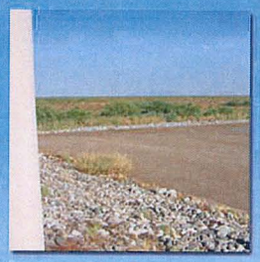

Centennial Levee
Structure Assessment Report

Registered Professional Engineer (Civil)
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ARIZONA, U.S.A.

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CENTENNIAL LEVEE
INDIVIDUAL STRUCTURE ASSESSMENT REPORT

FLOOD CONTROL DISTRICT
OF MARICOPA COUNTY
FCD 2003C015

KIMLEY-HORN AND ASSOCIATES, INC.
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NOVEMBER 2005

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ARIZONA

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INDIVIDUAL STRUCTURE ASSESSMENT REPORT for CENTENNIAL LEVEE

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1.0 EXECUTIVE SUMMARY

The Centennial Levee is a structural plan element of the Harquahala Valley Watershed Work Plan for the Harquahala Valley Watershed, Maricopa and Yuma 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 District is the local sponsor of the Watershed Work Plan. 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 (AWC). The Watershed Work Plan was developed under the authority of the Watershed Protection and Flood Prevention Act (Public Law 566, 83d Congress, 68 Stat. 666).

1.1 Description and Purpose of Levee

The Centennial Levee is a structural plan element of the Watershed Work Plan. The other structural plan elements designed under the Watershed Work Plan include the Harquahala and Saddleback Flood Retarding Structures, the Harquahala Floodway, and the Saddleback Diversion. The purpose of the structural plan elements is to provide flood protection and erosion control benefits to over 19,000 acres of farmland in the Harquahala Valley, as well as agricultural infrastructure, the Central Arizona Project canal, Interstate-10, county roads, the El Paso Natural Gas Line, the AT&T line, and minor residential and commercial properties. The Centennial Levee was designed to control runoff for the 100-year storm event and discharge it into the Centennial Wash. Centennial Levee floodway discharges into the Centennial Wash, which is a tributary of the Gila River. Centennial Levee itself was constructed upstream of the Westside Canal and protects that canal and 7,440 acres of land that are either irrigated or have a history of irrigation from the damaging effects of flooding and sedimentation (NRCS, 1976).

Levee Location. The Centennial Levee is located in Sections 5, 6, and 7 of Township 2 North, Range 9 West and Sections 11, 12, 14, and 15 of Township 3 North Range 10 West. The structure is approximately 75 miles west of downtown Phoenix, Arizona. **Figure 1 (Appendix Figures)** shows a location map of Centennial Levee. The project consists of the levee embankment and an upstream floodway.

Physical Features. The Centennial Levee consists of two segments. The main segment (referred to as Reach 1 in the design plans) is 22,400 feet long (4.3 miles). The embankment is homogeneous earthfill with 4:1 upstream slopes, 3:1 downstream slopes, and a crest width of 12 feet. The embankment has a cutoff trench that is 12 feet wide and varies in depth from three to five feet. The side slope of the cutoff trench is 0.5:1 (H:V). The cutoff trench was excavated to the depth of the Westside Canal invert to disrupt the natural flow of any sub-surface channels and to protect the lining of the Westside Canal from external hydraulic forces.

The maximum height of the embankment is 10.5 feet above the invert of the upstream floodway. The centerline of the Centennial Levee Floodway is 260 feet upstream of the Levee centerline. The floodway has a trapezoidal cross-section and has a total top width of approximately 415 feet wide. The bottom width is 80 feet wide and it has side slopes of 50:1. The bottom slope of the floodway is 0.00024 feet/feet.

The second segment of the Centennial Levee is 4,118 feet long and is parallel to Centennial Wash. This segment protects the Westside Canal siphon from being damaged by flows in Centennial Wash. The embankment is also homogeneous earthfill and has a crest width of 12 feet. The downstream side of the embankment has a compound slope that is 4:1 and 10:1 and the upstream slope is 3:1. There is no cutoff trench.

The levee was constructed across several local drainage washes conveying runoff from the watershed between the CAP and the levee. The contributing watershed is 21 square miles. The sediment storage accumulation was estimated to be 1 acre-feet per square mile per year by the Arizona Department of Water Resources (AWC 1974).

1.2 Hydrologic and Hydraulic Technical Review Summary

Levee Design Criteria. The Centennial Levee was designed according to SCS "Dike Standard," which was the NRCS design criteria at that time and the precursor to today's NRCS Conservation Practice Standard 356 for Arizona-Dikes (NRCS, 2002). This standard is referred to as Code 356, which was updated by the NRCS for Arizona in 2002. The upstream floodway was designed according to the "Open Channel Standard," which was the precursor to Conservation Practice Standard 582 for Arizona-Open Channel (NRCS, 2002a). Code 582 specifies that Technical Release-25 (TR-25) "Design of Open Channels" be used to develop the water-surface profile and to determine the stability of the channel.

Levee Classification. The NRCS, based on Code 356, has three classes of levees, or dikes. Class I dikes are constructed on sites where failure may cause loss of life or serious damage to homes, industrial and commercial buildings, important public utilities, main highways or railroads, and high value land, crops, or other improvements. Class I dikes are also specified for unusual or complex site condition or where the height of the dike exceeds 12 feet. Class II dikes are those constructed in highly developed and productive agricultural area where a failure may damage isolated homes, highways or minor railroads, or cause interruption of relatively important public utilities or where the maximum height of the levee does not exceed 12 feet. Class III dikes are those constructed in rural or agricultural areas where damage from a failure is likely to be minimal or the maximum height of the levee is less than six feet for mineral soils or four feet for organic soils.

Centennial Levee was designated as a Class II dike, but it was designed for the 100-year water-surface elevation, which is a Class I level.

Modifications Related Levee 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 Centennial Levee since original construction related to safety.

Non-Safety Levee Modifications. A rainfall gage was installed at the site in 1984 and a stage gage to measure steamflow in the floodway was constructed in 2003. The rainfall gage is located at approximately Station 165+00 along the levee and was put in its current location in 1992. The old rainfall gage was near end of the levee in Centennial Wash. As a part of the District's Dam Safety Program, the streamflow gage was constructed at the Centennial Levee in 2003 to measure the flow in the Centennial Levee floodway.

Hydrology and Hydraulics Review. In the Supplement to the Watershed Work Plan (NRCS, 1977), the new Centennial Levee was designed to replace the Burnt Mountain Floodway, and the Big Horn FRS and Floodway. The levee was designed to be 9.45 miles long and split into two reaches. Reach 1 went from just south of I-10 to the Centennial Wash, a distance of 3.7 miles. Reach 2 paralleled Centennial Wash like the design called for in the 1967 Watershed Work Plan and was 5.75 miles long. Only Reach 1 was constructed and it was 4.24 miles long (22,400 feet). A small dike parallel to Centennial Wash to protect the Westside Canal siphon was the only section of Reach 2 to be constructed. That dike is 0.8 miles long (4,118 feet).

Reach 1 of the Centennial Levee and associated floodway were designed to divert the runoff from the 100-year, 24-hour duration storm from the contributing watershed to the Centennial Wash. The contributing drainage area to Reach 1 is 21 square miles (NRCS, 1976). Interstate-10 does dissect the watershed and controls some of the flow in the watershed through culverts under the roadway.

The watershed for Centennial Levee watershed was divided into four subbasins by the NRCS. Area-capacity tables for the basins were developed and the 100-year, 24-hour runoff and 100-year sediment yield estimates were developed. Floodway sections were then developed to contain the peak inflow with two-feet of freeboard on the levee. Peak flows at Centennial Levee are 7,540 cfs at the outlet to Centennial Wash.

The NRCS completed a hydrology study for the Centennial Wash in 1983 (NRCS, 1983a). The study was completed for the entire Centennial Wash watershed including the watershed for Centennial Levee. For the Centennial Levee sub-basin, the contributing drainage area was estimated to be 24.46 square miles.

Cella Barr and Associates (CBA) completed a detailed flood insurance restudy on the Centennial Wash in Maricopa County, Arizona from the La Paz County line downstream a distance of nearly 40 miles to the confluence of the Centennial Wash with the Gila River near Gillespie Dam (CBA, 1988). At the outfall of the Centennial Levee into Centennial Wash, CBA estimated the total contributing watershed for Centennial Wash was 1,110 square miles, which compared favorably to an earlier NRCS study from 1984. CBA determined that the peak discharge in Centennial Wash was 52,300 cfs at the Centennial Levee compared to 55,300 in the earlier NRCS study (NRCS, 1983a).

CBA estimated that the peak discharge from the Centennial Wash watershed would be 3,294 cfs.

Overall, there are three estimates for the peak discharge for the contributing watershed for Centennial Levee. The 1976 NRCS study that was completed for the Watershed Work Plan estimates a peak discharge of 7,540 cfs for the contributing watershed for the 100-year 24-hour storm. The 1983 NRCS study, of which the complete document was not available, indicates a peak discharge of 4,910 during the 100-year, 6-hour storm event. The 1988 Cella Barr Study, showed an estimated peak discharge of 3,294 cfs for the 100-year, 24-hour storm.

Floodway Design. The upstream floodway was designed for a maximum discharge of 7,540 cfs throughout the entire length of the floodway and retains two feet of freeboard to the crest of the levee. Even though the design inflow varies from zero (0) cfs to 7,539 cfs, the floodway is designed for the maximum 7,540 cfs throughout its length. The slope of the floodway is 0.00024 feet/feet, which results in a maximum design velocity of 1.5 feet per second which is below the threshold for erosive velocities for earth lined channels. Allowable velocity in the floodway was estimated to be 1.9 feet per second. The floodway was designed with an 'aged' *n*-value of 0.035. The tailwater condition used for the design of the floodway was set for 26,400 cfs flowing in Centennial Wash (NRCS, 1977).

The NRCS completed a study to determine the erosion potential of flow from the floodway outfall to the invert of the Centennial Wash (NRCS, 1984). NRCS determined that there could be erosion potential associated with the outfall of the floodway into the Centennial Levee. NRCS completed the necessary hydraulic calculations that included a water-surface profile model of the area and detailed the soil cement protection that would be required, as well as calculated the size and thickness of the protection that would be required. The discharge rates used in this analysis was 4,010 cfs. Ultimately, this soil cement protection scheme was not constructed, but a rock sill was installed at Station 22+75 to prevent any erosion from headcutting back upstream into the floodway. Rock riprap was placed on the slope of the embankment at the Centennial Wash outfall to protect the embankment against erosion.

Since construction of the floodway stream gage in 2003, the maximum flows measured were 13 cfs in July 2003 and 126 cfs in August of 2003. Measurements on the stage gage were 2.2 and 1.0 feet for these storms, respectively.

The estimated peak discharges during the 100-year, 24-hour duration storm range from 3,294 cfs in the Cella Barr Study (CBA, 1988) to 7,540 cfs in the Watershed Work Plan (NRCS, 1977). The 1983 NRCS study (NRCS, 1983a) estimated a peak discharge of 4,910 cfs for the 100-year, 6-hour storm. The NRCS also used discharge rate of 4,010 cfs for the erosion potential at the outfall (NRCS, 1984). With these discharge rates, the capacity of the floodway could be greater than the 100-year event, including the necessary freeboard. An analysis of the floodway including a completed water surface profile model should be completed to determine if the Levee does exceed current design

standards. This analysis could show that the current levee and floodway exceed the FEMA minimum three (3) feet of freeboard requirement.

A sediment yield analysis was completed (AWC, 1974). Total sediment yield was estimated to be 0.91 acre-feet per year for the entire watershed. For a design life of 50-years this would result in a total expected sediment volume of 45.6 acre-feet. The sediment erosion yield rates for the soil types found within the four subbasins of the entire watershed were estimated by the AWC to range between 0.055 to 0.08 acre feet per square mile per year.

The Centennial Levee annual yield rate appears to be low compared to other sediment yield rates at District dams/watersheds other than Harquahala FRS and Saddleback FRS but appears to be consistent with the Harquahala FRS and Saddleback FRS. An average annual sediment yield rate within Maricopa County ranges from 0.2 to 0.3 acre-feet per square mile per year. Using a value of 0.3 acre-feet per square mile per year provides a 50-year sediment volume of 315 acre-feet. KHA recommends that an updated sediment yield analysis be conducted for Harquahala Valley watershed that includes the Centennial Levee, Harquahala FRS, and Saddleback FRS.

1.3 Geologic and Geotechnical Technical Review Summary

Geologic Review. The Centennial Levee 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. Centennial Levee lies in the valley between the Eagletail Mountains and the Big Horn Mountains.

Levee Centerline Geology. The description of the surface and subsurface conditions along the Centennial Levee are excerpted from a SCS memorandum dated December 1, 1983 from A.D. Elkin, Geologist to R. Arrington, State Conservation Engineer.

The Reach 1 levee will cross a broad alluvial plain sloping towards the southeast. Flood flows across this plain are probably infrequent, and there are no well defined stream channels. Materials encountered in the test holes consisted of clay, silts, and sands of mostly alluvial origin except in the youngest surficial deposits where some reworking of the material by wind is evident. All of these deposits are of late Quaternary geologic age but can usually be divided into older or younger age by the presence of a distinct zone of material moderately cemented with calcium carbonate [caliche]. *Often the materials at the top of this zone have a light reddish brown color.*

Younger Alluvial and Eolian Deposits

The uppermost alluvial and eolian deposits in the area consist of cemented silty fine sands (SM) and silts with fine sand (ML) that occur in the surface throughout the length of Reach 1. They range in thickness from one foot at T.P. [Test Pit] *1017 to 9.5 feet at T.P. 1031 but average about 3 to 4 feet in thickness through most of the area. Only one density test was taken in the top three feet of these deposits (T.P. 1013) but the upper few feet generally appear to be soft and of relatively low density. Numerous animal burrows are present and a cutoff trench should be excavated at least through the upper three feet of these deposits.*

In places, the uppermost alluvial and eolian deposits lie directly on the older alluvial deposits or on younger alluvial deposits (SC, SM, CL, ML) that are slightly cemented with calcium carbonate. But along most of Reach 1 they overlie younger alluvial sands that appear moderately permeable and consist generally of fine to medium sand with some coarse sand and some silt (SM, SP-SM, and SW-SM). The greatest thicknesses occur at T.P. 1014 (5.5 feet), T.P. 1018 (5.5 feet), T.P. 1025 (7 feet), T.P. 1029 (4.5 feet), T.P. 1030 (5.5 feet), and T.P. 1035 (8 feet). Design of both the canal and levee should take into account the possibility of water occasionally moving through these layers.

Older Alluvial Deposits

The older alluvial deposits are characterized by usually being moderately cemented with calcium carbonate. The cementation is enough to sometimes make excavation with a backhoe slow but not enough to stop excavation. Also, the cementation can usually be broken down by the application of water. In many places, the uppermost few feet of the older alluvium has a light reddish brown color. The materials in the older alluvium are mostly clays (CL) and silts (ML) with much fine sand, but there are also some clayey and silty fine sands (SM and SC).

Some of the older alluvial deposits are only slightly cemented with calcium carbonate and in some places layers of moderately permeable sands consisting of fine to coarse sand with some gravel and some silt are encountered. The average dry density of the slightly cemented material was about 98 pounds per cubic foot, and that of the moderately cemented materials was about 99 pounds [per cubic foot]. Again there was much variability with low ranges about 90 pounds and high ranges around 110 pounds [per cubic foot].

Seismic Evaluation. A seismic evaluation conducted by Euge, Schell, & Lam (1992), the Centennial Levee lies within the Sonoran Seismic Source Zone (**Appendix A**). 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 Centennial Levee, Sand Tank Fault, is about 60 miles southeast of the structure (**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 0.2 millimeters per year with the last occurrence less than 130,000 years ago (USGS, 2005). In **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 Centennial Levee.

Groundwater in the Harquahala Groundwater Basin. The Centennial Levee 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 8000 deep in the center of the basin (Oppenheimer, 1980). 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.

Groundwater in the Project Vicinity. The closest well (B-02-09 7ABB) to the Centennial Levee with a long-term hydrograph record is approximately one mile east. The hydrograph shows a water level decline of about 5 feet from 1999 through 2004. A hydrograph for well (B-02-10 14DCA) has a long term record through 1991 and shows a continuing slight downward decline in the water level of about 5 feet. Two wells about 3 miles east and two wells about 2.5 miles south of the Levee also show a decline in the water level. Hydrographs for wells within approximately three to four miles of the Centennial Levee were obtained from the Arizona Department of Water Resources (**Appendix A**).

Regional Subsidence. Prior to the utilization of groundwater in south-central Arizona, the water 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 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.

Study Area Subsidence. Historic National Geodetic Survey (NGS) level line data is not available in the vicinity of the Centennial Levee nor in the central portions of Harquahala Valley. Several survey monuments within the basin that are located along Interstate 10, along the CAP alignment, and scattered location throughout the basin; however, there is insufficient level data available to calculate the amount of subsidence that has historically occurred throughout the entire basin. The fact that earth fissures have opened up in the basin relatively recently (Roger's Fissure in 1997) indicates the basin sediment are being stressed, probably due to land subsidence in the basin.

Earth Fissures. 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 **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.

Known Earth Fissures in the Project Vicinity. There are three earth fissures reported in the Harquahala Valley. The closest fissure to the Centennial Levee is the Rogers fissure that was discovered in 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 1.6 miles west of the levee. The fissure made its first 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 (Corkhill, 1998). Development of the surface expression of the Rogers fissure was unusual in that there were no reported noticeable 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.

Another earth fissure is approximately 1.9 miles east of the northeast end of Centennial Levee in Section 9, Township 2 North, Range 9 West. This fissure was first discovered in 1958, visible in an aerial photo. The fissure was examined in 1978 and appeared to have been dormant for many years (Graf, 1980). In 1983, Scott Geotechnical Services trenched this feature and did not confirm the presence of any subsurface earth fissure trace in the area, but this finding has not been confirmed by any other consultant or agency.

The third earth fissure was documented in 1961 in a farm field about 6.8 miles southeast of Centennial Levee 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.

Foundation Conditions. A Preliminary Geologic Investigation was conducted in 1974 (AWC 1974) along an early project alignment approximately 1 mile downstream from the final alignment. A subsurface investigation consisting of test pits and soil sampling was performed by SCS in 1983 along the final project alignment, near the centerline for the proposed Westside Main Canal. No formal report of the 1983 investigation was available for review during the preparation of this Structure Assessment Report; however, the test pit logs and soil test data were reviewed. It is not known whether a formal investigation report was prepared for the project. Because of the relocation of the levee alignment and because of the limited scope of the Preliminary Geologic Investigation, the information contained in this report is based on the results of the 1983 investigation. However, it should be noted that the results of the Preliminary Geologic Investigation are similar to the results obtained in 1983.

The 1983 soils investigation included installation of forty backhoe test pits and collection and testing of sixty-one soil samples along the proposed centerline of the Westside Main Canal. The results of laboratory testing on these soil samples are included in Attachment A of **Appendix B**. Backhoe test pits from the 1983 soils investigation indicate that "younger alluvium and eolian" deposits overlie "older alluvium" along the length of the levee alignment. The thickness of the "younger alluvium and eolian" deposits ranges from 2.0 feet to 15.0 feet, based on information contained in the test pit logs.

Foundation soil laboratory test data from the 1983 soils investigation are summarized in **Appendix B**. More than 3/4 of all soil samples were fine grained, mostly silts, clayey silts and clays. Although the majority of soil samples were classified as ML, they were reportedly expected to behave like fine-grained silty sand (SM), possibly susceptible to erosion. These soils were also characterized as having relatively strong bearing strength for fine-grained soils (SCS, 1984).

All soil samples contained some clay particles: between 1% and 37% with average of 16% by weight, suggesting that clay mineral behavior will influence the soils. Palygorskite (also known as attapulgite), a tubular clay mineral, was reportedly present in nearly all soils samples, most of which have approximately equal amounts of palygorskite, montmorillonite and illite (SCS, 1984). Little was known of the engineering properties of palygorskite at the time Centennial Levee was designed and constructed (SCS, 1984). Palygorskite is a hydrated magnesium aluminum silicate similar to bentonite in its ability to absorb water (SCS, 1984) but differs from bentonite in its thin elongate chain type structure. The clay minerals present in the soil samples are moisture sensitive and tend to swell when wet and shrink when dry.

Embankment. The Centennial Levee embankment section was based on the design criteria from the Soil Conservation Service National Engineering Handbook, Section 2,

Engineering Design Standards, Part 1: Dike and Levee for a Class II dike. Centennial Levee was designed as a homogenous earth embankment with 4H:1V upstream and 3H:1V downstream slopes, maximum height of 10.5 feet, 12-foot crest width and a continuous central foundation cut-off trench with a depth of at least 6 feet and base width of 12 feet. A typical cross-section of the embankment is shown as Appendix B.

The Centennial Levee crest elevation varies from 1271.80 feet above mean sea level (amsl) at Station 0+00 to 1268.92 feet amsl at Station 224+00. The elevation slope is generally 0.0002 ft/ft toward Centennial Wash, with elevation transitions at three locations along the embankment.

Embankment Materials. The materials used to construct the embankment were derived from local borrow sources in the vicinity of the Levee. Eight of the sixty-one soil samples collected and tested during the 1983 soils investigation were identified by SCS as being representative of borrow materials. Four of the representative borrow material samples were collected from along the Westside Main Canal centerline and four were collected from the floodway area. The results of the laboratory testing are summarized in Appendix B. The as-built plans indicate that the embankment was to be constructed of soils containing not less than 15% passing the No. 200 sieve. No as-built soil data for the levee were available for review for this Assessment.

According to the soil test data, the borrow materials consist of silts (ML), clays (CL), clayey silts (CL-ML) and silty sands (SM). The average fines content for the borrow materials was 60%. The fines were generally non-plastic or had low PI values. The permeability of the borrow soil samples (measured at 90% Standard Proctor density) ranged from 0.01 ft/day to 1.2 ft/day.

Standard Proctor test data for the borrow area soils indicated that the maximum dry density of the soil ranges from 112.5 pcf to 121.5 pcf and can be achieved over a moisture content of between 9.5% and 14.5%. The SCS recommended that the project specifications require "a workable mix with a minimum moisture content of 2% less than the optimum moisture" (SCS, 1984). Results of the swell tests performed on the alluvial materials collected from the borrow areas are summarized on in **Appendix B**. SCS reported that the swelling would likely result in the levee surface swelling and shrinking in response to changes in moisture content and that this may cause or aggravate embankment cracking (SCS, 1984).

Embankment Penetration. In 1986, US Sprint Communications Company constructed a cable crossing of Centennial Levee. The construction consisted of removing approximately 120 feet of the embankment near the left abutment, within the existing Arizona Department of Transportation right-or-way, placing a cable within a 4-inch black iron pipe conduit and reconstructing the compacted embankment.

Embankment Construction. Foundation treatment recommendations called for removal of the loose surface soils to a depth of at least 6 feet, with the final depths to be determined by the engineer after inspection of the materials encountered. The as-built

plans indicate that the surface soils were removed to depths of between 3 and 9 feet. According to the SCS (1984), the geologist recommended removal of at least 3 feet of surface soils to destroy animal burrows and consolidate surface soils. There is no indication in the files that consideration was given to removing all of the less dense younger alluvial soils to prevent potential collapse consolidation and the often associated embankment cracking. This, taken together with the depth of the cutoff indicated on the as-built plan set (3 to 9 feet), suggests that potentially collapsible younger alluvial soil deposits may be present beneath the levee. No construction inspection reports were available for review during the preparation of this report.

Floodway. A floodway was constructed upstream from Centennial Levee to direct floodwaters toward Centennial Wash. The floodway channel centerline is 260 feet upstream from the centerline of the Levee. The floodway was constructed with 50H:1V side slopes. The channel bottom is 80 feet wide and slopes toward Centennial Wash at 0.0002 ft/ft.

The floodway was designed to accommodate a relatively low flow velocity to prevent scour in the easily erodible soils (SCS, 1976). The floodway includes a soil cement section from Station 215+87 to Station 225+75 for erosion protection. The soil cement extends 415 feet from the upstream toe of the Levee and has a compacted thickness of at least 6 inches. A rock sill was specified at the floodway outlet. The rock sill is 505 feet long, perpendicular to the floodway at Station 225+75. The sill consists of a 6 foot deep and 10 foot wide section filled with "4 inch to 24 inch rock sill material" according to the construction plans. The rock sill was presumably constructed to prevent headcutting and scour along the floodway as water discharges to Centennial Wash.

Original Slope Stability Analysis. No record of slope stability analyses performed during levee design was available for review. It is not know whether these analyses were performed for design.

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:

- The land between the Central Arizona Project (CAP) canal and the levee (contributing watershed) is all public owned land. The area is dissected by Interstate-10, which is a major intercontinental transportation route.
- A portion of the Centennial Reach of the Westside Canal is immediately downstream of the Levee. The Westside Canal delivers irrigation water from the CAP to agricultural lands irrigated by the Harquahala Valley Irrigation District.
- Local irrigation infrastructure downstream of the levee.
- There are no inhabited structures identified within a two-mile radius of the levee.
- No new residential development was recorded near this levee.

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 the levee.

1.5 Field Inspection

Based on the field inspection performed by the Kimley-Horn team, previous inspection reports by the District and NRCS and the results of FMEA for the levee, major signs of distress in the form of confirmed transverse and longitudinal cracking that have been identified along Centennial Levee.

Based on the field inspection performed by the Kimley-Horn team, previous inspection reports by the District and NRCS and the results of FMEA, no safety deficiencies have been identified for Centennial Levee.

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

1.6 Failure Modes and Effects Analysis

Kimley-Horn conducted a FMEA for Centennial Levee 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 Centennial Levee.

The FMEA did not identify any Category I potential failure modes. The FMEA team did develop two Category II, three Category III, and one Category IV potential failure modes. These are:

- Failure Mode H1 - Overtopping Of Levee For Runoff Events Exceeding 100-Year Event And Exceeding Freeboard Leading To Erosion And Breach Of The Levee (Category II).
- Failure Mode S1 – Erosion And Breach Of Levee During Runoff Events Due To Flow Through Existing Transverse Cracks In Levee Embankment (Category II).
- Failure Mode H2 - Scour And Erosion Of Levee Due To Flows In Floodway Starting At Upstream Toe Leading To Breach (Category III).
- Failure Mode H3 - Overtopping Due To Dynamics Of Flows Coming In From Steeper (High Velocity) Upstream Drainages Impinging On Levee And Floodway Channel Delayed Diversion (Category III).
- Failure Mode S2 - Erosion And Breach Of Levee Due To Flow Transverse Cracks Or Earth Fissures Extending Under Structure Foundation (Category III).
- Failure Mode H4 - Overtopping Due To Sediment Build Up From Major Drainage (E.G. Tiger Wash) Raising The Bottom Of Floodway Channel In Isolated Locations (Category IV).

The qualitative risk of the two Category II failure modes were ranked by the FMEA team to be low consequence and medium likelihood.

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) Conduct an updated hydrologic analysis for the levee. Include or update, if necessary, the latest District hydrologic model for Centennial Wash.
(2) Conduct a crest level survey and collect cross sections of the levee/floodway system.
(3) Update the hydraulic steady flow analysis to ascertain existing freeboard available.
(4) Conduct an unsteady flow model (HEC-RAS) of the levee (since the levee is a structure that is 4.2 miles in length within lateral inflows). Ascertain existing freeboard and potential segments of the levee that potentially could overtop in large storm events.
(5) Conduct a levee break analysis. The downstream inundation area is presently a mix of active and dormant irrigated agriculture fields. The future land use plans provided by the District indicate that the land use around and downstream of the levee will be changed from agricultural to residential.
(6) Prepare a levee emergency action plan.
(7) Map the 100-year floodplain for the Centennial Levee floodway. Confirm District fee land versus the mapped floodplain.
(8) Consider upgrading the levee to meet current FEMA levee certification requirements. Existing lands downstream of the levee depend on the flood control protection provided by the levee. Rapid urbanization around and downstream of the levee may require that ultimately the levee become FEMA certified.
(9) Evaluate what size upstream floodway would be required to remove the levee.
(10) Update the sediment yield analysis.
(11) Evaluate raising the levee crest elevation to provide additional freeboard.
(12) Add ALERT gage to measure flow in floodway to top of levee crest elevation (at downstream end of levee).

B. Geotechnical and Geological Recommendations

1. Supplemental Geotechnical Analyses

Potentially collapsible younger alluvial soil deposits may be present beneath the levee. The remaining collapse potential and risk of wetting of potentially collapsible soils in the foundation could be evaluated through appropriate sampling, testing, and analysis. The foundation soils, particularly those beneath the crest and upstream section of the levee could be sampled using split-spoon (California) samplers, pitcher sampler, or block

samples and collapse consolidation tests could be performed to determine the current state of stress and collapse potential.

2. Documentation of Slope Stability Factors of Safety

Although at the time Centennial Levee was designed, the Federal Emergency Management Agency (FEMA) did not have a comprehensive levee policy, an interim levee policy had been established in 1981 (FEMA, 1981). The interim policy indicated that guidance on new levee construction should be addressed by referring to the minimum requirements contained in the United States Army Corps of Engineers (USACE) "Design and Construction of Levees" (EM 1110-2-1913). Current USACE guidelines (as of April 2000) include provisions for slope stability analysis.

Under reasonable loading conditions for Centennial Levee, it is expected that both upstream and downstream slopes will be stable. However, documentation of slope stability factors of safety for specified loading and design criteria is not available at the time of this review. Slope stability analyses are recommended to document factors of safety for Centennial Levee. In addition, this information would be required for FEMA certification of Centennial Levee. The stability analyses and factors of safety required by the USACE for existing levees are summarized in **Appendix B**:

C. Additional Recommendations from Inspection Report and FMEA Report

(1) Coordinate with HVID to facilitate required level of maintenance activities on the levee and canal system, particularly for the V-shaped channel.
(2) Coordinate with HVID to ensure that the sediment removal from the V-shaped channel between the downstream toe and the canal does not adversely impact the downstream toe of the levee.
(3) Continue active vegetation management program.
(4) Continue active rodent management program.
(5) Develop mitigation strategies to protect the structure against transverse and longitudinal cracking.
(6) 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 the levee where there has been a noted propensity of cracks.
(7) Determine if gravel mulch or vegetative seeding would be beneficial for the crest and embankment slopes.
(8) Survey the crest of the Levee to begin a subsidence monitoring program for the levee.
(9) Ensure the levee is on a rotation to survey the crest regularly with the Flood Retarding Structures.
(10) Update crest survey. Survey Centennial Levee crest on the same frequency as significant hazard dams.
(11) Evaluate levee to determine if the levee meets current FEMA criteria for certification.

(12) Re-station the crest of levee. The station markers on the crest of the levee should match the as-built plans.

(13) Survey profile and cross-section of floodway. This will allow an estimate of the capacity of the floodway.

(14) Update hydraulics to confirm flood channel capacity. Check sensitivity to n-values. No subsequent analysis was found since the original design documents and analysis.

(15) Check for possible correlation between location of transverse cracks and washes. Note the location of washes into the floodway and the extension of the washes downstream beyond the Westside Canal. Note the locations of observed or suspected transverse cracks on a set of as-built plans. Attempt to derive a correlation of wash location and crack location in the embankment.

(16) Update slope stability analysis. Potential weakening of soils on saturation could alter the results of a slope stability analysis. Conduct an updated or new slope stability analysis.

(17) Need to locate flood hydrographs for design event. The design flood hydrographs were not located in the project literature prior to the FMEA (however these have been found and reviewed).

(18) Revetments near the wash crossings. Revetments could be beneficial on the upstream slope of the levee embankment where large washes could impinge and cause concentrated erosion. Erosion control measures or stabilized inlets from the washes into the floodway may be investigated and evaluated.

(19) Operation and maintenance update. Verify floodway operation and maintenance standard procedures (sediment buildup, excessive vegetation, etc).

(20) Future crack investigations. Follow up on Phase I Structure Assessment recommendations to conduct additional crack investigations. Transverse and longitudinal cracks have been identified at the structure.

(21) Earth fissuring studies. Consider addition studies to investigate earth fissuring in the central Harquahala Valley. Earth fissures have been noted within 2-miles of the levee. Consider baseline elevation survey tied to reliable benchmark. Incorporate studies with studies for Harquahala FRS and Saddleback FRS.

(22) Future FMEA considerations. Consider updated FMEA if development is proposed downstream of levee. Downstream development would cause increased consequences from potential levee failure.

2.0 DESCRIPTION OF LEVEE

The Centennial Levee is a structural plan element of the Harquahala Valley Watershed Work Plan for the Harquahala Valley Watershed, Maricopa and Yuma 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 District is the local sponsor of the Watershed Work Plan. 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 (AWC). The Watershed Work 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 Levee

The Centennial Levee is a structural plan element of the Watershed Work Plan. The other structural plan elements designed under the Watershed Work Plan include the Harquahala and Saddleback Flood Retarding Structures, the Harquahala Floodway, and the Saddleback Diversion. The purpose of the structural plan elements is to provide flood protection and erosion control benefits to over 19,000 acres of farmland in the Harquahala Valley, as well as agricultural infrastructure, the Central Arizona Project canal, Interstate-10, county roads, the El Paso Natural Gas Line, the AT&T line, and minor residential and commercial properties. The Centennial Levee was designed to control runoff for the 100-year storm event and discharge it into the Centennial Wash. Centennial Levee floodway discharges into the Centennial Wash, which is a tributary of the Gila River. Centennial Levee itself was constructed upstream of the Westside Canal and protects that canal and 7,440 acres of land that are either irrigated or have a history of irrigation from the damaging effects of flooding and sedimentation (NRCS, 1976).

The federal sponsor for the Centennial Levee was the Soil Conservation Service (SCS). The local sponsor for Centennial Levee is the District. The Levee was designed between 1976 and 1984 by an engineering consultant (Franzoy Corey Engineering Company) for the Harquahala Valley Irrigation District (HVID). The levee was designed and constructed in conjunction with the Westside Canal, which is an irrigation canal that delivers irrigation water from a turnout at the Central Arizona Project canal to irrigated farmland in the Harquahala Valley. The levee is immediately upstream of the Westside Canal. The design contract was administered by the United States Bureau of Reclamation (Reclamation). The levee was designed to NRCS standards and the NRCS completed a design review report dated May 21, 1983 (NRCS 1983). The NRCS also completed the hydrologic analysis on the levee. Construction of both the Centennial

Levee and Westside Canal was completed in September 1985 under the supervision of Reclamation.

2.2 Levee Location

The Centennial Levee is located in Sections 5, 6, and 7 of Township 2 North, Range 9 West and Sections 11, 12, 14, and 15 of Township 3 North Range 10 West. The structure is approximately 75 miles west of downtown Phoenix, Arizona. **Figure 1 (Appendix Figures)** shows a location map of Centennial Levee. The project consists of the levee embankment and an upstream floodway.

2.3 Physical Features

The Centennial Levee consists of two segments. The main segment (referred to as Reach 1 in the design plans) is 22,400 feet long (4.3 miles). The embankment is homogeneous earthfill with 4:1 upstream slopes, 3:1 downstream slopes, and a crest width of 12 feet. The embankment has a cutoff trench that is 12 feet wide and varies in depth from three to five feet. The side slope of the cutoff trench is 0.5:1 (H:V). The cutoff trench was excavated to the depth of the Westside Canal invert to disrupt the natural flow of any sub-surface channels and to protect the lining of the Westside Canal from external hydraulic forces.

The maximum height of the embankment is 10.5 feet above the invert of the upstream floodway. The centerline of the Centennial Levee Floodway is 260 feet upstream of the Levee centerline. The floodway has a trapezoidal cross-section and has a total top width of approximately 415 feet wide. The bottom width is 80 feet wide and it has side slopes of 50:1. The bottom slope of the floodway is 0.00024 feet/feet.

The second segment of the Centennial Levee is 4,118 feet long and is parallel to Centennial Wash. This segment protects the Westside Canal siphon from being damages by flows in Centennial Wash. The embankment is also homogeneous earthfill and has a crest width of 12 feet. The downstream side of the embankment has a compound slope that is 4:1 and 10:1 and the upstream slope is 3:1. There is no cutoff trench.

The levee crest elevation varies as shown in **Table 1 (Appendix Tables)**. There is no active flood pool associated with the Centennial Levee and there are no conduit penetrations through the embankment. Instrumentation on Centennial Levee includes are rain gage and a stream flow gage. Both of the gages are part of the District's flood warning system. The rain gage was first installed in 1984 and the stream flow gage was installed in 2003. **Table 2 (Appendix Tables)** provides a summary of the physical data for Centennial Levee.

2.3.1 Watershed

The levee was constructed across several local drainage washes conveying runoff from the watershed between the CAP and the levee. The contributing watershed is 21 square

miles. Additional watershed areas that were in the historic contributing watershed for Centennial levee are controlled by the Tiger Wash Detention Dam and the Harquahala Flood Retarding Structure (FRS). Both of these dams are immediately upstream of the CAP and are engineered detention dams. The Tiger Wash Detention Dam was designed by Reclamation. The Harquahala FRS was designed by the NRCS and both structures were constructed in conjunction with the CAP. The Tiger Wash Detention Dam is located immediately west of Harquahala FRS. The emergency spillway for Tiger Wash Detention Dam is located as the far west end of the dam. Both of these dams have emergency spillways that discharge away from Centennial Levee. The total combined contributing watershed for both of these structures is over 246 square miles; only a portion of which would be in the historic contributing watershed for the levee. The existence of these structures made it possible to design and construct Centennial Levee as a levee instead of a Flood Retarding Structure. The sediment storage accumulation was estimated to be 1 acre-feet per year by the Arizona Department of Water Resources (AWC 1974).

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 Centennial Levee 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 assisted in the engineering assessment of the structure. Second, to review original design criteria and design guidelines under which the Centennial Levee was constructed. This report presents a discussion of the data review and dam design criteria under which the levee was originally constructed. The original levee design criteria was compared to the current NRCS standards 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, Reclamation, HVID, and ADWR-Office of Dam Safety. Kimley-Horn has prepared under separate cover, a data collection report, summarizing the information collected for Centennial Levee.

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 and structural stability of the levee.

3.1 Levee Design Criteria

Centennial Levee was first analyzed and designed by the NRCS in the mid 1960's in the original Watershed Work Plan (NRCS 1967). The Work Plan was updated in 1977. The Centennial Levee was designed according to SCS "Dike Standard," which was the NRCS design criteria at that time and the precursor to today's NRCS Conservation Practice Standard 356 for Arizona-Dikes (NRCS, 2002). This standard is referred to as Code 356, which was updated by the NRCS for Arizona in 2002. The upstream floodway was designed according to the "Open Channel Standard," which was the precursor to Conservation Practice Standard 582 for Arizona-Open Channel (NRCS, 2002a). Code 582 specifies that Technical Release-25 (TR-25) "Design of Open Channels" be used to develop the water-surface profile and to determine the stability of the channel.

The purpose of the Centennial Levee, along with the other structural elements in the Harquhala Valley Watershed Work Plan, 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 Central Arizona Project canal (CAP), Interstate-10, county roads, an El Paso Natural Gas Line, an AT&T phone line, and minor residential and commercial properties (NRCS, 1977). The 100-year, 24-hour precipitation event was used to size the height of the levee and the floodway. The structure was designed to have 2.0 feet of freeboard. **Table 3 (Appendix Tables)** provides a summary of the original

NRCS design criteria based on the dike and open channel standard and compares these criteria with current NRCS Code 356 and 582 and FEMA guidelines.

3.2 Levee Classification

The NRCS, based on Code 356, has three classes of levees, or dikes. Class I dikes are constructed on sites where failure may cause loss of life or serious damage to homes, industrial and commercial buildings, important public utilities, main highways or railroads, and high value land, crops, or other improvements. Class I dikes are also specified for unusual or complex site condition or where the height of the dike exceeds 12 feet. Class II dikes are those constructed in highly developed and productive agricultural area where a failure may damage isolated homes, highways or minor railroads, or cause interruption of relatively important public utilities or where the maximum height of the levee does not exceed 12 feet. Class III dikes are those constructed in rural or agricultural areas where damage from a failure is likely to be minimal or the maximum height of the levee is less than six feet for mineral soils or four feet for organic soils.

Centennial Levee was designated as a Class II dike, but it was designed for the 100-year water-surface elevation, which is a Class I level. The return frequency for the hydrologic design was the 100-year, 24-hour storm event (NRCS, 1976). The minimum freeboard extends at least two feet above the calculated water-surface elevation. NRCS Code 356 for Arizona states that the freeboard must be either two feet about the water-surface elevation or one-foot above the water-surface elevation plus an allowance for wave height for a Class I dike.

A. Modifications Related Levee 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 Centennial Levee since original construction related to safety. Active and normal operation and maintenance activities are conducted at the levee by both the District, who maintains the levee and the HVID who maintains the Westside Canal.

B. Non-Safety Levee Modifications

A rainfall gage was installed at the site in 1984 and a stage gage to measure steamflow in the floodway was constructed in 2003. The rainfall gage is located at approximately Station 165+00 along the levee and was put in its current location in 1992. The old rainfall gage was near end of the levee in Centennial Wash. As a part of the District's Dam Safety Program, the streamflow gage was constructed at the Centennial Levee in 2003 to measure the flow in the Centennial Levee floodway. The gage is a pressure transducer type instrument and the rating curve is based upon a HEC-RAS water-surface profile model developed by the District using survey data gathered by the District. The stream flow gage is located in the same place as the rainfall gage, approximately Station 165+00.

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). Design documentation (NRCS, 1976) developed in 1976 by the NRCS were used to update the Watershed Work Plan. The location and nature of Centennial Levee changed from the 1967 work plan. In the original Watershed Work Plan (NRCS, 1967), Centennial Levee was parallel to Centennial Wash and extended from approximately the current location of the Westside Canal siphon protective dike to the southeast for 4.3 miles. Reach 1 of Centennial Levee was designed to be a series of flood retarding structures and floodways that included the Burnt Mountain Floodway, the Big Horn Flood Retarding Structure, and the Big Horn Floodway. Centennial Levee protected the flow from being redirected behind the FRS and Floodways by keeping the flow in the Centennial Wash floodplain. The Levee extended to where the natural flow would be contained in the natural Centennial Wash floodplain.

Discussions with Reclamation led to the design of the Harquahala Flood Retarding Structure designed by the NRCS and the Tiger Wash Detention Dam designed by Reclamation. Both of these structures are located directly upstream of the Central Arizona Project Canal and the requirements for the Big Horn FRS were reduced and a more economical diversion levee concept was developed (NRCS, 1976).

In the supplement to the Watershed Work Plan (NRCS, 1977), the new Centennial Levee was designed to replace the Burnt Mountain Floodway, and the Big Horn FRS and Floodway. The levee was designed to be 9.45 miles long and split into two reaches. Reach 1 went from just south of I-10 to the Centennial Wash, a distance of 3.7 miles. Reach 2 paralleled Centennial Wash like the design called for in the 1967 Watershed Work Plan and was 5.75 miles long. Only Reach 1 was constructed and it was 4.24 miles long (22,400 feet). A small dike to protect the Westside Canal siphon was the only portion constructed and is 0.8 miles long (4,118 feet).

Reach 1 of Centennial Levee and the associated floodway were designed to divert the runoff from the 100-year, 24-hour duration storm from the contributing watershed to the Centennial Wash. The contributing drainage area to Reach 1 was identified as 21 square miles (NRCS, 1976). Interstate-10 does dissect the watershed and controls some of the flow in the watershed through culverts under the roadway. Additional historic contributing watershed is controlled by Tiger Wash Detention Basin and the Harquahala Flood Retarding Structure. The contributing drainage area for these two structures is 246 square miles; a portion of which would be in the historic contributing watershed for the levee.

The watershed for Centennial Levee was divided into four sub-basins by the NRCS. Area-capacity tables for the basins were developed and the 100-year, 24-hour runoff and 100-year sediment yield estimates were developed. Floodway sections were then

developed to contain the peak inflow with two-feet of freeboard on the levee. Peak flows at Centennial Levee are 7,540 cfs at the outlet to Centennial Wash

The NRCS completed a hydrology study for the Centennial Wash in 1983 (NRCS, 1983a). The study was completed for the entire Centennial Wash watershed including the watershed for Centennial Levee. For the Centennial Levee sub-basin, the contributing drainage area was estimated to be 24.46 square miles. The total rainfall was estimated to be 3.9 inches and the time of concentration was estimated to be 2.07 hours. The rainfall was distributed according to the SCS Type II method. The curve number for the 24-hour storm was 75. The full design hydrology report from 1983 was not located in the data collection phase of this project, but it appears large portions of the report were duplicated and attached to other design documents that were located in the data collection phase. The results found from this study indicated the expected peak discharge for 24.46 square mile watershed is 4,910 cfs for the 100-year, 6-hour storm.

Cella Barr and Associates (CBA) completed a detailed flood insurance restudy on the Centennial Wash in Maricopa County, Arizona from the La Paz County line downstream a distance of nearly 40 miles to the confluence of the Centennial Wash with the Gila River near Gillespie Dam (CBA, 1988). At the outfall of the Centennial Levee into Centennial Wash, CBA estimated the total contributing watershed for Centennial Wash was 1,110 square miles, which compared favorably to an earlier NRCS study from 1983. CBA determined that the peak discharge in Centennial Wash was 52,300 cfs at the Centennial Levee compared to 55,300 in the earlier NRCS study (NRCS, 1983a).

CBA indicated that the sub-basin contributing to flow at the Centennial Levee was 24.9 square miles. The 100-year, 24-hour duration precipitation depth was estimated to be 3.88 inches, which was taken from the NOAA Atlas II. CBA estimated the lag time to be 4.2 hours, which is equivalent to a time of concentration of 2.5 hours. CBA estimated the average curve number for the Centennial Levee sub-basin to be 77, which resulted in an initial abstraction rate of 0.60 inches. The total rainfall loss was 2.16 inches and the total rainfall runoff was 1.72 inches. The time of the peak occurred 16.5 hours after the initial of the 24-hour duration rainfall event. Ultimately, CBA estimated that the peak discharge from the Centennial Wash watershed would be 3,294 cfs.

The District completed a study for Lower Centennial Wash (FCDMC, 2005) for a FEMA Flood Insurance Study, but the upper limits did not extend beyond Interstate-10 so a direct comparison to earlier studies could not be completed.

Overall, there are three estimates for the peak discharge for the contributing watershed for Centennial Levee. The 1976 NRCS study that was completed for the Watershed Work Plan estimates a peak discharge of 7,540 cfs for the contributing watershed for the 100-year 24-hour storm. The 1983 NRCS study, of which the complete document was not available, indicates a peak discharge of 4,910 during the 100-year, 6-hour storm event. The 1988 Cella Barr Study, showed an estimated peak discharge of 3,294 cfs for the 100-year, 24-hour storm. A new hydrologic study for the watershed should be completed to verify the 100-year peak discharge.

3.3.2 Floodway Design

The upstream floodway was designed for a maximum discharge of 7,540 cfs throughout the entire length of the floodway with an additional two feet of freeboard to the crest of the levee. Even though the design inflow varies from zero (0) cfs to 7,539 cfs, the floodway is designed for the maximum 7,540 cfs throughout its length. The slope of the floodway is 0.00024 feet/feet, which results in a maximum design velocity of 1.5 feet per second which is below the threshold for erosive velocities for earth lined channels. Allowable velocity in the floodway was estimated to be 1.9 feet per second. The floodway was designed with an 'aged' *n*-value of 0.035. The tailwater condition used for the design of the floodway was set for 26,400 cfs flowing in Centennial Wash (NRCS, 1977).

A channel stability analysis was completed (NRCS, 1976). It was assumed that the channel will be stable if the allowable velocity is not exceeded and the overall sediment deposition is greater than the sediment removal (erosion). The minimum allowable velocity was determined to be 1.91 feet per second which was referenced from a Preliminary Geological Investigation of Centennial Levee authored by A. Roesler (AWC 1974). The maximum velocity of the floodway was determined to be 1.5 feet per second, so the actual velocity is below the minimum allowable velocity. With this low maximum velocity, it was determined that the sediment deposition would be far greater than any erosion potential. In fact, the sediment yield was estimated to be 0.91 acre-feet per year for the full watershed and the crest of the levee was raised slightly to accommodate for this sediment deposition.

The NRCS completed a study to determine the erosion potential of flow from the floodway outfall to the invert of the Centennial Wash (NRCS, 1984). NRCS determined that there could be erosion potential associated with the outfall of the floodway into the Centennial Levee. NRCS completed the necessary hydraulic calculations that included a water-surface profile model of the area and detailed the soil cement protection that would be required, as well as calculated the size and thickness of the protection that would be required. The design discharge used for this analysis was 4,010 cfs. Ultimately, this soil cement protection scheme was not constructed, but a rock sill was installed at Station 22+75 to prevent any erosion from headcutting back upstream into the floodway. Rock riprap was placed on the slope of the embankment at the Centennial Wash outfall to protect the embankment against erosion.

Since construction of the floodway stream gage in 2003, the maximum flows measured were 13 cfs in July 2003 and 126 cfs in August of 2003. Measurements on the stage gage were 2.2 and 1.0 feet for these storms, respectively. Flows in the floodway with a depth less than 0.65 feet are not measured by the gage.

A HEC-2 water surface profile model was prepared by Franzoy Corey for the levee and floodway for the 100-year flood event approximately 25 years ago. The computer model results available for review for this Assessment were in the form of hardcopy and PDF

files of the model output. A cross section location map was neither available nor included with the HEC-2 model results. It appears that the levee crest elevation included 2 feet of freeboard over the 100-year storm event (see Section 3.3.4 below). Federal Management Agency (FEMA) levee certification requirements require mapped levees to include a minimum of three (3) feet of freeboard over the 100-year flood event.

The estimated peak discharges during the 100-year, 24-hour duration storm range from 3,294 cfs in the Cella Barr Study (CBA, 1988) to 7,540 cfs in the Watershed Work Plan (NRCS, 1977). The 1983 NRCS study (NRCS, 1983a) estimated a peak discharge of 4,910 cfs for the 100-year, 6-hour storm. The NRCS also used discharge rate of 4,010 cfs for the erosion potential at the outfall (NRCS, 1984). With these discharge rates, the capacity of the floodway could be greater than the 100-year event, including the necessary freeboard. An analysis of the floodway including a completed water surface profile model should be completed to determine if the Levee does exceed current design standards. This analysis could show that the current levee and floodway exceed the FEMA minimum three (3) feet of freeboard requirement.

3.3.3 Hydrology and Hydraulic Recommendations

Kimley-Horn recommends the following updated hydrologic and hydraulic investigations for Centennial Levee (given the vintage of the existing analyses and changed design criteria for levees):

- Conduct an updated hydrologic analysis for the levee. Include the latest District hydrologic model for Centennial Wash and add the entire contributing watershed sub-basin for Centennial Levee. There is quite a change in hydrologic results between design flows from the Watershed Work Plan and the Cella Barr hydrologic model results.
- Conduct a crest level survey and collect cross sections of the levee/floodway system.
- Update the hydraulic steady flow analysis to ascertain existing freeboard available.
- Conduct an unsteady flow model (HEC-RAS) of the levee (since the levee is a structure that is 4.2 miles in length within lateral inflows). Ascertain existing freeboard and potential segments of the levee that potentially could overtop in large storm events.
- Conduct a levee break analysis. The downstream inundation area is presently active and dormant irrigated agriculture fields. The future land use plans from the District indicate that the land use around and downstream of the levee will be changed from agricultural to residential.
- Prepare a levee emergency action plan.
- Map the 100-year floodplain for the Centennial Levee floodway. Confirm District fee land versus the mapped floodplain.
- Consider upgrading the levee to meet current FEMA levee certification requirements. Existing lands downstream of the levee depend on the flood control protection provided by the levee. Rapid urbanization around and

downstream of the levee may require that ultimately the levee become FEMA certified.

- Evaluate what size upstream floodway would be required to remove the levee.
- Update the sediment yield analysis
- Evaluate raising the levee crest elevation to provide additional freeboard.
- Add ALERT gage to measure flow in floodway to top of levee crest elevation (at downstream end of levee).

3.3.4 Sedimentation

As stated, the design documents indicated that a low velocity of flow be maintained in the floodway to prevent scour in the floodway. A sediment yield analysis was completed (AWC, 1974). **Table 4** shows the results of the sediment yield analysis. Total sediment yield was estimated to be 0.91 acre-feet per year for the entire watershed. For a design life of 50-years this would result in a total expected sediment volume of 45.6 acre-feet. The design report also indicates that 2 feet of extra embankment height was added to Reach 1 to allow for the expected sediment deposition behind the levee. The sediment erosion yield rates for the soil types found within the four subbasins of the watershed were estimated by the AWC to range between 0.055 to 0.08 acre feet per square mile per year.

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 Commission 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 per square mile per year. 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 per square mile per year.

The Centennial Levee annual yield rate appears to be low compared to other sediment yield rates at District dams/watersheds other than Harquahala FRS and Saddleback FRS but appears to be consistent with the Harquahala FRS and Saddleback FRS. An average annual sediment yield rate within Maricopa County ranges from 0.2 to 0.3 acre-feet per square mile per year. Using a value of 0.3 acre-feet per square mile per year provides a 50-year sediment volume of 315 acre-feet. KHA recommends that an updated sediment yield analysis be conducted for Harquahala Valley watershed that includes the Centennial Levee, Harquahala FRS, and Saddleback FRS.

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.

3.4 Geological and Geotechnical Review

3.4.1 Geologic Review

The Centennial Levee 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. Centennial Levee lies in the valley between the Eagletail Mountains and the Big Horn Mountains.

A. Levee Centerline Geology

The description of the surface and subsurface conditions along the Centennial Levee are excerpted from a SCS memorandum dated December 1, 1983 from A.D. Elkin, Geologist to R. Arrington, State Conservation Engineer. Geologic profiles along the Levee centerline are presented in **Appendix A**.

The Reach 1 levee will cross a broad alluvial plain sloping towards the southeast. Flood flows across this plain are probably infrequent, and there are no well defined stream channels. Materials encountered in the test holes consisted of clay, silts, and sands of mostly alluvial origin except in the youngest surficial deposits where some reworking of the material by wind is evident. All of these deposits are of late Quaternary geologic age but can usually be divided into older or younger age by the presence of a distinct zone of material moderately cemented with calcium carbonate [caliche]. *Often the materials at the top of this zone have a light reddish brown color.*

<u>Younger Alluvial and Eolian Deposits</u>

The uppermost alluvial and eolian deposits in the area consist of cemented silty fine sands (SM) and silts with fine sand (ML) that occur in the surface throughout the length of Reach 1. They range in thickness from one foot at T.P. [Test Pit] *1017 to 9.5 feet at T.P. 1031 but average about 3 to 4 feet in thickness through most of the area. Only one density test was taken in the top three feet of these deposits (T.P. 1013) but the upper few feet generally appear to be soft and of relatively low density. Numerous animal burrows are present and a cutoff trench should be excavated at least through the upper three feet of these deposits.*

In places, the uppermost alluvial and eolian deposits lie directly on the older alluvial deposits or on younger alluvial deposits (SC, SM, CL, ML) that are slightly cemented with calcium carbonate. But along most of Reach 1 they overlie younger alluvial sands that appear moderately permeable and consist generally of fine to medium

sand with some coarse sand and some silt (SM, SP-SM, and SW-SM). The greatest thicknesses occur at T.P. 1014 (5.5 feet), T.P. 1018 (5.5 feet), T.P. 1025 (7 feet), T.P. 1029 (4.5 feet), T.P. 1030 (5.5 feet), and T.P. 1035 (8 feet). Design of both the canal and levee should take into account the possibility of water occasionally moving through these layers.

Density test taken in the more permeable sand layers of younger alluvium average about 101 pounds per cubic foot dry density. Density tests in the other younger alluvial and eolian deposits below a depth of three feet average about 96 pounds per cubic foot dry density but are quite variable, ranging from some low values below 90 pounds to some highs around 110 pounds [per cubic foot].

Older Alluvial Deposits

The older alluvial deposits are characterized by usually being moderately cemented with calcium carbonate. The cementation is enough to sometimes make excavation with a backhoe slow but not enough to stop excavation. Also, the cementation can usually be broken down by the application of water. In many places, the uppermost few feet of the older alluvium has a light reddish brown color. The materials in the older alluvium are mostly clays (CL) and silts (ML) with much fine sand, but there are also some clayey and silty fine sands (SM and SC).

Some of the older alluvial deposits are only slightly cemented with calcium carbonate and in some places layers of moderately permeable sands consisting of fine to coarse sand with some gravel and some silt are encountered. The average dry density of the slightly cemented material was about 98 pounds per cubic foot, and that of the moderately cemented materials was about 99 pounds [per cubic foot]. Again there was much variability with low ranges about 90 pounds and high ranges around 110 pounds [per cubic foot].

