahala 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

Harquahala FRS is an earthfill dam has a crest length of 62,308 ft, a crest width of 14 ft, upstream slope of 3:1 and downstream slope of 2:1 and has a maximum height of 38.0 ft. The emergency spillway is a reinforced concrete baffle-block chute spillway with a downstream rip-rap stilling basin. The ungated outlet consists of a multilevel, reinforced concrete tower at the upstream toe of the dam and a 48-inch reinforced concrete pipe conduit constructed through the dam near the left abutment. The structure and impoundment was designed not to have a permanent storage pool. The Central Arizona Project canal is located immediately downstream of the FRS embankment.

3.0 Summary of Inspection Reports

Flood Control District inspection reports dating from 1998 to 2004 were collected and reviewed. The January 2000 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 for the western portion of the dam.

A record of impoundment prepared by the District includes both dates and depths of impoundments for the period from 1994 to 2003. The maximum gage depth of impoundment was 21.5-ft in 2001. Gravel mulch was applied in 2003 at the far western end of the dam for control of erosion rills. Gravel mulch application for the remaining portions of the dam is pending evaluation of this approach during the Failure Mode and Effects Analysis (FMEA) (to be conducted under a subsequent task for the Structure Assessment). The inspection reports document erosion rills and gullies of various sizes along both the upstream and downstream slopes particularly from Station 450+00 to 591+75.

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 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. 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.

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. The following are important issues that require review before the formal FMEA.
   a. Qualitatively assess downstream effects due to discharge from the emergency spillway (existing and future downstream land use conditions).
   b. Qualitatively assess whether or not there would be an emergency spillway discharge during the 100-year event.
   c. Evaluate to the extent practical, the magnitude or frequency of storms that would result in spillway discharge. Should a low-flow notch be constructed in spillway crest to provide warning of potential spillway discharge?
   d. What is the EAP mapping?
   e. How fast does the reservoir fill and drain? Should District EAP trigger levels be re-evaluated?
   f. Effects/relation of CAP canal and potential canal overtopping of spillway flows and canal breach.
   g. Future considerations: Redirection of flows due to downstream development. Case history on other District dams are impacts of development changing downstream drainage patterns from existing conditions.

3. **Central Arizona Project Canal**: The CAP canal is located immediately downstream of the dam. The CAP canal has been constructed in some reaches in a cut (below existing grade) situation and in other reaches the CAP canal is in a fill (perched) condition. The following are important issues that require assessment before or during the formal FMEA.
   a. Assess the impacts of CAP on flow distribution from potential dambreach.
   b. Assess the effects of the CAP tunnel (located downstream of left abutment of dam) failure; blocking of flows into tunnel and backwater effects.
   c. Failure from impacts of CAP gate system at tunnel head.
   d. Plot canal bank profile and locate low points for incipient overtopping.

4. **Failure of Principal Outlet**: The principal outlet for the dam is a reinforced concrete pipe 48-inches in diameter. 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 the intake tower of the principal outlet to assess and document is the walls have deflected due to instabilities.
d. Review available geotechnical information to assess is the principal outlet is underlain by collapsible soils.
e. Seepage collars around principal spillway.
f. Visually inspect the intake tower for cracking.
g. Does drain fill wrap fully around pipe?
h. Is seepage out of joint possible?

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 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 a 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 Harquahala 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.

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. Structural Failure Modes

1. **Failure Mechanisms Associated with Emergency Spillway Structural Failure.**
    a. Inadequate or failure of under-slab drainage system.
    b. Loss of structural integrity of spillway approach slab, energy dissipater, spillway sidewalls, stilling basin, and/or wing walls.

E. 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 Harquahala 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

HARQUAHALA 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 January 31, 2005 and on February 1, 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 (01/31/2005)
Dennis Duffy, P.E., R.G.	Dam Safety Engineer, Flood Control District of Maricopa County (01/31/2005)
Dan Lawrence, P.E.	Dam Safety Engineer, Flood Control District of Maricopa County (02/01/2005)
Earl Pearcy	Operation and Management, Flood Control District of Maricopa County
Robert Eichinger, P.E., CFM	Project Manager, Kimley-Horn and Associates
Ken Euge, P.G.	Principal Geologist, Geological Consultants
Dean Durkey, Ph.D., P.E	Principal Geotechnical Engineer, Gannett-Fleming
Frances, Ackerman, E.I.T., R.G.	Geotech, Gannett-Fleming
David Jensen, P.E.	Engineer, Kimley-Horn and Associates (02/01/2005)
Kelli Blanchard, E.I.T.	Hydrologist, Kimley-Horn and Associates

Several inspection team members were only present for one of the two day inspection as noted.

Operational Summary

Inspection Frequency: Harquahala 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 Harquahala FRS. The District conducts quarterly operation and maintenance inspections.

Maximum Water Surface Elevations: The maximum recorded impoundment for Harquahala FRS was during the October thru November 2000 time period. The impoundment was recorded at 21.47-feet which is approximately 14-feet below the emergency spillway crest elevation of 1408.4 ft (NGVD29 datum).

Emergency Spillway Discharge: Based on District records, there has been no recorded discharges from the emergency spillway at Harquahala FRS. The emergency spillway is a concrete-lined chute spillway with seven rows of baffle block energy dissipaters. The emergency spillway is located between Stations 9+92 and 10+66 along the centerline of the structure. The width of the emergency spillway is 150 feet.

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 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 Phase I Structures Assessment;
- Replace survey monument A28 at Station 470+56;
- Upstream slope has become over steepened due to erosion. Repair with compacted fill to match as-built slopes (see inspection report for station limits);
- Clear silt from central filter drain outlet conduits;
- Level crest (add 0.2-feet) with AB material in vicinity of Station 999+17.

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

District Operation and Maintenance Responsibilities: The District maintains operational control of the Harquahala 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 the preparation and implementation of the individual emergency action plan.

Field Examination Results Summary

Embankment Crest: The crest of the dam is gravel plated. All crest settlement monuments located on the crest were located. Survey monument A28 has been damaged and should be replaced. There are station markers on the dam. Currently station markers are labeled on one side; they should be labeled on both sides of the sign post. 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). Some previously reported cracks could not be located while some other new cracks were observed and noted.

Abutments: There is a distinction between the two abutments due to the foundation. The left abutment is founded and contacted against bed rock. The right abutment is founded on fill. 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 adverse vegetation.

Upstream Slope: Small animal burrows were scattered on the slope face. There was no evidence of seepage, undermining, settlement or sloughing. There are several large erosion gullies located between Stations 450+00 and 579+74 and between Stations 600+70 and 656+00. Recommendations for gravel mulching of the upstream slope will be provided during the Phase I Structures Assessment currently being completed by Kimley-Horn. The upstream slope has become over steepened due to erosion between Stations 638+94 and 639+93. Longitudinal cracks/transverse cracks, depressions, and holes were observed on the slope face (see inspection report for specific locations).

On the east end of the structure, it appears that overburden material was placed on top of the compacted embankment that was not fully compacted. This may have been done to protect the embankment from erosion. Depressions in this material are evident, but do not affect the integrity of the embankment.

There is no low flow channel along the east end of the structure that keeps low flow runoff away from the upstream toe of the embankment. The maintenance follows the natural topography in many areas. Consequently, runoff ponds up against the toe of the embankment at the wash crossings and is eliminated only through seepage or evaporation. As there is no record of collapsible soils, this should not adversely impact the foundation.

The vegetation maintenance program of the District should continue along the structure. Some vegetation is currently over five feet tall.

Downstream Slope: Small animal burrows were scattered on the slope face. There was no evidence of seepage, settlement or sloughing. Slope was prepared for gravel mulch installation between Stations 762+00 and 637+70 but mulch was not installed. Gravel mulch is currently installed between Stations 637+70 and 443+80. Longitudinal cracks/transverse cracks, depressions, and holes were observed on the slope face (see inspection report for specific locations).

Sediment accumulation was noted in many of the central filter outlet drain conduits like in previous investigations. Sediment buildup in the drain outlet conduits should be removed.

It appears that the toe has been cut back by as much as 18 feet between Stations 646+50-724+50. A survey is recommended to compare the current downstream embankment profile and the as-built plans of the structure.

Principal Spillway and Reservoir: The approach channel is clear of debris and obstructions. The exterior of the inlet structure was clean. The concrete for the inlet structure showed no signs of structural distress. There were several minor shrinkage cracks on the exterior portion of the inlet structure. The trash rack was clear of debris and obstructions. The interior of the principal spillway conduit was inspected visually by deflecting sunlight by mirror into the conduit barrel. The conduit was clean and there were no apparent signs of seepage. It is recommended that the outlet conduit be videotaped for subsequent office review.

The discharge outlet structure of the principal spillway was clear of debris. The joints of the outlet structure were straight and appeared tight. The slope north of the principal spillway outlet structure has recently been repaired. There were no signs of seepage.

The 24-inch diameter Drain Outlet and the New Tank Outlet should be video inspected on a regular basis along with the principal spillway conduit.

The toe drain outlet located at the principal spillway outlet structure was located. The service manhole for the toe drain outlet system was located but not inspected. The manhole lid was bolted. The toe drain outlet system should be video inspected to the extent possible.

Emergency Spillway: The emergency spillway is located between stations 9+92 and 10+66 of the dam. The emergency spillway is a 150-foot wide broad crested concrete chute. The spillway is clear of any obstructions and debris. The emergency spillway control structure shows minor cracking, no repairs are required at this time. The joint material has aged and is separating and cracking and should be replaced. The emergency spillway is concrete lined to outfall with baffle block energy dissipaters with no stilling basin. Discharge from the emergency spillways flows into the Granite Reef Aqueduct (GRA) which could cause the GRA to overtop, allowing flow to reach I-10.

Instrumentation: The settlement monuments for Harquahala FRS are located on the 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 A28 at Station 470+56 has been damaged and should be replaced. There are station markers on the Harquahala 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 Harquahala FRS.

There are seven staff gages on the dam at the principal outlet. Two of the staff gages are located on the outlet tower. The other five are separate posts mounted to the upstream slope. Staff gages are used to indicate the level of water impounded in the reservoir. A pressure transducer is also located at the principal outlet.

There is no staff gage or ALERT station at the western end of the dam.

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 relative to Harquahala FRS.

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 relative to Harquahala FRS. An EAP for Harquahala FRS needs to be prepared and developed to meet the minimum guidelines from ADWR and FEMA.

Conclusions

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

Recommendations from Inspection

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

a. Repair the damaged survey monument A28;
b. Continue active vegetation management program;
c. Remove sediment and any obstructions in the central filter drain outlet conduits;
d. Complete a survey to compare the downstream embankment profile to the as-built plans as it appears the toe may have been cut back;
e. Add watershed instrumentation (stream gages and rain gages in the upper watershed);
f. Develop an Emergency Action Plan to meet FEMA 64 and ADWR requirements
g. Conduct a video inspection of central filter drain outlet conduits;

h. Conduct a video inspection of the toe drain outlet system at the principal spillway.
i. Add a staff gage and/or ALERT station at the west end of the dam.
j. Repair Toe Drain

Next Annual Inspection

The next annual inspection is scheduled for November 2005.

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.: **07.53** FCDMC NO.: **330**	DAM NAME: **Harquahala FRS**	TYPE: **Earthfill**
CONTACTS: **Brett Howey – Flood Control 506-1501** **ADWR (Invited – declined attendance)** **NRCS (Invited – declined attendance)**	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), Frances Ackerman (Gannett-Fleming), Tom Renkly (FCD), Brett Howey (FCD), Dennis Duffy (FCD), Dan Lawrence (FCD) , Earl Pearcy (FCD O&M), Kelli Blanchard (Kimley-Horn)**	INSPECTION DATES: **January 31 and February 1, 2005**	
REVIEWED BY: **Bob Eichinger, P.E., CFM**	DATE: **June 8, 2005**	PAGE: **1 of 13**
SPILLWAY DESIGN CREST ELEVATION: **1408.4 ft (NGVD 29)**	HAZARD CLASS: **Significant**	SIZE: **Intermediate**
INFLOW DESIGN FLOOD: **½ PMF**	SPILLWAY CREST WIDTH: **150 ft.**	ADWR DAM HEIGHT: **38 ft.**
DAM CREST LENGTH: **62,308 ft.**	DAM CREST WIDTH: **14 ft.** Note: Crest elevation data from November 2004 Inspection report	CREST ELEV.: **Varies (datum NGVD 29)** **Sta. 433+50 to 540+00: 1422.2** **Sta. 540+00 to 710+00: 1422.1** **Sta. 717+00 to 742+00: 1420.6** **Sta. 752+00 to 810+00: 1420.5** **Sta. 820+00 to 910+00: slopes 1420.45 to 1420.0** **Sta. 930+00 to 1045+08: 1419.7**
CURRENT RESERVOIR LEVEL: **Empty**	TOTAL DESIGN FREEBOARD: **11.3 ft.**	PHOTOS: **Yes**

Item / Comments	NOT APPLIC.	NO	YES	MONITOR	REPAIR	INVESTIGATE
1. CREST **Maximum dam height is 49.3 ft. and the crest elevation is variable 1419.7 ft. (NGVD 29) near the left abutment to 1422.2 ft. near the right abutment**						
a. Settlements, slides, depressions **Depressions/holes noted at stations 433+60, 434+16519+90. Piping holes notes at station 492+60 to 492+87, 493+13 and 493+51. See list at end of report for descriptions of the holes at the various locations.**			✓		✓	
b. Misalignment?		✓				
c. Longitudinal/Transverse cracking? **See list at the end of the report for specific locations. No repairs recommended at this time.**			✓	✓		
d. Animal burrows? **Evidence of many inactive burrows from stations 400 through 600 along upstream and downstream shoulders, periodic active areas. Continue rodent control following standard procedures.**			✓		✓	
e. Adverse Vegetation? **Minor occurrences along upstream and downstream shoulders. Continue adverse vegetation control following standard procedures.**			✓		✓	
f. Erosion?		✓				
2. UPSTREAM SLOPE **3H:1V**						
a. Erosion? • **Rilling scattered along length of dam, particularly heavy and deep from stations 400 through 600.** • **Upstream slope has become over steepened due to erosion. Repair with compacted fill to match as-built slopes. See list at the end of the report for specific locations. .**			✓		✓	
b. Inadequate ground cover?		✓				
c. Adverse vegetation? **Scattered throughout. Continue adverse vegetation control following standard procedures.**			✓		✓	
d. Longitudinal/Transverse cracking? **See list at the end of the report for specific locations. No repairs recommended at this time.**		✓				
e. Inadequate riprap?			✓		✓	
f. Stone deterioration?		✓				

Item	Comments	N/A	NO	YES	MON	REP	INV
g. Settlements, slides, depressions, bulges?	**A toe depression at Sta. 541+84 is one of several low areas at toe of the upstream slope that ponds water at the slope. A slope depression noted from sta.480 to 650. Depression from sta. 900 to 1000 depressions noted. Generally along the east portion of Reach 1 the upstream slope appears to be benched or overbuilt without final grading.**			✓	✓		
h. Animal burrows?	**Evidence of many inactive burrows with periodic active areas from stations 400 through 700. Noted active burrow at station 1021+00. Continue rodent control following standard procedures.**			✓		✓	
3. DOWNSTREAM SLOPE **2H:1V**							
a. Erosion?	**Some rilling scattered throughout**			✓		✓	
b. Inadequate ground cover?	**Shows uneven grading or benching from sta. 760 to 790.**		✓				
c. Adverse vegetation?	**Scattered throughout. Continue adverse vegetation control following standard procedures.**			✓		✓	
d. Longitudinal/Transverse cracking?	**See list at the end of the report for specific locations. No repairs recommended at this time.**			✓	✓		
e. Animal burrows?	**Evidence of many inactive burrows from stations 400 through 800, periodic active areas. Continue rodent control following standard procedures.**			✓		✓	
f. Settlements, slides, depressions, bulges?	**Various depressions see list at the end of the report for specific locations and details. A possible 15-foot wide slump at sta. 766+30.**			✓	✓	✓	
g. Soft spots or boggy areas?	**Reservoir dry during inspection. There was some local ponding agains the upstream toe of the embankment. There is no low flow channel on the eastern end of the embankment and the access road is not cut into native ground, so there are areas of shallow ponding against the downstream toe.**		✓				
h. Movement at or beyond toe?			✓				
4. DRAINAGE-SEEPAGE CONTROL	**A 5 ft. wide filter/drain was installed during dam construction. The upstream filter section is 3 ft. wide and the downstream drain section is 2 ft. wide. The drain extends from Sta. 450+00 to 717+00 with the top 7 ft. below the dam crest and elevation varying with the variable dam crest elevation. Drain outlets are located at 400 ft. intervals between Sta. 452+00 and 716+00.**						
a. Internal drains flowing?	**Reservoir dry during inspection.**		✓				
b. Boils at or beyond toe?	**Reservoir dry during inspection.**		✓				
c. Seepage at or beyond toe?	**Reservoir dry during inspection.**		✓				
d. Does seepage contain fines?	**Some drain outlets silted in**	✓					
5. ABUTMENT CONTACTS							
a. Erosion?			✓				
b. Differential movement?			✓				
c. Cracks?			✓				
d. Settlements, slides, depressions, bulges?			✓				
e. Seepage?	**Reservoir dry during inspection.**		✓				
f. Animal burrows?			✓				
6. IRRIGATION OUTLET - INLET STRUCTURES							
a. Seepage into structure?	**Reservoir dry during inspection with no indications of seepage noted.**		✓				
b. Debris or obstructions?			✓				
[illegible]. If concrete, do surfaces show:							
1. Spalling or Scaling?			✓				
2. Cracking?			✓				
3. Erosion?			✓				

