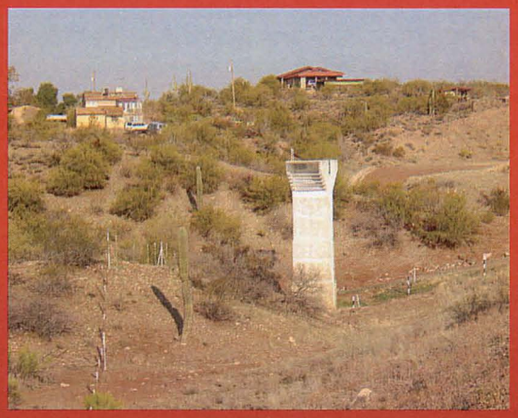

Sunnycove Flood Retarding Structure Individual Structures Assessment

Prepared for
Flood Control District of Maricopa County
On-Call Phase I Assessment
FCD 2003C015
PCN 050.36.31
June 2004

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KIMLEY-HORN AND ASSOCIATES, INC.
7878 NORTH 16TH STREET, SUITE 300
PHOENIX, ARIZONA 85020
(602) 944-5500

June 2004

Registered Professional Engineer (Civil)
CERTIFICATE NO. 24767
ROBERT A. EICHINGER
DATE SIGNED 7-4-2004
ARIZONA, U.S.A.

Kimley-Horn and Associates, Inc.

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Registered Professional Engineer (Civil) CERTIFICATE NO. 24767 ROBERT A. EICHINGER DATE SIGNED July 2 2008 ARIZONA U.S.A.

INDIVIDUAL STRUCTURE ASSESSMENT for SUNNYCOVE FLOOD RETARDING STRUCTURE

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1.0 EXECUTIVE SUMMARY

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

1.1 Dam Description

Sunnycove FRS is located approximately one mile southwest of the center of the Town of Wickenburg, Arizona. The Sunnycove FRS is located on Sunnycove Wash, a tributary to the Hassayampa River in the southeast corner of Section 11, Township 7 North, and Range 5 West. The project consists of the FRS embankment, principal spillway, and an earth-lined emergency spillway.

The reservoir behind the FRS is 18 acres with a capacity of 219 acre-feet (AWC, 1979). A permanent pool is not retained in the reservoir. The FRS and reservoir are designed to detain the 100-year floodwater and store the impoundment for a slow release of 10 days (AWC, 1979) [AWC is the Arizona Water Commission which is now the Arizona Department of Water Resources, ADWR, AWC is being used for reference purposes]. Reservoir capacity is then restored to handle future floods.

The emergency spillway is an earth-lined channel located through a saddle adjacent to the left abutment of the dam. Construction of the FRS and appurtenant structures was completed by the Soil Conservation Service in September 1976.

Sunnycove FRS is an earthfill structure with a central chimney drain/filter. The length of the FRS is 714 feet with a maximum height of 48.5 feet and crest width of 14 feet (AWC, 1979). The reservoir capacity is approximately 219 acre-feet at spillway crest. The FRS was designed with 18.2 acre feet of sediment storage (100-year). Sunnycove FRS is

located in the Town of Wickenburg, Arizona south of U.S. 60 and west of Kellis Road. The maximum recorded impoundment for Sunnycove FRS is 53 acre-feet with a stage of 21.68 feet at the FRS (August 22, 1992) (ADWR; 2004b).

Watershed

The dam was constructed across Sunnycove Wash, a tributary to the Hassayampa River. The drainage area contributing to the dam is 1.35 square miles (AWC, 1979). The sediment yield rate was determined to be 0.14 acre-feet per square mile per year delivered to the reservoir. With a trap efficiency of 95 percent, the sediment storage requirement is 18.2 acre-feet for the 100-year life of the structure (SCS, 1974).

Flood Pool

The emergency spillway crest was constructed at an elevation of 2170 feet (NGVD29) in order to contain the routed 100-year storm event. The peak inflow into the reservoir during the 100-year flood and PMF were estimated to be 1486 cfs (SCS, 1975) and 7709 cfs (AWC, 1979), respectively. The design allowed for 219 acre feet of storage to the emergency spillway crest. The 100-year flood pool is designed to drain in less than 10 days.

Dam Embankment

The dam is a zoned-earthfill dam with a central vertical chimney drain and blanket drain for outfall purposes (along the downstream portion of the outlet pipe). The embankment was constructed with 3:1 (horizontal to vertical) upstream slope and a 2:1 downstream slope. The as-built top of dam crest elevation is 2178.5. The minimum settled top of dam crest elevation (based on survey monuments) is 2178.7 ft.

Zone I, the core of the dam, was constructed of clayey sands and silty sands with some gravelly clayey sand. The fine contents of the chimney drain, Zone II, were limited to less than 5 percent. The shell (Zone III) consists of gravelly sands to sandy gravel. The Zone IV transition was similar to Zone III except that it contains more fines. All embankment material was obtained locally except for Zone II. See Appendix A sheet 4R for typical cross section of the Dam.

Principal Outlet Works

The principal spillway structure consists of several elements: intake tower, trash rack, low level gated intake, and an 8-inch by 8-inch orifice at elevation 2148.0-ft, spillway weir, principal spillway conduit, and an outfall manhole connecting the outlet to the downstream pipeline system. The reinforced concrete intake tower includes an uncontrolled overflow weir at elevation 2169.5 feet. Note that this elevation is 0.5-ft below the crest elevation of the emergency spillway (1270.0-ft). Under normal operating conditions the outlet conduit will not flow full as it is controlled by an 18-inch slide gate on the intake tower until a flood great than the 100-year storm occurs.

The 8-inch by 8-inch orifice was designed to drawdown the 100-year reservoir pool in less than 10-days to an elevation of 2148.0-ft.

Discharge through the 30-inch diameter reinforced concrete pipe principal spillway conduit enters into a discharge manhole at the downstream toe of the dam. From the manhole the flow is conveyed through the Sunset-Sunnycove Pipeline to an outfall at the Hassayampa River, approximately 1.5 miles away.

Emergency Spillway

The emergency spillway was designed to pass the PMF with 0.7 ft of freeboard with a spillway crest elevation of 2170 ft. The emergency spillway discharge capacity is 6,300 cfs. The emergency spillway is a 100- foot wide, uncontrolled earth-lined channel constructed through a narrow ridge located 200 feet to the left of the left abutment of the dam. The channel conveys floodwater to a discharge point several hundred feet below the downstream toe of the dam. The in-place material, firm fanglomerate, is adequate to resist serious erosion for the infrequent spills that may occur (AWC, 1979).

1.2 Hydrologic and Hydraulic Considerations

The Watershed Work Plan – Wickenburg Watershed was prepared by the NRCS (NRCS, 1974). The structural elements of the watershed project include two flood retarding structures, Sunset and Sunnycove. The two flood retarding structures capture and impound stormwater from their respective upstream watersheds. Discharge from the principal spillways of the Sunnycove FRS and Sunset FRS flow in a common outfall pipeline which ultimately discharges into an outlet structure located in the Hassayampa River. The NRCS designed the Sunnycove FRS to detain the 100-yr runoff volume

The principal spillway hydrograph (PSH) is the hydrograph used to determine the minimum crest elevation of the emergency spillway. It is used to establish the principal spillway capacity and determine the associated minimum floodwater retarding storage. For a Class C structure, the PSH is based on the one hundred-year precipitation (P_{100}).

The emergency spillway hydrograph (ESH) is the hydrograph used to establish the dimensions of the emergency spillway. For a Class C hazard structure, the ESH is based on a watershed precipitation depth according to the following formula: $\{P_{100} + 0.26*(PMP - P_{100})\}$. The freeboard hydrograph (FBH) is the hydrograph used to establish the minimum settled elevation of the top of the dam. It is also used to evaluate the structural integrity of the spillway system. For a Class C hazard structure, the FBH is based on a watershed precipitation depth for the probable maximum precipitation (PMP).

The U.S. Army Corps of Engineer conducted a Phase I Inspection Report as part of the National Dam Safety Program (Corps, January 1979). The Arizona Water Commission (now the Arizona Department of Water Resources) prepared the study on behalf of the Corps.

The Phase I study developed a local six-hour Probable Maximum Storm using the procedures outlined in HMR-49, "Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages". This storm was routed through the structure using the SCS watershed modeling program "TR-20". The routing was performed with an initial

water surface elevation at the spillway crest elevation. The study concluded that the spillway is capable of passing 100% of the PMF with 0.7 feet of residual freeboard.

The Phase I study used a drainage area of 1.35 square miles which agrees with the SCS determination of contributing watershed area. The Corps study estimated a maximum spillway discharge of 8,250 cfs. It is not clear from the Phase I report how this quantity was derived for the spillway.

In May 1994 the Flood Control District of Maricopa County completed the Wickenburg Area Drainage Master Study (ADMS). As part of this study the District conducted a hydrologic analysis for various streams within the ADMS. The study included the Sunnycove Wash drainage area and Sunnycove FRS. The hydrologic study included a routing routine to route the 100-year flows through the Sunnycove FRS. The study assumed that the principal spillway was the lower gated outlet (incorrect assumption) located at the inlet tower and did not account for the low stage orifice. As a result the study assumed no outflow from the principal spillway for the purposes of hydrologic routing. The study results indicate an inflow into the dam of 1232 cfs for the 100-year 24-hour storm and no discharges through the emergency spillway. The results show that an impoundment occurs and produces a water surface elevation in the pool to 2162.8 ft, which is approximately 7.2 ft below the emergency spillway elevation.

It is interesting to compare the ADMS results to the SCS hydrology results for the 100-year storm event. The District study used the 24-hour duration while the SCS hydrology was based on the 6-hour duration. Rainfall loss parameters were based on the Green-Amp methodology and Curve Numbers for the District and SCS studies, respectively. The 100-year 24-hour inflow into the structure for the District study was 1232 cfs with no outflow from the dam. The 100-year 6-hour inflow into the structure for the SCS study was 1486 cfs with outflow in the principal spillway.

1.3 Geologic and Geotechnical Considerations

Geologic Setting. Sunnycove FRS is located in hilly terrain within the northeast-central portion of the Sonoran Desert section of the Basin and Range Physiographic Province near its boundary with the Arizona Transition Zone Section. The latitude and longitude of the center part of the structure is approximately 33° 57' 27" N and 112° 44' 18" W based on NAD 27 datum. This portion of the Basin and Range is characterized by broad alluvial fans that are locally dissected and gently sloping connected valleys bounded by high, rugged northwest, north, and northeast trending mountains including the Date Creek and Weaver Mountains to the north, the Vulture Mountains to the south and the Wickenburg Mountains to the east that rise abruptly to form broad, elongated, deep, sediment-filled valleys produced by block faulting and folding during past episodes of mountain/basin bounding fault movements. The dam is within the city limits of Wickenburg, Arizona off the northeastern flank of the Vulture Mountains in the southeast quarter of Section 11, Township 7 North, Range 5 West.

Seismicity. No seismicity or earthquake evaluation was conducted for the Sunnycove FRS dam design based on a review of the project files. However, a seismicity evaluation for all of the FCDMC dam structures was conducted in 2002. The report entitled "Seismic Exposure Evaluation, Dam Safety Program, Flood Control District of Maricopa County" describes the various seismotectonic zones, fault zones, design earthquake, and characteristic ground motion affecting FCDMC structures (AMEC, 2002).

Land Subsidence. No unconsolidated, compressible basin fill soils are believed to be present beneath the Sunnycove FRS. The subsurface geological conditions in the embankment dam area consists of relatively hard, cemented Tertiary age fanglomerate (at the surface) and in the subsurface deposited on crystalline bedrock indicate the potential for land subsidence due to groundwater withdrawal does not exist at the Sunnycove FRS site.

According to Staedicke (1995) because there is no history of extensive groundwater pumping or subsidence, the NRCS has never surveyed the Sunnycove FRS structure. Although land subsidence is not expected to affect Sunnycove FRS, Kimley-Horn recommends that the structure be periodically surveyed. The Flood Control District of Maricopa County surveyed the structure in 2003 and 2004. Appendix B contains the current surveys for the Sunnycove FRS.

Earth Fissures. No earth fissures, related to land subsidence, are documented nor reported as occurring within the Sunnycove FRS project area. Geological conditions in the Sunnycove FRS area preclude the development of earth fissures at this site.

Foundation Conditions. The dam is founded on competent, generally well-cemented fanglomerate bedrock which is generally free of jointing. Recent alluvial and colluvial deposits, and weathered bedrock overlying the unweathered bedrock (fanglomerate) were removed as part of foundation preparation. A high gravel terrace deposit was identified in the upper left abutment on the original dam alignment. The terrace deposit consisted of uncemented, silty gravels and sands containing some very loose pockets of material. The terrace deposit was recognized as a potential seepage problem during the design process, and the dam alignment was shifted upstream to avoid founding the left abutment on these materials. The cutoff trench was excavated into competent fanglomerate in the upper left abutment along the revised alignment.

Foundation Surface Irregularities and Embankment Settlement. As-built plans indicate that the excavated cutoff trench was shaped to a smooth profile. However, potential foundation bedrock surface irregularities are indicated on the drawings along the dam axis outside of the cutoff trench. In particular, a bedrock ridge is evident crossing the dam alignment on the lower left valley section. The ridge spur can be seen upstream from the dam. The ridge extends upstream to downstream (east to west) across the dam axis, and is skewed to the NW-SE oriented axis of the dam. The ridge crosses the centerline of the dam at about Station 14+30 near the outlet works alignment. The as-built drawings indicate that the ridge was substantially removed and shaped. However, a trapezoidal trench was cut through the remnant, shaped ridge to install the outlet pipe.

The most notable foundation irregularity is left of the principal outlet conduit where as-built plans show a high point in the foundation. Based on the profile on dam centerline (Sheet No. 3 in the as-built drawing set), the trench cut slope was approximately 2H:1V and about 6.5 feet high at dam centerline. Another subtle bedrock ridge is evident at the base of right abutment, crossing the dam axis at centerline near Station 12+00. These subtle foundation surface irregularities occur under the deeper sections of the embankment, and are not considered to be problematic with regard to transverse cracking. The mechanism for potential transverse crack formation is tension cracking over the irregularity due to differential settlement of the embankment locally. Monitoring for transverse tension cracking in the dam crest above these foundation irregularities has been recommended as part of the FMEA.

Embankment Materials. The dam was constructed as a zoned embankment. The materials used to construct Zones I, III, and IV were derived from local borrow sources consisting of alluvial deposits in the vicinity of the dam. The Geologic Report (SCS, 1974) provides logs for test pits and borings, and laboratory test results for bulk samples obtained from two designated borrow areas: Borrow Area 1 located within the limits of the sediment pool, and Borrow Area 2 located within the floodplain downstream from the dam. Materials in the borrow areas consisted primarily of silty sand (SM), clayey sand (SC), and clean to slightly silty sand (SP, SW-SM, SP-SM). Materials in Borrow Area 2 appeared to have higher fines content, and this area was designated as the primary source for the core section (Zone I). Supplemental borrow investigations were also performed in two tributary washes upstream from the dam, and in the main Sunnycove wash above the sediment pool elevation. The main purpose for these supplemental investigations was to locate additional quantities of granular materials for Zone III.

Original Slope Stability Analyses. The Design Report (SCS, 1975) indicated that the factor of safety of 1.25 for the 2H:1V downstream slope was acceptable because the Zone III outer shell material was anticipated to have a higher shear strength than the $\varphi = 32^\circ$, c=0 strength assumption used in the analysis. The shear strength assumption was reported to be conservative because the materials specified for Zone III would have lower fines contents (<10%) than the materials tested in the laboratory. However, based on the laboratory tests, the materials with higher fines contents actually exhibited higher shear strengths. The shear strength assumption of $\varphi = 32^\circ$, c=0 is considered to be reasonable and conservative for the broadly graded Zone III materials, but not for the reason stated in the Design Report.

The design intent was for the core section to have higher fines content such that it served as the primary water barrier, and for Zone III to have lower fines content and provide strength and faster drainage to improve stability during drawdown. Ultimately, however, the gradational distinction between Zones I, III, and IV is subtle, and for practical purposes the differences in engineering behavior of these materials is small or negligible. The Specifications limited fines content in Zone III to 15% (revising the original Design Report assumption of 10% fines in Zone III) to accommodate materials from the available local borrow sources. This high fines content does not provide a "free draining" zone as was assumed in the stability analyses for rapid drawdown. Also, the

designers assumed full development of a phreatic line within Zones I and IV, and used this as primary justification for incorporating a drain zone (Zone II) in the dam. A more critical purpose for Zone II is actually as a filter to protect against internal erosion and piping.

1.4 Land Use

Sunnycove FRS is located in the Town of Wickenburg, Arizona. The surrounding land use is primarily single family residential, commercial, and office. The downstream residential area is almost fully developed. However, open space exists for further development.

1.5 Field Inspection

Sunnycove FRS is regularly inspected by the District and the Arizona Department of Water Resources. The Kimley-Horn team inspected the facility as part of the Phase I Assessment. None of these inspections identified conditions that indicated an imminent risk to the integrity of the structure. The structure is well maintained and appears to be in a satisfactory operable condition.

1.6 Failure Modes and Effects Analysis

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

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

- Adverse Consequences Resulting from Emergency Spillway Discharges During Major Rainfall Events (Category I).
- Potential for Cracking due to Foundation Irregularities – Seepage Erosion Through Crack. (Category II)

The potential failure modes range from a low likelihood of occurrence, high consequence to a high likelihood, medium consequence. None of the potential failure modes have a high likelihood, high consequence.

1.7 Recommendations

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

1. A spillway inundation study should be conducted for Sunnycove FRS following District methods for such studies.

2. An updated dambreak analysis and inundation mapping should be prepared for Sunnycove FRS.

3. Kimley-Horn suggests that the EAP flowchart percent levels be reduced and consolidated given consideration for the time to fill the impoundment during large event storms.

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

5. Probable Maximum Precipitation. Prepare PMP/PMF using 24-hr and 72-hour durations. Compare routings of these events to PMP 6-hr duration flood.

6. Monitoring. KHA recommends visual monitoring for transverse tension cracking in the dam crest near Stations 12+00 and 14+30.

7. Phase II Documentation of Slope Stability and Seepage Analyses for Main Dam. Adequate documentation could not be located of slope stability factors of safety for specified loading and design criteria that have been established by appropriate jurisdictional agencies. The original stability analysis and KHA's preliminary stability analyses do not completely document factors of safety for all the loading conditions that would need to be evaluated under current NRCS or ADWR criteria.

8. Rapid Drawdown Stability (upstream slope): Preliminary analyses were conducted as part of this Phase I study that simulated a plausible scenario for development of the seepage line into the dam under multiple temporary impoundment events, and to assess the upstream slope stability under normal drawdown conditions. These analyses show that it is very unlikely that a steady state phreatic line will develop in the Sunnycove FRS, assuming the outlet works is operational and is not clogged for sustained periods of time following a flood event. ADWR criteria require that an "instantaneous" drawdown analysis be performed. The ADWR guidance and rules were developed for water retention dams, and the criteria are interpreted to mean that rapid drawdown stability should be evaluated assuming that a steady state phreatic line has developed from the normal high reservoir pool elevation. It is not clear what is meant in the ADWR regulations by "instantaneous" drawdown, and this may be subject to interpretation. Additional analyses would be required to assess and document rapid drawdown factors of safety under these assumptions.

9. Downstream Slope Stability Under Steady Seepage: The minimum factors of safety that have been computed for the dry downstream slope do not achieve the minimum criteria of 1.5 (see Table 6 Appendix H). Gannett Fleming does not consider the infinite slope analysis that was done in the original design as representative of a "critical" failure scenario. Our preliminary analysis evaluated more substantial failure surfaces which resulted in a minimum factor of safety of

1.4. This factor of safety was computed using the original strength assumptions of $\varphi = 32^{\circ}$ and c=0. It is likely that this strength assumption is conservative, based on the results of three strength tests that were done on the original borrow materials.

10. Upstream slope stability under steady seepage, partial pool: The original analysis evaluated upstream slope stability under steady seepage for the maximum pool elevation only, resulting in a minimum factor of safety of 1.8. The ADWR criteria for partial pool conditions is intended for water retention dams, in which a steady state phreatic line may develop for intermediate pool elevations. The factor of safety may be lower for the intermediate pool conditions than the steady state condition under maximum pool. The analysis could be done to document the minimum partial pool factor of safety, under the scenario that the outlet works is clogged such that the steady state phreatic line develops.

11. Pseudo-static stability analysis (critical downstream slope section): Seismic stability analyses were not performed as part of the original design. Seismic stability under current design criteria should be documented.

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

2.0 DESCRIPTION OF DAM

The Sunnycove Flood Retarding Structure (FRS) is a structural plan element of the Watershed Work Plan for the Wickenburg Watershed, Maricopa and Yavapai Counties, Arizona. The Watershed Work Plan was prepared by the Natural Resources Conservation Service (NRCS; formerly the Soil Conservation Services, SCS) in December 1974. The Wickenburg watershed is in west central Arizona in Maricopa and Yavapai Counties between the Vulture and Date Creek Mountains. That part of the watershed within Maricopa County is in the Wickenburg Natural Resources Conservation District (NRCD), and that part within Yavapai County is in the Triangle NRCD. The total original watershed area is 100,000 acres (156 square miles) and includes the towns of Wickenburg and Congress.

2.1 Purpose of Dam

The Sunnycove FRS is one of two flood retarding structural measures designed and constructed under the Watershed Work Plan. The other flood retarding structure is the Sunset FRS. The purpose of the Sunnycove FRS is to provide flood and erosion control benefits for downstream developments (agriculture, commercial and urban areas). The Sunnycove FRS was designed to control runoff from the 100-year event.

2.2 Dam Location and Features

Sunnycove FRS is located approximately one mile southwest of the center of the Town of Wickenburg, Arizona. The Sunnycove FRS is located on Sunnycove Wash, a tributary to the Hassayampa River in the southeast corner of Section 11, Township 7 North, and Range 5 West. **Figure 1** provides a location map of Sunset FRS. The project consists of the FRS embankment, principal spillway, and an earth-lined emergency spillway.

The reservoir behind the FRS is 18 acres with a capacity of 219 acre-feet (SCS, 1974). A permanent pool is not retained in the reservoir. The FRS and reservoir are designed to detain the 100-year floodwater and store the impoundment for a slow release of 10 days (AWC, 1979). Reservoir capacity is then restored to handle future floods.

The emergency spillway is an earth-lined channel located through a saddle adjacent to the left abutment of the dam. Construction of the FRS and appurtenant structures was completed by the Soil Conservation Service in September 1976.

2.3 Physical Features

Sunnycove FRS is an earthfill structure with a central chimney drain/filter. The length of the FRS is 714 feet with a maximum height of 48.5 feet and crest width of 14 feet (AWC, 1979). The reservoir capacity is approximately 219 acre-feet at spillway crest. The FRS was designed with 18.2 acre feet of sediment storage (100-year). Sunnycove FRS is located in the Town of Wickenburg, Arizona south of U.S. 60 and west of Kellis Road. The maximum recorded impoundment for Sunnycove FRS is 53 acre-feet with a stage of

21.68 feet at the FRS (August 22, 1992) (ADWR; 2004b). **Table 1** provides a summary of the physical data for Sunnycove FRS.

Watershed

The dam was constructed across Sunnycove Wash, a tributary to the Hassayampa River. The drainage area contributing to the dam is 1.35 square miles (AWC, 1979). The sediment yield rate was determined to be 0.14 acre-feet per square mile per year delivered to the reservoir. With a trap efficiency of 95 percent, the sediment storage requirement is 18.2 acre-feet for the 100-year life of the structure (SCS, 1974).

Flood Pool

The emergency spillway crest was constructed at an elevation of 2170 feet (NGVD29) in order to contain the routed 100-year storm event. The peak inflow into the reservoir during the 100-year flood and PMF were estimated to be 1486 cfs (SCS;1975) and 7709 cfs (AWC, 1979), respectively. The design allowed for 218 acre feet of storage to the emergency spillway crest. The 100-year flood pool is designed to drain in less than 10 days.

Dam Embankment

The dam is a zoned-earthfill dam with a central vertical chimney drain and blanket drain for outfall purposes (along the downstream portion of the outlet pipe). The embankment was constructed with 3:1 (horizontal to vertical) upstream slope and a 2:1 downstream slope. The as-built top of dam crest elevation is 2178.5. The minimum settled top of dam crest elevation (based on survey monuments) is 2178.7 ft.

Zone I, the core of the dam, was constructed of clayey sands and silty sands with some gravelly clayey sand. The fine contents of the chimney drain, Zone II, were limited to less than 5 percent. The shell (Zone III) consists of gravelly sands to sandy gravel. The Zone IV transition was similar to Zone III except that it contains more fines. All embankment material was obtained locally except for Zone II.

Principal Outlet Works

The principal spillway structure consists of several elements: intake tower, trash rack, low level gated intake, and an 8-inch by 8-inch orifice at elevation 2148.0-ft, spillway weir, principal spillway conduit, and an outfall manhole connecting the outlet to the downstream pipeline system. The reinforced concrete intake tower includes an uncontrolled overflow weir at elevation 2169.5 feet. Note that this elevation if 0.5-ft below the crest elevation of the emergency spillway (1270.0-ft). Under normal operating conditions the outlet conduit will not flow full as it is controlled by an 18-inch slide gate on the intake tower until a flood great than the 100-year storm occurs.

The 8-inch by 8-inch orifice was designed to drawdown the 100-year reservoir pool in less than 10-days to an elevation of 2148.0-ft.

Discharge through the 30-inch diameter reinforced concrete pipe principal spillway conduit enters into a discharge manhole at the downstream toe of the dam. From the

manhole the flow is conveyed through the Sunset-Sunnycove Pipeline to an outfall at the Hassayampa River, approximately 1.5 miles away.

Emergency Spillway

The emergency spillway was designed to pass the PMF with 0.7 ft of freeboard with a spillway crest elevation of 2170 ft. The emergency spillway discharge capacity is 6,300 cfs. The emergency spillway is a 100- foot wide, uncontrolled earth-lined channel constructed through a narrow ridge located 200 feet to the left of the left abutment of the dam. The channel conveys floodwater to a discharge point several hundred feet below the downstream toe of the dam. The in-place material, firm fanglomerate, is adequate to resist serious erosion for the infrequent spills that may occur (AWC, 1979).

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 dam and its construction. Secondly, through this review the project assessment team became familiar with the structure, became familiar with the history of the structure, and acquainted the team with the basis of analysis and design. The review also provides for a review of original design criteria and design guidelines under which the dam was constructed. The report presents a discussion of the dam design criteria under which the dam was originally constructed versus the Arizona Department of Water Resources dam safety rules and regulations for jurisdictional dams.

This section of the report presents a review of the technical documentation for the structure. 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 purpose of the review of the technical documents is to assist in the engineering assessment of the structure. The technical document review, along with the field examinations and the failure mode and effects analysis (FMEA), provided a basis to evaluate the structure regarding operational adequacy, structural stability, and compliance with current dam safety rules and regulations.

The information and data reviewed in this assessment were collected from several sources/repositories. These repositories included the libraries and office files of the District, Natural Resources Conservation Service (NRCS), and ADWR, Office of Dam Safety. Kimley-Horn has prepared under separate cover, a data collection report, summarizing the information collected for Sunnycove FRS.

3.1 Dam Design Criteria

Sunnycove FRS was analyzed and designed by the NRCS in the late 1960's and early 1970's. The basis of design for the FRS was originally founded in the NRCS publication "Engineering Memorandum EM-27" which is the precursor manual to "Technical Release TR-60: Earth Dams and Reservoirs" the present NRCS design guideline for earth dams. The FRS has been analyzed and designed according to EM-27.