3.4.1.1 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 (AMEC 2002). However, this evaluation did not include the Centennial Levee. A seismic evaluation conducted by Euge, Schell, & Lam (1992), the Centennial Levee lies within the Sonoran Seismic Source Zone (**Appendix A**). 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 Centennial Levee, Sand Tank Fault, is about 60 miles southeast of the structure (**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 0.2 millimeters per year with the last occurrence less than 130,000 years ago (USGS, 2005). In **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 Centennial Levee.

3.4.1.2 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.

A. Groundwater in the Harquahala Groundwater Basin

The Centennial Levee 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 8000 deep in the center of the basin (Oppenheimer, 1980). 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).

B. Groundwater in the Project Vicinity

The closest well (B-02-09 7ABB) to the Centennial Levee with a long-term hydrograph record is approximately one mile east. The hydrograph show a water level decline of about 5 feet from 1999 through 2004. A hydrograph for well (B-02-10 14DCA) has a long term record through 1991 and show a continuing slight downward decline in the water level of about 5 feet. Two wells about 3 miles east and two well about 2.5 miles south of the Levee also show a decline in the water level. Hydrographs for wells within approximately three to four miles of the Centennial Levee were obtained from the Arizona Department of Water Resources (**Appendix A**). Nineteen hydrographs were obtained, with the oldest dating back to 1952. Ten of these hydrographs show a net decline or static groundwater levels over their period of record. Three of well

hydrographs show an increase in water levels. Six of the well hydrographs either have a short record period or no record. The wells in the vicinity of the Centennial Levee show a slow but continuous decline in groundwater levels suggesting the development of a pumping cone of depression is or has formed in the agricultural area southeast of the Levee.

3.4.1.3 Regional Subsidence

Prior to the utilization of groundwater in south-central Arizona, the water 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 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.

A. Study Area Subsidence

Historic National Geodetic Survey (NGS) level line data is not available in the vicinity of the Centennial Levee nor in the central portions of Harquahala Valley. Several survey monuments within the basin that are located along Interstate 10, along the CAP alignment, and scattered location throughout the basin; however, there is insufficient level data available to calculate the amount of subsidence that has historically occurred throughout the entire basin. The fact that earth fissures have opened up in the basin relatively recently (Roger's Fissure in 1997) indicates the basin sediment are being stressed, probably due to land subsidence in the basin.

3.4.1.4 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 **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 Centennial Levee vicinity (Oppenheimer, 1980). The Oppenheimer map estimated the depth to bedrock under the study area to be about 4,800 to 7,000 feet below ground surface, with the depth to bedrock depth increasing away from the mountain front (**Appendix A**). No unusual buried bedrock highs were interpreted within the project area from this data.

In **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 cover the northeast end of Centennial Levee, Geological Consultants, Inc. has extrapolated the contour lines into the project vicinity. As depicted in **Appendix A**, a relatively prominent gravity low exists centered along the axis of the basin. This gravity low could reflect the presence of a salt body similar to the Luke Salt Body known to be present in the West Salt River Valley, beneath Luke Air Force Base. Centennial Levee is located near the center of the basin. The Levee does not appear to be located over the bedrock pediment adjacent to the mountain front. In many of the basins of south-central Arizona, the position of the buried bedrock pediment acts as a focus for

earth fissure formation. However, as with the Luke Salt Body, the perimeter of the suspected salt body beneath Centennial Levee could represent a "relatively ridged" geologic structure boundary between the salt body and the basis fill over which the draped basin fill sediments might be stressed enough to induce earth fissure formation.

A. Known Earth Fissures in the Project Vicinity

There are three earth fissures reported in the Harquahala Valley. The closest fissure to the Centennial Levee is the Rogers fissure that was discovered in 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 1.6 miles west of the levee. The fissure made its first 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 noticeable 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.

Another earth fissure is approximately 1.9 miles east of the northeast end of Centennial Levee in Section 9, Township 2 North, Range 9 West. This fissure was first discovered in 1958, visible in an aerial photo. The fissure was examined in 1978 and appeared to have been dormant for many years (Graf, 1980). In 1983, Scott Geotechnical Services trenched this feature and did not confirm the presence of any subsurface earth fissure trace in the area, but this finding has not been confirmed by any other consultant or agency.

The third earth fissure was documented in 1961 in a farm field about 6.8 miles southeast of Centennial Levee 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.

Photo 1: View of Rogers earth fissure with gulley headcutting upslope along the fissure alignment. Photograph was taken October 16, 2001.

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. Photograph was taken October 16, 2001.

3.4.2 Geotechnical Review

The Centennial Wash Levee, referred to in the project files as the Centennial Reach of the Westside Canal, includes a southwest-trending levee and floodway extending from Interstate 10 approximately 5.1 miles to the Centennial Wash floodplain. Design documents indicate that the original design consisted of this section of the levee and floodway (Reach 1) as well as a southeast-trending section along Centennial Wash (Reach 2). However, Reach 2 was not constructed.

Centennial Levee is located immediately upstream of the Westside Main Canal, a portion of the Central Arizona Project (CAP) canal which conveys water from the Granite Reef Aqueduct to the Harquahala Valley Irrigation District. Upstream flood flows are diverted by the levee and floodway into Centennial Wash.

A. Regional Setting

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.

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 Township 2 North, Range 9 West. Farther west, near Sections 34, 35 and 36, Township 3 North, Range 11 West, 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-grained or moderately fine-grained surface soils and subsoils are present on the smoother slopes near the center of the valley. Coarse-grained or moderately coarse-grained soils are present on the upper fans of washes from the granitic mountains. Along the foot of the mountains, there is usually an area of shallow to moderately deep residual soils. These residual soils often have a medium-textured surface with gravel that is covered with dark desert varnish, and have slightly finer subsoils underlain at 12 to 28 inches by a strongly-cemented lime hardpan. Valley–fill alluvial soils originate in the granite, granite gneiss, schist, limestone, andesite, basalt, and shale rocks of the adjacent mountains. The soils in the plain are slightly to moderately erosive. Because the land surface is relatively flat and a sheet flow runoff condition prevails, erosion is generally not significant. Erosion is active in some of the channels and diversions constructed in and around the cultivated areas where flood flows are concentrated. Generally, the soils have a slow to very slow rate of water transmission and a slow to very slow infiltration rate when thoroughly wetted because of moderately-fine to fine texture or a layer that impedes downward movement of water.

3.4.2.1 Foundation Conditions

A Preliminary Geologic Investigation was conducted in 1974 (AWC 1974) along an early project alignment approximately 1 mile downstream from the final alignment. A subsurface investigation consisting of test pits and soil sampling was performed by SCS in 1983 along the final project alignment, near the centerline for the proposed Westside Main Canal. No formal report of the 1983 investigation was available for review during the preparation of this Structure Assessment Report; however, the test pit logs and soil test data were reviewed. It is not known whether a formal investigation report was prepared for the project. Because of the relocation of the levee alignment and because of the limited scope of the Preliminary Geologic Investigation, the information contained in this report is based on the results of the 1983 investigation. However, it should be noted that the results of the Preliminary Geologic Investigation are similar to the results obtained in 1983.

Centennial Levee crosses a broad, southeastward-sloping alluvial plain. Well-defined stream channels are not present and flood flows are generally infrequent. Subsurface materials are generally alluvial in origin, with some surficial eolian deposits, and consist primarily of silts and clays with some sands. Younger alluvial deposits overlie older alluvial deposits, with a zone of moderately (calcium carbonate) cemented materials marking the division between the deposits.

The 1983 soils investigation included installation of forty backhoe test pits and collection and testing of sixty-one soil samples along the proposed centerline of the Westside Main Canal. The results of laboratory testing on these soil samples are included in Attachment A of **Appendix B**. Backhoe test pits from the 1983 soils investigation indicate that "younger alluvium and eolian" deposits overlie "older alluvium" along the length of the levee alignment. The thickness of the "younger alluvium and eolian" deposits ranges from 2.0 feet to 15.0 feet, based on information contained in the test pit logs.

Foundation soil laboratory test data from the 1983 soils investigation are summarized in **Appendix B**. More than 3/4 of all soil samples were fine grained, mostly silts, clayey silts and clays. Although the majority of soil samples were classified as ML, they were reportedly expected to behave like fine-grained silty sand (SM), possibly susceptible to erosion. These soils were also characterized as having relatively strong bearing strength for fine-grained soils (SCS, 1984). It was also noted that field classification of these soils based on the #200 sieve (fine-grained vs. coarse-grained) was very difficult. This was reflected in the soil descriptions contained on the test pit logs; thirty-three of the sixty-one soil samples were classified incorrectly in the field. The field classifications generally described the soils as being coarser-grained than the laboratory classifications.

All soil samples contained some clay particles: between 1% and 37% with average of 16% by weight, suggesting that clay mineral behavior will influence the soils. Palygorskite (also known as attapulgite), a tubular clay mineral, was reportedly present in nearly all soils samples, most of which have approximately equal amounts of palygorskite, montmorillonite and illite (SCS, 1984). Little was known of the engineering properties of palygorskite at the time Centennial Levee was designed and constructed (SCS, 1984). Palygorskite is a hydrated magnesium aluminum silicate similar to bentonite in its ability to absorb water (SCS, 1984) but differs from bentonite in its thin elongate chain type structure. The clay minerals present in the soil samples are moisture sensitive and tend to swell when wet and shrink when dry.

1. Younger Alluvium and Eolian Deposits

The results of the laboratory testing of the younger alluvial soils are summarized in **Appendix B**. The uppermost alluvial and eolian deposits consist of uncemeted silty fine sands (SM), silts with fine sand (ML), clayey silt (CL-ML), and clay (CL). These deposits have an average thickness over the length of Centennial Levee of approximately 3 to 4 feet, but were reported to be present to depths of as much as 15 feet (Test Pit TP 1031; SCS, 1983a). The younger alluvial deposits have average in-field densities of approximately 96 pounds per cubic foot (pcf) and average fines content of 53%. The fines were generally non-plastic or had low plasticity index (PI) values.

The strength of the younger alluvial soil was evaluated using four triaxial shear tests (one from the centerline of the embankment and three from the borrow areas). The tests were performed wet of optimum at densities of 90% of Standard Proctor density. The specimens were back-pressure saturated and pore pressures were measured so that effective stress parameters as well as total stress parameters could be measured. The total

stress cohesion of the younger alluvial soil samples ranged from 400 pounds per square foot (psf) to 675 psf and the effective cohesion ranged from 100 psf to 250 psf. The total stress angles of internal friction ranged from 11.5 degrees to 16.5 degrees and the effective stress angles of internal friction ranged from 28.5 degrees to 33 degrees.

Swell tests were performed on three younger alluvial soil samples from the borrow areas. Tests were performed on samples compacted at optimum moisture content and densities of 90% and 95% of Standard Proctor density. The swell measured at 90% Standard Proctor density ranged from 0.44% to 0.84% and the swell measured at 95% Standard Proctor density ranged from 0.52% to 1.32%.

2. Older Alluvium

The results of the laboratory testing of the older alluvial soils are summarized in **Appendix B**. The older alluvial deposits are generally moderately cemented with calcium carbonate. The older alluvial material consists primarily of clays (CL), silts (ML), and clayey silt (CL-ML) with some fine-grained silty sand (SM). The amount of cementation is variable and, in some places, layers of moderately permeable fine- to coarse-grained sands with some gravel were encountered (SCS, 1983a). The average dry density of the cemented alluvial deposits was approximately 102 pcf and average fines content of 62%. The fines were generally non-plastic or had low PI values.

The strength of the older alluvial soil was evaluated with one triaxial shear test using the same methods as described above. The total stress cohesion and effective stress cohesion values obtained from tests on the older alluvial soil samples were 575 psf and 0 psf, respectively. The total stress angle of internal friction and the effective stress angle of internal friction of the older alluvial soils were 33 degrees and 37 degrees, respectively.

3.4.2.2 Embankment

The Centennial Levee embankment section was based on the design criteria from the Soil Conservation Service National Engineering Handbook, Section 2, Engineering Design Standards, Part 1: Dike and Levee for a Class II dike. Centennial Levee was designed as a homogenous earth embankment with 4H:1V upstream and 3H:1V downstream slopes, maximum height of 10.5 feet, 12-foot crest width and a continuous central foundation cut-off trench with a depth of at least 6 feet and base width of 12 feet. A typical cross-section of the embankment is shown as **Appendix B**.

The Centennial Levee crest elevation varies from 1271.80 feet above mean sea level (amsl) at Station 0+00 to 1268.92 feet amsl at Station 224+00. The elevation slope is generally 0.0002 ft/ft toward Centennial Wash, with elevation transitions at three locations along the embankment. Between Station 58+80 and Station 59+00 the elevation increases 1 foot; between Station 156+80 and Station 157+00 the elevation increases ½ foot; and between Station 196+80 and Station 197+00 the elevation increases 1 foot. The embankment crest and floodway elevations are represented graphically on **Appendix B**.

A. Embankment Materials

The materials used to construct the embankment were derived from local borrow sources in the vicinity of the Levee. Eight of the sixty-one soil samples collected and tested during the 1983 soils investigation were identified by SCS as being representative of borrow materials. Four of the representative borrow material samples were collected from along the Westside Main Canal centerline and four were collected from the floodway area. The results of the laboratory testing are summarized in **Appendix B**. The as-built plans indicate that the embankment was to be constructed of soils containing not less than 15% passing the No. 200 sieve. No as-built soil data for the levee were available for review for this Assessment.

According to the soil test data, the borrow materials consist of silts (ML), clays (CL), clayey silts (CL-ML) and silty sands (SM). The average fines content for the borrow materials was 60%. The fines were generally non-plastic or had low PI values. The permeability of the borrow soil samples (measured at 90% Standard Proctor density) ranged from 0.01 ft/day to 1.2 ft/day. Pinhole tests performed on two of the borrow area samples indicate that the soil is not dispersive. Calcium carbonate was present in all eight borrow area samples between 2% and 9%. This reportedly represented lime nodules dispersed in the soil matrix or possibly some cementation (SCS, 1984).

Standard Proctor test data for the borrow area soils indicated that the maximum dry density of the soil ranges from 112.5 pcf to 121.5 pcf and can be achieved over a moisture content of between 9.5% and 14.5%. The SCS recommended that the project specifications require "a workable mix with a minimum moisture content of 2% less than the optimum moisture" (SCS, 1984).

Results of the swell tests performed on the alluvial materials collected from the borrow areas are summarized on in **Appendix B**. SCS reported that the swelling would likely result in the levee surface swelling and shrinking in response to changes in moisture content and that this may cause or aggravate embankment cracking (SCS, 1984). Furthermore, one clay sample (#1005.1) swelled more when compacted at 95% Standard Proctor density than at 90% Standard Proctor density, indicating that the soils may react poorly to overcompaction. Based on this information, it was recommended that the levee not be used as a haul road and that the final density/compactive effort be limited (SCS, 1984). This is not an accepted method for mitigating moisture sensitive soils. Because construction inspection reports were not available for review during the preparation of this report, it is not known whether this recommendation was implemented during construction.

B. Embankment Construction

Foundation treatment recommendations called for removal of the loose surface soils to a depth of at least 6 feet, with the final depths to be determined by the engineer after inspection of the materials encountered. The as-built plans indicate that the surface soils

were removed to depths of between 3 and 9 feet. According to the SCS (1984), the geologist recommended removal of at least 3 feet of surface soils to destroy animal burrows and consolidate surface soils. There is no indication in the files that consideration was given to removing all of the less dense younger alluvial soils to prevent potential collapse consolidation and the often associated embankment cracking. This, taken together with the depth of the cutoff indicated on the as-built plan set (3 to 9 feet), suggests that potentially collapsible younger alluvial soil deposits may be present beneath the Levee. No construction inspection reports were available for review during the preparation of this report.

C. Embankment Penetrations

In 1986, US Sprint Communications Company constructed a cable crossing of Centennial Levee. The construction consisted of removing approximately 120 feet of the embankment near the left abutment, within the existing Arizona Department of Transportation right-or-way, placing a cable within a 4-inch black iron pipe conduit and reconstructing the compacted embankment.

The cable crossing design plan included provision of a filter diaphragm surrounding the pipe conduit. The design specified a 4 foot by 4 foot, 3 foot thick compacted sand diaphragm located beneath the upstream side of the levee crest. The sand was specified to be ASTM C-33 material. Beneath the conduit, low density or other unsuitable material was to be excavated and replaced with a minimum of 1 foot of select compacted material. The pipe bedding was specified to be 0.37 foot (same as outside diameter of pipe) select bedding material compacted by saturation and internal vibration. The cable crossing design plans were approved by ADWR and the District.

3.4.2.3 Floodway

A floodway was constructed upstream from Centennial Levee to direct floodwaters toward Centennial Wash. The floodway channel centerline is 260 feet upstream from the centerline of the Levee. The floodway was constructed with 50H:1V side slopes. The channel bottom is 80 feet wide and slopes toward Centennial Wash at 0.0002 ft/ft.

The floodway was designed to accommodate a relatively low flow velocity to prevent scour in the easily erodible soils (SCS, 1976). The floodway includes a soil cement section from Station 215+87 to Station 225+75 for erosion protection. The soil cement extends 415 feet from the upstream toe of the Levee and has a compacted thickness of at least 6 inches. A rock sill was specified at the floodway outlet. The rock sill is 505 feet long, perpendicular to the floodway at Station 225+75. The sill consists of a 6 foot deep and 10 foot wide section filled with "4 inch to 24 inch rock sill material" according to the construction plans. The rock sill was presumably constructed to prevent headcutting and scour along the floodway as water discharges to Centennial Wash.

3.4.2.4 Original Slope Stability Analysis

No record of slope stability analyses performed during levee design was available for review. It is not know whether these analyses were performed for design.

3.4.2.5 Geotechnical Recommendations

A. Supplemental Geotechnical Analyses

Potentially collapsible younger alluvial soil deposits may be present beneath the levee. The remaining collapse potential and risk of wetting of potentially collapsible soils in the foundation could be evaluated through appropriate sampling, testing, and analysis. The foundation soils, particularly those beneath the crest and upstream section of the levee could be sampled using split-spoon (California) samplers, pitcher sampler, or block samples and collapse consolidation tests could be performed to determine the current state of stress and collapse potential.

B. Documentation of Slope Stability Factors of Safety

Although at the time Centennial Levee was designed, the Federal Emergency Management Agency (FEMA) did not have a comprehensive levee policy, an interim levee policy had been established in 1981 (FEMA, 1981). The interim policy indicated that guidance on new levee construction should be addressed by referring to the minimum requirements contained in the United States Army Corps of Engineers (USACE) "Design and Construction of Levees" (EM 1110-2-1913). Current USACE guidelines (as of April 2000) include provisions for slope stability analysis.

Under reasonable loading conditions for Centennial Levee, it is expected that both upstream and downstream slopes will be stable. However, documentation of slope stability factors of safety for specified loading and design criteria is not available at the time of this review. Slope stability analyses are recommended to document factors of safety for Centennial Levee. In addition, this information would be required for FEMA certification of Centennial Levee. The stability analyses and factors of safety required by the USACE for existing levees are summarized in **Appendix B**:

4.0 PRELIMINARY LIST OF POTENTIAL FAILURE MODES

The Failure Mode and Effects Analysis team developed a preliminary list of potential failure modes for Centennial Levee. The purpose of the list of potential Failure Modes was to:

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

A detailed Failure Modes and Effect Analysis (FMEA) was conducted subsequent to the potential Failure Modes identification. The preliminary list of potential failure modes and items are as follows:

1. **Embankment Overtopping**: Overtopping of the levee embankment could lead to erosion and formation of a breach. This scenario could be due to high local runoff in one or more of the contributing washes; or due to the floodway outlet channel being plugged by debris; or due to a failure of an upstream dam such as the Harquahala FRS or Tiger Wash Detention Basin.

2. **Internal Erosion/Piping During a Major Flood Event**. Transverse cracking exists at several locations in the embankment and there is no filter zone to protect the embankment from erosion that will result during major flood events.

3. **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 embankment is founded upon Holocene soils that are erosive.

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

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

6. **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. A fissure has been identified in the general vicinity of the levee

7. **Failure Mechanisms Associated with Seismic Event**. A seismic event in the vicinity of Centennial Levee has the potential for exacerbating existing transverse/longitudinal cracks.

8. **Other Considerations**: This section addresses issues that are not directly related to a failure of the levee but which nonetheless may be relevant to the FMEA:
    - Foundation treatment
    - Compaction
    - Use of construction materials (borrow areas)
    - Placement of embankment lifts

5.0 Land Ownership and Land Use

This section discusses data on the existing and future land use upstream and downstream of Centennial Levee. Land use information for Centennial Levee was collected to allow a qualitative assessment of the consequence of levee failure. The scope of the study required review of 2 miles upstream and downstream of the levee.

5.1 Source of Data

The Flood Control District of Maricopa County provided aerial photography, information regarding land ownership and land use information. **Figure 5 (Figures Appendix)** provides a map demonstrating land ownership at Centennial Levee.

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 land owned by the US Department of the Interior-Bureau of Land Management or by the Arizona State Trust. This information is depicted on **Figure 6 (Figures Appendix)** and is summarized as follow:

- The land between the Central Arizona Project (CAP) canal and the Levee (contributing watershed) is all public owned land. The area is dissected by Interstate-10, which is a major intercontinental transportation route.
- A portion of the Centennial Reach of the Westside Canal is immediately downstream of the Levee. The Westside Canal delivers irrigation water from the CAP to agricultural lands irrigated by the Harquahala Valley Irrigation District.
- Local irrigation infrastructure downstream of the levee.
- There are no inhabited structures identified within a two-mile radius of the levee.
- No new residential development was recorded near this levee.

5.4 Proposed Land Use

Future land use maps were provided by 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 Centennial Levee. 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 Centennial Levee and the current full cash value of those properties.

5.6 Population Densities

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

5.7 Critical Facilities

Critical facilities exist within approximately a 2-mile radius from the Centennial Levee. These facilities include Interstate 10, the Westside Canal, and Westside Canal siphon crossing of Centennial Wash.

6.0 FIELD INSPECTION

6.1 Previous Inspections

Kimley-Horn and Associates, Inc. reviewed previous field inspection reports for Centennial Levee obtained from the project files at the District. Regular inspections have been completed on the structure since 1988. The reports collected from these sources date to March 16, 1998. A total of 29 regular, maintenance, and special inspection reports from March 1988 to January 2005 were reviewed as part of this task. An inspection was completed for the Individual Structures Assessment in August 2005 and the report is provided in **Appendix E**.

Key findings documented in the above mentioned field inspection reports include the following:

- Since 1988 the annual inspections have noted regular inspection and maintenance tasks for repairing fences and controlled access points, controlling unwanted vegetation; and controlling rodents and insects.
- Slope erosion and rilling on both upstream and downstream slopes of the levee is prevalent.
- Gravel mulch was proposed for the embankment slopes in October 2004 in the maintenance inspection.
- Embankment depressions were first noted at Station 32+56 in the 2003 maintenance inspection report.
- Evidence of longitudinal cracks was first noted in 2003.
- Possible transverse cracks were first noted in February 2002.
- Minor erosion damage from flows has occurred several times such as in 1990, 2000, and 2002. After each flow event, damages were repaired through the District's routine maintenance program.
- Erosion rilling was first noted in 1988 by the NRCS. Establishment of vegetation by seeding was an internal recommendation from the NRCS. The District report of 1989 indicates that seeding was completed in November 1988. The success rate of the seeding was estimated to be 10% and no further vegetation development was initiated.

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 levee project features are reviewed and evaluated to determine if deficiencies exist at the levee and associated project features. The examination was conducted by walking the length of the levee 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 August 11, 2005 by the following technical examination team:

Technical Examination Team

Tom Renckly, P.E.	Structures Branch Manager, Flood Control District of Maricopa County
Dan Lawrence, P.E., R.L.S.	Dam Safety Engineer, Flood Control District of Maricopa County
Bill Leal,	Dam Safety Technician, Flood Control District of Maricopa County
Mike Luecker, P.E.	State Hydraulic Engineer, Natural Resources Conservation Service
Robert Eichinger, P.E., CFM	Project Manager, Kimley-Horn and Associates
Lu Gan, E.I.T.	Analyst, Kimley-Horn and Associates
Ken Euge, P.G.	Principal Geologist, Geological Consultants Inc.
Jason Williams, R.G.	Geologist, Geological Consultants Inc.
Dean Durkee, Ph.D., P.E	Principal Geotechnical Engineer, Gannett-Fleming
Frances, Ackerman, E.I.T., R.G.	Geotechnical Engineer, Gannett-Fleming

Operational Summary: Floodwaters from the 21 square mile contributing watershed are diverted by the Centennial Levee, flow in the floodway, which is just upstream of the Levee, and ultimately discharges into Centennial Wash.

Inspection Frequency: Inspections are completed by District Operation and Maintenance Staff at least annually and 'window' inspections are completed quarterly. The levee is also inspected after significant rainfall or flow events as reported by the Districts' Flood Warning System.

Maximum Water Surface Elevations: Since construction of the streamflow gage in 2003, the maximum flows measured in the floodway were 126 cfs in August of 2003 and 13 cfs in July 2003. Measurements on the stage gage were 2.2 and 1.0 feet for these storms, respectively. Any flow in the floodway with a stage less than 0.65 feet is not measured by the gage. High water marks from an October 2000 rainfall event indicate that the 70% of the levee had water near the toe. No estimate of the flow rate was given.

Distress Observations Corrected or Operation and Maintenance Conducted Since Last Inspection: One new crack at Reach 2 Station 30+32 was documented in January 2005. At Reach 1 Station 19+96, a new three inch diameter hole was found in the upstream shoulder. A new possible transverse crack was found at Stations 1+70 and 4+32 on the upstream mid-slope and upstream shoulder, respectively.

Past Distress Observations Not Yet Corrected:

- Fill erosion rills on upstream and downstream slopes with compacted fill, if greater than 12-inches deep;
- Fill erosion holes on crest with compacted fill, if greater than 12-inches deep;
- Initiate gravel mulch recommendations resulting from Phase I Structures Assessment;
- Work with HVID to ensure maintenance of the canal does not adversely impact the levee.

District Operation and Maintenance Responsibilities: An operation and maintenance agreement was made on August 28, 1984 between the NRCS (SCS) and the Flood Control District. The Flood Control District is the local sponsor of the federally funded project. The District maintains operational control of the Centennial Levee and is responsible for the structural and functional integrity of the levee and floodway, 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: Longitudinal cracks/transverse cracks, depressions, and erosion holes were observed on the crest of the levee. There were also signs of rodent activity, which could be the initiator of some of the crest holes.

Upstream Slope: Small animal burrows were scattered on the slope face. There was no evidence of seepage, undermining, settlement or sloughing. There is erosion gullies located on many parts of the upstream slope. Recommendations for gravel mulching of the upstream slope will be provided. Longitudinal cracks/transverse cracks, depressions, and holes were observed on the slope face. The vegetation maintenance program of the District should continue along the levee.

Downstream Slope: Similar to the upstream slope, small animal burrows were scattered on the slope face. There was no evidence of seepage, undermining, settlement or sloughing. There are erosion gullies located on many parts of the upstream slope. Recommendations for gravel mulching of the upstream slope will be provided. Longitudinal cracks/transverse cracks, depressions, and holes were observed on the slope face (see inspection report for specific locations). There is a triangular shaped earth-lined channel downstream of the toe that is designed to carry flow away from the toe. Water ponds in the ditch.

Instrumentation: Instrumentation on Centennial Levee includes a rain gage (5120) and a streamflow gage (5123). Both of the gages are part of the District's flood warning system. The rain gage was first installed in 1984 and the streamflow gage was installed in 2003.

Outlet Structure: The grade control structure at the outlet is in good condition. The soil cement plating is in good condition.

Floodway: The floodway is in good condition. The District regularly inspects the vegetation density in the floodway and maintains the channel according to the design intent.

6.3 Signs of Distress

Based on the field inspection performed by the Kimley-Horn team, previous inspection reports by the District and NRCS and the results of FMEA for the Levee, major signs of distress in the form of confirmed transverse and longitudinal cracking that have been identified along Centennial Levee.

6.4 Safety Deficiencies

Based on the field inspection performed by the Kimley-Horn team, previous inspection reports by the District and NRCS and the results of FMEA, no safety deficiencies have been identified for Centennial Levee.

6.5 Conclusions

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

6.6 Recommendations from Inspection

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

- Work with Harquahala Valley Irrigation District (HVID) to facilitate better communication and understanding of maintenance activities on the levee and canal system;
- Work with HVID to ensure that the sediment removal from the v-notch ditch between the downstream toe and the canal does not adversely impact the downstream toe of the levee;
- Continue active vegetation management program;
- Continue active rodent management program;
- Develop mitigation strategies to protect the structure against transverse and longitudinal cracking;
- Map all cracks on set of as-built plans and profiles as well as aerial photo of levee. Continue to map cracks after all levee safety inspections. Monitor, over time, reaches of levee where there has been a noted propensity of cracks.
- Ensure the levee is on a rotation to survey the crest regularly.

7.0 FAILURE MODES AND EFFECTS ANALYSIS

7.1 Introduction

Kimley-Horn and Associates, Inc. and the Failure Modes and Effects (FMEA) team conducted a failure modes and effects analysis for Centennial Levee. 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 experienced 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 September 7, 2005. The workshop participants included:

Debora Miller, P.E., PhD, FMEA Facilitator, Gannett Fleming, Inc.
Dan Lawrence, P.E., R.L.S., Flood Control District of Maricopa County, Dam Safety Engineer
David Jensen P.E., Session Recorder, Kimley-Horn and Associates, Inc., Project Engineer
Dean Durkee, P.E., PhD, Geotechnical Engineer, Gannett Fleming, Inc.
Frances Ackerman, R.G., E.I.T., Gannett Fleming, Geotechnical Engineer
Ken Euge, R.G., Geological Consultants, Geology

The detailed Failure Mode and Effects Analysis Report is provided in **Appendix F** of this report.

The purpose and scope of the FMEA exercise was to:

- Identify potential site-specific failure modes for the levee.
- 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 levee is defined as the uncontrolled release due to breach of the levee.
- 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 D**.

7.3 Failure Mode Categories

The FMEA process as currently undertaken and carried out does not provide quantitative likelihood estimates for the identified potential failure modes. The FMEA does put the potential failure modes into categories as follows:

Category I - *Highlighted 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 was evident, fundamental flaw or weakness is identified and conditions and events leading to failure seemed reasonable and credible) are highlighted.*

Category II - *Failure Modes Considered but not Highlighted – These are judged to be of lesser significance and likelihood. Each failure mode is described and included with reasons for and against the potential occurrence of the 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 on significance thus a dam safety investigative action or analyses can be recommended. Because these actions are required before resolution these may also be highlighted.*

Category IV - *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 for the failure mode, or the failure mode is clearly non-credible or reasonable to postulate.

7.4 FMEA Results

The FMEA for Centennial Levee did not identify any Category I potential failure modes and only two (2) Category II failure modes. These two Category II failure modes were assigned a low likelihood of occurrence and a medium consequence. The following presents the Category II, III, and IV potential failure mode descriptions (see **Appendix D** for the detailed FMEA report).

Failure Mode H1 - Overtopping Of Levee For Runoff Events Exceeding 100-Year Event And Exceeding Freeboard Leading To Erosion And Breach Of The Levee (Category II).

Failure Mode Description: Flood flows contributing to the levee/floodway are greater than the capacity of the floodway and levee. The depth of flows against the levee is sufficient to overtop the levee initially at low spots and cause concentrated flow erosion. Erosion continues on the levee crest until a breach occurs releasing a portion or all of floodway flows from behind the levee. Potential causative mechanisms of overtopping could be due to failure of an upstream dam (Tiger Wash Detention Dam or Harquahala FRS); or a rainfall-runoff event greater than the design event (say 200-year storm on the watershed).

Failure Mode S1 – Erosion And Breach Of Levee During Runoff Events Due To Flow Through Existing Transverse Cracks In Levee Embankment (Category II).

Failure Mode Description: Flood runoff event of sufficient duration and depth of flow against levee occurs. Water flows through a transverse crack and the resulting highly localized flow velocities erode the soils materials in the crack. As the crack widens, more flow passes through the crack until embankment breaches releasing a significant portion or all the floodway flows through breach to the downstream inundation area.

Failure Mode H2 - Scour And Erosion Of Levee Due To Flows In Floodway Starting At Upstream Toe Leading To Breach (Category III).

Failure Mode Description: Flood flows in the upstream floodway flow against upstream levee toe. This results in scouring or lateral erosion of upstream toe eventually causing caving of embankment materials. Continued lateral erosion and caving results in breach of levee. Floodway designed for low flow velocity for design storm event. Other flows and concentrated flows not evaluated.

Failure Mode H3 – Embankment Erosion Due To Dynamics Of Flows Coming In From Steeper (High Velocity) Upstream Drainages Impinging On Levee And Floodway Channel (Category III).

Failure Mode Description: Concentrated high velocity flows emerging from upgradient steep drainage channels direct flows straight into the upstream slope of the levee. The force of water locally erodes the upstream slope of the levee causing caving. Continued erosion and caving results in breach of the levee. No evaluation could be located in the project literature to address this condition.

Failure Mode S2 - Erosion And Breach Of Levee Due To Flow Through Transverse Cracks Or Earth Fissures Extending Under Structure Foundation (Category III).

Failure Mode Description: An existing transverse crack or earth fissure extends through the levee foundation. During periods of flow in the floodway concentrated seepage flows through the crack, leading to erosion and widening of the crack. Continued widening of the foundation crack occurs during long sustained flood events, or progressively over multiple events over a longer period of time. Levee embankment cannot support itself over continued widening of the crack and caves/sloughs into eroded foundation. Levee material is washed away downstream resulting in breach of the levee.

Failure Mode H4 - Overtopping Due To Sediment Build Up From Major Drainage (E.G. Tiger Wash) Raising The Bottom Of Floodway Channel In Isolated Locations (Category IV). (Failure mode description not formulated by FMEA team – ruled out)

7.5 FMEA Limitations

Centennial Levee was constructed pursuant to a relatively modern levee design. Construction appears to have been without any particular issues. The levee has performed normally and satisfactorily for 23 years. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exist for all earth structures (levees and 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 levee is about to fail or even necessarily that there is a levee 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 levee should be aware.

No Category I potential failure modes were identified by the FMEA team. Two Category II potential failure modes were identified and discussed. Normal operations of the levee will discharge flood waters into Centennial Wash. The flows will enter Centennial Wash and then spread out over the Centennial Wash floodplain. There is very little structure development downstream of Centennial Levee at this time.

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 levee safety site inspection for the dam. This section provides a summary of the recommendations.

8.1 Hydrologic and Hydraulic Recommendations

(1) Conduct an updated hydrologic analysis for the levee. Include the latest District hydrologic model for Centennial Wash.
(2) Conduct a crest level survey and collect cross sections of the levee/floodway system.
(3) Update the hydraulic steady flow analysis to ascertain existing freeboard available.
(4) Conduct an unsteady flow model (HEC-RAS) of the levee (since the levee is a structure that is 4.2 miles in length within lateral inflows). Ascertain existing freeboard and potential segments of the levee that potentially could overtop in large storm events.
(5) Conduct a levee break analysis. The downstream inundation area is presently a mix of active and dormant irrigated agriculture fields. The future land use plans provided by the District indicate that the land use around and downstream of the levee will be changed from agricultural to residential.
(6) Prepare a levee emergency action plan.
(7) Map the 100-year floodplain for the Centennial Levee floodway. Confirm District fee land versus the mapped floodplain.
(8) Consider upgrading the levee to meet current FEMA levee certification requirements. Existing lands downstream of the levee depend on the flood control protection provided by the levee. Rapid urbanization around and downstream of the levee may require that ultimately the levee become FEMA certified.
(9) Evaluate what size upstream floodway would be required to remove the levee.
(10) Update the sediment yield analysis.
(11) Evaluate raising the levee crest elevation to provide additional freeboard.
(12) Add ALERT gage to measure flow in floodway to top of levee crest elevation (at downstream end of levee).

8.2 Geotechnical and Geological Recommendations

A. Supplemental Geotechnical Analyses

Potentially collapsible younger alluvial soil deposits may be present beneath the levee. The remaining collapse potential and risk of wetting of potentially collapsible soils in the foundation could be evaluated through appropriate sampling, testing, and analysis. The foundation soils, particularly those beneath the crest and upstream section of the levee could be sampled using split-spoon (California) samplers, pitcher sampler, or block samples and collapse consolidation tests could be performed to determine the current state of stress and collapse potential.

B. Documentation of Slope Stability Factors of Safety

Although at the time Centennial Levee was designed, the Federal Emergency Management Agency (FEMA) did not have a comprehensive levee policy, an interim levee policy had been established in 1981 (FEMA, 1981). The interim policy indicated that guidance on new levee construction should be addressed by referring to the minimum requirements contained in the United States Army Corps of Engineers (USACE) "Design and Construction of Levees" (EM 1110-2-1913). Current USACE guidelines (as of April 2000) include provisions for slope stability analysis.

Under reasonable loading conditions for Centennial Levee, it is expected that both upstream and downstream slopes will be stable. However, documentation of slope stability factors of safety for specified loading and design criteria is not available at the time of this review. Slope stability analyses are recommended to document factors of safety for Centennial Levee. In addition, this information would be required for FEMA certification of Centennial Levee. The stability analyses and factors of safety required by the USACE for existing levees are summarized in **Appendix B**:

8.3 Additional Recommendations from Inspection Report and FMEA Report

(1) Coordinate with HVID to facilitate required level of maintenance activities on the levee and canal system, particularly for the V-shaped channel.
(2) Coordinate with HVID to ensure that the sediment removal from the V-shaped channel between the downstream toe and the canal does not adversely impact the downstream toe of the levee.
(3) Continue active vegetation management program.
(4) Continue active rodent management program.
(5) Survey the crest of the Levee to begin a subsidence monitoring program for the levee.
(6) Develop mitigation strategies to protect the structure against transverse and longitudinal cracking.

(7) 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 the levee where there has been a noted propensity of cracks.

(8) Determine if a gravel mulch or vegetative seeding would be beneficial for the crest and embankment slopes.

(9) Ensure the levee is on a rotation to survey the crest regularly with the Flood Retarding Structures.

(10) Evaluate levee to determine if the levee meets current FEMA criteria for certification.

(11) Re-station the crest of levee. The station markers on the crest of the levee should match the as-built plans.

(12) Update crest survey. Survey Centennial Levee crest on the same frequency as significant hazard dams.

(13) Survey profile and cross-section of floodway. This will allow an estimate of the capacity of the floodway.

(14) Update hydraulics to confirm flood channel capacity. Check sensitivity to n-values. No subsequent analysis was found since the original design documents and analysis.

(15) Check for possible correlation between location of transverse cracks and washes. Note the location of washes into the floodway and the extension of the washes downstream beyond the Westside Canal. Note the locations of observed or suspected transverse cracks on a set of as-built plans. Attempt to derive a correlation of wash location and crack location in the embankment.

(16) Update slope stability analysis. Potential weakening of soils on saturation could alter the results of a slope stability analysis. Conduct an updated or new slope stability analysis.

(17) Need to locate flood hydrographs for design event. The design flood hydrographs were not located in the project literature prior to the FMEA (but have subsequently been found and reviewed).

(18) Revetments near the wash crossings. Revetments could be beneficial on the upstream slope of the levee embankment where large washes could impinge and cause concentrated erosion. Erosion control measures or stabilized inlets from the washes into the floodway may be investigated and evaluated.

(19) Operation and maintenance update. Verify floodway operation and maintenance standard procedures (sediment buildup, excessive vegetation, etc).

(20) Future crack investigations. Follow up on Phase I Structure Assessment recommendations to conduct additional crack investigations. Transverse and longitudinal cracks have been identified at the structure.

(21) Earth fissuring studies. Consider addition studies to investigate earth fissuring in the central Harquahala Valley. Earth fissures have been noted within 2-miles of the levee. Consider baseline elevation survey tied to reliable benchmark. Incorporate studies with studies for Harquahala FRS and Saddleback FRS.

(22) Future FMEA considerations. Consider updated FMEA if development is proposed to occur downstream of levee. Downstream development would cause increased consequences from potential levee failure.

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Table 1. Centennial Levee Crest Elevations (NGVD29).

Beginning Station	Ending Station	Distance (ft)	Top of Dam Design Crest Elevation (ft)
Reach 1			
0+00	58+80	5,880	1271.80-1270.39 (0.00024 ft/ft slope)
58+80	59+00	20	1270.39-1271.38
59+00	156+80	9,780	1271.38-1269.04 (0.00024 ft/ft slope)
156+80	157+00	20	1269.04-1269.53
157+00	196+80	3,980	1269.53-1268.58 (0.00024 ft/ft slope)
196+80	197+00	20	1268.58-1269.57
197+00	224+00	2,700	1269.57-1268.92 (0.00024 ft/ft slope)
Siphon Protection Dike			
0+00 (Reach 1 224+00)	3+45.24	345	1268.92-1263.0
3+45.24	25+55	2,210	1263.0-1257.31 (0.00257 ft/ft slope)
25+55	41+18	1,563	1257.31
Total Embankment Length (ft)		**26,518**	

Table 2. Centennial Levee Physical Data Summary.

Item	Unit	Design Value
Class of Levee		II [1]
Drainage Area (Uncontrolled)	square miles	22.0
Average Curve Number (1-Day AMC II)		81
Elevation-Top of Levee	ft	1271.8-1257.31 [2]
Maximum Height of Levee	ft	10.5
Volume of Fill	yd^3	325,000
Length	ft	26,518 (5.02 miles)
Top Width	ft	12
Upstream Slope (waterside)	H:V	4:1
Downstream Slope (landside)	H:V	3:1
Sediment (50-Year)	acre-feet/year	1
100-Year 24-Hour Rainfall Depth	inches	4.1 [3]
Design Runoff	acre-feet	116.7
Design Runoff	inches	2.19
Design Storm Event		100-year
Design Storm Duration	hours	24
Freeboard Minimum	feet	2
Peak Inflow	cfs	7,540
Floodway		
Bottom Width	feet	80
Approximate Top Width	feet	415
Sideslopes	H:V	50:1
Invert Slope	feet/feet	0.00024
Aged *n*-value		0.035
Maximum Floodway Velocity	feet per second	1.91

[1] Designed to Class I standard (100-year, 24-hour with 2 feet of freeboard)
[2] Elevation Data is based upon NGVD 29 vertical datum
[3] Estimated Rainfall Depth. NRCS Design Documents utilized runoff depth of 2.19 inches

Table 3. NRCS Levee Design Criteria.

Item	NRCS Original Design Criteria		NRCS Current Criteria	FEMA Criteria	Comment/Remarks
Publications and References for NRCS and FEMA Criteria			NRCS Conservation Practice Standard Arizona-Dike (CODE 356)		
1. Dike Classification			Class I		
2. Mannings "n"	NEH-5, Supplement B				
	Open Channel Standard				
	EDS-FWS				
3. Height of Dike:	a. Water-Surface Aged Condition	Open Channel Standard	100-Year WSEL	100-YearWSEL	
	b. Wave Height (Unsteady Flow)	EDS-FWS			
	c. Superelevation	EDS-FWS			
	d. Sediment Load	EIS P. 72-74			
	e. Freeboard-EDS-FWS	EDS-FWS	Minimum 2 feet or 1-foot + Wave Height	3 feet	
	f. Settlement-Dike Standard	Dike Standard	No less than 5% total height		
4. Cross Section/Embankment:	a. Top Width	12 feet	Minimum 10 feet for Class I dike height less than 15 feet		12 feet for ease during construction
	b. Side Slope/Stability	Dike Standard, EDS-FWS	Perform slope stability analysis. Maximum slope of 4:1 on water side for unprotected earth		
	c. Banquette (Landside)	Dike Standard	Required for excessively porous fills or poor foundation conditions. Required for construction access and added stability if channel crossings are saturated. Top width must equal height of dike at a minimum. Minimum height is 1-foot above existing ground.		
	d. Foundation Cutoff	Dike Standard	Required if foundation materials are pervious and subject to piping.		
	e. Berm (Waterside)	Dike Standard	Minimum berm width of 18 feet for dike height greater than 6 feet. Drain system can be used if necessary and require a graded sand-gravel filter.		
	f. Ramps and Turnouts	Dike Standard			
5. Design Procedures:	a. Water-Surface Profile	TR-25, Chapter 5			
6. Transitions	TR-25, Chapter 5, Appendix I				
7. Stability	a. Flow Conditions	Open Channel Standard			
	b. Procedures (to set curves, slopes, channel cross-section	TR-25, Chapter 6			
8. Riprap	ENG-PO-18				
9. Filter	Soil Mechanics Note 1				

Table 4. Sediment Yield Results (AWC, 1974).

Drainage Basin	Soil Unit	Erosion Rate (af/mi^2/yr)	Area (mi^2)	Gross Erosion (af/yr)	Delivery Rate (%)	Sediment Yield (af/yr)
I-1	2	0.055	6.08	0.334	77	0.257
I-2	2	0.055	8.63	0.475	75	0.356
I-3	2	0.055	2.28	0.125	82	0.103
I-4	2	0.055	3.0	0.165	80	0.192
	4	0.08	1.0	0.08		
		Total	**21**	**1.19**		**0.91**

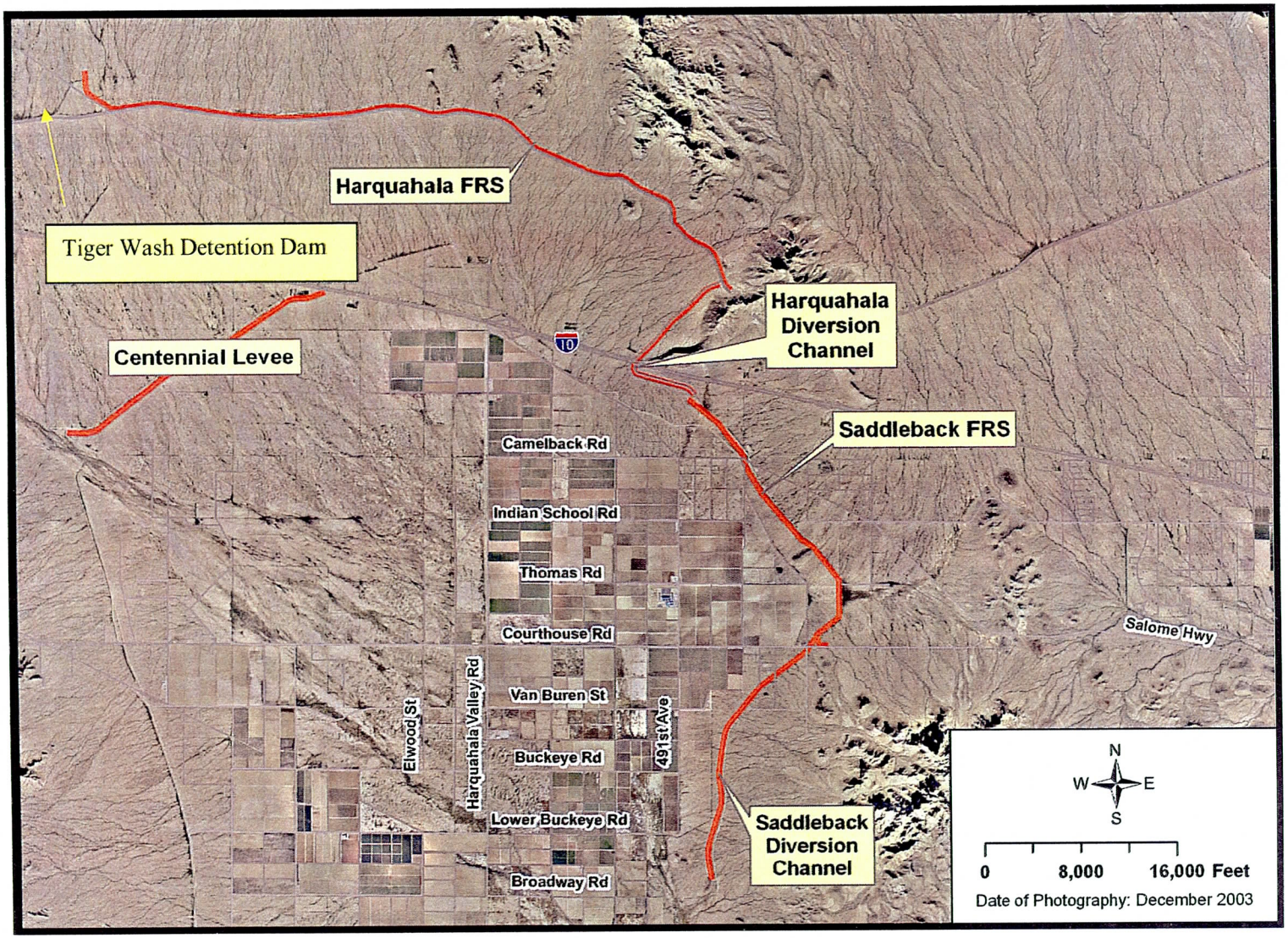
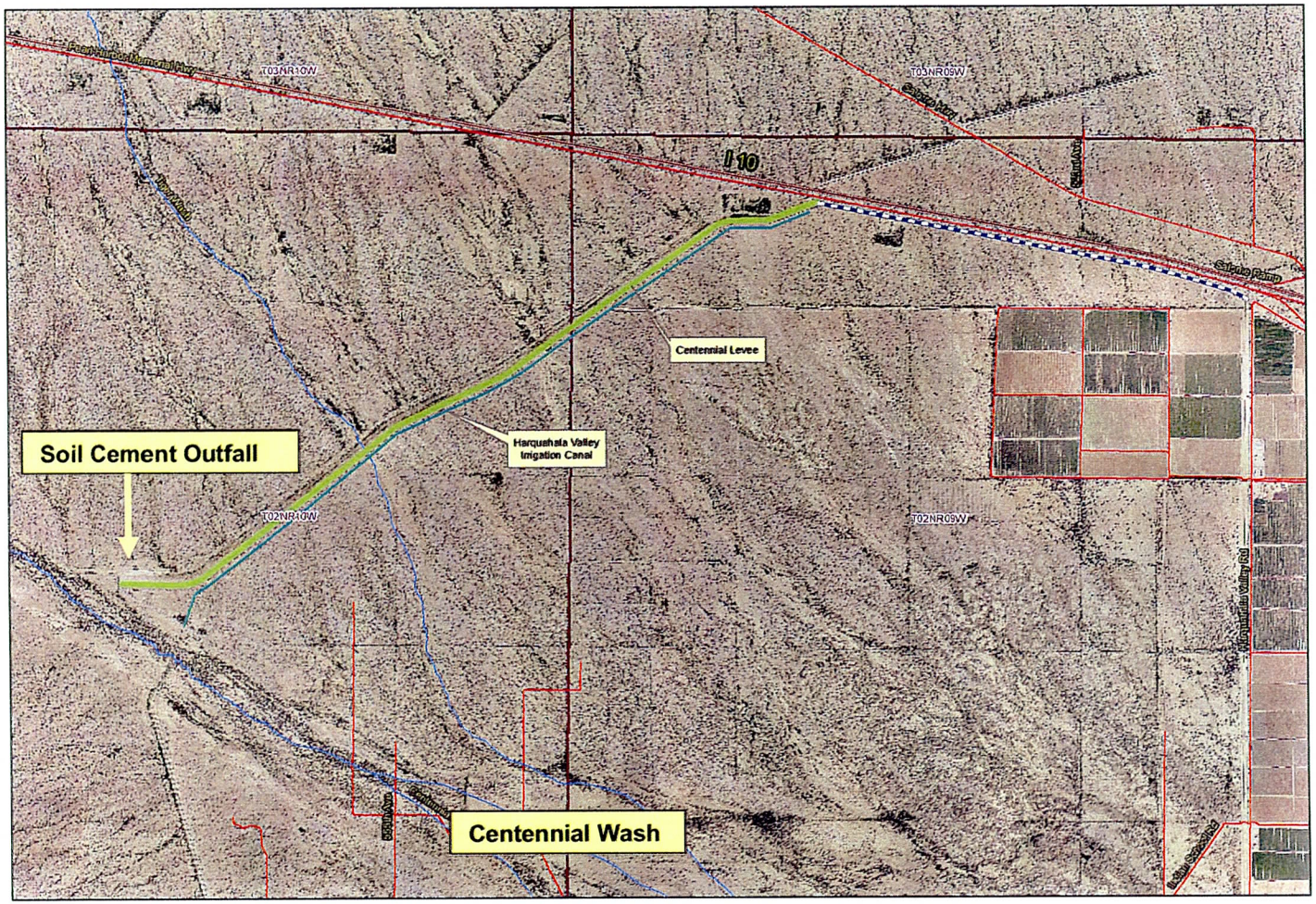


Figure 1. Location Map
KHA Project No. 091131011

Kimley-Horn and Associates, Inc.

Flood Control District of Maricopa County

Centennial Levee



N

Legend

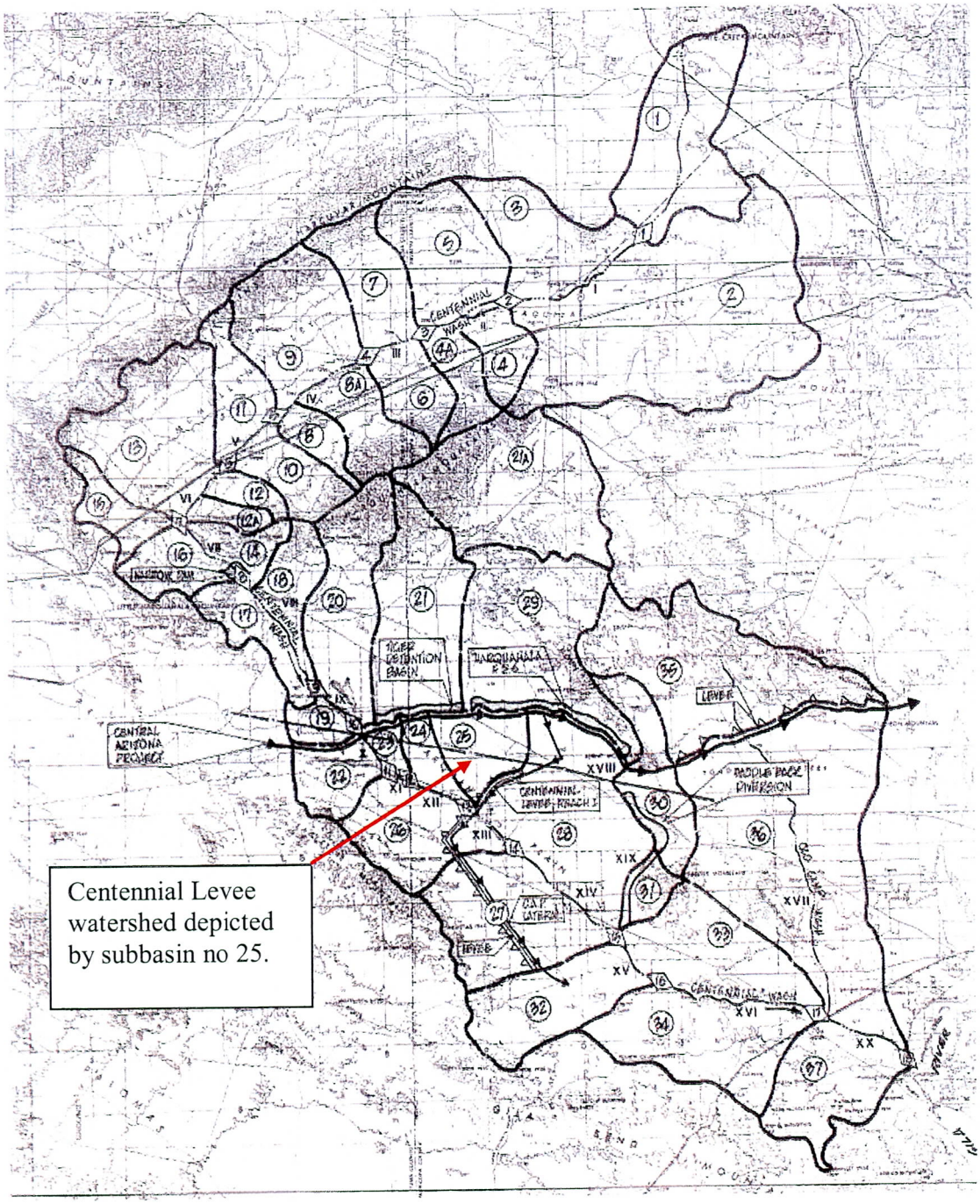
- Rivers
- Township & Range
- FCDMC Access Roads

Date of Photography 12/2003

0 0.25 0.5 1 Miles

Figure 1A. System Map Centennial Levee
KHA Project No. 091131011

FCD2003C015
PCN: 050.03.01



Reference: Centennial Wash Flood Insurance Study Maricopa County, Arizona. Figure 3. Topographic Map Showing Flood Control Structures along Centennial Wash. Hydrologic Analysis of the Centennial Wash in Maricopa County, Arizona. Prepared for FEMA by Cella Barr Associates, September 1988, Revised January 1989.