HARQUAHALA FRS DAM INSPECTION REPORT	PAGE **3**of **13**	ADWR NO.: **07.53** FCDMC NO.: **330**						
INSPECTED BY: **Kimley-Horn Inspection Team**		DATE: **January 31 and February 1, 2005**	N/A	NO	YES	MON	REP	INV
Item	Comments							
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. Irrigation Outlet Gate(s): **Slide gates (24-inch)**								
1. Broken or bent?				✓				
2. Corroded or rusted?				✓				
3. Leaking? Unknown				✓				
4. Not seated properly?				✓				
5. Not operational?				✓				
6. Not periodically maintained?				✓				
7. Date last operated? **Operated quarterly**								
7. IRRIGATION OUTLET CONDUITS: **The Drain Outlet (located at Sta. 746+00 that runs below the CAP) and New Tank Outlet (located at Sta. 583+80 that runs above the CAP: steel portion over the CAP). Both are 24-inch RCP and protected with anti-seep collars.** **(NOTE: June 8 2005: Video inspections have been performed and the video is being reviewed by the District).**								
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? **Minor cracking (non-structural) – no repairs required**								
3. Erosion?								
d. If metal, do surfaces show:								
1. Corrosion?			✓					
2. Protective coating deficient?			✓					
3. Misalignment or spilt seams?			✓					
e. Do the joints show:								

Item / Comments	N/A	NO	YES	MON	REP	INV
1. Displacement or offset?						
2. Loss of joint material?						
3. Leakage? **Reservoir dry during inspection – No indications of seepage**		✓				
8. PRINCIPAL SPILLWAY – APPROACH CHANNEL **Unlined channel**						
a. Eroding or back cutting?		✓				
b. Sloughing?		✓				
c. Restricted by vegetation?		✓				
d. Obstructed with debris?		✓				
e. Silted in?		✓				
9. PRINCIPAL SPILLWAY - INLET STRUCTURE **Reinforced concrete "T" structure with a crest elevation of 1387.3 ft. (NGVD 29) and 18-inch slide gate with invert at elevation 1373.3 ft.**						
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? **Minor cracking (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**		✓				
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? **Reservoir empty – No indications of seepage**		✓				
4. Not seated properly?		✓				

Item / Comments	N/A	NO	YES	MON	REP	INV
5. Not operational?		✓				
6. Not periodically maintained?		✓				
7. Date last operated? **Operated quarterly**						
10. PRINCIPAL SPILLWAY CONDUIT: **48-inch diameter RCP protected with anti-seep collars.** **NOTE: see discussion in section 7 regarding video inspection.**						
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?						
3. Erosion?						
d. If metal, do surfaces show:						
1. Corrosion?	✓					
2. Protective coating deficient?	✓					
3. Misalignment or spilt seams?	✓					
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? **Reservoir empty – No indications of seepage**		✓				
b. Debris present?		✓				
c. If concrete, do surfaces show:						
1. Spalling or scaling?		✓				
2. Cracking? **Minor cracking (non-structural) – no repairs required**			✓	✓		
3. Erosion?		✓				
4. Exposed reinforcement?		✓				
5. Other?	✓					
12. PRINCIPAL SPILLWAY - STILLING BASIN/POOL						
a. If concrete, do surfaces show:						
1. Spalling or Scaling?		✓				
Cracking? **Minor cracking (non-structural) – no repairs required**			✓	✓		
3. Erosion?		✓				
4. Exposed reinforcement?		✓				
b. If concrete, do joints show:						

Item / Comments	N/A	NO	YES	MON	REP	INV
1. Displacement?		✓				
2. Loss of joint material?		✓				
3. 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						
a. Eroding or back cutting? **The slope north of the principal spillway outlet structure has recently been repaired. Gravel mulch and/or hydro seed should be completed to protect the freshly graded slope.**			✓		✓	
b. Sloughing?		✓				
c. Obstructed?		✓				
d. Poorly rip rapped?		✓				
e. Tailwater elevation and flow condition: **Unknown**						
14. EMERGENCY SPILLWAY-APPROACH CHANNEL **40 ft. riprap section upstream of the 150 ft. wide broad crested concrete weir**						
a. Eroding or back cutting?		✓				
b. Sloughing?		✓				
c. Restricted by vegetation?		✓				
d. Obstructed with debris?		✓				
e. Silted in?		✓				
15. EMERGENCY SPILLWAY-CONTROL STRUCTURE **Concrete broad crested weir 150 ft. wide**						
a. If concrete, do surfaces show:						
1. Spalling or scaling?		✓				
2. Cracking? **Minor cracking (non-structural) – no repairs required**			✓	✓		
3. Erosion?		✓				
4. Exposed reinforcement?		✓				
b. If concrete, do joints show:						
1. Displacement or offset?		✓				
2. Loss of joint material? **Replace joint material that has aged and is separating and cracking.**			✓		✓	
3. Leakage?		✓				
c. If spillway is unlined:						
1. Are slopes eroding?	✓					
2. Are slopes sloughing?	✓					
3. Is crest eroding?	✓					
d. Is weir in poor condition?		✓				

Item / Comments	N/A	NO	YES	MON	REP	INV
e. Where is control structure? **Centerline of dam crest**						
16. EMERGENCY SPILLWAY – CHANNEL **Concrete lined to outfall with baffle block energy dissipaters. The emergency spillway does not have a stilling basin.**						
a. Obstructions or restrictions?		✓				
b. If concrete, do surfaces show:						
1. Spalling or scaling?		✓				
2. Cracking? **Minor cracking (non-structural) – no repairs required**			✓	✓		
3. Erosion?		✓				
4. Exposed reinforcement?		✓				
c. If concrete, do joints show:						
1. Displacement or offset?		✓				
2. Loss of joint material? **Replace joint material that has aged and is separating and cracking.**			✓		✓	
3. Leakage? **Reservoir dry during inspection.**		✓				
d. If an unlined channel, does it show:						
1. Erosion?	✓					
2. Slopes sloughing?	✓					
3. Poorly protected w/ vegetation/riprap?	✓					
17. EMERGENCY SPILLWAY-TERMINAL STRUCTURE **None Present**						
a. If concrete, do surfaces show:						
1. Spalling or scaling?	✓					
2. Cracking?	✓					
3. Erosion?	✓					
4. Exposed reinforcement?	✓					
b. If concrete, do joints show:						
1. Displacement or offset?	✓					
2. Loss of joint material?	✓					
3. Leakage?	✓					
c. Do the energy dissipaters show:						
1. Signs of deterioration?	✓					
2. Covered with debris?	✓					
3. Signs of inadequacy?	✓					
18. EMERGENCY SPILLWAY – OUTLET CHANNEL **Riprap lined**						
a. Eroding or back cutting?		✓				
b. Sloughing?		✓				
c. Obstructed or restricted?		✓				
19. RESERVOIR						

INSPECTED BY: **Kimley-Horn Inspection Team** | DATE: **January 31 and February 1, 2005**

Item / Comments	N/A	NO	YES	MON	REP	INV
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? **Reservoir empty**		✓				
f. Low ridges/saddles allowing overflow?		✓				
g. Structures below dam crest elevation?		✓				
20. INSTRUMENTATION						
a. List type(s) of instrumentation: **ALERT gage and settlement survey markers**						
b. Any repair or replacement required? **Monument A28 at Sta. 470+56 has been damaged.**			✓		✓	
c. Last monitoring report: **Surveys completed in 2002 and 2003. See additional comments.**						
21. CONDITION SUMMARY / EAP / MAINTENANCE RECOMMENDATIONS / NEXT INSPECTION						
a. Any safety deficiencies?		✓				
b. Safe storage level on License: **Principal spillway crest elevation 1387.3 ft. (NGVD 29)**						
c. Date of current ADWR License: **April 7, 2000**						
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: **Intermediate** Should size be revised? **No**		✓				
f. Recorded downstream hazard: **Significant** Should hazard be revised? **No. The Phase I Structures Assessment is scheduled to be completed May 2005. A task in the Phase I include 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 21.d.**			✓		✓	
h. Normal inspection frequency: Should inspection frequency be revised? **Annual**		✓				

i Maintenance Recommendations:

(1) Fill erosion rills upstream and downstream with compacted fill if greater than 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) Replace survey monument A28 at Sta. 470+56.
(5) Repair erosion on D/S shoulder impacting crest road at Station 541+84
(6) Removal of beehive at central filter drain outlet conduit required at Station 599+80
(7) Clear sediment from all central filter drain outlet conduits
(8) Repair large rodent hole on D/S slope, 2-feet below crest at Station 577+75. Possible badger hole.
(9) Repair slopes as detailed in 2.a.
(10) Continue control of adverse vegetation
(11) Level crest (add 0.2-ft.) with AB material in vicinity of Sta. 999+17
(12) Gravel mulch and/or hydro seed freshly graded slope around principal spillway outlet structure
(13) D/S toe appears to be cut in at many locations by as much as 18 feet between Stations 646+50-724+50. Check or survey existing embankment profile against as-builts at this location.

Item / Comments	N/A	NO	YES	MON	REP	INV
j. Is Supplemental Inspection required? Recommend flood event inspections			✓			
k. Recommended date for next inspection: **November 2005**						
Status of Structure Assessment Program: **In progress 1/31/2005.**						

ADDITIONAL COMMENTS:

2005 **Specific Location Details of Significant Conditions**

Station	Locati	Type	Date	2005 vs.	January 31 and February 1, 2005 Inspection Results

	on		First Reported	First Report	
440+35	D/S slope	Trans.	2001	No change	Possible transverse crack on downstream slope.
440+54	D/S slope	Trans.	2002	No Change	Possible transverse crack on downstream slope
440+95	U/S slope	Trans.	2005		Possible transverse crack on upstream slope, 3- to 4-feet below crest, 3-inches in diameter. Probe depth to 18-inches, probed hole horizontally 3-feet
441+00	D/S slope	Trans.	2002	Did not find	Possible transverse crack on downstream slope, probe inserted 20-inch.
441+92	U/S slope	Hole	2005		Rodent holes.
441+92	D/S slope	Trans.	2002	Did not find	Possible transverse crack on downstream slope
443+11	U/S slope	Erosion gulley	2005		Erosion gullies, 2- to 3-inches deep
443+68	D/S slope	Trans.	2004	Did not find	Possible transverse crack on downstream slope
446+67	U/S slope	Trans.	2005		Possible transverse crack on upstream slope.
447+00	U/S slope	Slope break	2005		Gentle gradient slope below crest with steeper slope below with rill erosion.
449+50	U/S slope	Erosion channel	2005		Erosion channel into low flow channel at edge of upstream slope access road.
450+00	U/S slope	Erosion gulley	2005		Erosion gulley. Picture is a panorama of typical erosion gullies in this section of the dam.
466+44	U/S slope	Erosion gulley	2005		Erosion gullies typical of this upstream slope section of the dam.
467+43 and 467+75	U/S slope	Erosion gulley	2005		Erosion gullies approximately 20-inches deep.
469+58	D/S crest	Hole	2005		2.5-feet deep hole off of downstream shoulder.
469+50	D/S slope	Slope depression	2005		Depression on D/S slope 2 feet below crest. Probed to depth 2 feet.
469+52	D/S slope	Slope depression	2005		Depression on D/S slope. Probed to depth 2.5 feet. Some rodent activity in this area.
475+85	U/S slope	Slope depression	2005		Depression on U/S slope 3 feet below crest. Probed to depth 2 feet.
478+76	U/S slope	Slope depression	2005		4 feet wide slightly depressed area on U/S slope. Probed to depth 3 feet.
484+85	U/S slope	Slope depression	2005		Slight depression in upstream slope below crest.
486+00	U/S slope	Borrow Area	2005		Head cut erosion channel into borrow excavation (stratifies gravelly silty caliche filaments and fine disseminated caliche).
487+39 to 491+13	Crest	Long.	2002	Change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline and only observed between Sta. 490+58 to 491+13. Few locations in 2003 where probe could be inserted to 30-inch. In Jan. 2004 three 1-inch diameter holes probed 12-inch at Sta. 490+79. In Nov. 2004 more obscure
490+92	Crest	Long	2005		At Station 490+92, two holes along centerline of crest, possible longitudinal crack. (Photo 7)
490+92	U/S slope	Erosion gulley	2005		Rill erosion gullies.(Photo 9)

491+97 to 495+21	Crest	Long.	2002	No Change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline, except at station 492+74 where probe was inserted to handle (34-inch). Few locations in 2003 where probe could be inserted to 30-inch depth. In 2004 a 3-inch diameter hole probed 34-inch at Sta. 492+55 and 1-inch diameter hole was probed 24-inch at Sta. 493+00.
492+00	U/S slope	Slope depression	2005		Low swale below crest.
492+60 to 492+87	Crest	Piping holes	2005		Array of seven possible erosion piping holes in crest.
493+21	D/S slope	Hole	2005		Small hole on crest at CL. Probed to depth of 3-feet.
493+12	Crest	Piping holes	2005		Pipe hole 0.6-feet by 0.5-feet; probed more than 20-inches at centerline of dam.
493+51	Crest	Piping holes	2005		¼-inch diameter hole in crest, probed over 3-feet.
498+54	D/S slope	Depression	2005		Depression on downstream slope, 3-feet below crest. Probed to depth of 2-feet
498+14 to 498+69	Crest	Long.	2002	Did not find	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline.
500+00	Crest	Long.	2002	Did not find	Longitudinal crack wanders along the crest centerline to +/- 2-to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted in holes along the crack to a depth of 18- to 30-inch.
500+78	D/S slope	Rodent hole	2005		Hole on d/s slope 3-feet below crest. Probed to depth of 1.5-feet.
501+33	D/S slope	Hole	2005		Large animal hole, possibly badger. Located near crest at d/s shoulder. Recommend repair.
505+96	Crest	Hole	2002		Found hole previously identified. Located on crest at CL. Probed to depth of 2'
506+99	Crest	Hole	2005		Two holes located at crest on centerline. One of the holes was probed to 3-feet depth.
516+50 to 518+00	U/S slope	Erosion gulley	2005		Erosion gulley, approximately 14-inches deep about 10-feet below upstream crest.
517+22	U/S slope	Erosion gulley	2005		Erosion gulley 14-inches deep about 10-feet below upstream crest.
517+99 to 518+64	Crest	Long.	2002	Did not find	Longitudinal hairline crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Documented as not found during the 2003 inspection.
522+89 to 523+03	Crest	Long.	2002	Did not find	Longitudinal hairline crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Small holes observed at Sta. 522+89, 522+93 and 523+03. Documented as not found during the 2003 inspection.
526+34	D/S toe	Holes	2005		Large inactive rodent hole located 5-feet above downstream toe.
527+42	U/S slope	Erosion gulley	2005		Erosion gulley rill 15- to 20-inches deep; may be related to erosion of gulley previously filled by slope repair and seeding. (Photo 10)
550+32	D/S slope	Depression	2005		Shallow depression on downstream slope, 3-feet below crest. Probed to depth of 2-feet.
554+02	U/S slope	Slope depression	2005		Shallow depression below crest of embankment.
562+09	U/S slope	Erosion gulley	2005		Erosion gulley extends from toe to top of upstream slope.
556+39	D/S slope	Holes	2005		Two rodent holes 2-feet below crest, probed to 2.5-feet. At 15' above DS toe, probed 2-feet.
561+85	D/S slope	Hole	2005		Large rodent hole on D/S shoulder. Probed to 3-feet.
574+77	U/S	Low flow	2005		Low berm across low flow channel that appears to interrupt flow.