The basis of design for Sunnycove FRS was to provide a 100-year level of protection (NRCS, December 1974). This design event was used to size the principal spillway and reservoir volume. The hydrology for the emergency spillway design and freeboard design flood is discussed below in the Hydrology section following NRCS criteria. According to ADWR criteria, the Sunnycove FRS Inflow Design Flood (IDF) is the ½ probable maximum flood (PMF). The NRCS, in their hydrologic study of Sunnycove FRS, has designed the dam not to overtop during the passage of the freeboard hydrograph, which was based on the PMP/PMF (see below – Hydrology). **Table 2** provides a summary of the original NRCS design criteria (based on EM-27) and current

TR-60 criteria for the dam and compares these criteria with current ADWR dam safety rules and regulations for jurisdictional dams.

3.2 Dam Classification

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

The NRCS classifies Sunnycove FRS as a Class C structure. Class C structures are structures located where failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, main highways, or railroads. The Arizona Department of Water Resources rules and regulations for jurisdictional dams classifies Spook Hill FRS as a high hazard, small dam.

3.3 Hydrology and Hydraulic Review

3.3.1 Hydrology. The Watershed Work Plan – Wickenburg Watershed was prepared by the NRCS (NRCS, 1974). The structural elements of the watershed project include two flood retarding structures, Sunset and Sunnycove. The two flood retarding structures capture and impound stormwater from their respective upstream watersheds. Discharge from the principal spillways of the Sunnycove FRS and Sunset FRS flow in a common outfall pipeline which ultimately discharges into an outlet structure located in the Hassayampa River.

The NRCS designed the Sunnycove FRS to detain the 100-yr runoff volume calculated using the principles outlined in Chapter 21, National Engineering Handbook, Section 4. Rainfall amounts from the rain gage (at the time of original analysis) in the watershed were analyzed and found to be lower than the revised TP-40 map amounts for 24-hour duration storms. The design rainfall was determined by using the revised TP-40 map rainfall and ES-1020 sheet 5 of 5. Runoff curve numbers were calculated from the SCS soil and cover reconnaissance surveys using procedures outlined in Chapters 7, 8, and 9 of the National Engineering Handbook (NEH), Section 4.

Times of concentration were derived from stream channel hydraulics. Channel cross sections were taken at several locations and velocities computed. Procedures outlined in Chapter 15, NEH-4 were used.

The principal spillway hydrograph (PSH) is the hydrograph used to determine the minimum crest elevation of the emergency spillway. It is used to establish the principal spillway capacity and determine the associated minimum floodwater retarding storage. For a Class C structure, the PSH is based on the one hundred-year precipitation (P_{100}).

The Watershed Work Plan states "the Sunnycove principal spillway is designed to outlet into a 12-inch concrete pipeline and junctions with the Sunset pipeline utilizing an 18-

inch pipeline to carry all flows to the Hassayampa River. The pipelines were designed to drain the flood pool at the Sunset site in less than 10 days and the flood pool at the Sunnycove site in less than 17 days". It should be noted that the principal spillway pipe diameters for both dams were increased in a subsequent NRCS analysis.

The Watershed Work Plan was updated by the SCS in their Design Report for Sunnycove FRS (NRCS, April 1975). The Design Report indicates that the hydrologic data and sediment yields were prepared by SCS Arizona staff. The report states that a review of the preliminary hydraulic design of the principal spillway pipeline system indicated an error was made in computing the hydraulic gradient. Hence the next larger standard pipe was required to maintain the same depths of flow. This allowed the principal spillway peak design discharges to be increased from 5.5 cfs to 8.7 cfs for the Sunset FRS and from 6.0 cfs to 9.1 cfs for Sunnycove FRS.

The SCS computer program FW-HY2-1130F Principal Spillway Routing was used for development of design storms and flood routing through the reservoir. Several alternate runs were made to determine the orifice size and the minimum crest elevation of the emergency spillway crest which would restrict the 100-year discharge as outlined in the previous paragraph. Results of the principal spillway routings were for Sunnycove FRS:

			Rainfall – 100 Yr				
Weighted CN	T_C [hr]	DA [sq. mi]	1 day [in]	10 day [in]	Sediment Pool Elevation [ft]	Orifice Size	Emergency Spillway Crest [ft]
83	1.10	1.35	4.20	6.50	2148.0	8-in × 8-in	2168.6

The routings for Sunnycove FRS resulted in a minimum crest elevation 6.4 feet lower that that used in the Work Plan. This routing does not empty the flood pool in 10 days; about 33 acre-feet remains at elevation 2155. According to the Design Report the 30-inch diameter principal spillway conduits will not prime during the routed 100-year storm.

The emergency spillway hydrograph (ESH) is the hydrograph used to establish the dimensions of the emergency spillway. For a Class C hazard structure, the ESH is based on a watershed precipitation depth according to the following formula: $\{P_{100} + 0.26*(PMP - P_{100})\}$. The freeboard hydrograph (FBH) is the hydrograph used to establish the minimum settled elevation of the top of the dam. It is also used to evaluate the structural integrity of the spillway system. For a Class C hazard structure, the FBH is based on a watershed precipitation depth for the probable maximum precipitation (PMP).

The PT-HY11-1130F computer program was used to route the emergency spillway design and freeboard hydrographs. Rainfall distribution/duration/amounts were determined for the six-hour thunderstorm using a preliminary draft report prepared by the National Weather Service for Probable Maximum Thunderstorm Precipitation Estimates, Southwest States, dated August 1972.

Stage-discharge for the spillway was developed by the computer program using Case 1 (level inlet, n = 0.06) with various entrance lengths, side slopes and bottom width combinations. Routings of the freeboard and emergency spillway hydrograph was started at the 10-day drawdown elevation. Results are as follows:

				Rainfall		Freeboard	
Emergency Spillway Crest [ft]	10 day drawdown elevation [ft]	Bottom Width [ft]	Side slope	ESH [in]	FBH [in]	Peak [cfs]	Elev. [ft]
2170.0	2154.9	100	3:1	6.37	14.95	6374	2177.75

Principal spillway discharge was not included in the routing for Sunnycove FRS.

The Design Report provided a stage-storage table for Sunnycove FRS which is reproduced as follows:

Item	Elevation [Ft]	Area [Ac]	Sum Storage [Af]
Bottom of Pool	2132.1	0	0
Top of Sediment Pool	2148.0	3.2	18.2
Crest of Principal Spillway			
Low Stage	2148.0	3.2	18.2
High Stage	2170.0	18.0	218.7
Crest of Emergency Spillway	2170.0	18.0	218.7
Crest of Dam (w/o camber)	2178.5	21.5	375.0

The NRCS further updated the design hydraulics for Sunnycove in their Supplement No. 1 to Design Report Dated April 10, 1975 (NRCS, June 1975). In the Supplement the NRCS stated that only minor changes in hydraulic design were made in final design of the Sunnycove FRS and Outlet Pipeline.

The stage-storage for the emergency spillway was developed by the PT-HY11-1130F computer program using Case 4 (control section length equal to 50 feet and Manning's roughness coefficient equal to 0.04). Routings of the freeboard and emergency spillway hydrograhs were started at the 10-day drawdown elevation (2154.0 ft). The results of the routing are summarized in the following table:

				Rainfall	Rainfall	Freeboard	Freeboard
Emergency Spillway Crest [ft]	10- day drawdown elevation [ft]	Bottom width [ft]	Side Slope	ESH [in]	FBH [in]	Peak [cfs]	Elevation [ft]
2170.0	2154.0	100	2:1	6.37	14.95	6460	2177.27

Principal spillway discharge was not included in the emergency spillway or freeboard hydrograph routings.

The computed stage-storage for Sunnycove FRS from Supplement No. 1 is as follows:

Item	Elevation [Ft]	Area [Ac]	Sum Storage [Af]
Bottom of Pool	2128.0	0	0
Top of Sediment Pool	2148.0	3.2	18.2
Crest of Principal Spillway			
Low Stage (orifice)	2148.0	3.2	18.2
High Stage	2169.5	17.5	212.0
Crest of Emergency Spillway	2170.0	18.0	218.7
Crest of Dam (w/o camber)	2178.5	21.5	375.0

The U.S. Army Corps of Engineer conducted a Phase I Inspection Report as part of the National Dam Safety Program (Corps, January 1979). The Arizona Water Commission (now the Arizona Department of Water Resources) prepared the study on behalf of the Corps.

The Phase I study developed a local six-hour Probable Maximum Storm using the procedures outlined in HMR-49, "Probable Maximum Precipitation Estimates, Colorado River and Great Basin Drainages". This storm was routed through the structure using the SCS watershed modeling program "TR-20". The routing was performed with an initial water surface elevation at the spillway crest elevation. The study concluded that the spillway is capable of passing 100% of the PMF with 0.7 feet of residual freeboard.

The Phase I study used a drainage area of 1.35 square miles which agrees with the SCS determination of contributing watershed area. The runoff curve number used by the Phase I study is higher (86) than that used by the SCS (83). The reservoir capacity of 219 acre-feet was used by both the Phase I study and the Supplemental No 1 revision. The Corps study estimated a maximum spillway discharge of 8,250 cfs. It is not clear from the Phase I report how this quantity was derived for the spillway. The following table from the Phase I report provides a summary of the flood hydrology from the Corps/ADWR evaluation. It should be noted that the study assumed that the initial reservoir water surface was at the spillway crest and that the outlet was plugged.

Flood Type	PMF*	PMF**
Storm Precipitation [in]	14.9	14.9
Precipitation duration [hr]	6.0	6.0
Peak Intensity [in/hr]		11.3
Time of concentration [hr]	1.1	1.1
Peak Inflow [cfs]	5168	7709
***Peak Inflow [csm]	3828	5710
Runoff [ac-ft]	943	942
Runoff [in]	13.1	13.1
Runoff Coefficient	86	86
Routed	Yes	Yes
Peak Outflow [cfs]	4858	7188
Peak Outflow [csm]	3598	5324
Maximum Water Surface Elevation [ft]	2176.1	2177.8
Residual Freeboard	2.4	0.7

* Precipitation time distribution per SCS 6-hout storm
** Precipitation time distribution per HMR-49 thunderstorm criteria
*** The units csm represent (cfs)/(sq miles)

In May 1994 the Flood Control District of Maricopa County completed the Wickenburg Area Drainage Master Study (ADMS). As part of this study the District conducted a hydrologic analysis for various streams within the ADMS. The study included the Sunnycove Wash drainage area and Sunnycove FRS. The study hydrologic criterion was based on the 100-year 24-hour storm using the SCS Type II distribution. Rainfall loss parameters used in the study were based on the Green-Ampt methodology. The hydrologic study included a routing routine to route the 100-year flows through the Sunnycove FRS. The study assumed that the principal spillway was the lower gated outlet (incorrect assumption) located at the inlet tower and did not account for the low stage orifice. As a result the study assumed no outflow from the principal spillway for the purposes of hydrologic routing. The study results indicate an inflow into the dam of 1232 cfs for the 100-year 24-hour storm and no discharges through the emergency spillway. The results show that an impoundment occurs and produces a water surface elevation in the pool to 2162.8 ft, which is approximately 7.2 ft below the emergency spillway elevation.

It is interesting to compare the ADMS results to the SCS hydrology results for the 100-year storm event. The District study used the 24-hour duration while the SCS hydrology was based on the 6-hour duration. Rainfall loss parameters were based on the Green-Amp methodology and Curve Numbers for the District and SCS studies, respectively. The 100-year 24-hour inflow into the structure for the District study was 1232 cfs with no outflow from the dam. The 100-year 6-hour inflow into the structure for the SCS study was 1486 cfs with outflow in the principal spillway.

3.3.2. Spillway Inundation Study. The Emergency Action Plan for Sunset FRS, Sunnycove FRS, and Casandro Wash in the Town of Wickenburg (FCD, November

2003) includes a spillway inundation exhibit portraying the downstream inundation due to emergency spillway discharge. The report is unclear however as to the reference for the hydraulic study that documents the evaluation of the inundation limits. The EAP text states that the exhibit portrays the inundation limits for spillway flows at 1/3, 2/3, and full. The report is not clear whether the reference to spillway flows is rated spillway capacity or referenced to PMF discharge. Discussions with FCD staff indicate that the development of the inundation mapping for Sunnycove FRS was conceptually evaluated in-house at the District and that back-up documentation is limited. **Figure 4 (Figures Appendix)** illustrates the emergency spillway inundation area.

Kimley-Horn recommends that a spillway inundation study be conducted for Sunnycove FRS following District methods for such studies. The EAP inundation boundary exhibit virtually depicts very little difference in the boundaries for the three flows. Kimley-Horn recommends that the exhibit only display the inundation boundary for full flows.

3.3.3. Dambreak Analysis. The Flood Control District conducted the dambreak analysis for Sunnycove FRS and documented the analysis in their report titled Dambreak Analysis of Sunnycove Dam on Sunset Wash Wickenburg, Arizona (FCD, January 1987). The analysis used the National Weather Service (NWS) dambreak model SMPDBK. This is a scaled down model of the more rigorous unsteady flow dambreak model DAMBRK by the NWS. The DAMBRK model has now been phased out of service by the NWS and replaced with their more up-to-date model FLDWAV. **Figure 3 (Figures Appendix)** illustrates the dam break inundation area.

The District analysis conducted the dambreak evaluation using a piping breach failure mode. The selection of this failure mode was based on the premise that previous hydrologic studies by the NRCS indicated that no overtopping occurs from routing the PMF through the dam. The report provides the breach parameters used to model the dambreak. These parameters and resulting dambreak peak discharge appear to be too conservative to develop the size of breach modeled given the pool volume that is available. The following table provides the breach parameters used in the dambreak study.

Time To Failure [min]	24
Final breach width [ft]	140
Initial Water Surface Elevation [ft]	2170.0 {emergency spillway crest}
Final Breach Elevation [ft]	2131.5
Volume of Reservoir [af]	218.0
Breach Outflow [cfs]	17,000
PMF Outflow [cfs]	7,700

The dam breach parameters appear to be reasonable, however, not for the reservoir volume behind the dam. Von Thun and Gillette (1990) provide breach parameter relationships for average breach width and breach formation times. Based on their relationships the breach width predicted by the District analysis would be based on a dam

with a reservoir with a volume of 5,000 acre-feet. The derivation of the breach parameters used in the District study is not documented in the report.

The EAP for Sunnycove FRS provides a dambreak inundation exhibit. The exhibit was developed showing dambreak inundation limits for Sunnycove FRS concurrent with the inundation limits from Sunset FRS. It is not clear from the EAP if it was the intent of the EAP to demonstrate the potential for concurrent dambreaks at each of Sunnycove FRS and Sunset FRS.

Kimley-Horn recommends that an updated dambreak analysis and inundation mapping be prepared for Sunnycove FRS. New integrated hydraulic models such as HEC-RAS (unsteady flow and dambreak options) could be used to prepare the updated study. The dambreak update should develop reasonable dambreach parameters using published guidelines and the District's dambreach model currently under development. The inundation mapping for Sunnycove FRS should be prepared without indicating the inundation limits for Sunset FRS.

3.3.4 Sedimentation – The Watershed Work Plan summarizes the sedimentation investigation conducted for Sunnycove FRS. The sediment storage requirements for the FRS is based on local stock pond surveys, studies of sediment sources, and factors that influence sediment yields. The major sources of sediment is from all areas above the dam site. Based on the sediment storage investigation, the NRCS estimated that the sediment storage requirements for the 100-year period was estimated at 18.2 acre-feet for the Sunnycove FRS. The sediment yield rate was determined to be 0.14 acre-feet per square mile per year or 0.19 acre-feet per year delivered to the reservoir. The NRCS used a 95 percent trap efficiency.

Kimley-Horn recently prepared 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 within Maricopa County and Pinal County. The average annual sediment yield was determined to be 0.2 acre-feet per square mile. This is in agreement with the NRCS sediment yield for Sunnycove FRS. Based on this observation, no further evaluation of sediment yield is required for Sunnycove FRS at this time. Future re-evaluation may be considered pending upstream land use changes.

3.4 Geological and Geotechnical Review

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

3.4.1. Geologic Setting. Sunnycove FRS is located in hilly terrain within the northeast-central portion of the Sonoran Desert section of the Basin and Range Physiographic Province near its boundary with the Arizona Transition Zone Section. The latitude and longitude of the center part of the structure is approximately 33° 57' 27" N and 112° 44' 18" W based on NAD 27 datum. This portion of the Basin and Range is characterized by broad alluvial fans that are locally dissected and gently sloping connected valleys bounded by high, rugged northwest, north, and northeast trending mountains including the Date Creek and Weaver Mountains to the north, the Vulture Mountains to the south and the Wickenburg Mountains to the east that rise abruptly to form broad, elongated, deep, sediment-filled valleys produced by block faulting and folding during past episodes of mountain/basin bounding fault movements (Cooley, 1977). The dam is within the city limits of Wickenburg, Arizona off the northeastern flank of the Vulture Mountains in the southeast quarter of Section 11, Township 7 North, Range 5 West.

3.4.2 Seismicity. No seismicity or earthquake evaluation was conducted for the Sunnycove FRS dam design based on a review of the project files. However, a seismicity evaluation for all of the FCDMC dam structures was conducted in 2002. The report entitled "Seismic Exposure Evaluation, Dam Safety Program, Flood Control District of Maricopa County" describes the various seismotectonic zones, fault zones, design earthquake, and characteristic ground motion affecting FCDMC structures (AMEC, 2002).

Sunnycove FRS is situated within the Southern Basin and Range (SBR) Source Zone as defined by AMEC (2002) which includes the Sonoran Seismic Source Zone defined by ADOT (1992). The SBR source zone appears to be tectonically quiescent, with a low level of seismicity and few neotectonic faults that would be considered active or potentially active sources of earthquakes (Bausch and Brumbaugh, 1994; ADOT, 1992). The largest historic earthquake within this zone was a magnitude 5.0 that occurred in the southern part of the source zone in 1965. Only a few minor faults occur in the SBR (AMEC, 2002; ADOT, 1992).

The deterministic and probabilistic analysis of seismic hazard affecting the Sunnycove FRS area was conducted by AMEC (2002) to establish seismic attenuation relationships and the maximum probable earthquake. The closest Quaternary age fault is the Sand Tank Fault located about 77 miles south of the site. According to AMEC (2002) the maximum credible earthquake for this fault source ranges between M6.2 and M6.6. The background earthquake, which is estimated to have a higher maximum magnitude of M7.2, was applied to the regression relationship to derive the horizontal ground acceleration. The recommended peak ground acceleration calculated for the Sunnycove FRS area, based on the background seismic source, is 0.10 g (10 percent of gravitational acceleration) (AMEC, 2002).

3.4.3. Land Subsidence. Land subsidence is known to occur in alluvium-filled valleys of Arizona where agricultural activities and urban development have caused substantial over-drafting or removal of groundwater from thick basin aquifers. The magnitude of

subsidence is directly related to the subsurface geology, the thickness and compressibility of the alluvial sediments deposited in the valleys, and the net groundwater decline.

No unconsolidated, compressible basin fill soils are believed to be present beneath the Sunnycove FRS. The subsurface geological conditions in the embankment dam area consists of relatively hard, cemented Tertiary age fanglomerate (at the surface) and in the subsurface deposited on crystalline bedrock indicate the potential for land subsidence due to groundwater withdrawal does not exist at the Sunnycove FRS site.

According to Staedicke (1995) because there is no history of extensive groundwater pumping or subsidence, the NRCS has never surveyed the Sunnycove FRS structure. Although land subsidence is not expected to affect Sunnycove FRS, Kimley-Horn recommends that the structure be periodically surveyed. Periodic embankment surveys have been initiated by the District as part of the Dam Safety Recurrence Activities for the dam.

3.4.4. Earth Fissures. No earth fissures, related to land subsidence, are documented nor reported as occurring within the Sunnycove FRS project area. Geological conditions in the Sunnycove FRS area preclude the development of earth fissures at this site.

3.4.5. Foundation Conditions. The dam is founded on competent, generally well-cemented fanglomerate bedrock which is generally free of jointing. Recent alluvial and colluvial deposits, and weathered bedrock overlying the unweathered bedrock (fanglomerate) were removed as part of foundation preparation. A high gravel terrace deposit was identified in the upper left abutment on the original dam alignment. The terrace deposit consisted of uncemented, silty gravels and sands containing some very loose pockets of material. The terrace deposit was recognized as a potential seepage problem during the design process, and the dam alignment was shifted upstream to avoid founding the left abutment on these materials. The cutoff trench was excavated into competent fanglomerate in the upper left abutment along the revised alignment.

3.4.6. Foundation Surface Irregularities and Embankment Settlement. As-built plans indicate that the excavated cutoff trench was shaped to a smooth profile. However, potential foundation bedrock surface irregularities are indicated on the drawings along the dam axis outside of the cutoff trench. In particular, a bedrock ridge is evident crossing the dam alignment on the lower left valley section. The ridge spur can be seen upstream from the dam (see Photo 2 in Geologic Section provided by Ken Euge). The ridge extends upstream to downstream (east to west) across the dam axis, and is skewed to the NW-SE oriented axis of the dam. The ridge crosses the centerline of the dam at about Station 14+30 near the outlet works alignment. The as-built drawings indicate that the ridge was substantially removed and shaped. However, a trapezoidal trench was cut through the remnant, shaped ridge to install the outlet pipe. The most notable foundation irregularity is left of the principal outlet conduit where as-built plans show a high point in the foundation. Based on the profile on dam centerline (Sheet No. 3 in the as-built drawing set), the trench cut slope was approximately 2H:1V and about 6.5 feet high at dam centerline. Another subtle bedrock ridge is evident at the base of right abutment,

crossing the dam axis at centerline near Station 12+00. These subtle foundation surface irregularities occur under the deeper sections of the embankment, and are not considered to be problematic with regard to transverse cracking. The mechanism for potential transverse crack formation is tension cracking over the irregularity due to differential settlement of the embankment locally. Monitoring for transverse tension cracking in the dam crest above these foundation irregularities has been recommended as part of the FMEA.

3.4.7. Embankment Soils. The dam was constructed as a zoned embankment, with material zones as shown on Table 1. A typical cross-section of the embankment is shown as Figure 1 of Appendix H.

The materials used to construct Zones I, III, and IV were derived from local borrow sources consisting of alluvial deposits in the vicinity of the dam. The Geologic Report (SCS, 1974) provides logs for test pits and borings, and laboratory test results for bulk samples obtained from two designated borrow areas: Borrow Area 1 located within the limits of the sediment pool, and Borrow Area 2 located within the floodplain downstream from the dam. Materials in the borrow areas consisted primarily of silty sand (SM), clayey sand (SC), and clean to slightly silty sand (SP, SW-SM, SP-SM). Materials in Borrow Area 2 appeared to have higher fines content, and this area was designated as the primary source for the core section (Zone I). Supplemental borrow investigations were also performed in two tributary washes upstream from the dam, and in the main Sunnycove wash above the sediment pool elevation. The main purpose for these supplemental investigations was to locate additional quantities of granular materials for Zone III.

Laboratory testing of representative borrow soils was reported in the Geologic Report (SCS, 1974). Table 2 of Appendix H provides a summary of the laboratory testing results for representative samples from Borrow Areas 1 and 2. The results on Table 2 show that materials within the borrow areas were variable, and that segregating materials by fines content and plasticity would have been challenging.

3.4.8. Original Slope Stability Analyses

Based in part on the laboratory tests (as summarized on Table 2, derived from SCS, 1974), the designers assumed the parameters shown on Table 3 for the slope stability analyses. Slope stability analysis results were obtained for the loading conditions shown on Table 4.

The Design Report (SCS, 1975) indicated that the factor of safety of 1.25 for the 2H:1V downstream slope was acceptable because the Zone III outer shell material was anticipated to have a higher shear strength than the $\varphi = 32^\circ$, c=0 strength assumption used in the analysis. The shear strength assumption was reported to be conservative because the materials specified for Zone III would have lower fines contents (<10%) than the materials tested in the laboratory. However, based on the laboratory tests, the materials with higher fines contents actually exhibited higher shear strengths. The shear strength assumption of $\varphi = 32^\circ$, c=0 is considered to be reasonable and conservative for the

broadly graded Zone III materials, but not for the reason stated in the Design Report. The assumed shear strength is conservative because it is based on the lowest value from direct shear testing of three samples, as summarized on Table 2.

The design intent was for the core section to have higher fines content such that it served as the primary water barrier, and for Zone III to have lower fines content and provide strength and faster drainage to improve stability during drawdown. Ultimately, however, the gradational distinction between Zones I, III, and IV is subtle, and for practical purposes the differences in engineering behavior of these materials is small or negligible. The Specifications limited fines content in Zone III to 15% (revising the original Design Report assumption of 10% fines in Zone III) to accommodate materials from the available local borrow sources. This high fines content does not provide a "free draining" zone as was assumed in the stability analyses for rapid drawdown. Also, the designers assumed full development of a phreatic line within Zones I and IV, and used this as primary justification for incorporating a drain zone (Zone II) in the dam. A more critical purpose for Zone II is actually as a filter to protect against internal erosion and piping. Supplemental geotechnical analysis have been performed as part of this Phase I Structures Assessment to correctly document the slope stability and filter compatibility based on current criteria and our understanding of the structure and zoning. These analyses are described in the following section.

3.4.9. Supplemental Geotechnical Analysis

3.4.9.1 Supplemental Seepage and Slope Stability Analysis. In support of the Phase I Structures Assessment, Gannett Fleming has conducted preliminary supplemental seepage and slope stability analysis for the Sunnycove FRS to document the expected stability of the structure under anticipated loading conditions. The assumptions used in the original stability analysis are suspect for the following reasons:

- Assumption No. 1: Development of a steady state phreatic line – The original design assumed that a steady state phreatic line would be likely to develop within Zones I and IV because of the "...slow release rate of the low stage inlet."
- Revised Assumption No. 1: Development of a steady state phreatic line is unlikely. The maximum detention time for a 100-year event will be less than 10 days, assuming the outlet does not clog. In our estimation, this is insufficient time for a steady state seepage line to develop.
- Assumption No. 2: Zone III is free-draining - Rapid drawdown was not considered in the original design because the Zone III shell was thought to be "free-draining" in comparison to the "slow drawdown rate" of the principal spillway.
- Revised Assumption No. 2: Zone III is not free draining. The Zone III materials, contain up to 15% fines content and are therefore probably not free-draining. However, since the wetting front advance into the upstream zone during a detention event is expected to be very minimal (see following seepage analysis), rapid drawdown is not anticipated to cause slope instability.

Gannett Fleming conducted preliminary seepage analyses using a numerical model (SEEP/W) that allows simulation of the transient wetting front advance into the upstream shell of the dam during a storm detention event, or sequence of events. The results are shown on Figure 2 in Appendix H for a sequence of two back-to-back 100-year floods.

The SEEP/W model correctly accounts for unsaturated and saturated hydraulic conductivities and gradients within the soil to predict the rate of infiltration during a transient event. A standard "Silty Sand" material type was selected from the model's database to represent the Sunnycove embankment materials. The database provides the necessary unsaturated hydraulic parameters for use in the simulation. The vertical saturated hydraulic conductivity was assumed to be 0.6 ft/day (2×10^{-4} cm/s), which was the value used by the SCS designers for the filter/drain design as reported in the Design Report (SCS, 1975). The embankment was modeled as a homogeneous section (ignoring the subtle differences between zones), with a horizontal:vertical anisotropy (kh/kv) ratio of 10:1 for the hydraulic conductivity.