Figure 1B. Drainage Area Map for Centennial Wash
KHA Project No. 091131011

FCD2003C015
PCN: 050.03.01

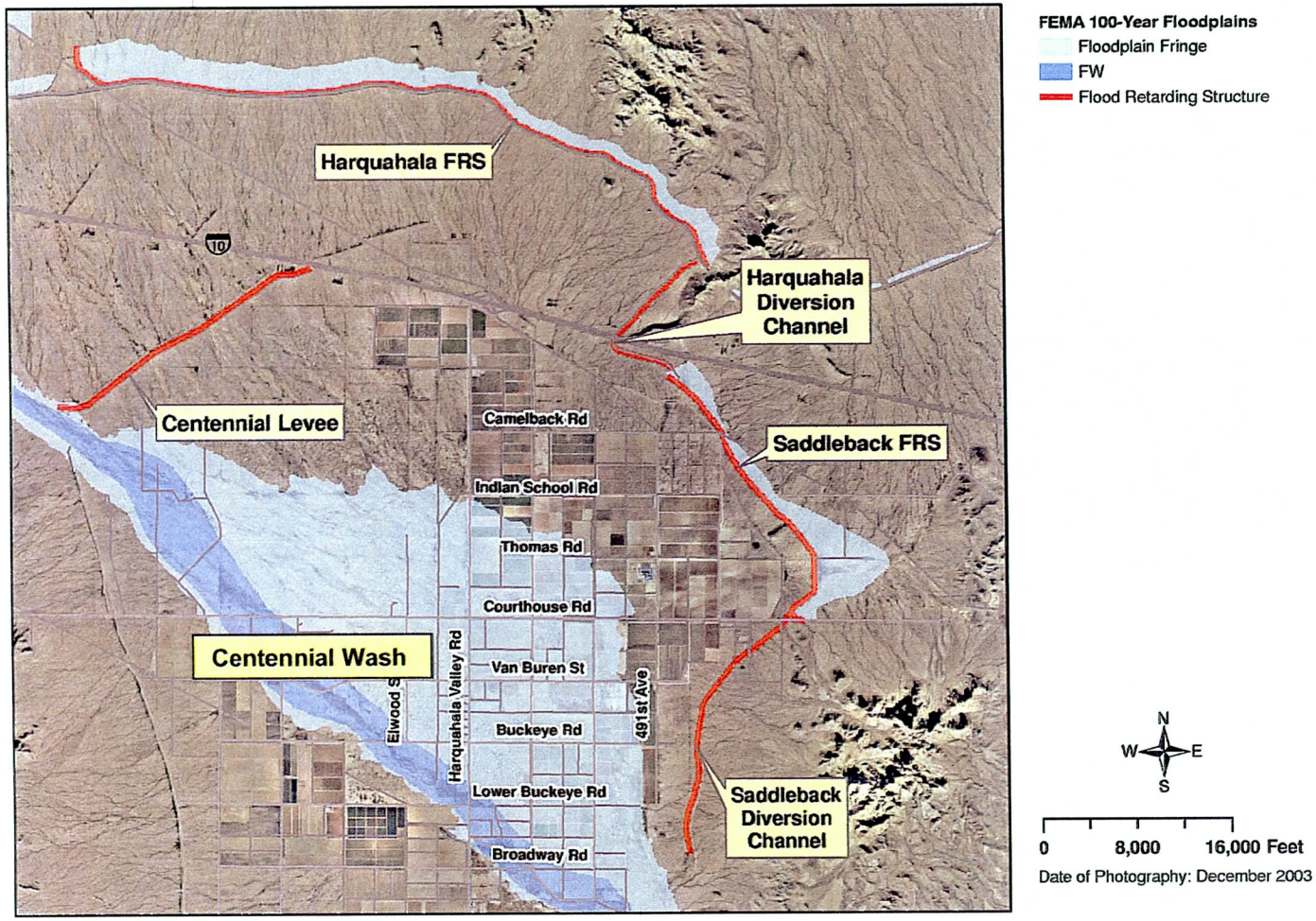


Figure 2. Harquahala Valley Floodplains Map.
KHA Project No. 091131011

Kimley-Horn and Associates, Inc.

Flood Control District of Maricopa County

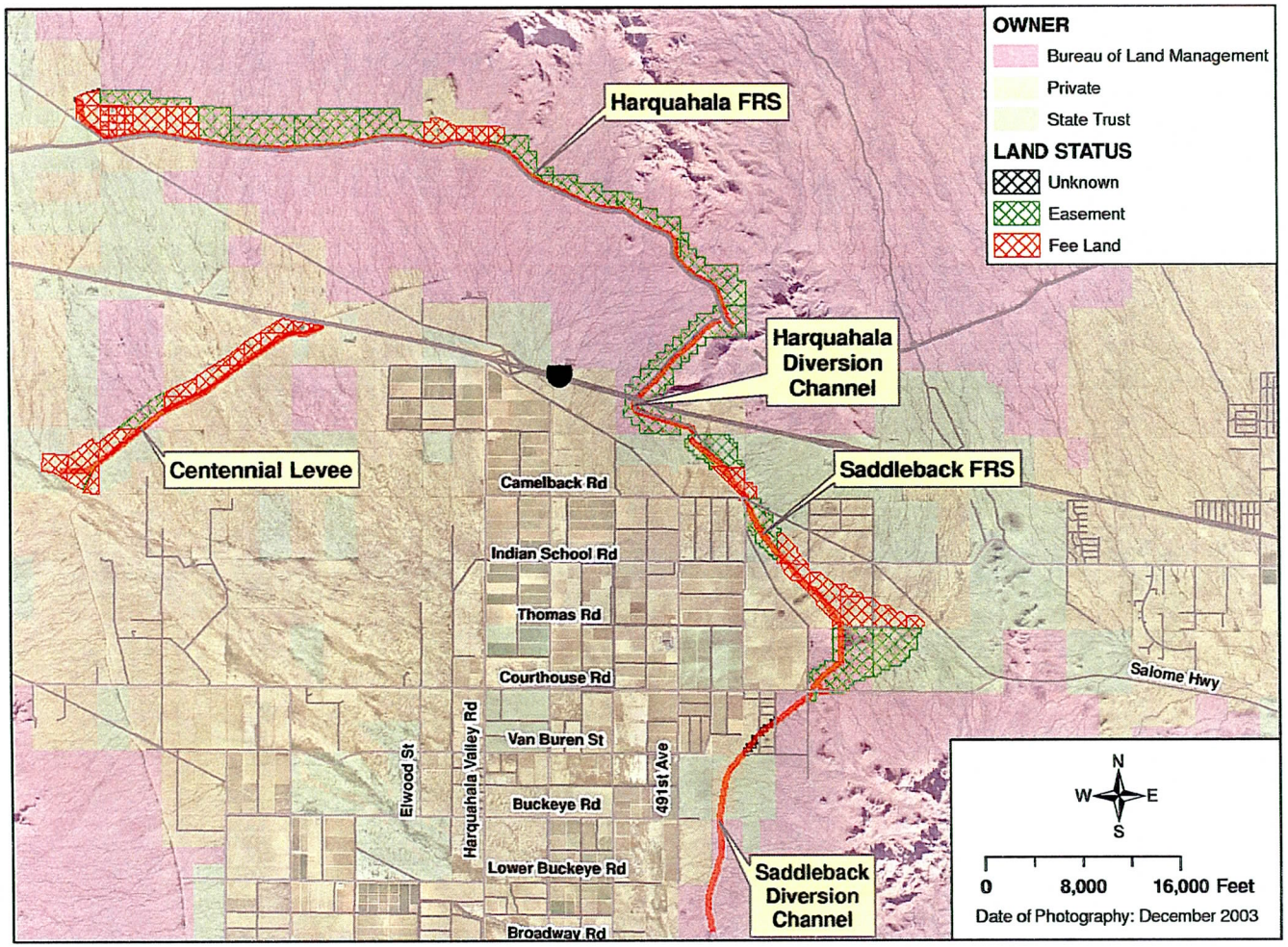


Figure 3. Landownership Map
KHA Project No. 091131011

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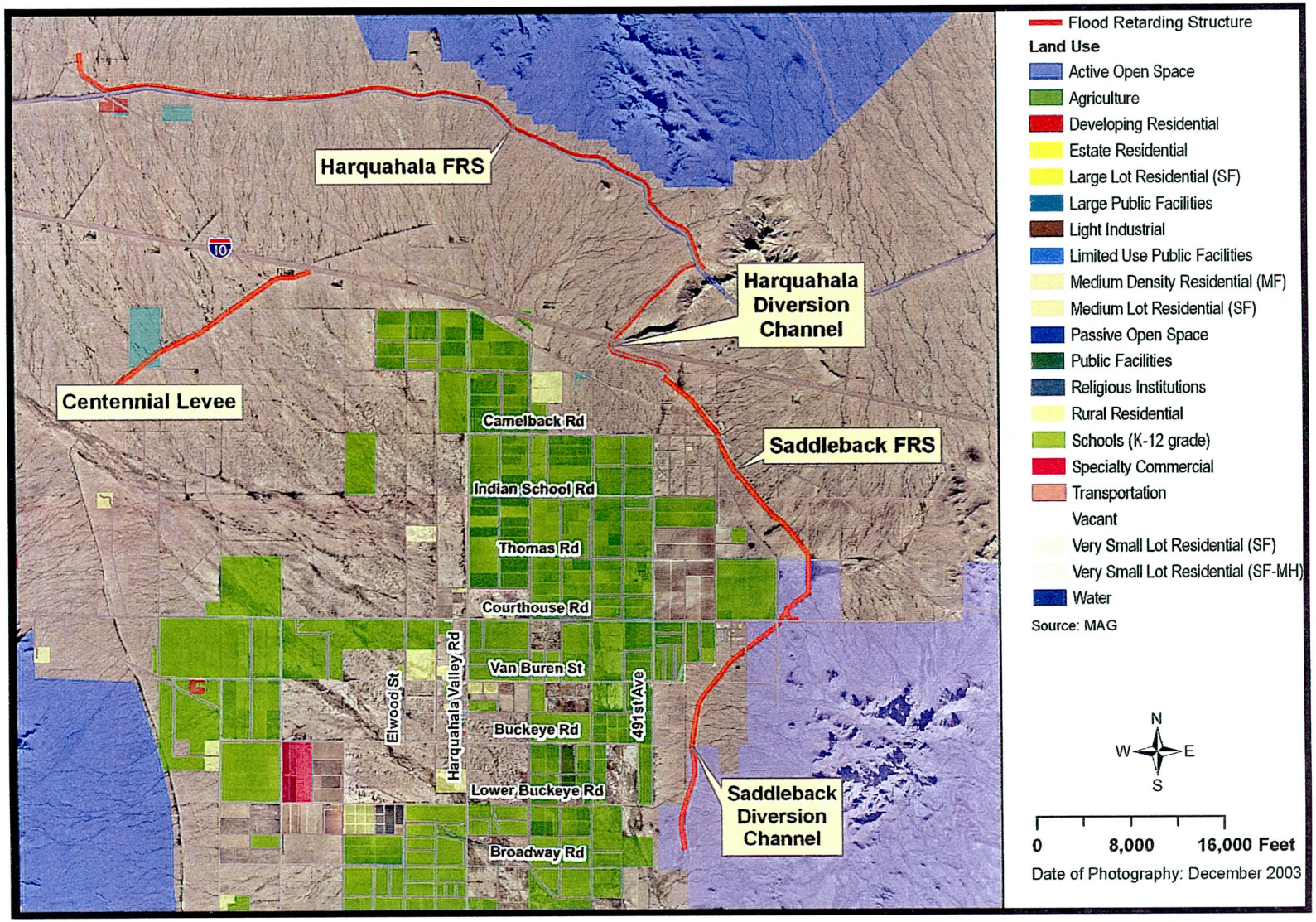

Figure 4. Current Land Use Map.
KHA Project No. 091131011

FCD2003C015
PCN: 050.03.01

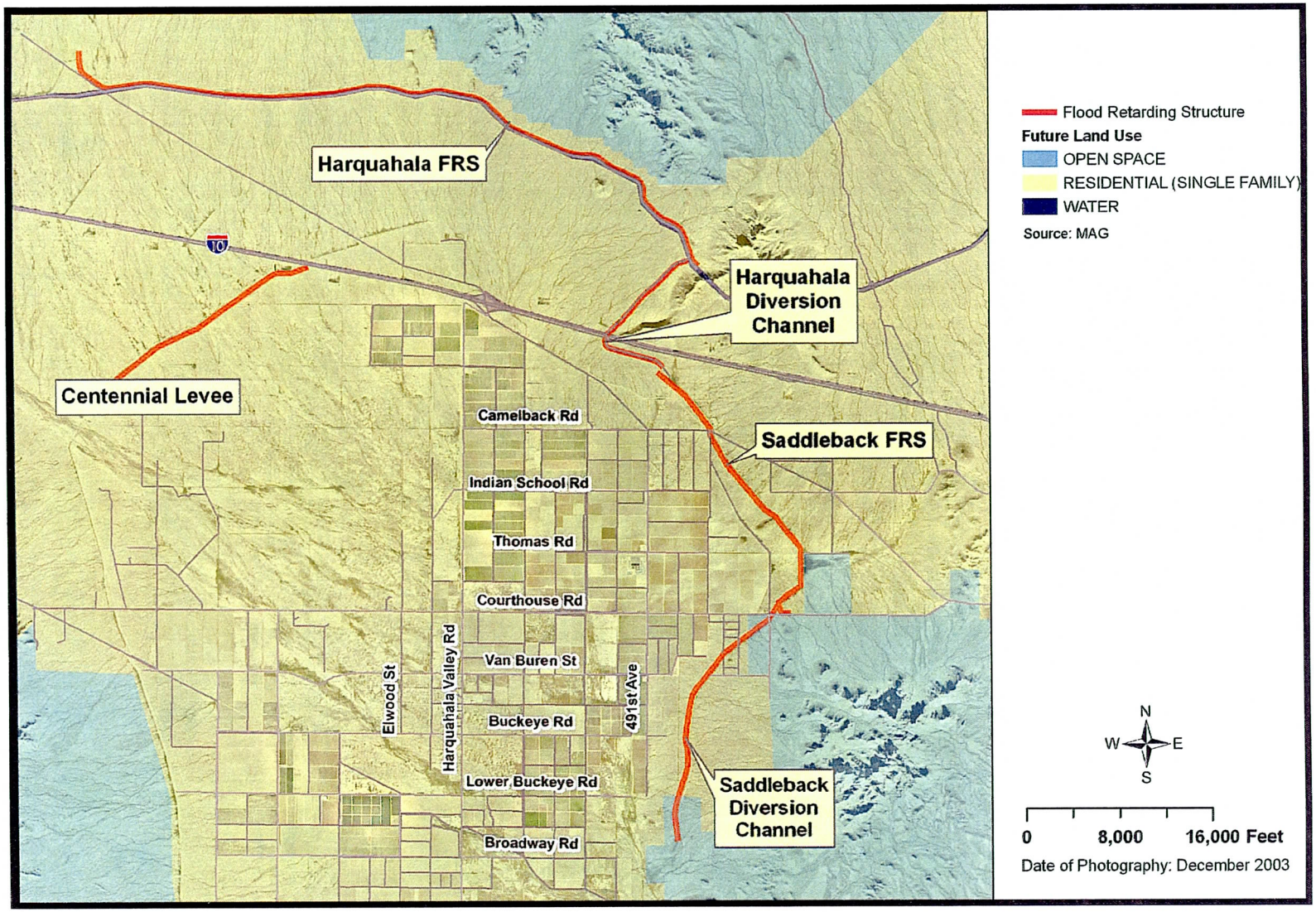

Figure 5. Future Land Use Map.
KHA Project No. 091131011

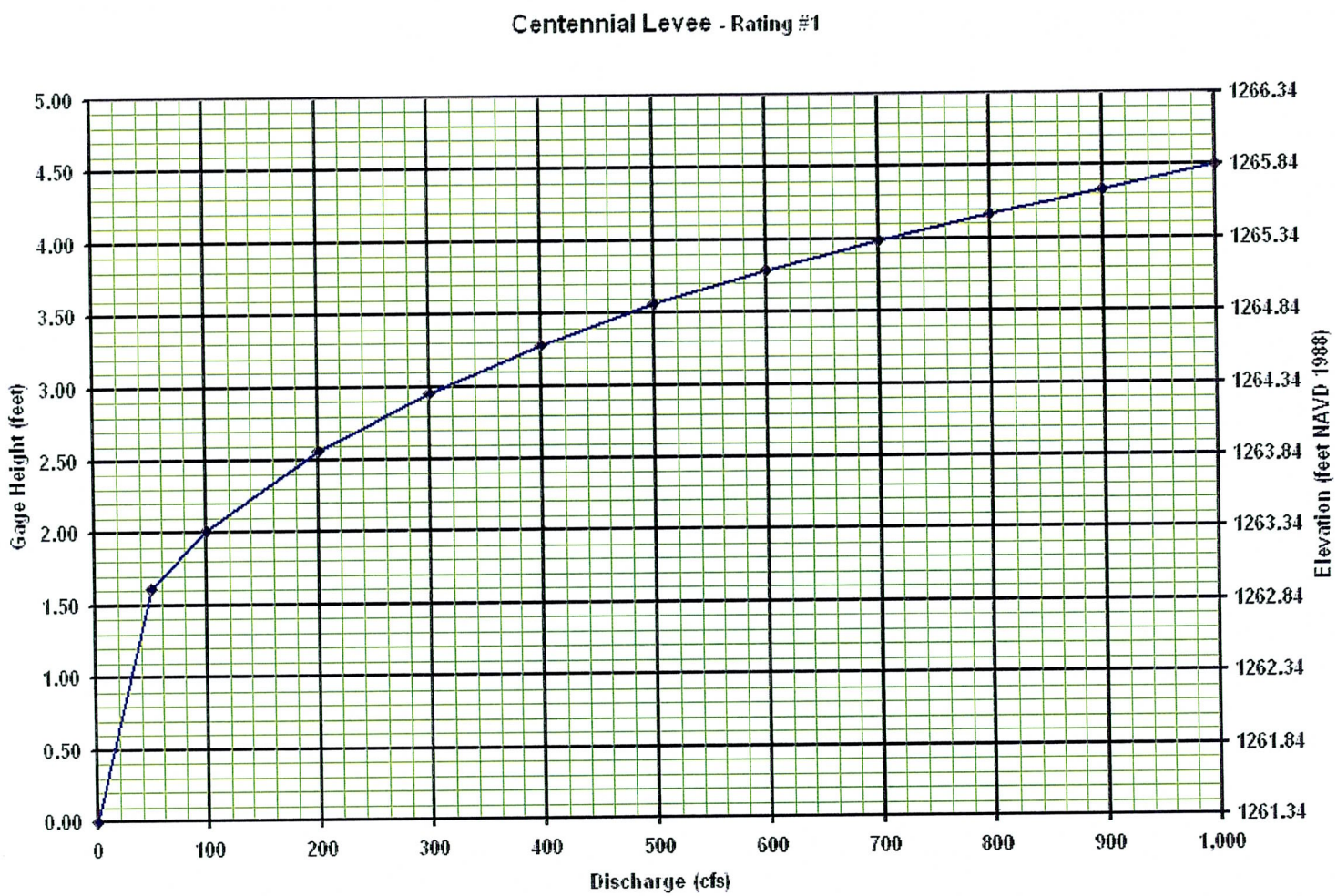

Figure 6. Centennial Levee Gage Rating Curve.

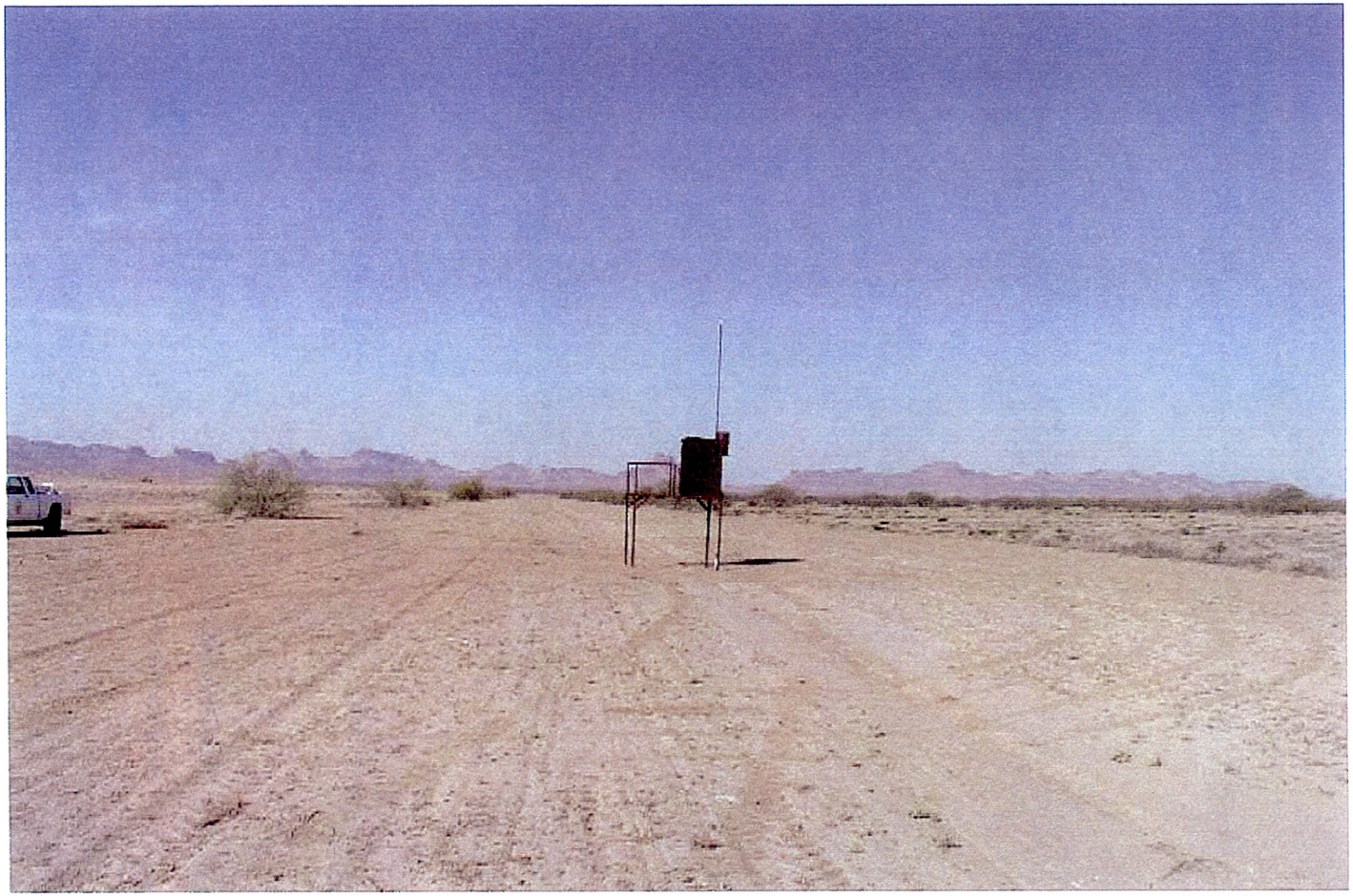

Figure 7. Centennial Levee ALERT Gage.

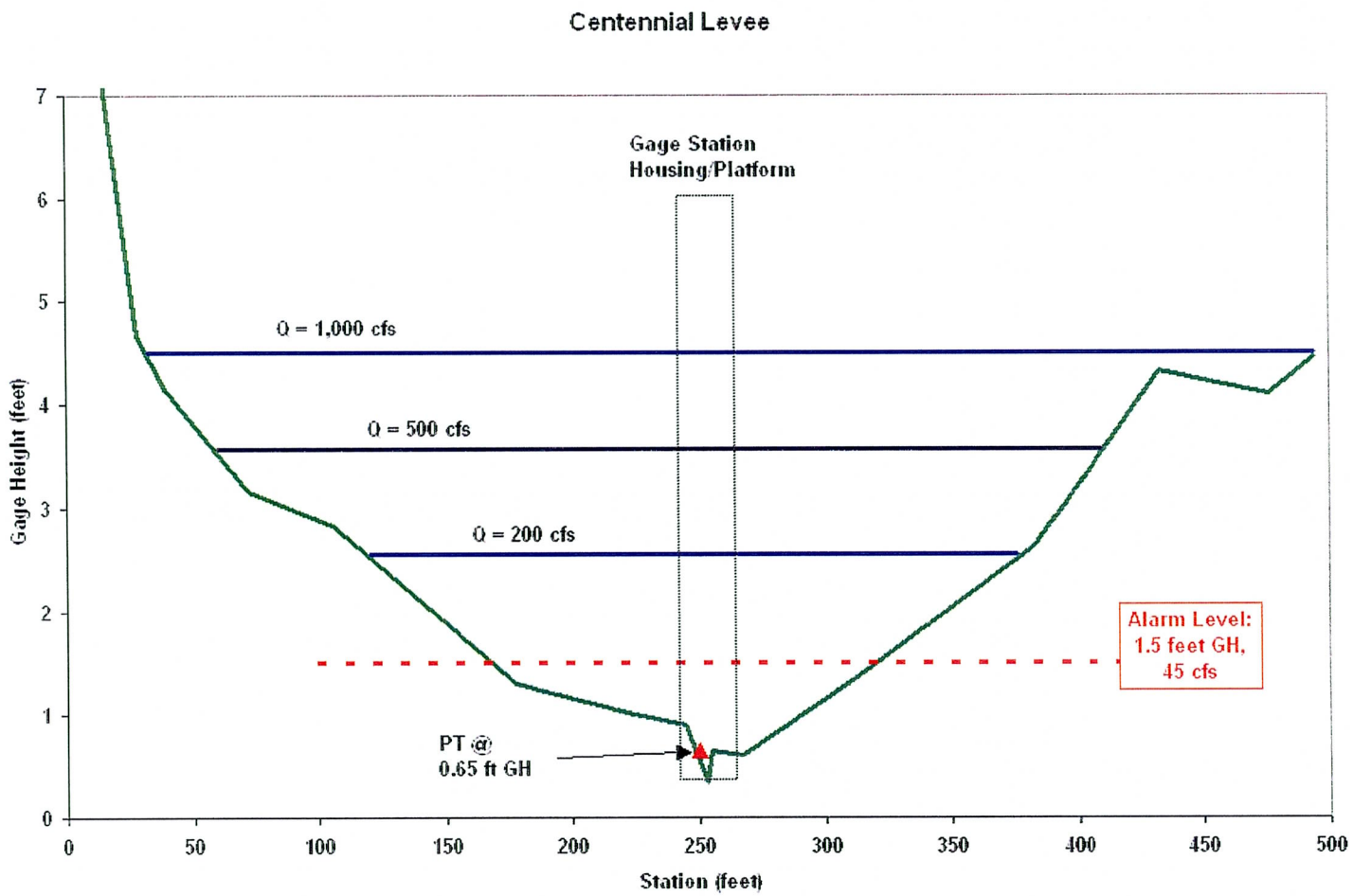

Figure 8. Centennial Levee ALERT Alarm Level.

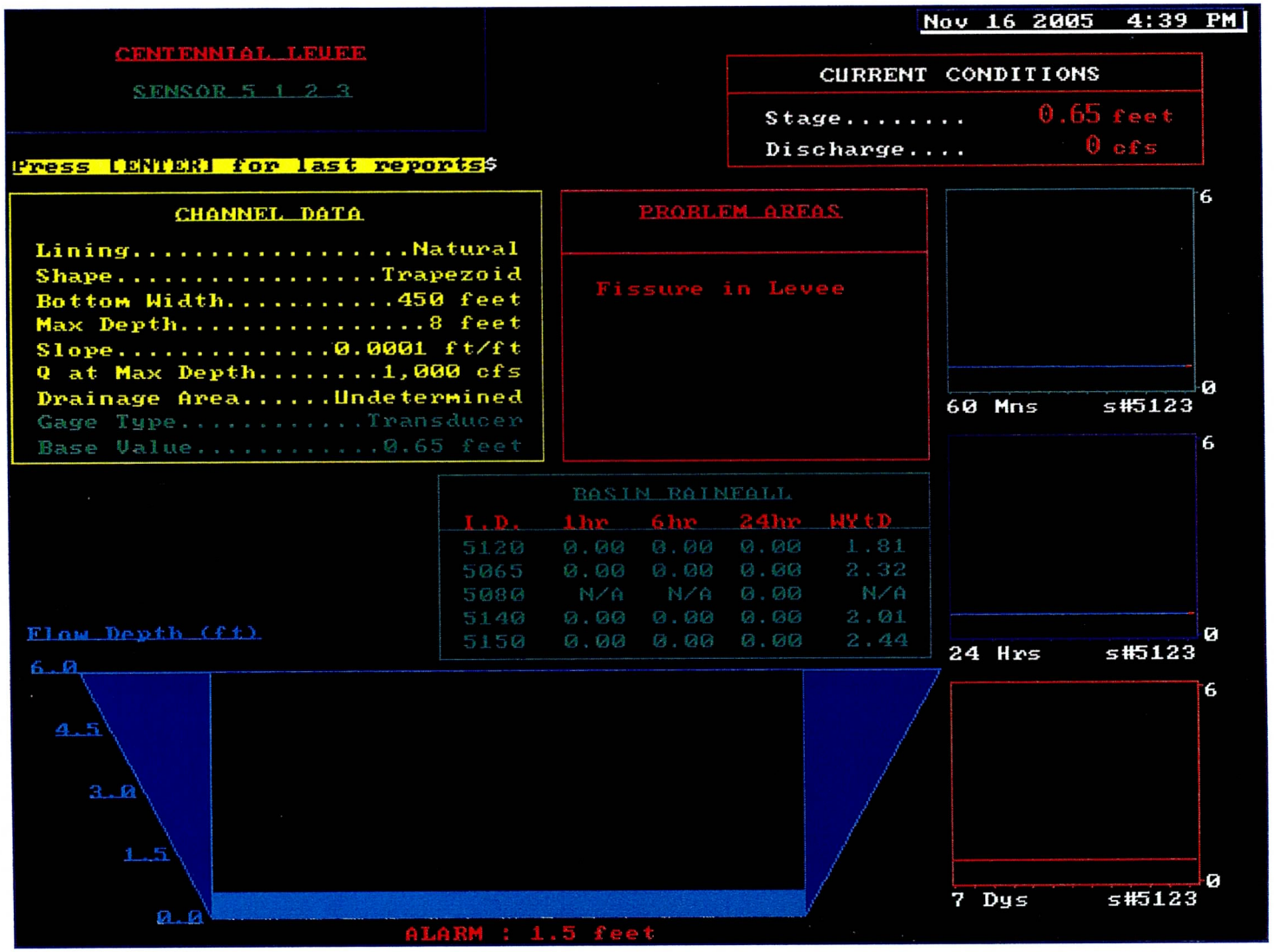

Figure 9. Centennial Levee Sensor Data Webpage.

GEOLOGICAL CONSULTANTS INC

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

Memorandum

September 30, 2005
Revised November 25, 2005

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

From: Ken Euge, R.G.
Principal Geologist

Subject: Geological Input to Structures Assessment Program, Phase I
Centennial Levee
Maricopa County, Arizona
FCDMC Contract No. FCD 2003C015, Work Assignment 4
Geological Consultants Project No. 2003-161-WA4

Geological Consultants Inc. is pleased to submit the geologic, seismic and ground subsidence information for the Centennial Levee structures assessment report. The Centennial Levee is one component of the flood control projects constructed within the Harquahala Valley Watershed. The original watershed work plan include two reaches for the levee: Reach 1 beginning about three miles west of the Buckeye-Salome Road and I-10 interchange and extended diagonally in a southwesterly direction to Centennial Wash (Figure 1); Reach 2 was proposed to extend southeasterly about five miles from the south end of Reach 1 and parallel the north side of Centennial Wash. Reach 2 was never constructed.

Centennial Levee Reach 1 is located immediately upstream of the West Side Canal within portions of Sections 5, 6, and 7 of Township 2 North, Range 9 West and Sections 11, 12, 14, and 15 of Township 2 North, Range 10 West. The total length of Reach 1 is about five and one-half miles. The purpose of Centennial Levee is to protect the West Side Canal and downstream agricultural lands.

1.0 Previous Reports

1.1 Soils Investigation as part of the Harquahala Valley Watershed design program for project PL-06; prepared by R. L. Graner (1966) for the Soil Conservation Service (now the National Resources Conservation Service (NRCS)). Report referenced in the 1974 Preliminary Geologic Investigation Report.

1.2 *Preliminary Geologic Investigation of Centennial Levee, Maricopa County, Arizona; 1974; prepared by the Flood Control Division, Arizona Water Commission:* This report documented the subsurface conditions along the proposed levee alignment and supplemented an earlier investigation (1966) conducted by the former Soil Conservation Service (now the National Resources Conservation Service (NRCS)). This report includes a summary of

the soil conditions, the presence of shallow caliche, soil erodibility. It also includes soil test data and exploration trench logs and photographs.

1.3 *Supplement No. 1, Harquahala Valley Watershed, Engineering Documentation, Volume II, Centennial Levee; 1976;* report ". . . describes the engineering design procedures, criteria, and reasoning used to develop the structural components and estimated construction cost of Centennial Levee (CENLEV)."

1.4 *Land Subsidence Potential Harquahala Valley Irrigation District; 1983;* prepared by Scott Geotechnical Services for Franzoy Corey & Associates Inc., Tempe, Arizona; report briefly describes the structural geology and groundwater conditions where irrigation ditches are to be constructed within the Harquahala Valley Irrigation District area as these conditions relate to land subsidence impacts on the proposed structures. The report documents three suspect earth fissures that according to the author ". . . are not earth fissures . . . " Additionally, survey level lines, reviewed by the author, ". . . established in 1980 showed no subsidence when rerun in May 1983." The author concluded that "No problems due to land subsidence or earth fissuring are anticipated . . . and therefore, does not have to be allowed for in design or maintenance planning."

2.0 Geologic Setting

The Centennial Levee 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 central 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. Centennial Levee lies in the valley between the Eagletail Mountains and the Big Horn Mountains.

The Eagletail Mountains predominately consist of pre-Tertiary granitoid rocks intruded by Tertiary mafic, intermediate, and felsic dikes, with some Jurassic to early Proterozoic metamorphics (Spencer, Gilbert, and Richard, 1992). This sequence is overlain by Miocene volcanics including basalt, tuff, and andesite. The mountains were affected by Tertiary normal faulting and tilting (Spencer, Richard, and Ort, 1997).

The Big Horn 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 presence of a gravity low in the central part of the valley suggests the possible presence of a large salt body (Figure 2). The Middle Alluvial Unit varies in thickness and it is may be a few hundred feet thick 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. According Oppenheimer (1980), the depth to bedrock in the central portion of the Valley beneath the Centennial Levee is about 7,000 feet (Figure 3).

1.1 Levee Centerline Geology

The description of the surface and subsurface conditions along the Centennial Levee are excerpted from a SCS memorandum dated December 1, 1983 from A.D. Elkin, Geologist to R. Arrington, State Conservation Engineer. Geologic profiles along the Levee centerline are presented in Figure 4.

The Reach 1 levee will cross a broad alluvial plain sloping towards the southeast. Flood flows across this plain are probably infrequent, and there are no well defined stream channels. Materials encountered in the test holes consisted of clay, silts, and sands of mostly alluvial origin except in the youngest surficial deposits where some reworking of the material by wind is evident. All of these deposits are of late Quaternary geologic age but can usually be divided into older or younger age by the presence of a distinct zone of material moderately cemented with calcium carbonate [caliche]. *Often the materials at the top of this zone have a light reddish brown color.*

<u>Younger Alluvial and Eolian Deposits</u>

The uppermost alluvial and eolian deposits in the area consist of cemented silty fine sands (SM) and silts with fine sand (ML) that occur in the surface throughout the length of Reach 1. they range in thickness from one foot at T.P. [Test Pit] *1017 to 9.5 feet at T.P. 1031 but average about 3 to 4 feet in thickness through most of the area.*

Only one density test was taken in the top three feet of these deposits (T.P. 1013) but the upper few feet generally appear to be soft and of relatively low density. Numerous animal burrows are present and a cutoff trench should be excavated at least through the upper three feet of these deposits.

In places, the uppermost alluvial and eolian deposits lie directly on the older alluvial deposits or on younger alluvial deposits (SC, SM, CL, ML) that are slightly cemented with calcium carbonate. But along most of Reach 1 they overlie younger alluvial sands that appear moderately permeable and consist generally of fine to medium sand with some coarse sand and some silt (SM, SP-SM, and SW-SM). The greatest thicknesses occur at T.P. 1014 (5.5 feet), T.P. 1018 (5.5 feet), T.P. 1025 (7 feet), T.P. 1029 (4.5 feet), T.P. 1030 (5.5 feet), and T.P. 1035 (8 feet). Design of both the canal and levee should take into account the possibility of water occasionally moving through these layers.

Density test taken in the more permeable sand layers of younger alluvium average about 101 pounds per cubic foot dry density. Density tests in the other younger alluvial and eolian deposits below a depth of three feet average about 96 pounds per cubic foot dry density but are quite variable, ranging from some low values below 90 pounds to some highs around 110 pounds [per cubic foot].

Older Alluvial Deposits

The older alluvial deposits are characterized by usually being moderately cemented with calcium carbonate. The cementation is enough to sometimes make excavation with a backhoe slow but not enough to stop excavation. Also, the cementation can usually be broken down by the application of water. In many places, the uppermost few feet of the older alluvium has a light reddish brown color. The materials in the older alluvium are mostly clays (CL) and silts (ML) with much fine sand, but there are also some clayey and silty fine sands (SM and SC).

Some of the older alluvial deposits are only slightly cemented with calcium carbonate and in some places layers of moderately permeable sands consisting of fine to coarse sand with some gravel and some silt are encountered. The average dry density of the slightly cemented material was about 98 pounds per cubic foot, and that of the moderately cemented materials was about 99 pounds [per cubic foot]. Again there was much variability with low ranges about 90 pounds and high ranges around 110 pounds [per cubic foot].

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. However, this evaluation did not include the Centennial Levee. A seismic evaluation conducted by Euge, Schell, & Lam (1992), the Centennial Levee lies within the Sonoran

Seismic Source Zone (Figure 5). 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 Centennial Levee, Sand Tank Fault, is about 60 miles southeast of the structure (Figure 6). 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 0.2 millimeters per year with the last occurrence less than 130,000 years ago (USGS, 2005). 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 Centennial Levee.

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 Centennial Levee 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 8000 deep in the center of the basin (Oppenheimer, 1980). 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 well (B-02-09 7ABB) to the Centennial Levee with a long-term hydrograph record is approximately one mile east. The hydrograph show a water level decline of about5 feet from 1999 through 2004. A hydrograph for well (B-02-10 14DCA) has a long term record through 1991 and show a continuing slight downward decline in the water level of about 5 feet. Two wells about 3 miles east and two well about 2.5 miles south of the Levee also show a decline in the water level. Hydrographs for wells within approximately three to four miles of the Centennial Levee were obtained from the Arizona Department of Water Resources (Appendix A) (Figure 7). Nineteen hydrographs were obtained, with the oldest dating back to 1952. Ten of these hydrographs show a net decline or static groundwater levels over their period of record. Three of well hydrographs show an increase in water levels. Six of the well hydrographs either have a short record period or no record. The wells in the vicinity of the Centennial Levee show a slow but continuous decline in groundwater levels suggesting the development of a pumping cone of depression is or has formed in the agricultural area southeast of the Levee.

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 Centennial Levee nor in the central portions of Harquahala Valley. Several survey monuments within the basin that are located along Interstate 10, along the CAP alignment, and scattered location throughout the basin; however, there is insufficient level data available to calculate the amount of subsidence that has historically occurred throughout the entire basin. The fact that earth fissures have opened up in the basin relatively recently (Roger's Fissure in 1997) indicates the basin sediment are being stressed, probably due to land subsidence in the basin..

There are six NGS monuments along Salome Road, south of I-10 identified in the NGS historic level line survey data (Figure 8). The six monuments are the closest ones to Centennial Levee that have been resurvey at least twice during the period of 1948 to 1982. A comparison of the level data over the period from 1967 through 1982 suggest land subsidence has taken place at a relatively slow rate ranging from 0.006 feet per year to 0.061 feet per year and it probably reflect the limited groundwater level decline in the basin. One anomalous data set (monument R476) documents a calculated subsidence of 0.27 feet per year for a two-year period of 1981 and 1982. A summary of the level line survey data is summarized in Table 1. Discounting the suspected anomalous reading and assuming the same rate of land subsidence continued to 2005, additional subsidence that may have occurred could range from 0.14 feet to 1.4 feet. If development in the basin occurs, either increased agricultural activity or urban growth, groundwater use could increase and cause groundwater levels to decline. Should this occur, land subsidence will be reactivated within the basin.

Table 1
NGS Monument Level Summary Summary
w/ Estimated Subsidence Rate

PID	Designation	Elevation						Subsidence Rate (ft/yr)	Interval (years)
		1948	1962	1967	1980	1981	1982		
DV1064	Z262	1213.553	1213.696	--	--	--	1213.517	0.009	20
DV1065	A263	1213.980	1214.145	1214.293	1214.190	1213.975	--	0.023	14
DV1066	B263	1203.507	--	1203.828	--	1203.520	--	0.022	14
DV1067	D13	1208.909	--	1209.233	--	--	--	--	--
DV1074	C263	1227.458	--	1227.774	1227.693	--	--	0.006	13
DV1291	R476	--	--	--	1234.385	1234.174	1233.845	0.270	2
DVO980	PLEASANTON RM1	--	1204.450	--	--	--	1203.237	0.061	20

PID	Designation	Elevation Change							
		1948-1962	1948-1967	1962-1967	1967-1980	1967-1981	1980-1981	1962-1982	1981-1982
DV1064	Z262	0.143	--	--	--	--	--	-0.179	--
DV1065	A263	0.166	0.313	0.147	-0.103	-0.317	-0.215	--	--
DV1066	B263	--	0.321	--	--	-0.309	--	--	--
DV1067	D13	--	0.324	--	--	--	--	--	--
DV1074	C263	--	0.315	--	-0.081	--	--	--	--
DV1291	R476	--	--	--	--	--	-0.211	--	-0.329
DVO980	PLEASANTON RM1	--	--	--	--	--	--	-1.214	--

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 9. 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 Centennial Levee vicinity (Oppenheimer, 1980). The Oppenheimer map estimated the depth to bedrock under the study area to be about 4,800 to 7,000 feet below ground surface, with the depth to bedrock depth increasing away from the mountain front (Figure 3). No unusual buried bedrock highs were interpreted within the project area from this data.

Figure 2 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 cover the northeast end of Centennial Levee, Geological Consultants, Inc. has extrapolated the contour lines into the project vicinity. As depicted in Figure 2, a relatively prominent gravity low exists centered along the axis of the basin. This gravity low could reflect the presence of a salt body similar to the Luke Salt Body known to be present in the West Salt River Valley, beneath Luke Air Force Base. Centennial Levee is located near the center of the basin. The Levee does not appear to be located over the bedrock pediment adjacent to the mountain front. In many of the basins of south-central Arizona, the position of the buried bedrock pediment acts as a focus for earth fissure formation. However, as with the Luke Salt Body, the perimeter of the suspected salt body beneath Centennial Levee could represent a "relatively ridged" geologic structure boundary between the salt body and the basis fill over which the draped basin fill sediments might be stressed enough to induce earth fissure formation.

3.3.1 Known Earth Fissures in the Project Vicinity

There are three earth fissures reported in the Harquahala Valley (Figure 10). The closest fissure to the Centennial Levee is the Rogers fissure that opened on September 25, 1997 in Sections 20 and 21, Township 2 North, Range 10 West, approximately 1.6 miles west of the dam. The fissure made its first 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; October 16, 2001) (Corkhill et al, 1998). Development of the surface expression of the Rogers fissure was unusual in that there were no reported noticeable 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.

Another earth is approximately 1.9 miles east of the northeast end of Centennial Levee in Section 9, Township 2 North, Range 9 West (Figure 11). 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). In 1983, Scott Geotechnical Services trenched this feature and did not confirm the presence of any subsurface earth fissure trace in the area, but this finding has not been confirmed by any other consultant or agency.

The third earth fissure was documented in 1961 in a farm field about 6.8 miles southeast of Centennial Levee 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.

REGISTERED GEOLOGIST
CERTIFICATE No.
10338
KENNETH M.
EUGE
Date Signed 11/25/05
ARIZONA U.S.A.

Photo 1: View of Rogers earth fissure with gulley headcutting upslope along the fissure alignment. Photograph taken October 16, 2001,

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. Photograph taken October 16, 2001.

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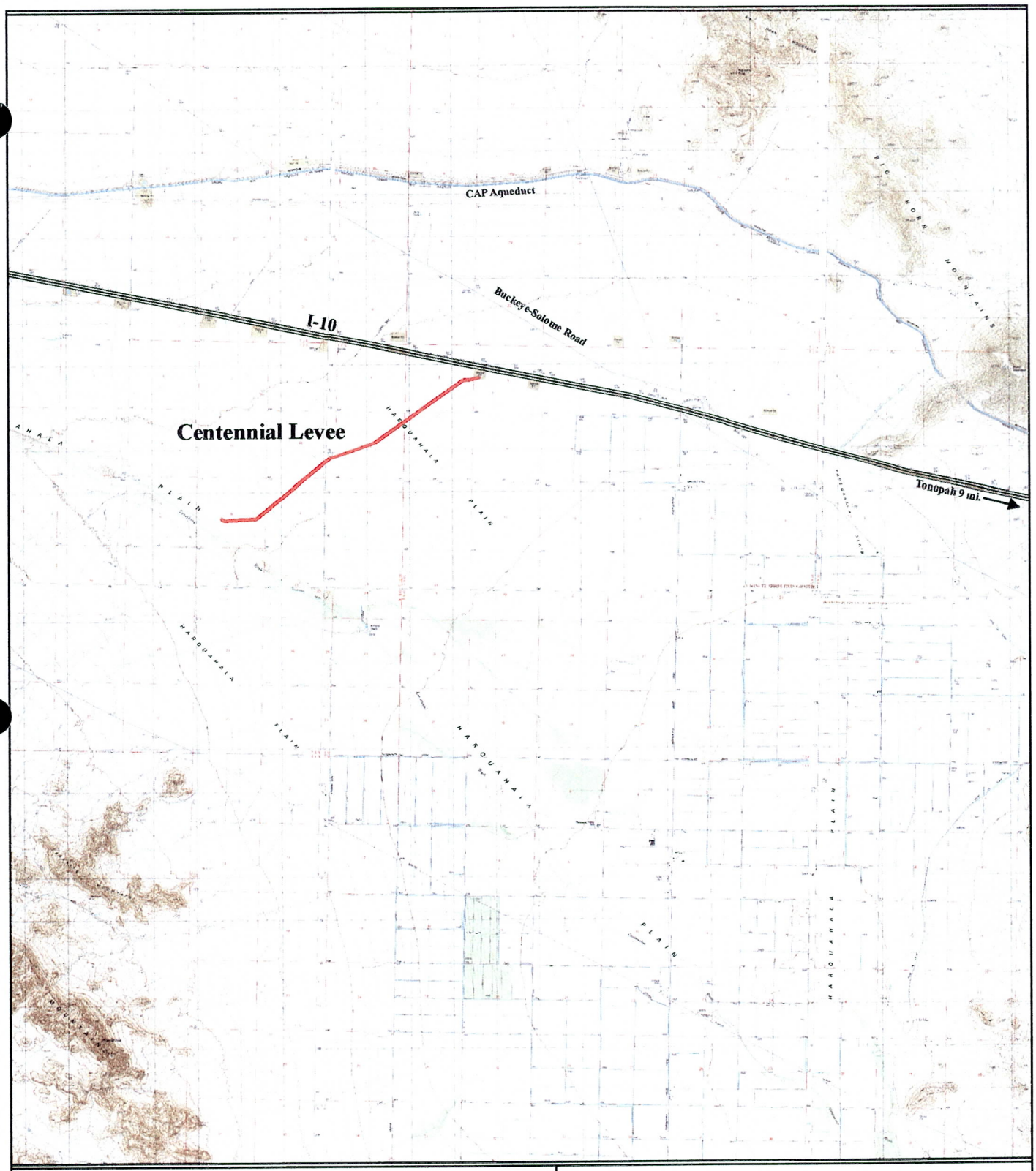
Figures



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Centennial Levee
Location Map
Figure 1

GEOLOGICAL CONSULTANTS INC.
2333 West Northern Ave, Suite 1A
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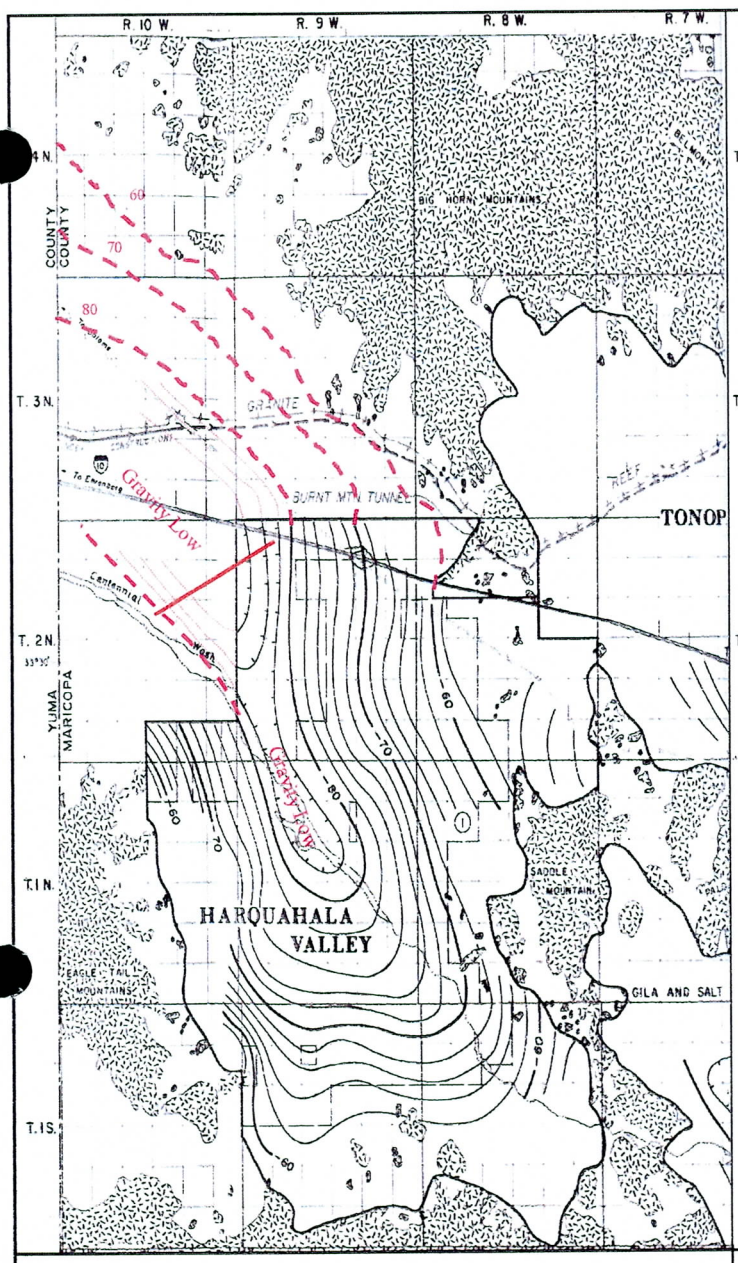
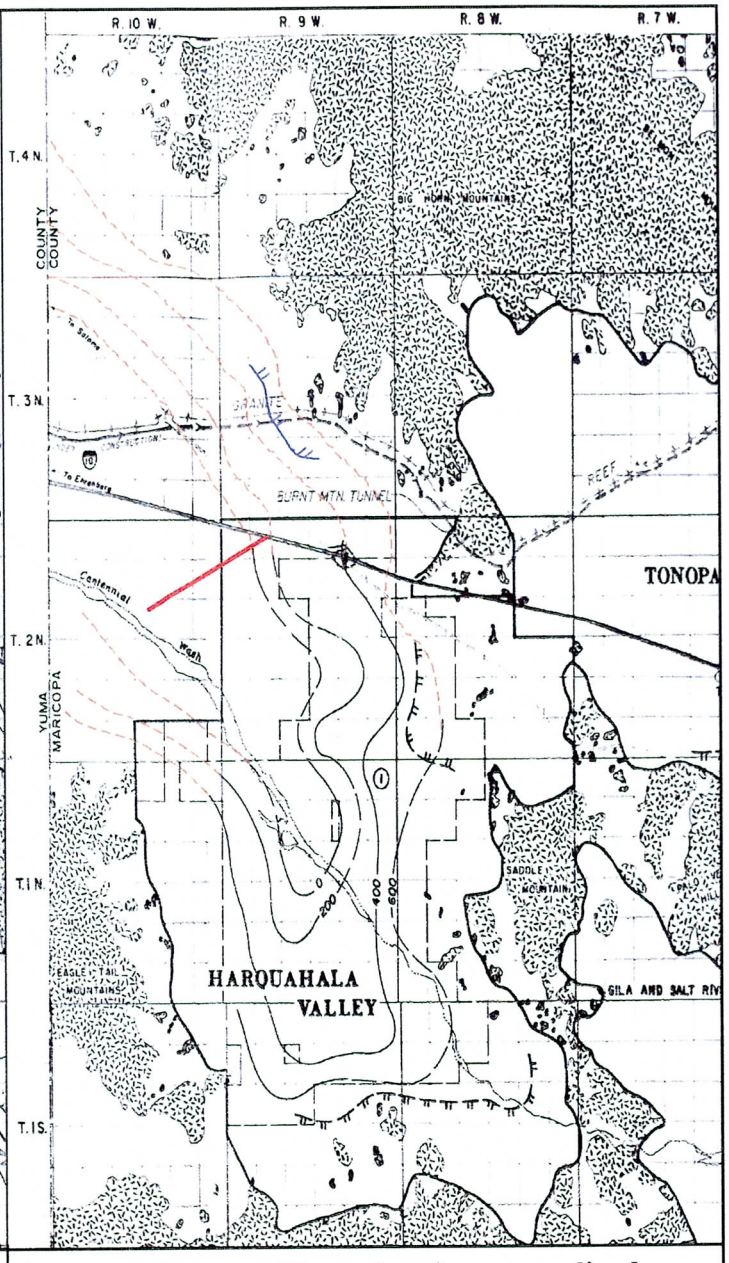


Bouguer Anomaly Map* showing gravity contours (dashed where inferred), contour interval 2 milligals.

- - - - 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)

*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.