Item	Comments	N/A	NO	YES	MON	REP	INV

	slope	channel			
577+75	D/S slope	Hole	2005		Large rodent hole 2-feet below crest. Probed to 2-feet. Possible badger hole. Recommend repair.
579+74	U/S slope	Erosion gulley	2005		Deep rill gullies in lower one-quarter of upstream slope.
584+44	U/S slope	Slope Repair	2004	No Change	Section of repaired US slope, repair complete October 2004.
589+07	U/S slope	Low flow channel	2005		Low berm across the low flow channel appears to interrupt flow.
592+58	U/S slope	Toe bulge	2005		Accumulate sluff at toe of slope about 2-feet thick with rill erosion 20-inches deep.
596+89	U/S slope	Slope depression	2005		Shallow depression 10-feet below crest of embankment.
600+70	U/S slope	Erosion gulley	2005		Prominent rill erosion in lower portion of upstream embankment slope.
601+78	U/S slope	Slope break	2005		Hummocky embankment slope from slope mid-height to toe.
607+23	U/S slope	Slope depression	2005		Erosional slope depression at mid-slope, fan soil debris at toe.
608+00	U/S slope	Slope repair	2005		Repaired section of upstream embankment slope with rill erosion gullies.
610+97	U/S slope	Slope depression	2005		4½-foot deep depression/break in slope in lower one-third of upstream slope.
618+63	U/S slope	Slope break	2005		Irregular upstream slope surface.
637+49	D/S slope	Mulch	2005		Note: Gravel mulch ends at this station
638+94 to 639+93	U/S slope	Over-steepened slope	2005		Over-steepened slope section with erosional scarp about 5-feet high.
641+24	U/S slope	Slope depression	2005		Erosional scarp on upstream slope about 8-feet below crest
642+99	U/S slope	Erosion gulley	2005		Large rill erosion gulley with erosional depression at upstream edge of crest.
643+49	U/S slope	Slope depression	2005		Erosional depression at upstream edge of crest.
643+76	U/S slope	Slope depression	2005		Erosional depression 10-feet below upstream edge of crest.
645+08	U/S slope	Erosion gulley	2005		Erosional depression with debris fan formed at toe of upstream embankment slope.
647+00	U/S slope	Erosion gulley w/ depression	2005		Gullied section of upstream slope 300-feet in length with erosional scarp near crest or mid-height of embankment slope.
651+00	U/S slope	Erosion gulley w/ depression	2005		Erosional scarp below upstream crest and mid-height of upstream embankment slope.
656+00	U/S slope	Erosion gulley w/ depression	2005		Erosional scarp about 10-feet high below the crest on the upstream portion of the embankment.
671+41	U/S slope	Slope break	2005		Change in slope angle (flatter) for 260-feet. Section is 6-feet below crest, may be due to construction staking.
687+22	U/S slope	Erosion scarp w/ gulley			Erosional scarp area drained by deep rill gully at mid-height of upstream slope.
715+72	U/S slope	Blocked low-flow channel	2005		Access ramp on upstream embankment slope. Ramp extends upstream to block low-flow channel interrupting flow.
729+00	U/S	Rodent holes	2005		Rodent holes 6- to 8-inches in diameter near toe of upstream

	slope				embankment slope.
738+55	U/S slope	Blocked low-flow channel	2005		Upstream access road on berm blocking low-flow channel.
756+00	D/S slope	Hole	2005		Depression/rodent hole midway up downstream slope.
762+50	U/S slope	Longitudinal Crack	2005		Possible longitudinal crack located approximately 15-feet above the upstream toe. Evidence continued west, possible sloughing.
764+00	U/S slope	Gullies	2005		Evidence of drainage/erosion in the form of small channels (approximately 1- to 2-feet wide and 6-inches deep) along upstream slope.
765+00	D/S slope	Grading	2005		Downstream slope shows some uneven grading (benching).
766+30	D/S slope	Slump	2005		Possible small (15-feet wide) slump at toe of downstream slope.
767+00	D/S slope	Hole	2005		Depression/rodent hole midway up downstream slope
767+00 to 780+00	U/S slope	Gullies	2005		Evidence of drainage/erosion in the form of small channels (approximately 1- to 2-feet wide and 6-inches deep) along upstream slope.
775+00 to 770+00	D/S slope	Depressions	2005		2-foot diameter depression located on downstream slope, several others in a longitudinally line.
770+50	D/S slope	Depression	2005		2-foot diameter depression located on downstream slope several others in a longitudinally line.
778+00	D/S slope	Holes	2005		Several small holes located on downstream slope.
780+00-					
787+50	U/S slope	Gullies	2005		Small animal trail or drainage from crest to toe of upstream slope. Depression about 15-feet up on the embankment. No evidence of crakcing
791+00	U/S slope	High water mark	2005		Evidence of high water approximately 12-feet up from upstream toe, debris and small logs.
794+50 to 790+00	D/S slope	Grading	2005		Downstream slope shows some uneven grading (benching).
796+50 to 809+50	D/S slope	Depression	2005		Several 2-foot diameter depressions, approximately 12-inches deep on downstream slope.
819+50	D/S slope	Depression	2005		2-foot diameter depression, approximately 12-inches deep on downstream slope.
820+20	D/S slope	Depression	2005		Series of depressions on downstream slope, appear to be connected.
824+50	D/S slope	Holes	2005		Several small (2-inch diameter) holes on lower downstream slope. Can probe 12- to 18-inches in holes.
829+00	D/S slope	Depression	2005		2-foot diameter depression, approximately 12-inches deep on downstream slope.
830+00	D/S slope	Grading	2005		Downstream slope appears somewhat over steepened, larger boulders have moved down slope.
830+80	D/S slope	Erosion	2005		Erosion on the downstream slope, possible crack.
834+00	D/S shoulder	Depression/hole	2005		Depression/Rodent hole on downstream shoulder.
834+82	D/S slope	Trans.	2004	No Change	Inactive burrow with probe insert 18-inch, possible transverse crack.
842+00	D/S shoulder	Hole	2005		Depression/rodent hole on downstream shoulder, soft in-fill. (Photo 13)
General Observation	U/S toe	Depressions	2005		Beginning west of Sta. 900+00 there were localized depressions between the upstream toe and the road where moisture accumulates. There is relatively thick vegetation in this area.

931+50	U/S toe	Depressions	2005		Large depression, 12-feet long, 2- to 3-feet wide, within 3-feet of upstream toe. No visible erosion.
971+00 and 971+50	Crest	Piezometer	2005		Piezometer indicated on station marker at shoulder of crest, piezometer not located, vegetation noted in center of crest.
996+00	U/S slope	Depressions	2005		Surface depressions (2) on upstream slope, 2-feet in diameter, inspection probe inserted 18-inches, another noted at Sta. 995+50.
1001+00	U/S slope	Pit/Excavation	2005		Pit or excavation observed on upstream slope near mid slope, 5-feet in diameter, 2- to 3-feet deep.
1021+00	U/S slope	Holes	2005		Recent rodent activity, several holes observed on upstream slope (mid slope) 3- to 6-inches in diameter, inspection probe inserted 1- to 1 ½-feet.
1042+00	U/S slope	Pipe	2005		2 ½-inch diameter galvanized pipe embedded in slope vertically, approximately 2 ½-feet tall, mid slope, unknown origin.
General Observation	U/S slope	Bench	2005		Generally along the eastern portion of Reach 1 the upstream slope appears to be benched or overbuilt and left without final grading. Outside edge of bench is soft, low density material, while the bench is dense and compacted. It is possible that this was done to prevent larger particles from rolling to the bottom of the slope.

(2) Review of 2002 & 2003 Subsidence Survey*

A review of the 2003 subsidence survey data (attached) indicates a low spot (-0.15 ft.) in the crest around Sta. 999+17. A recommendation has been made to level the crest in the vicinity of the subject station. Several toe survey points appear to be inconsistent (spikes) with previously recorded values. The raw survey data has been checked against the reduced data and there does not appear to be any data entry errors. The 2002 and 2003 data appears relatively consistent over the one year period. Maintenance activities between 1986 and 2002 may have disturbed several of the toe monuments resulting in inconsistent reading between 1986 and 2002. Historic data and its relationship to more recent data should be reassessed after the FY 05-06 survey has been performed. The Phase I team should assess the data and recommend explanations for the inconsistencies and any further actions.

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

Photo 1 – Principal Outlet: Generally wet/damp conditions, dense low growing vegetation. Outlet structure and operation features appeared to be in good working condition, no significant sediment was observed.

Photo 2 – Harquahala FRS Floodway: At principal outlet structure, looking downstream.

Photo 2a. Toe drain outlet at principal spillway outlet structure.

Photo 3 – Principal Outlet Structure and Energy Dissipator.

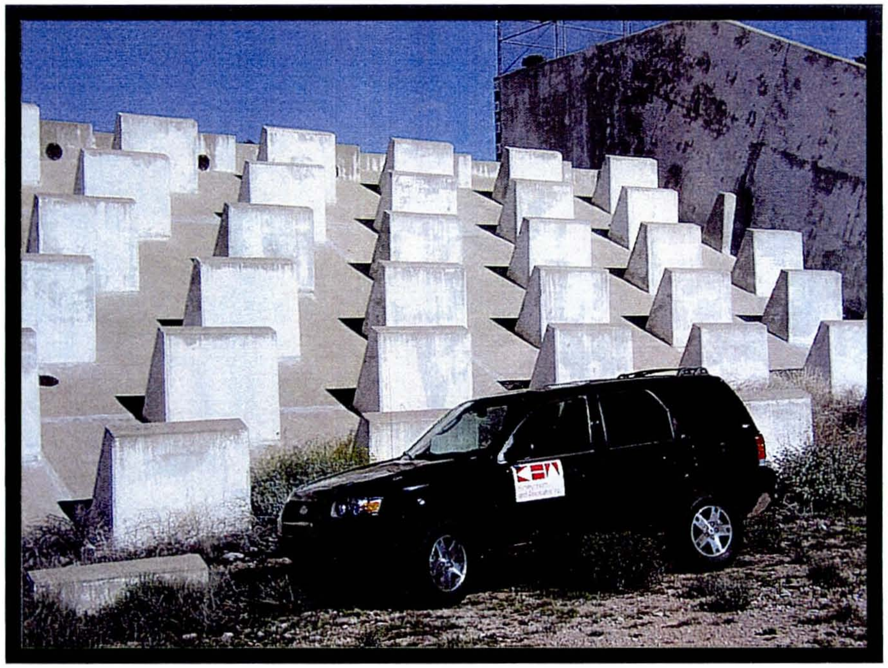


Photo 4 – Emergency Spillway (baffle block chute spillway).

Photo 5 – Drain Outlet at Station 746+00: Gate was closed.

Photo 6 – Outlet Drain at Station 467+72: Drain pipe is disconnected approximately 4-feet from downstream end otherwise appears clear. Recommend video inspection.

Photo 7 – Sta. 490+92: Two holes along centerline of crest, possible longitudinal crack.

Photo 8 – Reservoir Area, Reach 2. Note large vegetation on embankment

Photo 9 – Upstream Slope, Reach 2: Typical section of erosion gullies (2- to 3-inches deep).

Photo 10 – Upstream Slope, Reach 2: Erosion gulley 20-inches deep.

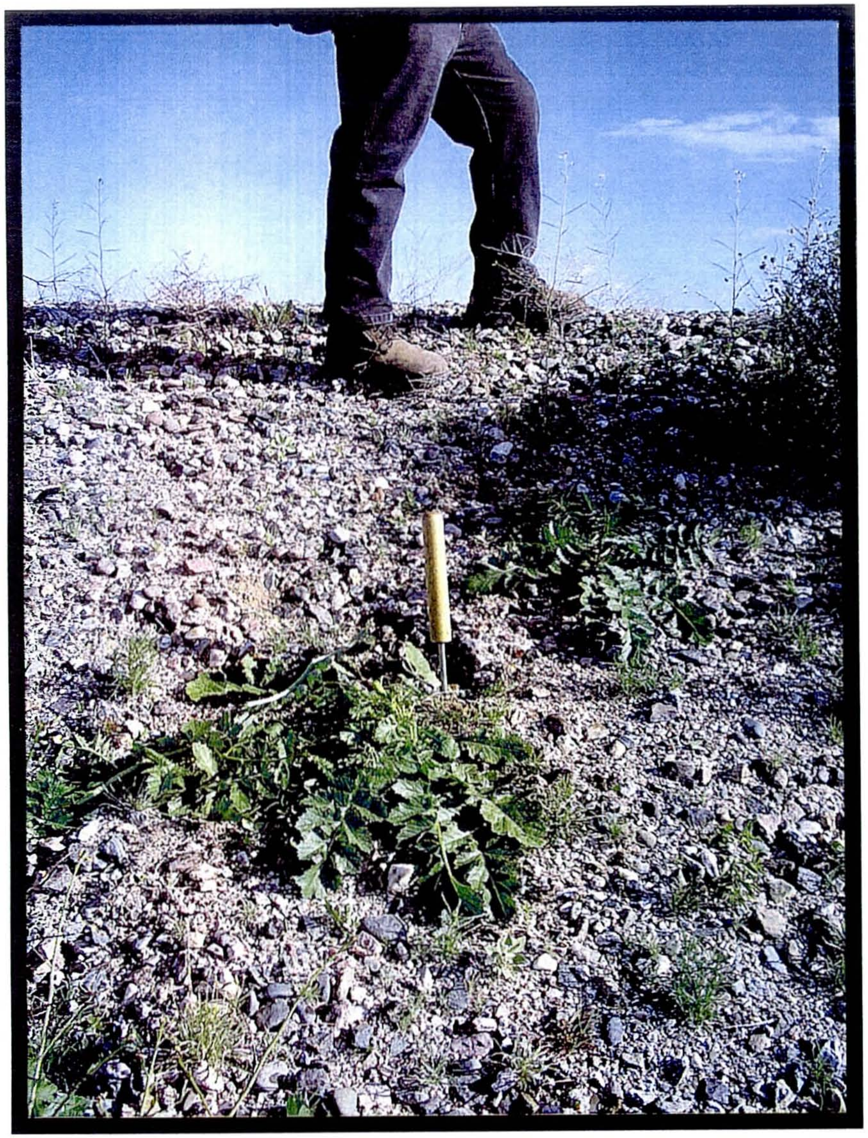


Photo 11 – Reach 2, Station 469+48: 2.5-feet deep hole off of downstream shoulder.

Photo 12 – Dam Crest, Reach 1: Piezometer indicated on station marker at shoulder of crest, piezometer not located. Vegetation noted in center of crest.

Photo 13 – Approximate Sta. 842+00: Depression/Rodent hole on downstream shoulder, soft in-fill.

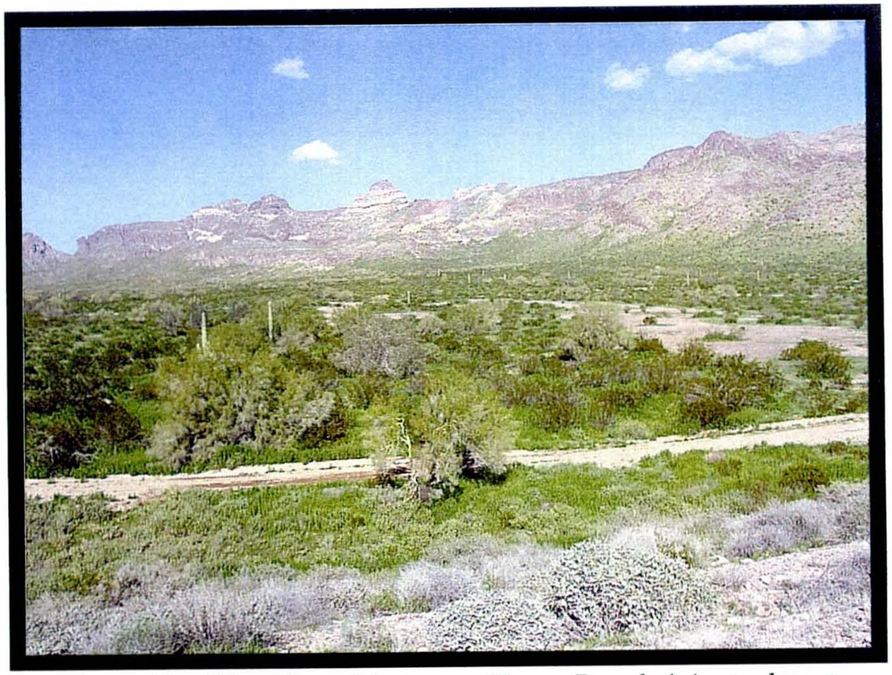


Photo 14 – View from Upstream Slope, Reach 1 (note desert pavement).

Photo 15 – East Abutment, Upstream Slope, Reach 1: Generally wet/damp conditions, dense low growing vegetation. The upstream slope appeared to be in good condition with no signs of cracking.

Photo 16 – Upstream Slope, Reach , Station 931+50: Large depression approximately 12-feet long, 2- to 3-feet wide, within 3-feet of upstream toe. No visible erosion.

Photo 17 – Downstream Slope, Reach 1.

Photo 18 – Downstream Slope, Reach 1, Station 965+00:
Downstream slope erosion probed 30-inches.

March 17,1998 Inspection Conducted by K.M. Hussain Reviewed by Jon Benoist
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
450+00 to 591+75	U/S slope	Erosion Repairs	1998	No Comment	The FCDMC staff regularly repairs the severe erosion gullies at both slopes on a phased program. Photo shows repairing of u/s slope between Sat. 450+00 and Sta. 591+75 during the last phase. The U/S slope after filling the erosion gullies was compacted with a D-8 dozer
			1998	No Comment	The HDPE drain pipes from the central gravel drain at the dam discharges into the Central Arizona Project Canal. The pipes are located at 1000-feet intervals.
			1998	No Comment	The inner rod of the operating wheel for the gated inlet is broken. The FCDMC staff was asked to fix the problem.
			1998	No Comment	The vehicular ramp at the immediate d/s of the emergency spillway is encroaching on the spillway channel width. The access road ramp must be pushed back.

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
591+75	U/S and D/S slope	Erosion Repairs	1999	No Comment	Erosion and riling on both the upstream and downstream slopes is evident for several miles east of Sta. 591+75.
975+00 to 980+00	D/S slope	Holes	1999	No Comment	On the downstream slope, may large holes were noticed with the largest concentration between Sta. 980+00 and 975+00.