Figure 2 of Appendix H shows the simulated development of the seepage line into the embankment with time during a sequence of two consecutive 100-year events (multiple storm scenario). This impoundment scenario was modeled to estimate realistic, but conservative phreatic lines for use in evaluating upstream slope stability. It is evident that even following multiple storm events, the wetting front will advance to a very limited extent into the dam. During the first impoundment event, the phreatic line does not even reach the drain, and barely penetrates into the foundation. The model results indicate rapid dissipation of the upstream pore pressures as the pool level drops after each event.

Slope stability was analyzed using the program SLOPE/W, which imports the estimated pore pressures from the SEEP/W analysis. Stability was evaluated using the same material property assumptions that the SCS designers used except that a small cohesion intercept (c = 10 psf) was assigned for the strength estimate in order to exclude trivial, extremely shallow (infinite slope) failure surface results. Figures 3 and 4 of Appendix H show the estimated minimum factors of safety for the upstream slope at two times: (1) during drawdown after the 2nd flood impoundment (factor of safety = 2.1), and (2) after drawdown immediately following two consecutive impoundment events (factor of safety = 2.0). The factor of safety is slightly higher at the intermediate impoundment stage because the pool provides additional buttressing against the slope. Note that the slope is predicted to be nearly completely drained (low phreatic line) immediately following the events, based on the assumptions used in the model for impoundment times, drawdown times, and hydraulic conductivity of the materials.

Figure 5 of Appendix H shows the results of slope stability for the downstream slope. Downstream slope stability was modeled to document the factor of safety for a more meaningful failure mode than the shallow, infinite slope analysis that was completed during original design. The minimum factor of safety shown (1.4) is representative of a substantial slope failure that would impact a portion of the crest of the dam. As previously discussed, this factor of safety is based on a conservative shear strength

assumption for the Zone III materials, which may not be representative of the materials actually used to construct the dam.

The results of the preliminary supplemental seepage and slope stability analyses are summarized on Table 5 of Appendix H.

3.4.9.2. Compatibility of Zone II Drain Fill as Filter for Zone I. Zone II is shown on the as-built drawings as a 6-ft wide, vertical chimney drain immediately downgradient from the Zone I core. This zone was designed to act as a drain, but its most important function is to serve as a filter to protect against potential internal erosion and piping of the core materials if transverse cracks were to develop.

Because of its critical function as a filter, the Zone II gradation was checked against current filter criteria in accordance with the NRCS, National Engineering Handbook, Chapter 26 "Gradation Design of Sand and Gravel Filters" (NRCS, 1994). Figure 6 of Appendix H shows what is believed to be a representative gradation curve for the finer materials used in the Zone I "Base Soil" (graphed with solid red triangular symbols). This gradation curve is re-plotted from the gradation curve for Field Sample 2104.1 from the Geological Report (SCS, 1974). The sample was taken from Borrow Area #2 downstream from the dam, and is described as a "reddish brown, calcareous, coarse clayey sand, that classifies as SC according to the Unified Soil Classification system (USCS).

The base soil gradation curve was adjusted for gravel content as shown by the curve graphed with green open triangular symbols. The filtering and permeability (k) criteria which are the basis for the required filter gradation are shown for the adjusted base soil curve by the solid circles on the 15% passing line. The Zone II specification band is shown to fit within the filtering and permeability limits. Thus Zone II is compatible as a protective filter against piping of Zone 1 materials assuming it was placed in accordance with the specified gradation limits.

3.5 Construction History

The January 1979 Phase report provided the following synopsis of construction history for Sunnycove FRS.

> "The application for construction was approved by the State Engineer on September 5, 1975, but the contract for construction was not awarded until March 22, 1976. The first foundation inspection by the Water Commission (now ADWR) engineers was on May 13, 1976 at which time foundation preparation was well along. Contrary to their normal method of operation, the Soil Conservation Service contracted with a private engineering firm for all quality control and construction supervision for the project. Engineers Testing Laboratory of Phoenix performed this service. Construction of the dam was by M. M. Sundt Construction Company. Quality control was in accordance with specifications, but records of tests are not available in commission (ADWR) files.

There were no unusual problems associated with construction and all work was completed and accepted on September 15, 1976."

3.6 Utilities

There are no major utilities directly affecting the dam. **Figure 5 (Figures Appendix)** shows the location of utilities in relationship to Sunnycove FRS. The data used for this figure was obtained through the Town of Wickenburg and site visits and compiled by Hoque and Associates.

3.7 Emergency Action Plan

The Flood Control District has an Emergency Action Plan for Sunnycove FRS (FCD, November 2003). The EAP appears to meet the minimum requirements published in the Federal Emergency Management Agency guidelines FEMA 64 Emergency Action Planning for Dam Owners (FEMA, October 1998). The EAP provides an EAP flowchart based on percent reservoir impoundment on reservoir filling. However, the text (page 5 of the EAP) presents the flowchart based on percent spillway capacity. This discrepancy should be corrected in an updated EAP.

The EAP provides inundation mapping for spillway discharges as well as for potential dambreak. The inundation mapping for Sunnycove FRS is displayed on the same figures as shown for Sunset FRS. Although there is a remote probability that both dams may incur impoundments at the same time it is unlikely that spillway discharges and/or dambreaks will occur concurrently. The inundation mapping exhibit in the EAP for Sunnycove should be shown independent of Sunset FRS.

The EAP flowchart is divided into actions items based on percent reservoir filling. The flowchart is divided into 10, 25, 50, 90, and 100 percent response actions. The five levels of action may occur in a relatively short time frame given the expected rapid filling of the reservoir. Kimley-Horn suggests that the EAP flowchart percent levels be reduced and given consideration for the time to fill the impoundment during large event storms.

The Maricopa County Department of Emergency Management currently has an Emergency Operation Plan (McDEM, 1999) that outlines the procedures and duties of various agencies which are activated in emergency flood situations. Sunnycove FRS is included the McDEM Plan.

The District has prepared a Flood Emergency Response Manual (FERM) (FCD, January 2002) that presents the most current duties for District personnel during significant rainfall events and/or flood emergencies. The FERM indicates that District personnel will be sent to observe the dam during flood emergencies or when weather conditions merit observation. The manual states that the District Operation and Maintenance Division will be notified at an impoundment depth of 21.8 feet. In addition, McDEM would be notified at an impoundment depth of 28.5 feet (7 foot difference).

The notification levels form the FERM and the Emergency Operation Plan are presented in the table below. The table shows a discrepancy in the notification levels in the two plans

	Emergency Operations Plan	**FERM (January 2002)**
	Pool Level [ft]	**Pool Level [ft]**
District Alarm	-	15.0
Notify FCD O&M	-	21.8
Notify McDEM	19	28.5

4.0 PRELIMINARY FAILURE MODES

Kimley-Horn and Associates, Inc. (KHA) facilitated a Preliminary Failure Modes Identification workshop for Sunnycove FRS conducted on February 24, 2004. The overall objective of the workshop was to develop a comprehensive list of potential failure modes for the structure and appurtenances. The workshop was conducted at the offices of the Flood Control District of Maricopa County. The following individuals participated in the workshop:

Tom Renckly, P.E.	Flood Control District
Bob Eichinger, P.E., CFM	Kimley-Horn and Associates, Inc.
Kelli Blanchard, EIT	Kimley-Horn and Associates, Inc.
Debora Miller, Ph.D, P.E.	Gannett Fleming, Inc.
Ken Euge, R.G.	Geological Consultants, Inc.

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

1. **Embankment Overtopping**: The embankment crest and downstream slope are protected against erosion. Overtopping of the embankment could lead to erosion and formation of a breach.

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

3. **Erosion at the Emergency Spillway (Sunnycove FRS):** There is no stabilized control section in the emergency spillway. The downstream limit of the emergency spillway terminates at a drop into a natural wash. The emergency spillway in an earth-lined channel. An erosion headcut in the emergency spillway could encroach into the reservoir pool and cause a breach to form.

4. **Failure of Principal Outlet**: The principal outlet for the dam is a reinforced concrete pipe 30 inches in diameter.

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

6. **Erosion and Piping through the Embankment**: This failure mode relates to the concentrated leak piping along a transverse crack, or along a penetration through the dam (outlet pipes and utility conduits).

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

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

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

10. **Failure Mechanisms Associated with 6-inch AC Chimney/Filter drain pipe**. The chimney/filter drain in Sunset and Sunnycove incorporates a 6-inch asbestos cement perforated drain pipe to collect seepage water. There may be a potential for failure of the drain pipe system by either clogging or structural failure by collapse.

11. **Other considerations**: This section addresses issues that are not directly related to a failure of the dam or its appurtenant facilities, but which nonetheless may be relevant to the FMEA:
    a. Assess the impact of discharge from the emergency spillway on the downstream areas.
    b. Assess the impact of groundwater withdrawal in the vicinity of the dam.

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

5.0 Land Use

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

5.1 Source of Data

The Flood Control District of Maricopa County provided aerial photography, information regarding dam pools and flood retention structures, and land use information.

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 (Jurisdiction defined) (see **Figures 6 and 7** in the **Figures Appendix**). These categories are described briefly below:

- *Residential* land uses include estate residential (1/5-1 unit per acre), single family (small lot 4-6 units per acre to medium lot 2-4 units per acre) and multi-family (10-15 units per acre). There are several areas designated as single-family (small lot) residential land located directly downstream and upstream of the dam. Land area designated as estate residential land is located throughout the 2 mile radius, as shown on Figure 6 (Figures Appendix).
- *Commercial* land uses include retail establishments, office buildings, hotels, and warehouses. Commercial land that contains 50,000 to 100,000 sq. ft is classified as neighborhood commercial land. Commercial land of 100,000-500,000 sq ft is classified as community commercial. There are several areas designated as community commercial land located within 2,500 ft north of the dam.
- *Public Facilities* include community centers, power sub-stations, libraries, city halls, police/fire stations, and other government facilities). There is a large area designated as a public facility located just downstream of the dam.
- *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 residential, commercial, or as public facilities. This information is depicted on Figure 6 and is summarized as follow:

- Wickenburg Way/US 60 is a major road for the Town of Wickenburg and contains a large portion of land designated as community commercial lots. This road is located approximately 300 ft north of Sunnycove FRS and runs upstream and downstream of the dam.
- Residential land surrounds the dam and is located directly downstream of the dam and within 200 ft upstream of the dam.

- There is a public facility located directly upstream of Sunnycove FRS.
- No new residential development was recorded for this dam.

5.4 Proposed Land Use

Future land use plans were obtained through the District. There are no significant changes. A portion of the active open space has changed to industrial land use and is locate within 13,000 feet upstream of the dam. Also downstream of the dam, on the east side of the Gila River the active open space has changed to residential land. These trends illustrate a trend from converting open space into more intense land use categories ("infilling").

6.0 FIELD INSPECTIONS

6.1 Previous Inspections

Kimley-Horn reviewed previous field inspection reports for Sunnycove FRS from project files at the Flood Control District and Arizona Department of Water Resources. The reports collected from these sources date to July 23, 1986. A total of 12 inspections from April 1980 to November 2003 were reviewed as part of this task and are summarized in the table below.

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

- Received ADWR License of Approval in 1986
- Rodent activity noted on slopes in 1994
- Removal of 150 cubic yards of sediment form west end of impoundment area in 1995
- Minor erosion rill on the upstream and downstream slopes reported in 1996
- Erosion gullies on downstream slope were report in 1997 with a note of repair needed
- In 2002 scattered rills throughout both slopes and erosion in emergency spillway approach channel
- In 2003 gravel mulch was applied to the upstream and downstream slopes
- In 2003 the principal spillway conduit and a portion of the Sunnycove/Sunset pipeline was videotaped.
- Significant impoundment events have occurred in 1988, 1989, 1990, 1992, 1993, 1995, 1996, and 1997. The highest impoundment of record was 21.7 ft in 1999.

Summary of Sunnycove FRS Inspection Reports

Date	ADWR Inspector (s)	District Personnel	Other Personnel	Principal Findings
23-Jul-86	Ken Hussain	Paul DiPierro	Bill Kutter	Slight depression at the left abutment contact
				Received License of Approval from ADWR Division of Safety of Dames
15-Jun-88		Paul DiPierro Fred Fuller	Carmella Apodaca-SCS Jon Hall- SCS	Impoundment on 10/29/87 of 12ft
				Impoundment on 11/01/87 of 16.5ft
				Impoundment on 1/18/88 of 7.5ft
				The upstream and downstream slopes have been seeded and mulched since last inspection
14-Jun-89	Ken Hussian	Ellery Biathrow Paul DiPierro Fred Fuller	Jack Elder-SCS	Impoundment on 8/26/88 of 24ft
				Impoundment on 1/4/89 of 3ft
				On the discharge end of the spillway there is an erosion gully that has a depth of 1.5 ft deep and 2 ft wide.
				Seeding and mulching operation conducted the previous year on both slopes does not appear to have been successful. This is probably due to insufficient rainfall.
2-Jul-91		Jim Bening Bob Panasewicz		Northwest groin has recently been rebuilt and is in good condition
				Impoundment on 7/6/1990 of 9.2ft
24-Jun-92		Mike Meng Bob Panasewicz		Highest impoundment the past year was 11.8ft
23-Jun-93	Ken Hussain	Ernie Hamer		Impoundment on 8/22/92 of 21.7ft
				Two enlarged rodent holes at bottom area of downstream slope need to be excavated, backfilled, and compacted.

Summary of Sunnycove FRS Inspection Reports

Date	ADWR Inspector (s)	District Personnel	Other Personnel	Principal Findings
23-Jun-94		Ernie Hamer	John Harrington - SCS	Rodent activity was noted on slopes
		Bob Panasewicz	Rob Genualdi - SCS	
27-Sep-95		Ernie Hamer	John Harrington - NRCS (SCS)	Impoundment on 08/15/95 of 15.2 ft
		Chuck Smith	Rob Genualdi - NRCS (SCS)	150 yards of slit was in the process of being removed from the west end of the impoundment area
				FCD crews recently performed routine maintenance throughout the project
26-Oct-96	Ken Hussain	Ernie Hamer	Steve Smerik -NRCS	On the upstream slope (bottom area) need erosion gullies repaired
		Chuck Smith	Richard Hansen-NRCS	Impoundment on 09/11/96 of 7.15 ft
				Exposed central drain outlet at the downstream toe area needs a screen installed at the end pipe to keep rodents from entering.
				At the bottom of the south groin upstream slope additional rip rap needs to be installed and grouted where scour is occurring
27-Oct-97	Ken Hussain	Ernie Hamer		Several 8x8in erosion gullies along upstream and downstream slopes
		Chuck Smith		Grouted rip-rap at the upstream abutment (left side) need cleaning and repair
6-Nov-02		Michael Greenslade		Scattered rills throughout the upstream and downstream slopes with rills up to 10 inches deep right of the inlet operator guide.
				Gravel mulch application scheduled for 2003
				Video inspection of principal spillway conduit and Sunnycove/sunset pipeline scheduled Y 2002-2003 Need to Survey dam

Summary of Sunnycove FRS Inspection Reports

Date	ADWR Inspector (s)	District Personnel	Other Personnel	Principal Findings
3-Nov-03		Larry Lambert	John Chua-NRCS	Few mesquite and Palo Verde trees on right groin
				Scatter rills right of the inlet operator guide repaired when gravel mulch installed in 2003
				Gravel mulch applied to upstream and downstream face in 2003
				Video taped the Principal spillway conduit and Sunnycove/Sunset pipeline in 2003. Inspected the first manhole (cleanout) downstream of the dam. It was clear with no debris present. Water standing at invert to pipe. Need to complete the video tape inspection report.

6.2 Field Inspection for Structure Assessment

As part of the Phase I Assessment for Sunnycove FRS, a visual inspection of Sunnycove FRS and its appurtenant structures was performed on February 25, 2004. The inspection team included Mike Meng of the District, Bob Eichinger, P.E., CFM, Kimley-Horn and Associates, Inc., Debora Miller, Ph.D, P.E., Gannett Fleming, Inc., Ken Euge, R.G. Geological Consultants, Inc. and Enamul Hoque P. E., Hoque & Associates, Inc. Key elements of the day system including the emergency spillway and the principal outlet were inspected by all members of the inspection team.

The inspection team spilt into two groups to inspect the dam embankment. The team walked along the upstream and downstream embankment toes and the crest of the dam embankment. Key features observed during the inspection were documented in field notes and photographed. A detailed inspection report is included in **Appendix E**. Appendix E also includes photographs and the inspection form used to document the field conditions. Key findings are summarized as follow:

- Gravel mulch place on upstream and downstream slopes in 2003.
- From Station 10+04 to 17+00 several "patches" of finer materials were noticed near the top of the crest on the upstream slope (similar to those observed at Sunset FRS). The patches were placed approximately 10 to 30 feet apart and 4 to 5 ft below crest.
- From the lower 1/3 to 1/2 up the upstream slope the slope appears to have a bulge. The team suspects variability in mulch thickness, but inspected with shovel, mulch is thin on top of bulge.
- Erosion occurring downstream of emergency spillway channel in natural wash.

6.3 Signs of Distress

Based on the field inspection performed by the Kimley-Horn team, historic inspection reports by ADWR and the District and the results of FMEA for the FRS, no major signs of distress have been identified relative to Sunnycove FRS and its appurtenant facilities.

6.4 Safety Deficiencies

Based on the field inspection performed by the Kimley-Horn team, historic inspection reports by ADWR and the District and the results of FMEA for the FRS, no safety deficiencies have been identified relative to Sunnycove FRS and its appurtenant facilities.

7.0 FAILURE MODES AND EFFECTS ANALYSIS

7.1 Introduction

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

The workshop was conducted on March 2, 2004. The workshop participant included:

Tom Renckly, P.E., Flood Control District of Maricopa County, Project Manager,
Larry Lambert, P.E., Flood Control District of Maricopa County, Dam Safety Engineer
Bob Eichinger, P.E., CFM, Kimley-Horn and Associates, Inc., Project Manager
Larry Von Thun, P.E, Dam Consultant and FMEA Facilitator
Debbie Miller, P.E., PhD, Gannett Fleming, Inc. Geotechnical
Ken Euge, R.G., Geological Consultants, Geology
Kelli Blanchard, E.I.T, Kimley-Horn and Associates, Inc, Session Recorder
Nicole Spence Gibson, Arizona Department of Water Resources, Office of Dam Safety

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

The purpose and scope of the FMEA exercise was to:

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

7.2 FEMA Procedure

The FMEA workshop was conducted in the following steps:

- Define the System: This process involves developing a detailed description of the dam system and its components. This is an important step in

understanding how the system components operate and relate and how the components or system may fail.

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

7.3 FMEA Results

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

Adverse Consequences Resulting from Emergency Spillway Discharges During Major Rainfall Events (Category I).

Failure Mode Description: The Sunnycove FRS emergency spillway is a 100 foot wide unlined earth and rock cut channel, located off the left abutment. The spillway channel is separated from the dam embankment by a ridge in the native rock and soil materials. The emergency spillway was constructed into sound fanglomerate as indicated in the as-built plan. Normal flood discharges from the spillway are directed into a natural wash. The flows continue in the wash and are directed toward residential development within the Town of Wickenburg. This potential "failure mode" does not "fail" the dam or emergency spillway but could result in severe adverse consequences for major flooding events. This potential failure mode was rated as a Category I failure mode because normal "successful" operation of the emergency spillway can produce discharges that could have significant adverse consequences and the likelihood of occurrence of these adverse consequences is associated with floods of reasonably probable frequency. The floodwaters will pass through the emergency spillway. From that point the water will flow into a large housing development downstream away from the dam.

Potential for Cracking due to Foundation Irregularities – Seepage Erosion Through Crack. (Category II)

As-built plans indicate potential foundation bedrock surface irregularities along the axis of dam outside of the cutoff trench. In particular, a bedrock ridge is evident crossing the dam alignment on the lower left valley section. The ridge extends upstream to downstream (east to west) across the dam axis, and is skewed to the NW-SE oriented axis of the dam. The ridge crosses the centerline of the dam at about Station 14+30 near the outlet works alignment. The as-built drawings indicate that the ridge was substantially removed and shaped. However, a trapezoidal trench was cut through the remnant, shaped ridge to install the outlet pipe. The most notable foundation irregularity is left of principal outlet conduit where as-built plans show a high point in foundation. Based on the profile on dam centerline (Sheet No. 3 in the as-built drawing set), the trench cut slope was approximately 2H:1V and about 6.5 feet high at dam centerline. Another subtle bedrock ridge is evident at the base of right abutment near Station 12+00. Transverse cracks could potentially form over these surface irregularities. The mechanism for potential transverse crack formation is tension cracking over the irregularity due to differential settlement of the embankment locally.

Failure Mode Description: The potential failure mode development sequence is: Transverse crack forms prior to or at onset of impoundment, impoundment occurs, seepage flows through crack, seepage erosion occurs along a crack, crack widens flow/erosion rate increase and breach forms.

7.4 FMEA Limitations

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

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 for the dam. This section provides a summary of the recommendations.

8.1 Hydrologic and Hydraulic Recommendations

1. A spillway inundation study should be conducted for Sunnycove FRS following District methods for such studies. The EAP inundation boundary exhibit virtually depicts very little difference in the boundaries for the three flows. Kimley-Horn recommends that the exhibit only display the inundation boundary for full flows.

2. An updated dambreak analysis and inundation mapping should be prepared for Sunnycove FRS. New integrated hydraulic models such as HEC-RAS (unsteady flow and dambreak options) could be used to prepare the updated study. The dambreak update should develop reasonable dambreach parameters using published guidelines and the District's dambreach model currently under development. The inundation mapping for Sunnycove FRS should be prepared without indicating the inundation limits for Sunset FRS.

3. The EAP flowchart is divided into actions items based on percent reservoir filling. The flowchart is divided into 10, 25, 50, 90, and 100 percent response actions. The five levels of action may occur in a relatively short time frame given the expected rapid filling of the reservoir. Kimley-Horn suggests that the EAP flowchart percent levels be reduced and consolidated given consideration for the time to fill the impoundment during large event storms.

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

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

8.2 Geotechnical and Geological Recommendations

1. Monitoring. In recognition of the presence of substantial zones of uncemented granular materials in the bedrock ridge that forms the upper left abutment, we recommend monitoring for seepage through the left abutment and on the south side of the emergency spillway cut during any sustained impoundment events. In

recognition of foundation irregularities, KHA recommends visual monitoring for transverse tension cracking in the dam crest near Stations 12+00 and 14+30.

2. Phase II Documentation of Slope Stability and Seepage Analyses for Main Dam. Gannett Fleming does not anticipate any problems with slope stability under any reasonable loading conditions for Sunnycove FRS. However, Gannett could not find adequate documentation of slope stability factors of safety for specified loading and design criteria that have been established by appropriate jurisdictional agencies. Table 6 in Appendix H shows the definitions of various loading conditions and a comparison between the current NRCS design criteria that are outlined in TR-60 (SCS, 1985), and the current criteria as presented in the ADWR dam safety rules and regulations for jurisdictional dams.

The original stability analysis, and our preliminary (Phase I) stability analyses do not completely document factors of safety for all the loading conditions that would need to be evaluated under current NRCS or ADWR criteria. Table 7 Appendix H summarizes the results from the original stability analysis, results from the preliminary supplemental analysis performed as part of this Phase I study, and indicates where additional analysis is required to document factors of safety under all loading conditions.

(1) **Rapid Drawdown Stability (upstream slope)**: Preliminary analyses were conducted as part of this Phase I study that simulated a plausible scenario for development of the seepage line into the dam under multiple temporary impoundment events, and to assess the upstream slope stability under normal drawdown conditions. These analyses show that it is very unlikely that a steady state phreatic line will develop in the Sunnycove FRS, assuming the outlet works is operational and is not clogged for sustained periods of time following a flood event. ADWR criteria require that an "instantaneous" drawdown analysis be performed. The ADWR guidance and rules were developed for water retention dams, and the criteria are interpreted to mean that rapid drawdown stability should be evaluated assuming that a steady state phreatic line has developed from the normal high reservoir pool elevation. It is not clear what is meant in the ADWR regulations by "instantaneous" drawdown, and this may be subject to interpretation. Additional analyses would be required to assess and document rapid drawdown factors of safety under these assumptions. A recommended analysis procedure is outlined as follows:

   a. Establish the steady state phreatic line and pore pressure distribution using 2-D seepage analysis. Use reasonable assumptions for hydraulic conductivity and anisotropy for the embankment materials based on available information.
   b. Model the dissipation of pore pressures with time, starting from the steady state initial condition, and assuming a worst case drawdown rate. The drawdown rate should be based on the current outlet capacity, or an adjusted (higher) capacity if the outlet is modified. This is not an "instantaneous" drawdown assumption, but is much more realistic given the physical constraints on the rate of drawdown. Pore pressure

dissipation with time from the steady state condition can be estimated using either a transient numerical flow analysis or a suitable analytical procedure.

c. Evaluate the upstream slope stability at various stages of the drawdown by inputting the instantaneous pore pressure grids and reservoir levels from the seepage analysis. Report the minimum value, and compare against the design criteria (minimum factor of safety = 1.2).

(2) **Downstream Slope Stability Under Steady Seepage**: The minimum factors of safety that have been computed for the dry downstream slope do not achieve the minimum criteria of 1.5 (see Table 6 Appendix H). Gannett Fleming does not consider the infinite slope analysis that was done in the original design as representative of a "critical" failure scenario. Our preliminary analysis evaluated more substantial failure surfaces which resulted in a minimum factor of safety of 1.4. This factor of safety was computed using the original strength assumptions of $\varphi = 32^\circ$ and c=0. It is likely that this strength assumption is conservative, based on the results of three strength tests that were done on the original borrow materials (Table 2 Appendix H). The following additional analyses could be performed to further evaluate and document the stability of the downstream slope:

a. Conduct a search for reasonable "critical" failure surfaces on the dry downstream slope using a higher shear strength, which may be more representative of the in-place materials. The average shear strength for three tests on borrow samples is $\varphi = 34^\circ$ and $c = 0$.

b. Re-test the shear strength using representative samples of materials from the downstream Zone III shell. Sample representative materials from the downstream slope using a backhoe or drill rig, and evaluate gradation, plasticity, and compaction characteristics. Compact shear strength test samples to 95% maximum dry density (per the placement specifications), and test in direct shear or triaxial shear to estimate the actual shear strength of the Zone III fill. Re-compute the factor of safety based on the revised shear strength.

(3) **Upstream slope stability under steady seepage, partial pool**: The original analysis evaluated upstream slope stability under steady seepage for the maximum pool elevation only, resulting in a minimum factor of safety of 1.8. The ADWR criteria for partial pool conditions is intended for water retention dams, in which a steady state phreatic line may develop for intermediate pool elevations. The factor of safety may be lower for the intermediate pool conditions than the steady state condition under maximum pool. The following analysis could be done to document the minimum partial pool factor of safety, under the scenario that the outlet works is clogged such that the steady state phreatic line develops:

a. Perform seepage analyses under various partial pool elevations to establish the steady state pore pressure distributions within the dam at each pool elevation.

b. Conduct slope stability analyses for each partial pool seepage analysis result, and graph the results as factor of safety versus pool elevation.
c. Report the minimum factor of safety and corresponding pool elevation.