Centennial Levee
Bouguer Anomaly and Structure Contours
Figure 2

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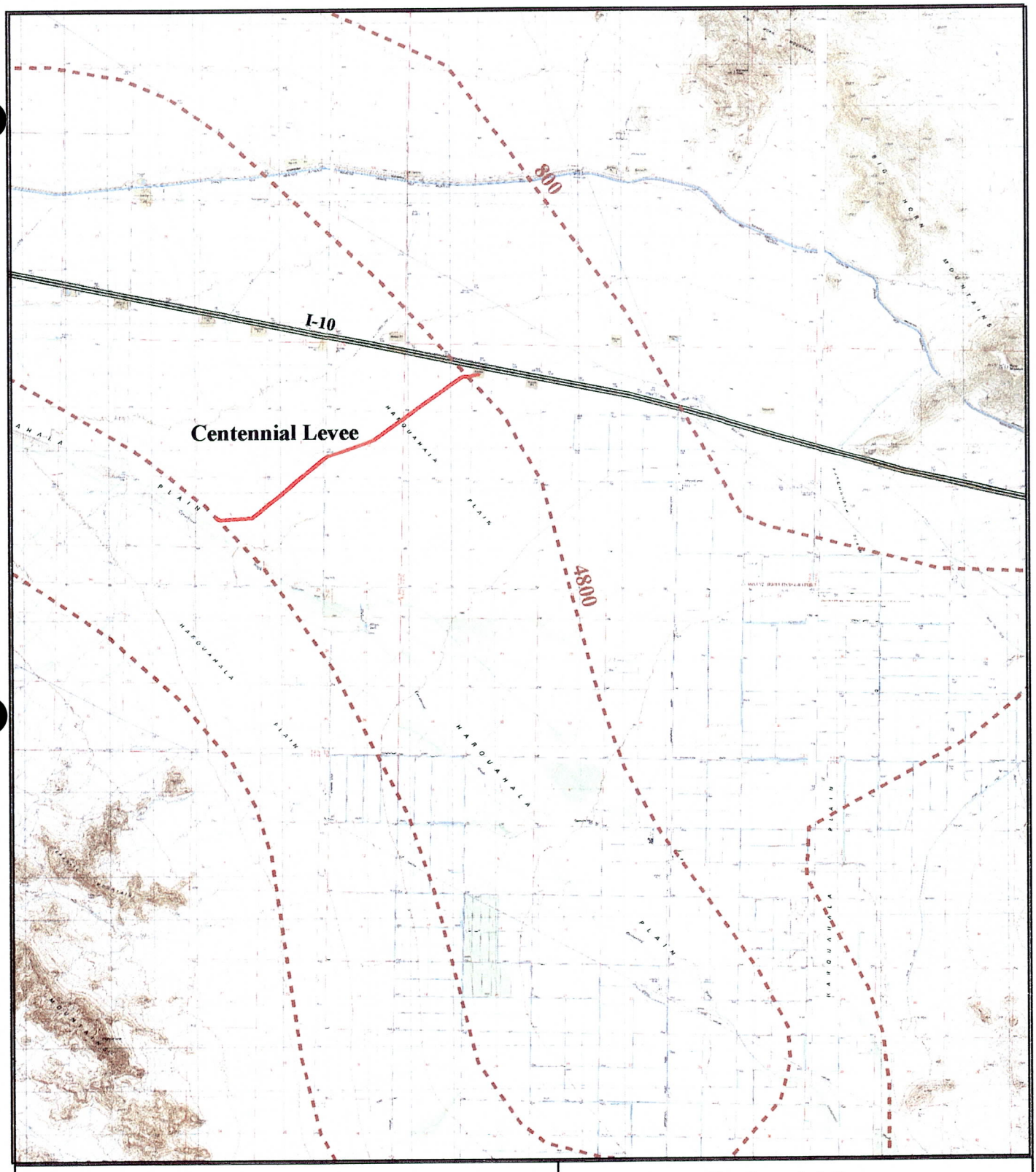

(modified from Oppenheimer, 1980)

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Centennial Levee
Depth to Bedrock Map
Figure 3

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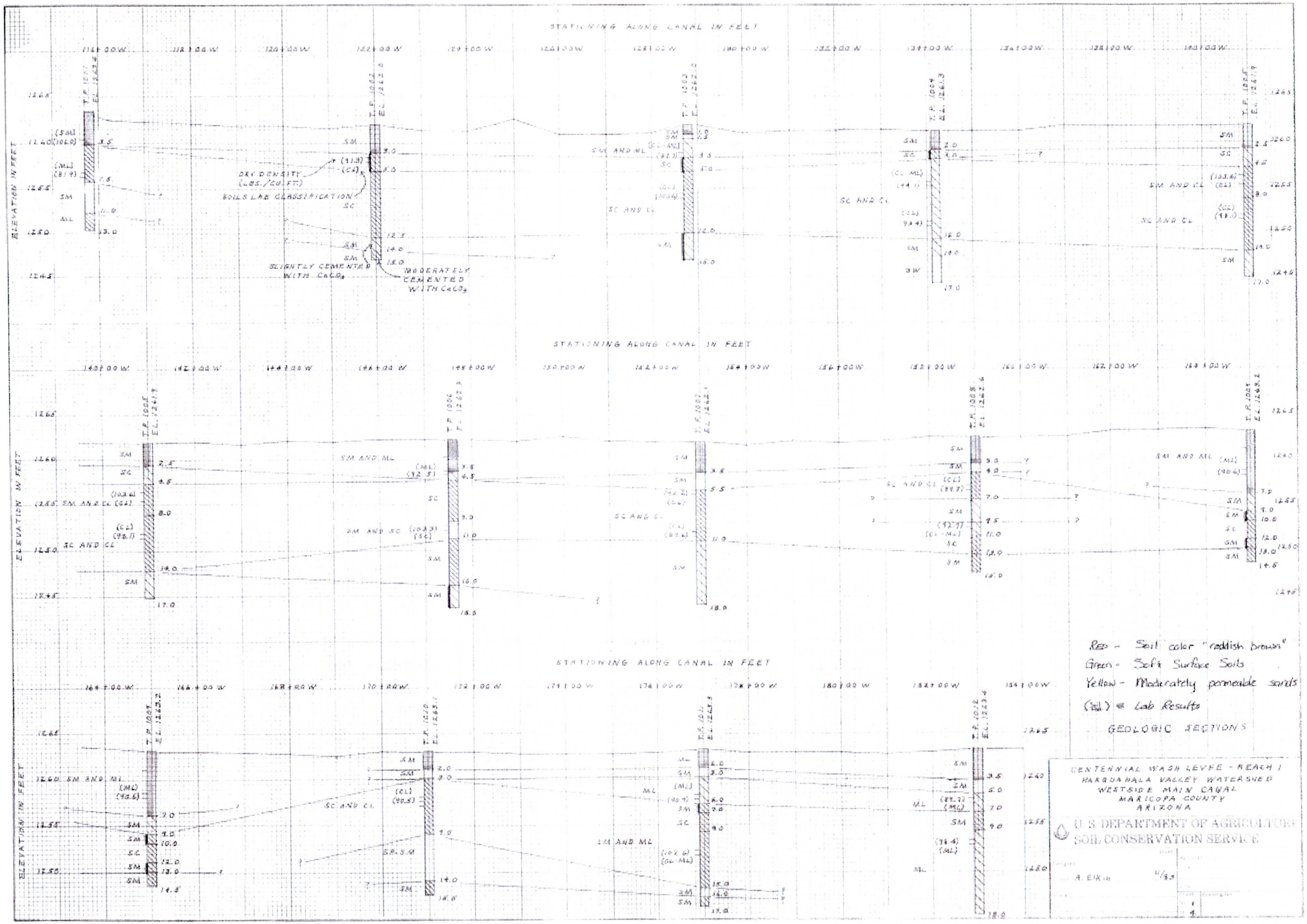

Not to Scale

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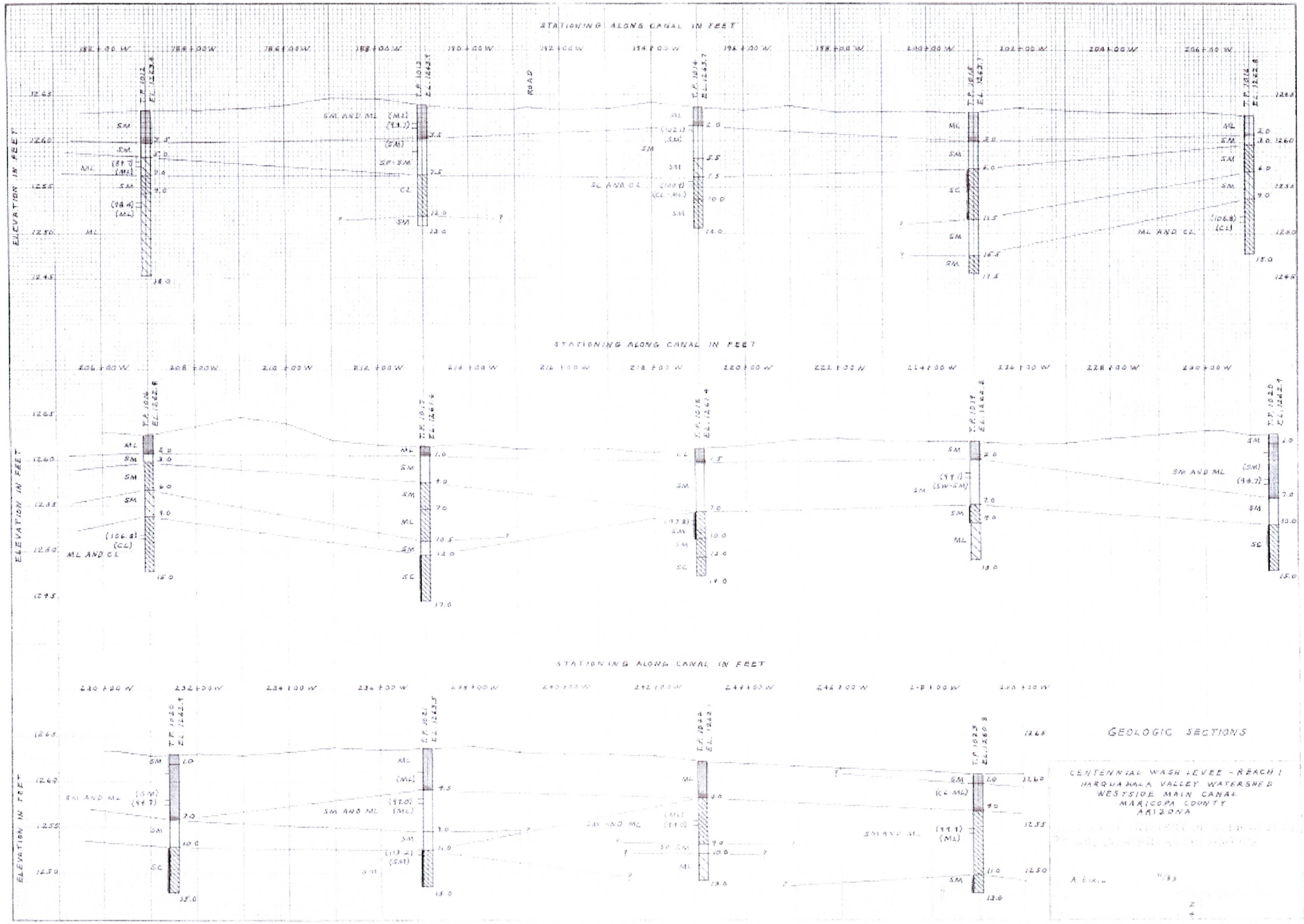
Centennial Levee
Figure 4a
Geologic Cross Section Sheet 1 of 4



Not to Scale

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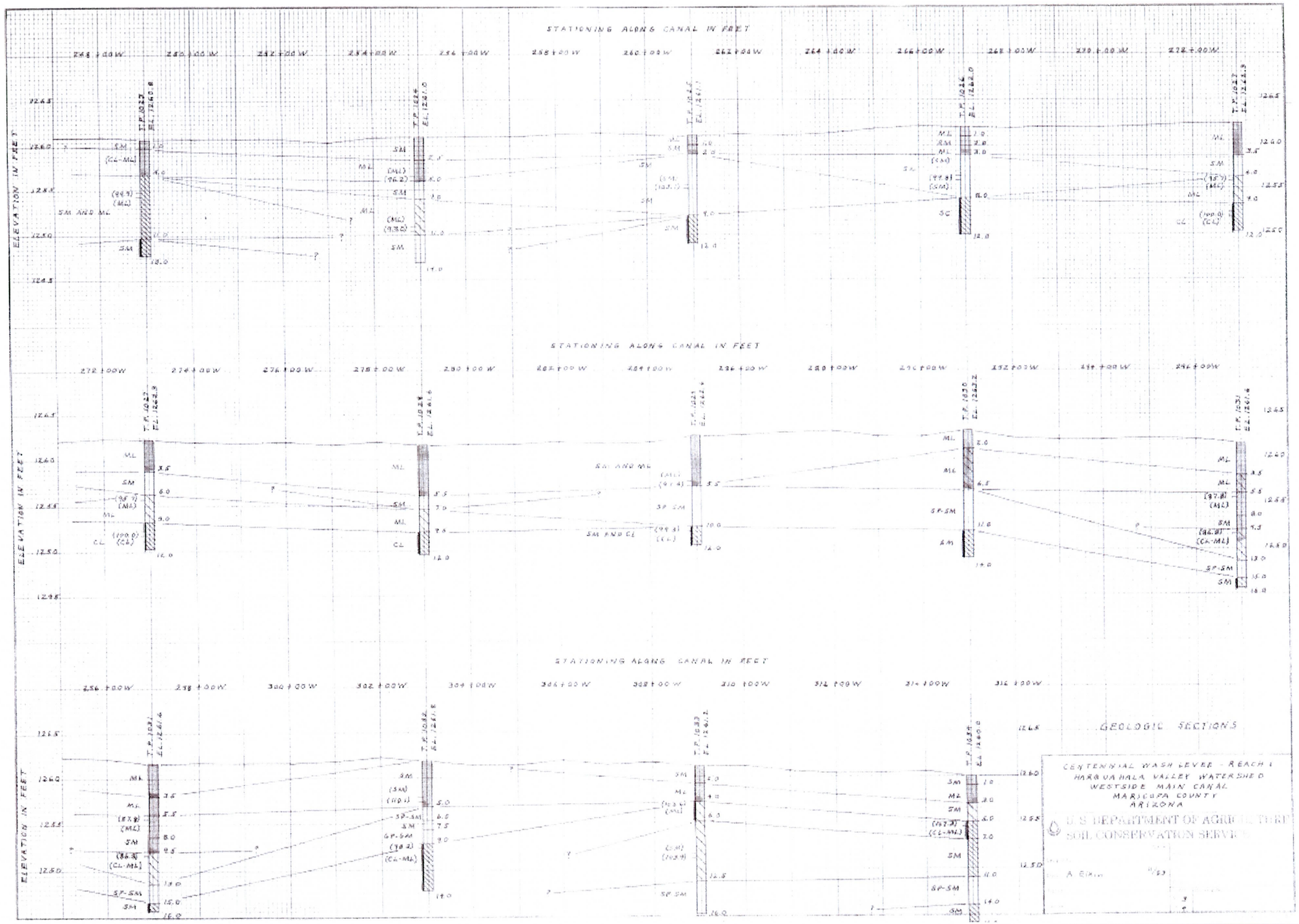
Centennial Levee
Figure 4b
Geologic Cross Section Sheet 2 of 4



Not to Scale

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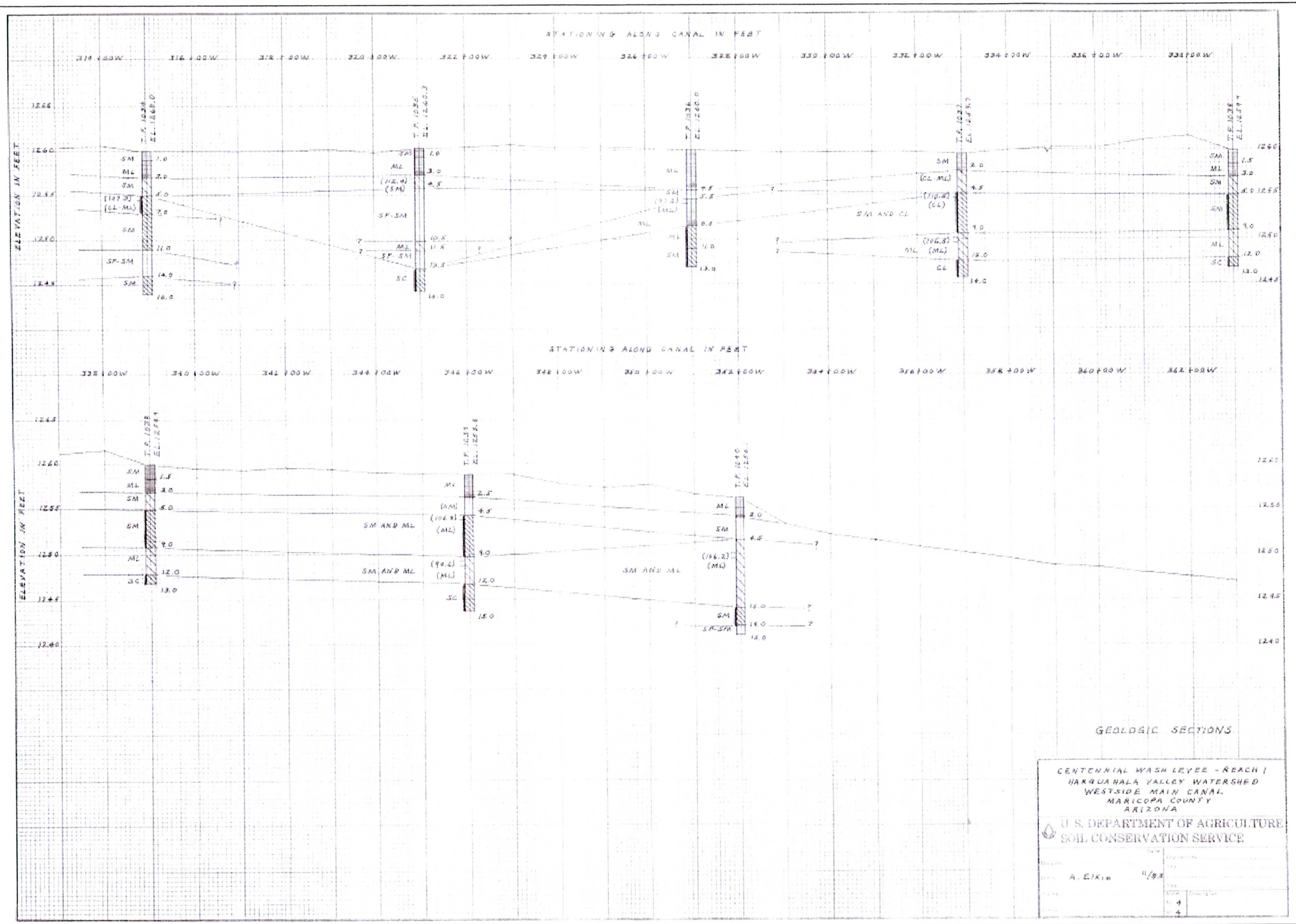
Centennial Levee
Figure 4c
Geologic Cross Section Sheet 3 of 4



Not to Scale

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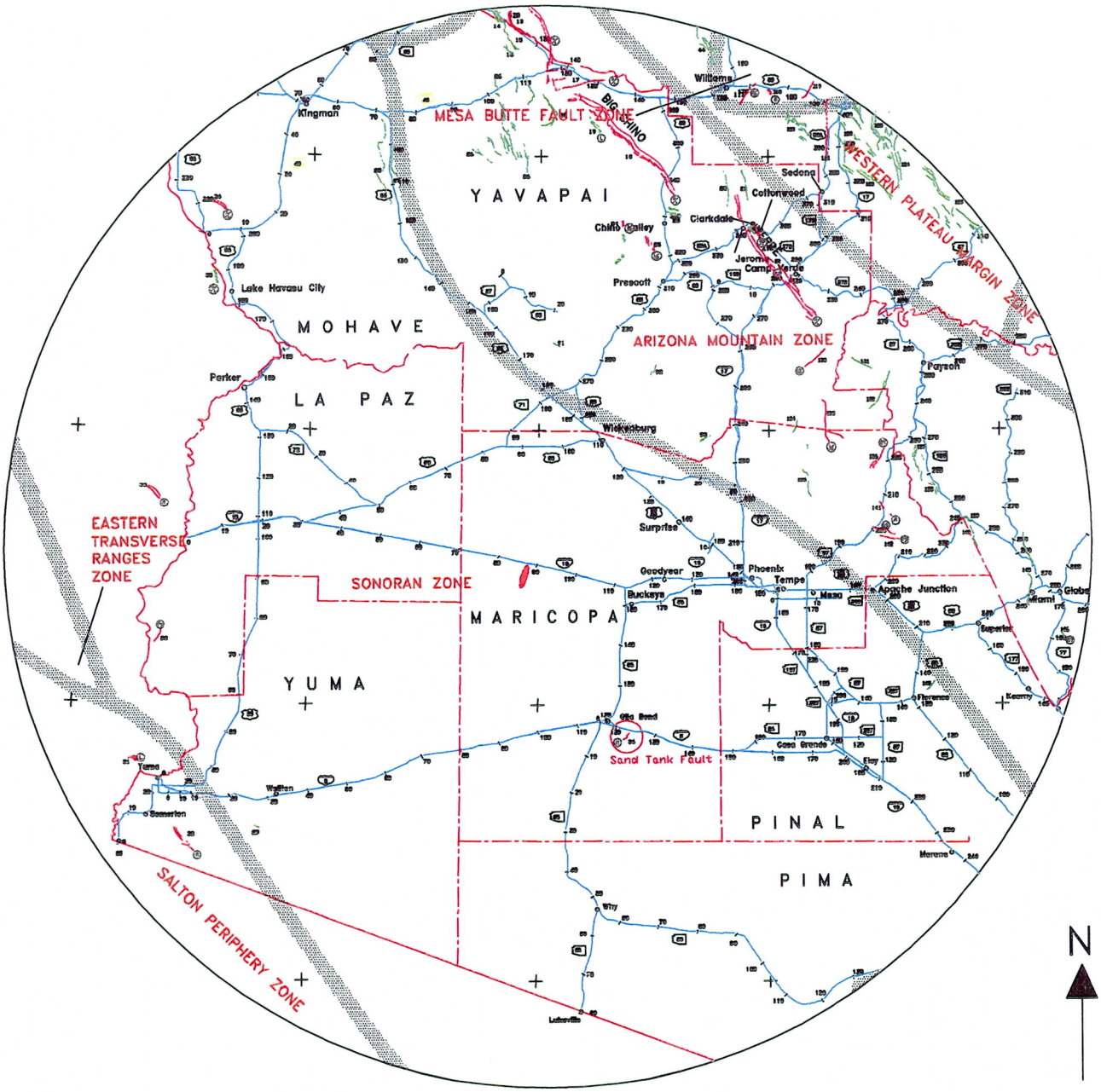
Centennial Levee
Figure 4d
Geologic Cross Section Sheet 4 of 4



EXPLANATION

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

			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

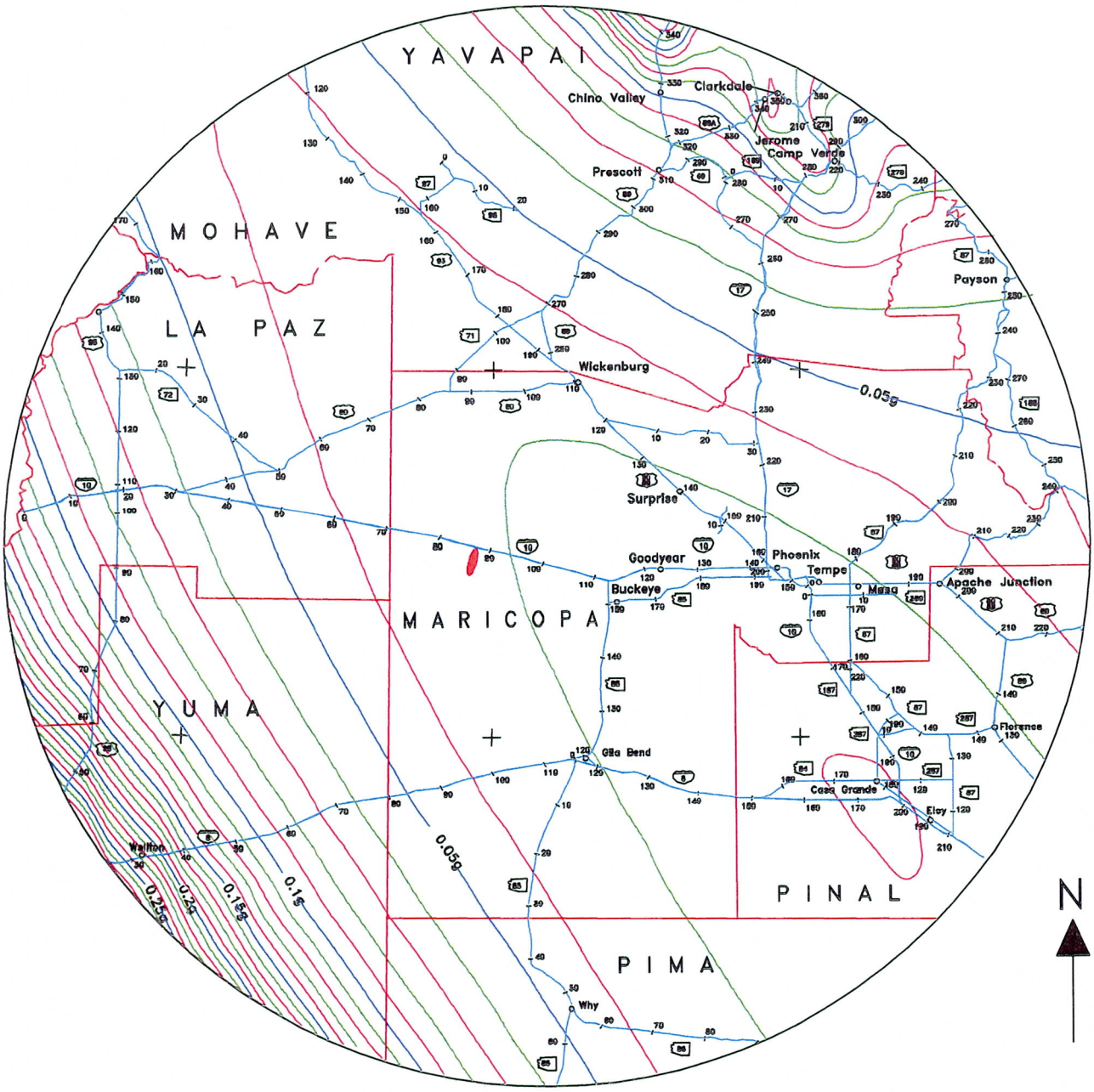
Seismic source zone boundary

Approximate Vicinity of Centennial Levee

(modified from: Fault Map of Arizona Area by Bruce A. Schell, Kenneth M. Sugg, and Ignatius Po Lam, 1992)

Centennial Levee Fault Map Figure 5

2333 West Northern Ave, Ste 1A
Phoenix, Arizona 85021
Phone 602-864-1888
Fax 602-864-1899



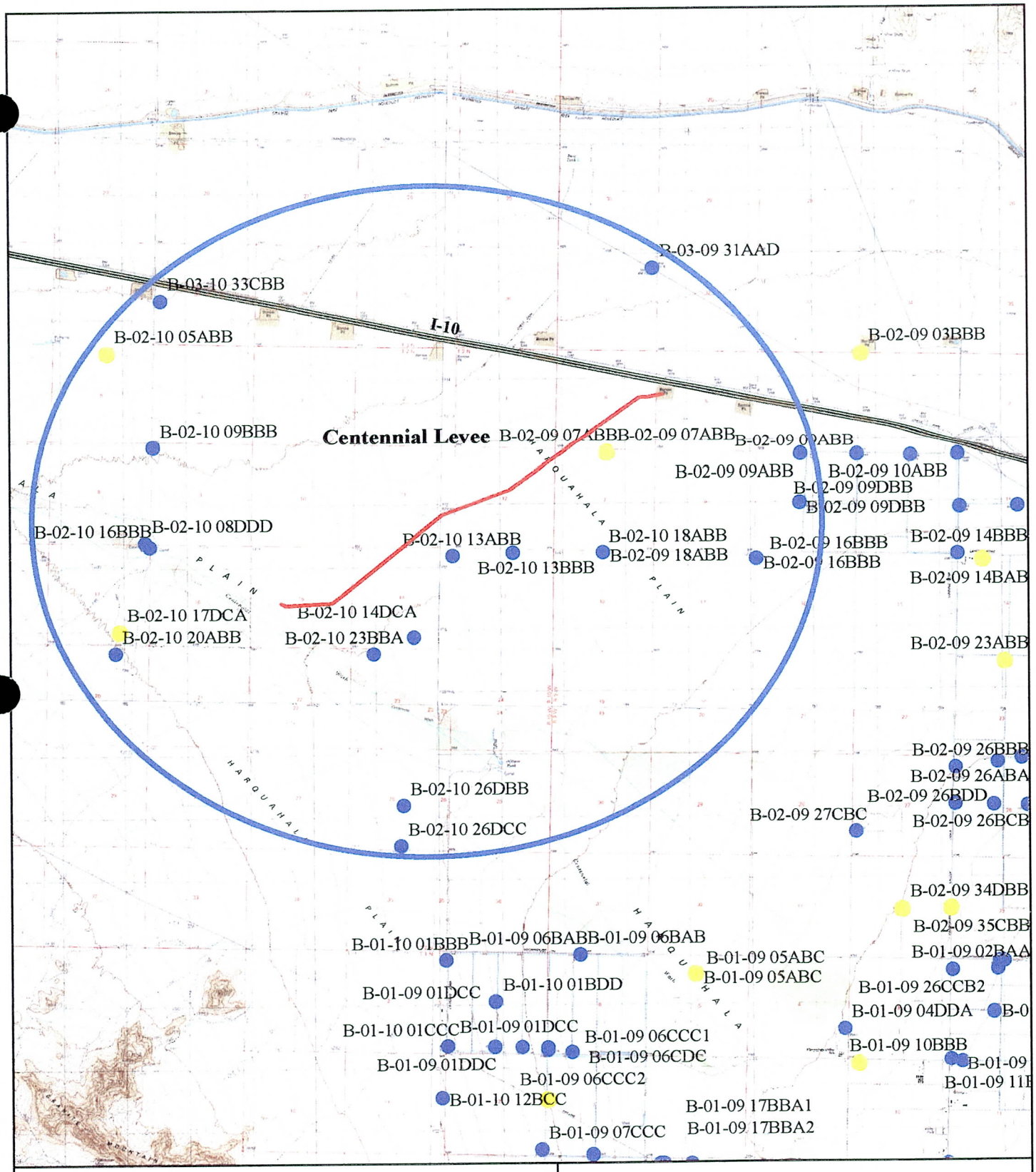
Legend

Approximate location of Centennial Levee

(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)

Centennial Levee Horizontal Acceleration Map Figure 6

2333 West Northern Ave, Ste 1A
Phoenix, Arizona 85021
Phone 602-864-1888
Fax 602-864-1899



GWSI Well Points:

- Index Wells (See Appendix A)
- Other Wells (See Appendix A)
- Area of well included in Appendix A

N

1:80,741

Centennial Levee Well Location Map Figure 7

GEOLOGICAL CONSULTANTS INC.
2333 West Northern Ave. Suite 1A
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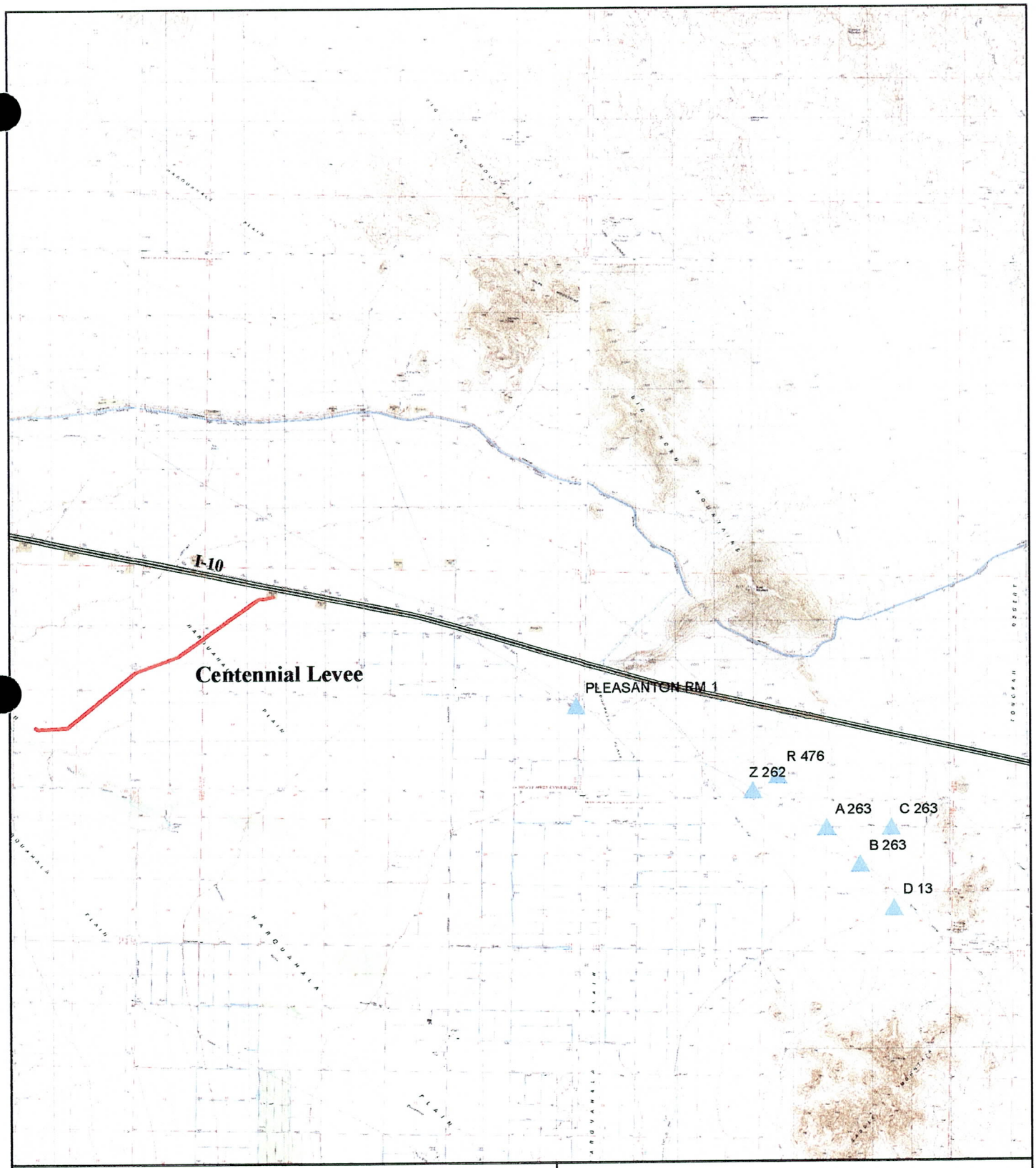
phone 602-864-1888
fax 602-864-1899



Legend

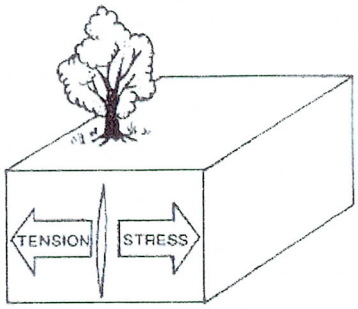
NGS Survey Monuments

N

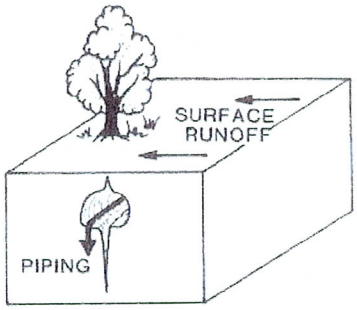
1:130,150

Centennial Levee
NGS Monument Location Map
Figure 8

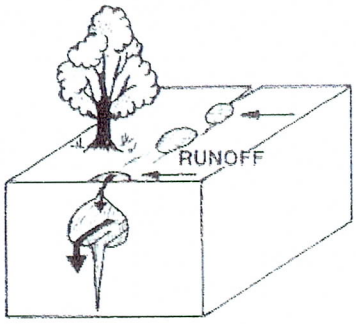
GEOLOGICAL CONSULTANTS INC.
2333 West Northern Ave, Suite 1A
Phoenix, AZ 85021
phone 602-864-1888
fax 602-864-1899



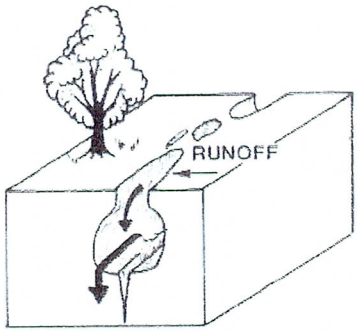
1. Lateral stresses induce tension cracking



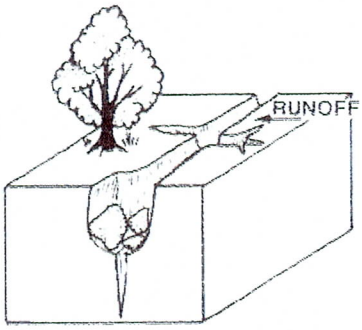
2. Surface runoff and infiltration enlarge crack through subsurface piping



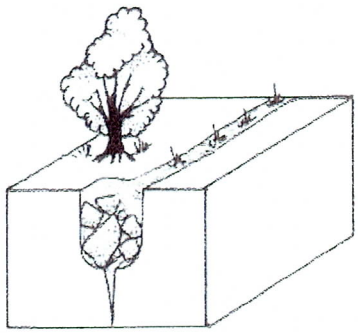
3. As piping continues, fissure begins to appear at surface as series of potholes and small cracks



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

Centennial Levee
Generalized States of Earth Fissure Development
Figure 9

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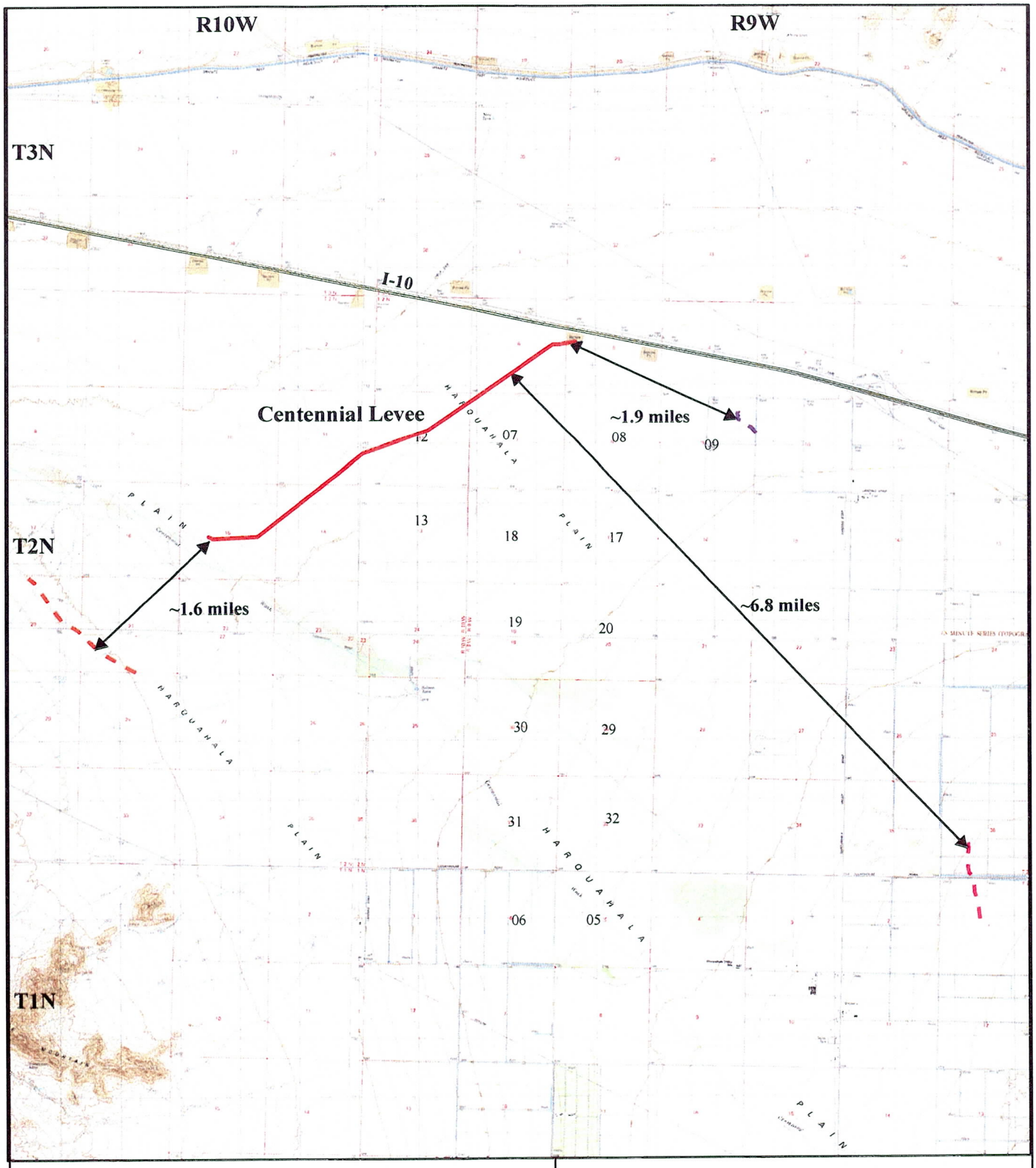
phone 602-864-1888
fax 602-864-1899



Legend

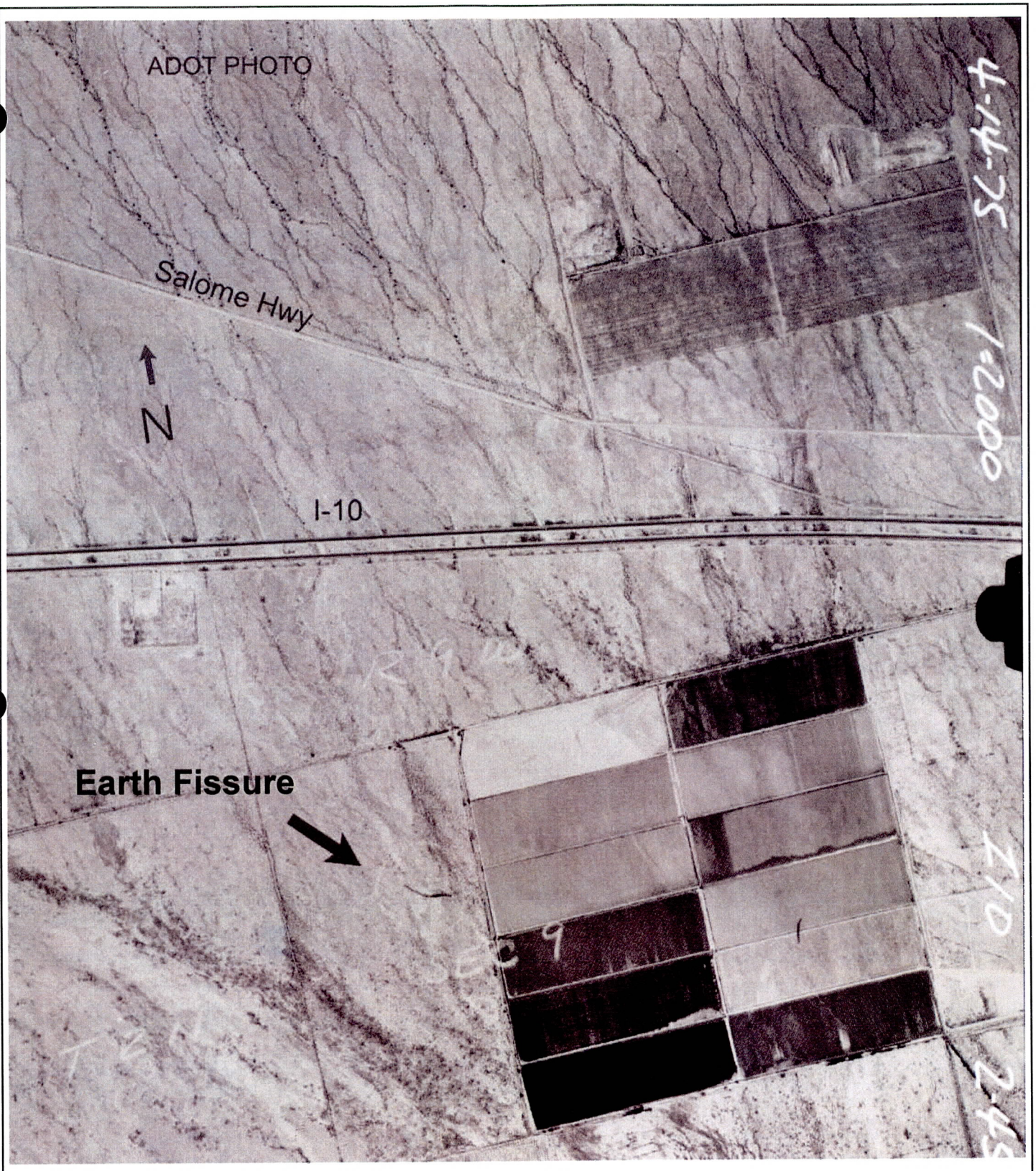
- "Rogers" Earth Fissure (Corkhill, 1998)
- Earth Fissure (Harris, 2001)
- Fissure Sec. 9, T2N,R9W (Graf, 1980) See Figure 11.

N

1:90,000

Centennial Levee
Earth Fissure Location Map
Figure 10

GEOLOGICAL CONSULTANTS INC.
2333 West Northern Ave, Suite 1A
Phoenix, AZ 85021
phone 602-864-1888
fax 602-864-1899



Refer to Figure 10 for earth fissure location relative to Centennial Levee.

Centennial Levee
Earth Fissure Section 9, T2N, R9W
Figure 11

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Phoenix, Arizona 85021
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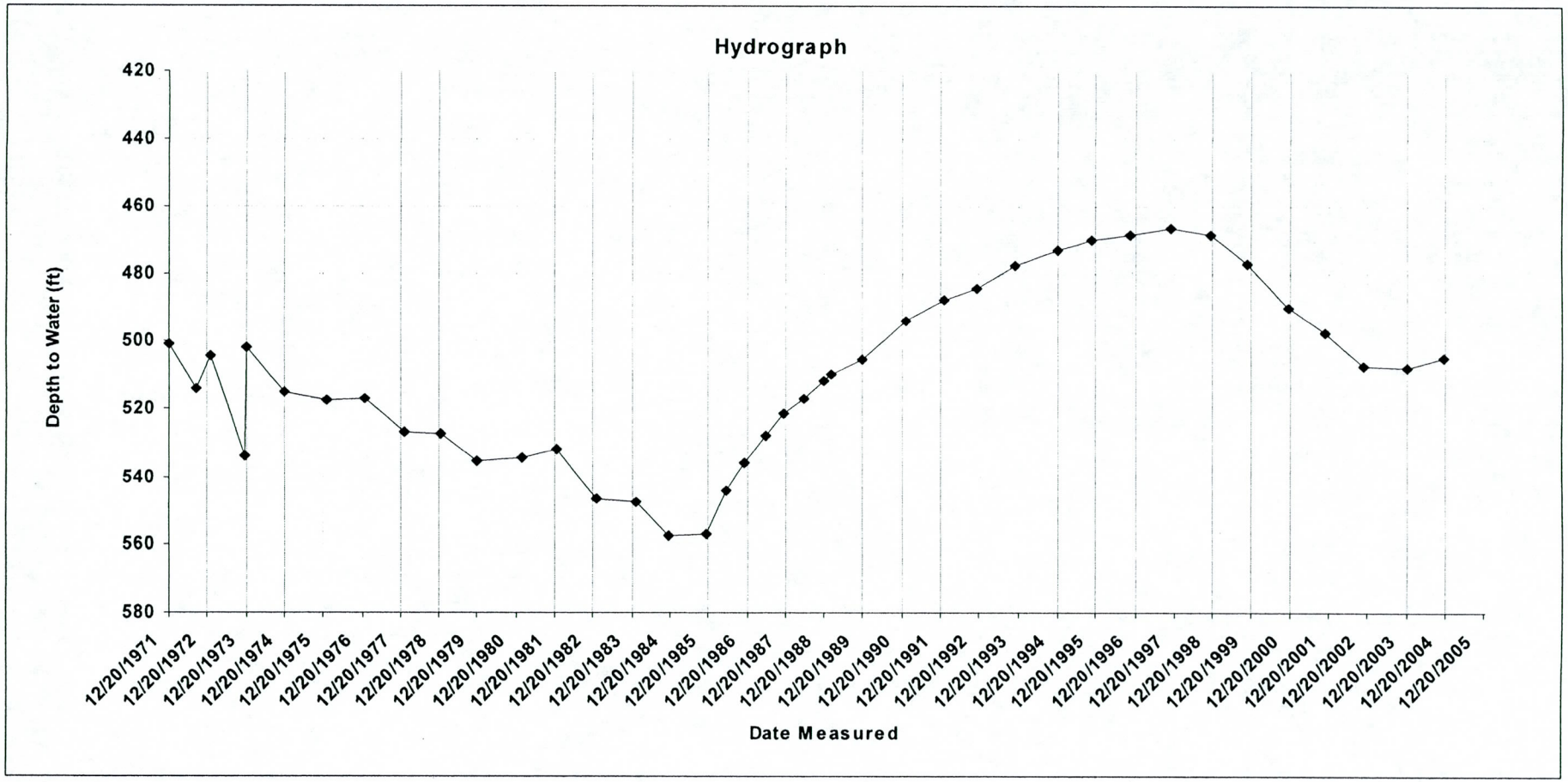
Fax 602-864-1899

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	11/30/2004	505.10	754.9	40

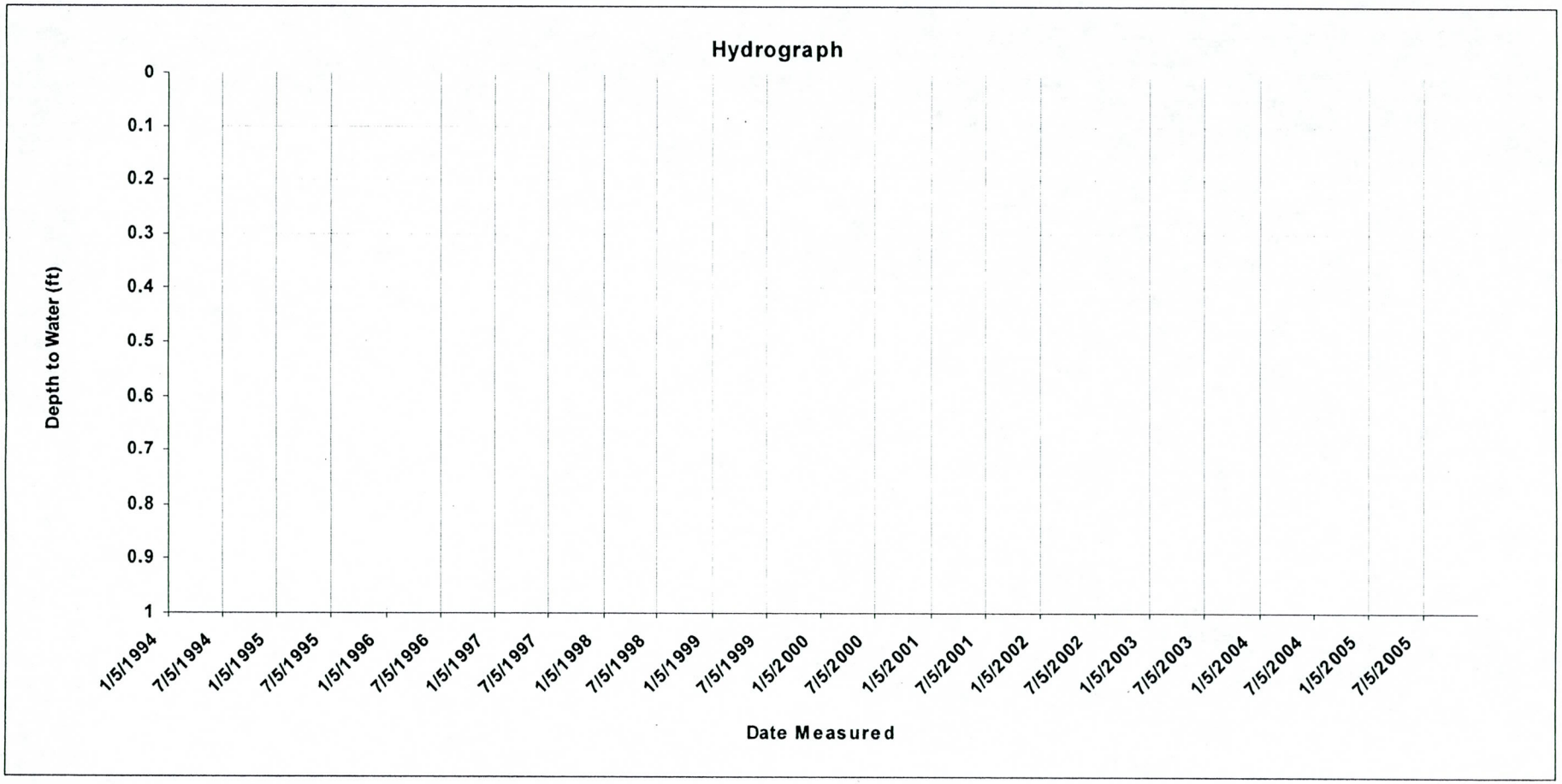


GWSI is ADWR's technical database of well locations, construction data, and water levels.