January 31 & February 1, 2000 Inspection Conducted by Michael Greenslade (FCDMC) Reviewed by Jon Benoist
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
491+50 to 493+50	Crest	Long.	2000	No Comment	A longitudinal crack was traced over the central filter/drain from Sta. 491+50 to Sta. 493+50. This crack was observed intermittently between Sta. 494+70 to Sta. 541+00, and again at Sta. 670+00. At Sta. 493+50 the crack was probed to a depth of 2-feet
444+00 to 614+50	D/S slope	Gravel	2000	No Comment	Gravel mulch material has been placed from Sta. 444+00 to 614+50
846+92	Crest	Hole	2000	No Comment	At Sta. 846+92 a depression and small hole was probed and revealed an abandoned piezometer covered with a plastic bag buried under about 12-inch of soil. Material around the outside of the piezometer was loose and the piezometer was open. Another uncapped piezometer was observed flush with the crest at Sta. 1022+60 and a capped piezometer was observed at Sta. 1022+80. The piezometers appear to have been constructed of a very thin walled PVC. All of these piezometers should be located and properly abandoned. If needed for monitoring, new piezometers should be constructed to appropriate standards.

January 22-24 2001 Inspection Conducted by Larry Lambert (FCDMC)
Contacts: Mike Greenslade (ADWR) and Noller Hebert (NRCS)

Inspection Notes					
Station	Location	Type	Date First Reported	2001 FY vs. First Report	2001 FY Inspection Results
440+45	Crest	Trans.	2001	No Comment	Transverse crack found at stations 440+45.
493+87	Crest	Long. & Hole	2001	No Comment	Longitudinal crack at 493+87. Hole near centerline of crest at 493+87, probe inserted to approximately 2.5-feet
710+00	Crest	Hole	2001	No Comment	Rodent holes and a few scattered larger burrows between the right abutment and station 710+00.
710+00	U/S slope	Rills	2001	No Comment	Some rills scattered along the dam from the right abutment to Sta. 710+00. Repair following standard maintenance procedures. One deep gully at Sta. 535+00 (12- to 18-inch deep). Ultimately may need to gravel mulch upstream slope from right abutment to Sta. 710+00
738+00	U/S slope	Depression	2001	No Comment	Depression at Sta. 738+00 that appears to be a natural wash area (45-feet by 20-feet by 5-feet deep).
685+00 to 707+00	D/S slope	No Type	2001	No Comment	Slopes appear to be steeper than 2:1
892+16	D/S slope	Gulley	2001	No Comment	One deep gully at Sta. 892+16 (12- to 18-inch deep) needs to be repaired.
957+15	D/S slope	Depression	2000	No change	Bulges and depressions previously reported at Sta. 957+15 show no change
965+00 to 966+00	D/S slope	Depression	2000	No change	Possible transverse crack on downstream slope, probe inserted 20-inch.

January 13-14, 2003 Inspection Conducted by Larry Lambert (FCDMC)
Contacts: Brett Howey (ADWR) and Noller Hebert (NRCS)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2003 FY vs. First Report**	**2003 FY Inspection Results**
433+24	D/S slope	Trans.	2002	Did not find	Transverse crack on downstream slope.
433+60	Crest	Hole	2003	No Comment	Hole at centerline of dam on crest (1 1/2-inch diameter by 12-inch deep).
434+16	Crest	Trans. Hole	2002	No change	Hole on crest and transverse crack on downstream slope.
436+51	D/S slope	Trans.	2002	No change	Transverse crack on downstream slope, holes along crack and probe inserted 18-inch.
436+62	D/S slope	Trans.	2003	No Comment	Transverse crack on downstream slope at shoulder.
437+22	D/S slope	Trans.	2002	No change	Transverse crack on downstream slope.
440+35	D/S slope	Trans.	2001	No change	Transverse crack on downstream slope, no change in 2002 or 2003.
440+54	D/S slope	Trans.	2002	No change	Transverse crack on downstream slope.
440+82	D/S slope	Trans.	2002	Did not find	Possible transverse crack on downstream slope, probe inserted 20-inch.
441+00	D/S slope	Trans.	2002	No change	Transverse crack on downstream slope, probe inserted 20-inch.
441+43	D/S slope	Trans.	2002	No change	Transverse crack on downstream slope.
441+92	D/S slope	Trans.	2002	No change	Transverse crack on downstream slope.
449+42	U/S slope	Hole	2002	No change	Typical animal dug hole. These are scattered throughout the dam and should be repaired following approved repair procedure
451+51 to 451+75	U/S slope	Long.	2002	Did not find	Longitudinal crack along upstream slope varies from 3- to 6-feet from crest. Crack 20-inch +/- deep

January 13-14, 2003 Inspection Conducted by Larry Lambert (FCDMC)
Contacts: Brett Howey (ADWR) and Noller Hebert (NRCS)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2003 FY vs. First Report**	**2003 FY Inspection Results**
490+58 to 491+13	Crest	Long.	2002	No change	Longitudinal crack wanders along the crest centerline to+/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Few locations in 2003 where probe could be inserted to 30-inch.
491+97 to 495+21	Crest	Long.	2002	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline, except at station 492+74 where probe was inserted to handle (34-inch). Few locations in 2003 where probe could be inserted to 30-inch depth.
495+65	D/S toe	Long.	2003	No Comment	Longitudinal crack at downstream toe that extends for about 30-feet.
498+14 to 498+69	Crest	Long.	2002	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline.
500+00	Crest	Long.	2002	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted in holes along the crack to a depth of 18- to 30-inch.
501+06 to 506+40	Crest	Long.	2003	No Comment	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted in holes along the crack to a depth of 18- to 30-inch.
506+25 to 507+02	Crest	Long.	2002	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted 18- to 30-inch in holes along the crack in 2003.
506+90	Crest	Long.	2003	No Comment	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Probe inserted in holes along the crack to a depth of 18- to 30-inch.
512+53	Crest	Long.	2003	No Comment	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Probe inserted in holes along the crack to a depth of 18- to 30-inch.
519+90	Crest	Holes	2003	No Comment	Two holes on crest at downstream shoulder. Probe inserted 24-inch.
529+94 to 530+00	Crest	Long.	2002	Did not find	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline.

January 13-14, 2003 Inspection Conducted by Larry Lambert (FCDMC)
Contacts: Brett Howey (ADWR) and Noller Hebert (NRCS)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2003 FY vs. First Report**	**2003 FY Inspection Results**
577+62 to 578+03	Crest	Long.	2002	Did not find	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline.
578+27 to 578+55	Crest	Long.	2002	Did not find	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline.
652+30	Slope		No Date	No Comment	Slope appears to be over steepened in this area. Need to check upstream slope.

January 12 & 14, 2004 Inspection Conducted by Michael Greenslade, P.E. (FCDMC) and reviewed by Brett Howey, P.E. (FCDMC)
Contacts: Michael Johnson, Ph.D., P.E. (ADWR) and John Chua, P.E. (NRCS)

Inspection Notes					
Station	**Location**	**Type**	**Date First Reported**	**2004 FY vs. First Report**	**2004 FY Inspection Results**
433+60	Crest	Hole	2003	No change	Hole at centerline of dam on crest (1 1/2-inch diameter by 12-inch deep).
434+16	Crest	Trans. & Hole	2002	No change	Hole on crest and possible transverse crack on downstream slope.
436+51	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope, holes along crack and probe inserted 18-inch
436+62	D/S slope	Trans.	2003	No Comment	Possible transverse crack on downstream slope at shoulder
437+24	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope
440+35	D/S slope	Trans.	2001	No change	Possible transverse crack on downstream slope, no change in 2002 or 2003
440+54	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope
440+82	D/S slope	Trans.	2002	Did not find	Possible transverse crack on downstream slope
441+00	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope, probe inserted 20-inch.
441+43	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope
441+92	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope
443+68	D/S slope	Trans.	2004	No change	Possible transverse crack on downstream slope
487+39 to 491+13	Crest	Long.	2002	Change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline and only observed between Sta. 490+58 to 491+13. Few locations in 2003 where probe could be inserted to 30-inch. In 2004 three 1-inch diameter holes probe 12-inch at Sta. 490+79

January 12 & 14, 2004 Inspection Conducted by Michael Greenslade, P.E. (FCDMC) and reviewed by Brett Howey, P.E. (FCDMC)
Contacts: Michael Johnson, Ph.D., P.E. (ADWR) and John Chua, P.E. (NRCS)

Inspection Notes					
Station	Location	Type	Date First Reported	2004 FY vs. First Report	2004 FY Inspection Results
491+97 to 495+21	Crest	Long.	2002	Change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline, except at station 492+74 where probe was inserted to handle (34-inch). Few locations in 2003 where probe could be inserted to 30-inch depth. In 2004 a 3-inch diameter hole probed 34-inch at Sta. 492+55 and 1-inch diameter hole was probed 24-inch at Sta. 493+00.
495+65	D/S toe	Long.	2003	Did not find	Longitudinal crack at downstream toe that extends for about 30-feet.
498+14 to 498+69	Crest	Long.	2002	Did not find	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline.
500+00	Crest	Long.	2002	Did not find	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted in holes along the crack to a depth of 18 to 30-inch.
501+06 to 506+40	Crest	Long.	2003	Change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted in holes along the crack to a depth of 18- to 30-inch. In 2004 a 1-inch diameter hole probed 12-inch at Sta. 505+96 and four 1-inch diameter holes were probed 30-inch at Sta. 506+89.
506+25 to 507+02	Crest	Long.	2002	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted 18- to 30-inch in holes along the crack in 2003.
506+90	Crest	Long.	2003	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Probe inserted in holes along the crack to a depth of 18- to 30-inch. Observed four holes in 2004.
511+71 to 513+24	Crest	Long.	2003	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline.
512+53	Crest	Long.	2003	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Probe inserted in holes along the crack to a depth of 18- to 30-inch.

January 12 & 14, 2004 Inspection Conducted by Michael Greenslade, P.E. (FCDMC) and reviewed by Brett Howey, P.E. (FCDMC)
Contacts: Michael Johnson, Ph.D., P.E. (ADWR) and John Chua, P.E. (NRCS)

Inspection Notes

Station	Location	Type	Date First Reported	2004 FY vs. First Report	2004 FY Inspection Results
517+99 to 518+64	Crest	Long.	2002	Change	Longitudinal hairline crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Documented as not found during the 2003 inspection.
519+90	Crest	Holes	2003	No change	Two holes on crest at downstream shoulder. Probe inserted 24-inch.
522+89 to 523+03	Crest	Long.	2002	Change	Longitudinal hairline crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Small holes observed at Sta. 522+89, 522+93 and 523+03 Documented as not found during the 2003 inspection.

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 Ilde Chavez, P.E. (NRCS)

Inspection Notes					
Station	Location	Type	Date First Reported	2005 FY vs. First Report	2005 FY Inspection Results
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434+16	Crest	Trans. & Hole	2002	No change	Hole on crest and possible transverse crack on downstream slope.
436+51	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope, holes along crack and probe inserted 18-inch.
436+62	D/S slope	Trans.	2003	Did not find	Possible transverse crack on downstream slope at shoulder.
437+24	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope
440+35	D/S slope	Trans.	2001	Did not find	Possible transverse crack on downstream slope
440+54	D/S slope	Trans.	2002	Did not find	Possible transverse crack on downstream slope
440+82	D/S slope	Trans.	2002	Did not find	Possible transverse crack on downstream slope
441+00	D/S slope	Trans.	2002	No change	Possible transverse crack on downstream slope, probe inserted 20-inch.
441+43	D/S slope	Trans.	2002	Did not find	Possible transverse crack on downstream slope
441+92	D/S slope	Trans.	2002	Did not find	Possible transverse crack on downstream slope
443+68	D/S slope	Trans.	2004	No change	Possible transverse crack on downstream slope
487+39 to 491+13	Crest	Long.	2002	Change	Longitudinal crack wanders along the crest centerline to +/- 2 to 3-feet from centerline. Crack width in 2002 was hairline and only observed between Sta. 490+58 to 491+13. Few locations in 2003 where probe could be inserted to 30-inch. In Jan. 2004 three 1-inch diameter holes probed 12-inch at Sta. 490+79.

Nov. 15, 2004 Inspection Conducted by Brett Howey, P.E. (FCDMC) and reviewed by Michael Greenslade, P.E. (FCDMC)
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495+65	D/S Toe	Long.	2003	Did not find	Longitudinal crack at downstream toe that extends for about 30-feet.
498+14 to 498+69	Crest	Long.	2002	Did not find	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline.
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501+06 to 506+40	Crest	Long.	2003	Change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted in holes along the crack to a depth of 18- to 30-inch. In Jan 2004 a 1-inch diameter hole probed 12-inch at Sta. 505+96 and four 1-inch diameter holes were probed 30-inch at Sta. 506+89. In Nov. 2004 small holes around station 501+23.
506+25 to 507+02	Crest	Long.	2002	Change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Crack width in 2002 was hairline. Probe inserted 18- to 30-inch in holes along the crack in 2003. In Nov. 2004 holes in crest located at Sta. 503+34 and 506+01.
506+90	Crest	Long.	2003	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Probe inserted in holes along the crack to a depth of 18- to 30-inch. Observed four holes in 2004.
511+71 to 513+24	Crest	Long.	2003	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline.
512+53	Crest	Long.	2003	No change	Longitudinal crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Probe inserted in holes along the crack to a depth of 18- to 30-inch.

Nov. 15, 2004 Inspection Conducted by Brett Howey, P.E. (FCDMC) and reviewed by Michael Greenslade, P.E. (FCDMC)
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519+90	Crest	Holes	2003	No change	Two holes on crest at downstream shoulder. Probe inserted 24-inch.
522+89 to 523+03	Crest	Long.	2002	Change	Longitudinal hairline crack wanders along the crest centerline to +/- 2- to 3-feet from centerline. Small holes observed at Sta. 522+89, 522+93 and 523+03. Documented as not found during the 2003 inspection.
834+82	D/S slope	Trans.	2004	No Comment	Inactive burrow with probe insert 18-inch, possible transverse crack

FINAL

FAILURE MODE AND EFFECTS ANALYSIS FOR HARQUAHALA FLOOD RETARDING STRUCTURE MARICOPA COUNTY, ARIZONA

Submitted to:

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7878 N. 16th Street
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Submitted by:

Gannett Fleming

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

Registered Professional Engineer (Civil) CERTIFICATE NO. 35506 DEBORA J. MILLER Date Signed 11/3/05 ARIZONA, U.S.A.

Dean B. Durkee, Ph.D., P.E.
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FMEA Facilitator

November 2005

043394

1.0 INTRODUCTION

General Description

The Harquahala Flood Retarding Structure (FRS) is located in the Harquahala Valley of Maricopa County, approximately 10 miles west of Tonopah, Arizona and southwest of the Bighorn Mountains. Harquahala FR+S consists of a homogeneous earth embankment dam with two toe drain outlets. An inclined central filter drain was installed over a portion of the structure. The principal outlet consists of a 48-inch reinforced concrete pipe located near the left abutment. The emergency spillway consists of a concrete baffled chute located at approximately Station 839+00. There are two vegetative outlets located at Station 746+00 (below the Granite Reef Aqueduct – GRA: also know as the Central Arizona Project or CAP) and at Station 583+80 (above Granite Reef Aqueduct at New Tank Outlet). The FRS was designed not to have a permanent storage pool.

Harquahala FRS was assigned with an NRCS hazard classification of B but ADWR correspondence indicates that the dam was designed as an average of Class B and Class C. In light of this information, the dam was analyzed based on the average of the Class B and Class C NRCS criteria. The average is $0.3(P_{100}) + 0.7PMP$. The current ADWR hazard potential classification is significant and the size of the dam is intermediate. The surface area of a full reservoir behind the dam is 1,231 acres with a capacity of 8,404 acre-feet at the principal outlet elevation. Design of Harquahala 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 1983 by MM Sundt Construction Company.

The Harquahala FRS has performed satisfactorily to date and has experienced impoundments of various depths since construction. The maximum gage depth of impoundment was 21.5 feet recorded in October 2000. Gravel mulch was applied to the slopes of the embankment at the far western end of the structure to control erosion rilling.

Dam Data

Dam type: Homogeneous Compacted Earthfill
Dam height: 45 feet (49.3 ft max)
Dam length: 62,058 feet
Dam crest width: 14 feet
Dam crest elevation: 1420.7 feet-1422.2 (Sta. 433+50 to 450+00); 1422.20 (Sta. 450+00 to 530+00); 1422.2-1422.1 (Sta. 530+00 to Sta. 540+00); 1422.10 (Sta. 540+00 to Sta. 710+00); 1421.1-1420.6 (Sta. 710+00-717+00); 1420.6 (Sta. 717+00-742+00); 1420.6-1420.5 (Sta. 742+00-752+00); 1420.5 (Sta. 752+00-810+00); 1420.5-1420.0 (Sta. 810+00-910+00); 1420.0-1419.7 (Sta. 910+00-930+00); and 1419.7 (Sta. 930+00-1054+08).
Spillways: Principal Spillway - 48-inch diameter reinforced concrete pipe (RCP) protected with anti-seep collars, 48-inch invert at elevation 1,367.5 ft (NGVD29);
Emergency Spillway – 150-foot wide broad-crested concrete weir with baffled chute spillway: crest at elevation 1,408.4 feet (NGVD29).
Freeboard: 11.3 feet
Reservoir Surface: 1,231 acres at principal outlet elevation
Storage: 8,404 acre-feet at principal outlet elevation of 1,387.3 ft.
Hazard Classification: Significant per ADWR; assigned Class B by NRCS but designed higher than Class B requirements.