(4) **Pseudo-static stability analysis (critical downstream slope section)**: Seismic stability analyses were not performed as part of the original design. To document seismic stability under current design criteria, the following analysis could be conducted:

a. Based on the regional seismicity review performed for the Casandro Wash Dam, as documented the design report for that structure (CH2M Hill, 1995), a reasonable estimate for the peak ground acceleration (PGA) for the area is 0.1g. ADWR guidance recommends using a pseudo-static coefficient = 60% of the PGA, or 0.06.
b. Conduct the pseudo-static analysis on the downstream stability section with the lowest static factor of safety, and report the result.

8.3 Additional Recommendations

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

9.0 REFERENCES

AWC; 1979. "Sunny Cove Flood Retarding – Arizona Dam No. 7-48, Maricopa County, Arizona, Phase I Inspection Report, National Dam Safety Program," prepared for Los Angeles District Corps of Engineers, by State of Arizona, Arizona Water Commission, Office of the State Water Engineer, January.

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SCS; 1975a. Construction Plans and Specifications, Sunnycove FRS, Wickenburg Watershed Protection and Flood Prevention Project

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Table 1. Sunnycove FRS
Physical Data

	As-Designed Sunset FRS and Sunnycove FRS Design Report, 1975		Sunnycove FRS Phase I Inspection Report, 1979		As-Builts Plans Sunnycove FRS 1975	
Item		Reference Page		Reference Page		Reference Sheet
Inflow Design Flood						
PMF						
Watershed Characteristics						
Watershed Area (acres) (square miles)	908	*See Note Below	864 (1.35)	10	864 (1.35)	15
Maximum Elevation (ft MSL)	2460	*See Note Below				
Minimum Elevation (ft MSL)	2080	*See Note Below	2130	10		
Reservoir or Flood Pool						
Peak Inflow Flood During 100 year, cfs	1486	*See Note Below				
Peak Inflow Flood During PMF,cfs			7709	2 (Flood Estimate)		
Storage at Emergency Spillway Crest (ac-ft)	218.7	6	219	10	218.7	15
100-yr Sediment Accumulation (ac-ft)	18.2	6				
Reservoir Storage at PMF Water Level (ac-ft)	375	6				
Main Embankment						
Type			Zoned Earth fill			
Length (ft)			714	1	714	3
Maximum Height above Stream bed (ft)			48.5	1		
Crest Width (ft)			14	1	14	4R
Crest Elevation (ft MSL)	2178.5	6	2178.5	1 (Information Sheet)	2178.5	3
Upstream Slope (H:V)			3:1	1	3:1	4R
Downstream Slope (H:V)			2:1	1	2:1	4R
Outlet Works						
Flood Control Inlet Tower			gated	1 (Information Sheet)	perforated gate well	4R
Type	RCP	2	RCP	1 (Information Sheet)	RCP	6R
Length (ft)					297	4R
Diameter	30	15	30	1 (Information Sheet)	30	12
Maximum Discharge (cfs)	9.1	4				
Principal Spillway Crest (ft MSL)	2170	15	2169.5	1 (Information Sheet)	2169.5	4R
Emergency Spillway						
Type			unlined earth channel	1 (Information Sheet)		
Approach Channel Length (ft)	n/a				307	13R
Crest Length (FT)	100	5	100	3	100	13R
Crest Elevation (ft)	2170	5	2170	10	2170	3
PMF- Maximum Water Surface Elevation (ft MSL)			2177.8	2 (Flood Estimate)		
PMF- Peak Outflow			7188	2 (Flood Estimate)		
ADWR Size Classification		Small				
ADWR Hazard Potential Classification		High				

*The reference for the Peak Inflow Flood During 100 year is the Wickenburg WPP Arizona Sunset FRS, Sunnycove FRS and Pipeline Hydrology computation sheets

Item	NRCS Original Design Criteria	NRCS Current Criteria	ADWR Criteria Eff.June 12, 2000	Comment/Remarks
Publications and References for NRCS and ADWR Criteria	1) "Engineering Memorandum 27 - Earth Dams" SCS March 19, 1965 (EM-27) 2) Frye Creek Stockton Wash Watershed Work Plan SCS September 1958	Technical Release No. 60 TR-60. Earth Dams and Reservoirs. Oct. 1985. Amended Jan 1991	Arizona Administrative Code Title 12, Chapter 15 Effective June 12, 2000	
Size			**Small:** Storage capacity 50 to 1,000 Acft and height 25 to 40 ft	
Hazard	Class C. Failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, major highways, or railroads	Class C. Failure may cause loss of life, serious damage to homes, industrial and commercial buildings, important public utilities, major highways, or railroads	**High**: failure or improper operation of a dam would be likey to cause loss of human life because of residential, commercial, or industrial development. Intangible losses may be major and potentially impossible to mitigate, critical lifeline services may be significantly disrupted, and property losses may be extensive.	High: Probable loss of human life - Probable - one or more expected Probable Economic, Lifeline, and Intagible Losses - Low to High
Inflow Design Flood (IDF)	One-percent event	For high hazard dams; PMF. The design height of the dam is to be sufficient to prevent overtopping during the passageof either the (1) freeboard hydrograph or (2) the emergency spillway hydrograph plus freeboard required for wave action, whichever is larger.	High: All Sizes **0.5PMF to PMF**: High hazard class with any size class will vary with size increased based on downstream population (persons at risk) and potential economic losses. The applicant shall consider foreseeable future conditions.	Phase I report states that IDF is PMF . ADWR database shows IDF for Sunset is PMF and Sunnycove is PMF. No overtopping of PMF for both dams.
Total Freeboard (between Emergency Spillway crest and the settled top of the dam crest)			The applicant shall ensure that the total freeboard is the largest of the following: a) The sum of the IDF maximum water depth above the spillway crest plus wave runup. b) The sum of the IDF maximum water depth above the spillway crest plus 3 feet.	
Residual Freeboard (between maximum IDF water surface elevation to dam crest)		between maximum water surface elevation to dam crest	means the vertical distance between the highest water surface elevation during the IDF and the lowest point at the top of the dam	
Principal Spillway Design Flood	100-year	100-year. A storm duration of not less than 10 days is to be used for sizing the principal spillway. Use NEH-5, TR-29, Design Note 8	N/A	100-year
Principal Spillway Capacity	(a) Discharge through the emergency spillway will not occur (b) Adequate to empty the retarding pool in 10 days or less. Or adequate to empty 80 percent or more of the maximum volume of retarding storage after 10 days. The 10-day is measured starting from the time the maximum water surface elevation is attained during the passage of the principal spillway flood (EM -27 Page E-1 Supplement 6)	(a) Discharge through the emergency spillway will not occur (b) Adequate to empty the retarding pool in 10 days or less. Or adequate to empty 80 percent or more of the maximum volume of retarding storage after 10 days. The 10-day is measured starting from the time the maximum water surface elevation is attained during the passage of the principal spillway flood (c) The minimum diameter of the principal spillway conduit is to be 30 inches.	Low level outlet that is capable of: i) draining the reservoir pool to the sediment pool level ii) high hazard dams - Outlet works shall be a minimum of 36-inch diameter b. high hazard dams: capacity to drain 90% of storage capacity of reservoir within 30 days. c. has diaphram filter or other current practice measure to reduce potential for piping along conduit. e. has an emergency manual override system or can be operated manually	(a) Discharge through the emergency spillway will not occur (b) Adequate to empty the retarding pool in 10 days or less. Or adequate to empty 80 percent or more of the maximum volume of retarding storage after 10 days. The 10-day is measured starting from the time the maximum water surface elevation is attained during the passage of the principal spillway flood
Initial Reservoir Stage for Principal Spillway Hydrograph Routing	Crest elevation of the lowest ungated principal spillway inlet or the anticipated elevation of the sediment storage, whichever is higher	Crest elevation of the lowest ungated principal spillway inlet or the anticipated elevation of the sediment storage, whichever is higher	N/A	Crest elevation of the lowest ungated principal spillway inlet or the anticipated elevation of the sediment storage, whichever is higher
Runoff Volume Estimation Procedures for Principal Spillway Sizing	National Engineering Handbook No 4 Hydrology	Part 630 and NEH 4. Use CN method and AMC II	N/A	
Design Procedures for Principal Spillways	EM -27 Appendix E Principal Spillways	TR 60 Chapt 6 Principal Spillways	for high and significant hazard dams principal spillway shall be 36-inches or greater; all high and significant hazard dams shall have the capacity to evacuate 90% of storage capacity of reservoir within 30 days, excluding reservoir inflows; corrugated metal pipe not acceptable	
PMP Storm Types	NA	General and local. HMR No. 49. the storm duration and distribution that result in the maximum reservoir stage when the hydrograph is routed through the structure should be used.	Both frontal and thunderstorm (tropical) type storms should be studied with due consideration given to tropical storm potential and orographic influences that may greatly increase rainfall. Local Storm duration 6 hour; General Storm duration 72 hour (whichever is greater)	See ADWR guidlelines "PMF Studies for Evaluation of Spillway Adequacy General Guidelines" Revised March 2004. Site-specific PMP studies are acceptable.
Reservoir Stage-Storage Curve for Routing PMP Hydrograph and Stability Design Storm Hydrograph		For Class C Structure 1: emergency spillway hydrograph P100 + .26x(PMP - P100) 2: freeboard hydrograph = PMP	The adequacy of the emergency spillway is normally determined by routing the IDF through the reservoir and spillway. Flood routings for spillway capacity determinations will normally be required to begin with reservoir storage at the spillway crest elevation. An infrequent exception is that the reservoir is used exclusively for flood control and would	
Emergency Spillway Capacity	(a) Pass the emergency spillway hydrograph resulting from P100 at the safe velocity (b) Pass the freeboard hydrograph with the water surface elevation at or below the design top of the dam (c) Capacity must not be less than that determined from Figure F-1 on Page F-3 in EM-27	(a) Pass the emergency spillway hydrograph resulting from P100 at the safe velocity (b) Pass the freeboard hydrograph with the water surface elevation at or below the design top of the dam (c) Capacity must not be less than that determined from Figure 7-1 on Page 7-8 in TR-60	Spillways and outlets of flood control dams shall be able to pass all the flood water at a discharge rate as calculated on the basis of the spillway design flood. Emergency spillways must be designed to safely discharge the PMF while maintaining adequate freeboard.	Additional ADWR criteria: i. include a control structure to avoid head cutting and lowering o fthe spillway crest for spillways excavated in soils or soft rock. Ii. Ensure each spillway, in combination with outlet, is able to safetly pass the peak discharge flow rate, as calculated on the basis of the IDF.

Item	NRCS Original Design Criteria	NRCS Current Criteria	ADWR Criteria Eff.June 12, 2000	Comment/Remarks
Emergency Spillway Crest Elevation	(a) Satisfy the 2500 ac-ft total capacity limit (PL 83-566, NWM 500.20) (b) The discharge through the emergency spillway will not occur during the routing of the principal spillway hydrograph (c) If the 10-day drawdown requirement is not met for principal spillway capacity design, then the crest elevation of the emergency spillway will be raised as noted on Page 6-1, Capacity	(a) Satisfy the 2500 ac-ft total capacity limit (PL 83-566, NWM 500.20) (b) The discharge through the emergency spillway will not occur during the routing of the principal spillway hydrograph (c) If the 10-day drawdown requirement is not met for principal spillway capacity design, then the crest elevation of the emergency spillway will be raised as noted on Page 6-1, Capacity	N/A	(a) Satisfy the 2500 ac-ft total capacity limit (PL 83-566, NWM 500.20) (b) The discharge through the emergency spillway will not occur during the routing of the principal spillway hydrograph (c) If the 10-day drawdown requirement is not met for principal spillway capacity design, then the crest elevation of the emergency spillway will be raised as noted on Page 6-1, Capacity of Principal Spillway.
Initial Reservoir Stage for Emergency Spillway Hydrograph Routing	The highest value from the following elevations: (a) Elevation of the lowest ungated principal spillway inlet (b) The anticipated elevation of the sediment storage (c) The elevation of the water surface associated with significant base flow (d) The pool elevation after 10 days of drawdown from the maximum stage attained when routing the principal spillway hydrograph. (Page 7-2 in TR 60)	The highest value from the following elevations: (a) Elevation of the lowest ungated principal spillway inlet (b) The anticipated elevation of the sediment storage (c) The elevation of the water surface associated with significant base flow (d) The pool elevation after 10 days of drawdown from the maximum stage attained when routing the principal spillway hydrograph. (Page 7-2 in TR 60)	Deviations from the normal starting level of routing at the spillway crest elevation must be considered on the basis of risk and reservoir operating procedure, and are evaluated by the Department on a case-by-case basis.	
Sedimentation	50-year sediment reservoir per reference no 2.	100-year sediment reservoir	N/A	50-year sediment reservoir per reference no 2.
Dam Breach		See TR-60 for Qmax for depth of water less than 103 feet	Unless waived by the Director, owners of high and significant hazard potential dams shall prepare, maintain, and exercise Emergency Action Plans for immediate defensive action to prevent failure of the dam and minimize threat to downstrem development.	Develop EAP to FEMA 64 guidelines and ADWR requirements.
Special Requirement for Storage	2500 ac-ft (total reservoir capacity = water volume plus the anticipated sediment volume) according to Table 500-2 in Public Law 83-566, National Watershed Manual-Part 500.20. Based on Table 500-2, any amount for construction costs and >4,000 ac-ft of total capacity require a committee on Environment and Public Works of the Senate and committee on Public Works and Transportation of the House of Representatives.	2500 ac-ft (total reservoir capacity = water volume plus the anticipated sediment volume) according to Table 500-2 in Public Law 83-566, National Watershed Manual-Part 500.20. Based on Table 500-2, any amount for construction costs and >4,000 ac-ft of total capacity require a committee on Environment and Public Works of the Senate and committee on Public Works and Transportation of the House of Representatives.	The temporary storage will be evacuated as soon as possible following such periods of flood.(from License)	
Seismic		See NEH-8 and Part 531, 210-v	Design the dam to withstand the maximum credible earthquake (MCE)	AAC R12-15-1216.B.2. Seismic Requirements
Design for Vegetated and Earth Emergency Spillways	(a) From EM - 27 Pages Appendix F (b) Spillway will not breach during passage of the freeboard storm (f) Maximum permissible velocity in vegetated emergency spillways: Table F-II in EM-27 (g) Maximum permissible velocity in earth emergency spillways: Table F-III in Em-27(Fortier and Scobey's Study) (h) Manning's n = 0.02 for design velocity in earth spillways; Capacity of earth spillways will be based on a appraisal of the Manning's n at the site (Page F-2 EM-27). (i) Manning's n = 0.04 for vegetated spillways (Page F-2 in EM-27)	(a) From EM - 27 Pages Appendix F (b) Spillway will not breach during passage of the freeboard storm (f) Maximum permissible velocity in vegetated emergency spillways: Table 7-1 in TR-60 (g) Maximum permissible velocity in earth emergency spillways: Table 7-2 in TR-60(Fortier and Scobey's Study) (h) Manning's n = 0.02 for design velocity in earth spillways; Capacity of earth spillways will be based on a appraisal of the Manning's n at the site. (i) Manning's n = 0.04 for vegetated spillways	Criteria depends on whether earthen spillway is located on soils subject to liquefaction.	
Miscellaneous Design Criteria		Minimum top width is 14 feet.	a. the design ...shall include seepage collection and prevent internal erosion or piping due to embankment cracking.. B. the minimum top width of an embankment dam is equal to the structrual height of the dam divided by 5 plus an additional 5 feet. The required minimum top width for any embankment dam is 12 feet. The maximum top width for any embankment dam is 25 feet. c.the applicant shall keep the top of the dam and appurtenant structures accessible by equipment and vehicles for emergency operations and maintenance.	

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FIGURE 1: VICINITY MAP

FCD2003C015
PCN 050.36.31

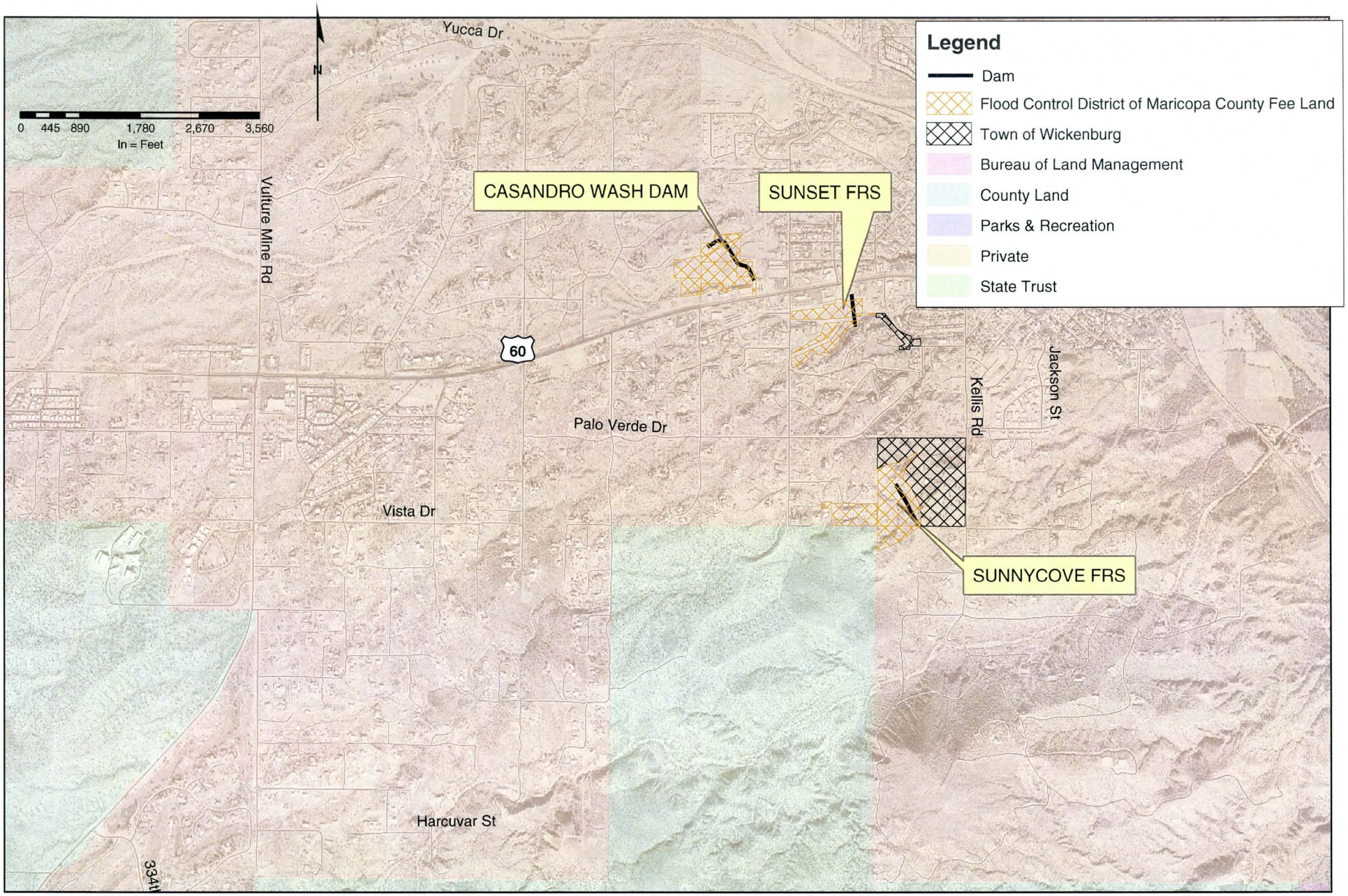

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FIGURE 2: LANDOWNERSHIP

FCD2003C015
PCN 050.36.31

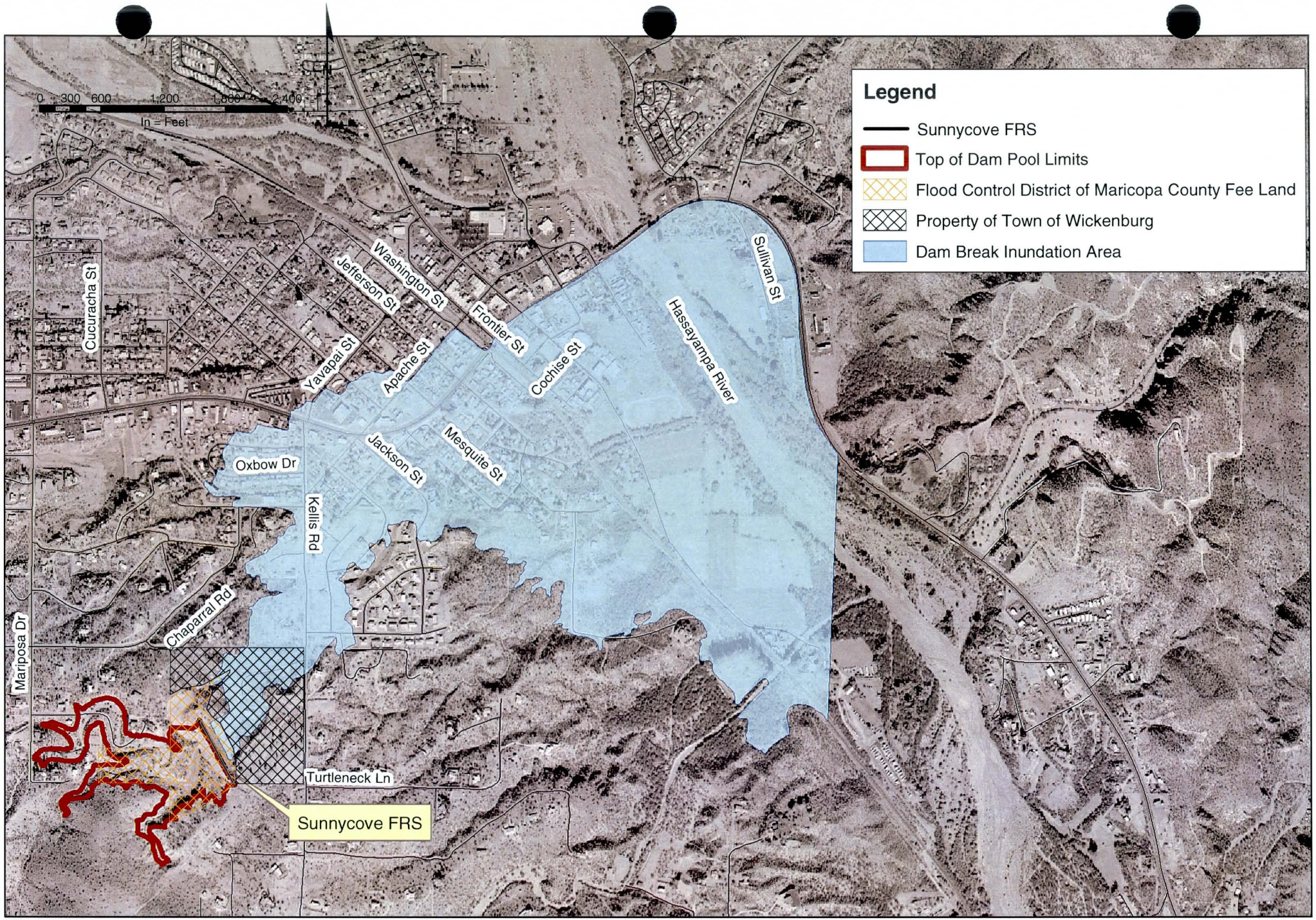

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FIGURE 3: SUNNYCOVE FRS BREAK INUNDATION AREA

FCD2003C015
PCN 050.36.31

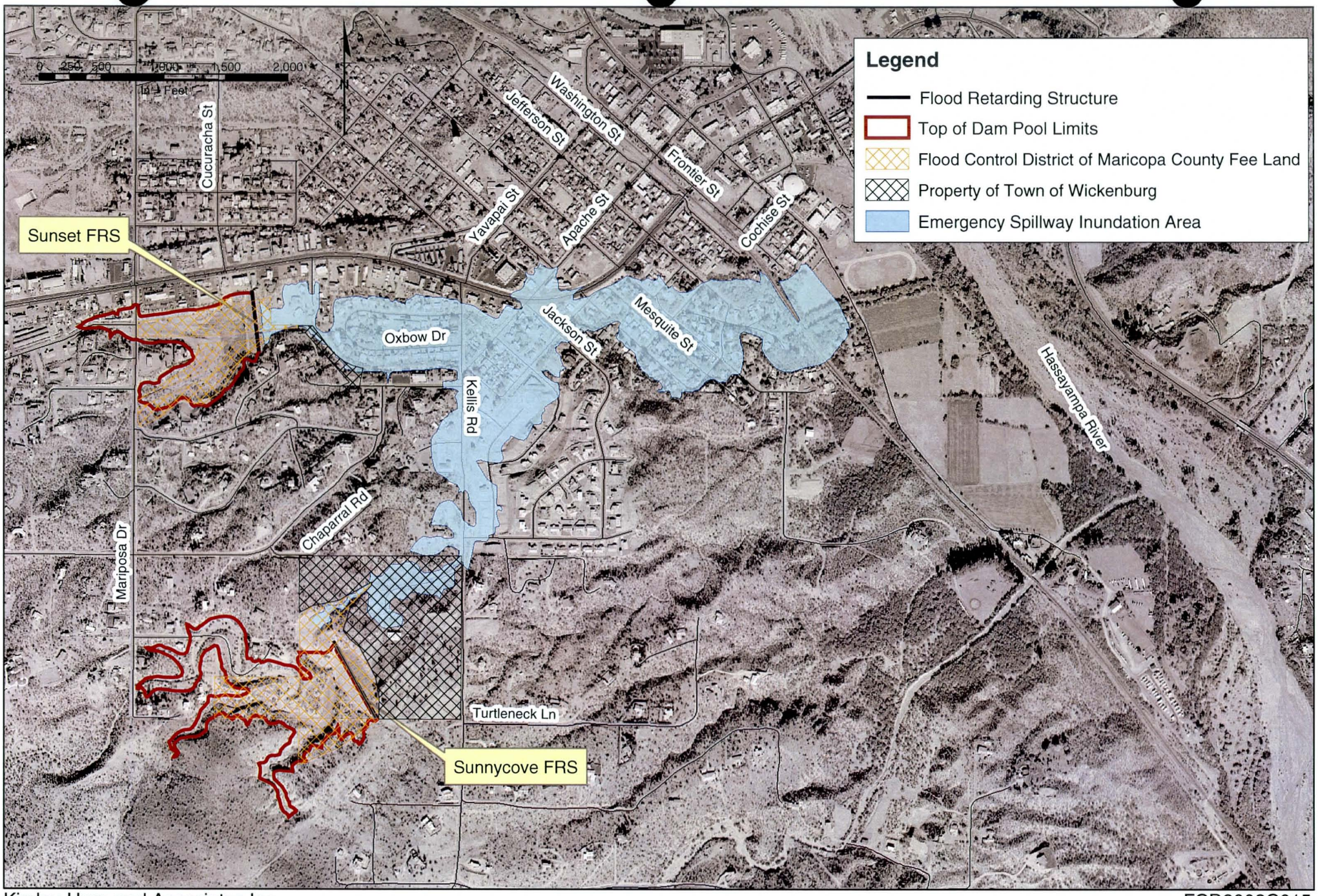

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FIGURE 4: SUNNYCOVE FRS AND SUNSET FRS EMERGENCY SPILLWAY INUNDATION AREA