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-02-10 09BBB	333211113175801		33° 32′ 11″	113° 17′ 58″	UNUSED	56		16	1/5/1994			1



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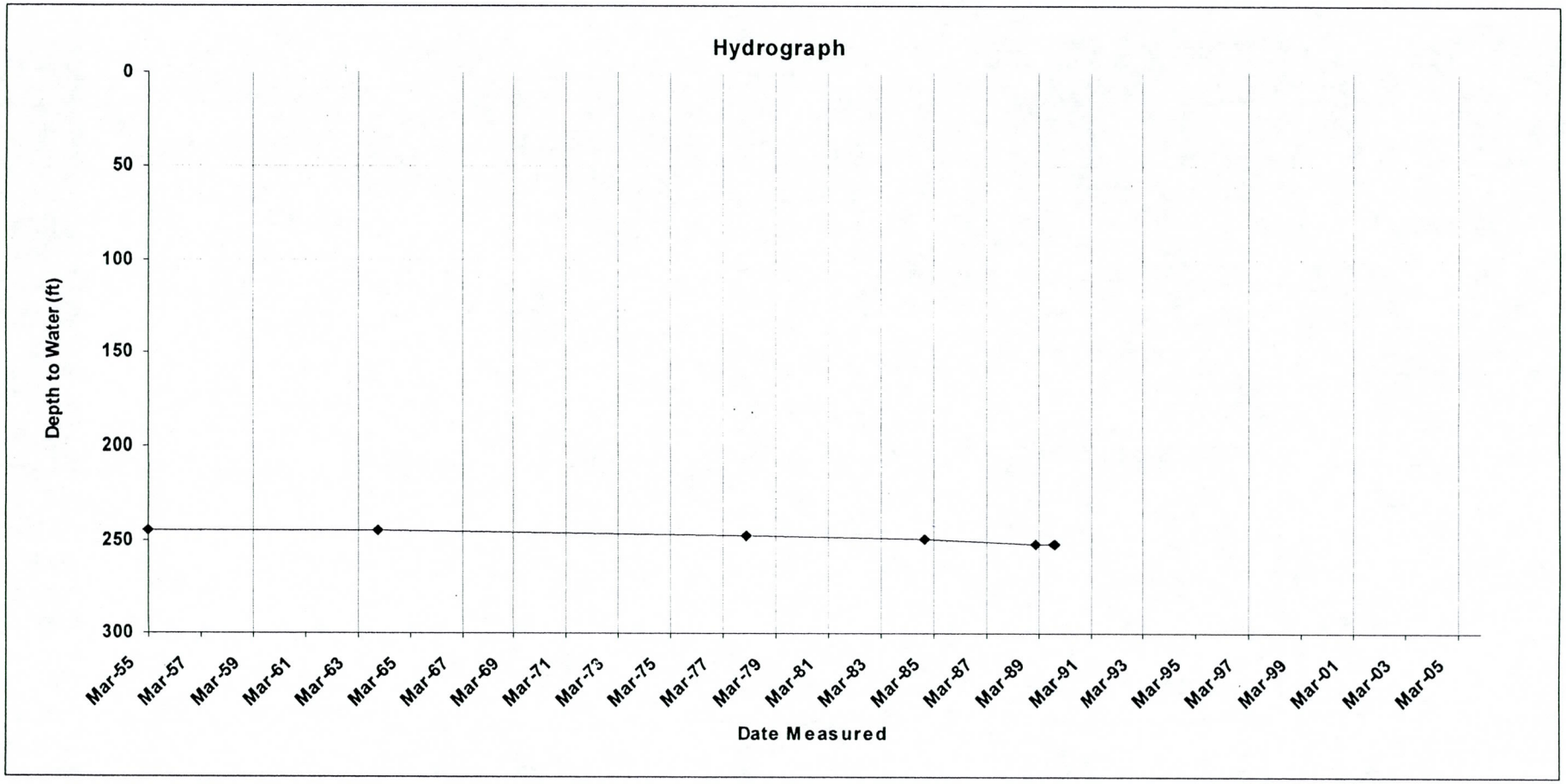
Sunday, October 02, 2005

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-02-10 14DCA	333035113151501		33° 30′ 37.2″	113° 15′ 13.9″	UNUSED	253.8			12/20/2004			8



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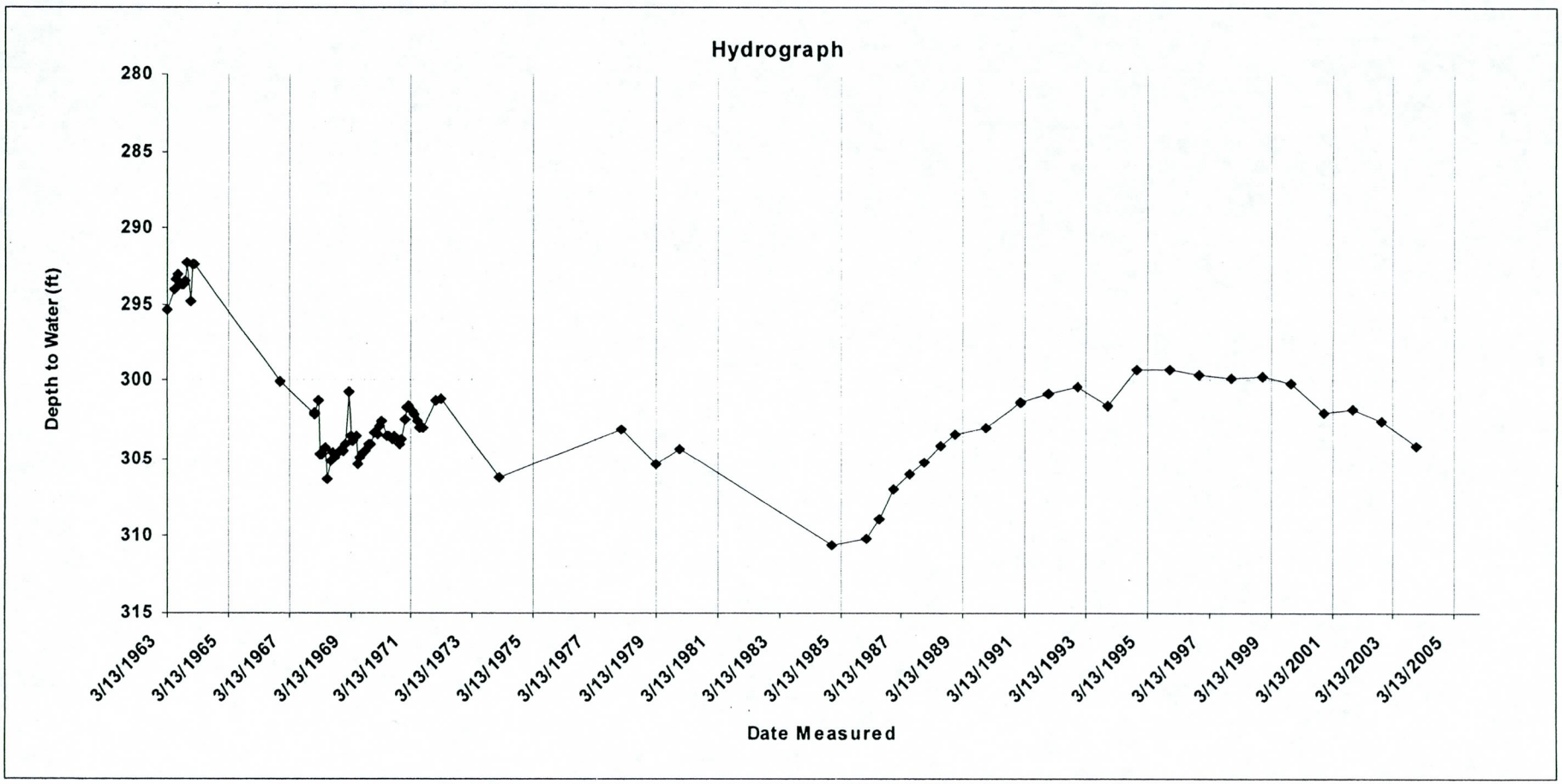
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-09 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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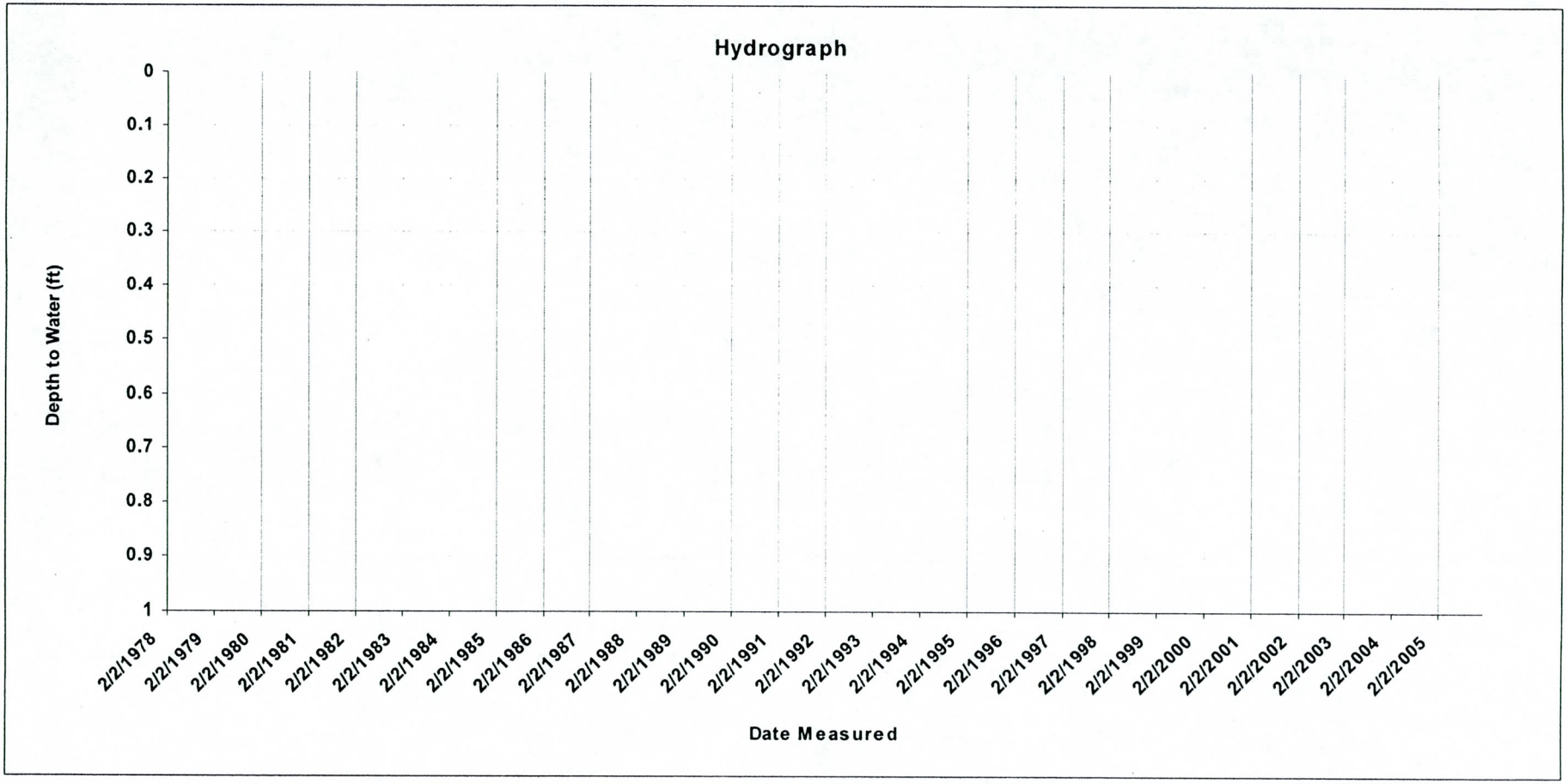
AZ Dept of Water Resources

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-02-10 13ABB	333120113141501		33° 31′ 20″	113° 14′ 15″	UNUSED	6		16.5	2/2/1978			1



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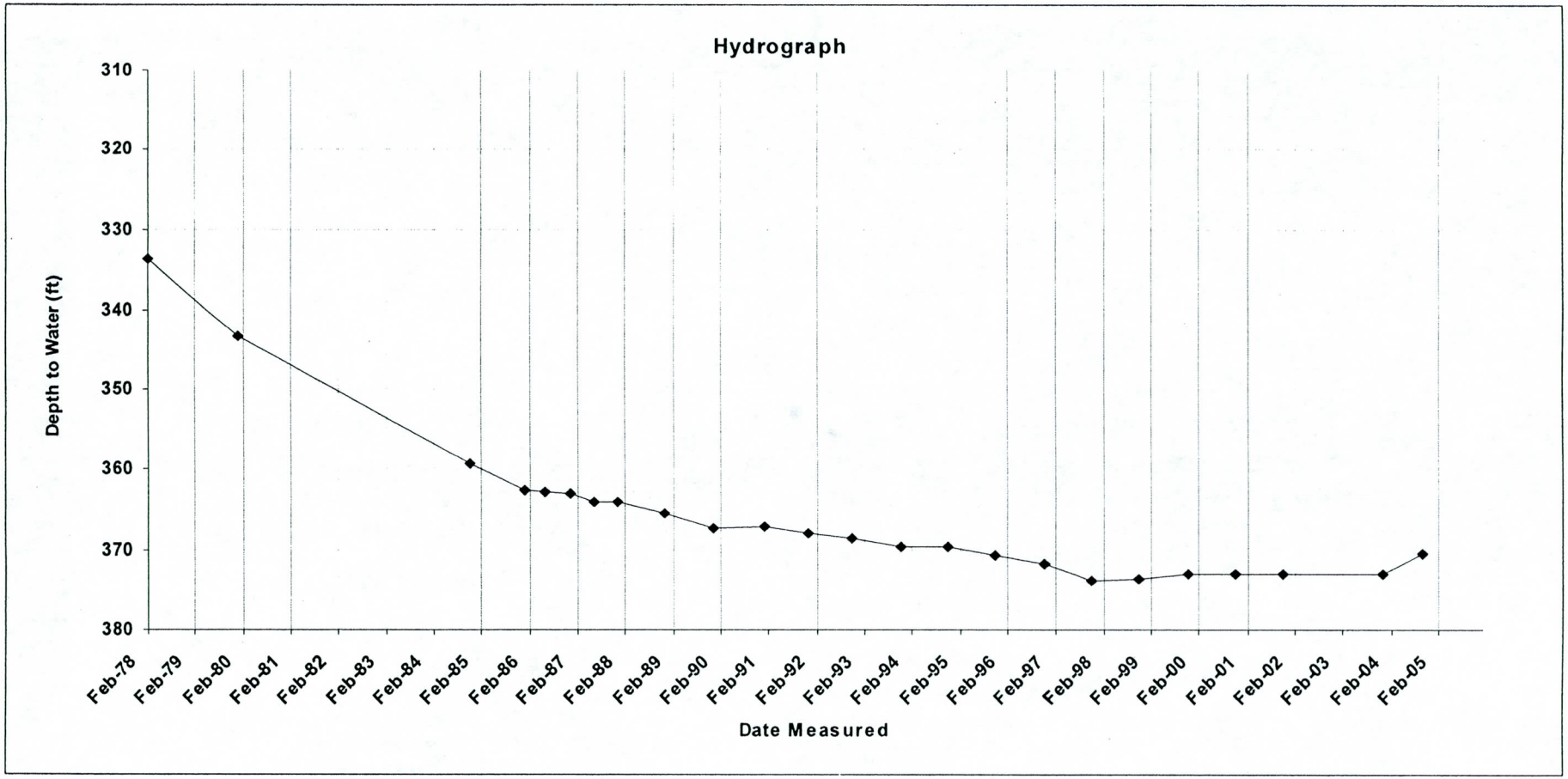
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-10 17DCA	333035113181701	621523	33° 30′ 35″	113° 18′ 17″	DOMESTIC	400			10/27/2004	370.40	925.6	24



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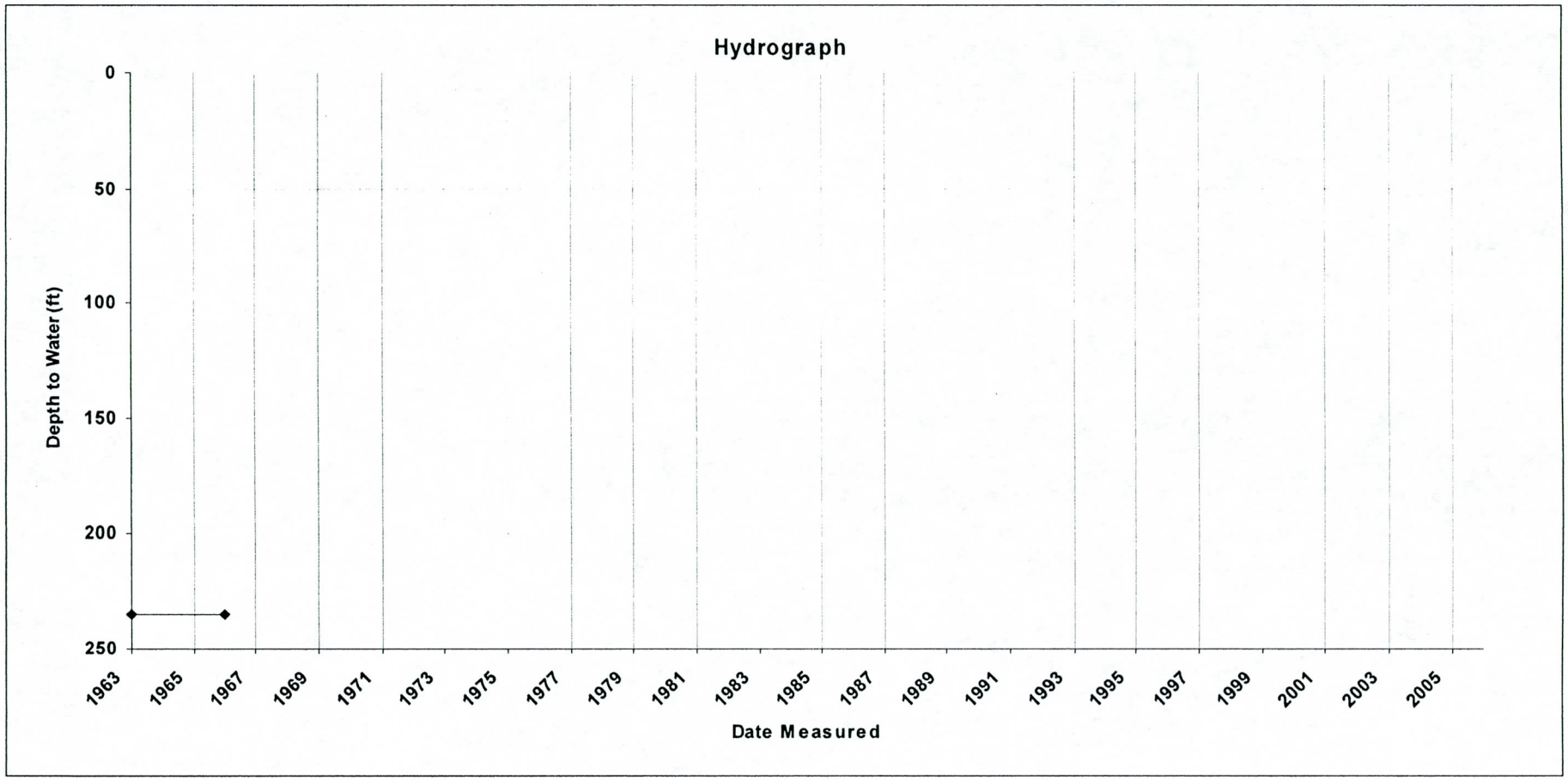
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-09 18ABB	333121113131901		33° 31′ 21″	113° 13′ 19″	UNUSED				12/2/1993			3



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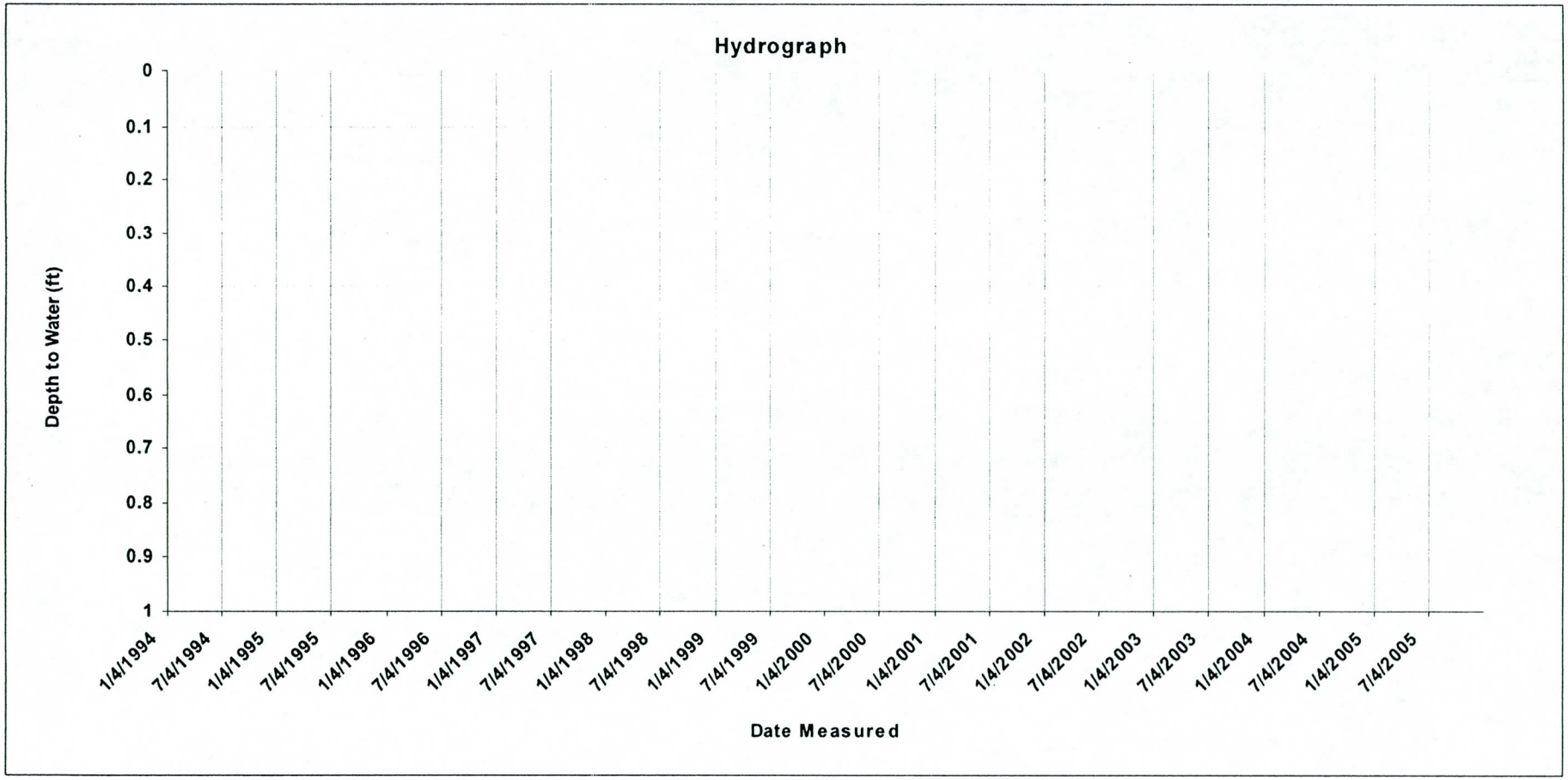
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above' Mean Sea Level	Times Meas.
B-02-10 13BBB	333118113145201		33° 31′ 18″	113° 14′ 52″	UNUSED	65		16	1/4/1994			1



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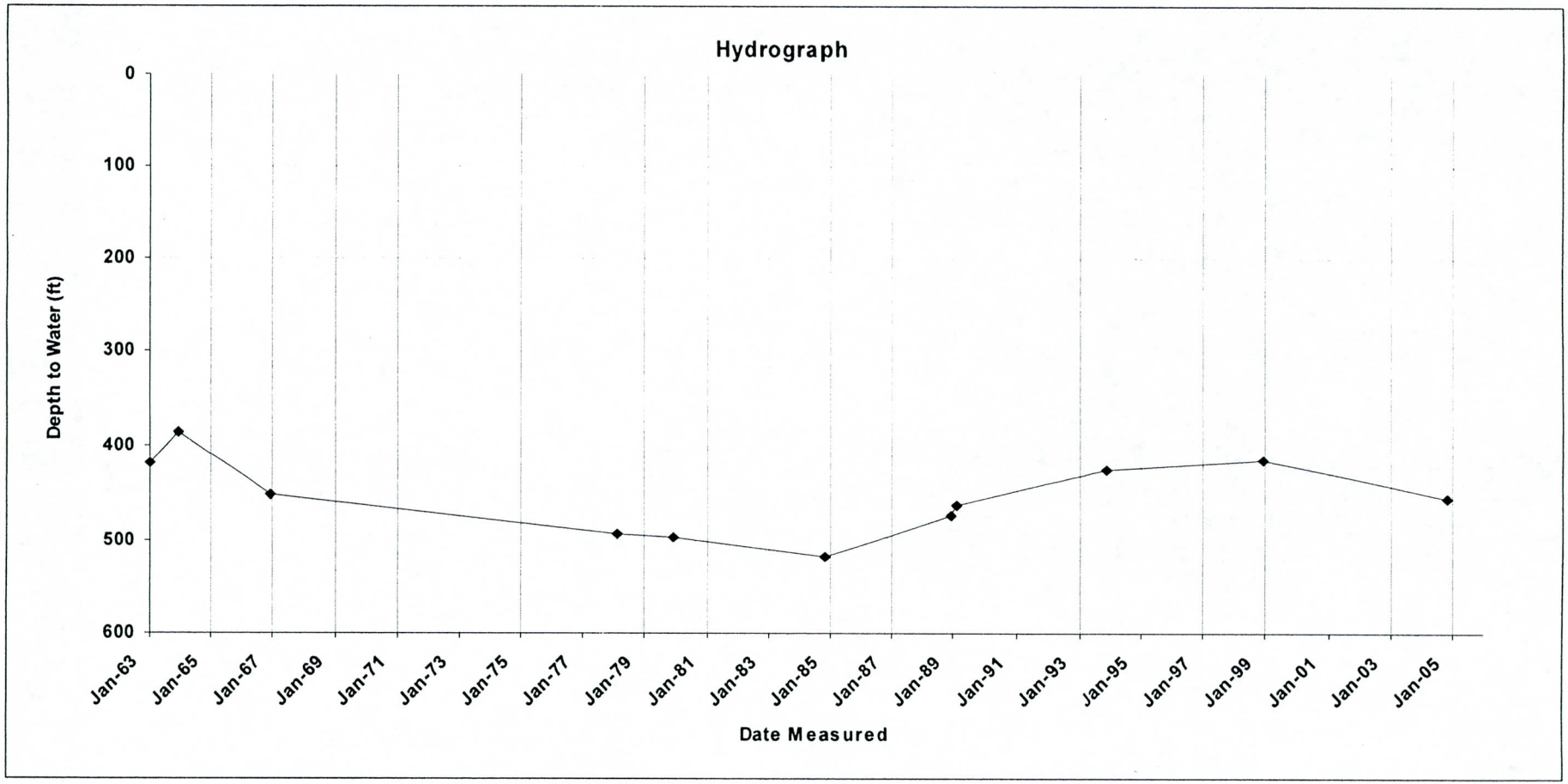
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-09 16BBB	333119113114401	629624	33° 31′ 19.5″	113° 11′ 42.3″	UNUSED	1400		20	11/30/2004	455.90	759.1	11



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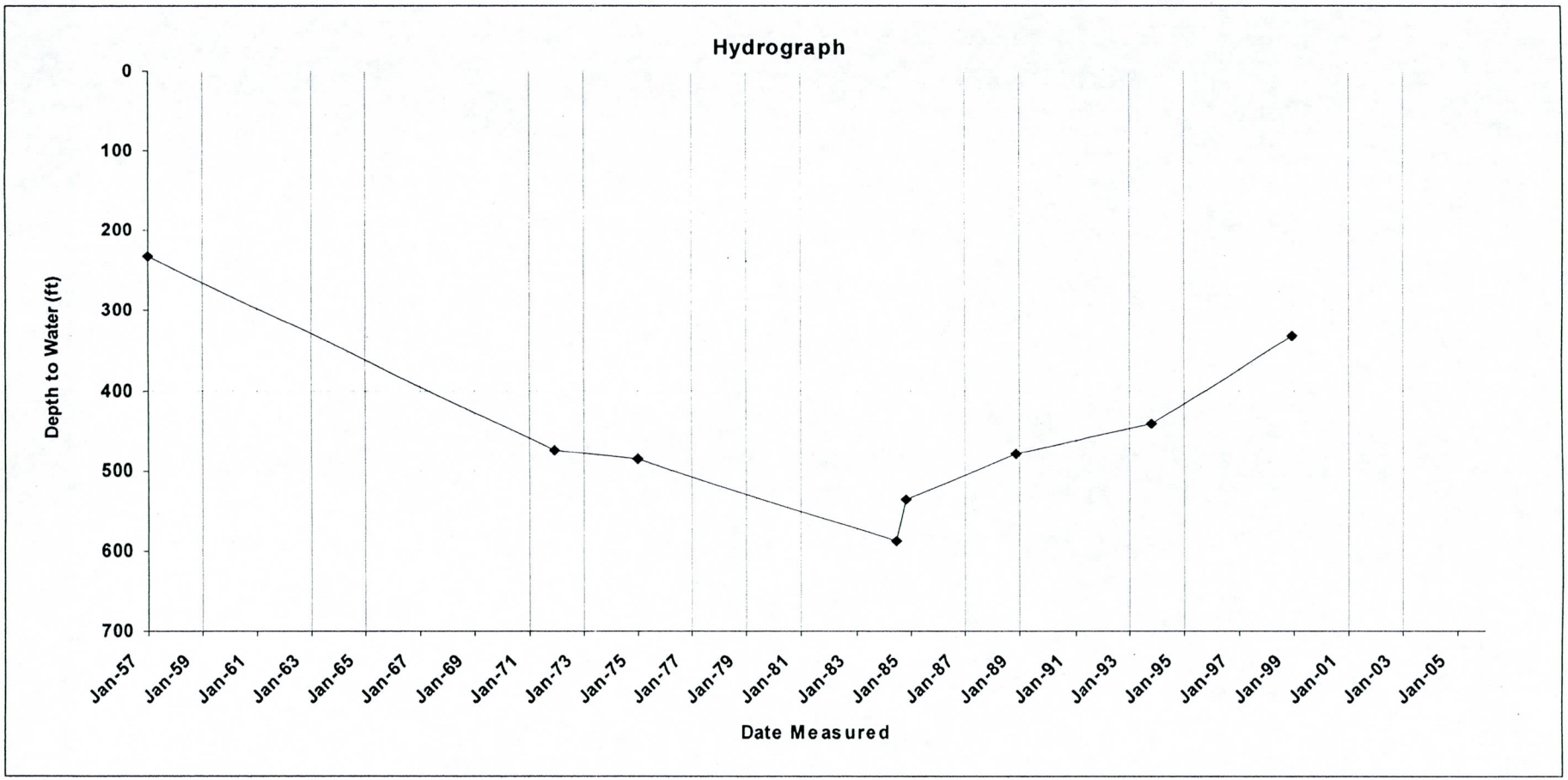
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-09 10ABB	333213113101001	611125	33° 32′ 13″	113° 10′ 10″	IRRIGATION	1500	1/1/1957	20	12/1/1998	333.40	889.6	8



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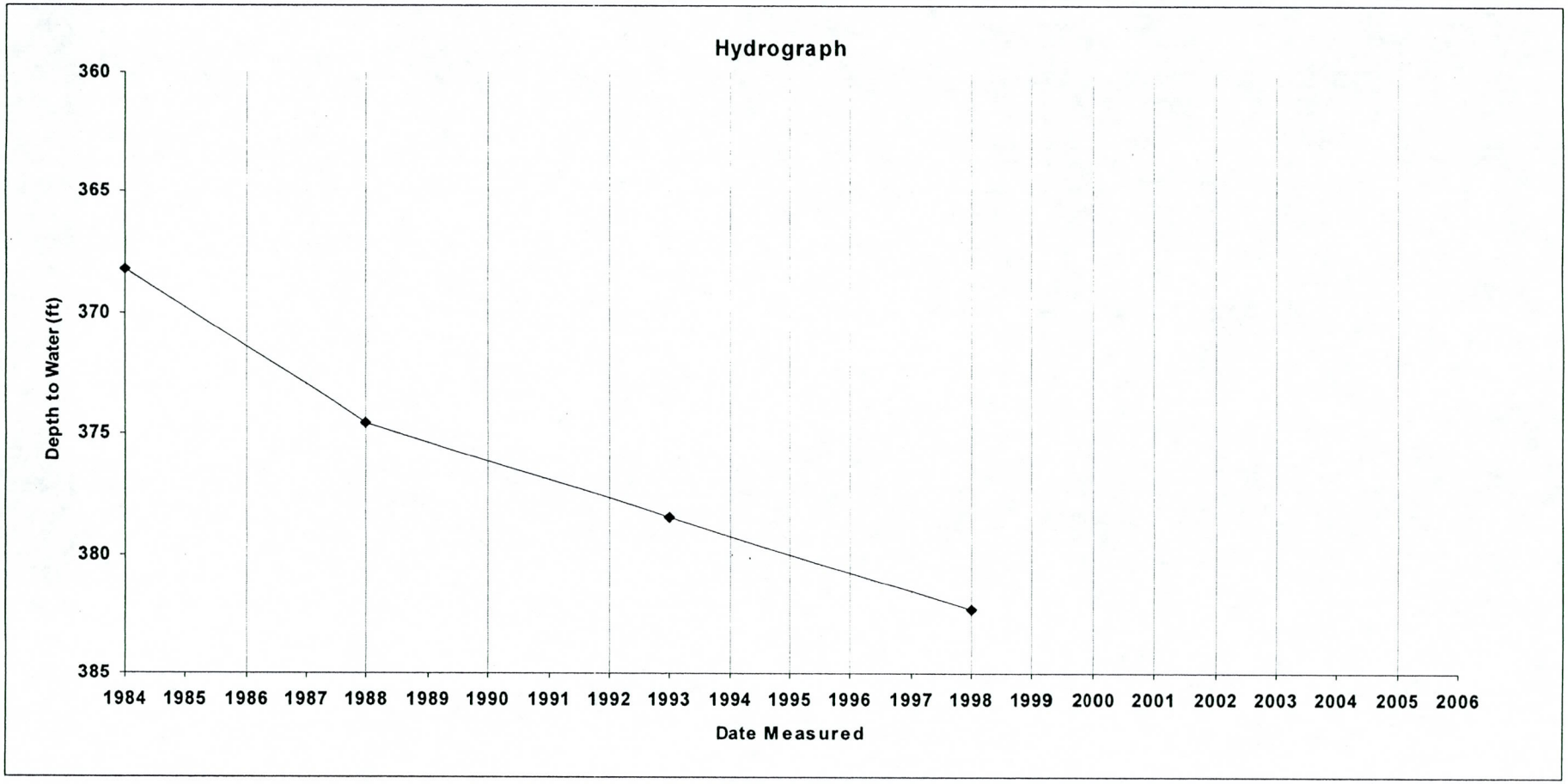
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-10 20ABB	333024113031901	803947	33° 30′ 24″	113° 18′ 19″	DOMESTIC	390	12/4/1975	8	11/30/1998	382.30	921	4



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Sunday, October 02, 2005

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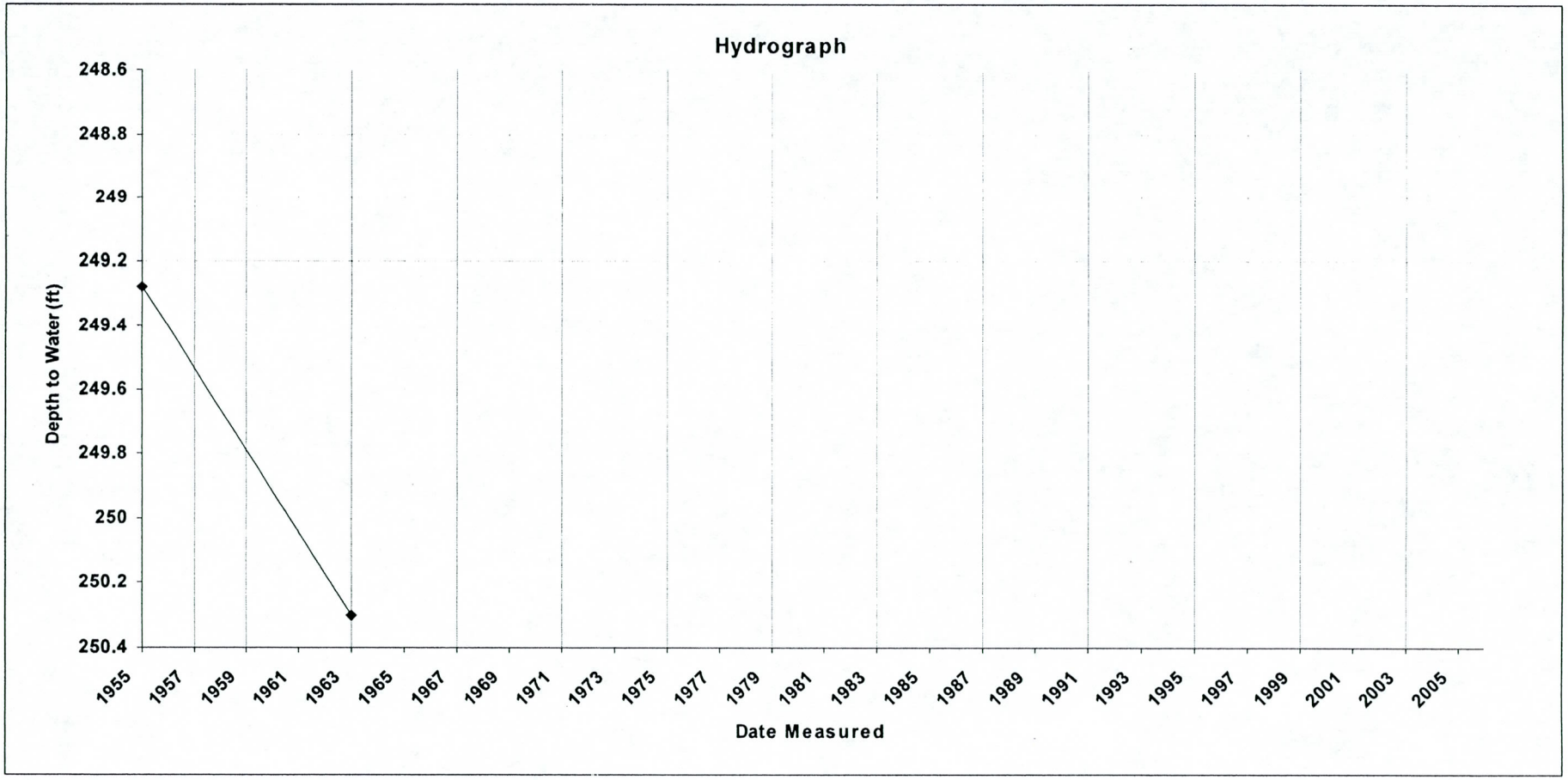
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-10 23BBA	333026113154001		33° 30′ 26″	113° 15′ 40″	UNUSED				12/2/1963	250.30	997.7	2



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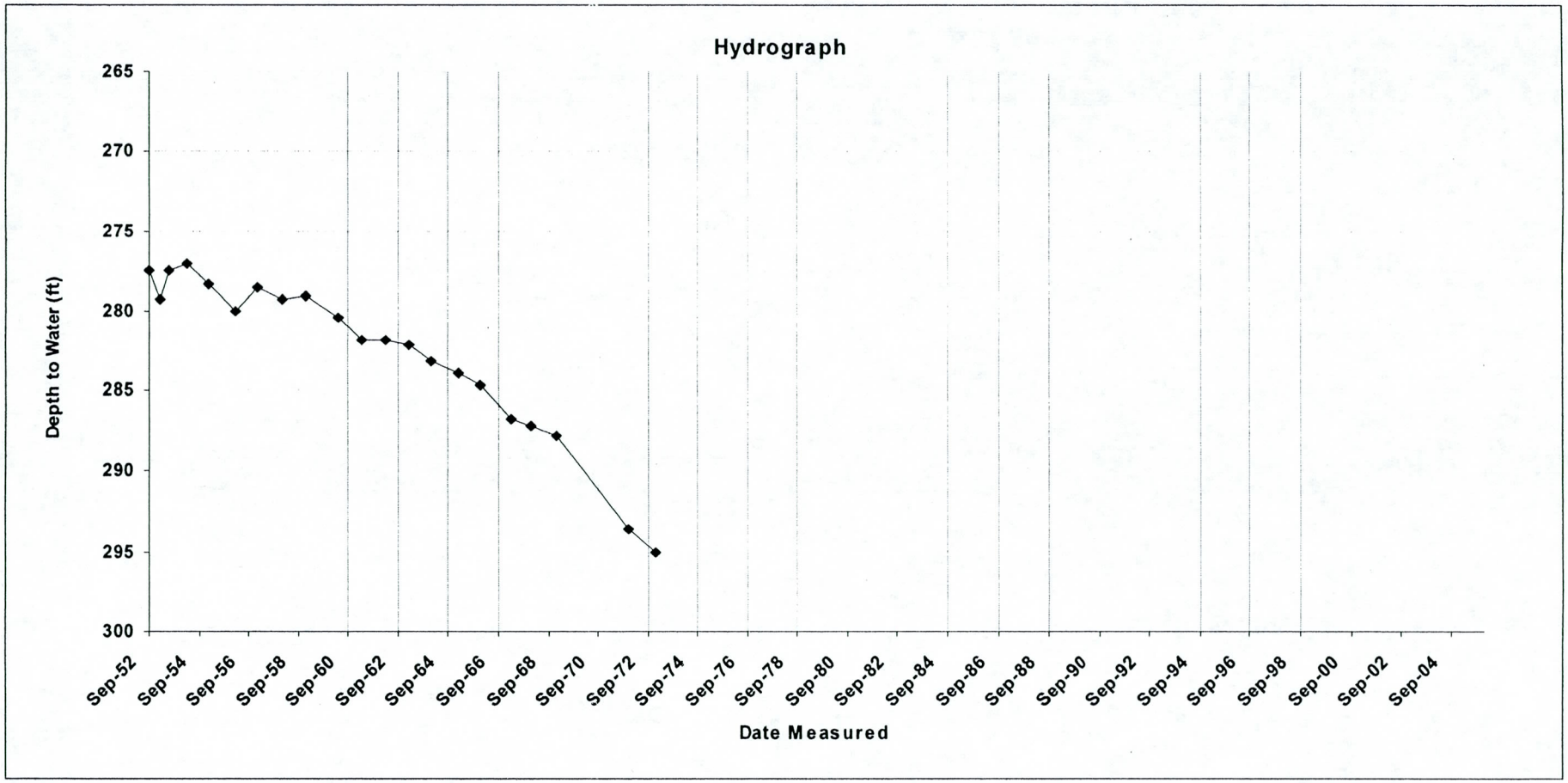
AZ Dept of Water Resources

Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-10 16BBB	333120113175901	614431	33° 31′ 20″	113° 17′ 59″	UNUSED	494			1/30/1973	295.00	983	21



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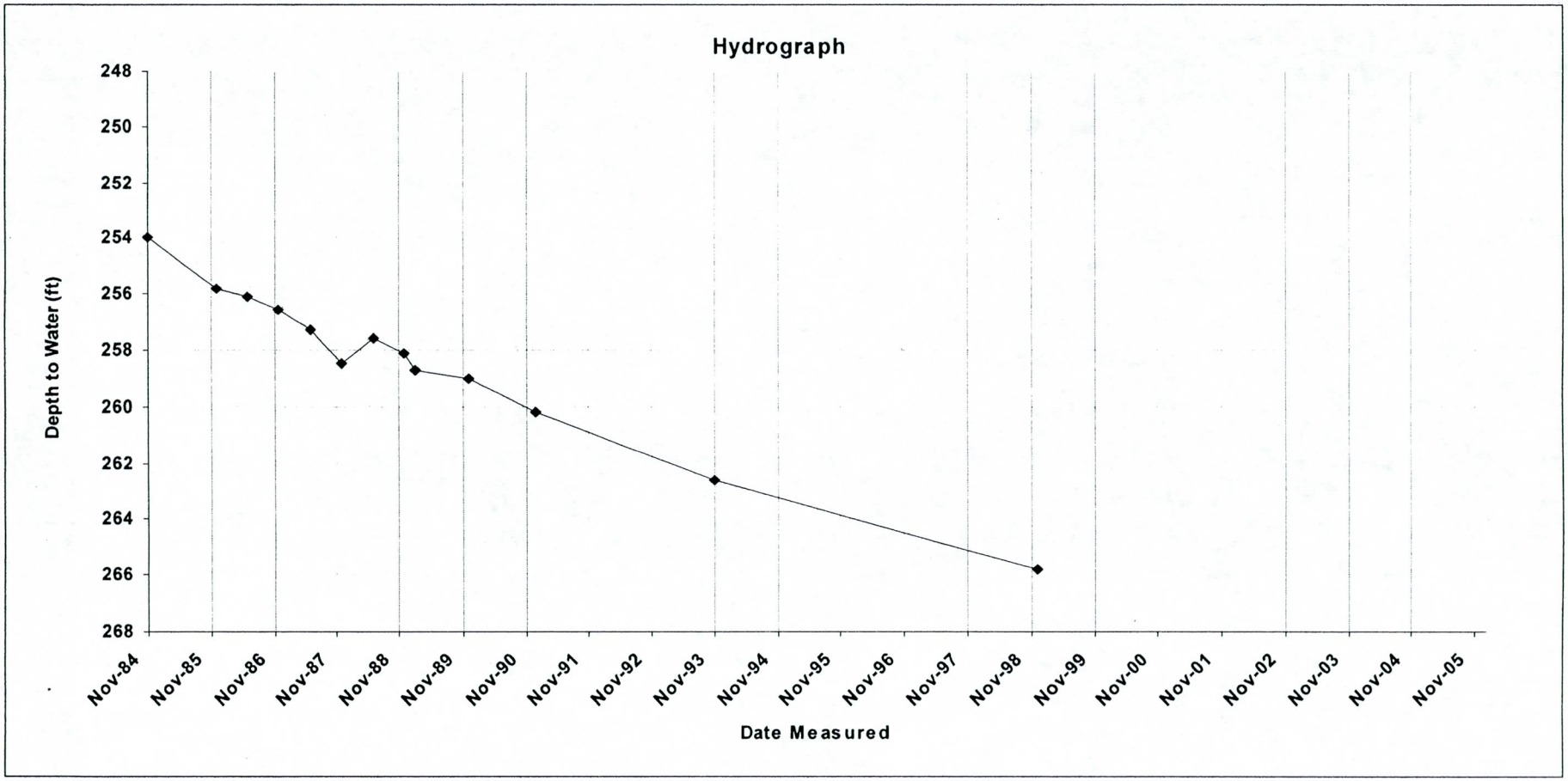
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-10 26DBB	332908113152001	607664	33° 29′ 8″	113° 15′ 20″	UNUSED	755	9/20/1981	14	12/9/1998	265.80	964.2	13



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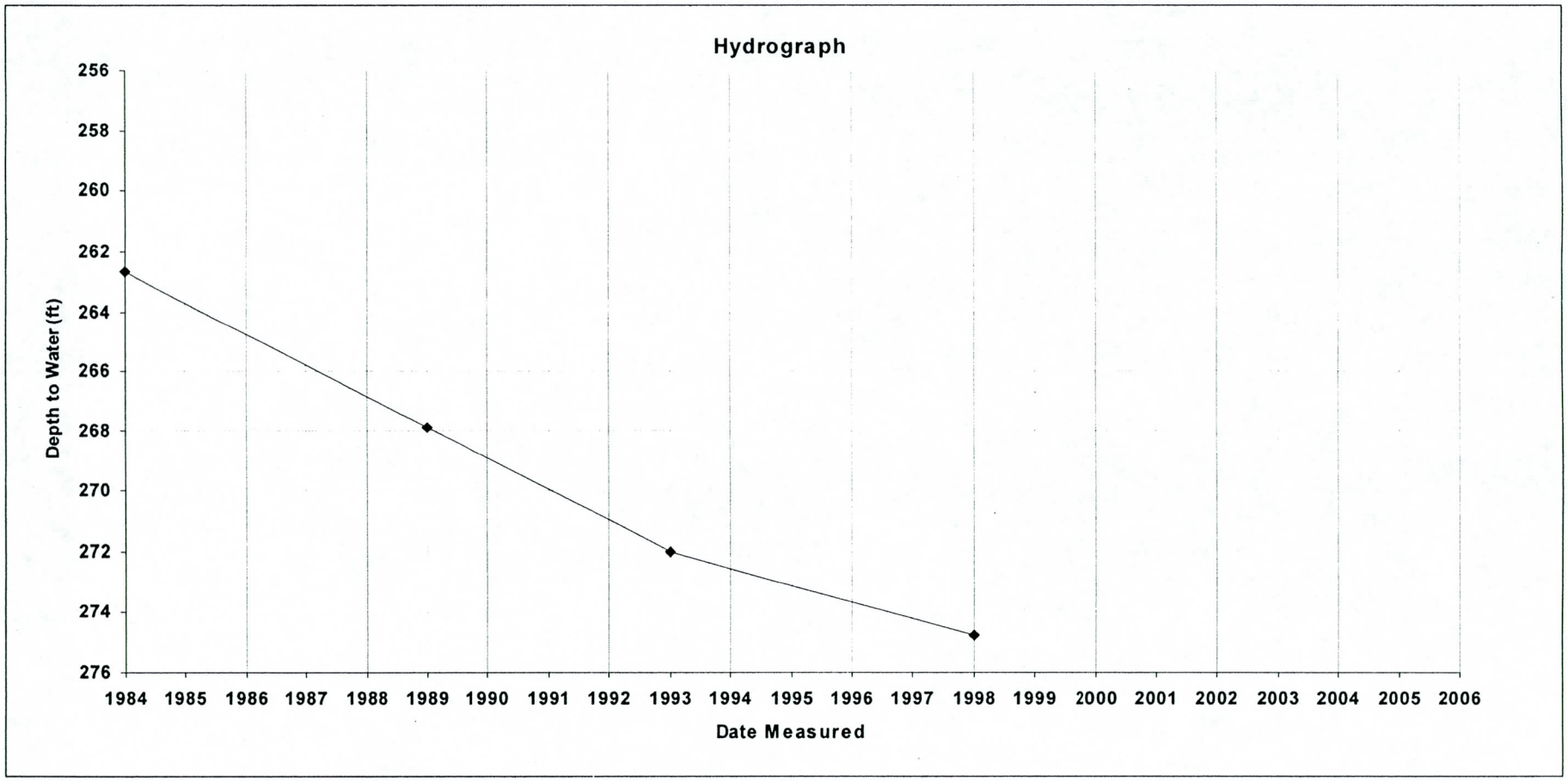
Local ID	Site ID	ADWR Reg. No.	Latitude	Longitude	Water Uses	Well Depth	Drill Date	Case Dia.	Latest WL Date	Depth to Water	WL Alt. above Mean Sea Level	Times Meas.
B-02-10 26DCC	332847113152101	607665	33° 28′ 47″	113° 15′ 21″	UNUSED	810	12/4/1981	16	12/9/1998	274.80	957.2	4



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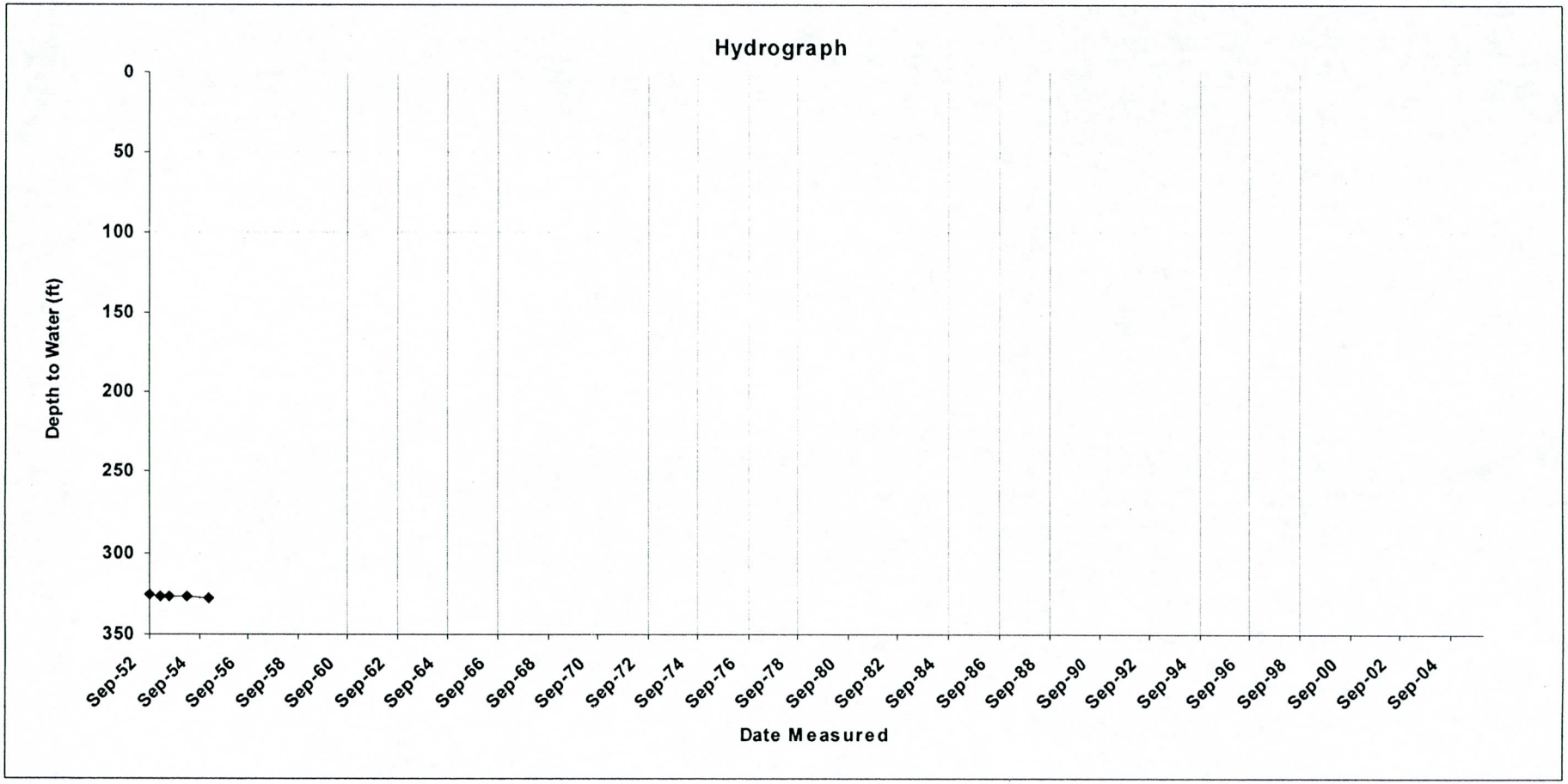
GWSI Well Report and Hydrograph

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



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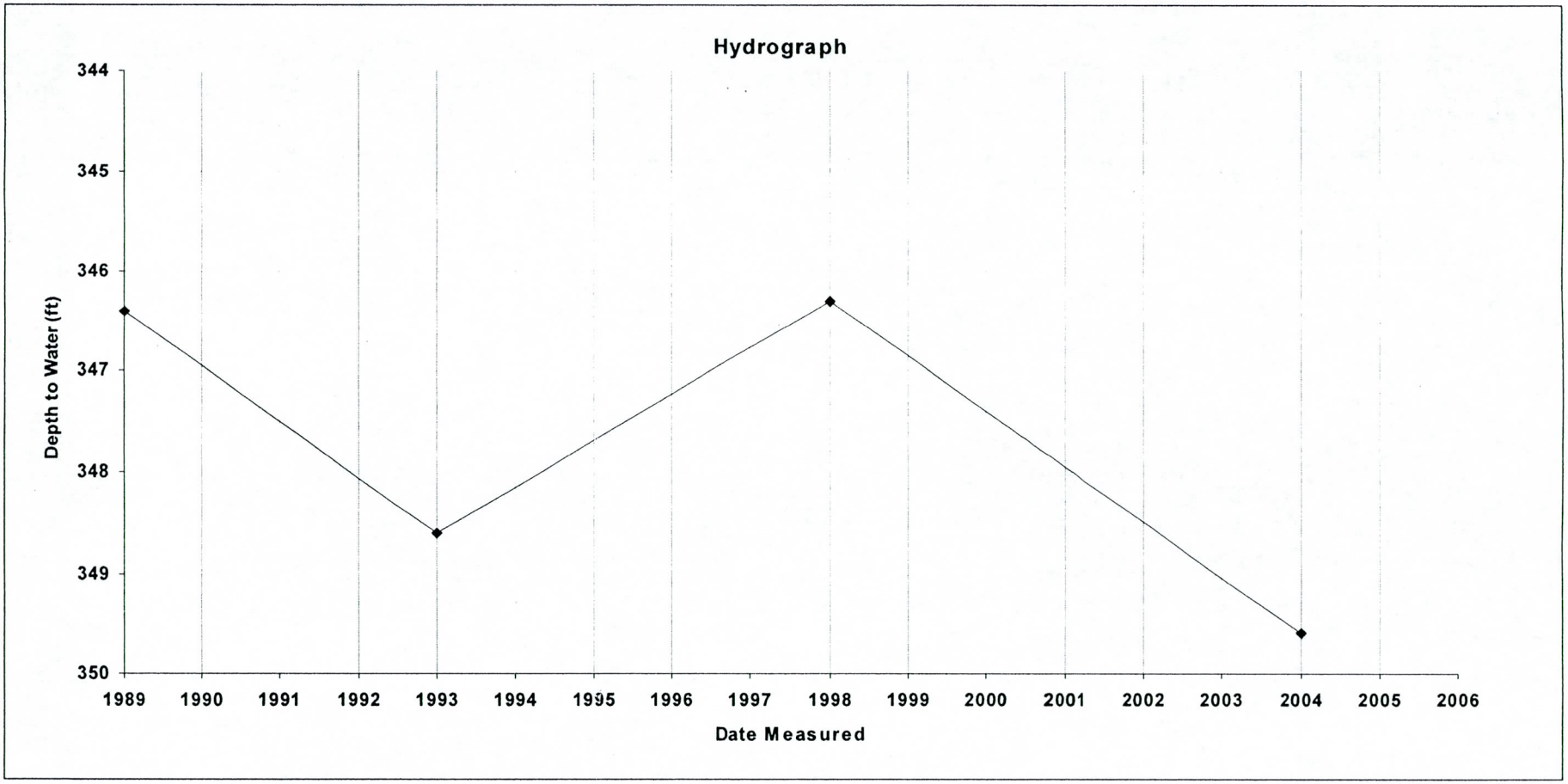
GWSI Well Report and Hydrograph

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-02-10 08DDD	333122113180201	520284	33° 31′ 21.2″	113° 17′ 59.5″	STOCK	605	2/23/1988	5	12/20/2004	349.60	926.4	4



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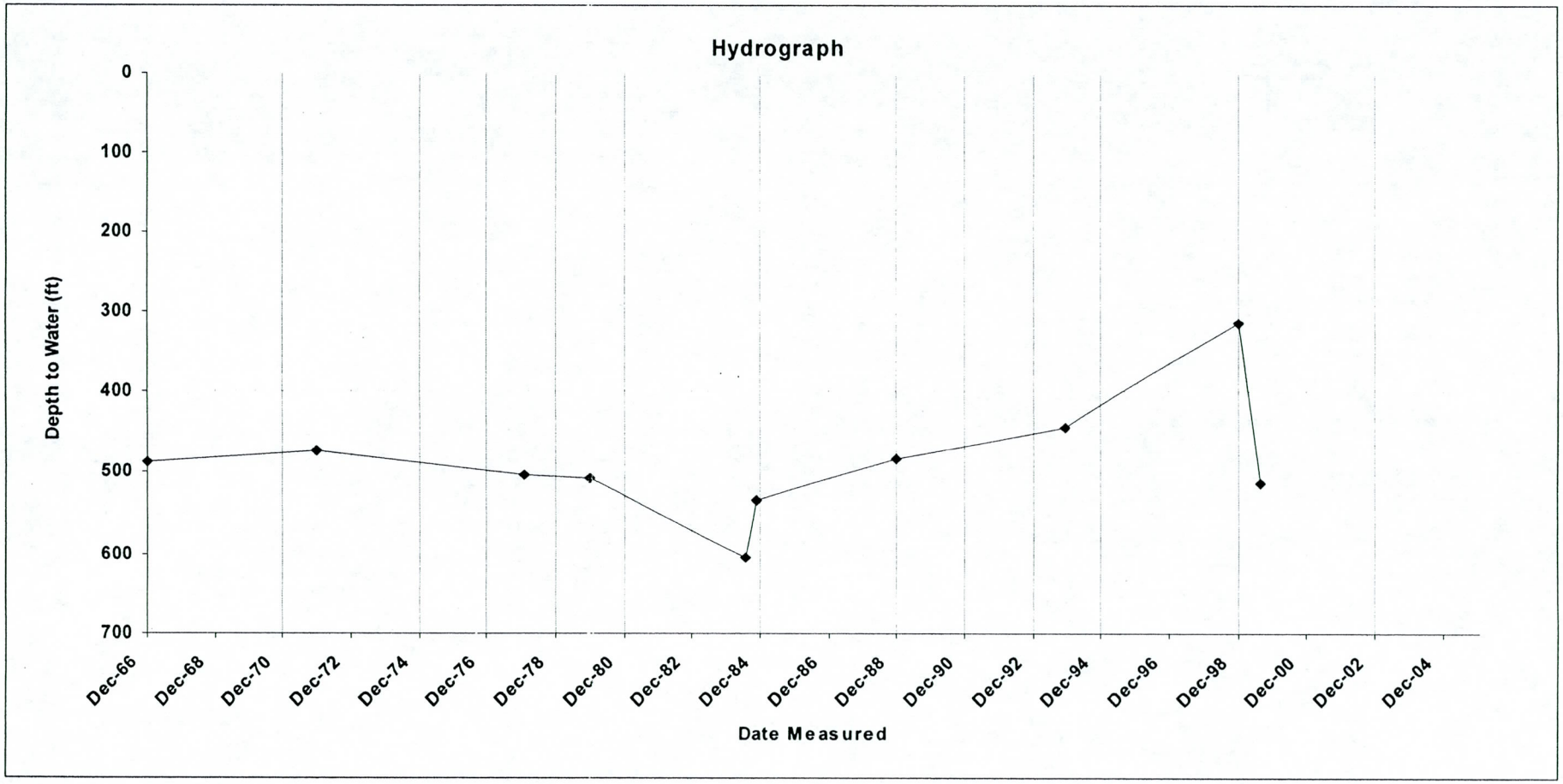
GWSI Well Report and Hydrograph

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 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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GEOTECHNICAL MEMORANDUM
Centennial Levee
Phase I Structures Assessment

Submitted to:
Kimley-Horn and Associates, Inc.
7878 N. 16th Street
Phoenix, AZ 85020

Submitted by:

Gannett Fleming, Inc.
4722 N. 24th Street Suite 250
Phoenix, AZ 85016

Registered Professional Engineer (Civil)
CERTIFICATE NO.
35654
DEAN B.
DURKEE
Date Signed 11-28-05
ARIZONA, U.S.A.