Hydrology Data (elevations in NGVD 1929 datum)

- Probable Maximum Precipitation (6-hr) = 9.31 inches (ADWR, 1981) (Note that NRCS and ADWR computed design storm precipitation for five storm durations – 6-hr, 24-hr, 36-hr, 48-hr, and 72-hr).
- 100-year 24-hour = 4.10 inches (ADWR, 1981).
- PMF Inflow Estimate: Carter & Burgess dambreak report – 162,500 cfs for 6-hour storm and 101,500 cfs for 72-hr storm duration
- ½ PMF Inflow Estimate: Carter & Burgess dambreak report – 81,252 cfs for 6-hour storm and 50,760 cfs for 72-hr storm duration
- Principal Spillway Capacity: 500 cfs per Carter & Burgess report.
- Reservoir Volume: Storage below dam crest – 12,000 acre-feet
- Drawdown flood pool: less than 10 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

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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
Frances Ackerman, R.G., E.I.T., Gannett Fleming, Inc. Geotechnical
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
Dennis Duffy, Ph.D., Flood Control District of Maricopa County

2.0 Major Findings And Understandings Gained

The following is a summary of the major findings and understandings for Harquahala FRS as a result of the Failure Mode and Effects Analysis (FMEA). Harquahala 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 Saddleback 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 Harquahala FRS are given. Findings related to failure modes or adverse consequences for overtopping and spillway discharge are given next, followed by findings related to consequences and action items (risk reduction and investigations). Finally, general findings that are informational and/or generally similar for the dam are provided.

Key Findings/Differences Related To Failure Mode Development – "Static Loading Failures – Seepage Erosion – Fissuring – Foundation Erosion –Etc."

1) There are Two Distinct Foundation Soil Conditions on the East and West Ends of the Dam.. Based on the information presented in the Geologic Investigation Report, the east end of the structure is underlain by coarse-grained gravels while the west end is underlain by finer-grained sands and silts. The change from coarse- to fine-grained materials occurs between Stations 722+00 and 712+00. The eastern portion of the alignment was designated Reach 1 (Station 717+00 to Station 1054+20), and the western portion designated as Reach 2 (Station 431+00 to Station 717+00).

2) Embankment Cracks were Noted as Early as March 1984 (FRS approximately 1 year Old at that Time).. Observed cracking has been limited to the west end of the structure where the foundation consists primarily of finer-grained sands and silts. The west-end of the structure was recently covered with gravel mulch to control surface erosion and reduce evaporation and shrinkage cracking.

3) Longitudinal Cracking and Voids above the Filter Indicate That Other Cracking is Present and In Communication with the Voids. The cause of longitudinal cracking is not known but the occurrence indicates tensile strains have developed longitudinally either due to shrinkage or differential settlement of the embankment.

4) Emergency Spillway is Founded on Bedrock and Cemented Fanglomerate. According to the Supplemental Geologic Investigation Report of Emergency Spillway Site, dated January 7, 1980, the original emergency spillway location was changed to the as-built location which straddles a low rhyolite knob.

5) There is No Evidence that Subsidence is affecting the Structure at this Time. Although subsidence has been documented regionally in the area, there has been no evidence in the survey data, to date, to suggest that subsidence is a problem for the structure.

6) Excavation of the Central Arizona Project Canal provided Borrow for the Dam; the same Contractor was used for both. Dam Construction Inspection Reports and CAP Construction Progress Report document the concurrent construction of the Harquahala FRS and the CAP canal.

7) The Two-Zone Filter over Collapse Zones Could Provide Significant Advantage over a Single Filter for Purposes of Stopping Cracks. The Harquahala FRS was designed with an inclined, two-zone filter/drain. If constructed properly the two-zone filter/drain system provides filtered drainage of the embankment during impoundment events. However, in the event of transverse cracking the finer-grained filter material on the upstream side serves as a crack stopper. Note: construction photos indicate that the filter-drain width and inclined

nature of the system were difficult to control, thereby causing some doubt as to the effectiveness of the as-built system.

8) Design Issues for Reach 1 Were Related to High Permeability Zones in the Foundation, and in Reach 2, to Loose, Collapsible Soils in the Foundation. The design specifically considered the differing foundation conditions along the length of the structure. A filter-drain system was installed in Reach 2 to protect the embankment from internal erosion through transverse cracks that were anticipated in the finer-grained embankment soils. In addition, one and one-half feet of overfill was provided to allow for post-construction foundation settlement.

9) Performance Differences are Evident for Each Reach (more cracks, surface erosion in Reach 2). Reach 1 is Performing Well Structurally. Inspection reports reviewed by the FMEA team, and confirmed by the detailed site inspection conducted by the team in March 2005, indicate that the western end of the structure (Reach 2) is experiencing more structural issues than the eastern end (Reach 1). Twenty-five of the twenty-six significant conditions noted on the November 2004 Inspection Report were located in Reach 2.

10) The Effective Width of the Filter/Drain was Reduced Due to Contamination During Construction. The FMEA Team Concluded that it Likely Still Functions as Intended. Dam Construction Inspection Reports and construction photos indicated that the contractor had difficulty with filter/drain material placement and that the filter/drain materials were contaminated with embankment materials. The contractor was not required to perform repair work at the time of construction, suggesting that the contamination was not considered to be significant enough to affect the functionality of the filter/drain.

11) There are Potentially Collapsible Soils Present Below Stripping Depth under Reach 2. The presence of collapsible soil was identified by the designers and provisions were incorporated into the design to address them. However, the stripping depth and depth of cutoff is variable and it is possible that collapsible soils are still present downstream of the cutoff in Reach 2, where cracking has been observed.

12) The Depth of Excavation for the Cutoff and the Stripping Depth are Highly Variable. The as-built drawings indicate up to 9 feet of stripping and excavation of between 5 feet and 23.5 feet for the cutoff.

13) Gravel/Sand "Buried Channel" Zones are Present Under the Dam. Dam Construction Inspection Reports indicate that at some locations, over-excavation was conducted to remove buried channels beneath the structure. Over-excavation did not reach to the bottom of at least one of these buried channels.

14) The Likelihood for Potential Collapse/Settlement under Reach 1 is Negligible Due to Nature of Foundation Soils. Reach 1 is underlain by coarse-grained gravels and there has been no indication to date that these materials have exhibited any collapse characteristics.

15) Badgers, the largest mammals observed at this dam, can burrow very deep.

Key Findings/Differences Related To Failure Mode Development – "Flooding – Overtopping – Spillway Discharges – Etc."

16) There is No Filter Diaphragm on the Principal Outlet of East Drain Outlet. The as-built construction plans indicate that there is no filter diaphragm surrounding the principal spillway conduit or the east drain outlet conduit (Station 746+00). There is a filter around the New Tank Outlet conduit (Station 583+80). All the conduits were provided with anti-seep collars. This results in less than ideal conditions relating to potential piping of finer-grained material around and in particular above the pipe in the embankment soil. However, given the well-graded and somewhat coarse-grained characteristics of the embankment in Reach 1 (east end of embankment) the principal outlet is not highly susceptible to piping.

17) No Overtopping for the ½ Probable Maximum Flood (PMF) Event. The March 9, 1981 ADWR memorandum documenting ADWR's review of the design hydrology for Harquahala FRS indicates that the dam will not be overtopped by the ½ PMF. This conclusion was based on the assumption that precipitation equal to the ½ PMP would yield a flood hydrograph approximately equal to the ½ PMF and by comparing the ½ PMP values to the NRCS precipitation values used it can be seen that the flood used by the NRCS in designing the emergency spillway was approximately 1.6 times greater than the ½ PMP required by the ADWR Safety of Dams. The NRCS criteria were to design the dam with no freeboard for their design flood.

18) CAP Canal Provides Considerable Uncertainty Regarding Flows and Overtopping Events. The Central Arizona Project (CAP) canal is located immediately downstream from the dam. The influence or impacts of the CAP canal upon dambreak flows or overtopping events from the dam have not been evaluated and therefore are not completely understood. The CAP canal may provide a measure of risk reduction or risk mitigation from dambreaks or overtopping flows of the dam. The canal has considerable freeboard and therefore available storage. Depending on the nature and degree of the breach or overtopping event the CAP canal could attenuate breach or overtopping flows.

19) The Dam Crest is not Level by Design, Reach 2 (Station 431+00 to Station 717+00) is Higher Than Reach 1 (Station 717+00 to Station 1054+20). The dam crest elevation is 1.5 feet higher in Reach 2 by design to allow for post-construction settlement. Survey data indicates that there is currently a low spot in the crest at Station 999+17.

20) There is No Well-Defined Low-Flow Upstream Channel. The Harquahala FRS is an 11-mile long dam. Typical NRCS design of flood retarding structures includes an upstream low flow channel to direct collected and impounded flows toward the principal spillway. Field review of the dam and upstream conditions as well as review of the as-built plans indicates that there is not a complete low flow channel. The result of this finding on dam operations is that small storms may pond behind the dam and may not be positively drained toward the New Tank Outlet, East Drain Outlet, or the Principal Spillway.

21) The Two 24-Inch Pipes (New Tank Outlet and East Drain Outlet) Could be Used to Assist Drawdown in the Event of Plugging of the Principal Spillway with Debris. The dam was designed with two outlets – the New Tank Outlet (Station 583+80) and the East Drain Outlet (Station 746+00). The two outlets are 24-inch reinforced concrete pipes that drain the pool and pass flows downstream. In the unlikely event that the principal spillway becomes partially or completely plugged with debris during an impoundment, the two 24-inch outlets may assist in pool drawdown.

22) Drawdown is Less Than 10 Days for the 100-Year Storm. NRCS hydrologic computations indicate that the reservoir pool (from emergency spillway crest elevation) will be drawn down in less than 10-days. This meets NRCS design criteria.

23) The Dam has Experienced Significant Impoundments (up to 21 feet) Without Adverse Impacts. The District stage records for the dam indicate that there have been impoundments up to 21 feet in depth. The inspection records and/or operation records for the dam indicate that there have been no documented adverse impacts on dam operations from these relatively deeper impoundments.

Consequence Evaluation

24) CAP Canal/Interstate-10 Provides Unknown Level Of Downstream Consequence Protection/Mitigation. The CAP canal and the Interstate 10 roadway embankment provide a measure of risk reduction and mitigation in the event of normal emergency spillway discharges, dambreaks, or overtopping of the dam embankment (see item 17 above). Both of these facilities may potentially attenuate significant discharges from the dam. However, no studies have been conducted to evaluate the impacts and influence of the canal and I-10 embankment on such large flows.

25) Downstream Consequences Are Low At This Time. The areas downstream from Harquahala FRS to Centennial Wash are relatively undeveloped. There is a low population at risk and downstream land uses are primarily open desert and agriculture. Major infrastructure includes the CAP canal and the I-10 roadway. Damage will be sustained to the CAP canal from normal discharges from the emergency spillway, dambreaks, or overtopping of the dam.

26) Access to dam crest is limited during major impoundments. Access to the dam crest during impoundments is limited to approaches at the left and right abutments. Access to the dam at the left abutment may be limited further during emergency spillway discharges depending on the nature of the downstream flow distribution. The CAP canal does not have an accommodation for vehicular crossings along the length of the dam. Ramps are provided along the length of the dam on both the upstream and downstream slopes. The upstream ramps would be inaccessible.

General Findings

27) Two Distinct Reaches (Reach 1 - Station 717+00 to Station 1054+20 and Reach 2 - Station 431+00 to Station 717+00) were Recognized by Design. The design of the dam explicitly accounted for the varying foundation conditions over the length of the structure. A filter/drain system was installed in Reach 2 to mitigate potential internal erosion through transverse cracks that were anticipated to be a problem in the embankment in Reach 2.

28) CAP Canal Provides Opportunity for Early Warning of Potential Problems. The CAP canal is instrumented and monitored by CAP personnel. Spills from Harquahala FRS either from the emergency spillway, a dambreak, or overtopping, would result in flows entering the CAP canal. The increase in flows in the canal would set off instruments that monitor canal operations. The CAP canal could also provide indications of local or regional settlement or land subsidence. See the description for Item 32 above.

29) Potential for Harquahala - Centennial Levee System Alternative. Potential to Install Second Outlet at West End. See description for Item 26 above. Since a majority of the drainage area is located at the western reach of the dam and hence most of the inflow from storm events, an opportunity was identified for construction of an additional principal spillway and floodway. The floodway would convey flows from the west principal spillway to Centennial Levee. Centennial Levee, in turn, would direct flows to Centennial Wash. The concept for this alternative could be extended to evaluate the potential for a second emergency spillway located in the western reach of the dam. In this fashion Harquahala FRS would be furnished with two sets of principal and emergency spillways. This concept, at some future time, could segment the dam into two smaller structures.

30) Designers and Contractors Made Good use of Previous Experience with Flood Retarding Structures in the Southwest. Original design included a central filter/drain system (Reach 2), 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.

31) Don't See overall Need for Large Scale Rehabilitation or Replacement at This Time. The FRS has performed well to date and was constructed in a manner consistent with most current understanding of the mechanisms of failure. The question of internal erosion through transverse cracks in Reach 2 should be answered before concluding that major rehabilitation or replacement is necessary.

32) No clear evidence that the dam has gone to high hazard.

33) Two Category I Failure Modes were Identified. Transverse cracking through the crest above the filter/drain zone in Reach 2 leading to failure (S5a) and Overtopping the crest during a major flood event (H1) were the two Category I failure modes identified as a result of the FMEA process.

34) Land Rights Above Top of Dam. The Flood Control District has upstream land rights extending above the elevation of the crest of the dam. This makes the opportunity for a future dam raise a feasible concept.

35) FMEA Has Provided Important Recommendations for District's Dam Safety Program.

Action Items – Risk Reduction Measures or Investigations

1) Level gage needed due to long dam length and large drainage area in west end. A review of the watershed map for Harquahala FRS indicates the majority of the drainage area is situated above the western portion of the facility. A flood event in the western drainage area may not report to the stage gage at the principal spillway located at the extreme eastern end of the dam. It is recommended that a staff gage and pressure transducer be installed in the western reach of the dam.

2) Transverse cracking should be evaluated and repaired. The existing transverse cracking should be evaluated to determine the cause (i.e. shrinkage, collapse settlement, ground subsidence) and a method for mitigating the cracking should be developed. The method of mitigation will be dependent upon the cause and anticipated future performance/behavior of the structure.

3) All penetrations without filter diaphragms need more attention. Penetrations through the embankment that do not have a filter diaphragm should be evaluated to determine whether one is needed. Foundation soils in Reach 1 are different from the soils in Reach 2, and the need/effectiveness of a filter diaphragm will differ depending on those conditions.

4) Site-specific PMP is recommended. Site specific Probable Maximum Precipitation studies in Arizona have resulted in PMP values lower that those resulting from using the HMR-49 procedures. Harquahala FRS is an 11-mile long dam with significantly different contributing sub-basins from the west end to the east end of the dam. The west reach of the dam contributing drainage area is from alluvial plains and fans. The eastern reach contributing drainage area is from the Big Horn Mountains. HMR-49 procedures cannot adequately account for this mix of differing drainage area types and elevation differences.

5) Need PMF Routing. The Probable Maximum Flood or the ½ PMF have not been routed through the impoundment. A review of the project hydrologic and hydraulic records indicate that the dam was designed as an average between the design storm for the Class B and Class C criteria (see second paragraph of General Description and Item 16 above). The March 19, 1981 ADWR memorandum indicates that the Safety of Dams inflow design-flood for the dam should be the ½ PMF. The memo states that the ½ PMF has not bee routed through the dam.

6) Recommend Dynamic Routing be Conducted. Harquahala FRS is an 11-mile long dam that functions as a diversion/levee system. Flows are collected along the dam from many contributing streams. The timing of the hydrographs from these streams will result in a sloping water surface for the reservoir from the west end of the dam toward the east end of the dam. Normal hydrologic routing used in HEC-1 uses the modified Puls routing method that results in a level pool (level pool routing). The use of an unsteady flow model, such as the dynamic capabilities of the HEC-RAS model, will provide the water surface profile/elevation of the inflow design flood (and other storm events) for Harquahala FRS. Knowing the actual water surface profile along the dam will provide the opportunity for determining residual freeboard and the potential for overtopping.

7) CAP Canal Elevation Data and Performance Records Need to be Reviewed. The CAP canal and canal embankment provides a measure of mitigation for potential dambreaks or overtopping events. Also the canal has a concrete lining to reduce infiltration losses. CAP canal survey data should be obtained to evaluate elevation changes that may be signs of settlement or regional land subsidence. The CAP canal inspection and maintenance records should also be reviewed to ascertain potential canal lining crack repairs. Cracking of the concrete lining could be an early indicator of regional settlement and land subsidence.

8) Need for Reliable Survey Database to Ensure Adequate Documentation over the Length of the Long Dam. The Harquahala FRS is an 11-mile long dam. A good survey program of the dam crest over the length of the dam is required to monitor for settlement of the dam and other potential impacts (land subsidence, collapsible soils, etc). The long-term survey monitoring of the dam is needed to assist in the development of potential improvements to the dam.

9) Need to Update Dambreak Analysis; Breach Parameters are not Realistic. Based on a review of the dambreak study conducted for Harquahala FRS, the failure mode modeled in the Carter & Burgess Dambreak study was piping failure under the "sunny-day" scenario. This failure mode was chosen because the inflow design flood (IDF), which is the ½ PMF,

does not overtop the dam. The dam breach parameters used in the study appear to be unrealistic. The time to failure and final breach bottom width are 3.5 hours and 793 feet, respectively for the 72-hr PMF for the west breach location. The time to breach, based on case history for earth embankment dams of similar height, is on the order of 20 to 40 minutes.