FCD2003C015
PCN 050.36.31

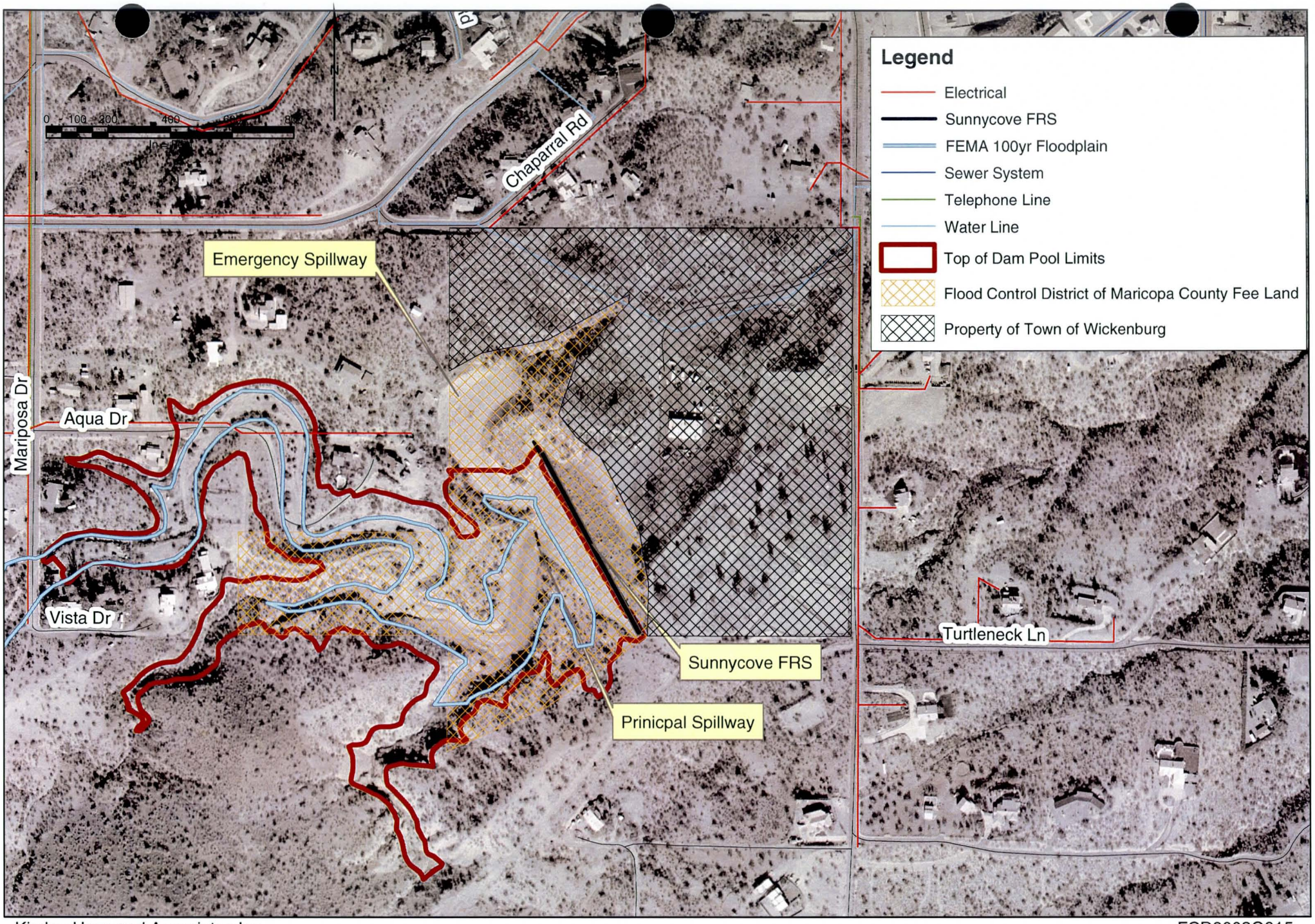

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FIGURE 5: SUNNYCOVE FRS TOP OF DAM POOL LIMITS

FCD2003C015
PCN 050.36.31

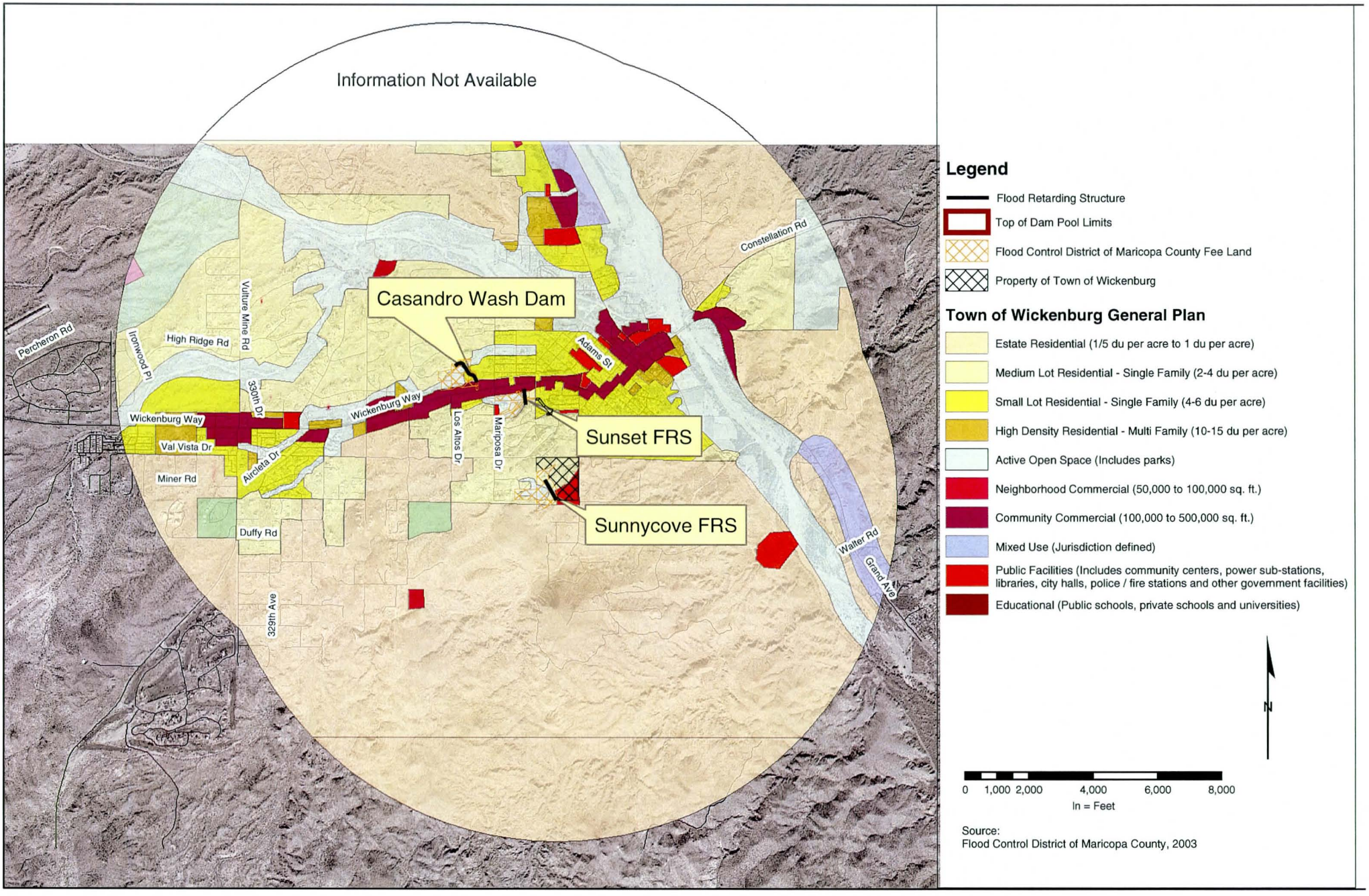

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FIGURE 6: GENERAL LAND USE

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PCN 050.36.31

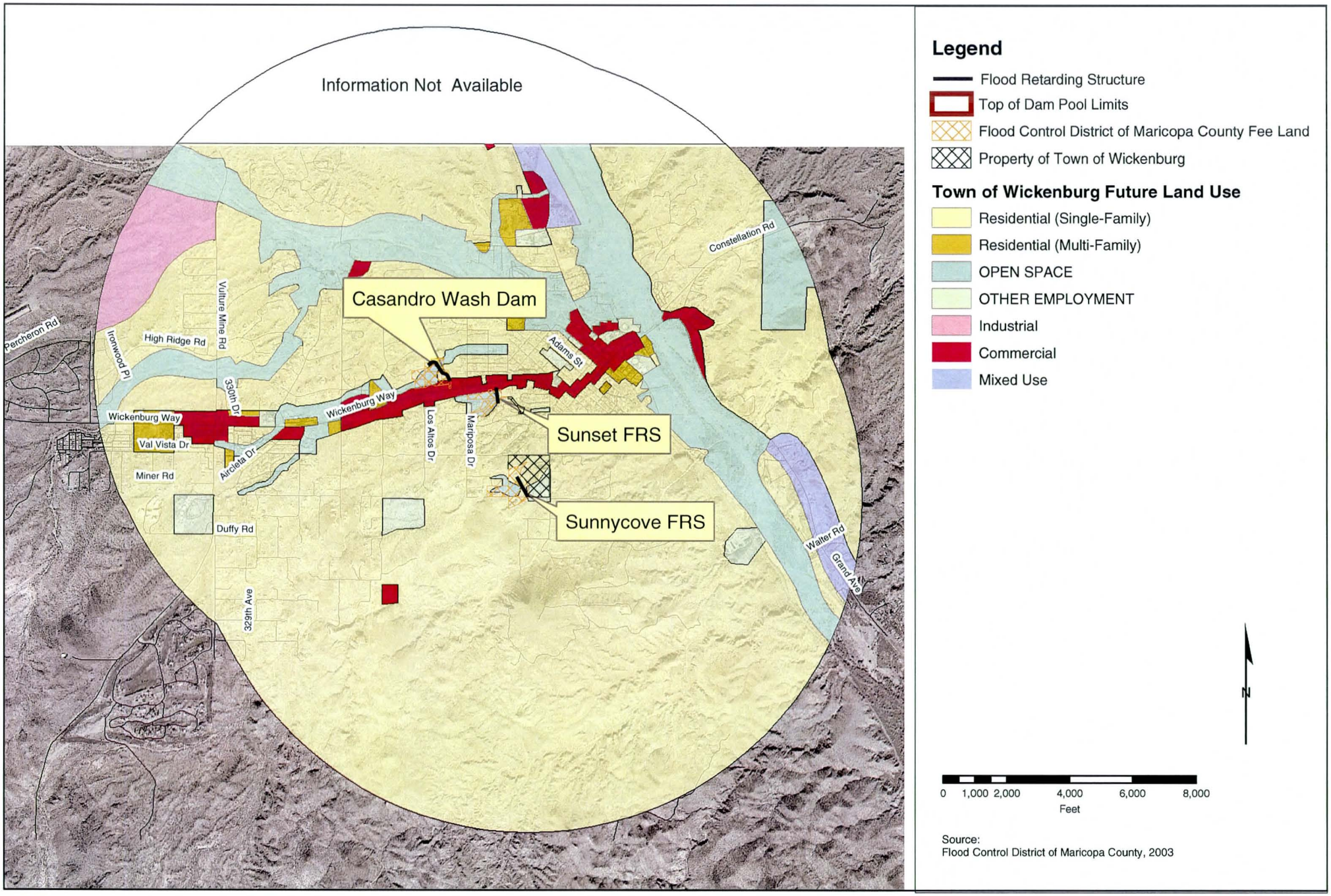

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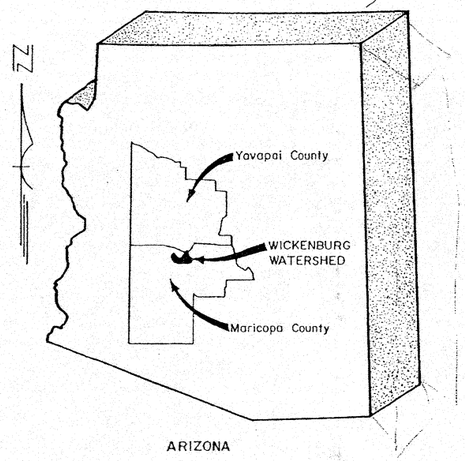
FIGURE 7: FUTURE LAND USE

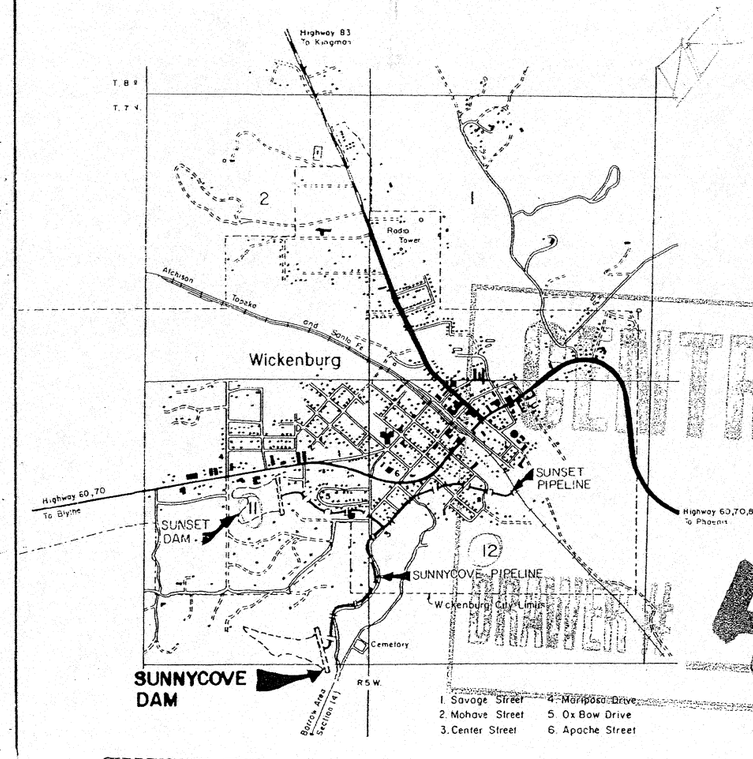
FCD2003C018
PCN 050.36.31

WICKENBURG WATERSHED
PROTECTION AND FLOOD PREVENTION PROJECT
MARICOPA AND YAVAPAI COUNTIES, ARIZONA
PLANS FOR THE CONSTRUCTION OF
SUNNYCOVE FLOODWATER RETARDING STRUCTURE



PREPARED FOR THE
MARICOPA COUNTY FLOOD CONTROL DISTRICT
TRIANGLE NATURAL RESOURCE CONSERVATION DISTRICT
WICKENBURG NATURAL RESOURCE CONSERVATION DISTRICT
YAVAPAI COUNTY BOARD OF SUPERVISORS
AND
TOWN OF WICKENBURG
BY
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

OFFICE USE ONLY
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
CONSTRUCTION AND OPERATIONS DIVISION
BORROW #
NOT RECORD COPY



CENTRAL FILES
DRAWING

INDEX OF DRAWINGS

SHT. NO.	TITLE
1.	INDEX OF DRAWINGS
2.R	PLAN OF DAM AND RESERVOIR
2A.R	ALIGNMENT LOCATION MAP
3.	PROFILE ON ℄ DAM AND EXCAVATION SCHEDULES
4.R	PROFILE ON ℄ PRINCIPAL SPILLWAY & TYPICAL CROSS SECTION
5.R	DRAIN LAYOUT DETAILS
6.R	RISER LAYOUT DETAILS
7.	TRASH GUARD DETAILS
8.	RISER DETAILS
9.	RISER DETAILS
10.	RISER DETAILS
11.R	GATE CONTROL DETAILS
12.	PRINCIPAL SPILLWAY CONDUIT DETAILS
13.R	EMERGENCY SPILLWAY PLAN, PROFILE AND PAYLINES
14.	FENCING DETAILS
15.	SIGN DETAILS
16.	CACTUS SALVAGING DETAILS
17.	SOIL LOGS
18.	LOCATION OF BORROW AREA (SECTION 14)

40

AS BUILT
CONSTRUCTION COMPLETED
3-15-76

GENERAL NOTES

1. Elevations are in feet above mean sea level datum.
2. All stationing refers to centerline of construction and is the measured horizontal distance.
3. All cross sections are taken looking in the direction of increasing stations unless otherwise shown.
4. Soil logs on these drawings show general condition at the site and are the basis for this design. Log of foundation investigation borings and test pits are contained in the exploration data report. A complete set of all soils investigation logs and geology report is available at the project office for reference.
5. Unified soil descriptions are based on field identifications except where an asterisk is shown the classification has been based on laboratory analysis.

STRUCTURAL NOTES

1. All exposed concrete edges shall be chamfered one inch or rounded.
2. Spacing of reinforcement bars is measured center to center of bars. Bar cover is the clear distance between surface of bar and face of concrete, and unless otherwise shown is two inches for formed and top surfaces and three inches for surfaces against earth.
3. All bar splices not shown on these drawings shall be staggered with bars lapped a minimum of 30 diameters of the smaller bar in the splice.
4. All nuts, bolts, washers, etc., used in the permanent works of this project shall be galvanized steel unless otherwise noted.

Section or Detail
Sheet where section or detail is shown.
Sheet where section or detail is taken.

SUNNYCOVE FRS
AS BUILT

7-E-23090

341

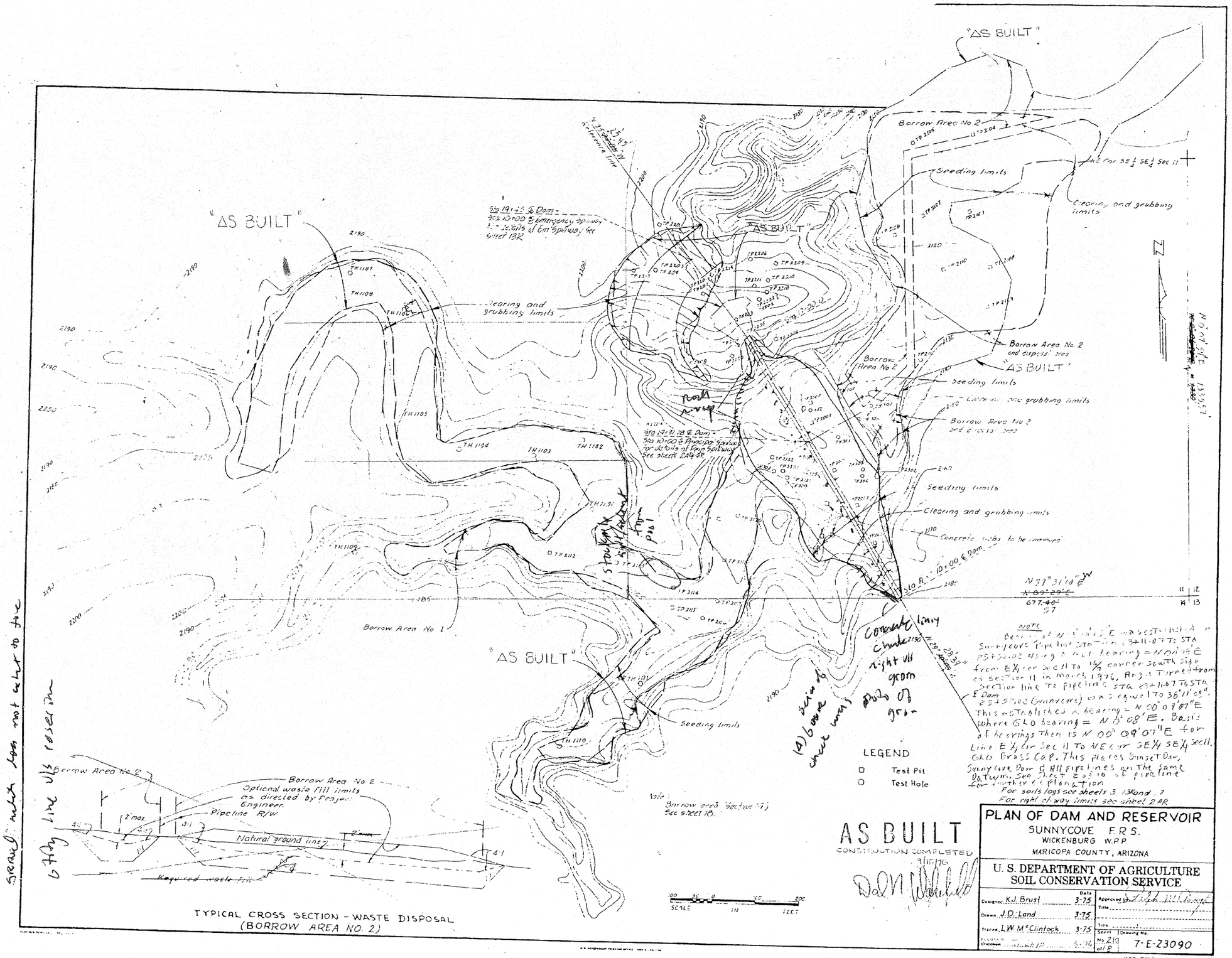
YAVAPAI COUNTY BOARD OF SUPERVISORS	WICKENBURG NATURAL RESOURCE CONSERVATION DISTRICT	TRIANGLE NATURAL RESOURCE CONSERVATION DISTRICT	MARICOPA COUNTY FLOOD CONTROL DISTRICT
APPROVED	APPROVED	APPROVED	APPROVED
DATE: 6-16-75 Mayor	DATE: 6/16/75 Chairman	DATE: 5-12-75 Secretary - Treasure	DATE: 6/16/75 Chairman
		DATE: 6/16/75 Chairman	DATE: 6/13/75 Chief Engineer

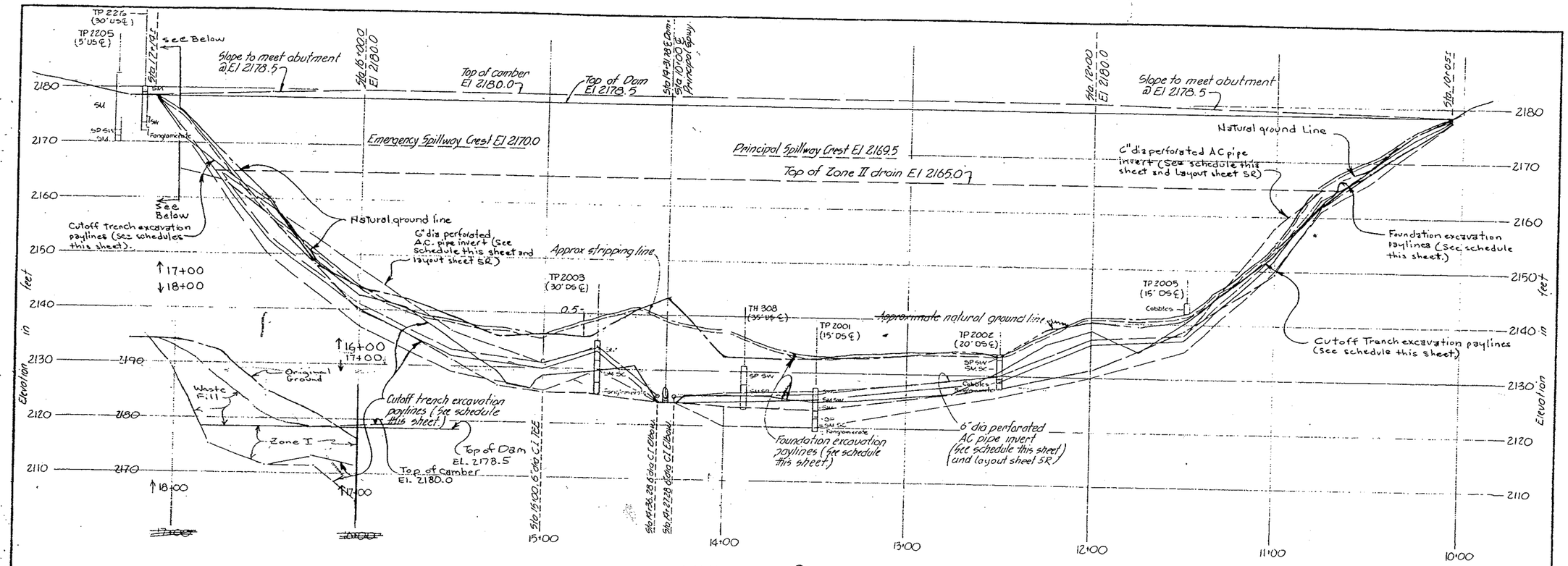
INDEX OF DRAWINGS
SUNNYCOVE F.R.S.
MARICOPA AND YAVAPAI COUNTIES, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date	Approved by:
Designed Soil Conservation Service	3-75	Title: Head E.B.W.R. Work
Drawn Soil Conservation Service	3-75	Title: State Conservation Engineer
Traced		Sheet No. 1 of 18
Checked		Drawing No. 7-E-23090

SCS-ENG-313-B Rev. 7-69


"AS BUILT"
Borrow Area No 2
Seeding limits
Clearing and grubbing limits
Borrow Area No. 2 and disposal area
Borrow Area No 2
Borrow Area No 1
Sta 19+73 ℄ Dam - Sta 10+00 ℄ Emergency Spillway
Sta 19+73.78 ℄ Dam - Sta 10+00 ℄ Principal Spillway
Concrete cubes to be removed
Stockpile
Concrete chute
Right of way
Seeding limits
Clearing and grubbing limits
N 00°09'07"E
677.40
57
NOTE
Borrow Area No 2
Optional waste fill limits as directed by Project Engineer.
Pipeline R/W
Natural ground line
Required waste fill
TYPICAL CROSS SECTION - WASTE DISPOSAL
(BORROW AREA NO. 2)
LEGEND
Test Pit
Test Hole
AS BUILT
CONSTRUCTION COMPLETED
SCALE IN FEET
PLAN OF DAM AND RESERVOIR
SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Designed K.J. Brust 3-75
Drawn J.D. Land 3-75
Traced L.W. McClintock 3-75
Approved
Title
Sheet No. 2 of 8
Drawing No. 7-E-23090
SCS-ENG-313-B Rev. 7-69



PROFILE ON ℄ DAM
(Looking downstream)

SCHEDULES

FOUNDATION EXCAVATION		
STATION	ELEVATION	
10+05 ±	~~2177.5~~	2177.7
10+50	~~2168.0~~	2168.9
10+75	~~2163.0~~	2162.1
11+00	~~2152.0~~	2153.1
11+50	~~2137.0~~	2139.8
12+00	~~2136.0~~	2138.5
12+50	~~2129.0~~	2131.1
——	——	
13+50	~~2125.0~~	2124.7
14+27.28	2124.1	
14+36.28	2124.1	
14+70	~~2133.0~~	2130.6
15+00	~~2130.0~~	2127.9
15+50	~~2132.0~~	2134.9
16+00	~~2140.0~~	2140.0
16+50	~~2153.0~~	2159.7
17+00	~~2174.0~~	2171.4
17+14 ±	~~2177.5~~	2172.4

[1] Elevations at ℄ Dam. Excavated surface in cross sections shall be to firm fanglomerate except from Sta 13+50 to Sta 14+50 where foundation excavation shall be uniformly shaped to the foundation excavation lines required for Principal spillway. See Profile on ℄ Principal Spillway sheet 4R

CUTOFF TRENCH EXCAVATION		
STATION	ELEVATION	
10+05 ±	~~2177.5~~	2177.4
10+50	~~2166.0~~	2168.0
~~10+75~~	~~2161.0~~	
11+00	~~2150.0~~	2149.4
11+50	~~2135.0~~	2139.0
12+00	~~2130.0~~	2131.5
12+50	~~2126.0~~	2127.7
13+50	~~2120.0~~	2123.3
14+00	~~2120.0~~	2123.3
14+27.28	2124.1	
14+36.28	2124.1	
14+70	~~2130.0~~	2124.3
15+00	~~2126.0~~	2125.5
15+50	~~2129.0~~	2131.0
16+00	~~2137.0~~	2139.9
16+50	~~2150.0~~	2157.0
17+00	~~2172.0~~	2165.1
17+14	~~2177.5~~	2171.3

[1] Elevation at 7.0' upstream from ℄ of Dam.