Frances Ackerman, R.G., E.I.T.
Project Geotechnical Engineering

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

November 28, 2005

043394

1.0 INTRODUCTION

The Centennial Wash Levee, referred to in the project files as the Centennial Reach of the Westside Canal, includes a southwest-trending levee and floodway extending from Interstate 10 approximately 5.1 miles to the Centennial Wash floodplain. Design documents indicate that the original design consisted of this section of the levee and floodway (Reach 1) as well as a southeast-trending section along Centennial Wash (Reach 2). However, Reach 2 was not constructed.

Centennial Levee is located immediately upstream of the Westside Main Canal, a portion of the Central Arizona Project (CAP) canal which conveys water from the Granite Reef Aqueduct to the Harquahala Valley Irrigation District. Upstream flood flows are diverted by the levee and floodway into Centennial Wash.

This Draft Geotechnical Memorandum presents the results of data review and interpretation conducted in support of an Individual Structure Assessment of Centennial Levee.

2.0 REVIEW OF PREVIOUS GEOTECHNICAL DOCUMENTATION

A comprehensive review of existing geotechnical reports and information 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)
- Preliminary Geologic Investigation of Centennial Levee (Flood Control Division Arizona Water Commission, October, 1974)
- Supplement No. 1 Harquahala Valley Watershed Engineering Documentation, Volume II Centennial Levee (May, 1976)
- Supplemental Watershed Workplan, Harquahala Valley Watershed (Flood Control District of Maricopa County, 1977)
- Land Subsidence Potential Hawuahala (sic; Lewis E. Scott, June, 1983)
- SCS Memorandum Re: Geology – Centennial Wash Levee, Reach 1, Harquahala Valley Watershed, Maricopa County, Arizona (SCS, December 1, 1983)
- SCS Memorandum Re: ENG Soil Mechanics – Arizona (WE-08) Centennial Wash Levee, Reach 1, (SCS, December 21, 1983)
- SCS Memorandum Re: ENG Soils – Centennial Wash Levee, Reach 1, Harquahala Valley Watershed, Maricopa County, Arizona (SCS, March 9, 1984)
- US Sprint Cable Crossings Plan Sheet 1 of 1 (Lemme Engineering Inc., December 22, 1986)
- Centennial Wash/Westside Canal soil data binder from Natural Resources Conservation Service (NRCS, formerly SCS) files
- NRCS correspondence files for Centennial Levee, Reach 1
- NRCS correspondence files for Centennial Levee, Reach 2
- Harquahala Valley Irrigation District Central Arizona Project Distribution System Design Memorandum No. 1 (Franzoy, Corey and Associates, Inc., undated)

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

2.1 Regional Geologic Setting

Information on the regional setting of the Centennial Levee was summarized from FCDMC (1967).

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 Township 2 North, Range 9 West. Farther west, near Sections 34, 35 and 36, Township 3 North, Range 11 West, 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-grained or moderately fine-grained surface soils and subsoils are present on the smoother slopes near the center of the valley. Coarse-grained or moderately coarse-grained soils are present on the upper fans of washes from the granitic mountains. Along the foot of the mountains, there is usually an area of shallow to

moderately deep residual soils. These residual soils often have a medium-textured surface with gravel that is covered with dark desert varnish, and have slightly finer subsoils underlain at 12 to 28 inches by a strongly-cemented lime hardpan. Valley–fill alluvial soils originate in the granite, granite gneiss, schist, limestone, andesite, basalt, and shale rocks of the adjacent mountains. The soils in the plain are slightly to moderately erosive. Because the land surface is relatively flat and a sheet flow runoff condition prevails, erosion is generally not significant. Erosion is active in some of the channels and diversions constructed in and around the cultivated areas where flood flows are concentrated. Generally, the soils have a slow to very slow rate of water transmission and a slow to very slow infiltration rate when thoroughly wetted because of moderately-fine to fine texture or a layer that impedes downward movement of water.

2.2 Foundation Conditions

A Preliminary Geologic Investigation was conducted in 1974 (Flood Control Division Arizona Water Commission, 1974) along an early project alignment approximately 1 mile downstream from the final alignment. A subsurface investigation consisting of test pits and soil sampling was performed by SCS in 1983 along the final project alignment, near the centerline for the proposed Westside Main Canal. No formal report of the 1983 investigation was available for review during the preparation of this Geotechnical Memorandum, however, the test pit logs and soil test data were reviewed. It is not known whether a formal investigation report was prepared for the project. Because of the relocation of the levee alignment and because of the limited scope of the Preliminary Geologic Investigation, the information contained in this Memorandum is based on the results of the 1983 investigation. However, it should be noted that the results of the Preliminary Geologic Investigation are similar to the results obtained in 1983.

Centennial Levee crosses a broad, southeastward-sloping alluvial plain. Well-defined stream channels are not present and flood flows are generally infrequent. Subsurface materials are generally alluvial in origin, with some surficial eolian deposits, and consist primarily of silts and clays with some sands. Younger alluvial deposits overlie older alluvial deposits, with a zone of moderately (calcium carbonate) cemented materials marking the division between the deposits.

The 1983 soils investigation included excavation of forty backhoe test pits and collection and testing of sixty-one soil samples along the proposed centerline of the Westside Main Canal. The results of laboratory testing on these soil samples are included in Attachment A. Backhoe test pits from the 1983 soils investigation indicate that "younger alluvium and eolian" deposits overlie "older alluvium" along the length of the levee alignment. The thickness of the "younger alluvium and eolian" deposits ranges from 2.0 feet to 15.0 feet based on information contained in the test pit logs.

Foundation soil laboratory test data from the 1983 soils investigation are summarized in Tables 1 and 2. More than 3/4 of all soil samples were fine grained, mostly silts, clayey silts and clays. Although the majority of soil samples were classified as ML, they were reportedly expected to behave like fine-grained silty sand (SM), possibly susceptible to erosion. These soils were also characterized as having relatively strong bearing strength for fine-grained soils (SCS, 1984). It was noted that field classification of these soils

based on the #200 sieve (fine-grained vs. coarse-grained) was very difficult. This was reflected in the soil descriptions contained on the test pit logs; thirty-three of the sixty-one soil samples were classified incorrectly in the field. The field classifications generally described the soils as being coarser-grained than the laboratory classifications.

All soil samples contained some clay particles: between 1% and 37% with average of 16% by weight, suggesting that clay mineral behavior will influence the soils. Palygorskite (also known as attapulgite), a tubular clay mineral, was reportedly present in nearly all soils samples, most of which have approximately equal amounts of palygorskite, montmorillonite and illite (SCS, 1984). Little was known of the engineering properties of palygorskite at the time Centennial Levee was designed and constructed (SCS, 1984). Palygorskite is a hydrated magnesium aluminum silicate similar to bentonite in its ability to absorb water (SCS, 1984) but differs from bentonite in its thin elongate chain type structure. The clay minerals present in the soil samples are moisture sensitive and tend to swell when wet and shrink when dry.

2.2.1 Younger Alluvium and Eolian Deposits

The results of the laboratory testing of the younger alluvial soils are summarized in Table 1. The uppermost alluvial and eolian deposits consist of uncemeted silty fine sands (SM), silts with fine sand (ML), clayey silt (CL-ML), and clay (CL). These deposits have an average thickness over the length of Centennial Levee of approximately 3 to 4 feet, but were reported to be present to depths of 15 feet (TP 1031; SCS, 1983a). The younger alluvial deposits have average in-field densities of approximately 96 pounds per cubic foot (pcf) and average fines content of 53%. The fines were generally non-plastic or had low plasticity index (PI) values.

The strength of the younger alluvial soil was evaluated using four triaxial shear tests (one from the centerline of the embankment and three from the borrow areas). The tests were performed wet of optimum at densities of 90% of Standard Proctor density. The specimens were back-pressure saturated and pore pressures were measured so that effective stress parameters as well as total stress parameters could be measured. The total stress cohesion of the younger alluvial soil samples ranged from 400 pounds per square foot (psf) to 675 psf and the effective cohesion ranged from 100 psf to 250 psf. The total stress angles of internal friction ranged from 11.5 degrees to 16.5 degrees and the effective stress angles of internal friction ranged from 28.5 degrees to 33 degrees.

Swell tests were performed on three younger alluvial soil samples from the borrow areas. Tests were performed on samples compacted at optimum moisture content and densities of 90% and 95% of Standard Proctor density. The swell measured at 90% Standard Proctor density ranged from 0.44% to 0.84% and the swell measured at 95% Standard Proctor density ranged from 0.52% to 1.32%.

Table 1. Summary of younger alluvial soils laboratory test data

Sample ID	USCS	PI	γ_d (pcf; field)	% fines	Swell (%)		Shear strength (@90% Std Proctor density)			
					90% Std Proctor density	95% Std Proctor density	ϕ (deg)	c (lb/ft^2)	ϕ' (deg)	c' (lb/ft^2)
1001.1	SM	NP	106.0	25						
1001.2	ML	6	81.9	60						
1002.1	CL	17	91.3	69						
1003.1	CL-ML	6	89.7	52	0.44	0.52	11.5	475	28.5	225
1003.2	CL	9	100.6	64						
1004.1	CL-ML	7	94.1	68						
1004.2	CL	13	93.4	67						
1005.1	CL	11	103.6	73	0.84	1.32	13.5	400	28.5	250
1005.2	CL	16	98.1	65						
1006.2	SC	20	103.3	15						
1007.1	CL	12	92.2	71						
1007.2	CL	15	89.6	66						
1008.1	CL	13	89.3	63						
1008.2	CL-ML	4	92.9	55						
1009.1	ML	2	90.6	77						
1010.1	CL	13	90.5	68	0.54	0.44				
1011.1	ML	3	90.9	72						
1011.2	CL-ML	5	102.6	67						
1011.3	ML	NP		66			16.5	675	33	100
1012.1	ML	NP	89.7	67						
1013.1	ML	NP	93.1	54			13	675	32	100
1013.2	SM	NP		40						
1014.1	SM	NP	102.1	25						
1019.1	SW-SM	NP	99.0	7						
1020.1	SM	NP	94.7	49						
1021.1	ML	1	97.0	52						
1023.2	CL-ML	6	99.9	73						
1024.1	ML	1	96.2	55						
1024.2	ML	NP	93.0	70						
1025.1	SM	NP	103.1	21						
1026.1	SM	1	99.8	40						
1026.2	SM	NP		31						
1027.1	ML	2	95.7	62						
1029.1	ML	NP	97.4	51						
1031.1	ML	NP	87.8	80						
1031.2	CL-ML	6	86.8	58						
1032.1	SM	NP		41						
1033.1	ML	2	103.6	58						
1035.1	SM	1	112.4	42						
1036.1	ML	NP		62						
1037.3	CL-ML	4		74						
1039.3	SM	NP		23						

2.2.2 Older Alluvium

The results of the laboratory testing of the older alluvial soils are summarized in Table 2. The older alluvial deposits are generally moderately cemented with calcium carbonate. The older alluvial material consists primarily of clays (CL), silts (ML), and clayey silt (CL-ML) with some fine-grained silty sand (SM). The amount of cementation is variable and, in some places, layers of moderately permeable fine- to coarse-grained sands with some gravel were encountered (SCS, 1983a). The average dry density of the cemented alluvial deposits was approximately 102 pcf and average fines content of 62%. The fines were generally non-plastic or had low PI values.

The strength of the older alluvial soil was evaluated with one triaxial shear test using the same methods as described above. The total stress cohesion and effective stress cohesion values obtained from tests on the older alluvial soil samples were 575 psf and 0 psf, respectively. The total stress angle of internal friction and the effective stress angle of internal friction of the older alluvial soils were 33 degrees and 37 degrees, respectively.

Table 2. Summary of older alluvial soils laboratory test data

					Shear strength (@90% Std Proctor density)			
Sample ID	**USCS**	**PI**	**γ_d (pcf; field)**	**% fines**	**ϕ (deg)**	**c (lb/ft^2)**	**ϕ' (deg)**	**c' (lb/ft^2)**
1012.2	ML	4	98.4	84				
1014.2	CL-ML	6	100.9	51				
1016.1	CL	8	106.8	66				
1021.2	SM	NP	113.2	24				
1021.3	ML	2		65				
1022.1	ML	2	94.0	60				
1023.1	ML	2		62				
1027.2	CL	9	100.0	71				
1029.2	CL	12	99.3	58				
1032.2	CL-ML	7		83				
1033.2	SM	2	103.9	41				
1034.1	CL-ML	4		59				
1037.1	CL	8	110.5	67				
1037.2	ML	NP	106.5	62				
1039.1	ML	1	106.8	56				
1039.2	ML	2	94.6	69	33	575	37	0
1040.1	ML	NP	106.2	53				

2.3 Embankment

The Centennial Levee embankment section was based on the design criteria from the Soil Conservation Service National Engineering Handbook, Section 2, Engineering Design Standards, Part 1: Dike and Levee for a Class II dike. Centennial Levee was designed as a homogenous earth embankment with 4H:1V upstream and 3H:1V downstream slopes, maximum height of 9.5 feet, 12-foot crest width and a continuous central foundation cut-off trench with a depth of at least 6 feet and base width of 12 feet. A typical cross-section of the embankment is shown as Figure 1.

The Centennial Levee crest elevation varies from 1271.80 feet above mean sea level (amsl) at Station 0+00 to 1268.92 feet amsl at Station 224+00. The elevation slope is generally 0.0002 ft/ft toward Centennial Wash, with elevation transitions at three locations along the embankment. Between Station 58+80 and Station 59+00 the elevation increases 1 foot; between Station 156+80 and Station 157+00 the elevation increases ½ foot; and between Station 196+80 and Station 197+00 the elevation increases 1 foot. The embankment crest and floodway elevations are represented graphically on Figure 2.

2.3.1 Embankment Materials

The materials used to construct the embankment were derived from local borrow sources in the vicinity of the Levee. Eight of the sixty-one soil samples collected and tested during the 1983 soils investigation were identified by SCS as being representative of borrow materials. Four of the representative borrow material samples were collected from along the Westside Main Canal centerline and four were collected from the floodway area. The results of the laboratory testing are summarized in Table 3. The as-built plans indicate that the embankment was to be constructed of soils containing not less than 15% passing the No. 200 sieve. No as-built soil data for the Levee were available for review.

According to the soil test data, the borrow materials consist of silts (ML), clays (CL), clayey silts (CL-ML) and silty sands (SM). The average fines content for the borrow materials was 60%. The fines were generally non-plastic or had low PI values. The permeability of the borrow soil samples (measured at 90% Standard Proctor density) ranged from 0.01 ft/day to 1.2 ft/day. Pinhole tests performed on two of the borrow area samples indicate that the soil is not dispersive. Calcium carbonate was present in all eight borrow area samples between 2% and 9%. This reportedly represented lime nodules dispersed in the soil matrix or possibly some cementation (SCS, 1984).

Standard Proctor test data for the borrow area soils indicated that the maximum dry density of the soil ranges from 112.5 pcf to 121.5 pcf and can be achieved at moisture contents of between 9.5% and 14.5%. The SCS recommended that the project specifications require "a workable mix with a minimum moisture content of 2% less than the optimum moisture" (SCS, 1984).

Table 3. Summary of borrow soils laboratory test data

Sample ID	USCS	PI	γ_d (pcf; field)	% fines	Swell (%)		Permeability (90% SP; ft/day)	Moisture-density relationship γ_{dmax} (pcf)/opt m (%)	Shear strength (@90% Std Proctor density)				Compaction		Pinhole test	$CaCO_3$ (% by weight)
					90% Std Proctor density	95% Std Proctor density			ϕ (deg)	c (lb/ft^2)	ϕ' (deg)	c' (lb/ft^2)	90% Std Proctor density	95% Std Proctor density		
1003.1	CL-ML	6	89.7	52	0.44	0.52	0.03	118.0/13.0	11.5	475	28.5	225	0.44	0.52	ND	8
1005.1	CL	11	103.6		0.84	1.32	0.03	112.5/14.5	13.5	400	28.5	250	0.84	1.32	ND	9
1006.1	ML	2	92.5													5
1009.1	ML	2	90.6	77			0.02	114.0/13.5								3
1010.1	CL	13	90.5	68	0.54	0.44	0.01	113.5/14.5					0.54	0.44		
1013.1	ML	NP	93.1	54			0.14	114.0/12.0	13	675	32	100				2
1020.1	SM	NP	94.7	49			0.12	114.5/12.0								
1035.1	SM	1	112.4				1.2	121.5/9.5					0.16			2

The results of the swell tests performed on the alluvial materials collected from the borrow areas are summarized on Section 1.2.1. SCS reported that the swelling would likely result in the levee surface swelling and shrinking in response to changes in moisture content and that this may cause or aggravate embankment cracking (SCS, 1984). Furthermore, one clay sample (#1005.1) swelled more when compacted at 95% Standard Proctor density than at 90% Standard Proctor density, indicating that the soils may react poorly to overcompaction. Based on this information, it was recommended that the levee not be used as a haul road and that the final density/compactive effort be limited (SCS, 1984). Though this is not an accepted method for mitigating moisture sensitive soils, the swell potential based on tests performed at 90% and 95% of standard Proctor maximum density is quite low. Because construction inspection reports were not available for review during the preparation of this Draft Geotechnical Memorandum, it is not known whether this recommendation was implemented during construction but based on the swell potential reported it is not likely that expansive soils within the compacted embankment are presently an adverse condition.

2.3.2 Embankment Construction

Foundation treatment recommendations called for removal of the loose surface soils to a depth of at least 6 feet, with the final depths to be determined by the engineer after inspection of the materials encountered. The as-built plans indicate that the surface soils were removed to depths of between 3 and 9 feet. According to the SCS (1984), the geologist recommended removal of at least 3 feet of surface soils to destroy animal burrows and consolidate surface soils. There is no indication in the files that consideration was given to removing all of the less dense younger alluvial soils to prevent potential collapse consolidation and the often associated embankment cracking. This, taken together with the depth of the cutoff indicated on the as-built plan set (3 to 9 feet), suggests that potentially collapsible younger alluvial soil deposits may be present beneath the Levee. No construction inspection reports were available for review during the preparation of this Draft Geotechnical Memorandum.

2.3.3 Embankment Penetrations

In 1986, US Sprint Communications Company constructed a cable crossing of Centennial Levee. The construction consisted of removing approximately 120 feet of the embankment near the left abutment, within the existing Arizona Department of Transportation right-or-way, placing a cable within a 4-inch black iron pipe conduit and reconstructing the compacted embankment.

The cable crossing design plan included provision of a filter diaphragm surrounding the pipe conduit. The design specified a 4 foot by 4 foot, 3 foot thick compacted sand diaphragm located beneath the upstream side of the Levee crest. The sand was specified to be ASTM C-33 material. Beneath the conduit, low density or other unsuitable material was to be excavated and replaced with a minimum of 1 foot of select compacted material. The pipe bedding was specified to be 0.37 foot (same as outside diameter of pipe) select bedding material compacted by saturation and internal vibration. As-built information was not available for review.

2.4 Floodway

A floodway was constructed upstream from Centennial Levee to direct floodwaters toward Centennial Wash. The floodway channel centerline is 260 feet upstream from the centerline of the Levee. The floodway was constructed with 50H:1V side slopes. The channel bottom is 80 feet wide and slopes toward Centennial Wash at 0.0002 ft/ft (see Figures 1 and 2).

The floodway was designed to accommodate a relatively low flow velocity to prevent scour in the easily erodible soils (SCS, 1976). The floodway includes a soil cement section from Station 215+87 to Station 225+75 for erosion protection. The soil cement extends 415 feet from the upstream toe of the Levee and has a compacted thickness of at least 6 inches. A rock sill was specified at the floodway outlet. The rock sill is 505 feet long, perpendicular to the floodway at Station 225+75. The sill consists of a 6 foot deep and 10 foot wide section filled with "4 inch to 24 inch rock sill material" according to the construction plans. The rock sill was presumably constructed to prevent headcutting and scour along the floodway as water discharges to Centennial Wash.

2.5 Original Slope Stability Analysis

No record of slope stability analyses performed during levee design was available for review. It is not know whether these analyses were performed for design.

3.0 RECOMMENDATIONS

3.1 Supplemental Geotechnical Analyses

A cutoff trench was constructed in the area beneath the crest of the levee to a depth of between 3 and 9 feet, which likely resulted in removal of collapsible soil in that area. However, potentially collapsible younger alluvial soil deposits may be present beneath upstream and downstream sections of the levee. The remaining collapse potential and risk of wetting of potentially collapsible soils in the foundation could be evaluated through appropriate sampling, testing, and analysis.

The foundation soils beneath the crest and upstream section of the dam can be sampled using split-spoon (California) samplers, pitcher sampler, or block samples from test pits and collapse consolidation tests could be performed to determine the current state of stress and collapse potential.

3.2 Documentation of Slope Stability Factors of Safety

Although at the time Centennial Levee was designed, the Federal Emergency Management Agency (FEMA) did not have a comprehensive levee policy, an interim levee policy had been established in 1981 (FEMA, 1981). The interim policy indicated that guidance on new levee construction should be addressed by referring to the minimum requirements contained in the United States Army Corps of Engineers (USACE) "Design and Construction of Levees" (EM 1110-2-1913). Current USACE guidelines (as of April 2000) include provisions for slope stability analysis.

Under reasonable loading conditions for Centennial Levee, it is expected that both upstream and downstream slopes will be stable. However, documentation of slope stability factors of safety for specified loading and design criteria is not available. Slope stability analyses are recommended to document factors of safety for Centennial Levee. In addition, this information would be required for FEMA certification of Centennial Levee. The stability analyses and factors of safety required by the USACE for existing levees are summarized in Table 4.

Table 4. USACE Slope Stability Design Criteria for Existing Levees

Loading Condition	Required Factor of Safety
Long-Term (steady seepage)	1.4
Rapid Drawdown (upstream slope)	1.0 to 1.2
Earthquake	[a]

[a] See ER 1110-2-1806 for guidance. An EM for seismic stability analysis is under preparation.

4.0 REFERENCES

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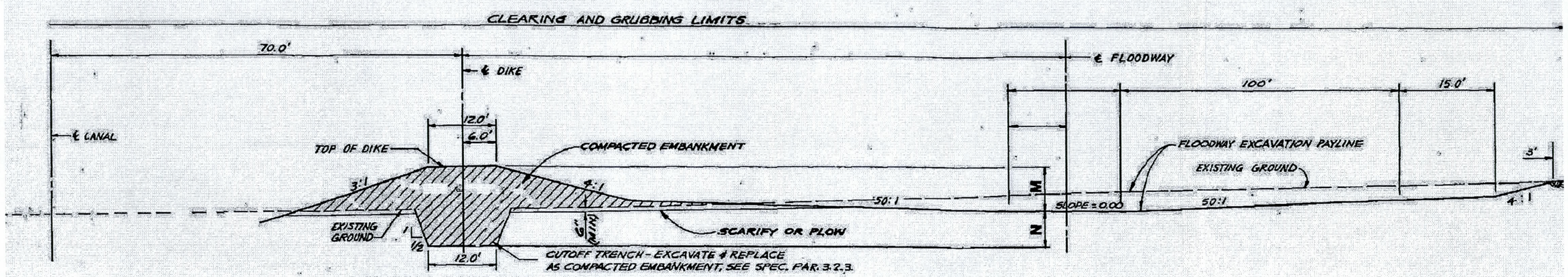
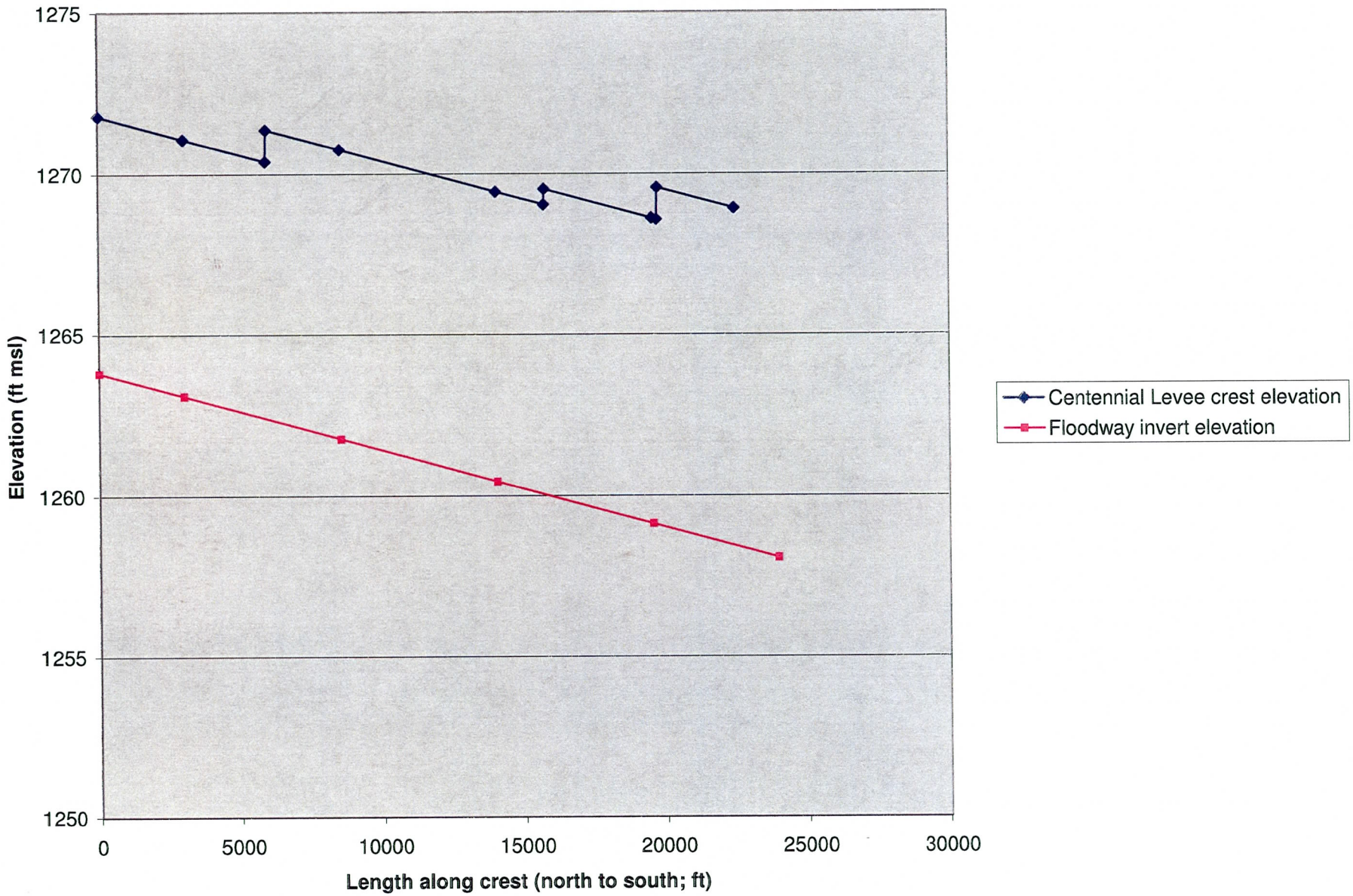
SCS, 1983b. Memorandum to Ralph M. Arrington. Subject: ENG – Soil Mechanics – Arizona (WE-08) Centennial Wash Levee, Reach 1, SCS. December 21, 1983.

SCS, 1984. Memorandum to Ralph M. Arrington. Subject: ENG – Soils – Centennial Wash Levee, Reach 1, Harquahala Valley Watershed, Maricopa County, AZ, from United States Department of Agriculture Soil Conservation Service. March 9, 1984.



TYPICAL SECTION - DIKE & FLOODWAY

Gannett Fleming	TYPICAL EMBANKMENT SECTION CENTENNIAL LEVEE	FIGURE 1



Gannett Fleming

CREST AND FLOODWAY ELEVATIONS
CENTENNIAL LEVEE

FIGURE
2

ATTACHMENT A

Soil Laboratory Test Results

Table 1. Summary of laboratory testing of soil samples
Centennial Levee Draft Geotechnical Memorandum

Sample ID	Sample Location	Sample Depth (ft)	Sample Date	Sample Method	Depth to older alluvium (ft)	USCS	Geologic unit	LL	PI	Clay minerology	Soluble salts (%)	Specific gravity	γ_d (pcf; lab)	γ_d (pcf; field)	% fines	Permeability (90% SP; ft/day)	Compaction γ_{dmax} SP(pcf)/opt m (%)	Swell (%) 90% Std Proctor density	Swell (%) 95% Std Proctor density	Shear strength (@90% Std Proctor density) ϕ (^{0})	Shear strength c (lb/ft^2)	Shear strength ϕ' (^{0})	Shear strength c' (lb/ft^2)	Compaction 90% Std Proctor density	Compaction 95% Std Proctor density	Pinhole test	$CaCO_3$ (% by weight)	Comments
1001.1	115+75	3.0-3.5	9/26/1983	Backhoe	3.5	SM	Younger alluvium	NP	NP		<0.5	2.64		106.0	25													
1001.2	115+75	6.5-7.0	9/26/1983	Backhoe		ML	Younger alluvium	29	6		<0.5	2.67	81.9	81.9	60													
1002.1	122+00	3.5-4.0	9/26/1983	Backhoe	3.0	CL	Younger alluvium	39	17		<0.5	2.68		91.3	69	0.03												
1003.1	128+00	3.0-3.5	9/26/1983	Backhoe	3.5	CL-ML	Younger alluvium	25	6	MI-PG-CLMT	<0.5	2.65	89.7	89.7	52	0.03	118.0/13.0	0.44	0.52	11.5	475	28.5	225				9	
1003.2	128+00	7.5-8.0	9/26/1983	Backhoe		CL	Younger alluvium	31	9		<0.5	2.67		100.6	64													
1004.1	134+16	5.5-6.0	9/26/1983	Backhoe	2.0	CL-ML	Younger alluvium	27	7		<0.5	2.68		94.1	68												2	Representative of borrow material
1004.2	134+16	10.0-10.5	9/26/1983	Backhoe		CL	Younger alluvium	31	13		<0.5	2.67		93.4	67													
1005.1	141+00	5.5-6.5	9/26/1983	Backhoe	2.5	CL	Younger alluvium	29	11	MI-PG-MT	<0.5	2.67		103.6	73	0.03	112.5/14.5	0.84	1.32	13.5	400	28.5	250					
1005.2	141+00	10.0-10.5	9/26/1983	Backhoe		CL	Younger alluvium	34	16		<0.5	2.68		98.1	65													
1006.1	147+62	2.0-4.0	9/26/1983	Backhoe	4.5	ML	Younger alluvium	21	2	MI-PG-MT	<0.5	2.67		92.5	59													
1006.2	147+62	9.5-10.0	9/26/1983	Backhoe		SC	Younger alluvium	41	20		<0.5	2.63		103.3	15													
1007.1	153+00	5.5-6.0	9/26/1983	Backhoe	5.5	CL	Younger alluvium	30	12		<0.5	2.66		92.2	71													Representative of borrow material
1007.2	153+00	10.0-10.5	9/26/1983	Backhoe		CL	Younger alluvium	36	15		<0.5	2.65	89.6	89.6	66													
1008.1	159+00	5.5-6.0	9/26/1983	Backhoe	4.0	CL	Younger alluvium	31	13		<0.5	2.66	89.3	89.3	63													
1008.2	159+00	9.5-10.0	9/26/1983	Backhoe		CL-ML	Younger alluvium	26	4		<0.5	2.66		92.9	55													
1009.1	165+00	2.0-5.0	9/26/1983	Backhoe	9.0	ML	Younger alluvium	23	2	MT-MI-PG	<0.5	2.68	90.6	90.6	77	0.02	114.0/13.5											
1010.1	171+00	5.0-5.5	9/26/1983	Backhoe	3.0	CL	Younger alluvium	31	13		<0.5	2.68	90.5	90.5	68	0.01	113.5/14.5	0.54	0.44									
1011.1	177+00	5.5-6.0	9/26/1983	Backhoe	6.0	ML	Younger alluvium	23	3		<0.5	2.67	90.9	90.9	72													
1011.2	177+00	11.0-11.5	9/26/1983	Backhoe		CL-ML	Younger alluvium	25	5		<0.5	2.66		102.6	67													
1011.3	177+00	3.0-5.0	9/26/1983	Backhoe		ML	Younger alluvium	NP	NP		<0.5	2.67			66	0.04	117.0/12.5			16.5	675	33	100					
1012.1	183+00	5.5-6.0	9/26/1983	Backhoe	7.0	ML	Younger alluvium	NP	NP	PG-MI-CLMT	<0.5	2.68	89.7	89.7	67													
1012.2	183+00	10.0-10.5	9/26/1983	Backhoe		ML	Older alluvium	27	4		<0.5	2.68		98.4	84													
1013.1	189+00	2.0-2.5	9/26/1983	Backhoe	7.5	ML	Younger alluvium	NP	NP	MT-MI	<0.5	2.69	93.1	93.1	54	0.14	114.0/12.0			13	675	32	100					
1013.2	189+00	3.5-5.0	9/26/1983	Backhoe		SM	Younger alluvium	NP	NP		<0.5	2.68			40													
1014.1	195+00	2.0-2.5	9/26/1983	Backhoe	7.5	SM	Younger alluvium	NP	NP		<0.5	2.69		102.1	25													
1014.2	195+00	8.0-8.5	9/26/1983	Backhoe		CL-ML	Older alluvium	23	6		<0.5	2.68		100.9	51													
1016.1	207+00	11.0-11.5	9/26/1983	Backhoe	3.0	CL	Older alluvium	23	8		<0.5	2.69		106.8	66													
1019.1	225+00	3.5-4.0	9/26/1983	Backhoe	7.0	SW-SM	Younger alluvium	NP	NP		<0.5	2.67		99.0	7													
1020.1	231+00	2.0-5.5	9/26/1983	Backhoe	10.0	SM	Younger alluvium	NP	NP		<0.5	2.73	94.7	94.7	49	0.12	114.5/12.0											
1021.1	237+00	5.5-6.0	9/26/1983	Backhoe	11.0	ML	Younger alluvium	19	1		<0.5	2.70		97.0	52												2	
1021.2	237+00	11.0-11.5	9/26/1983	Backhoe		SM	Older alluvium	NP	NP		<0.5	2.70		113.2	24									0.44	0.52	ND	8	Representative of borrow material
1021.3	237+00	2.5-4.5	9/26/1983	Backhoe		ML	Older alluvium	22	2		<0.5	2.71			65													
1022.1	243+00	6.5-7.0	9/26/1983	Backhoe	4.0	ML	Older alluvium	21	2		<0.5	2.72		94.0	60													
1023.1	249+00	6.0-6.5	9/26/1983	Backhoe	4.0	ML	Older alluvium	23	2		<0.5	2.69			62													
1023.2	249+00	1.0-4.0	9/26/1983	Backhoe		CL-ML	Younger alluvium	26	6		<0.5	2.67		99.9	73													
1024.1	255+00	4.5-5.0	9/26/1983	Backhoe	11.0	ML	Younger alluvium	20	1		<0.5	2.70		96.2	55													
1024.2	255+00	10.0-10.5	9/26/1983	Backhoe		ML	Younger alluvium	NP	NP		<0.5	2.70		93.0	70													
1025.1	261+00	5.5-6.0	9/26/1983	Backhoe	9.0	SM	Younger alluvium	NP	NP		<0.5	2.70		103.1	21													
1026.1	267+00	5.5-6.0	9/26/1983	Backhoe	8.0	SM	Younger alluvium	17	1		<0.5	2.71		99.8	40													
1026.2	267+00	3.0-5.0	9/26/1983	Backhoe		SM	Younger alluvium	NP	NP		<0.5	2.68			31													
1027.1	273+00	6.0-6.5	9/26/1983	Backhoe	9.0	ML	Younger alluvium	20	2		<0.5	2.69		95.7	62													
1027.2	273+00	10.0-10.5	9/26/1983	Backhoe		CL	Older alluvium	28	9		<0.5	2.68		100.0	71									0.84	1.32	ND	9	Representative of borrow material
1029.1	285+00	5.0-5.5	9/26/1983	Backhoe	10.0	ML	Younger alluvium	NP	NP		<0.5	2.68		97.4	51									0.16			2	Representative of borrow material
1029.2	285+00	10.0-10.5	9/26/1983	Backhoe		CL	Older alluvium	32	12	MI-MT-CL	<0.5	2.65		99.3	58													
1031.1	297+00	5.5-6.0	9/26/1983	Backhoe	15.0	ML	Younger alluvium	NP	NP		<0.5	2.70	87.8	87.8	80													
1031.2	297+00	9.5-10.0	9/26/1983	Backhoe		CL-ML	Younger alluvium	24	6		<0.5	2.70	86.8	86.8	58													
1032.1	303+00	4.0-4.5	9/26/1983	Backhoe	9.0	SM	Younger alluvium	NP	NP		<0.5	2.71			41													
1032.2	303+00	9.0-9.5	9/26/1983	Backhoe		CL-ML	Older alluvium	25	7		<0.5	2.70			83												5	Representative of borrow material
1033.1	309+00	4.0-4.5	9/26/1983	Backhoe	4.0	ML	Younger alluvium	19	2		<0.5	2.70		103.6	58													
1033.2	309+00	9.5-10.0	9/26/1983	Backhoe		SM	Older alluvium	21	2		<0.5	2.69		103.9	41													
1034.1	315+00	5.0-5.5	9/26/1983	Backhoe	5.0	CL-ML	Older alluvium	21	4		<0.5	2.69			59													
1035.1	321+00	3.5-4.0	9/26/1983	Backhoe	13.5	SM	Younger alluvium	17	1	MT-MI-PG	<0.5	2.72		112.4	42	1.2	121.5/9.5											
1036.1	327+00	5.5-6.0	9/26/1983	Backhoe	8.5	ML	Younger alluvium	NP	NP		<0.5	2.71			62													
1037.1	333+00	4.5-5.0	9/26/1983	Backhoe	4.5	CL	Older alluvium	23	8		<0.5	2.71		110.5	67													
1037.2	333+00	9.5-10.0	9/26/1983	Backhoe		ML	Older alluvium	NP	NP		<0.5	2.70		106.5	62													
1037.3	333+00	2.0-4.5	9/26/1983	Backhoe		CL-ML	Younger alluvium	22	4		<0.5	2.68			74													
1039.1	346+00	4.5-5.0	9/26/1983	Backhoe	4.5	ML	Older alluvium	18	1		<0.5	2.72		106.8	56													
1039.2	346+00	9.5-10.0	9/26/1983	Backhoe		ML	Older alluvium	22	2		<0.5	2.71		94.6	69	0.75	122.5/9.0			33	575	37	0				3	Representative of borrow material
1039.3	346+00	2.5-4.5	9/26/1983	Backhoe		SM	Younger alluvium	NP	NP		<0.5	2.71			23													
1040.1	352+00	6.0-6.5	9/26/1983	Backhoe	4.5	ML	Older alluvium	NP	NP		<0.5	2.72		106.2	53									0.54	0.44			Representative of borrow material

From USDS-SCS Lab Report (12/21/83)

ON-CALL PHASE I ASSESSMENT

CENTENNIAL LEVEE

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 Centennial Levee features are reviewed and evaluated to determine if deficiencies exist at the levee and associated project features. The examination was conducted by walking the length of the levee and visually examining the crest, upstream and downstream slopes, upstream and downstream toes, vehicle cross-over ramps, 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 August 11, 2005 by the following technical examination team:

Technical Examination Team

Tom Renckly, P.E.	Structures Branch Manager, Flood Control District of Maricopa County
Dan Lawrence, P.E., R.L.S.	Dam Safety Engineer, Flood Control District of Maricopa County
Bill Leal	Dam Safety Technician, Flood Control District of Maricopa County
Mike Luecker, P.E.	State Hydraulic Engineer, Natural Resources Conservation Service
Robert Eichinger, P.E., CFM	Project Manager, Kimley-Horn and Associates, Inc.
Lu Gan, E.I.T.	Civil Analyst, Kimley-Horn and Associates, Inc.
Ken Euge, P.G.	Principal Geologist, Geological Consultants Inc.
Jason Williams, R.G.	Geologist, Geological Consultants Inc.
Dean Durkee, Ph.D., P.E	Principal Geotechnical Engineer, Gannett-Fleming, Inc.
Frances, Ackerman, E.I.T., R.G.	Geotechnical Engineer, Gannett-Fleming, Inc.

Operational Summary

Inspection Frequency: Quarterly inspections are completed by District Operation and Maintenance Staff. Centennial Levee is also inspected after significant rainfall or flow events when significant rainfall-runoff events are reported by the Districts' Flood

Warning System. A total of 29 annual inspections, maintenance inspections, and special inspection reports were recorded from March 1988 to January 2005.

Maximum Water Surface Elevations: Since construction of the streamflow gage in 2003 at the downstream end of the levee, the maximum flows measured in the floodway were 126 cfs in August of 2003 and 13 cfs in July 2003. Measurements on the stage gage were 2.2 and 1.0 feet for these storms, respectively. Any flow in the floodway with a stage less than 0.65 feet is not measured by the gage. High water marks from an October 2000 rainfall event indicate that the 70% of the levee had water near the toe. No estimate of the flow rate was given. If flow were near the upstream toe of the Levee, the top width of the flow in the floodway during this event could have been as wide as 300 feet with a depth as much as 2-feet.

Distress Observations Corrected or Operation and Maintenance Conducted Since Last Inspection (2005): One new crack at Reach 2 Station 30+32 was documented in January 2005. At Reach 1 Station 19+96, a new three inch diameter hole was found in the upstream shoulder. A new possible transverse crack was found at Stations 1+70 and 4+32 on the upstream mid-slope and upstream shoulder, respectively.

Past Distress Observations Not Yet Corrected:

- Fill erosion rills on upstream and downstream slopes with compacted fill, if rills are greater than 12-inches deep;
- Fill erosion holes on crest with compacted fill, if erosion holes are greater than 12-inches deep;
- Initiate gravel mulch recommendations resulting from Phase I Structures Assessment (see Individual Structure Assessment Reports for Harquahala FRS and/or Saddleback FRS);
- Coordinate with Harquahala Valley Irrigation District (HVID) to ensure maintenance of the Westside Canal does not adversely impact the levee.

District Operation and Maintenance Responsibilities: An operation and maintenance agreement was made on August 28, 1984 between the NRCS (SCS) and the Flood Control District. The Flood Control District is the local sponsor of the federally funded project. The District maintains operational control of the Centennial Levee and is responsible for the structural and functional integrity of the levee and floodway, erosion control of the embankments, and landscaping.

Field Examination Results Summary

Regular maintenance has been performed on the Levee and Floodway since 1988 by the District according the record inspection reports.

Embankment Crest: Suspected longitudinal cracks/transverse cracks, depressions, and erosion holes were observed on the crest of the dam (see inspection report for specific locations). There were also signs of rodent activity, which could be the initiator of some

of the crest holes. Gravel mulch was proposed in the District maintenance report of 2004.

Upstream Slope: Small animal burrows were scattered on the slope face. There was no evidence of seepage however there are apparent signs of sloughing. There are erosion gullies located on many parts of the upstream slope that were first identified in 1988. Recommendations for gravel mulching were provided in a previous inspection report. Suspected longitudinal cracks/transverse cracks, depressions, and erosion holes were observed on the slope face (see inspection report for specific locations). The vegetation maintenance program of the District should continue along the levee. Gravel mulch was proposed by the maintenance inspection of 2004.

Downstream Slope: Similar to the upstream slope, small animal burrows were scattered on the slope face. There was no evidence of seepage however there are apparent signs of sloughing. There is erosion gullies located on many parts of the upstream slope. Recommendations for gravel mulching of the upstream slope were provided in a previous inspection report. Suspected longitudinal cracks/transverse cracks, depressions, and holes were observed on the slope face (see inspection report for specific locations). There is an earth-line vee-shaped channel at the downstream toe of the levee that is designed to carry flow away from the toe of the levee and canal maintenance road. Water does pond in the ditch. HVID maintenance activities of the channel appear to be cutting into the downstream toe of the levee. The grading of the channel for maintenance activity appears to be developing windrows across the levee access ramps. This causes accessibility problems for District vehicles. Gravel mulch was proposed by the maintenance inspection of 2004.

Instrumentation: Instrumentation on Centennial Levee includes a rain gage (5120) and a streamflow gage (5123). Both of the gages are part of the District's flood warning system. The rain gage was first installed in 1984 and the streamflow gage was installed in 2003 as a part of the District's Dam Safety Program. The gages are located on the Levee crest at the downstream end of the Levee.

Outlet Structure: The grade control structure at the outlet is in good condition. The soil cement plating is in good condition.

Floodway: The floodway is in good condition. The District regularly inspects the vegetation density in the floodway and maintains the channel according to the design intent.

Conclusions

The overall conclusion of the field examination is that the Centennial Levee, floodway, and appurtenant structures are in satisfactory operational condition. Monitoring of transverse and longitudinal cracking, its causes and effects should continue.

Recommendations

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

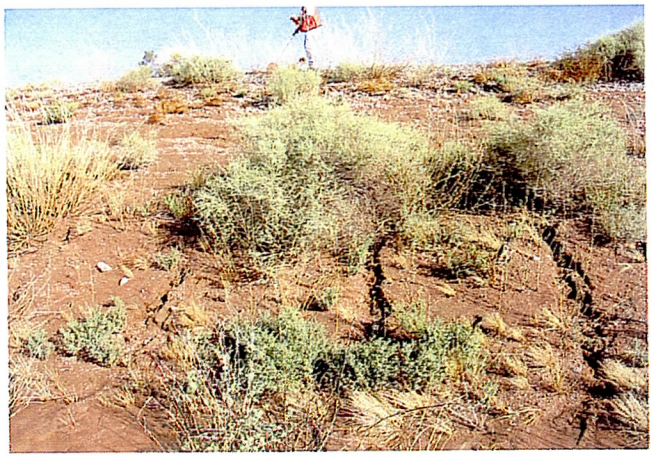
a. Coordinate with HVID to facilitate required level of maintenance activities on the levee and canal system, particularly for the V-shaped channel;
b. Coordinate with HVID to ensure that the sediment removal from the V-shaped channel between the downstream toe and the canal does not adversely impact the downstream toe of the levee;
c. Continue active vegetation management program ;
d. Continue active rodent management program;
e. Survey the crest of the Levee to begin a subsidence monitoring program for the levee;
f. Develop mitigation strategies to protect the structure against transverse and longitudinal cracking;
g. 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 the levee where there has been a noted propensity of cracks;
h. Determine if a gravel mulch or vegetative seeding would be beneficial for the crest and embankment slopes;
i. Ensure the levee is on a rotation to survey the crest regularly with the Flood Retarding Structures;
j. Evaluate levee to determine if the levee meets current FEMA criteria for certification.

Next Annual Inspection

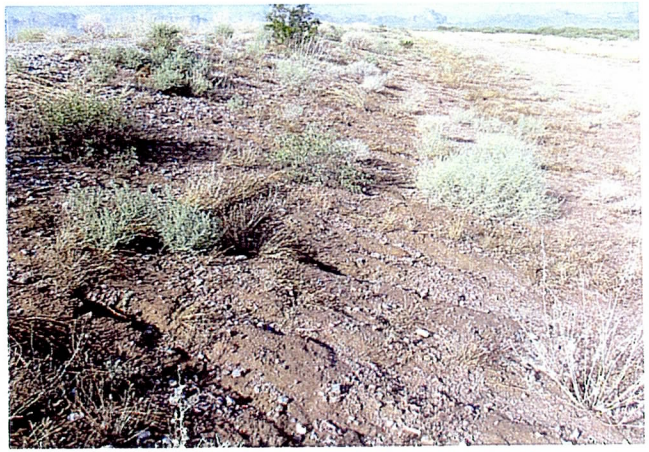
The next annual inspection is scheduled for January 2006.

1 Station 8+00

2 Station 55+00



3 Station 57+93



4 Station 62+00

5 Station 62+37

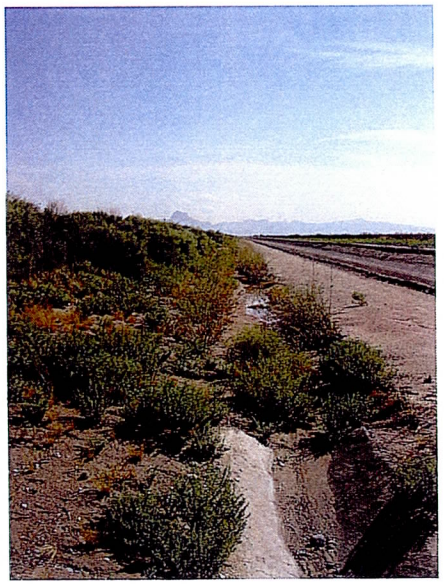
6 Station 100+18

7 Station132+53

8 Station 215+56

9 Station 216+16

10 Station 208+44

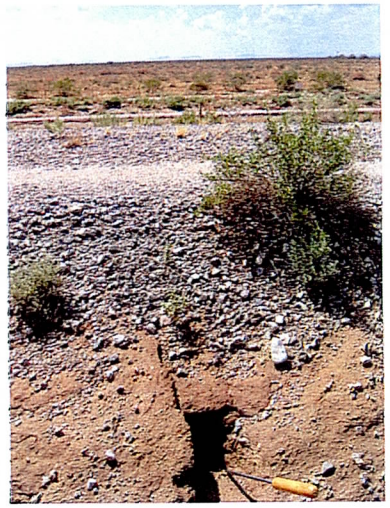


11 Station 186+59

12 Station 153+82

Beginning of Soil Cement
Spillway

13 Station 162+89

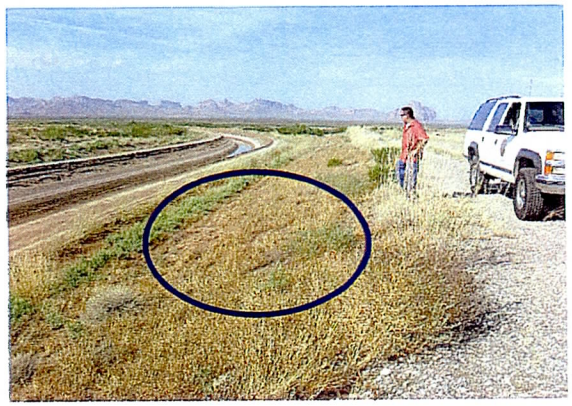


14 Station 125+15

15 Station 62+50

Erosion or sloughing on upstream slope

Station 215+38 Vehicle ruts on Downstream slope



Station 192+98 Some sloughing of the downstream slope

FAILURE MODE AND EFFECTS ANALYSIS

For

CENTENNIAL LEVEE

MARICOPA COUNTY, ARIZONA

September 7, 2005

1.0 INTRODUCTION

General Description

Centennial Levee is located in Sections 5, 6, and 7 of Township 2 North, Range 9 West and Sections 11, 12, 14, and 15 of Township 3 North Range 10 West. The structure is approximately 75 miles west of downtown Phoenix, Arizona. Centennial Levee consists of the levee embankment and a floodway that diverts up to the 100-year, 24-hour runoff event away from downstream irrigated farmland in the Harquahala Valley to Centennial Wash. The levee is a structural component of the Harquahala Valley Watershed Work Plan prepared by the Natural Resources Conservation Service.

Centennial Levee is 22,400 feet long (4.2 miles). The earth dike that protects the siphon inlet for the Westside canal crossing of Centennial Wash is 4,118 feet long. Centennial Levee is a homogeneous earthfill structure. The maximum height of the embankment is 10.5 feet above the invert of the floodway. The centerline of the Centennial Levee floodway is 260 feet upstream from the levee centerline. The floodway has a trapezoidal cross-section and has a maximum top with of approximately 415 feet wide. Construction of Centennial Levee was completed in September 1985.

The levee has performed satisfactorily to date. Stream gage for the levee floodway is provided on the District's ALERT website and data is available from 2003 to present.

Embankment Data

Embankment type: Homogeneous earthfill
Embankment height: 10.5-feet
Embankment length: 22,400 ft (4.2 miles)
Embankment crest: width 12-feet; elevation varies
Upstream Slope: 4:1; Downstream Slope: 3:1
No outlet pipe penetrations
Freeboard: 2-feet by design
Cutoff Trench: Width 12-feet; Depth varies 3-5-feet

Floodway/Outfall Data
Cross Section: Trapezoidal shape
Floodway Top Width: Approximately 415-feet
Floodway Bottom Width: 80-feet
Side Slopes: 50:1
Floodway Invert Slope: 0.00024 feet/feet by design
Design Discharge: 7,540 cfs (100-year 24-hour storm)
Design Velocity: 1.2 feet/second to 3.5 feet/second depending on floodway section
Estimated Coefficient of Roughness: 0.035 (n-value)
Outfall: Trapezoidal shape invert with soil cement lining and rock sill at terminus

Hydrology Data (elevations in NGVD29 datum)
100-year 24-hour Runoff Volume: 2.19 inches
100-year, 24-hour Runoff Volume: 116.7 acre-feet
Uncontrolled Contributing Drainage Area: 21 square miles (Watershed controlled by Tiger Wash Detention Basin and Harquahala Flood Retarding Structure total 246 square miles; only a portion of which would be in the historic contributing watershed for the Levee)
Estimate Sediment Accumulation: 0.95 acre-feet/year (Arizona Water Commission - 1979)
100-year, 24-hour inflow = 0 cfs at upstream end progressing to 7,540 cfs at downstream end

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 levee.
- Discuss qualitatively the likelihood of the occurrence of failure modes.
- Determine if 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 releases into Centennial Wash).
- 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.