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 is briefly described. 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 mode are provided.

CATEGORY I – HIGHLIGHTED POTENTIAL FAILURE MODES

S5a: Failure Due to Transverse Cracks Through Dam that Extend Through Crest Above Filter/Drain in Reach 2 Leading to Internal Erosion and Breach at Location of Crack During Impoundment (Category I; may go to Category II).

Failure Mode Description: Several possible transverse cracks above the filter/drain (in the upper 7 feet of the embankment) in Reach 2 have been identified. In addition, the Reach 2 soils are erosive. During a large flood, flow of impounded water into these cracks could result in continuing internal erosion and enlargement of the cracks through the upper portion of the dam, leading to a breach in the upper embankment that widens and erodes downward ultimately causing a breach of the embankment.

Adverse Factors:
(1) The upper portion of dam may be cracked.
(2) Evidence of rodent activity has been observed in the upper zone of the embankment.
(3) The filter/drain zone may have been contaminated during construction. The contractor reportedly had difficulty with material placement methods.
(4) Flow of water in a crack will down-cut and cannot be tolerated if it exists.
(5) Near surface soils with some plasticity may be vulnerable to shrinkage.
(6) There is no filter protection in the upper 7 feet of the structure.
(7) Erosion gullies have been observed on the structure and indicate that soils are erosive.
(8) Several possible transverse cracks have been identified.
(9) Longitudinal cracks on the upper upstream crest would feed the transverse cracks.
(10) There have been no crack studies/investigations on the structure to date.
(11) Cracks have been observed in the narrowest portion of dam.
(12) Cracks may exist that are not expressed on surface.

Positive Factors:
(1) A large, low probability flood event (greater than the 100-year event) would be needed to overtop the filter/drain.
(2) The flow in cracks would be for a short duration.
(3) The drain zone may intercept a portion of the flow in the cracks.
(4) CL, SC material infill may be less vulnerable to erosion.
(5) No upstream to downstream connected cracks have been observed to date.
(6) Low head and gradient.

Potential Actions for Risk Reduction (Potential Failure Mode S1a):
(1) Extend filter by excavating to top of filter and fill with filter material.

H1: Overtopping During Major Flood Event (Category I, may go to Category II).

Failure Mode Description: The Harquahala FRS is approximately 11 miles long, with the emergency spillway located near the left (eastern) abutment. Sub-basins from the western portion of the watershed are routed eastward by the relatively flat topography to the emergency spillway. In addition, the topography on the upstream side of the structure, and thus the routing of water, is topographically constricted by the presence of the Big Horn Mountains to the west of the Emergency Spillway (Station 839+00). Potentially slow routing of water to the emergency spillway, and/or backup of water behind the topographic constriction, after the occurrence of back-to back storm events in the watershed could lead to overtopping of the dam and subsequent erosion and embankment breach.

Adverse Factors:
(1) Large drainages to the west have to be routed up to several miles to the principal outlet or emergency spillway near the left (eastern) abutment.
(2) The topographic gradient is low toward the east and the emergency spillway.
(3) The upstream topography is constricted near the transition between Reach 1 and Reach 2 by the presence of the Big Horn Mountains.
(4) The pool topography is steep toward the north.

(5) Outlets (principal outlet and emergency spillway) are located at the far east end of dam.
(6) The western end of the structure (in Reach 2) is constructed of erodible soils.
(7) The dam crest has a low point at 999+17.
(8) The potential for back-to-back storms.
(9) Access to the structure would be difficult during overtopping.
(10) The structure was not designed for the full PMF event. ADWR records indicate that the structure can safely pass the ½ PMF.
(11) A large flood event that overtops the dam would impact I-10 downstream.
(12) Overtopping on the east end (Reach 1) has more potential for adverse downstream impacts because the area downstream from Reach 1 has been developed to a greater extent than the area downstream from Reach 2.

Positive Factors:
(1) The dam crest elevation is higher to the west than to the east.
(2) Small fetch length.
(3) The large size of the watershed makes it less likely that full PMF will occur over the entire watershed.
(4) The drawdown time is less than ten days from the emergency spillway crest elevation.
(5) Low spot on right (west end) abutment limits breach due to flat topography upstream and downstream.
(6) Less erodible material is present in Reach 1.
(7) Gravel mulch has been placed on Reach 2 and may provide some erosion protection.
(8) CAP canal may provide some additional storage/protection.
(9) The two 24-inch drain outlets have trash racks as well as the principal spillway intake structure.
(10) Low consequences are associated with failure due to the undeveloped downstream conditions.
(11) I-10 embankments, located downstream of the structure, will provide additional flood protection in the event of failure.
(12) Sparse/non-existent development downstream provides opportunities for various alternative spillway options.
(13) District has land rights above dam crest elevation upstream.

Potential Actions for Risk Reduction (Potential Failure Mode H1):
(1) Consider conducting dynamic reservoir routing.
(2) Consider raising dam crest/crest leveling near Station 999+17 and at the far right abutment.
(3) Check flood operation plan.
(4) Evaluate Emergency Action Plan procedures to incorporate an I-10 closure.
(5) Consider installing a second Principal Outlet or Emergency Spillway at west end of the structure (segment dam).
(6) Evaluate construction of a defined upstream low flow channel to route water to the east toward the principal outlet.
(7) Install additional instrumentation at the west end of dam.
(8) Re-evaluate PMP (orographic) and consider site-specific parameters in PMF estimation.

CATEGORY II – POTENTIAL FAILURE MODES CONSIDERED BUT NOT HIGHLIGHTED

S3a. Failure Associated with Piping along the Principal Outlet Leading to Undermining and Breach of Dam at the Location of the Principal Outlet (Category II).

Failure Mode Description: The principal outlet is a 48-inch RCP and has anti-seep collars. Construction issues associated with anti-seep collars (difficulty in compaction around the collars) can lead to areas of preferential flow paths around the collars. The preferential flow can result in piping of embankment materials and undermining, ultimately leading to breaching at the principal outlet location during impoundment events.

Adverse Factors:
(1) There is no central filter or diaphragm around the principal outlet conduit.
(2) Compaction in the backfill around the top of the cradle and seepage collars is difficult.
(3) CAP canal saturates the downstream toe area, and is a potential exit area for piping.

Positive Factors:
(1) The principal outlet is constructed on a cradle to the spring-line and is on a bedrock foundation.
(2) The outlet is very long and has a fairly low hydraulic head.
(3) The structure has experienced several feet of head in the past without incident. The principal spillway has been tested.
(4) The impoundment time for the structure is short (less than 10 days for a 100-year storm event).

Potential Actions for Risk Reduction (Potential Failure Mode S3a):
(1) Build a downstream stability berm and diaphragm.
(2) Evaluate the cost of installing a filter diaphragm around the principal outlet.

S3b. Failure Due to Piping along the Eastern 24-inch Drain Outlet (Station 746+00) that Leads to Undermining and Breach of Dam during Impoundment (Category II).

Failure Mode Description: The drain outlet has anti-seep collars. Construction issues associated with anti-seep collars (difficulty in compaction around the collars) can lead to areas of preferential flow paths around the collars. The preferential flow can result in piping of embankment materials and undermining, ultimately leading to breaching at the outlet location during impoundment events.

Adverse Factors:
(1) Drain outlet has issues with proximity of CAP canal related to saturation of the downstream toe area and presence of a potential piping exit, similar to those at the principal outlet.
(2) The eastern 24-inch drain outlet is located in Reach 2 (Station 746+00), which has more erodible foundation soils.
(3) No filter diaphragm is present around the east drain outlet.
(4) Compaction in the backfill at the top of the cradle around the anti-seep collars is difficult.
(5) Materials in Reach 2 are more erodible.
(6) The rigid outlet pipe may span voids if the foundation settles under the pipe, providing a potential "roof" over developing piping features.

Positive Factors:
(1) The outlet was constructed on a concrete cradle.
(2) The impoundment time for the structure is short (less than 10 days for a 100-year storm event)

S5b. Failure Due to Transverse Cracks through Dam and Filter/Drain in Reach 2 that Leads to Internal Erosion and Breach at Location of Crack during Impoundments. (Category II).

Failure Mode Description: A transverse crack extends from the crest of the embankment downward into the embankment and fully through the filter. During impoundment, flow may develop through the transverse crack and initiate the process of internal erosion of upstream embankment material which can then be transported through the crack and the cracked 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 contractor had difficulties constructing the filter (width, contamination, alignment).
(2) Foundation may be potentially collapsible at depths below stripping. Future settlement could lead to more transverse cracking.
(3) Deeper cracks would "activate" at lower pool elevations.
(4) Duration of flow through these deeper cracks may be longer than duration of flow in shallow crest crack features.
(5) Head will be higher than for Failure Mode S5a.

Positive Factors:
(1) Filter/drain is present.
(2) Foundation was treated to remove loose, potentially collapsible soils.
(3) Drain zone could provide protection even if filter zone fails.
(4) Contractor tried to use finer-grained materials on upstream zone of Reach 2.
(5) Longer flow length than Failure Mode S5a.

H2a. Potential Adverse Consequences Resulting from Emergency Spillway Discharges during Major Rainfall Events (Category II).

Potential Adverse Consequence Description: The Harquahala FRS emergency spillway is a 150 foot wide reinforced concrete baffle block chute spillway located in the eastern reach of the dam (Station 939+20) of the main dam embankment. Normal flood discharges from the spillway are directed into the CAP canal. If discharges are sufficient, the flows will overtop and flow past (overshoot) the canal into several small natural, shallow, ill-defined washes. The flows continue in the washes and are directed toward Interstate 10. This potential "failure mode" does not "fail" the dam or emergency spillway. However, any large appreciable flows from the spillway would likely cause adverse consequences downstream from the dam. Very large flows have the potential for resulting in extensive damage and potential loss of life. This potential adverse consequence was rated as Category II because normal "successful" operation of the emergency spillway can produce discharges that could have significant adverse consequences and the likelihood of occurrence of these adverse consequences is associated with floods of reasonably probable frequency. The floodwaters will pass through the emergency spillway.

From that point the water will flow towards and under Interstate 10 and into the agricultural and low density residential housing communities downstream from the dam.

Adverse Factors:
(1) The CAP canal could be damaged, plugged or breached.
(2) There is a potential for population-at-risk associated with spillway discharges.
(3) This failure mode has a higher probability than other failure modes (>100-year but <1/2 PMF)
(4) Minimal lowering time.

Positive Factors:
(1) CAP canal will capture some or all of the discharges and mitigate downstream impacts.
(2) Flow will occur first through the principal outlet (early warning).
(3) The structure is instrumented with ALERT system.
(4) I-10 embankment attenuates flows (shallow depth, slow velocities)
(5) Population at risk is downstream from the I-10 embankment, which provides some additional protection.
(6) The emergency spillway and channel are founded on bedrock.
(7) There is an Emergency Action Plan (EAP).
(8) Relatively short duration of flows.

Potential Actions for Risk Reduction (Potential Failure Mode H2a):
(1) Extend EAP mapping to include the emergency spillway discharge inundation area below I-10.
(2) Understand how CAP canal operations are impacted by emergency spillway discharges from the structure.
(3) Look at the duration of flows in the emergency spillway during 1/2 PMF event.
(4) Develop stage frequency curve for the dam.
(5) Work with Bureau of Reclamation to consider an engineered overflow structure on CAP embankment or lowering of the downstream embankment. Get profiles of CAP canal over dam length.

CATEGORY III – MORE INFORMATION OR ANALYSES ARE NEEDED IN ORDER TO CLASSIFY

S3b. Potential Failure Due to Piping Along 24-inch New Tank Drain Outlet that Leads to Undermining and Breach of Dam during Impoundment (Category III; may go to Category IV).

Failure Mode Description: The drain outlet has anti-seep collars. Construction issues associated with anti-seep collars (difficulty in compaction around the collars) can lead to areas of preferential flow paths around the collars. The preferential flow can result in piping of embankment materials and undermining, ultimately leading to breaching at the outlet location during impoundment events.

Adverse Factors:
(1) The 24-inch New Tank drain outlet has similar issues as Failure Modes S3a and S3b associated with proximity to the CAP canal related to saturation in the toe area and a potential piping exit.

(2) All potentially collapsible soils at depth under the New Tank outlet may not have been completely removed during construction.
(3) Materials in Reach 2, where the New Tank drain outlet is located, are more erodible.
(4) The rigid outlet pipe may span voids if the foundation settles under the pipe, providing a potential "roof" over developing piping features.
(5) Compaction in the backfill at the top of the cradle around the anti-seep collars is difficult.

Positive Factors:
(1) The filter/drain extends around the New Tank outlet pipe.
(2) The outlet was constructed on a concrete cradle.
(3) The impoundment time for the structure is short (less than 10 days for a 100-year storm event).
(4) Loose, potentially collapsible soils were stripped under the New Tanks outlet section.

S4. Failure Due to Internal Erosion through Transverse Cracks Caused by Collapse Settlement or Ground Subsidence/Earth Fissuring Involving Foundation Along Reach 2 Leading to Breach at Location of Cracks (Category III; may go to Category IV).

Failure Mode Description: The presence of earth fissures and the resulting ground subsidence can result in differential settlement of long earth embankments, which can in turn lead to transverse cracking of the structure. Similar to other failure modes involving transverse cracking, during impoundment, flow may develop through the transverse crack and initiate the process of internal erosion. As the erosion process continues and the crack widens, it may migrate downward under sustained reservoir head, and a breach of the embankment is conceivable.

Adverse Factors:
(1) The transition of foundation materials between Reaches 1 and 2 may result in differential settlement.
(2) Potentially collapsible materials may extend deeper than the stripping zone.
(3) Variable zones of permeability are present.
(4) Earth fissures have been documented in the region, about 3.4 miles southwest of the FRS.
(5) More erosive materials are present in Reach 2.

Positive Factors:
(1) Shallow, loose, potentially collapsible materials were removed and the foundation was closely inspected prior to construction of the embankment.
(2) The central filter drain will protect the Reach 2 embankment section from cracks.
(3) The structure has experienced significant impoundment and there has been minimal indication of longitudinal cracking.
(4) Short impoundment time.
(5) CAP canal lining provides potential early warning of earth fissure development.
(6) INSAR data could help in the evaluation of ground subsidence in the vicinity of the structure.
(7) An examination of recent aerial photos does not show evidence that this fissure is currently "active".

Potential Actions for Risk Reduction (Potential Failure Mode S4):
(1) Evaluate CAP infrastructure for collapsible soils problems on Reach 2 of FRS.

(2) Get INSAR data.
(3) Continue to monitor dam by survey

S5c. Failure Due to Transverse Cracks Through Dam in Reach 1 that Leads to internal Erosion and Breach at Location of Crack During Impoundments (Category III; may go to Category IV).

Failure Mode Description: As with other failure modes involving transverse cracking, such cracking can, during impoundment, allow flow and internal erosion at the crack location. As the erosion process continues and the crack widens, it may migrate downward under sustained reservoir head, and a breach of the embankment is conceivable. In Reach 1, no transverse cracking has been observed to date; however, there is the possibility that cracks are present with no surface expression.

Adverse Factors:
(1) No central filter/drain is present in the structure along Reach 1.
(2) Cracks may be present but not expressed on surface.

Positive Factors:
(1) Foundation consists of granular materials. It may be leaky, but it is unlikely to be compressible in Reach 1.
(2) Less erodible fill material is present in Reach 1.
(3) Inspection records indicate that there are fewer possible transverse cracks than in Reach 2.
(4) The reservoir head is lower in Reach 1 than it is in Reach 2.
(5) Materials in Reach 1 are less susceptible to shrinkage cracking.
(6) Short impoundment time.
(7) The foundation in reservoir area is pervious (it will drain more quickly).

Potential Actions for Risk Reduction (Potential Failure Mode S5c):
(1) Conduct crack inspection (trenching) in Reach 1 to confirm that there are no cracks.

H2b. Failure of Emergency Spillway due to Erosion of Channel or Failure of CAP Canal (Category III; may go to Category IV).

Failure Mode Description: Large discharges in the emergency spillway may cause erosion at the downstream end of the baffle block chute. The flows from the spillway may potentially damage and fail the CAP locally at the spillway crossing. A potential for undermining of the spillway may be caused by the head cut erosion from loss of the canal structure or baffle-block chute.

Adverse Factors:
(1) The CAP canal is not constructed of reinforced concrete.
(2) The rock type of fractured volcanics is variable.

Positive Factors:
(1) The CAP canal is lined.
(2) The chute spillway is constructed of reinforced concrete.
(3) The structure has energy dissipators.
(4) The spillway foundation is concrete and is founded on fanglomerate and ryholite bedrock.

(5) As scour holes develop, the erosion process will slow.
(6) The integrity of the emergency spillway may be tested by smaller flood rather than a larger event.

Potential Actions for Risk Reduction (Potential Failure Mode H2b):
(1) Confirm rock foundation conditions at the location of the emergency spillway and toe-down of baffle-block chute into competent material.

CATEGORY IV – POTENTIAL FAILURE MODE RULED OUT

S1. Potential Failure Due to Erosion Along the Outside of the Emergency Spillway Sidewalls Leading to Breach (Category IV).