CUTOFF TRENCH EXCAVATION	
17+50	2171.3
17+80	2175.2
18+01	2189.7
18+17	2195.0

DRAIN PIPE PLACEMENT		
STATION	ELEVATION	REMARKS
10+75	~~2164.0~~ 2165.0	Plug beginning end with mortar or AC cap.
11+00	~~2153.0~~ 2158.0	Grade change
11+50	2138.0	Grade change
12+00	2137.0	Grade change
12+50	2130.0	Grade change
~~13+00~~	~~2128.0~~	~~Grade change~~
13+50	~~2126.0~~ 2122.6	Grade change
14+27.28	2125.1	CI Elbow to 6" dia non-perforated AC outlet pipe
14+36.28	2125.1	CI Elbow to 6" dia non-perforated AC outlet pipe
14+70	2134.0	Grade change. Plug begining ends with mortar or two AC caps.
15+00	2131.0	CI Tee to 6" dia non-perforated AC outlet pipe.
15+50	~~2133.0~~ 2136.5	Grade change
16+00	~~2141.0~~ 2147.5	Grade change
16+50	~~2154.0~~ 2158.5	Grade change
16+75	2164 ±	Plug beginning end with mortar or AC cap

[1] Elevation at 7.0' Downstream from ℄ of Dam.

[2] All Elevations and Stations are approximate and may be adjusted by Engineer.

Notes:

1. Unified soil descriptions are based on field identifications except where an asterisk is shown the classification has been based on laboratory analysis.
2. Complete field logs, laboratory test data and geologic report are available for review in the project office
3. For locations of test pits and test holes see sheet 2R
4. Test pit or test hole location DS = Downstream from ℄ Dam and US = Upstream from ℄ Dam.

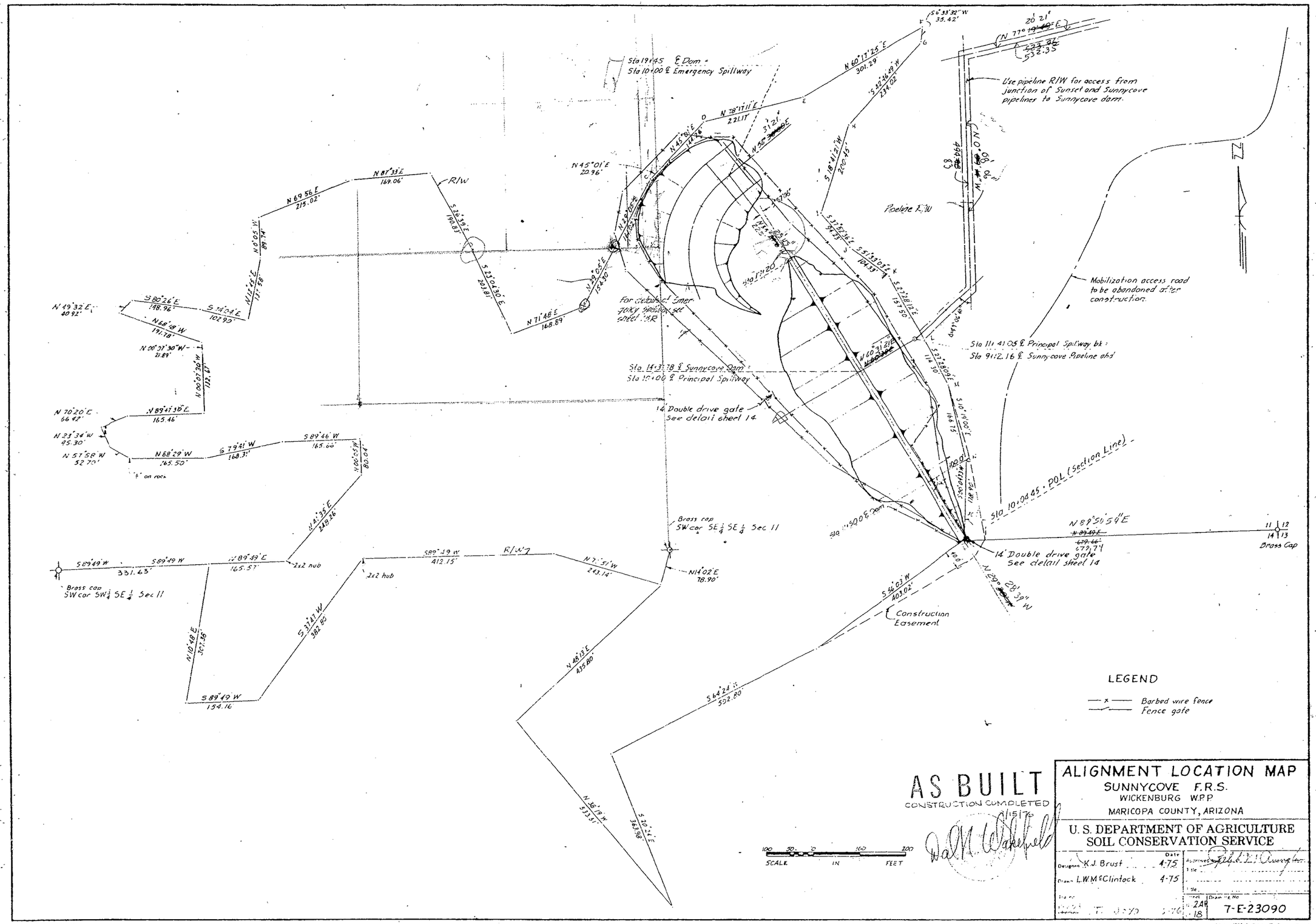
AS BUILT
CONSTRUCTION COMPLETED
9/15/76

PROFILE ON ℄ DAM AND EXCAVATION SCHEDULES
SUNNYCOVE F.R.S.
WICKENBURG, W.P.D.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed Greg Cunningham	3-75	Approved by	
Drawn Jack Long	4-75	Title	
Traced		Title	
Checked		Sheet 3 of 18	Drawing No. 7-E-23090

SCS-ENG-313-B Rev. 7-69



AS BUILT
CONSTRUCTION COMPLETED 1/15/76

ALIGNMENT LOCATION MAP

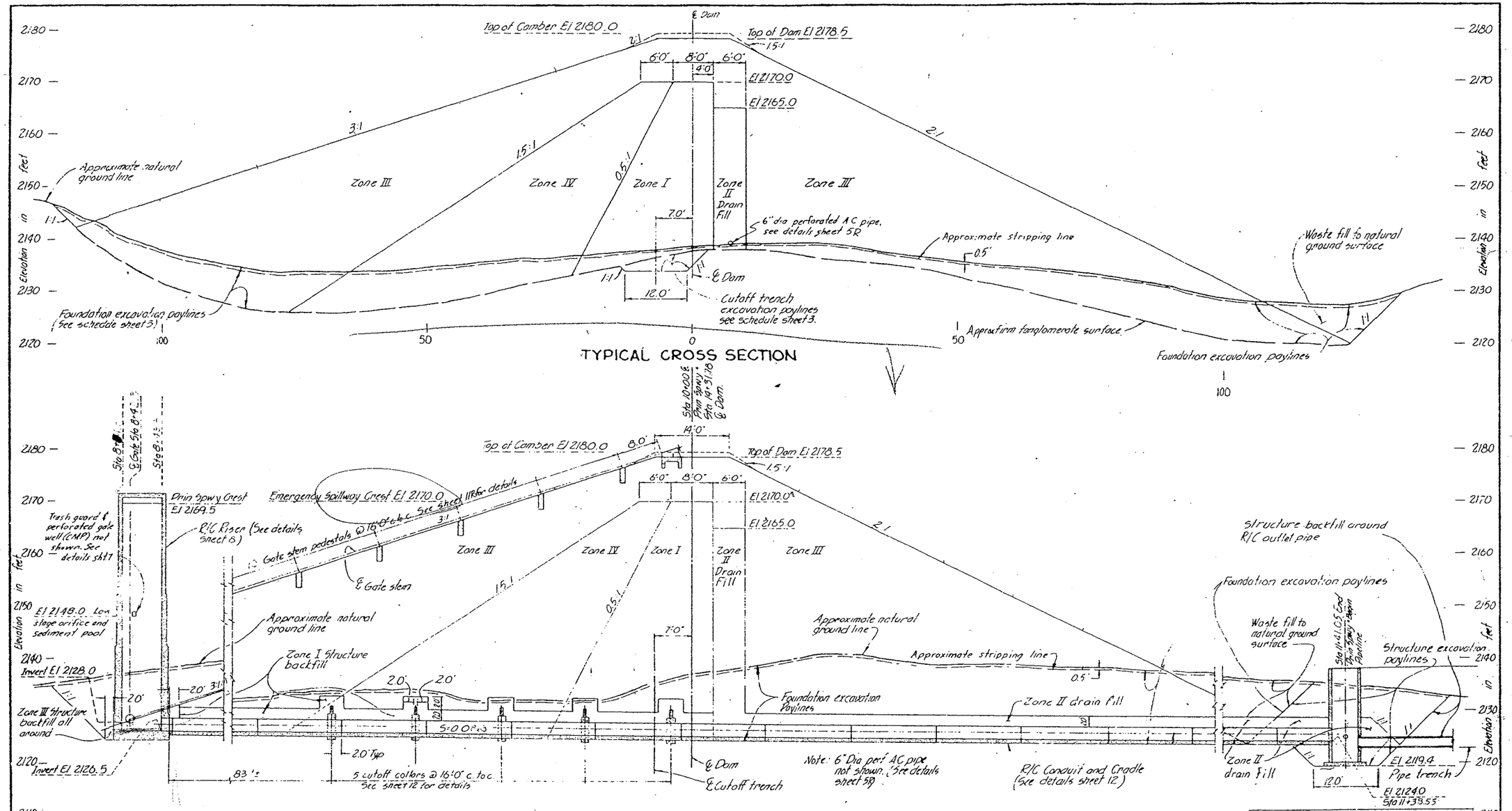
SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

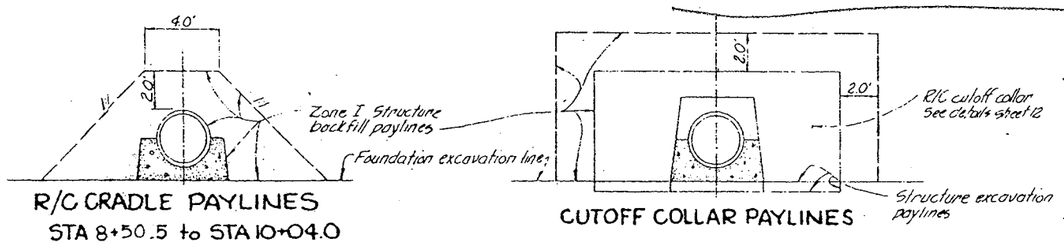
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

		Date
Designed	K.J. Brust	4-75
Drawn	L.W.McClintock	4-75
Traced	T. Lloyd	5-76

Approved: [illegible]
Title:
Title:
Sheet 2AR of 18
Drawing No. 7-E-23090

SCS-ENG-313-B Rev. 7-69





R/C CRADLE PAYLINES
STA 8+50.5 to STA 10+04.0

CUTOFF COLLAR PAYLINES

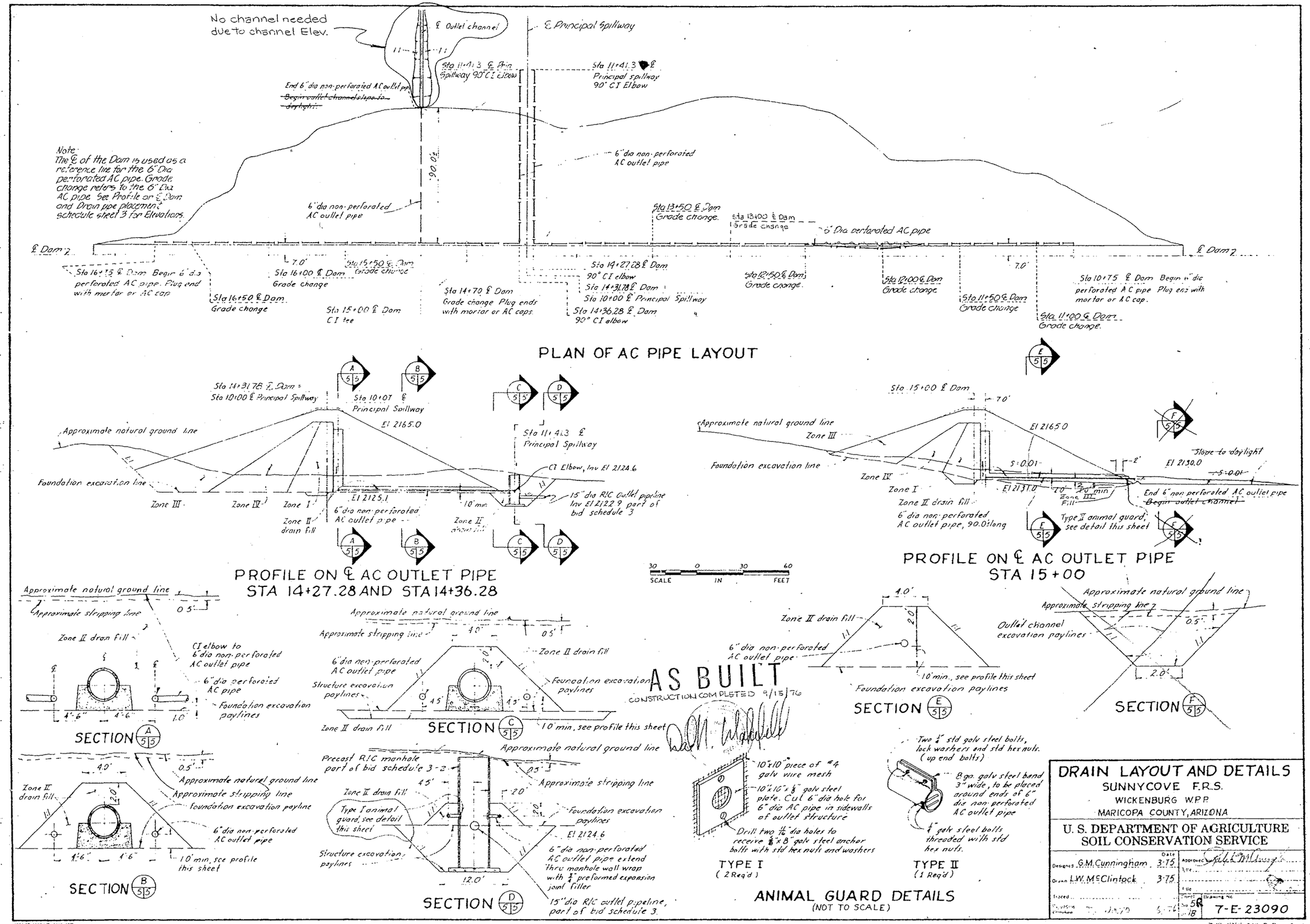
AS BUILT
CONSTRUCTION COMPLETED
9/15/76

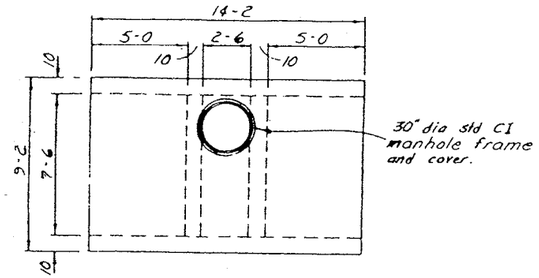
PROFILE ON ℄ PRINCIPAL SPILLWAY AND TYPICAL CROSS SECTION OF DAM
SUNNYCOVE F.R.S
WICKENBURG W.P.P
MARICOPA COUNTY, ARIZONA

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

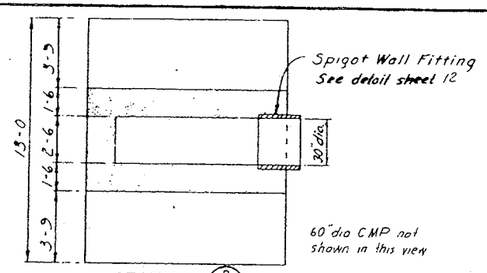
Designed Greg Cunningham 3-75
Drawn dock Wood 3-75
Traced
Checked 6-76
Approved
Title
Sheet No 4R of 18
Drawing No 7-E-23090

SCS-ENG-313-B Rev. 7-69

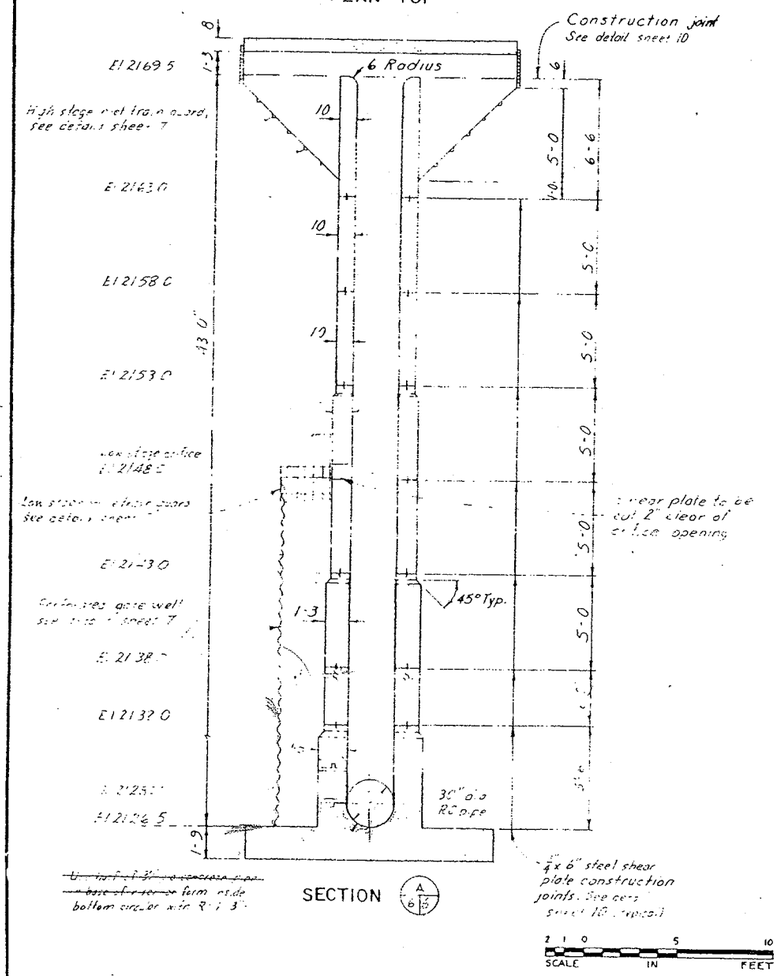

No channel needed due to channel Elev.
℄ Outlet channel
℄ Principal Spillway
Sta 11+41.3 ℄ Prin Spillway 90° CI Elbow
Sta 11+41.3 ℄ Principal spillway 90° CI Elbow
End 6" dia non-perforated AC outlet pipe
~~Begin outlet channel slope to daylight~~
Note
The ℄ of the Dam is used as a reference line for the 6" Dia perforated AC pipe. Grade change refers to the 6" Dia AC pipe. See Profile on ℄ Dam and Drain pipe placement schedule sheet 3 for Elevations.
90.0'
6" dia non-perforated AC outlet pipe
6" dia non-perforated AC outlet pipe
Sta 13+50 ℄ Dam Grade change.
Sta 13+00 ℄ Dam Grade change
6" Dia perforated AC pipe
℄ Dam
℄ Dam
Sta 16+75 ℄ Dam Begin 6" dia perforated AC pipe. Plug end with mortar or AC cap
7.0'
Sta 16+00 ℄ Dam Grade change
Sta 15+50 ℄ Dam Grade change
Sta 16+50 ℄ Dam Grade change
Sta 15+00 ℄ Dam CI Tee
Sta 14+70 ℄ Dam Grade change Plug ends with mortar or AC caps.
Sta 14+27.28 ℄ Dam 90° CI elbow
Sta 14+31.78 ℄ Dam
Sta 10+00 ℄ Principal Spillway
Sta 14+36.28 ℄ Dam 90° CI elbow
Sta 12+50 ℄ Dam Grade change.
Sta 12+00 ℄ Dam Grade change.
7.0'
Sta 11+50 ℄ Dam Grade change
Sta 11+00 ℄ Dam Grade change.
Sta 10+75 ℄ Dam Begin 6" dia perforated AC pipe Plug end with mortar or AC cap.
PLAN OF AC PIPE LAYOUT
Sta 14+31.78 ℄ Dam = Sta 10+00 ℄ Principal Spillway
Sta 10+07 ℄ Principal Spillway
El 2165.0
Approximate natural ground line
Foundation excavation line
Zone III
Zone IV
Zone I
Zone II drain fill
El 2125.1
6" dia non-perforated AC outlet pipe
10' min
Zone II drain fill
Sta 11+41.3 ℄ Principal Spillway
CI Elbow, Inv El 2124.6
15" dia RJC Outlet pipeline Inv El 2122.9 part of bid schedule 3
PROFILE ON ℄ AC OUTLET PIPE STA 14+27.28 AND STA 14+36.28
Sta 15+00 ℄ Dam
7.0'
Approximate natural ground line
Zone III
Foundation excavation line
Zone IV
Zone I
Zone II drain fill
6" dia non-perforated AC outlet pipe, 90.0' long
El 2165.0
S=0.01
El 2131.0
7.0'
3.0' min
Zone III Fill
Slope to daylight
El 2130.0
S=0.01
End 6" non-perforated AC outlet pipe
~~Begin outlet channel~~
Type II animal guard, see detail this sheet
PROFILE ON ℄ AC OUTLET PIPE STA 15+00
SCALE 30 0 30 60 IN FEET
Approximate natural ground line
Approximate stripping line
0.5'
Zone II drain fill
CI elbow to 6" dia non-perforated AC outlet pipe
6" dia perforated AC pipe
Foundation excavation paylines
4'-6"
4'-6"
1.0'
SECTION A 5/5
Approximate natural ground line
Approximate stripping line
4.0'
0.5'
Zone II drain fill
6" dia non-perforated AC outlet pipe
Structure excavation paylines
Foundation excavation paylines
4.5'
4.5'
Zone II drain fill
10' min, see profile this sheet
SECTION C 5/5
4.0'
Zone II drain fill
6" dia non-perforated AC outlet pipe
10' min., see profile this sheet
Foundation excavation paylines
SECTION E 5/5
Approximate natural ground line
Approximate stripping line
Outlet channel excavation paylines
2.0'
SECTION F 5/5
AS BUILT
CONSTRUCTION COMPLETED 9/15/76
4.0'
0.5'
Zone II drain fill
Approximate natural ground line
Approximate stripping line
Foundation excavation payline
6" dia non-perforated AC outlet pipe
1'-6"
1'-6"
10' min, see profile this sheet
SECTION B 5/5
Precast RJC manhole part of bid schedule 3-2
Zone II drain fill
Type I animal guard, see detail this sheet
Structure excavation paylines
Approximate natural ground line
0.5'
Approximate stripping line
Foundation excavation paylines
El 2124.6
6" dia non-perforated AC outlet pipe extend thru manhole wall wrap with 1/4" preformed expansion joint filler
12.0'
15" dia RJC outlet pipeline, part of bid schedule 3.
SECTION D 5/5
10"x10" piece of #4 galv wire mesh
10"x10"x 3/8" galv steel plate. Cut 6" dia hole for 6" dia AC pipe in sidewalls of outlet structure
Drill two 11/16" dia holes to receive 3/8"x8" galv steel anchor bolts with std hex nuts and washers
TYPE I
(2 Req'd)
Two 1/4" std galv steel bolts, lock washers and std hex nuts. (up end bolts)
8 ga. galv steel band 3" wide, to be placed around ends of 6" dia non-perforated AC outlet pipe
1/4" galv steel bolts threaded with std hex nuts.
TYPE II
(1 Req'd)
ANIMAL GUARD DETAILS
(NOT TO SCALE)
DRAIN LAYOUT AND DETAILS
SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Designed G.M. Cunningham 3-75
Drawn L.W. McClintock 3-75
Approved
Sheet 58 of 18
Drawing No. 7-E-23090
SCS ENG-313-B (Rev. 7-69)



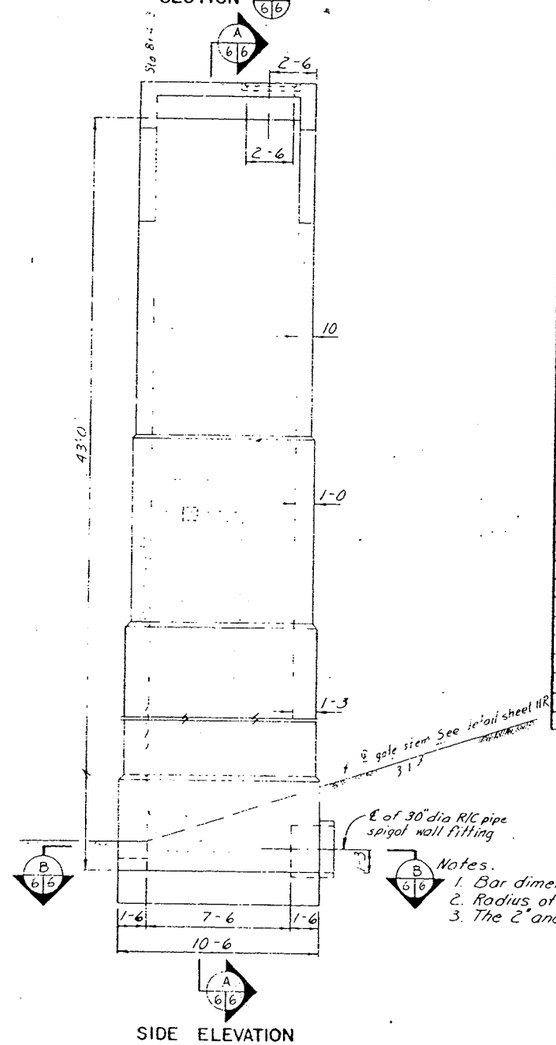
PLAN-TOP



SECTION B 6/6



SECTION A 6/6



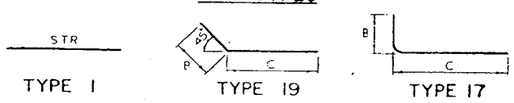
SIDE ELEVATION A 6/6

Notes.
1. Bar dimensions are out to out of bar.
2. Radius of bends equals 3 bar diameters for sizes equal to or less than #7.
3. The 2" and 3" dimensions from face of concrete to steel are clear distances.