(Note: 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 could be evaluated in a future risk assessment for the levee.)

Team Members

Debora Miller, P.E., PhD, FMEA Facilitator, Gannett Fleming, Inc.
Dan Lawrence, P.E., R.L.S., Flood Control District of Maricopa County, Dam Safety Engineer
David Jensen P.E., Session Recorder, Kimley-Horn and Associates, Inc., Project Engineer
Dean Durkee, P.E., PhD, Geotechnical Engineer, Gannett Fleming, Inc.
Frances Ackerman, R.G., E.I.T., Gannett Fleming, Geotechnical Engineer
Ken Euge, R.G., Geological Consultants, Geology

2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

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

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

1) No Crack Protection (No Central Filter). The embankment is a homogenous embankment with no central filter. Central filters are provided on most District/NRCS structures (flood retarding structures).
2) Easily Identifiable Geologic Layers. The Holocene and Pleistocene boundary is relatively shallow as it ranges from 2-feet to 15-feet in depth.
3) Known Transverse Cracking And Longitudinal Cracking. There is a history of transverse and longitudinal cracking on the levee that has been documented in past inspections and investigations.
4) Earth Fissures Have Been Documented In Area. There are earth fissures within 2-miles of the structure. One fissure is located to the south and west across Centennial Wash.
5) Subsidence. Centennial Levee is located in a known subsidence region.
6) Erosive Soils In Floodway. The Holocene soils that form the bottom and side slopes of the floodway are erosive but floodway design velocities are low. The design velocity in the floodway is approximately 1.2 feet per second.
7) Low Head On Foundation. There crest of the levee is broad and the slopes are flat compared to depth of flow against levee. The crest width (12 feet) is greater than the hydraulic head. The maximum depth of design flow against the embankment is 2-feet

below the height of the levee for the 100-year runoff event. The hydraulic head is approximately 8.5 feet.

8) Cutoff Does Not Extend Into Pleistocene Layer. The embankment cutoff trench does not extend entirely through shallow Holocene soils into the deeper caliche (Pleistocene) layer. The cutoff trench was excavated to the depth of the Westside Canal invert to disrupt the natural flow of sub-surface channels and to protect the lining of the Westside Canal from external hydraulic forces.
9) No Erosion Protection On Upstream Slope. During initial construction, the embankment slopes were treated to reduce rilling and for aesthetics. The treatment included scarifying and seeding with indigenous plants.
10) No Slope Stability Analysis. A slope stability analysis could not be located or identified as part of the data collection.

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

11) Minimum Available Freeboard Is Approximately 2-feet. The design documents indicate that floodway was designed to have 2-feet of freeboard above the 100-year water-surface elevation to the top of the levee crest. This is based upon the NRCS Dike Standard.
12) Uncertainty Regarding Floodway Profile And Channel Capacity. There are no recent surveys for the floodway profile. Recent studies by the District completed for the installation of the streamflow gage did not analyze the full capacity of the floodway. A rating curve was only developed up to 1,000 cfs.
13) Limited Duration And Infrequency Of Flooding. The limited duration of flow (design event 24-hours) in channel, low hydraulic head, and the infrequency of storm events are important mitigating factors for most failure modes.
14) Levee Is Not Designed To Impound Water. The design intent is to divert water to the Centennial Wash upstream from the historic confluence.
15) Upstream Structures Mitigate Sediment Transport And Inflows. The contributing area for the Levee is 21 square miles, which is only a portion of the historic drainage area. Tiger Wash Detention Basin and Harquahala Flood Retarding Structure control a large portion of the historic watershed and thus control inflows and the amount of sediment that is transported to Centennial Levee.
16) Attenuation Of Peak Inflow By Interstate-10. Within the contributing watershed to the levee, peak flows are attenuated by I-10 freeway embankment.
17) Unavailability Of Hydrologic Design Data. The hydrologic studies completed for the watershed could not be located in the data collection phase. Only select data were found in other project related documents. (see Item 26)

Consequence Evaluation

18) Low Consequences Of Failure Downstream (Population At Risk). There are no identifiable inhabited structures downstream from the levee. Failure of the levee would likely not result in the loss of life.

19) No Category I Failure Modes. The FMEA team developed only two Category II failure modes. These include (1) overtopping of the levee during a storm exceeding the 100-year storm event and (2) erosion or breaching of the levee during the runoff event that develops from erosion through an existing transverse crack in the embankment.

Action Items – Risk Reduction Measures or Investigations

20) Re-Station The Crest Of Levee. The station markers on the crest of the levee should match the as-built plans. (see Item 32)
21) Update Crest Survey. Survey Centennial Levee crest on the same frequency as significant hazard dams.
22) Survey Profile And Cross-Section Of Floodway. This will allow an estimate of the capacity of the floodway.
23) Update Hydraulics To Confirm Flood Channel Capacity. Check sensitivity to Manning's n-values to validate channel hydraulic capacity. No subsequent analysis was found since the original design documents and analysis.
24) Check For Possible Correlation Between Location Of Transverse Cracks And Washes. Note the location of washes into the floodway and the extension of the washes downstream beyond the Westside Canal. Note the locations of observed or suspected transverse cracks on a set of as-built plans. Attempt to derive a correlation of wash location and crack location in the embankment.
25) Update Slope Stability Analysis. Potential weakening of soils on saturation could alter the results of a slope stability analysis. Conduct an updated or new slope stability analysis.
26) Need To Locate Flood Hydrographs For Design Event. The design flood hydrographs were not located in the project literature prior to the FMEA (but have subsequently been found and reviewed).
27) Revetments Near The Wash Crossings. Revetments could be beneficial on the upstream slope of the levee embankment where large washes could impinge and cause concentrated erosion. Erosion control measures or stabilized inlets from the washes into the floodway may be investigated and evaluated.
28) Operation And Maintenance Update. Verify floodway operation and maintenance standard procedures (sediment buildup, excessive vegetation, etc).
29) Future Crack Investigations. Follow up on Phase I Structure Assessment recommendations to conduct additional crack investigations. Transverse and longitudinal cracks have been identified at the structure.
30) Earth Fissuring Studies. Consider additional studies to investigate earth fissuring in the central Harquahala Valley. Earth fissures have been noted within 2-miles of the levee. Consider baseline elevation survey tied to reliable benchmark. Incorporate studies with studies for Harquahala FRS and Saddleback FRS.
31) Future FMEA Considerations. Consider updated FMEA if development is proposed downstream from the levee. Downstream development would cause increased consequences from potential levee failure.

General Findings

32) Discrepancies In Levee Stationing. There is a discrepancy between the stationing indicated on the as-built plans and the station markers on the levee crest. This makes inspections and locating features (transverse cracks) difficult.
33) Triangular Channel at Downstream Slope Impounds Water At Downstream Toe Of Levee. Water is impounded by the triangular-channel on the downstream toe of the levee. The channel is designed to intercept runoff and sediment from the downstream side of the levee embankment and direct it to the Westside Canal. The channel is a design feature of the project. The channel discharges into a series of inlet/cross culverts under the canal maintenance road and then into the canal.

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 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 was evident, fundamental flaw or weakness is identified and conditions and events leading to failure seemed reasonable and credible) are highlighted.*

Category II - *Failure Modes Considered but not Highlighted – These are judged to be of lesser significance and likelihood. Each failure mode is described and included with reasons for and against the potential occurrence of the 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 on significance thus a dam safety investigative action or analyses can be recommended. Because these actions are required before resolution these may also be highlighted.*

Category IV - *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 for the failure mode, or the failure mode is clearly non-credible or reasonable to postulate.*

For each of the potential failure modes identified, a failure mode description is briefly described and the factors that make the failure mode more likely (adverse factors) or less likely (positive factors) to occur are listed following the failure mode description. In addition, any identified potential actions for risk reduction for each potential failure are then provided.

CATEGORY I – HIGHLIGHTED FAILURE MODES

The FMEA team judged that there are no Category I Failure Modes for Centennial Levee as this time. A future update to the FMEA may or may not result in having Category I failure modes.

CATEGORY II – FAILURE MODES CONSIDERED BUT NOT HIGHLIGHTED

Failure Mode H1 - Overtopping Of Levee For Runoff Events Exceeding 100-Year Event And Exceeding Freeboard Leading To Erosion And Breach Of The Levee (Category II).

Failure Mode Description: Flood flows contributing to the levee/floodway are greater than the capacity of the floodway and levee. The depth of flows against the levee is sufficient to overtop the levee initially at low spots and cause concentrated flow erosion. Erosion continues on the levee crest until a breach occurs releasing a portion or all of floodway flows from behind the levee. Potential causative mechanisms of overtopping could be due to failure of an upstream dam (Tiger Wash Detention Dam or Harquahala FRS); or a rainfall-runoff event greater than the design event (say 200-year storm on the watershed).

Adverse Factors:

(1) Possibility of concentrated flows over crest due to uneven crest profile or transverse cracks. The embankment has an stepped crest profile by design
(2) Soils are erosive
(3) Unusual clay mineralogy (attapulgite) may be more prone to erosion on saturation
(4) Extended duration or back-to-back runoff events could contribute to exceeding design event
(5) Slopes are unprotected. No rock mulch on slopes
(6) Flooding event for a potential overtopping occurrence could be due to a greater frequency (say 200-year event) event when compared to dam (which may be on the order of ½ PMF to PMF event).
(7) Upstream dams (Tiger Wash Detention Dam and/or Harquahala FRS) could potentially fail and cause levee overtopping.

Positive Factors:

(1) Short duration of runoff (design storm is 100-year 24-hour) and flows along length of levee. Potentially need longer duration flows
(2) Gravel plating on crest of levee

(3) No impoundment as provided in flood retarding structures (no sustained head on levee embankment)
(4) Westside canal will collect and convey some portion of overtopping flows. In particular if levee holds but only overtops with minor depths of flow canal may take flows without downstream consequences
(5) Floodway will have capacity for flows even if levee breaches
(6) Very low downstream consequences at this time
(7) Flows passing through levee will quickly return to Centennial Wash
(8) No history of major problems with levee
(9) Upstream contributing drainage area protected by another NRCS Watershed Workplan structure (Harquahala FRS) and Bureau of Reclamation Tiger Wash Detention Basin
(10) Central Arizona Project canal and Interstate 10 embankment provide upstream attenuation

Potential Risk Reduction Measure

(1) Consider Dynamic Routing model (HEC-RAS) to evaluate water surface profiles

Failure Mode S1 – Erosion And Breach Of Levee During Runoff Events Due To Flow Through Existing Transverse Cracks In Levee Embankment (Category II).

Failure Mode Description: Flood runoff event of sufficient duration and depth of flow against levee occurs. Water flows through a transverse crack and the resulting highly localized flow velocities erode the soils materials in the crack. As the crack widens, more flow passes through the crack until embankment breaches releasing a significant portion or all the floodway flows through breach to the downstream inundation area.

Adverse Factors:
(1) Evidence of transverse cracks and confirmation of transverse cracks in southern half of levee
(2) Test trenching at specific locations indicated that the transverse cracks extend through the embankment
(3) Cracks are wide ranging from ½ inch to ¾ inch in test trenches
(4) Soils are erosive
(5) No central filter
(6) Visible evidence of longitudinal cracks on crest (possible interconnecting transverse cracks)
(7) Potentially collapsing soils in upper foundation (may be thicker on southern end) were not completely removed under levee or under cutoff trench
(8) Long structure traverses many drainages leading to potential for differential settlement and cracking
(9) Saturation at downstream toe due to channel (Vee-shaped channel). Ponded water in channel may saturate collapsible foundation soils
(10) Documentation of through cracks in previous District special investigations.

Positive Factors:

(1) No impoundment
(2) Very low downstream hazard potential at this time (few structures)
(3) Short duration of flood flows against levee
(4) Soils testing indicate that the embankment soils are non-dispersive
(5) Low hydraulic head against levee
(6) Probably need successive events to develop a failure
(7) Very low downstream hazard
(8) Broad section relative to hydraulic head (flat slopes and 12 foot crest width)

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

Failure Mode H2 - Scour And Erosion Of Levee Due To Flows In Floodway Starting At Upstream Toe Leading To Breach (Category III).

Failure Mode Description: Flood flows in the upstream floodway flow against upstream levee toe. This results in scouring or lateral erosion of upstream toe eventually causing caving of embankment materials. Continued lateral erosion and caving results in breach of levee. Floodway designed for low flow velocity for design storm event. Other flows and concentrated flows not evaluated.

Adverse Factors:

(1) Could occur under normal operations with less than the 100-year event
(2) Concentrated flows from Tiger Wash and other major upstream drainages
(3) Erosive and unprotected embankment slopes
(4) Potential weakening of soils materials on saturation due to unusual clay mineralogy
(5) Successive storms could lead to longer duration storms in floodway and downstream toe channel
(6) Cracked sections may be more prone to lateral erosion

Positive Factors:

(1) Low floodway velocities by design
(2) Upstream floodway designed to keep low flows away from levee
(3) Flat (3:1) slope downstream and (4:1) slope upstream
(4) The cross section is thick perpendicular to the flow direction
(5) Higher velocity section is protected by hardened channel (soil cement) and rock sill end
(6) Flood event is relatively short (less than three days total flow period for 100-year event) and peak discharge duration is short (less than a few hours)
(7) Floodway channel profile will keep flows off levee until exceed some elevation.
(8) Centerline of upstream floodway is 260 feet from levee
(9) Very low downstream hazard at this time (few structures)

Failure Mode H3 – Embankment Erosion Due To Dynamics Of Flows Coming In From Steeper (Higher Velocity) Upstream Drainages Impinging On Levee And Floodway Channel (Category III).

Failure Mode Description: Concentrated high velocity flows emerging from upgradient steep drainage channels direct flows straight into the upstream slope of the levee. The force of water locally erodes the upstream slope of the levee causing caving. Continued erosion and caving results in breach of the levee. No evaluation could be located in the project literature to address this condition.

Adverse Factors:

(1) Possibility of concentrated flows over crest due to uneven crest profile or transverse cracks. The embankment has a stepped crest profile by design
(2) Soils are erosive
(3) Unusual clay mineralogy (attapulgite) may be more prone to erosion on saturation
(4) Extended duration or back-to-back runoff events could contribute to exceeding design event
(5) No rock mulch on slopes

Positive Factors:

(1) Short duration of runoff and flows along length of levee
(2) Gravel plating on crest of levee
(3) No impoundment as provided in flood retarding structures (no sustained head on levee embankment)
(4) Westside canal will collect and convey some portion of overtopping flows. In particular if levee holds but only overtops with minor depths of flow canal may take flows without downstream consequences
(5) Floodway will have capacity for flows even if levee breaches
(6) Very low downstream hazard at this time (few structures)
(7) Flows passing through levee will quickly return to Centennial Wash
(8) No history of major problems with levee
(9) Upstream contributing drainage area protected by other NRCS Watershed Workplan structure (Harquahala FRS) and Bureau of Reclamation Tiger Wash Detention Basin
(10) Central Arizona Project canal and Interstate 10 embankment provides upstream attenuation
(11) Very low downstream hazard potential at this time (few structures)

Potential Risk Reduction Measure

(1) Consider Dynamic Routing Model to evaluate water surface profiles

Failure Mode S2 - Erosion And Breach Of Levee Due To Flow Through Transverse Cracks Or Earth Fissures Extending Under Structure Foundation (Category III).

Failure Mode Description: An existing transverse crack or earth fissure extends through the levee foundation. During periods of flow in the floodway concentrated seepage flows through the crack, leading to erosion and widening of the crack. Continued widening of the foundation crack occurs during long sustained flood events, or progressively over multiple events over a longer period of time. Levee embankment cannot support itself over continued widening of the crack and caves/sloughs into eroded foundation. Levee material is washed away downstream resulting in breach of the levee.

Adverse Factors:

(1) Earth fissures have been documented in area. Closest fissure is within 1.9 miles of levee
(2) Known subsidence area due to overdraft of groundwater
(3) Highly erosive soils in embankment and foundation
(4) Power plant groundwater pumping continues and power plant operations will continue
(5) Holocene soils (down to 15 feet) may mask earth fissure

Positive Factors:

(1) No documented earth fissures in near vicinity of levee
(2) Would require a very long duration flood event (greater than 3 days for 100-year event)
(3) Cutoff may provide some protection
(4) Fissure should be well-defined (identifiable) and potentially less erodible due to high degree of cementing in deeper foundation
(5) Very low downstream hazard potential at this time (few structures)
(6) Relatively broad base section compared to head on structure (flat slopes)

CATEGORY IV – FAILURE MODES WHICH ARE RULED OUT

Failure Mode H4 - Overtopping Due To Sediment Build Up From Major Drainage (E.G. Tiger Wash) Raising The Bottom Of Floodway Channel In Isolated Locations (Category IV). (Failure mode description not formulated by FMEA team – ruled out)

Adverse Factors:

(1) Sediment prediction in design is evenly distributed along channel profile and not concentrated at drainages
(2) Soils in drainage area are highly erosive
(3) Drainage features in basin appear to verify that soils are erosive

Positive Factors:

(1) Operation and Maintenance procedures would detect sediment buildup and changes in channel capacity
(2) No visible evidence of sediment buildup at the levee

(3) Very limited runoff area for sediment source (major upstream catchments)

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) Erosion of Control Sill of Floodway. Sill is documented in inspection reports to be in good condition. The failure of the control sill or soil cement section would not result in significant adverse consequences (discharge is still into Centennial Wash).

(2) Slope Stability. The slope stability is not much of a concern due to the low embankment height and flat slopes but should be verified (factor of safety) for potential loss of strength due to clay mineralogy.

(3) Earthquake Loading. Earthquake/failure mode not identified due to low regional seismicity and low embankment height and flat slopes.

(4) Piping/Erosion in Embankment Where No Cracks Exist. Piping/erosion in embankment where no crack exists is not feasible due to short duration of flood flows, low head, broad embankment, and presence of cutoff trench

(5) Utility Crossing. No failure mode was identified in association with the only known utility crossing because it is adjacent to the I-10 frontage at the far upstream northern end of the levee where breach would be of very minor consequence.

(6) Normal Operation of Centennial Levee Discharges Flows into Centennial Wash. There is very minor potential for adverse downstream consequences. The levee and floodway discharges into Centennial Wash, a broad meandering floodplain that would attenuate flood flows from Centennial levee/floodway.

4.0 LIKELIHOOD AND CONSEQUENCE CATEGORIES

The likelihood of occurrence of each identified failure modes has been assigned to one of four 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 levee. 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 for Centennial Levee
Medium	Intermediate likelihood of occurrence for Centennial Levee
Low	Lowest likelihood of occurrence for Centennial Levee

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 levee. 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 for the FMEA. This relative measure of consequence is contained in the categories defined in Table 2.

Table 2. Consequence Categories

Category	Description
High	Highest inundation effects for Centennial Levee
Medium	Intermediate inundation effects for Centennial Levee
Low	Lowest inundation effects for Centennial Levee

5.0 FAILURE MODE AND EFFECTS TABLE

Construction of the Failure Mode and Effects Table (Table 3) summarizes the failure modes identified and evaluated in the FMEA workshop by the workshop FMEA team. The columns contain the following elements from left to right:

- Failure Mode – identifies the primary failure mechanism
- Initiating Condition – condition(s) giving rise to initiation of the failure mode/sequence
- Effects – distinguishes levee breach 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 – any supplemental remarks if needed

Table 3. Summary of Failure Mode and Effects Analysis
Centennial Levee
Maricopa County, Arizona

FAILURE MODE	INITIATING CONDITION	EFFECT	LIKELIHOOD	CONSEQUENCES	INFORMATION NEEDS	EXISTING RISK REDUCTION FACTORS	POTENTIAL RISK REDUCTION FACTORS/MEASURES	COMMENTS
Failure Mode H1 - Overtopping Of Levee For Runoff Events Exceeding 100-Year Event And Exceeding Freeboard Leading To Erosion And Breach Of The Levee (Category II).	Levee flows greater than 100-year flood with longer duration flows.	Downstream inundation	Low	Medium	Dynamic routing	Levee is instrumented as part of District ALERT system Discharge duration of 100-year event is short (design event is 24 hours)	Widen floodway to carry more flow and away from levee Provide bank protection	
Failure Mode S1 – Erosion And Breach Of Levee During Runoff Events Due To Flow Through Existing Transverse Cracks In Levee Embankment (Category II).	High levee flows and longer duration flows. Existing cracks.	Downstream inundation	Low	Medium	Dynamic routing Map cracks Consider ground penetrating radar	Cracks known to exist Low height levee	Widen floodway to carry more flow and away from levee Provide bank protection Monitor embankment and repair cracks	
Failure Mode H2 - Scour And Erosion Of Levee Due To Flows In Floodway Starting At Upstream Toe Leading To Breach (Category III).	Lateral erosion of levee embankment due to flows against levee.	Downstream inundation	Not determined	Not determined		Short duration of flows (design event is 24 hours) Low height levee	Provide bank protection	
Failure Mode H3 – Embankment Erosion Due To Dynamics Of Flows Coming In From Steeper (High Velocity) Upstream Drainages Impinging On Levee And Floodway Channel (Category III).	High velocity flows from upstream drainages impact perpendicular to levee embankment and cause local erosion. Embankment collapses and breaches levee.	Downstream inundation	Not determined	Not determined	Wave run-up computations	Existing floodway Flow is parallel to levee	Provide stabilized inlets for incoming drainages into floodway Provide bank protection	
Failure Mode S2 - Erosion And Breach Of Levee Due To Flow Through Transverse Cracks Or Earth Fissures Extending Under Structure Foundation (Category III).	Transverse crack or earth fissure under and/or through foundation.	Downstream inundation	Not determined	Not determined		Cutoff trench Low flow velocities in floodway	Monitor for settlement of levee crest Continue monitoring for subsidence	

6.0 FAILURE MODE BINNING

While the FMEA table contains the likelihood and consequence attributes of risk, it does not portray risk as such. Binning extends the FMEA to the final step of separating failure modes into rank-ordered groupings according to their respective relative risks. It is convenient to bin failure modes into a two-dimensional array as shown in Table 4, where each failure mode falls into a discrete region of risk space according to its particular likelihood and consequence attributes.

Table 4. Failure Mode Binning for Centennial Levee

(Numbers refer to failure mode identification numbers in Table 3 and shaded region represents comparatively greater risk)

Consequences

Likelihood	High	Medium	Low
High			
Medium			
Low		H1 S1	

In the format of Table 4, risk increases to the upper left of the array and decreases to the lower right. Thus the shaded region of Table 4 contains any failure modes of generally greater risk. Note that Table 4 indicates that both failure modes H1 and S1 are directly within the unshaded region. H1 (levee overtopping for storm event greater that the 100-year storm leading to levee breach) and S2 (erosion and breach of levee through existing transverse cracks) are depicted in the bin array for low likelihood and medium consequence.

7.0 SUMMARY AND CONCLUSIONS

Centennial Levee was constructed pursuant to a relatively modern levee design. Construction appears to have been without any particular issues. The levee has performed normally and satisfactorily for 23 years. The structure is satisfactorily maintained and monitored.

However, it is prudent to recognize that there exist for all earth structures (levees and 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 levee is about to fail or even necessarily that there is a levee 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 levee should be aware.

No Category I potential failure modes were identified by the FMEA team. Two Category II potential failure modes were identified and discussed. Normal operations of the levee will discharge flood waters into Centennial Wash. The flows will enter Centennial Wash and then spread out over the Centennial Wash floodplain. There is very little structure development downstream of Centennial Levee at this time.

Structure Assessment Program, Phase I
Hoque & Associates, Inc. Work Assignment No. 3

Centennial Levee

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 the area around Centennial Levee. The search was limited to areas located within 1/8 mile upstream and downstream of the levee. Information related to utility locations was gathered from utility companies as-built drawings and data provided to HA by Kimley-Horn. The following table includes all utilities found within 1/8 mile of the levee. All utility locations are input to and identified on the AutoCAD base maps provided by Kimley-Horn.

Sl. No.	Utility Name	Description
1.	Sprint Communication Fiber Optic Line	Runs across north end of Centennial Levee and parallel to I-10. It is located within 200 feet of I-10 and immediately south of the existing fence bordering the I-10 right-of-way.

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 levee 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 Kimley-Horn and maps available through the County Assessor's website, total land within two miles upstream and two miles downstream of Centennial Levee is 18,342 acres. 19 percent (3,487 acres) of this land is owned by the State Trust, 17 percent (3,147 acres) is owned by US Bureau of Land Management (USBLM), and 64 percent (11,708 acres) is owned by private parties. Detailed information on land use within two miles of the levee is provided in Table 1 – Land Uses for Areas in Two miles Radius of Centennial Levee, in Appendix A.

Based on the information provided, there is no information available on existing or proposed infrastructures except the existing roadways including Salom Highway and Interstate 10 Freeway and their right-of-ways within two-mile radius of the levee.

ADJACENT AREA PROPERTIES

HA collected data related to adjacent area properties, and current and future population densities within the prescribed distances from the levee as required under our Scope of Work. HA compiled related information in spreadsheets that are contained in Appendix A. 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 the levee 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 257 properties were located within the prescribed area of Centennial Levee with no properties developed with building structures. The Current Properties information obtained is presented in Tables 2 – List of Current Properties within Two Miles of Centennial Levee, in Appendix B. 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 levee do not have information available such as land owned by State Trust. The properties without an assessor parcel number (APN) and/or other necessary information are not included in Table 2.

Critical Facilities

HA collected information on Critical Facilities located within 2 miles upstream and downstream of the levee 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, no critical facility currently exists within a two-mile radius of Centennial Levee.

Present and Projected Populations

HA collected information on current and projected populations in the areas adjacent to the levee (up to two miles upstream and downstream of the levee) 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 included population density maps that are contained in Appendix C – 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

Centennial Levee

APPENDIX A

Data Review Tables

Table 1 Land Uses for Areas in Two Miles Radius of Centennial Levee

Sl. No.	Section Number	Section Area sq.ft.	Covered area (%Section)	Covered area sq.ft.	BLM		Private		State Trust	
					%Covered area	Area sq.ft.	%Covered area	Area Sq.ft	%Covered area	Area Sq.ft
1	03 N10 W25	27878400	10	2787840	0	0	0	0	100	2787840
2	03 N09 W30	27878400	50	13939200	50	6969600	50	6969600	0	0
3	03 N09 W29	27878400	55	15333120	100	15333120	0	0	0	0
4	03 N09 W28	27878400	35	9757440	100	9757440	0	0	0	0
5	03 N10 W35	27878400	40	11151360	0	0	0	0	100	11151360
6	03 N10 W36	27878400	85	23696640	15	3554496	0	0	85	20142144
7	03 N9 W31	27878400	100	27878400	25	6969600	75	20908800	0	0
8	03 N9 W32	27878400	100	27878400	50	13939200	50	13939200	0	0
9	03 N9 W33	27878400	100	27878400	75	20908800	25	6969600	0	0
10	03 N9 W34	27878400	25	6969600	100	6969600	0	0	0	0
11	02 N10 W4	27878400	10	2787840	0	0	100	2787840	0	0
12	02 N10 W3	27878400	12	3345408	0	0	40	1338163	60	2007245
13	02 N10 W2	27878400	65	18120960	100	18120960	0	0	0	0
14	02 N10 W1	27878400	100	27878400	0	0	100	27878400	0	0
15	02 N9 W6	27878400	100	27878400	0	0	100	27878400	0	0
16	02 N9 W5	27878400	100	27878400	0	0	70	19514880	30	8363520
17	02 N9 W4	27878400	100	27878400	0	0	100	27878400	0	0
18	02 N9 W3	27878400	35	9757440	0	0	40	3902976	60	5854464
19	02 N10 W9	27878400	70	19514880	0	0	100	19514880	0	0
20	02 N10 W10	27878400	100	27878400	0	0	50	13939200	50	13939200
21	02 N10 W11	27878400	100	27878400	100	27878400	0	0	0	0
22	02 N10 W12	27878400	100	27878400	0	0	100	27878400	0	0
23	02 N9 W7	27878400	100	27878400	0	0	100	27878400	0	0
24	02 N9 W8	27878400	24	6690816	100	6690816	0	0	0	0
25	02 N09 W9	27878400	85	23696640	0	0	100	23696640	0	0
26	02 N09 W10	27878400	10	2787840	0	0	100	2787840	0	0
27	02 N10 W18	27878400	15	4181760	0	0	0	0	100	4181760
28	02 N10 W16	27878400	100	27878400	0	0	0	0	100	27878400
29	02 N10 W15	27878400	100	27878400	0	0	50	13939200	50	13939200
30	02 N10 W14	27878400	100	27878400	0	0	35	9757440	65	18120960
31	02 N10 W13	27878400	100	27878400	0	0	90	25090560	10	2787840
32	02 N9 W18	27878400	100	27878400	0	0	100	27878400	0	0
33	02 N9 W17	27878400	55	15333120	0	0	33	5059930	67	10273190
34	02 N09 W16	27878400	10	2787840	0	0	100	2787840	0	0
35	02 N10 W19	27878400	10	2787840	0	0	100	2787840	0	0
36	02 N10 W21	27878400	90	25090560	0	0	100	25090560	0	0
37	02 N10 W22	27878400	100	27878400	0	0	100	27878400	0	0
38	02 N10 W23	27878400	100	27878400	0	0	100	27878400	0	0
39	02 N10 W24	27878400	85	23696640	0	0	100	23696640	0	0
40	02 N9 W19	27878400	25	6969600	0	0	70	4878720	30	2090880
41	02 N10 W28	27878400	30	8363520	0	0	0	0	100	8363520
42	02 N10 W27	27878400	70	19514880	0	0	100	19514880	0	0
43	02 N10 W26	27878400	55	15333120	0	0	100	15333120	0	0
44	02 N10 W25	27878400	10	2787840	0	0	100	2787840	0	0
	Total (sq. ft.)			796207104		137092032		507233549		151881523
				(18342 acre)		(3147 acre)		(11708 acre)		(3487 acre)

Table 2 List of Current Properties Within Two Mile of Centennial Levee

Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
1	506-22-113	$27,000	$27,000	NO	-
2	506-22-123	$17,000	$17,000	NO	-
3	506-22-112	$17,000	$17,000	NO	-
4	506-16-001-A	$21,000	$21,000	NO	-
5	506-16-015	$21,000	$21,000	NO	-
6	506-16-004-C	$14,500	$14,500	NO	-
7	506-16-004-A	$31,500	$31,500	NO	-
8	506-16-002-B	$18,000	$18,000	NO	-
9	506-16-003-A	$34,500	$34,500	NO	-
10	506-16-002-A	$30,500	$30,500	NO	-
11	506-26-002	$11,500	$11,500	NO	-
12	506-26-003	$9,500	$9,500	NO	-
13	506-26-004	$10,500	$6,917	NO	-
14	506-26-001	$11,500	$7,844	NO	-
15	506-26-005	$12,500	$8,563	NO	-
16	506-26-006	$10,500	$6,602	NO	-
17	506-26-007	$9,000	$5,585	NO	-
18	506-26-008	$12,000	$8,254	NO	-
19	506-26-009	$10,000	$6,508	NO	-
20	506-26-010	$11,500	$11,500	NO	-
21	506-26-012	$12,000	$12,000	NO	-
22	506-26-013	$12,000	$8,462	NO	-
23	506-26-014	$12,500	$8,746	NO	-
24	506-26-015	$12,500	$12,500	NO	-
25	506-26-016	$9,900	$8,011	NO	-
26	506-26-017	$11,500	$7,926	NO	-
27	506-26-018	$11,500	$7,982	NO	-
28	506-17-006-B	$43,000	$43,000	NO	-
29	506-17-006-A	$40,500	$40,500	NO	-
30	506-17-006-C	$43,000	$43,000	NO	-
31	506-20-003-J	$17,000	$17,000	NO	-
32	506-20-003-D	$17,000	$13,554	NO	-
33	506-20-003-F	$12,500	$8,497	NO	-
34	506-20-003-G	$12,500	$8,497	NO	-
35	506-20-003-K	$17,000	$17,000	NO	-
36	506-20-003-M	$17,000	$17,000	NO	-
37	506-20-003-E	$23,500	$23,500	NO	-
38	506-20-027	$19,000	$19,000	NO	-
39	506-20-004-D	$17,000	$17,000	NO	-
40	506-20-005	$24,000	$24,000	NO	-
41	506-20-004-B	$40,000	$30,983	NO	-
42	506-20-004-E	$17,000	$17,000	NO	-
43	506-20-028	$19,000	$19,000	NO	-
44	506-20-002-G	$12,000	$12,000	NO	-
45	506-20-002-C	$12,000	$12,000	NO	-
46	506-20-002-H	$19,500	$19,500	NO	-
47	506-20-002-E	$12,500	$12,500	NO	-
48	506-20-002-F	$15,500	$15,500	NO	-

Table 2 List of Current Properties Within Two Mile of Centennial Levee					
Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
49	506-20-002-B	$9,000	$9,000	NO	-
50	506-20-001	$33,000	$33,000	NO	-
51	506-19-026-A	$30,500	$30,500	NO	-
52	506-19-027	$33,000	$33,000	NO	-
53	506-19-029	$24,000	$24,000	NO	-
54	506-19-028-D	$17,000	$17,000	NO	-
55	506-19-028-B	$17,000	$17,000	NO	-
56	506-19-028-C	$12,500	$12,500	NO	-
57	506-19-028-E	$12,500	$12,500	NO	-
58	506-19-028-F	$14,500	$14,500	NO	-
59	506-19-030	$24,000	$24,000	NO	-
60	506-19-028-G	$9,500	$9,500	NO	-
61	506-19-008-C	$19,500	$19,500	NO	-
62	506-19-008-B	$11,515	$5,973	NO	-
63	506-19-015-B	$28,000	$28,000	NO	-
64	506-19-015-A	$34,500	$34,500	NO	-
65	506-19-015-E	$11,000	$11,000	NO	-
66	506-19-015-D	$27,500	$27,500	NO	-
67	506-19-037	$11,500	$11,500	NO	-
68	506-19-002-B	$14,000	$14,000	NO	-
69	506-19-001-F	$8,000	$8,000	NO	-
70	506-19-001-E	$11,000	$11,000	NO	-
71	506-19-001-G	$12,000	$12,000	NO	-
72	506-19-001-H	$3,000	$3,000	NO	-
73	506-19-004-B	$10,500	$7,274	NO	-
74	506-19-038	$14,500	$14,500	NO	-
75	506-19-003-C	$8,500	$8,500	NO	-
76	506-19-003-E	$5,000	$5,000	NO	-
77	506-19-003-F	$7,500	$7,500	NO	-
78	506-19-004-B	$10,500	$7,274	NO	-
79	506-19-006	$24,000	$24,000	NO	-
80	506-19-005	$24,000	$24,000	NO	-
81	506-18-003-A	$25,000	$19,163	NO	-
82	506-18-003-B	$62,500	$60,976	NO	-
83	506-18-002-A	$30,500	$22,660	NO	-
84	506-18-002-B	$24,500	$17,170	NO	-
85	506-18-001-A	$40,000	$32,800	NO	-
86	506-18-004-G	$17,500	$14,659	NO	-
87	506-18-004-F	$45,000	$38,103	NO	-
88	506-18-004-C	$37,500	$29,778	NO	-
89	506-18-003-B	$62,500	$60,976	NO	-
90	506-18-004-E	$68,000	$57,768	NO	-
91	506-18-006	$67,500	$67,500	NO	-
92	506-18-039	$33,000	$26,025	NO	-
93	506-20-018-A	$33,000	$33,000	NO	-
94	506-20-019-C	$32,500	$32,500	NO	-
95	506-20-018-B	$33,000	$33,000	NO	-
96	506-20-019-B	$32,500	$32,500	NO	-

Table 2 List of Current Properties Within Two Mile of Centennial Levee					
Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
97	506-20-020	$45,500	$45,500	NO	-
98	506-19-031	$2,303,500	$1,300,123	NO	-
99	506-19-033-G	$26,000	$26,000	NO	-
100	506-19-032-B	$27,500	$27,500	NO	-
101	506-19-033-F	$12,000	$12,000	NO	-
102	506-19-033-H	$8,500	$8,500	NO	-
103	506-19-032-A	$25,000	$25,000	NO	-
104	506-19-032-A	$25,000	$25,000	NO	-
105	506-19-033-J	$37,000	$37,000	NO	-
106	506-19-033-D	$8,000	$8,000	NO	-
107	506-19-033-C	$28,000	$28,000	NO	-
108	506-19-016-B	$9,000	$9,000	NO	-
109	506-19-016-D	$6,000	$6,000	NO	-
110	506-19-018-E	$23,500	$23,500	NO	-
111	506-19-018-D	$17,000	$17,000	NO	-
112	506-19-018-Q	$17,000	$17,000	NO	-
113	506-19-016-C	$31,500	$31,500	NO	-
114	506-19-018-J	$12,500	$7,285	NO	-
115	506-19-018-H	$21,000	$18,686	NO	-
116	506-19-017-A	$16,561	$16,561	NO	-
117	506-19-017-B	$15,939	$15,939	NO	-
118	506-19-018-B	$23,500	$23,500	NO	-
119	506-18-008-L	$43,000	$35,260	NO	-
120	506-18-008-F	$22,000	$18,040	NO	-
121	506-18-008-G	$22,000	$18,040	NO	-
122	506-18-008-H	$25,500	$20,910	NO	-
123	506-18-008-H	$25,500	$20,910	NO	-
124	506-18-008-J	$25,500	$20,910	NO	-
125	506-18-008-P	$58,500	$41,535	NO	-
126	506-18-008-K	$28,000	$22,960	NO	-
127	506-18-008-R	$51,500	$36,565	NO	-
128	506-18-007	$153,571	$131,881	NO	-
129	506-18-008-E	$21,500	$17,265	NO	-
130	506-18-008-D	$51,000	$40,953	NO	-
131	506-18-008-Q	$51,500	$36,565	NO	-
132	506-18-010-A	$27,603	$26,386	NO	-
133	506-18-015	$28,810	$28,810	NO	-
134	506-18-014	$76,819	$76,819	NO	-
135	506-18-009-A	$27,603	$26,386	NO	-
136	506-18-011-A	$28,022	$26,787	NO	-
137	506-18-013	$76,819	$76,819	NO	-
138	506-20-023-A	$48,000	$38,544	NO	-
139	506-20-022-R	$6,657	$6,657	NO	-
140	506-20-022-S	$5,000	$5,000	NO	-
141	506-20-022-J	$12,000	$12,000	NO	-
142	506-20-022-H	$12,000	$12,000	NO	-
143	506-20-022-T	$6,500	$6,500	NO	-
144	506-20-022-V	$6,657	$6,657	NO	-

Table 2 List of Current Properties Within Two Mile of Centennial Levee					
Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
145	506-20-022-U	$6,657	$6,657	NO	-
146	506-20-022-H	$5,000	$5,000	NO	-
147	506-20-022-F	$12,000	$12,000	NO	-
148	506-20-021-E	$24,500	$24,500	NO	-
149	506-20-026	$23,000	$23,000	NO	-
150	506-20-021	$33,000	$33,000	NO	-
151	506-20-021-J	$8,000	$8,000	NO	-
152	506-20-021-F	$12,000	$12,000	NO	-
153	506-20-021-H	$17,500	$17,500	NO	-
154	506-19-039	$27,000	$27,000	NO	-
155	506-22-137-B	$4,376	$4,376	NO	-
156	506-22-137-C	$4,376	$4,376	NO	-
157	506-22-137-A	$8,750	$8,750	NO	-
158	506-22-136	$17,500	$17,500	NO	-
159	506-22-138	$17,500	$17,500	NO	-
160	506-22-135	$17,500	$17,500	NO	-
161	506-19-035-D	$750	$750	NO	-
162	506-19-035-C	$375	$375	NO	-
163	506-19-035-E	$375	$375	NO	-
164	506-19-035-A	$43,000	$43,000	NO	-
165	506-19-034	$32,500	$32,500	NO	-
166	506-19-025-A	$18,000	$18,000	NO	-
167	506-19-025-B	$29,500	$29,500	NO	-
168	506-19-022	$34,000	$34,000	NO	-
169	506-19-024-J	$16,000	$16,000	NO	-
170	506-19-024-H	$9,000	$9,000	NO	-
171	506-19-024-C	$18,000	$18,000	NO	-
172	506-19-023-D	$17,000	$17,000	NO	-
173	506-19-023-C	$17,500	$17,500	NO	-
174	506-19-023-H	$12,000	$10,121	NO	-
175	506-19-023-J	$12,500	$10,660	NO	-
176	506-19-023-B	$18,000	$18,000	NO	-
177	506-19-024-F	$13,000	$13,000	NO	-
178	506-19-024-G	$12,500	$12,500	NO	-
179	506-19-024-E	$18,000	$18,000	NO	-
180	506-19-023-G	$3,000	$1,661	NO	-
181	506-19-021-A	$24,000	$24,000	NO	-
182	506-19-020-A	$24,000	$24,000	NO	-
183	506-19-020-B	$24,000	$24,000	NO	-
184	506-19-019	$24,000	$24,000	NO	-
185	506-18-033	$96,500	$89,443	NO	-
186	506-21-033-A	$6,500	$5,330	NO	-
187	506-21-033-B	$7,500	$6,150	NO	-
188	506-21-033-C	$14,500	$11,890	NO	-
189	506-21-002-J	$9,000	$9,000	NO	-
190	506-21-088-A	$11,000	$11,000	NO	-
191	506-21-008-B	$10,500	$6,626	NO	-
192	506-21-008-L	$11,000	$6,952	NO	-

Table 2 List of Current Properties Within Two Mile of Centennial Levee					
Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
193	506-21-008-K	$10,500	$10,500	NO	-
194	506-21-008-H	$10,500	$6,625	NO	-
195	506-21-008-C	$10,500	$6,626	NO	-
196	506-21-008-D	$10,500	$6,625	NO	-
197	506-21-007	$24,000	$24,000	NO	-
198	506-21-006	$24,000	$24,000	NO	-
199	506-21-008-M	$10,500	$8,663	NO	-
200	506-21-008-N	$10,500	$10,500	NO	-
201	506-21-008-P	$10,500	$8,663	NO	-
202	506-21-005	$33,000	$33,000	NO	-
203	506-21-010-A	$24,000	$17,250	NO	-
204	506-21-010-C	$17,000	$17,000	NO	-
205	506-21-010-D	$17,000	$17,000	NO	-
206	506-21-011-A	$32,500	$32,500	NO	-
207	506-21-012	$33,000	$33,000	NO	-
208	506-21-009	$33,000	$33,000	NO	-
209	506-22-139	$17,500	$17,500	NO	-
210	506-22-134	$17,500	$17,500	NO	-
211	506-22-133	$17,500	$17,500	NO	-
212	506-22-140	$17,500	$17,500	NO	-
213	506-22-022	$45,500	$45,500	NO	-
214	506-22-132	$17,500	$17,500	NO	-
215	506-22-142	$17,500	$17,500	NO	-
216	506-22-141	$17,500	$17,500	NO	-
217	506-22-131	$17,500	$17,500	NO	-
218	506-22-024-E	$19,000	$19,000	NO	-
219	506-22-024-F	$19,000	$19,000	NO	-
220	506-22-098	$17,000	$17,000	NO	-
221	506-22-093	$17,000	$17,000	NO	-
222	506-22-024-G	$12,000	$12,000	NO	-
223	506-22-024-H	$12,000	$12,000	NO	-
224	506-22-095	$17,500	$17,500	NO	-
225	506-22-100	$17,000	$17,000	NO	-
226	506-22-024-B	$24,000	$24,000	NO	-
227	506-22-122	$17,000	$17,000	NO	-
228	506-22-111	$17,000	$17,000	NO	-
229	506-22-101	$17,000	$17,000	NO	-
230	506-22-096	$17,500	$17,500	NO	-
231	506-22-094	$17,000	$17,000	NO	-
232	506-22-099	$17,000	$17,000	NO	-
233	506-21-016-B	$21,000	$21,000	NO	-
234	506-21-016-A	$12,500	$12,500	NO	-
235	506-21-023	$33,000	$33,000	NO	-
236	506-21-014-A	$17,000	$17,000	NO	-
237	506-21-023-C	$17,000	$17,000	NO	-
238	506-21-014-B	$17,000	$17,000	NO	-
239	506-21-013	$17,000	$17,000	NO	-
240	506-21-015	$17,000	$17,000	NO	-

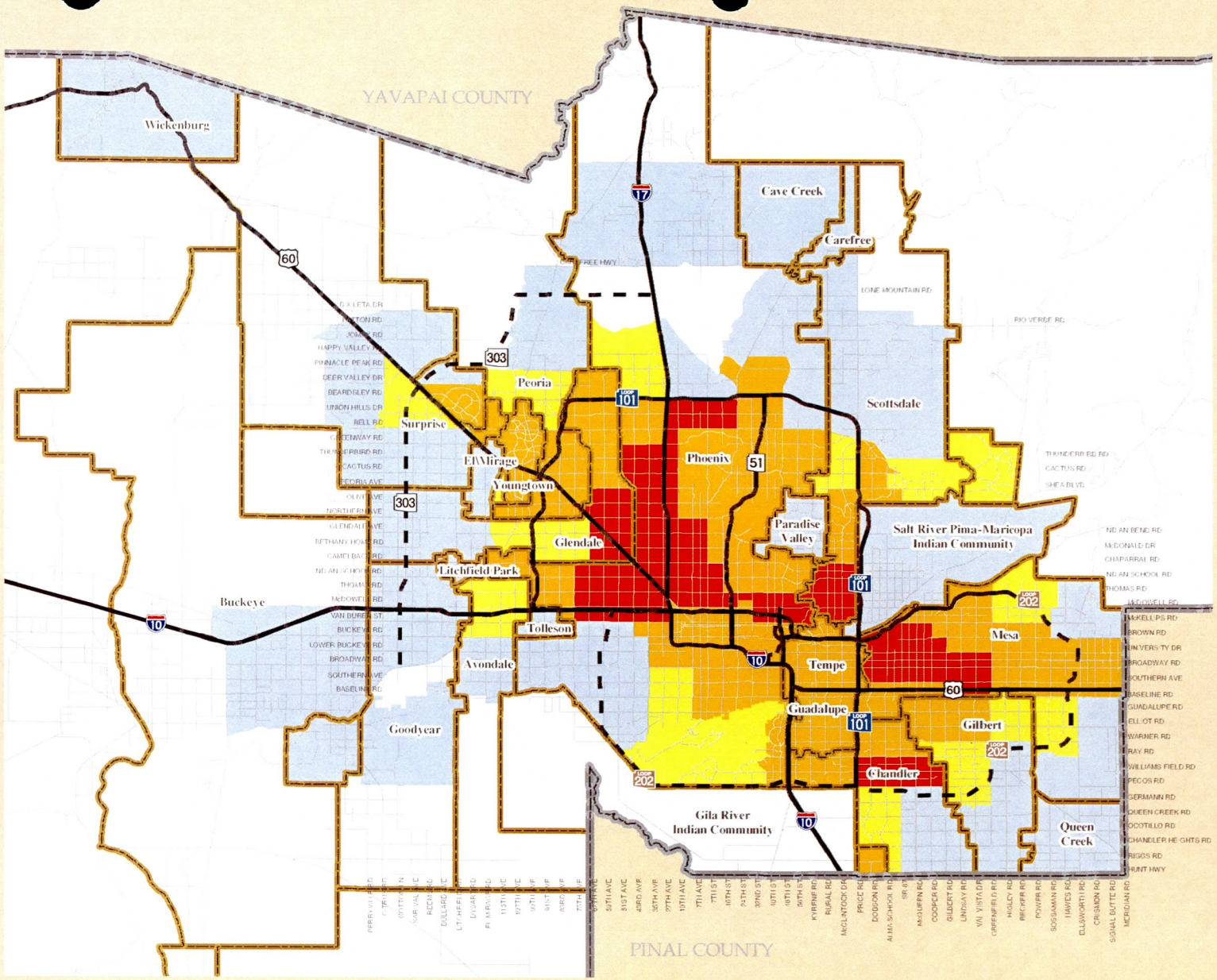
Table 2 List of Current Properties Within Two Mile of Centennial Levee					
Sl. No.	Parcel ID#	2005 Full Cash Value	2005 Limited Property Value	Structure	Condition
241	506-21-017	$17,000	$17,000	NO	-
242	506-21-018	$17,000	$17,000	NO	-
243	506-21-023-D	$29,000	$29,000	NO	-
244	506-22-156	$22,584	$22,584	NO	-
245	506-22-155	$17,500	$17,500	NO	-
246	506-22-154	$17,500	$17,500	NO	-
247	506-22-157	$17,500	$17,500	NO	-
248	506-22-158	$17,500	$17,500	NO	-
249	506-22-153-C	$4,376	$4,376	NO	-
250	506-22-153-D	$4,376	$4,376	NO	-
251	506-22-027	$45,500	$45,500	NO	-
252	506-22-153-B	$4,374	$4,374	NO	-
253	506-22-153-A	$4,374	$4,374	NO	-
254	506-22-152	$17,500	$17,500	NO	-
255	506-22-159	$17,500	$17,500	NO	-
256	506-26-011	$9,500	$8,182	NO	-
257	506-20-003-L	$17,000	$17,000	NO	-

APPENDIX B

Population Density Maps

MARICOPA ASSOCIATION of GOVERNMENTS

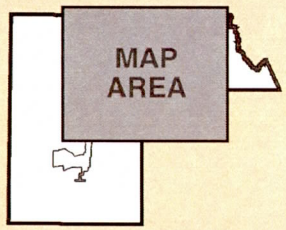
2000 Population Density for Interim Socioeconomic Projections by RAZ*



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



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

N

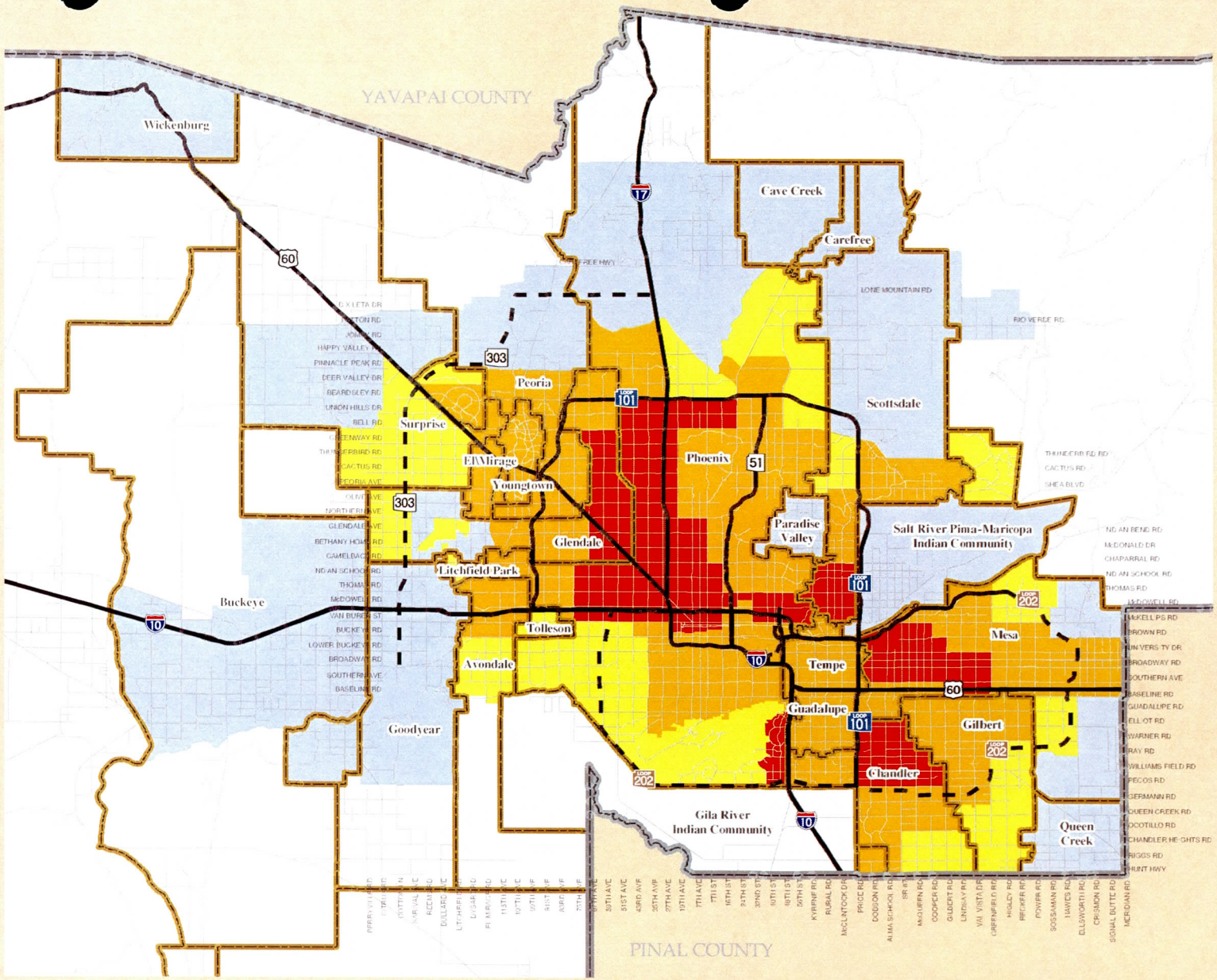
*Based on interim projections by Municipal Planning Area (MPA) and Regional Analysis Zone (RAZ) for 2010, 2020, 2025 and 2030 accepted by MAG Regional Council on June 25, 2003.

Prepared by Maricopa Association of Governments, July 2003

G:\Dev\work\seth\Densities by RAZ\ArcMap\2000 PopDens

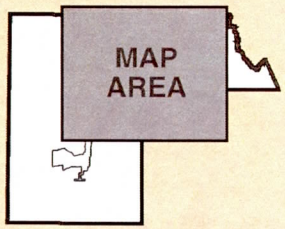
MARICOPA ASSOCIATION of GOVERNMENTS

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



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0 5 10 15 Miles

N

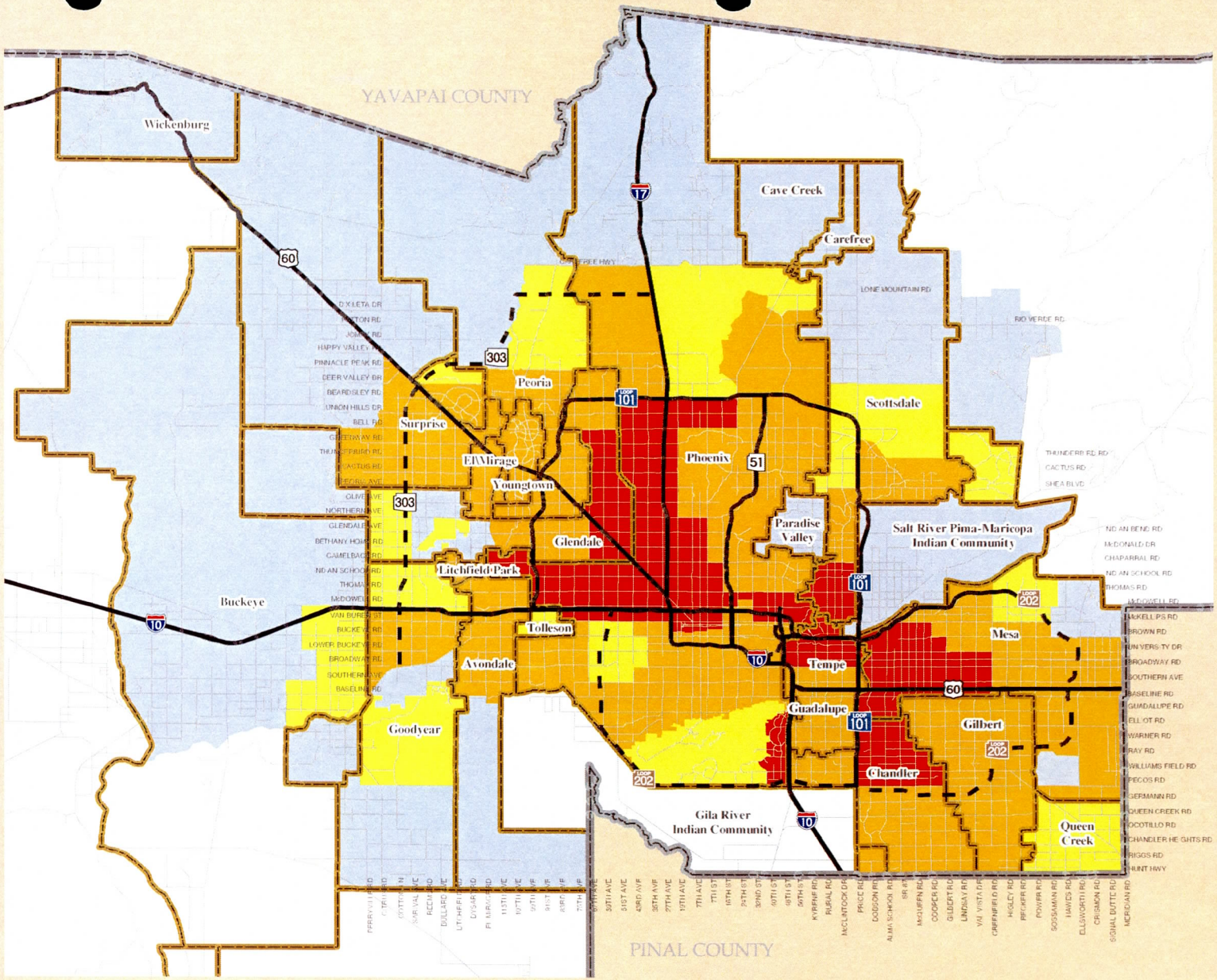
*Based on interim projections by Municipal Planning Area (MPA) and Regional Analysis Zone (RAZ) for 2010, 2020, 2025 and 2030 accepted by MAG Regional Council on June 25, 2003.