Failure Mode Description: The emergency spillway was designed and constructed with reinforced concrete spillway walls and upstream and downstream wingwalls. The upstream wingwalls extend laterally into the dam embankment 26 feet and the downstream wingwalls extend laterally 11 feet into ground. The failure mode under this scenario would be for the development of a preferential flowpath along the outside of the spillway walls. The preferential flow path could result in piping of embankment materials and ultimately lead to breach of the embankment. This failure mode was ruled out due to the short impoundment time at the high reservoir pool elevations, long flow path provided by the wing walls, evidently good contact between the embankment and the spillway sidewalls, and solid rock foundation that minimize risk of settlement of the structure and separation between the backfill and sidewalls.

Adverse Factors:
(1) Because there has never been flow through the emergency spillway, the potential for erosion along the spillway sidewalls has not been tested.

Positive Factors:
(1) Earth pressures are evident on sidewalls.
(2) There have been no visual observations of separation at the interface.
(3) Backfill material behind the sidewalls may be less pervious than surrounding fill.
(4) Short impoundment time.
(5) Wing walls increase flow path.
(6) Because the emergency spillway is founded on bedrock there is little risk of settlement.
(7) Surveys do not indicate movement to date.
(8) The emergency spillway sidewalls are battered.

S2. Potential Failure Due to Wetting/Collapse of Dam Foundations by Seepage From CAP Canal Leading to Transverse Crack and Breach at Crack on Impoundment (Category IV).

Failure Mode Description: This failure mode assumes water could migrate laterally from the CAP into the foundation soils beneath the upstream slope of the dam, induce collapse consolidation resulting in transverse cracking. The transverse cracks would then be subject to internal erosion during impoundment events, and ultimately lead to failure of the dam. This failure mode was ruled out because it is highly unlikely that the canal seepage would saturate the near-surface soils under the embankment, and potentially collapsible soils were removed from the upper foundation.

Adverse Factors:
(1) Water from the canal is known to leak/seep through cracks and joints in the concrete lining.

Positive Factors:
(1) To date, there is no evidence to suggest that collapse settlement is occurring.
(2) During construction, approximately 4 to 5 feet of collapsible soils were stripped from the foundation for the dam.
(3) Any seepage/wetting front that develops is directed downward under canal.
(4) The canal is concrete-lined.

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) Potential effects of the CAP canal on dam operations.
   (a) Potential for seepage and wetting/collapse of foundation under Reach 2. Although no evidence of this has been observed, it is likely that the canal leaks.
   (b) Potential plugging or collapse of CAP tunnel during a flood event could result in the possible submergence of the toe drain near the principal outlet. The FMEA team determined that the occurrence of a tunnel collapse concurrently with a significant flood event would be highly unlikely.
   (c) Overtopping of the CAP canal in the vicinity of the principal outlet could result in blockage of the principal outlet discharge.
(2) True piping between the base of the dam and the foundation in Reach 2 was ruled out as a potential failure mode due to the short duration of impoundments.
(3) The FMEA team considered potential "scour" of fine-grained fill materials in contact with a flowing buried channel deposit. Considerations for this potential failure mode include the possibility of borrow areas feeding the buried channel, short impoundment times, and the variations in permeability of both the embankment fill and the buried channels. Based on the relatively short impoundment time, the FMEA team ruled this mechanism out as a potential failure mode. It should be noted that on the as-built plans that a blanket was placed in reservoir borrow areas located within channels. It appears that a 2-foot thick soil blanket was placed in the borrow area at locations where an exposed clean sand or gravel strata was not intercepted by the cutoff trench.
(4) There is a toe drain at the principal outlet.
(5) There are several areas where the downstream slope has been over-steepened due to cuts (approximately 200 to 300 feet each) in the slope between Stations 700+00 to 724+00. These areas are not likely to cause instability; however, technically speaking the dam does not conform to the as-built slope.
(6) Different slope stability analyses were conducted for Reach 1 and Reach 2 during design of the dam. The analyses were conducted with appropriate shear strength and loading conditions (end of construction, steady-state seepage, rapid drawdown and pseudo-static) and all calculated Factors of Safety were above the minimum operation.

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.

4.0 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 from 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 - Harquahala FRS
Maricopa County, Arizona

Failure Mode	Initiating Condition	Effect	Likelihood	Consequences	Information Needs	Existing Risk Reduction Factors	Potential Risk Reduction Factors	Comments
1. Transverse cracks through dam crest above filter/drain in Reach 2 leading to internal erosion and breach (S5a) (Category I; may go to Category II)	Reservoir inflow equal to or greater than 100-year flood	Downstream inundation	Medium	Medium			Extend filter by excavating to top of filter and replacing with filter material.	
2. Overtopping during major flood event (H1)(Category I, may go to Category II)	Reservoir inflow at Probable Maximum Flood	Downstream inundation	Low to Medium	Low to Medium	Dynamic reservoir routing. Reevaluate PMP using site-specific PMF. Evaluate effect of a defined upstream low-flow channel.	There is an EAP.	Raise/level dam crest at 999+17 and right abutment. Install second PO or ES at west end of dam. Instrument west end of dam.	
3. Piping along Principal Outlet leading to undermining and breach (S3a) (Category II)	Reservoir impoundment	Downstream inundation	Low	Medium	Evaluate the cost of installing a filter diaphragm around PO.		Build downstream stability berm and diaphragm.	
4. Piping along the eastern 24-inch drain outlet leading to undermining and breach (S3b) (Category II)	Reservoir impoundment	Downstream inundation	Low	Medium				
5. Transverse cracks through dam and filter/drain in Reach 2 leading to internal erosion and breach (S5b) (Category II)	Reservoir impoundment	Downstream inundation	Low	Medium				
6. Potential adverse consequences of normal operation of emergency spillway (H2a) (Category II)		Discharges in emergency spillway and downstream inundation	High	Low to Medium	Extend EAP mapping. Effect of CAP canal operations on ES discharges. ES flows during ½ PMF event. Stage frequency.		Engineered overflow on CAP embankment or lowering of downstream embankment.	

Table 3. Summary of Failure Mode and Effects Analysis - Harquahala FRS
Maricopa County, Arizona

Failure Mode	Initiating Condition	Effect	Likelihood	Consequences	Information Needs	Existing Risk Reduction Factors	Potential Risk Reduction Factors	Comments
7. Piping along 24-inch New Tank drain outlet leading to undermining and breach (S3c) (Category III; may go to Category IV)	Reservoir impoundment	Downstream inundation	Not determined	Not determined				
8. Internal erosion on transverse cracks due to collapse settlement or ground subsidence/earth fissuring involving foundation in Reach 2 leading to breach (S4) (Category III; may go to Category IV).		Downstream inundation	Not determined	Not determined	Continued survey monitoring. Ground subsidence evaluation using INSAR. Evaluate collapsible soils problem in CAP infrastructure.	There is a central filter drain. CAP canal provides early warning.		
9. Transverse cracks through dam in Reach 1 leading to internal erosion and breach (S5c) (Category III; may go to Category IV).		Downstream inundation	Not determined	Not determined	Crack investigation in Reach 1.			
10. Emergency spillway failure due to erosion of channel or failure of CAP canal (H2b) (Category III; may go to Category IV)		Discharges in emergency spillway and downstream inundation	Not determined	Not determined	Confirm ES foundation conditions.			
11. Erosion along emergency spillway sidewalls leading erosion and breach (S1) (Category IV).		Downstream inundation	Not determined	Not determined				
12. Wetting/collapse of dam foundations due to seepage from CAP canal leading to transverse crack and breach at crack on impoundment (S2) (Category IV).		Downstream inundation	Not determined	Not determined				

5.0 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. The failure modes in Categories I and II were included in the binning process.

Table 4. Failure Mode Binning for Harquahala FRS
(Numbers refer to failure mode identification numbers in Table 3 and shaded region represents comparatively greater risk)

Likelihood

Consequence	Low	Medium	High
High			
Medium	S5b, S3a & S3b, H1	S5a	H2a
Low		H1	H2a

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. Failure Modes H2a (adverse impacts of normal operation of emergency spillway) and S5a (failure due to transverse cracking through dam that extend through crest above filter/drain in Reach 2 leading to internal erosion and breach at the location of cracking during impoundment) are within the shaded region and thus represent the failure modes of greatest risk.

Failure mode H2a represents the highest risk, associated with high likelihood and medium consequences. However, the FMEA team concluded that the consequences of this failure mode may be low. This determination would be made based on information regarding the inundation area below I-10, the flow duration for a ½ PMF event, and an understanding of the stage frequency for the dam and how CAP operations could be impacted from emergency spillway discharges.

Failure Mode S5a also represents a relatively high risk associated with medium likelihood of occurrence and medium consequences. Failure mode H1 (overtopping during major flood event) was determined to have a medium likelihood of occurrence and low consequences. This failure mode could be reclassified as having a low likelihood of occurrence and medium consequences based on the results of a dynamic reservoir routing analysis and a site-specific PMF estimation, and an understanding of the EAP procedures with respect to the closure of I-10. In addition, several mitigation measures (dam leveling near Station 999+17, installation of a second principal outlet or emergency spillway near the west end of the structure) could also reduce the risk associated with this failure mode.

Failure modes S5b (failure due to transverse cracks through dam and filter/drain in Reach 2 that leads to internal erosion and breach at location of crack during impoundments), S3a (failure associated with piping along Principal Outlet leading to undermining and breach of dam at the location of the Principal Outlet), and S3b (failure due to piping along the eastern 24-inch drain outlet that leads to undermining and breach of dam during impoundment are all failure mode that have a low likelihood of occurrence and medium consequences.

6.0 Summary and Conclusions

Harquahala FRS was constructed pursuant to a relatively modern dam design. Construction appears generally to have been consistent with design specifications, although some difficulty in filter/drain placement and associated contamination of the filter/drain occurred. The dam has performed normally and satisfactorily for 24 years. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exists 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. These 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.

Two Category I potential failure modes were identified by the FMEA team. Overtopping during a major flood event was one of the Category I potential failure modes. The second Category I potential failure mode was related to transverse cracking that has been observed in Reach 2 of the Harquahala FRS. The team concluded that both of the Category I potential failure modes may be reclassified into Category II pending additional data development and/or review.

During the binning process, the potential for failure associated with transverse cracking in Reach 2 was assigned a higher risk than the potential failure due to overtopping. In addition, the potential adverse impacts associated with the normal operation of the
Spillway, a Category II potential failure mode, was assigned a risk similar to that associated with the transverse crack risk. This is due to the higher probability of occurrence of the spillway failure mode. However, the FMEA team concluded that the normal spillway operation potential failure mode may be reassigned a lower risk pending a better understanding of the potential flow in the spillway, the effects of the CAP canal operations, and the spillway inundation area below I-10.

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

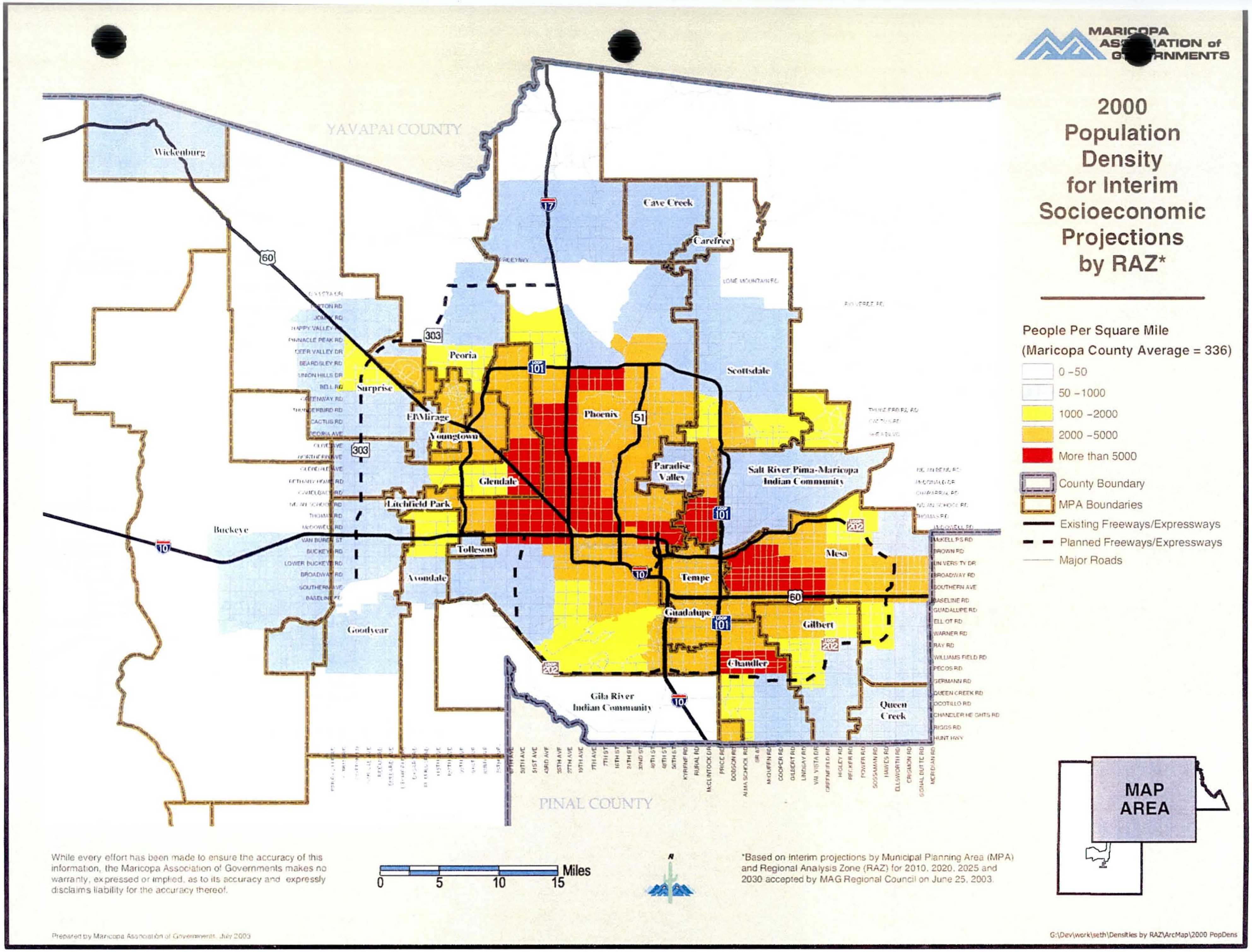
1. As-builts provided by Kimley-Horn
2. Maricopa County Assessor's Website – (http://www.maricopa.gov/assessor/gisPortal/gis_portal.asp
3. Maricopa Association of Governments Website - (http://www.mag.maricopa.gov)
4. Utility company furnished utility as-built drawings.
5. Land Use Maps provided by the State Land Department

Table 1 List of Current Properties Within Two Mile of Harquahala FRS

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
1	506-51-001	$9,000	$9,000	NO	-
2	506-51-027-B	$6,000	$6,000	NO	-
3	506-61-028-B	$6,000	$6,000	NO	-
4	506-61-028-A	$96,500	$50,188	NO	-
5	506-51-027-A	$89,000	$46,279	NO	-
6	506-61-026	$7,500	$7,500	NO	-
7	506-61-056	$4,000	$4,000	NO	-
8	506-61-055-B	$2,000	$2,000	NO	-
9	506-61-055-A	$2,500	$2,500	NO	-
10	506-61-054-B	$500	$500	NO	-
11	506-61-054-A	$3,500	$3,500	NO	-
12	506-61-053	$3,500	$3,500	NO	-
13	506-61-052	$3,500	$3,500	NO	-
14	506-61-051	$3,500	$3,500	NO	-
15	506-61-050	$5,000	$5,000	NO	-
16	506-61-049	$4,000	$4,000	NO	-
17	506-61-048	$4,500	$4,500	NO	-
18	506-61-047	$4,500	$4,500	NO	-
19	506-61-046	$3,500	$3,500	NO	-
20	506-61-045	$3,500	$3,500	NO	-
21	506-61-044	$3,500	$3,500	NO	-
22	506-61-043	$3,500	$3,500	NO	-
23	506-61-042	$3,500	$3,500	NO	-
24	506-61-041	$3,500	$3,500	NO	-
25	506-61-040	$3,500	$3,500	NO	-
26	506-61-039	$3,500	$3,500	NO	-
27	506-61-038	$3,500	$3,500	NO	-
28	506-61-037	$3,500	$3,500	NO	-
29	506-61-036	$3,500	$3,500	NO	-
30	506-61-035	$4,000	$4,000	NO	-
31	506-61-034	$4,000	$4,000	NO	-
32	506-61-033	$3,500	$3,500	NO	-
33	506-61-032	$3,500	$3,500	NO	-
34	506-61-031	$3,500	$3,500	NO	-
35	5062-61-017	$4,000	$4,000	NO	-
36	506-61-018	$4,500	$4,500	NO	-
37	506-61-021	$7,000	$7,000	NO	-
38	506-61-019-B	$4,500	$4,500	NO	-
39	506-61-019-B	$6,500	$6,500	NO	-
40	506-61-020-A	$8,000	$8,000	NO	-
41	506-61-020-B	$2,500	$2,500	NO	-
42	506-61-019-B	$4,500	$4,500	NO	-
43	506-16-006-G	$25,000	$12,432	NO	-
44	506-16-006-H	$10,500	$7,694	NO	-
45	506-16-007	$9,000	$9,000	NO	-
46	506-16-006-L	$362,000	$29,520	NO	-
47	506-61-013-B	$3,000	$3,000	NO	-

48	506-61-014-B	$500	$500	NO	-
49	506-16-006-C	$10,500	$10,500	NO	-
50	506-16-006-B	$33,000	$33,000	NO	-
51	506-16-013-A	$8,000	$8,000	NO	-
52	506-61-014-A	$8,500	$8,500	NO	-
53	506-61-012	$8,500	$8,500	NO	-
54	506-61-011	$8,500	$8,500	NO	-
55	506-61-006	$8,500	$8,500	NO	-
56	506-61-005	$8,500	$8,500	NO	-
57	506-61-004	$8,500	$8,500	NO	-
58	506-61-003	$8,500	$8,500	NO	-
59	506-61-012-D	$6,000	$6,000	NO	-
60	506-16-012-D	$18,000	$18,000	NO	-
61	506-16-011-A	$18,000	$18,000	NO	-
62	506-16-012-C	$24,500	$24,500	NO	-
63	506-16-013-F	$500	$500	NO	-
64	506-16-013-H	$7,000	$7,000	NO	-
65	506-16-013-E	$3,500	$3,500	NO	-
66	506-416-013-D	$6,500	$6,500	NO	-
67	506-16-013	$12,000	$12,000	NO	-
68	506-16-013-J	$18,500	$18,500	NO	-
69	506-16-010-B	$731,500	$380,200	NO	-
70	506-16-010-A	$353,000	$183,179	NO	-
71	506-16-014	$21,000	$21,000	NO	-
72	506-16-003-A	$34,500	$34,500	NO	-
73	506-16-002-A	$30,500	$30,500	NO	-
74	506-16-002-A	$30,500	$30,500	NO	-
75	506-16-002-B	$18,000	$18,000	NO	-
76	506-16-004-C	$14,500	$14,500	NO	-
77	506-16-004-A	$31,500	$31,500	NO	-
78	506-16-001-A	$21,000	$21,000	NO	-
79	506-16-001-B	$29,500	$29,500	NO	-
80	506-26-001	$11,500	$7,844	NO	-
81	506-26-002	$11,500	$11,500	NO	-
82	506-26-003	$9,500	$9,500	NO	-
83	506-26-004	$10,500	$6,917	NO	-
84	506-26-005	$12,500	$8,563	NO	-
85	506-26-007	$9,000	$5,585	NO	-
86	506-26-006	$10,500	$6,602	NO	-
87	506-26-009	$10,000	$6,508	NO	-
88	506-26-008	$12,000	$8,254	NO	-
89	506-26-012	$12,000	$12,000	NO	-
90	506-26-010	$11,500	$11,500	NO	-
91	506-26-011	$9,500	$8,182	NO	-
92	506-17-001	$70,000	$44,125	NO	-
93	506-17-002-A	$26,136	$18,185	NO	-
94	506-17-002-C	$37,000	$37,000	NO	-
95	506-17-002-D	$37,000	$37,000	NO	-
96	506-17-002-B	$37,000	$37,000	NO	-
97	506-17-003-H	$32,000	$31,319	NO	-
98	506-17-003-J	$21,500	$21,500	NO	-
99	506-17-003-F	$29,000	$27,453	NO	-

100	506-17-003-G	$24,000	$24,000	NO	-
101	506-17-003-D	$37,000	$37,000	NO	-
102	506-17-003-C	$37,000	$37,000	NO	-
103	506-17-002-D	$37,000	$37,000	NO	-
104	506-17-002-C	$37,000	$37,000	NO	-
105	506-17-004-A	$24,000	$24,000	NO	-
106	506-17-004-B	$23,000	$23,000	NO	-
107	506-17-004-C	$41,500	$25,128	NO	-
108	506-17-006-A	$40,500	$40,500	NO	-
109	506-17-006-C	$43,000	$43,000	NO	-
110	506-18-002-B	$24,500	$17,170	NO	-
111	506-18-002-A	$30,500	$22,660	NO	-
112	506-18-001-A	$40,000	$32,800	NO	-
113	506-18-004-G	$17,500	$14,659	NO	-
114	506-18-038	$72,000	$72,000	NO	-
115	506-18-037-A	$17,500	$13,605	NO	-
116	506-18-037-D	$71,000	$70,862	NO	-
117	506-17-005	$37,000	$37,000	NO	-
118	516-67-001	$8,000	$8,000	NO	-


MARICOPA ASSOCIATION of GOVERNMENTS
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
YAVAPAI COUNTY
PINAL COUNTY
Wickenburg
Cave Creek
Carefree
Scottsdale
Peoria
Surprise
El Mirage
Youngtown
Phoenix
Glendale
Paradise Valley
Salt River Pima-Maricopa Indian Community
Litchfield Park
Buckeye
Tolleson
Avondale
Tempe
Mesa
Guadalupe
Gilbert
Goodyear
Chandler
Gila River Indian Community
Queen Creek
MAP AREA
Miles
0
5
10
15
While every effort has been made to ensure the accuracy of this information, the Maricopa Association of Governments makes no warranty, expressed or implied, as to its accuracy and expressly disclaims liability for the accuracy thereof.
*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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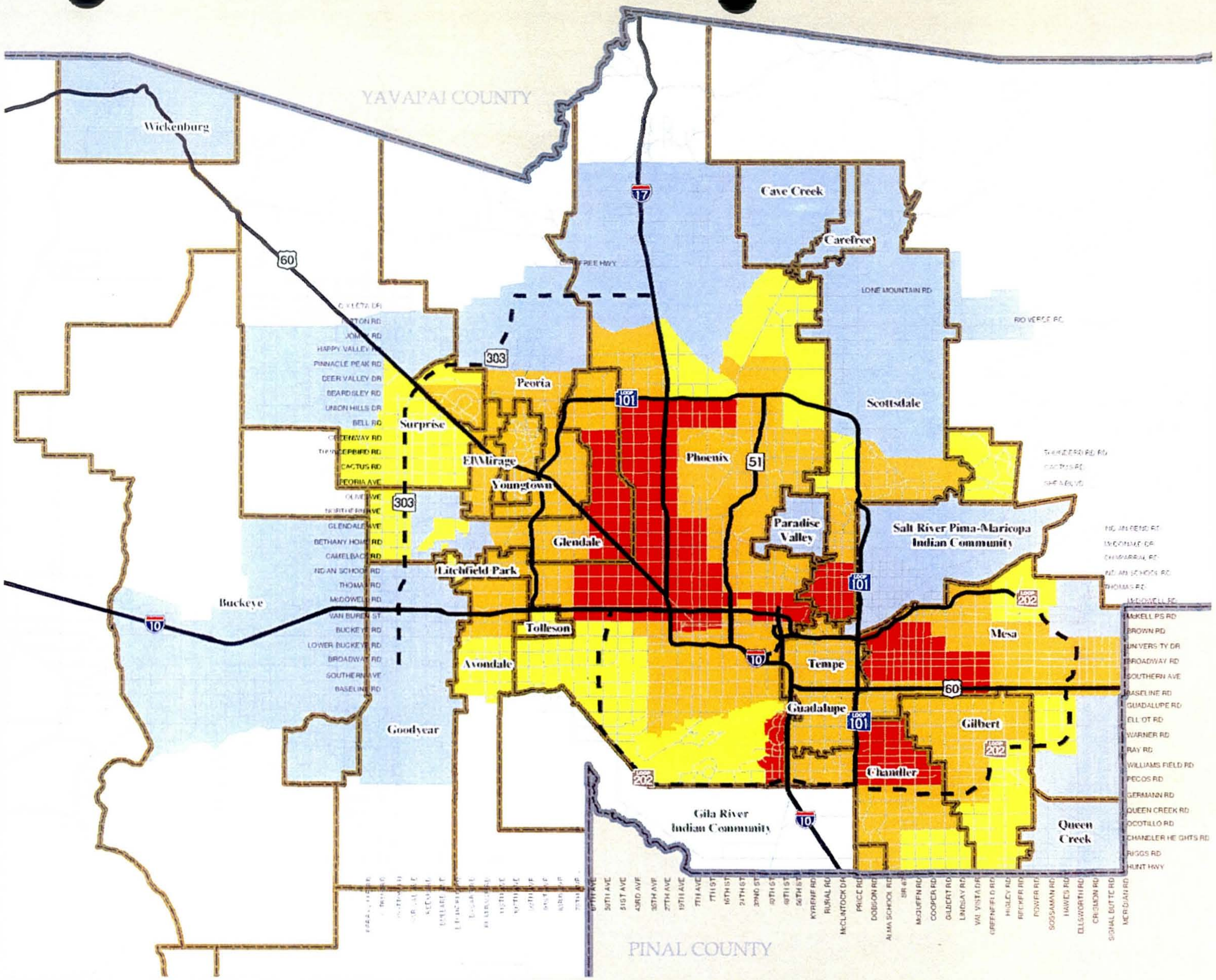
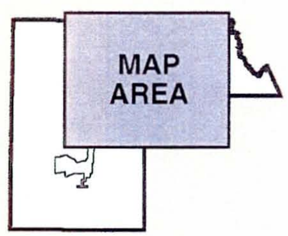
MARICOPA ASSOCIATION of GOVERNMENTS

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



While every effort has been made to ensure the accuracy of this information, the Maricopa Association of Governments makes no warranty, expressed or implied, as to its accuracy and expressly disclaims liability for the accuracy thereof.

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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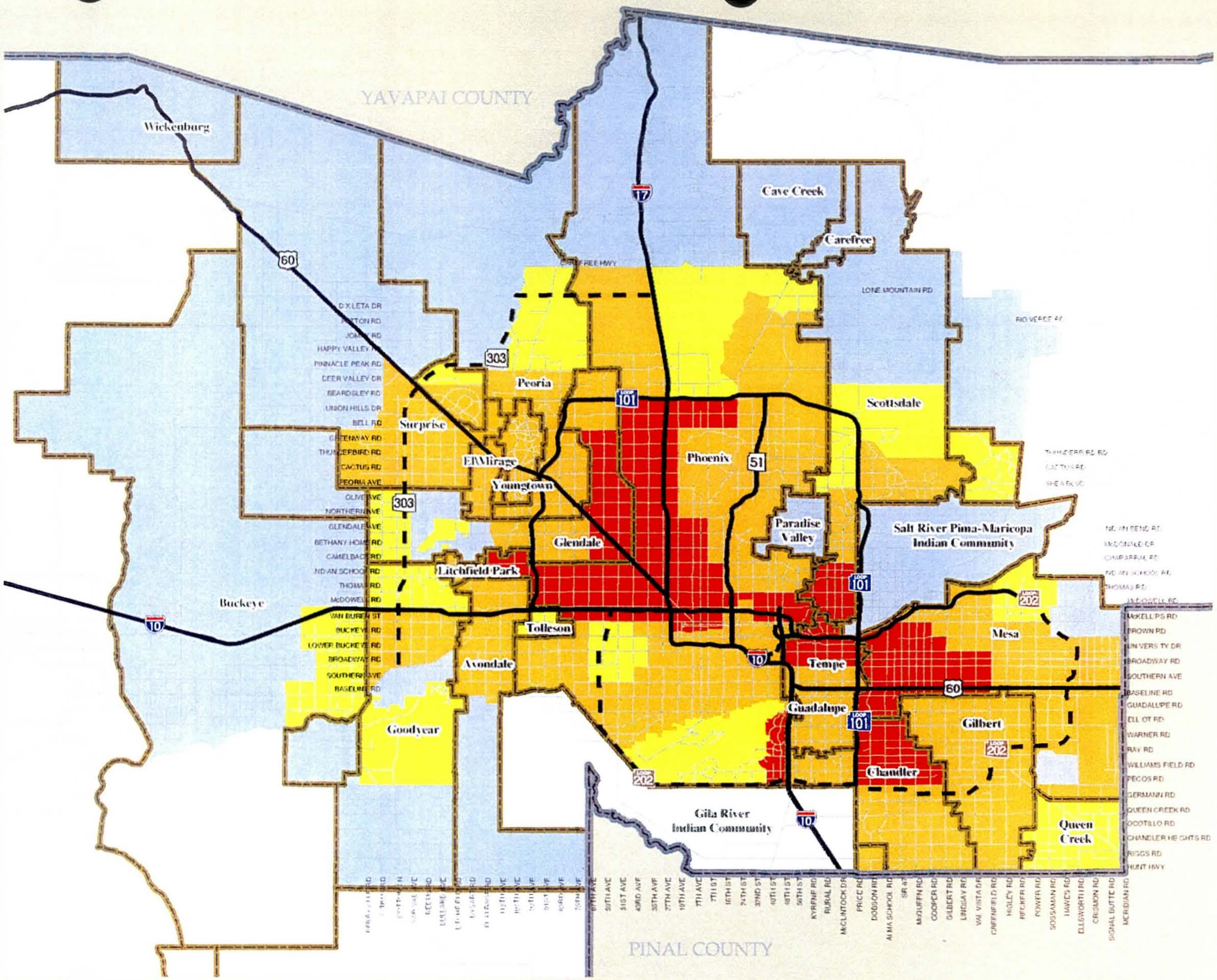
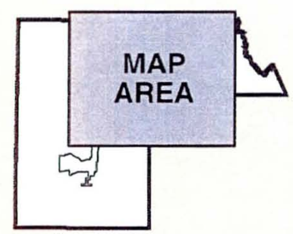
2020 Population Density for Interim Socioeconomic Projections by RAZ*



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



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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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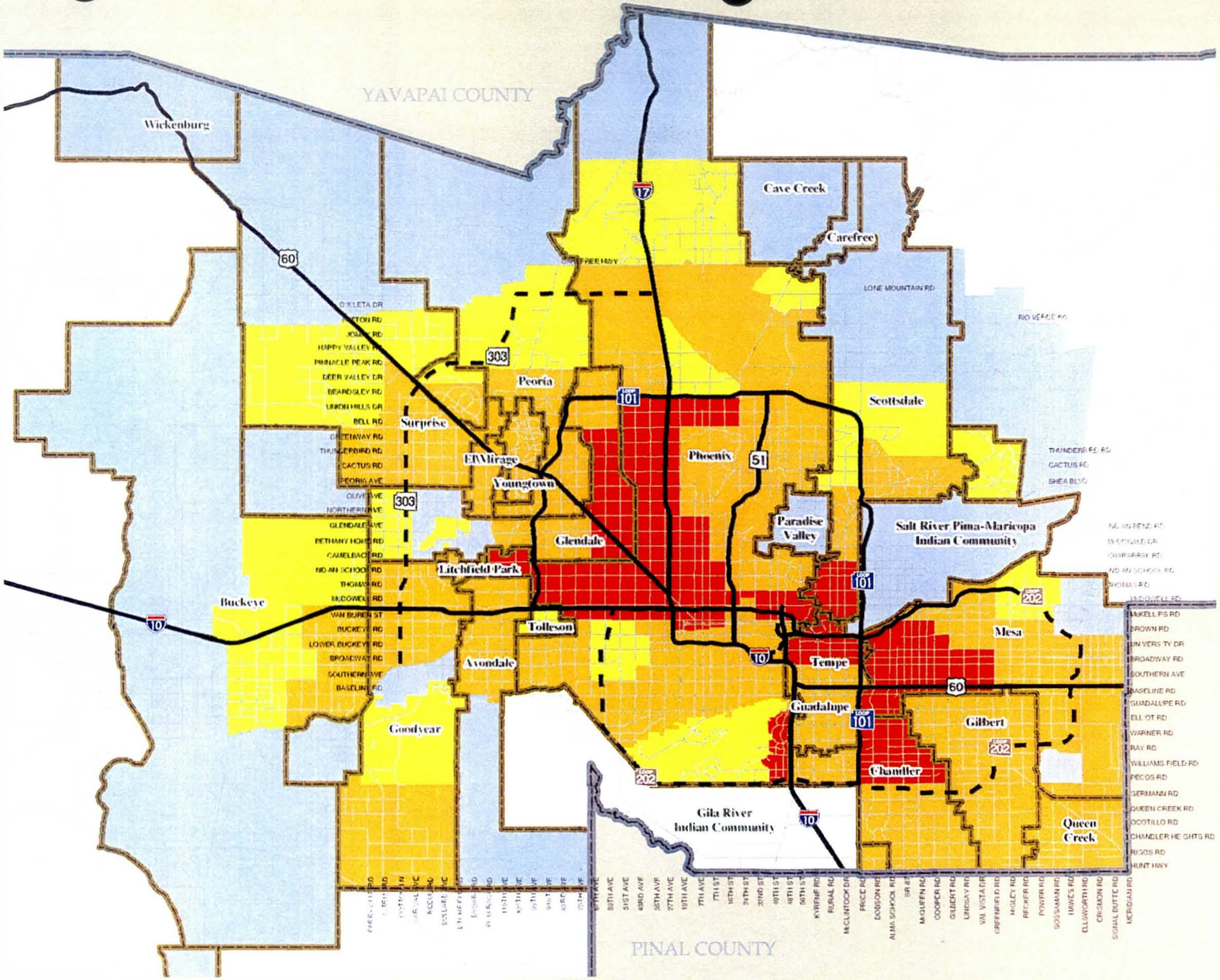
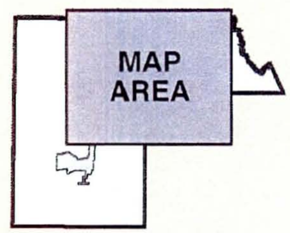
MARICOPA ASSOCIATION of GOVERNMENTS

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

*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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