Prepared by Maricopa Association of Governments, July 2003

G:\Dev\work\seth\Densities by RAZ\ArcMap\2010 PopDens

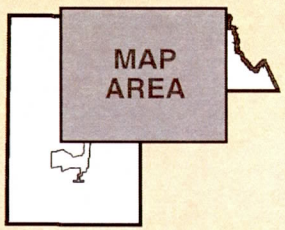
MARICOPA ASSOCIATION of GOVERNMENTS

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



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

N

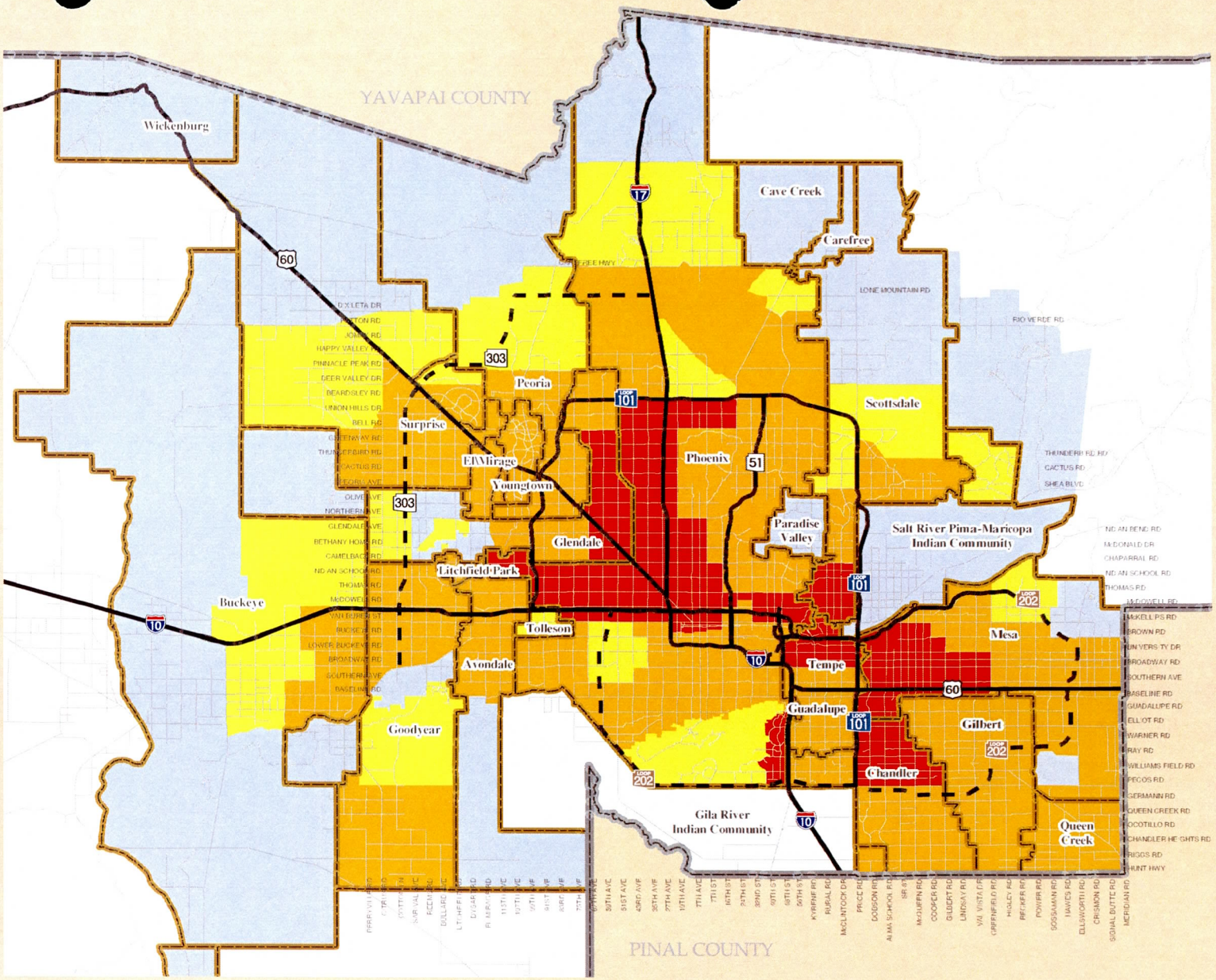
*Based on Interim projections by Municipal Planning Area (MPA) and Regional Analysis Zone (RAZ) for 2010, 2020, 2025 and 2030 accepted by MAG Regional Council on June 25, 2003.

Prepared by Maricopa Association of Governments, July 2003

G:\Dev\work\seth\Densities by RAZ\ArcMap\2020 PopDens

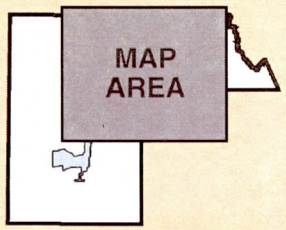
MARICOPA ASSOCIATION of GOVERNMENTS

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



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.

Miles 0 5 10 15

N

*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

G:\Dev\work\seth\Densities by RAZ\ArcMap\2030 PopDens

USDA NATURAL RESOURCES CONSERVATION SERVICE
CONSERVATION PRACTICE STANDARD
ARIZONA

DIKE

(feet)
CODE 356

DEFINITION

An embankment constructed of earth or other suitable materials to protect land against overflow or to regulate water.

SCOPE

This standard applies to dikes or levees used to prevent or reduce flood damage to land and property, for flow control in conjunction with floodways or to impound or regulate water for fish and wildlife management.

Dikes are divided into classes determined by the value of the land, crops, and other improvements and the hazard to life within the area to be protected.

PURPOSE

To permit improvement of agricultural land by preventing overflow and better use of drainage facilities, to prevent damage to land and property, and to facilitate water storage and control in connection with wildlife and other developments. Dikes can also be used to protect natural areas, scenic features and archeological sites from damage.

CONDITIONS WHERE PRACTICE APPLIES

Class I dikes are those constructed on sites where:

1. Failure may cause loss of life or serious damage to homes, industrial and

Conservation practice standards are reviewed periodically, and updated if needed. To obtain the current version of this standard, contact the Natural Resources Conservation Service.

commercial buildings, important public utilities, main highways or railroads, and high value land, crops, or other improvements.

2. Unusual or complex site conditions require special construction procedures to ensure satisfactory installations.
3. Protection is needed to withstand more than 12 ft (3.7 m) of water above normal ground surface, exclusive of crossing of sloughs, old channels, or low areas.

Class II dikes are those constructed in highly developed and productive agricultural areas where:

1. Failure may damage isolated homes, highways or minor railroads, or cause interruption in service of relatively important public utilities.
2. The maximum design water stage against the dike is 12 ft (3.7 m).

Class III dikes are those constructed in rural or agricultural areas where:

1. Damage likely to occur from dike failure is minimal.
2. The maximum design water stage against the dike is 6 ft (1.8 m) for mineral soils and 4 ft (1.2 m) for organic soils. (Exclude channels, sloughs, swales, and gullies in determining the design water stage.)

CRITERIA - ALL DIKES

In locating dikes, careful considerations shall be given to preserving natural areas, fish and wildlife habitat, woodland, and other environmental resources. If dike construction will adversely affect such values, concerned public agencies and private organizations shall be consulted about the project.

Protection. A protective cover of grasses shall be established on all exposed surfaces of the dike and other disturbed areas. Seedbed preparation, seeding, fertilizing, mulching, and fencing shall comply with recommendations in local technical guides.

If vegetation will not control erosion, riprap or other protective measures shall be installed.

Maintenance. All dikes must be adequately maintained to the required shape and height. The maintenance of dikes must include periodic removal of woody vegetation that may become established on the embankment. Provisions for maintenance access must be provided.

Cultural Resources

If this practice involves soil disturbance, the area of potential effect for each undertaking must be investigated for cultural resources under section 106 of the National Historical Preservation Act of 1966, as amended, before soil disturbance occurs. See the NRCS Arizona Handbook of Cultural Resources Procedures - Applicability and Exceptions Section - for identification of practices that are exempt from, or that require cultural resources surveys.

Endangered Species

Determine if installation of this practice, with any others proposed, will affect any federal, tribal, or state listed Threatened or Endangered species or their habitat. NRCS's objective is to benefit these species or at least not have any adverse effect on a listed species. If the Environmental Evaluation indicates the action may adversely affect a listed species or result in adverse modification of habitat of listed species which has been determined to be critical habitat, NRCS will advise the land user of the requirements of the Endangered Species Act and recommend alternative conservation treatments that avoid the adverse effects.

Further assistance will be provided only if the landowner selects one of the alternative conservation treatments for installation; or at the request of the landowners, NRCS may initiate consultation with the U.S. Fish and Wildlife Service. If the Environmental Evaluation indicates the action will not affect a listed species or result in adverse modification of critical habitat, consultation generally will not apply and usually would not be initiated. Document any special considerations for endangered species in the Practice Requirements Worksheet.

CRITERIA - CLASS I DIKES

Location. Conditions to be considered in designing Class I dikes are foundation soils, property lines, exposure to open water, adequate outlets for gravity or pump drainage, and access for construction and maintenance.

Mineral soils that will be stable in the dike embankment must be available.

Height. The design height of a dike shall be the design high water depth plus 2 ft (0.6 m) of freeboard or 1 ft (0.3 m) of freeboard plus an allowance for wave height, whichever is greater. Design elevation of high water shall be determined as follows:

1. If dike failure is likely to cause loss of life or extensive high-value crop or property damage, the elevation of design high water shall be that associated with the stage of the 100-year-frequency flood or of the maximum flood of record, whichever is greater.
2. If dike failure is unlikely to result in loss of life or extensive high-value crop or property damage, the elevation of design high water shall be that associated with the peak flow from the storm that will insure the desired level of protection or the 50-year-frequency flood whichever is greater.
3. If the dike will be subject to stages from more than one stream or source, the criteria indicated shall be met for the combination that causes the highest stage.
4. If the dike will be subject to tidal influence as well as streamflow, the streamflow peak shall be assumed to occur in conjunction with the mean high tide to determine the design high water depth.

The design height of the dike shall be increased by the amount needed to insure that the design top elevation is maintained after settlement. This increase shall be not less than 5 percent.

Interior drainage. If inflow from the area to be protected by the dike may result in loss of life or extensive high-value crop property damage, provisions shall be included in the plans to provide interior protection against a 100-year-frequency hydrograph, plus base flow, and an allowance for seepage, and may include storage areas, gravity outlets, or pumping plants, alone or in combination.

If inflow from the area to be protected by the dike is unlikely to result in loss of life or extensive high-value crop or property damage, storage areas, gravity outlets, or a pumping plant, alone or in combination, shall be included in the plans and designed to handle the discharge from the drainage area based on drainage requirements established for the local area or the peak flow from the storm that will insure the desired level of protection, whichever is greater.

In sizing outlet works in combination with available storage, the minimum design storm duration for interior drainage shall be 10 days. If outlet works are designed using peak flood frequency flows without considering storage, the minimum design storm duration shall be 24 hours.

Embankment and foundation. The embankment shall be constructed of mineral soils, which when placed and compacted will result in a stable earth fill. No organic soil shall be used in the dike. Soils must have high specific gravity and be capable of being formed into an embankment of low permeability. The design of the embankment and specifications for its construction shall give due consideration to the soil materials available, foundation conditions, and requirements for resisting the action of water on the face of the dike and excessive seepage through the embankment and the foundation. The design of the embankment and the foundation requirements shall be based on the length of time and height that water will stand against the dike.

Minimum requirements for certain features of the embankment, the foundation, and borrow pits are as follows:

Minimum top width of Class I dikes shall be 10 ft (3 m) for embankment heights of 15 ft (4.6 m) or less and 12 ft (3.6 m) for heights more than 15 ft (4.6 m). If maintenance roads are to be established on the dike top, "turnarounds" or passing areas shall be provided, as needed.

Side slopes shall be determined from a stability analysis, except that an unprotected earth slope on the water side shall not be steeper than 4 horizontal to 1 vertical if severe wave action is anticipated.

If dikes cross old channels or have excessively porous fills or poor foundation conditions, the landside toe shall be protected by a banquette or constructed berm. Banquettes shall be used to provide construction access and added stability if channel crossings are under water or saturated during construction. Banquettes

shall be designed on the basis of site investigations, laboratory analysis, and compaction methods. The finished top width of the banquettes shall not be less than the height of dike above mean ground. The finished top of the banquettes shall not be less than 1 ft (0.3 m) above mean ground and shall be sloped away from the dike.

A cutoff shall be used if foundation materials are sufficiently pervious to be subject to piping or undermining. The cutoff shall have a bottom width and side slopes adequate to accommodate the equipment to be used for excavation, backfill, and compaction operations. It shall be backfilled with suitable material placed and compacted as required for the earth embankment. If previous foundations are too deep to be penetrated by a foundation cutoff, a drainage system adequate to insure stability of the dike shall be used.

Ditches and borrow pits. Landside ditches or borrow pits shall be located so the hazard of failure is not increased. Ditches for borrow pits when excavated on the water side of dikes shall be wide and shallow. Plugs, at least 15 ft (4.6 m) in width, shall be left in the ditches at intervals not greater than 400 ft (121.9 m) to form a series of unconnected basins.

Minimum berm widths between the toe of the dike and the edge of the excavated channel or borrow shall be:

Fill height	Minimum berm width
Less than 6 ft (1.8 m)	12 ft (3.7 m)
More than 6 ft (1.8 m)	18 ft (5.5 m)

A drainage system shall be used if necessary to insure the safety of a dike. Toe drains, if used, shall be located on the landside and shall have a graded sand-gravel filter designed to prevent movement of the foundation material into the drain.

Subsurface drains shall not be installed, or permitted to remain without protection, closer to the landside toe of a dike than a distance three times the design water height for the dike. If subsurface drains are to be installed or remain closer than the distance stated, protection shall consist of a graded sand-gravel filter, as for a toe drain, or a closed pipe laid within the specified distances from the dike.

Pipes and conduits. Dikes shall be protected from scour at pump intakes and discharge locations by appropriate structural measures. A pump discharge pipe through a dike shall be installed above design high water, if feasible, or be equipped with antiseep collars.

All conduits through a dike below the design high waterline shall be equipped with antiseep collars designed to increase the distance of the seepage line along the conduit by at least 15 percent. Discharge conduits of pumps placed below the designed water line shall be equipped with a Dayton or a similar coupling to prevent vibration of the pumping plant being transmitted to the discharge conduits.

CRITERIA - CLASS II DIKES

Design water stage. The maximum design water stage permitted is 12 ft (3.7 m) above normal ground level exclusive of crossings at channels, sloughs, and gullies.

If the design water depth against dikes, based on the required level of protection, exceeds 4 ft (1.2 m) the design shall be based on at least a 25-year-frequency flood. If this degree of protection is not feasible, the design shall approach the 25-year flood level as nearly as possible, and planned fuse plug sections and other relief measures shall be installed where appropriate.

Height. The design height of an earth dike shall be the design water depth plus a free-board of at least 2 ft (0.6 m) or freeboard of 1 ft (0.1 m) plus an allowance for wave height, whichever is greater.

The constructed height of the dike shall be the design height plus allowance for settlement necessary to insure that the design top elevation is maintained but shall be no less than 5 percent of the design height.

Interior drainage. Provisions must be made for adequate drainage for the area to be protected by the dike.

Cross section. The minimum requirements for the cross section of the dike where fill is compacted by hauling or special equipment shall be as follows:

Design water height		Minimum top width		Steepest side slope
ft	*m*	*ft*	*m*	

0-6	(0-1.8)	6	(1.8)	1-1/2:1
6-12	(1.8-3.7)	8	(2.4)	2:1

If soils or water conditions make it impractical to compact the dike with hauling or special equipment, dumped fill may be used and shall have minimum cross section dimensions incorporated in the fill as follows:

Design water height		Minimum top width		Steepest side slope
ft	*m*	*ft*	*m*	
0-6	(0-1.8)	6	(1.8)	1-1/2:1
6-12	(1.8-3.7)	10	(3)	2:1

Side slopes of 3 horizontal to 1 vertical on waterside and 2:1 on landside may be used instead of 2-1/2:1 for both slopes.

The cross sections shall be strengthened or increased as required to provide additional protection against floods of long duration. The top width shall be less than 10 ft (3 m) if a maintenance road is planned on top the dike. "Turnarounds" or passing areas shall be provided as required on long dikes.

The side slopes shall be 3:1 or flatter on the waterside if severe wave action is expected or if a steeper slope would be unstable under rapid drawdown conditions. Side slopes shall be 3:1 or flatter on both sides where permeable soils of low plasticity, such as SM and ML, are used in construction.

A banquette (or constructed berm) shall reinforce the landside toe if a dike crosses an old channel or if excessively porous fill or poor foundation conditions justify such reinforcement. Such banquettes shall be used if, during construction, the channel crossing is under water or saturated. The top width of the banquette shall be equal to or greater than the fill height of the dike above the top of the banquette unless a detailed investigation and analyses show a different design is adequate.

Foundation cutoff. A cutoff shall be installed if there are layers of permeable soils or layers creating a piping hazard through the foundation at a depth less than the design water depth of the dike below natural ground level. The cutoff trench shall be of sufficient depth and width and filled with suitable soils to minimize such hazard.

Ditches and borrow pits. Minimum berm widths between the toe of the dike and the edge of the excavated channel or borrow shall be:

Fill height	Minimum berm width
Less than 6 ft (1.8 m)	10 ft (3 m)
More than 6 ft (1.8 m)	15 ft (4.6 m)

A landside ditch or borrow pit shall be far enough away from the dike to minimize any hazard to the dike because of piping through the foundation.

For dikes having a design water depth of more than 5 ft, (1.5 m), the landside ditch or borrow pit shall be far enough away from the dike so that a line drawn between the point of intersection of the design waterline with the waterside of the dike and the landside toe of a dike meeting minimum dimensional requirements shall not intersect the ditch or borrow pit cross section.

Pipes and conduits. The dike shall be protected from scour at a pump intake and discharge by appropriate structural measures. A pump discharge pipe through the dike shall be installed above design high water, if feasible, or else equipped with antiseep collars.

All conduits through the dike below the design high waterline shall be equipped with antiseep collars designed to increase the distance of the seepage line along the conduit by at least 15 percent. Discharge conduits of pumps placed below the designed waterline shall be equipped with a Dayton or a similar coupling to prevent vibrations of the pumping plant being transmitted to the discharge conduits.

Drains. Drains shall be used where necessary to insure safety of dikes and shall be located on the land side, have a graded sandgravel filter, and be designed and installed in accordance with Soil Conservation Service standards for such drains.

Field subsurface drains shall not be installed or permitted to remain without protection closer to the landside toe of a dike than a distance three times the design water height for the dike. If such drains are to be installed or remain closer than the distance stated above, protection shall consist of a graded sandgravel filter, as for a toe drain, or a closed pipe laid within the specified distances from the dike.

CRITERIA - CLASS III DIKES

The design criteria shall be based on site conditions for mineral or organic soils as applicable.

Top width. Minimum top width is 4 ft (1.2 m).

Side slopes. Minimum side slope is 1:1.

Freeboard. The minimum freeboard is 1 ft (0.3 m) plus wave height. The constructed height shall be increased by the amount necessary to insure that the settled top is at design elevation but not less that 5 percent.

Foundation cutoff. A cutoff shall be installed if necessary to insure dike stability.

Ditches and borrow pits. Minimum berm widths between the toe and the dike and the edge of the excavated channel or borrow shall be two times the depth of the ditch but not less than 8 ft (2.4 m).

PLANS AND SPECIFICATIONS

Plans and specifications for constructing dikes shall be in keeping with this standard and shall describe the requirements for applying the practice to achieve its intended purpose.

PLANNING CONSIDERATIONS FOR WATER QUANTITY AND QUALITY

Quantity

1. Effects upon components of the water budget, especially on volumes and rates of runoff, infiltration, evaporation, and transpiration.
2. Potential for changes in rates of plant growth and transpiration because of changes in the volume of soil water.
3. Effects on downstream flows or aquifers that would affect other water uses or users.
4. Effects on the rate or volume of downstream flow to prohibit environmental, social, or economic effects.

Quality

1. Effect on erosion and the movement of sediment and soluble and sediment-attached substances carried by runoff.
2. Effects on the movement of dissolved substances to ground water.
3. Short-term, construction, and maintenance related effects on the quality of water resources.
4. .Effects on temperature of water resources to prevent undesired effects on aquatic and wildlife communities.
5. Effects on wetlands or water-related wildlife habitats that would be associated with the practice.
6. Effects on the visual quality of water resources.

USDA NATURAL RESOURCES CONSERVATION SERVICE
CONSERVATION PRACTICE SPECIFICATION
ARIZONA

DIKE
CLASS II and III
(ft)
CODE 356

1. SCOPE

The work shall consist of all preparation, excavation, and furnishing and installation of materials required dike construction.

This specification applies to Class II and III dikes only. Specifications for Class I dikes will be developed using the National Engineering Handbook, Section 20.

2. GENERAL

Preparation of site for dike construction shall be performed in a manner which destroys as little vegetation outside the areas to be occupied by dikes and borrow pits as feasible. Special effort shall be made to save trees of significant value which are not in the area to be occupied by the dike.

Construction operations shall be carried out in a manner to minimize air and water pollution and hold such pollution within legal limits. Bare areas shall be revegetated as soon as practical after earthwork is completed. A minimum area shall be stripped of vegetation at any one time to provide an adequate work site.

Disposal of debris from site preparation shall be performed in a manner to cause minimum pollution to the environment.

3. SITE PREPARATION

All trees, brush, stumps, roots, rocks and other objectionable materials shall be removed from the foundation and borrow areas and disposed of beyond the limits of the work so as to not interfere with construction.

Following satisfactory completion of clearing operations, all channel banks and sharp breaks shall be sloped to no steeper than 2:1. Topsoil that is high in organic matter shall be removed. The surface area of the foundation area will be thoroughly scarified before placement of the embankment material.

4. CUT-OFF TRENCH

The cut-off trench excavation shall be to lines and grades shown on the plans.

The borrow areas for backfill material and moisture, placement, and compaction requirements for backfill shall be according to Section "Earthfill" below.

5. CONDUIT INSTALLATION

All conduits through a dike shall be placed on a firm foundation. Backfill material approved by the technician shall be placed in layers around the conduits and their component parts and each successive layer shall be thoroughly compacted.

6. EARTHFILL

All fill material shall be obtained from borrow areas shown on the plans or laid out at the site, or as approved by the technician. The fill material shall be free of organic matter and other objectionable material and shall contain a minimum percent of fine material as specified in the plans.

The placing and spreading of fill materials shall be started at the lowest point of the foundation and the fill brought up in approximately 8.0-inch horizontal lifts before compaction. The top shall be maintained as essentially a level surface during construction. Smooth surfaces resulting from the method of compaction shall be scarified. Where borrow yields material of varying texture and gradation, the more impervious material shall be placed toward the water side of the dike.

Material, when placed shall contain sufficient moisture so that a sample taken in the hand

and squeezed shall remain intact when released.

Compaction for Class II dikes shall be obtained by:

1. Two complete passes over the entire surface area of each lift with pneumatic rollers or sheepsfoot rollers.
2. Two complete passes over the entire surface area of each lift with heavily loaded rubber-tired scrapers.
3. Track type (crawler) equipment provided it is routed so the entire surface area of each lift is traversed by not less than four passes of the tracks.

Compaction for Class III dikes shall be obtained through normal routing of rubber-tired or track-type equipment when the preceding moisture requirements are met.

7. BLOCKS, ACROSS BORROW AREAS

When borrow areas are made behind dikes, undisturbed strips or compacted plugs will be placed every 660 feet, or closer as needed, to prohibit flow through the borrow area.

8. CULTURAL RESOURCES

This practice involves soil disturbance. The area of potential effect for each undertaking must be investigated for cultural resources under Section 106 of the National Historical Preservation Act of 1966, as amended, before soil disturbance occurs. (See the NRCS Arizona Handbook of Cultural Resources Procedures in Section I of the FOTG.)

9. SPECIAL REQUIREMENTS

Installation shall be in accordance with the following drawings, specifications and special requirements. NO CHANGES ARE TO BE MADE IN THE DRAWINGS OR SPECIFICATIONS WITHOUT PRIOR APPROVAL OF NRCS.

Other Requirements

10. REQUIRED ATTACHMENTS

☐ Plan Map showing location

☐ Designs showing alignment, width, side slopes, drainage, erosion control, surfacing, traffic safety, and construction operations.

Drawings, No.

11. OTHER ATTACHMENTS

☐ Associated Practice Specifications _____

☐ Water Quality Considerations

☐ Other _____

12. OPERATION AND MAINTENANCE

This conservation practice is an asset to your farm or ranch. This practice will need periodic operation and maintenance to maintain satisfactory performance. The life of this practice or system is at least 10 years. The life of this practice can be assured or extended by thorough and timely operation and maintenance. Here are some recommendations to help you develop a good operation and maintenance program.

GENERAL RECOMMENDATIONS

☐ Maintain the roadway surface in a good condition, which includes periodic grading or repair of the surface.

☐ Prevent surface ponding by grading to remove depressions.

☐ Limit livestock usage to periods that permit use without damage.

☐ If fences are installed, they shall be maintained to provide warning and/or prevent unauthorized human or livestock entry.

☐ Remove debris to prevent blockage of stream crossings, culverts or bridges.

☐ Control all rodents or burrowing animals. Immediately repair any damage caused by their activity.

☐ Immediately repair any damage from vandalism, vehicles, or livestock.

Specific Recommendations For Your Installation

STATEMENT OF WORK
Dike (356)
Arizona (5/13/04)

-These deliverables apply to this individual practice. For other planned practice deliverables refer to those specific Statements of Work.

DESIGN

Deliverables:

1. Design documentation that will demonstrate that the criteria in NRCS practice standard have been met and are compatible with other planned and applied practices.
    a. Practice purpose(s) as identified in the conservation plan
    b. List of required permits to be obtained by the client
    c. Compliance with NRCS national and state utility safety policy (NEM Part 503-Safety, Subpart A - Engineering Activities Affecting Utilities 503.00 through 503.06)
    d. Practice standard criteria related computations and analyses to develop plans and specifications including but not limited to:
        i. Geology and Soil Mechanics (NEM Subpart 531a)
        ii. Hydrology/Hydraulics
        iii. Structural including dike classification
        iv. Vegetation
        v. Environmental Considerations
        vi. Safety Considerations (NEM Part 503-Safety, Subpart A, 503.10 through 503.12)
2. Written plans and specifications including sketches and drawings shall be provided to the client that adequately describes the requirements to install the practice and obtain necessary permits...
3. Design Report and Inspection Plan as appropriate (NEM Part 511, Subpart B Documentation, 511.11 and Part 512, Subpart D Quality Assurance Activities, 512.30 through 512.32).
4. Operation and Maintenance Plan (National Operation and Maintenance Manual 500.70 and 500.40 through 500.42)
5. Certification that the design meets practice standard criteria and comply with applicable laws and regulations (NEM Subpart A, 505.03 (a) (3)).
6. Design modifications during installation as required.

INSTALLATION

Deliverables

1. Pre Installation conference with client and contractor.
2. Verification that client has obtained required permits.
3. Staking and layout according to plans and specifications including applicable layout notes.
4. Installation inspection (according to inspection plan as appropriate).
    a. Actual materials used (Part 512, Subchapter D Quality Assurance Activities, 512.33)
    b. Inspection records
5. Facilitate and implement required design modifications with client and original designer
6. Advise client/NRCS on compliance issues with all federal, state, tribal, and local laws, regulations and NRCS policies during installation.
7. Certification that the installation process and materials meets design and permit requirements.

CHECK OUT

Deliverables

1. As-Built documentation.
    a. Extent of practice units applied
    b. Drawings
    c. Final quantities
2. Certification that the installation meets NRCS standards and specifications and is in compliance with permits (NEM Subpart A, 505.03 (c) (1)).
3. Progress reporting.

REFERENCES

- NRCS Field Office Technical Guide (eFOTG), Section IV, Conservation Practice Standard - Dike, 356
- National Operation and Maintenance Manual
- NRCS Technical Release 60, Earth Dams and Reservoirs
- NRCS National Engineering Manual (NEM).
- NRCS National Environmental Compliance Handbook
- NRCS Cultural Resources Handbook

State Contact: Hydraulic Engineer

NATURAL RESOURCES CONSERVATION SERVICE
CONSERVATION PRACTICE STANDARD

DIKE

(Ft.)

CODE 356

DEFINITION

A barrier constructed of earth or manufactured materials.

PURPOSE

- To protect people and property from floods.
- To control water level in connection with crop production; fish and wildlife management; or wetland maintenance, improvement, restoration, or construction.

CONDITIONS WHERE PRACTICE APPLIES

All sites that are subject to damage by flooding or inundation and where it is desired to reduce the hazard to people and to reduce damage to land and property.

Sites where the control of water level is desired.

The dike standard does not apply to sites where NRCS conservation practice standards Pond (378), Water and Sediment Control Basin (638), Diversion (362), or Terrace (600) are appropriate. Dikes used to reduce flooding are normally constructed adjacent and/or parallel to a stream, river, wetland or water body and are not constructed across the stream, river or water body. Dikes used to control water levels usually have small interior drainage areas in relation to the surface area of the regulated water level.

CRITERIA

Classification. The dike classification is determined by the hazard to life, the design water height, and the value of the protected land, crops, and property. Classification must consider land use changes likely to occur over the life of the dike.

Dikes are classified as Class I when located on sites where failure may cause loss of life or serious damage to homes, primary highways, industrial buildings, commercial buildings, major railroads or important public utilities.

All dikes with a design water height of more than 12 feet above normal ground surface, exclusive of crossings of sloughs, old channels, or low areas shall be classified as Class I.

Dikes are classified as Class II when located on sites where failure may cause damage to isolated homes, secondary highways, minor railroads, relatively important public utilities, high value land, or high value crops.

Dikes are classified as Class III when located on sites where damage likely to occur from failure will be minimal.

Constructed Elevation. The constructed elevation of a dike whose purpose is to prevent flooding shall be the sum of the following:

- The water elevation attained by a flood or high tide of the design frequency shown in Table 1 with the critical duration and timing. This is the design high water.
- The larger of the minimum freeboard in Table 1 or the wave height caused by wind or boat traffic.
- The allowance for settlement.

The constructed elevation of a dike whose purpose is to control water level shall be the sum of the following:

- The water elevation at the highest water level control.
- The rise in water height above the highest water level control caused by a flood of the design frequency shown in Table 1. This is the design high water.
- The larger of the minimum freeboard shown in Table 1 or the wave height caused by wind of the design frequency

Conservation practice standards are reviewed periodically, and updated if needed. To obtain the current version of this standard, contact the Natural Resources Conservation Service.

shown in Table 1.

- The allowance for settlement.

Settlement. Settlement shall be based on an analysis of the fill material, foundation material and condition, and compaction methods.

In lieu of an analysis, the allowance for settlement shall be as follows:

1. For dikes constructed of compacted earth fill material shall be a minimum of 5% of the dike height.
2. For Class II or Class III dikes, constructed of fill material that is hauled from off-site, dumped, and shaped (referred to as "dumped and shaped"), the allowance for settlement shall be a minimum of 15% of the dike height. For fill material that is excavated adjacent to the dike and dropped from the excavator (referred to as "dropped"), the allowance for settlement shall be a minimum of 20% of the dike height. The allowance for settlement of dumped and shaped or dropped organic soil fill material shall be a minimum of 40% of the dike height. Organic soils are permitted only for Class III dikes 6 feet or less in height. Higher dike heights result in excessive settlement and decomposition.

For the purpose of this standard, organic soils are described as follows:

1. Soil layers that are not saturated with water for more than a few days at a time are organic if they have 20 percent or more organic carbon.

   or

2. Layers that are saturated for longer periods, or were saturated before being drained, are organic if:

   (a) They have 12 percent or more of organic carbon and no clay, or

   (b) 18 percent or more organic carbon and 60 percent or more clay, or

   (c) A proportional amount of organic carbon, between 12 and 18 percent, if the clay content is between 0 and 60 percent.

   or

3. All soils described in the local soil survey as an organic soil.

Top Width and Side Slopes. The minimum top widths and side slopes for earth embankments shall be as shown in Table 1.

All dikes must be accessible for maintenance activities. Typically, this may be along the top of the dike or along the berm. Access roads shall provide adequate width for the maintenance equipment and inspection vehicles. The minimum width for vehicular traffic should be 12 feet. Provide wider areas for passing and turning around at regular intervals. Access roads may need to be controlled to prevent vandalism, accidents, and damage.

Berms. The need for a constructed berm on an embankment will be based on the results of an embankment and foundation stability analysis. If a stability analysis is not performed, all earth dikes shall have berms either constructed or occurring naturally on both sides meeting the following criteria:

- Constructed berms shall be at a constant elevation and sloped away from the dike.
- Where dikes cross channels, ditches, borrow areas, streams, sloughs, swales, gullies, etc., they shall have a berm constructed on each side. The top elevation of these berms shall be at least 1 foot above the average ground surface on each side of the channel, ditch, borrow area, stream, slough, swales, gully, etc., and sloped away from the dike.
- The minimum top width of natural or constructed berms shall be as shown in Table 1.
- The minimum side slope ratio of constructed berms shall be 2:1 (Horizontal:Vertical).

Dike Materials. Manufactured materials are erosion resistant materials such as concrete, PVC, steel, or other material that provides the required structural strength and durability for the dike. Dikes constructed of manufactured materials shall have a structural analysis completed for the various loads the dike will be subjected to during its life. These include hydrostatic, ice, uplift, earth, and equipment.

The dike shall be analyzed for stability using acceptable safety factors for each loading condition.

Earth dike materials shall be obtained from required excavations and designated borrow areas. The selection, blending, routing, and disposition of materials in the various fills shall be subject to approval by the engineer or designer. Fill materials shall contain no frozen soil, sod, brush, roots, or other perishable materials. Rock particles larger than the maximum size specified for each type of fill shall be removed prior to placement and compaction of the fill. The types of materials used in the various fills shall be as listed and described in the specifications and drawings.

Embankment and Foundation Seepage. Embankment and foundation drainage and seepage control shall be designed on the basis of site investigation, laboratory data, seepage analysis, and stability analysis. The resulting design shall minimize seepage, prevent piping or undermining, and provide a stable embankment and foundation.

An analysis is required on all Class I dikes that have a height of six (6) feet or greater and Class II dikes that have a height of eight (8) feet or greater.

In the absence of more detailed data and analysis, the following criteria for a foundation cutoff apply for Class I dikes less than 6 feet in height, Class II dikes less than 8 feet in height and Class III dikes:

- Minimum of H feet deep for H<3 feet.
- Minimum of 3 feet deep for H≥3 feet.
- Minimum of 4 feet bottom width.
- 1:1 or flatter side slopes.

A stream, channel, ditch, borrow area, slough, swale, gully, etc. shall be far enough away from the dike so that the extension of a line drawn from the design high water elevation on one side of the dike to the dike toe on the opposite side shall not intersect any stream, channel, etc. (See figure 1). This line criterion applies to both sides of the dike. This criterion will minimize the hazard to the dike caused by piping through the foundation.

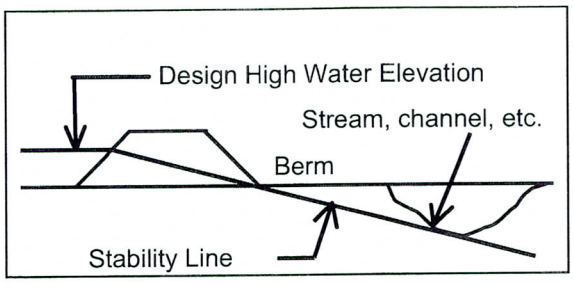


Figure 1

Interior Drainage. Dikes to prevent flooding shall be provided with interior drainage systems for the area being protected. The interior drainage system shall prevent flood damage to the interior area from a flood of the design frequency in Table 1 for both the 1-day and the 10-day storm duration. The interior drainage system may include storage areas, gravity outlets, and pumping plants as needed to provide the required level of flood protection.

Pipes. Pipes installed through a Class I dike below the design high water with a dike height greater than 12 feet shall meet the requirements for PRINCIPAL SPILLWAYS as found in NRCS TECHNICAL RELEASE 60 – Earth Dams and Reservoirs, except for the minimum size requirements.

Pipes through all other dikes shall meet the requirements for a principal spillway in NRCS Conservation Practice Standard, Ponds (378).

Dikes shall be protected from scour at pipe inlet and outlet locations by appropriate measures. A pump discharge pipe through a dike shall be installed above design high water, if feasible. Pump discharge pipes shall be equipped with a flexible connection or similar coupling to prevent vibration of the pumping plant being transmitted to the discharge pipe.

Slope Protection. Slopes of earthen dikes shall be protected from sheet, rill, and gully erosion; erosion from flowing floodwaters; and wave action created by wind and/or boat traffic. Erosion protection measures such as non-woody vegetation, berms, rock riprap, sand-gravel, or soil cement shall be utilized as needed.

Regulatory Requirements. Dikes shall meet the requirements of all federal, state, and local laws or regulations.

CONSIDERATIONS

Flood of Record. For Class I dikes, the flood of record should be considered when establishing the top of dike elevation.

Location. When locating the site for the dike, consider the foundation soils, property lines, setbacks from property lines, exposure to open water, distance to streambanks, availability of outlets by gravity or pumping, buried, utilities, cultural resources, and natural resources such as wetlands, natural areas, and fish and wildlife habitat.

Fluvial geomorphologic concepts contained in National Engineering Handbook (NEH) Part 653, Stream Corridor Restoration Principles, Processes and Practices should be considered when placing a dike near a stream.

Berms. Give special consideration to wider berms, additional setbacks, or protecting the berm side slope when adjacent to actively eroding or moving streams to protect the dike for its design life.

Adverse Impacts. Adverse environmental impacts from the proposed dike will be evaluated. Any increases in flood stage caused by dike-induced flow restrictions will be evaluated for adverse impacts to unprotected areas. Adverse impacts should be minimized.

DRAWINGS AND SPECIFICATIONS

Drawings and specifications shall be prepared in accordance with the criteria of this standard and shall describe the requirements for applying the practice to achieve its intended purpose.

OPERATION AND MAINTENANCE

Operation and maintenance requirements for all dikes will be provided to the landowners. For Class I dikes with a height greater than 12 feet, an emergency action plan meeting the requirements of 500.70 of the National Operation and Maintenance Manual shall be completed prior to construction of the dike. For Class I and Class II dikes, a detailed written Operation and Maintenance Plan in accordance with 500.40 through 500.42 of the National Operation and Maintenance Manual shall be completed and provided to the owner.

Table 1 – Minimum Design Criteria for Dikes

Classification	Material [1/]	Height (H) in Feet [2/]	Minimum Storm Design Frequency in Years	Minimum Freeboard in Feet	Minimum Top Width in Feet	Minimum Side Slope Ratio [3/] (H:V)	Berm Width in Feet
Class I	Earth	0 to 6	100	H/3	10	2:1	12
		>6 to 12	100	2	10	Note 4/	Note 4/
		>12 to 25	100	3	12	Note 4/	Note 4/
		>25	100	3	14	Note 4/	Note 4/
	Manufactured	0 to 8	100	H/4	N/A	N/A	Note 4/
		>8 to 12	100	2	N/A	N/A	Note 4/
		>12	100	3	N/A	N/A	Note 4/
Class II	Earth	0 to 6	25	H/3	6	2:1	12
		>6 to 12	25	2	8	2:1	15
	Manufactured	0 to 8	25	H/4	N/A	N/A	Note 4/
		>8 to 12	25	2	N/A	N/A	Note 4/
Class III	Mineral Soils	0 to 3	10	H/3	4	2:1	8
		>3 to 6	10	1	6	2:1	8
		>6 to 12	25	2	8	2:1	8
	Organic Soils [5/]	0 to 2	10	H/2	4	2:1	10
		>2 to 4	10	1	6	2:1	10
		>4 to 6	10	2	8	2:1	15

[1/] Earth includes rock. Manufactured materials are erosion resistant materials such as concrete, PVC and steel that provides the structural strength for the dike.

[2/] Height is the difference between normal ground elevation at the dike centerline and the design high water elevation. When determining normal ground elevation, exclude crossings of channels, sloughs, small low areas, small ridges, swales, or gullies.

[3/] Minimum side slope ratios are for compacted earth fill. Dumped earth fill without compaction will be flatter.

[4/] Side slope ratios and berm widths shall be determined by a stability analysis.

[5/] Organic soils are permitted only for Class III dikes 6 feet or less in height. Higher dike heights result in excessive settlement and decomposition.

USDA NATURAL RESOURCES CONSERVATION SERVICE
CONSERVATION PRACTICE STANDARD
ARIZONA

OPEN CHANNEL

(feet)
CODE 582

DEFINITION

Constructing or improving a channel either natural or artificial, in which water flows with a free surface.

PURPOSE

To provide discharge capacity required for flood prevention, drainage, other authorized water management purposes, or any combination of these purposes.

CONDITIONS WHERE PRACTICE APPLIES

This standard applies to construction of open channels or modifications of existing streams or ditches. Design criteria for channel stability and maintenance of Floodwater Diversions (400), Floodways (404), or Surface Drainage, Main or Lateral (608), having a drainage area in excess of 1 mi^2 (1.6 km^2) shall be in accord with this standard for open channels. It does not apply to Diversions (362), Grassed Waterways (412), Irrigation Field Ditches (388), Surface Drainage, Field Ditches (607), or Irrigation Canals or Laterals (320).

This standard applies to all earth channel construction or modification except as noted under "Scope."

It also applies where stability requirements can be met, where the impact of the proposed construction on water quality, fish and wildlife habitat, forest resources, and quality of the landscape is evaluated and the techniques and

Conservation practice standards are reviewed periodically, and updated if needed. To obtain the current version of this standard, contact the Natural Resources Conservation Service.

measures necessary to overcome the undesirable effects are made part of any planned work, where an adequate outlet for the modified channel reach is available for discharge by gravity flow or pumping, and where excavation or other channel work does not cause significant erosion, flooding, or sedimentation.

CRITERIA

Plan. Channel construction or modification shall be according to an approved plan prepared for the site. TR-25 shall be used in surveys, planning, and site investigations for channel work. Design criteria in TR-25 shall be followed, using the procedure best adapted to site conditions.

In selecting the location and design of channels, careful consideration shall be given to minimizing water pollution, damage to fish and wildlife habitat, and to protecting forest resources and the quality of the landscape. In considering requirements for construction and operation and maintenance, selected woody plants must be preserved. The overall landscape character, prominent views, and fish and wildlife habitat requirements must be considered.

Planned measures necessary to mitigate unavoidable losses to fish or wildlife habitat shall be included in the project. The quality of the landscape shall be maintained by both the location of channel works and plantings, as appropriate.

The alinement of channels undergoing modifications shall not be changed to the extent that the stability of the channel or laterals thereto is endangered.

Capacity. The capacity for open channels shall be determined according to procedures applicable to the purposes to be served and according to related engineering standards and guidelines in handbooks. The water surface profile or hydraulic gradeline for design flow shall be determined according to guidelines for hydraulic design in TR-25. The n value for aged channels shall be based on the expected vegetation, along with other retardance factors, considering the level of maintenance prescribed in the operation and maintenance plan prepared with the owners or sponsors. The required capacity may be established by considering volume-duration removal rates, peak flow, or a combination of the two, as determined by the topography, purpose of the channel, desired level of protection, and economic feasibility.

Cross section. The required channel cross section and grade shall be determined by the plan objectives, the design capacity, the materials in which the channel is to be constructed, the vegetative establishment program, and the requirements for operation and maintenance. A minimum depth may be required to provide adequate outlets for subsurface drains, tributary ditches, or streams. Urban and other high-value developments through which the channel is to be constructed must be considered in the design of the channel section.

Channel stability. Characteristics of a stable channel are:

1. The channel neither aggrades nor degrades beyond tolerable limits.
2. The channel banks do not erode to the extent that the channel cross section is changed appreciably.
3. Excessive sediment bars do not develop.
4. Gullies do not form or enlarge because of the entry of uncontrolled surface flow to the channel.

All channel construction and modification (including clearing and snagging) shall be according to a design that can be expected to result in a stable channel that can be maintained at reasonable cost. Vegetation, riprap, revetments, linings, structures, or other measures shall be used if necessary to insure stability.

The method applicable to site channel in TR-25 shall be used in determining the stability of proposed channel improvements.

Bankfull flow is the flow in a channel that creates a water surface at or near the normal ground elevation, or the tops of dikes or continuous spoil banks that confine the flow for a significant length of a channel reach.

Channels must be stable under conditions existing immediately after construction (as-built condition) and under conditions existing during effective design life (aged condition). Channel stability shall be determined for discharges under these conditions as follows:

1. As-built condition – Bankfull flow, design discharge, or 10-year frequency flow, whichever is smallest, but not less than 50 percent of design discharge.

   The allowable as-built velocity (regardless of type of stability analysis) in the newly constructed channel may be increased by a maximum of 20 percent if:

- The soil and site in which the channel is to be constructed are suitable for rapid establishment and support of erosion-controlling vegetation,
- Species of erosion-controlling vegetation adapted to the area and proven methods of establishment are known, and
- The channel design includes detailed plans for establishing vegetation on the channel side slopes.

2. Aged condition – Bankfull flow or design discharge, whichever is larger, except that it is not necessary to check stability for discharge greater than the 100-year frequency.

Stability checks that are flow related are not required if the velocity is 2 ft/s (0.6 m/s) or less.

For newly constructed channels is fine-grained soils and sands, the n values shall be determined according to procedures in chapter 6 of TR-25, and shall not exceed 0.025. The n value for channels to be modified by clearing and snagging only shall be determined by reaches according to the expected channel condition upon completion of the work.

Appurtenant structures. The channel design shall include all structures required for proper functioning of the channel and its laterals, as well as travelways for operation and maintenance. Inlets and structures needed for entry of surface and subsurface flow into channels without significant erosion or degradation shall be included in the channel design. The design also shall provide for necessary flood gates, water-level-control devices, bays used in connection with pumping plants, and any other appurtenances essential to the functioning of channels and contributing to attainment of the purposes for which they are built. If needed, protective structures or treatment shall be used at junctions between channels to insure stability at these critical locations.

The effect of channel work on existing culverts, bridges, buried cables, pipelines, irrigation flumes, and inlet structures for surface and subsurface drainage on the channel and laterals thereto shall be evaluated to determine the need for modification or replacement.

Culverts and bridges that are modified or added as part of channel projects shall meet reasonable standards for the type of structure and shall have a minimum capacity equal to the design discharge of state agency design requirements, whichever is greater. Capacity of some culverts and bridges may need to be increased above the design discharge.

Disposition of spoil. Spoil material from clearing, grubbing, and channel excavation shall be disposed of in a manner that will:

1. Not confine or direct flows so as to cause instability when the discharge is greater than the bankfull flow.
2. Provide for the free flow of water between the channel and flood plain unless the valley routing and water surface profile are based on continuous dikes being installed.
3. Not hinder the development of travelways for maintenance.
4. Leave the right-of-way in the best condition feasible, consistent with the project purposes and adjacent land uses.
5. Direct water accumulating on or behind spoil areas to protected outlets.
6. Maintain or improve the visual quality of the site to the extent feasible.

Vegetation of channel. Vegetation shall be established on all channel slopes, berms, spoil, and other disturbed areas according to the NRCS standard for Channel Vegetation (322).

Cultural Resources

If this practice involves soil disturbance, the area of potential effect for each undertaking must be investigated for cultural resources under section 106 of the National Historical Preservation Act of 1966, as amended, before soil disturbance occurs. See the NRCS Arizona Handbook of Cultural Resources Procedures - Applicability and Exceptions Section - for

identification of practices that are exempt from, or that require cultural resources surveys.

Endangered Species

Determine if installation of this practice with any others proposed practice will affect any federal, tribal, or state listed Threatened or Endangered species or their habitat. NRCS's objective is to benefit these species or at least not have any adverse effect on a listed species. If the Environmental Evaluation indicates the action may adversely affect a listed species or result in adverse modification of habitat of listed species which has been determined to be critical habitat, NRCS will advise the land user of the requirements of the Endangered Species Act and recommend alternative conservation treatments that avoid the adverse effects.

Further assistance will be provided only if the landowner selects one of the alternative conservation treatments for installation; or at the request of the landowners, NRCS may initiate consultation with the U.S. Fish and Wildlife Service. If the Environmental Evaluation indicates the action will not affect a listed species or result in adverse modification of critical habitat, consultation generally will not apply and usually would not be initiated. Document any special considerations for endangered species in the Practice Requirements Worksheet.

CONSIDERATIONS

Water Quantity

- Effects on components of the water budget, especially on volumes and rates of runoff and infiltration.

Water Quality

- Effects of erosion and the movement of sediment and soluble and sediment-attached substances in runoff during and immediately after construction.
- Effects of the use of chemicals during vegetation control.
- Effects of changes in channel vegetation on downstream water temperature.
- Potential for temporary and long-term effects on the visual quality of downstream waters.

PLANS AND SPECIFICATIONS

Plans and specifications for constructing open channels shall be in keeping with this standard and shall describe the requirements for properly installing the practice to achieve its intended purpose.

OPERATION AND MAINTENANCE

Plan. An operation and maintenance plan must be prepared for each channel system. Minimum requirements for operation, maintenance, and replacement shall be consistent with the design objectives. This includes consideration of fish and wildlife habitat, quality of the landscape, water quality, mitigation features, methods, equipment, costs, stability, function for design life, frequency, and time of year for accomplishing the work. Detailed provisions for operation and maintenance must be made if complex features, such as water-level-control structures and pumping plants, are required.

Maintenance access. Travelways for maintenance generally shall be provided as part of all channel work. This requirement may be met by providing reading access points to sections of the channel if this will permit adequate maintenance in conformance with the operation and maintenance plan.

A travelway shall be provided on each side of large channels if necessary for use of maintenance equipment. Travelways must be adequate for movement and operation of equipment required for maintenance of the channel. The travelway may be located adjacent to the channel on a berm or on the spread spoil. In some places the channel itself may be used as the travelway. The travelway, including access points, must blend into the topography, the landscape, and adjacent land uses.

Safety. Open channels can create a safety hazard. Appropriate safety features and devices should be installed to protect people and animals from accidents such as falling or drowning.

USDA NATURAL RESOURCES CONSERVATION SERVICE
CONSERVATION PRACTICE SPECIFICATION
ARIZONA

OPEN CHANNEL

(acre)
CODE NO. 582

1. SCOPE

The work shall consist of all preparation, excavation and spoil disposal as required to construct the channel.

2. GENERAL REQUIREMENTS

A. Site Preparation

All trees, logs, brush and stumps shall be completely burned or removed from the site and disposed of in locations where floodwater will not return it to the channel.

B. Excavation

The completed channel shall be constructed to the alignment, grades and dimensions as shown in the plans within the following tolerances:

(1) The alignment is uniform and as planned with no projections along the bank to deflect the flow.

(2) The bottom and top widths of the channel improvement are not more than 1-foot less than the designed width.

(3) At least ½ of the elevations checked across the bottom of the channel at the cross-section likely to pass the design discharge shall be equal to or lower than the planned elevation and no elevation more than 0.2 foot above grade will be permitted.

C. Spoil

The spoil shall be removed from the channel and placed so that sloughing or interference to flood flows will not occur. The spoil shall be placed or spread as required by the plans.

3. SPECIAL REQUIREMENTS

Installation shall be in accordance with the following drawings, specifications and special requirements. NO CHANGES ARE TO BE MADE IN THE DRAWINGS OR SPECIFICATIONS WITHOUT PRIOR APPROVAL OF NRCS.

Other Requirements

4. REQUIRED ATTACHMENTS

☐ Plan Map showing location

☐ Designs showing alignment, width, side slopes, drainage, erosion control, surfacing, traffic safety, and construction operations.

Drawings, No.

5. OTHER ATTACHMENTS

☐ Associated Practice Specifications _____

☐ Water Quality Considerations

☐ Other _____

6. OPERATION AND MAINTENANCE

This conservation practice is an asset to your farm or ranch. This practice will need periodic operation and maintenance to maintain satisfactory performance. The life of this practice or system is at least 10 years. The life of this practice can be assured or extended by thorough and timely operation and maintenance. Here are some recommendations to help you develop a good operation and maintenance program.

GENERAL RECOMMENDATIONS

☐ Maintain the roadway surface in a good condition, which includes periodic grading or repair of the surface.

☐ Prevent surface ponding by grading to remove depressions.

☐ Limit livestock usage to periods that permit use without damage.

☐ If fences are installed, they shall be maintained to provide warning and/or prevent unauthorized human or livestock entry.

- ☐ Remove debris to prevent blockage of stream crossings, culverts or bridges.
- ☐ Control all rodents or burrowing animals. Immediately repair any damage caused by their activity.
- ☐ Immediately repair any damage from vandalism, vehicles, or livestock.

Specific Recommendations For Your Installation

_____

STATEMENT OF WORK
Open Channel (582)
Arizona (5/17/04)

These deliverables apply to this individual practice. For other planned practice deliverables refer to those specific Statements of Work.

DESIGN

Deliverables:

1. Design documentation that will demonstrate that the criteria in NRCS practice standard have been met and are compatible with other planned and applied practices.
    a. Practice purpose(s) as identified in the conservation plan.
    b. List of required permits to be obtained by the client.
    c. Impacts on adjacent properties and structures.
    d. Compliance with NRCS national and state utility safety policy (NEM Part 503-Safety, Subpart A - Engineering Activities Affecting Utilities 503.00 through 503.06).
    e. Practice standard criteria related computations and analyses to develop plans and specifications including but not limited to:
        i. Hydrology/Hydraulics and Appurtenance Design
        ii. Channel Stability
        iii. Vegetation and Erosion Control
2. Written plans and specifications including sketches and drawings shall be provided to the client that adequately describes the requirements to install the practice and obtain necessary permits.
3. Design Report and Inspection Plan as appropriate (NEM Part 511, Subpart B Documentation, 511.11 and Part 512, Subpart D Quality Assurance Activities, 512.30 through 512.32).
4. Operation and Maintenance Plan
5. Certifications that the design meets practice standard criteria and comply with applicable laws and regulations (NEM Subpart A, 505.03(b)(2)).
6. Design modifications during installation as required.

INSTALLATION

Deliverables

1. Pre Installation conference with client and contractor.
2. Verification that client has obtained required permits.
3. Staking and layout according to plans and specifications including applicable layout notes.
4. Installation inspection (according to inspection plan as appropriate).
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    b. Inspection records
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CHECK OUT

Deliverables

1. As-Built documentation.
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    b. Drawings
    c. Final quantities
2. Certification that the installation meets NRCS standards and specifications and is in compliance with permits (NEM Subpart A, 505.03(c)(1)).
3. Progress reporting.

REFERENCES

- NRCS Field Office Technical Guide (eFOTG), Section IV, Conservation Practice Standard - Open Channel, 580.
- NRCS National Engineering Manual (NEM).
- NRCS National Environmental Compliance Handbook
- NRCS Cultural Resources Handbook

State Contact: Design Engineer