STEEL SCHEDULE

Mark	Size	Quantity	Length	Type	B	C	Total Length	Mark	Size	Quantity	Length	Type	B	C	Total Length
B1	#7	14	12-6	1			175-0	R28	#5	28	8-0	21	2-9	5-3	224-0
B2	#6	13	10-0	1			130-0								
B3	#7	40	8-0	17	3-8	6-4	320-0								
B4	#6	14	10-0	1			140-0								
B5	#6	10	12-6	1			125-0								
B6	#6	4	5-6	1			22-0								
B7	#6	7	5-6	17	1-2	4-4	52-6	T1	#5	18	6-0	1			108-0
B8	#6	19	7-9	17	1-0	6-0	147-3	T2	#5	6	8-0	1			48-0
B9	#5	14	8-6	1			119-0	T3	#5	4	4-9	1			19-0
B10	#6	6	3-6	1			21-6	T4	#5	4	3-6	1			14-0
B11	#6	3	2-6	1			7-6	T5	#5	4	2-3	1			9-0
B12	#6	2	2-6	1			5-0	T6	#5	4	9-0	19	2-0	7-0	36-0
B13	#6	6	4-4	1			26-0	T7	#5	12	8-3	1			99-0
B14	#7	20	4-8	1			93-4	T8	#5	2	3-3	1			6-6
B15	#6	10	7-3	17	1-1	6-2	72-6	T9	#5	2	5-9	1			11-6
B16	#6	14	9-9	17	3-7	6-2	136-6	T10	#5	2	10-9	1			21-6
B17	#6	4	9-3	17	3-4	5-11	37-0	T11	#5	2	13-3	1			26-6
								T12	#5	14	6-3	1			87-6
								T13	#5	6	8-0	1			48-0
								T14	#5	4	6-0	1			24-0
								T15	#5	4	4-9	1			19-0
								T16	#5	4	3-6	1			14-0
R1	#6	22	7-9	1			174-6	T17	#5	4	2-3	1			9-0
R2	#5	32	8-6	1			272-0	T18	#5	4	9-0	19	2-0	7-0	36-0
R3	#6	16	3-6	1			56-0	T19	#5	24	8-0	17	2-9	5-3	192-0
R4	#6	28	12-6	1			350-0	T20	#5	2	3-3	1			6-6
R5	#7	64	9-6	17	3-6	6-0	608-0	T21	#5	2	5-9	1			11-6
R6	#5	20	8-6	1			170-0	T22	#5	2	8-3	1			16-6
R7	#6	10	3-6	1			35-0	T23	#5	2	10-9	1			21-6
R8	#6	26	4-0	1			104-0	T24	#5	2	13-3	1			26-6
R9	#6	36	9-3	17	3-4	5-11	333-0	T25	#5	4	13-9	1			55-0
R10	#6	4	8-9	17	3-1	5-8	35-0	T26	#5	4	13-9	1			55-0
R11	#5	22	11-9	1			230-6	T27	#4	14	8-3	1			115-6
R12	#5	20	8-6	1			170-0	T28	#4	2	4-9	1			9-6
R13	#5	10	3-6	1			35-0	T29	#4	7	13-9	1			96-3
R14	#5	26	9-6	1			247-0	T30	#4	4	5-3	1			21-0
R15	#7	40	9-0	17	3-3	5-9	360-0	T31	#5	24	6-9	17	1-6	5-3	162-0
R16	#6	14	8-3	1			115-6	T32	#5	2	6-6	17	1-6	5-0	13-0
R17	#5	10	3-6	1			35-0	T33	#5	2	2-6	17	1-6	1-0	5-0
R18	#5	20	3-8	1			73-4	T34	#4	7	13-9	1			96-3
R19	#5	36	8-3	17	2-10½	5-4½	297-0	T35	#4	4	5-3	1			21-0
R20	#5	4	8-0	17	2-9	5-5	32-0								
R21	#5	20	11-9	1			235-0	S1	#5	88	2-0	1			176-0
R22	#6	14	8-3	1			115-6								
R23	#5	8	3-3	1			26-0								
R24	#5	20	11-9	1			235-0								
R25	#5	40	8-0	17	2-9	5-3	320-0								
R26	#5	10	8-3	1			82-6								
R27	#5	8	3-3	1			26-0								

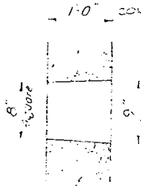
BAR TYPES



AS BUILT

CONSTRUCTION COMPLETED 9/15/76

Walt M. Wakefield



Low stage orifice
Invert El 2148.0

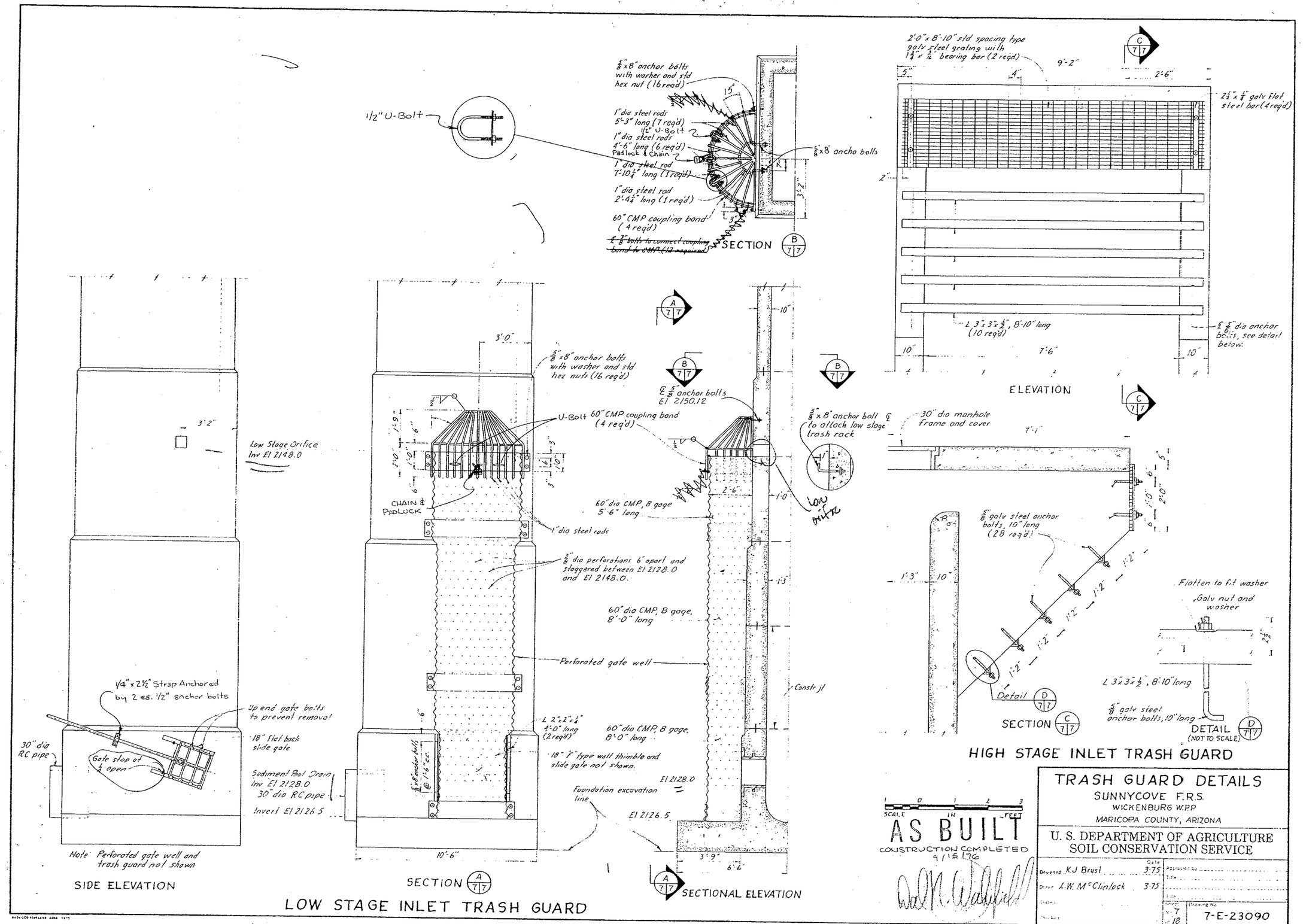
ORIFICE DETAIL
(NOT TO SCALE)

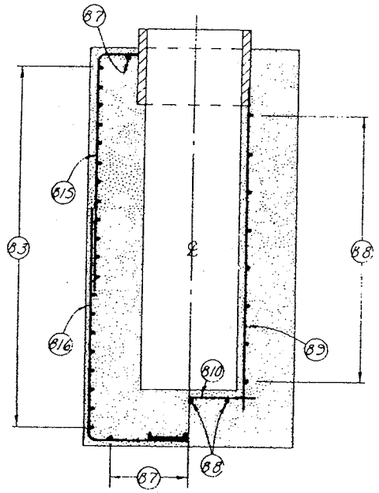
RISER LAYOUT DETAILS

SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust	Date 3-75	Approved Ralph Williams	
Drawn L.W. McClintock	3-75	Title	
Traced		Title	
Checked		Sheet No. 6A of 12	Drawing No. 7-E-23090

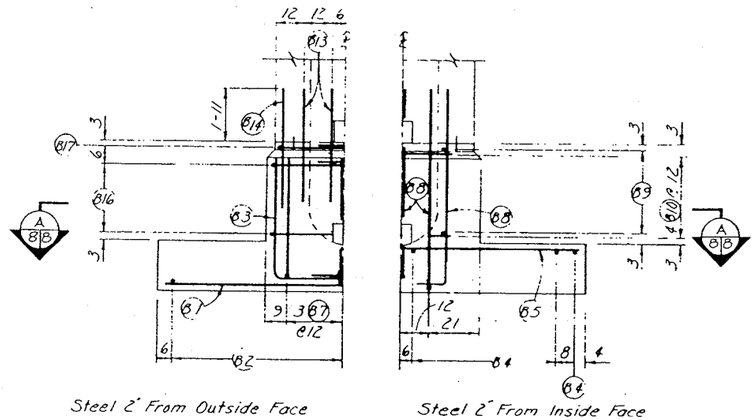

1/2" U-Bolt
5/8"x8" anchor bolts with washer and std hex nut (16 req'd)
15°
1" dia steel rods 5'-3" long (7 req'd)
1/2" U-Bolt
1" dia steel rods 4'-6" long (6 req'd)
Padlock & Chain
1" dia steel rod 7'-10 1/4" long (1 req'd)
1" dia steel rod 2'-4 1/4" long (1 req'd)
60" CMP coupling band (4 req'd)
5/8"x8" ancho bolts
3'-2"
3'
SECTION B 7|7
2'-0"x 8'-10" std spacing type galv steel grating with 1 3/4" x 3/16" bearing bar (2 req'd)
9'-2"
5"
4"
2'-6"
2 1/2" x 1/4" galv flat steel bar (4 req'd)
2"
L 3"x3"x1/2", 8'-10" long (10 req'd)
7'-6"
10"
℄ 5/8" dia anchor bolts, see detail below.
ELEVATION C 7|7
Low Stage Orifice Inv El 2148.0
3'-2"
1/4" x 2 1/2" Strap Anchored by 2 ea. 1/2" anchor bolts
Up end gate bolts to prevent removal
18" Flat back slide gate
30" dia RC pipe
Gate stop at 1/2 open
Sediment Pool Drain Inv El 2128.0 30" dia RC pipe
Invert El 2126.5
Note: Perforated gate well and trash guard not shown
SIDE ELEVATION
3'-0"
5/8" x 8" anchor bolts with washer and std hex nuts (16 req'd)
U-Bolt
60" CMP coupling band (4 req'd)
CHAIN & PADLOCK
1" dia steel rods
5/8" dia perforations 6" apart and staggered between El 2128.0 and El 2148.0.
60" dia CMP, 8 gage 8'-0" long
Perforated gate well
L 2"x2"x1/4" 4'-0" long (2 req'd)
60" dia CMP, 8 gage, 8'-0" long
18" F type wall thimble and slide gate not shown.
El 2128.0
Foundation excavation line
El 2126.5
10'-6"
SECTION A 7|7
℄ 5/8" anchor bolts El 2150.12
60" dia CMP, 8 gage 5'-6" long
Low Mitre
Constr jt
3'-9"
6'-6"
SECTIONAL ELEVATION A 7|7
5/8" x 8" anchor bolt ℄ to attach low stage trash rack
30" dia manhole frame and cover
7'-1"
5/8" galv steel anchor bolts, 10" long (28 req'd)
1'-3"
Detail D 7|7
SECTION C 7|7
Flatten to fit washer
Galv nut and washer
L 3"x3"x1/2", 8'-10" long
5/8" galv steel anchor bolts, 10" long
DETAIL (NOT TO SCALE) D 7|7
HIGH STAGE INLET TRASH GUARD
LOW STAGE INLET TRASH GUARD
SCALE IN FEET
AS BUILT
CONSTRUCTION COMPLETED 9/15/76
TRASH GUARD DETAILS
SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
Designed K.J. Brust 3-75
Drawn L.W. McClintock 3-75
Approved by
Sheet 7 of 18
Drawing No. 7-E-23090
SCS ENG-313-B Rev. 7-69



Outside Steel Inside Steel

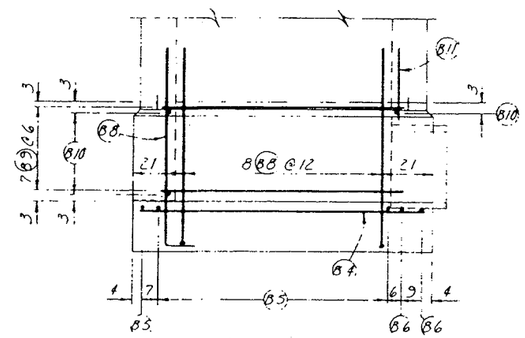
SECTION A/B8B

(NOT TO SCALE)



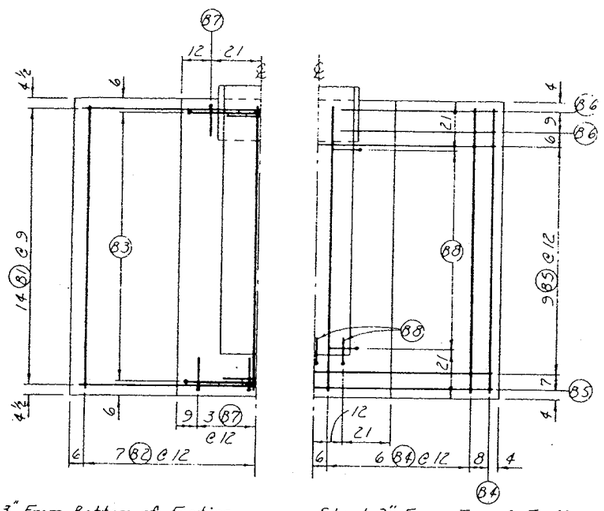
Steel 2" From Outside Face Steel 2" From Inside Face

UPSTREAM ELEVATION



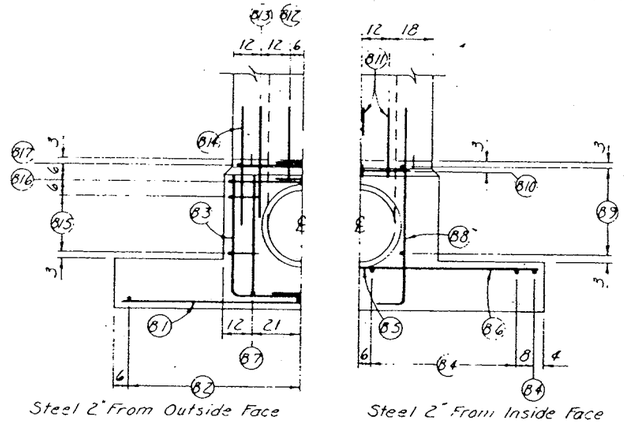
Steel 2" From Inside Face and 2" From Top of Footing

SIDEWALL ELEVATION



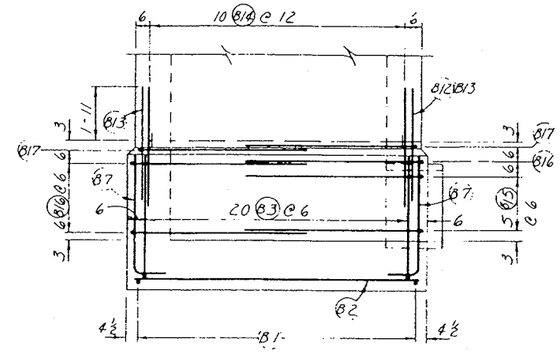
Steel 3" From Bottom of Footing Steel 2" From Top of Footing

PLAN—FOOTING



Steel 2" From Outside Face Steel 2" From Inside Face

DOWNSTREAM ELEVATION



Steel 2" From Outside Face and 3" From Bottom of Footing

SIDEWALL ELEVATION

AS BUILT

CONSTRUCTION COMPLETED 9/15/76

SCALE 3 2 1 0 5 IN FEET

RISER DETAILS

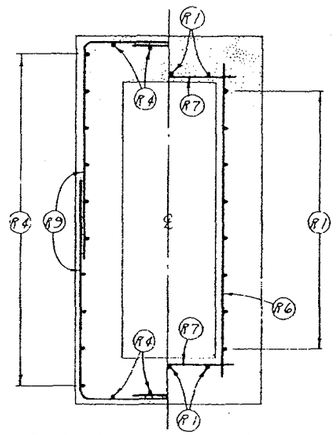
SUNNYCOVE F.R.S.

WICKENBURG W.P.P.

MARICOPA COUNTY, ARIZONA

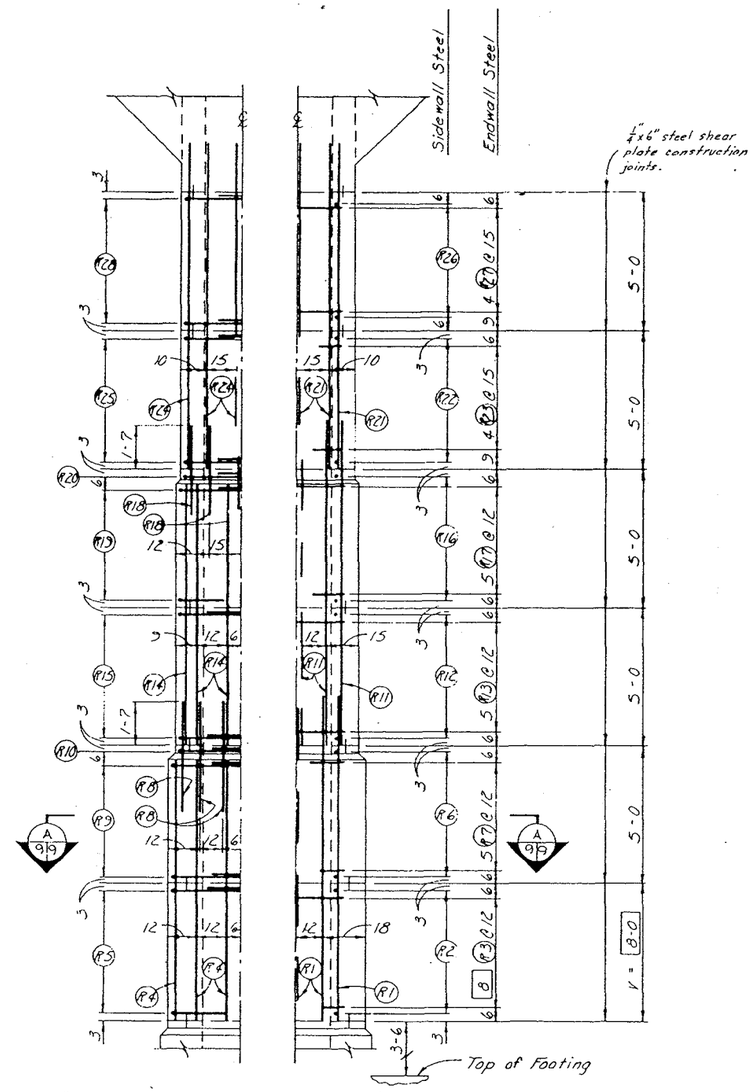
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust	Date 3-75	Approved by	
Drawn		Title	
Traced		Title	
Checked		Sheet No. 8 of 18	Drawing No. 7-E-23090



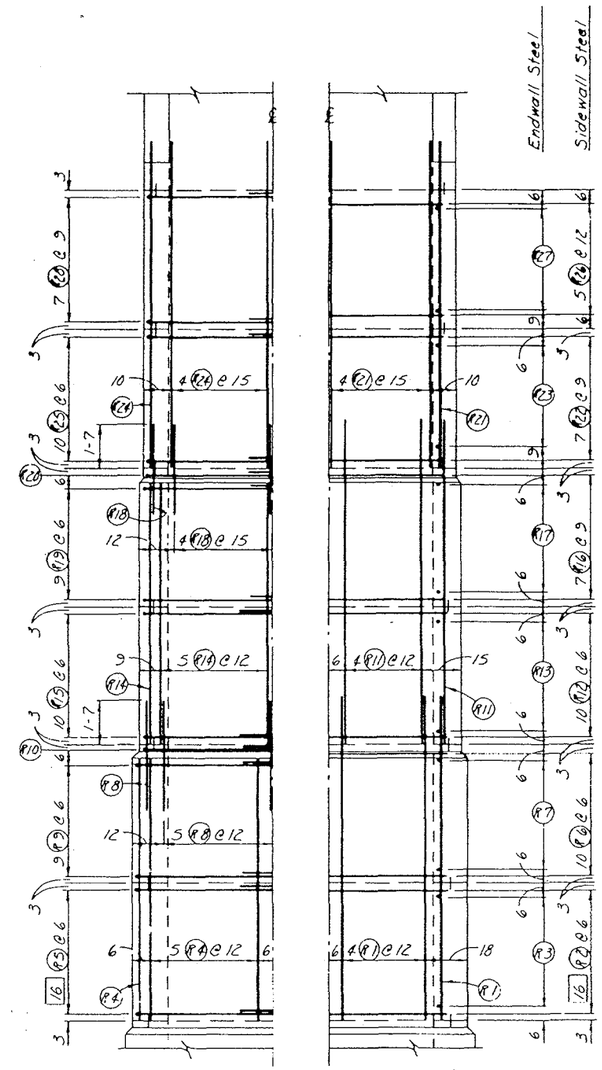
SECTION A/9

(NOT TO SCALE)



Steel 2" From Outside Face Steel 2" From Inside Face

ENDWALL ELEVATION



Steel 2" From Outside Face Steel 2" From Inside Face

SIDEWALL ELEVATION

AS BUILT
CONSTRUCTION COMPLETED
9/18/76

SCALE 3 2 1 0 5 IN FEET

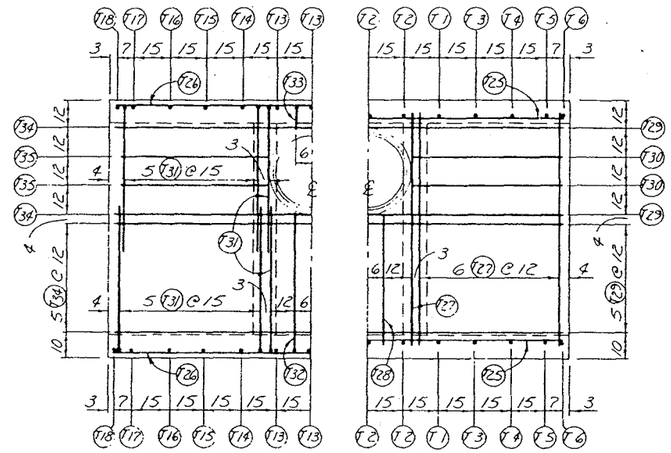
RISER DETAILS

SUNNYCOVE F.R.S.
WICKENBURG WPP
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

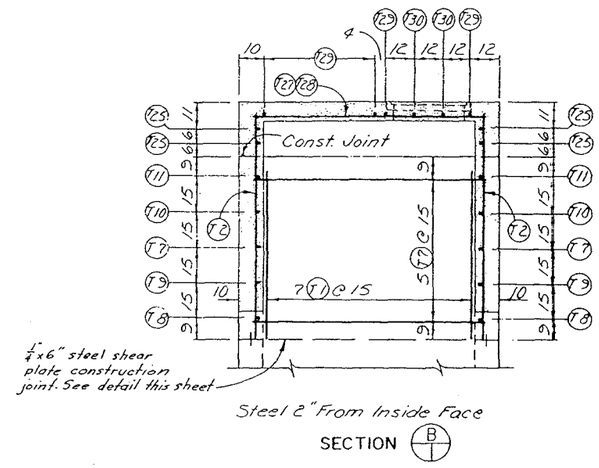
Designed K J Brust	Date 3-75	Approved by
Drawn		Title
Traced		Title
Checked		Sheet No. 9 of 18 — Drawing No. 7-E-23090

FORM SCS 313 (APRIL 1963)

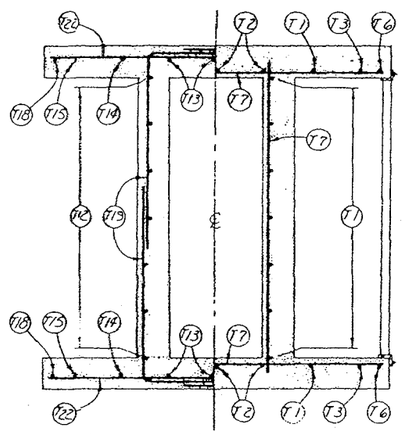


PLAN-COVER SLAB

Riser Sidewall Steel Not Shown

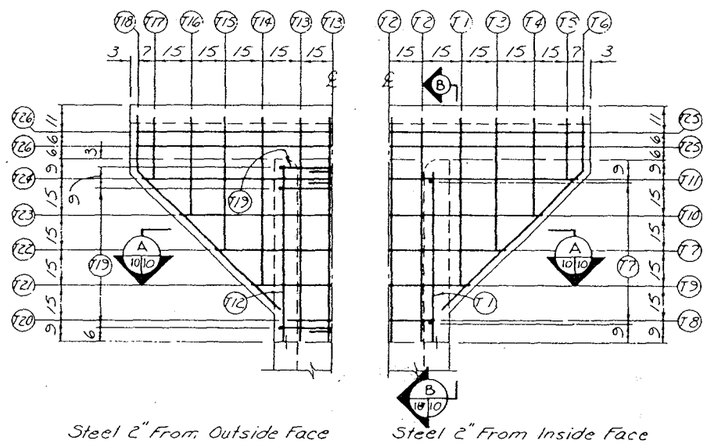


SECTION B



SECTION A 10/10

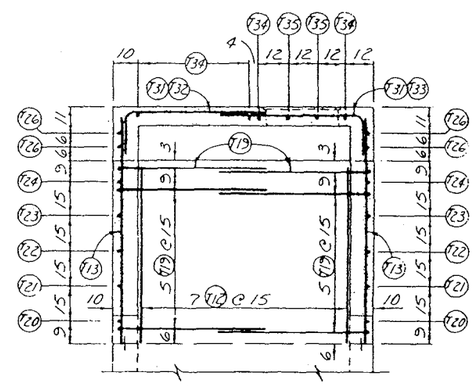
(NOT TO SCALE)



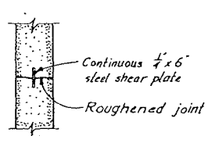
ENDWALL ELEVATION

Cover Slab Steel Not Shown

SCALE 3 2 1 0 5 IN FEET



SECTION B 10/10



TYPICAL CONST JOINT DETAIL

(NOT TO SCALE)

AS BUILT

CONSTRUCTION COMPLETED 9/15/76

RISER DETAILS

SUNNYCOVE F.R.S.

WICKENBURG W.P.P.

MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

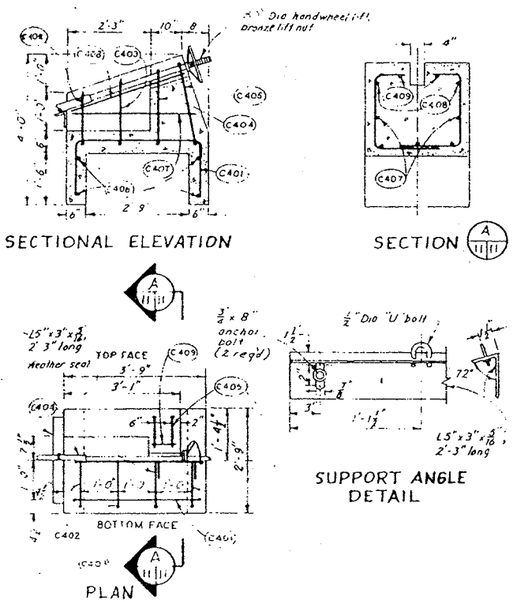
Designed K.J. Brust	Date 3-75	Approved by	
Drawn		Title	
Traced		Title	
Checked		Sheet No 10 of 18	Drawing No 7-E-23090

Notes
Fill stem encasement with approximately 14.0 Gallons of SAE 20 motor oil.

The stainless steel portion of the gate stem shall extend a sufficient length on each end so that no part of the carbon steel stem is exposed during operation of the gate. A carbon steel stem will be used between the upper and the lower splices inside the encasement.

STEEL SCHEDULES

LOCATION	MARK	SIZE	QUAN	LENGTH	TYPE	A	B	C	TOTAL LENGTH
GATE STEM PEDESTALS (9 req'd)									
	P401	4	27	3'-0"	TI	0'-2"	0'-8"	0'-8"	81'-0"
	P402	4	18	2'-3"	str				40'-6"
	P403	4	18	2'-9"	str				49'-6"
GATE LIFT PEDESTAL									
	C401	4	3	6'-3"	2	1'-6	3'-3"	1'-6"	18'-9"
	C402	4	1	4'-6"	2	1'-2"	2'-2"	1'-2"	4'-6"
	C403	4	1	5'-0"	2	1'-5"	2'-2"	1'-5"	5'-0"
	C404	4	2	4'-0"	2	1'-6"	1'-9"	0'-9"	8'-0"
	C405	4	2	4'-3"	2	1'-6"	2'-0"	0'-9"	8'-6"
	C406	4	4	2'-3"	Str				9'-0"
	C407	4	2	3'-0"	Str				6'-0"
	C408	4	2	2'-9"	Str				5'-6"
	C409	4	2	0'-6"	Str				1'-0"



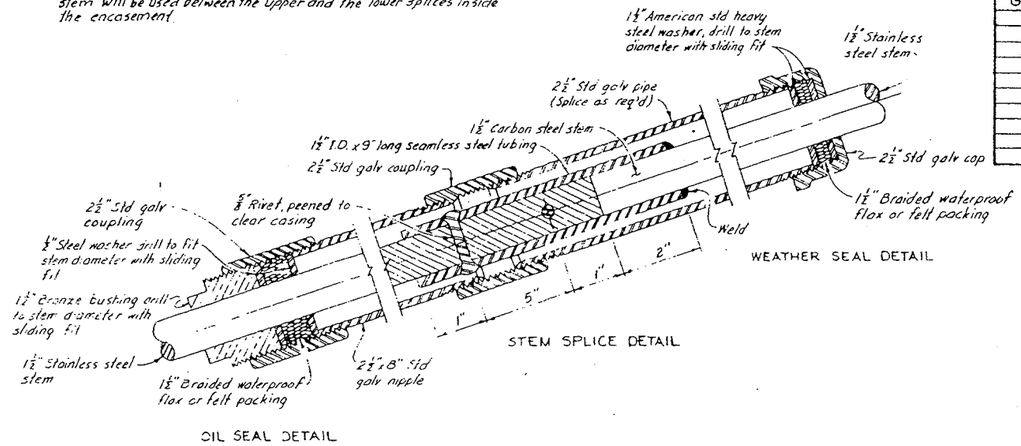
SECTIONAL ELEVATION

SECTION A

SUPPORT ANGLE DETAIL

PLAN

GATE LIFT PEDESTAL DETAILS

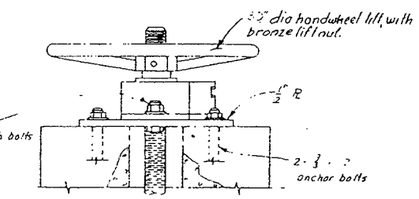


WEATHER SEAL DETAIL

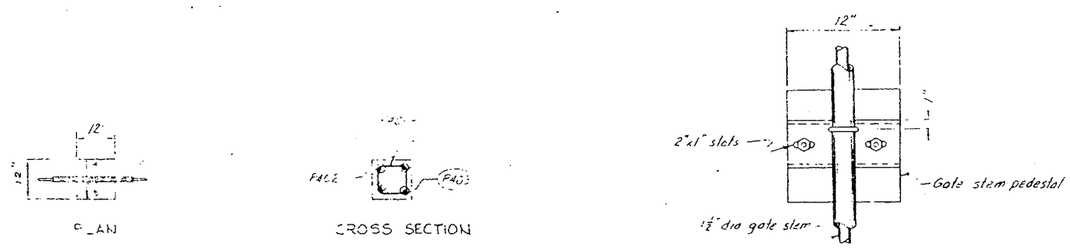
STEM SPLICE DETAIL

OIL SEAL DETAIL

GATE STEM SPLICE



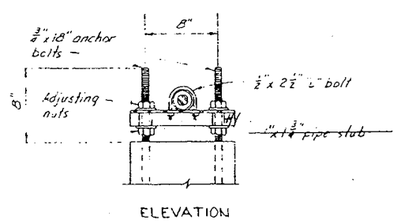
ELEVATION



SIDE ELEVATION

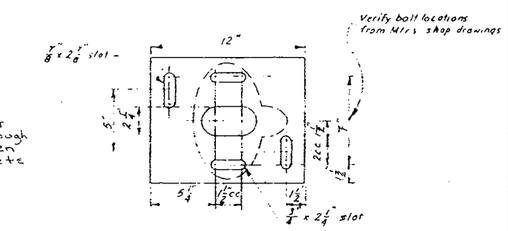
SECTIONAL ELEVATION

GATE STEM PEDESTAL



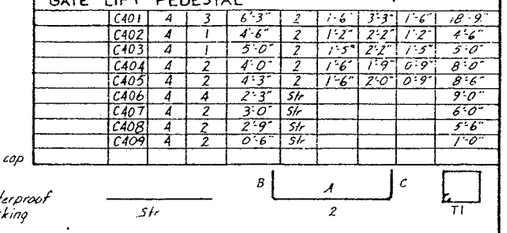
ELEVATION

GATE STEM GUIDE



PLAN

BASE PLATE
(For gate lift pedestal)



BAR TYPES

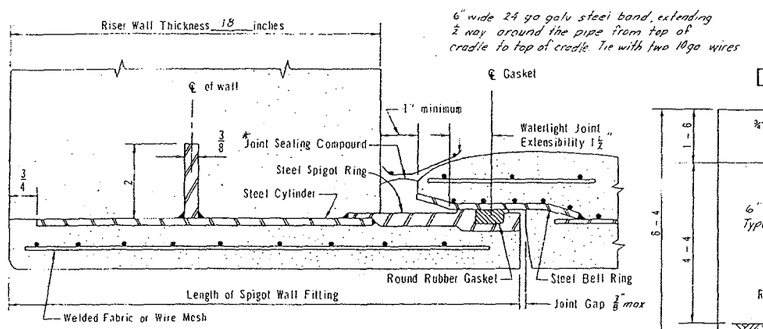
AS BUILT
CONSTRUCTION COMPLETED
9/15/76

GATE CONTROL DETAILS
SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust 3-75
Drawn L.W. McClintock 3-75
Traced
Revised
Approved
Date
Title
Sheet No. 11R of 18
Drawing No. 7-E-23090

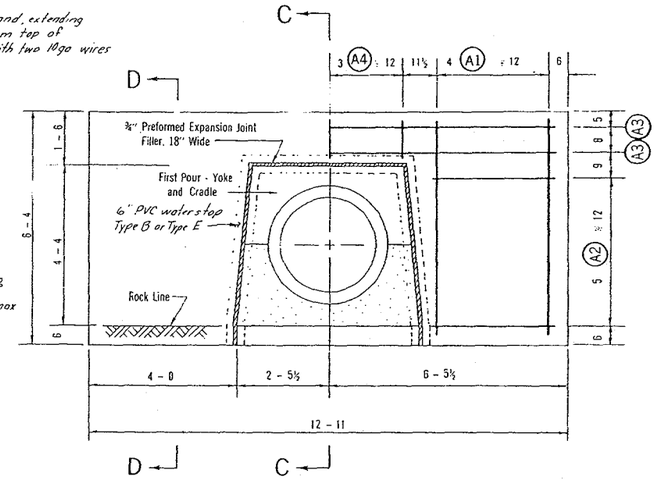
SCS ENG 413-B Rev. 7-69



6" wide 24 ga galv steel band, extending ½ way around the pipe from top of cradle to top of cradle. Tie with two 10ga wires

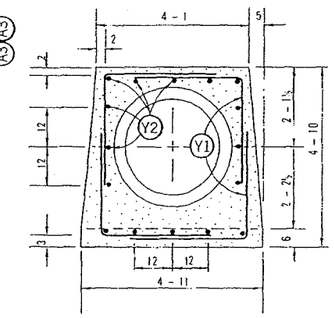
DETAIL A

* ¾" dia circular cross section, Butyl rubber sealer, Federal Specification SS-S-00210 applied over the manufactures recommended primer.



DETAIL OF ANTI-SEEP COLLAR

Yoke steel not shown.



DETAIL OF ANTI-SEEP COLLAR YOKE

STEEL SCHEDULE

Anti-seep Collar and Yoke. 5 Required.

Mark	Size	Quantity per Collar	Length	Type	Total Quantity	Total Length
A1	4	8	5 - 10	1	40	233-4
A2	4	10	3 - 6	1	50	175-0
A3	4	2	12 - 5	1	10	124-2
A4	4	5	1 - 0	1	25	25-0
Y1	4	12	5 - 8	21	60	340-0
Y2	4	16	1 - 2	1	80	93-4

QUANTITIES

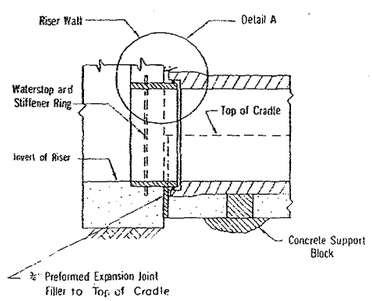
Concrete	Cu. Yds.
Anti-seep Collar including Yoke	
* Each, Additional to Cradle	1.91
Total	9.55
Cradle	
** Per Lineal Foot of Cradle	0.26
Total	75.0
Steel	**Pounds**
Anti-seep Collar including Yoke	662
Cradle	—

Concrete quantities are based on an outside diameter of pipe of 5 inches. Steel quantities do not change with outside diameter of pipe.

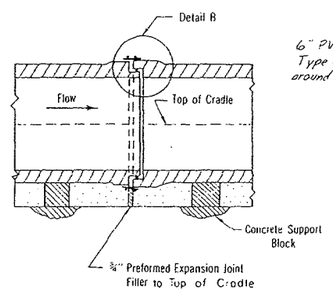
* This quantity is given by
1.95 - 0.0001515 (D1 - 38) (D1 - 38) cu yds

** This quantity is given by
0.2392 - 0.0001010 (D1 - 38) (D1 - 38) cu yds

D1 = outside diameter of pipe furnished, inches

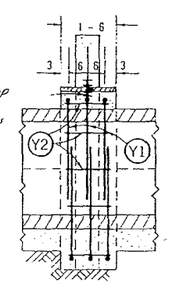


DETAIL OF SPIGOT WALL FITTING



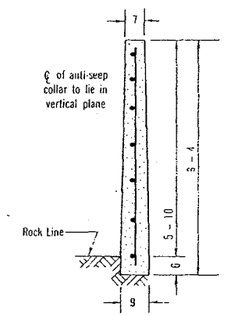
DETAIL OF PIPE JOINT

6" PVC waterstop Type B or Type E around top and sides

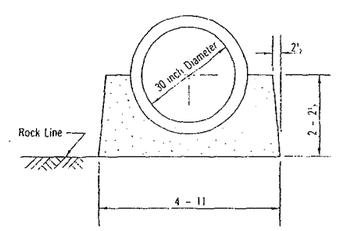


SECTION C-C

Anti-seep collar steel not shown.

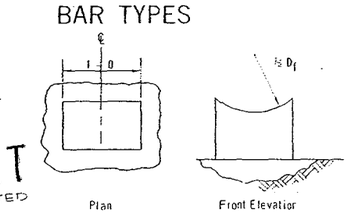


SECTION D-D



DETAIL OF CRADLE

BAR TYPES

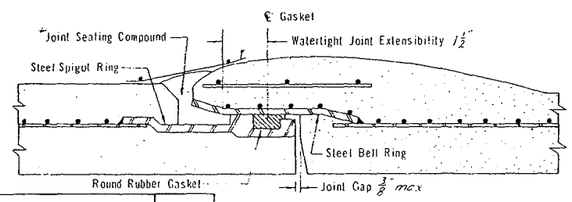


SUGGESTED SUPPORT BLOCKS

Sufficient blocks shall be provided to support the pipe to the required line and grade. The Contractor shall determine the number and size of blocks required. Wedges may be used as an alternate.

AS BUILT
CONSTRUCTION COMPLETED 9/15/76

6" wide 24 ga galv steel band all around Tie with two 10 ga wires



DETAIL B

DATE	REVISION	BY
1-73	Waterstop added to anti-seep collar	L.W.M.

STANDARD CONDUIT DETAILS
FOR
REINFORCED CONCRETE PRESSURE PIPE
PRINCIPAL SPILLWAY

STANDARD DWG. NO. ES-5030-CR

DATE 2-70 SHEET 1 OF 1

Joint length equals watertight joint extensibility plus joint gap.

The pipe shall be drawn together so that the maximum joint gap does not exceed ⅜ inch for pipe laid on a straight line. For cambered pipe or pipe laid on a curved line, the joint gap at the closest point shall not exceed ¾ inch.

JOINT REQUIREMENTS

Length of Pipe Section	Minimum Joint Length	Minimum Joint Limiting Angle	
feet	inches	radians	degrees
16.0	1⅞		0°45'

For pipe length other than shown, joint requirements will be determined by the Engineer.

Where pipes of different length are connected, adjoining pipes shall meet the requirements of the longer pipe.

Prior to delivery of pipe, the pipe joint detail proposed for use shall be submitted to the Engineer for approval.

STRENGTH REQUIREMENTS

Inside Diameter of Pipe	Internal Load	External Load	
	Hydrostatic Pressure	Minimum 3-Edge Bearing Strength in Pounds per Lineal Foot of Pipe	
		Applicable Standard Specification	
		AWWA C-301	AWWA C-300
	Head of Water	Load to produce 0.001 inch crack one foot long	Load to produce 0.01 inch crack one foot long
inches	feet		
30	55	9200	12,500

The outside diameter of pipe assumed in design is 35 inches. Where the pipe furnished has an outside diameter greater than assumed in design, the three-edge bearing strength of the pipe furnished must not be less than the specified three-edge bearing strength multiplied by the ratio of the outside diameter of the pipe furnished to the outside diameter assumed in design.

PRINCIPAL SPILLWAY CONDUIT DETAILS
SUNNYCOVE F.R.S.
NICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust G.M.C.	Date 3-75	Approved by	Title
Drawn L.W.McClintock	3-75		Title
Traced		Sheet No. 12 of 18	Drawing No. 7-E-23090
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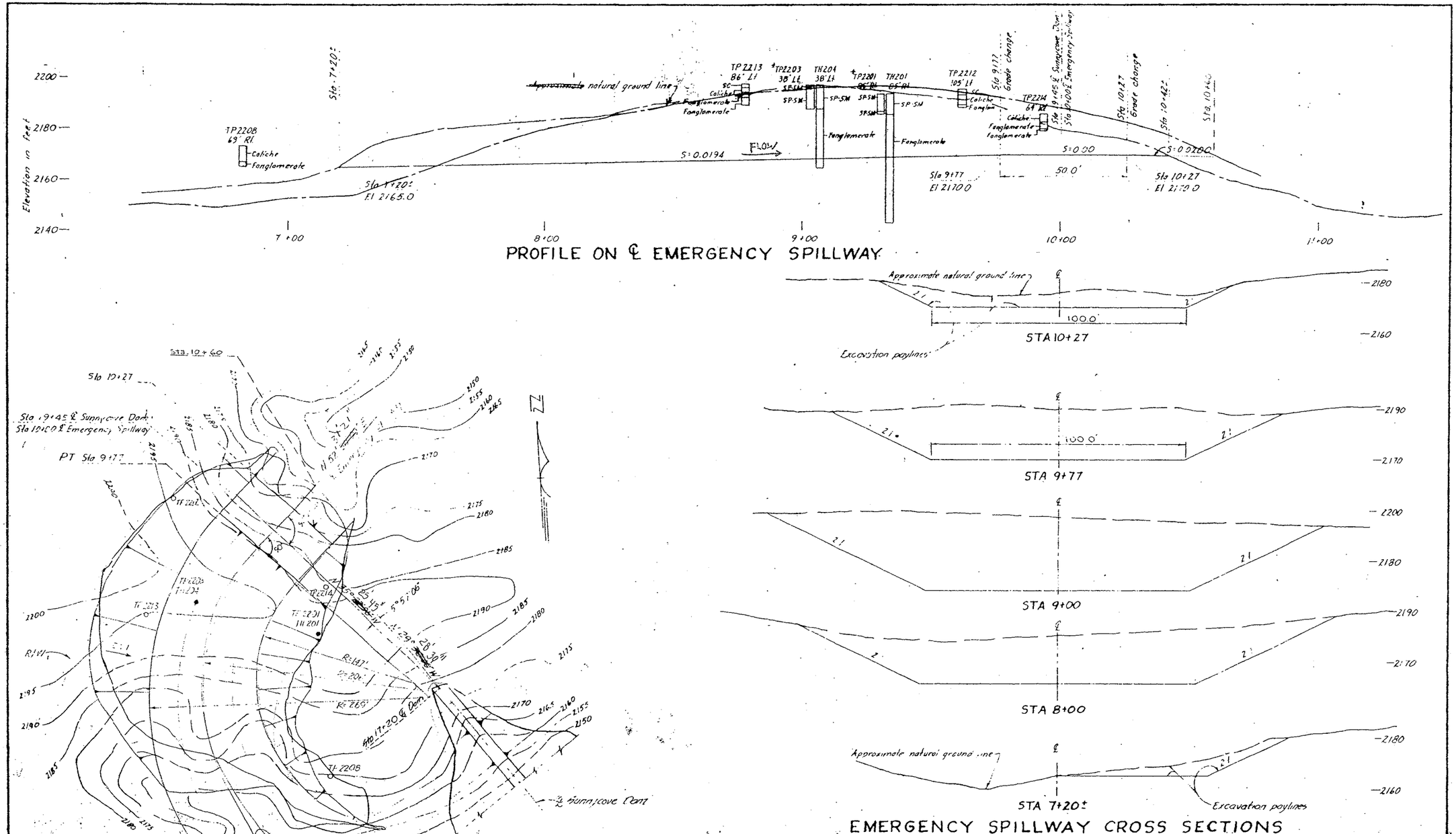
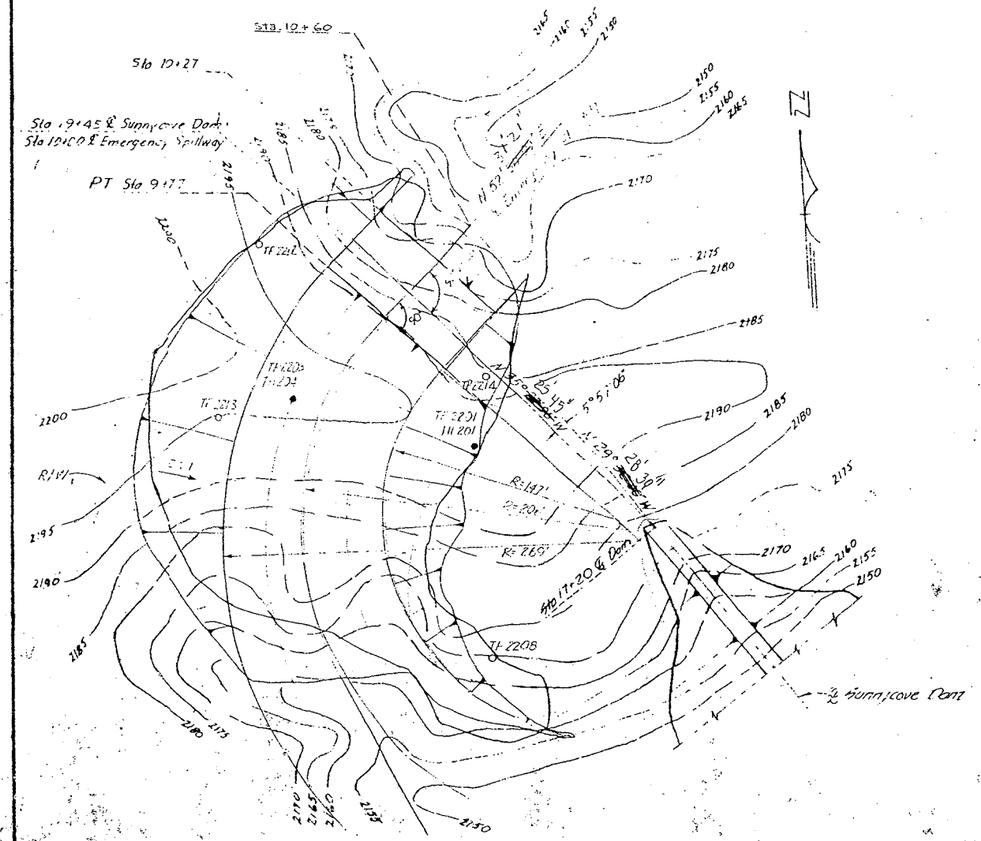


PROFILE ON ℄ EMERGENCY SPILLWAY

EMERGENCY SPILLWAY CROSS SECTIONS



PLAN OF EMERGENCY SPILLWAY

SCALE 50 0 50 100 IN FEET

SCALE 20 0 20 40 IN FEET

AS BUILT
CONSTRUCTION COMPLETED
9/15/76

Notes:

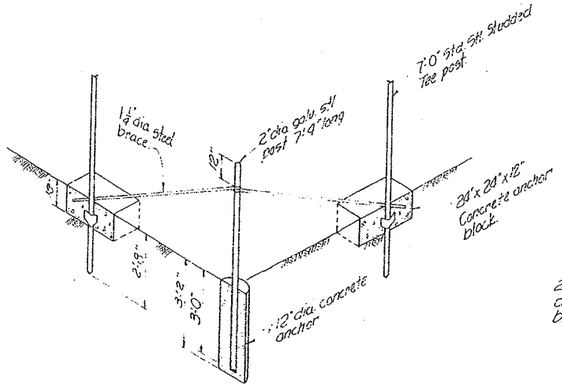
1. Unified soil descriptions are based on field identifications except where an asterisk is shown, the classification has been based on laboratory analysis.
2. Complete field logs, laboratory test data and geologic report are available for review in the project office.

EMERGENCY SPILLWAY PLAN
PROFILE AND PAYLINES
SUNNYCOVE F.P.S.
WICKENBURG W.P.D.
MARICOPA COUNTY, ARIZONA

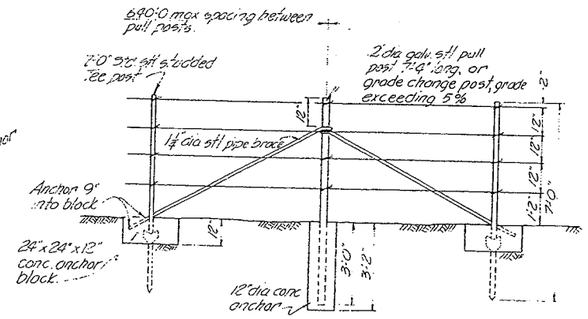
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust	Date 4-75	Approved Ralph M. Amyotte	Date
Drawn J.W. McClintock	4-75		Title
Traced		Sheet 13 of 18	Drawing No. 7-E-23090
Revised J. Jayo	6-76		

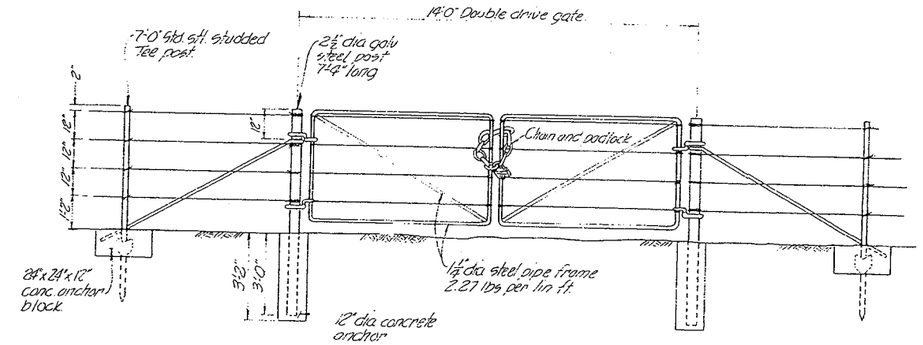
SCS ENG 313 B REV 7-69



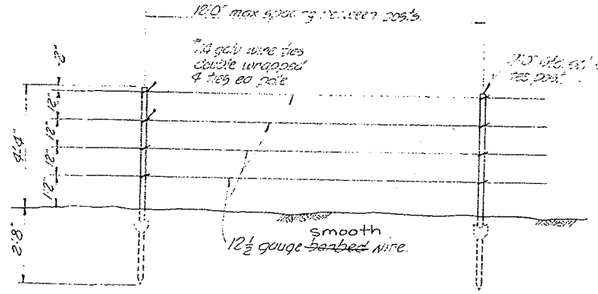
END OR CORNER POST ASSEMBLY



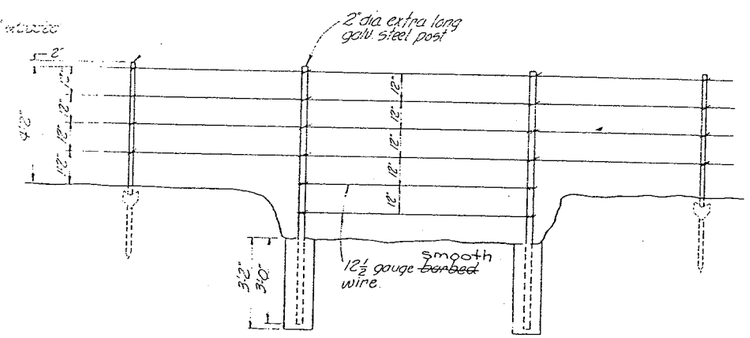
PULL POST OR CHANGE IN GRADE ASSEMBLY



GATE AND POST ASSEMBLY
(2 required)



LINE POST ASSEMBLY



WATERCOURSE OR DEPRESSION CROSSING ASSEMBLY

Drawings on this sheet not to scale

Notes:

1. 1½" dia. std. pipe shall be a minimum weight of 2.27 lbs per lin ft before galvanizing.
2. 1½" dia. std. pipe shall be a minimum weight of 2.72 lbs per lin ft before galvanizing.
3. 2" dia. std. pipe shall be a minimum weight of 3.65 lbs per lin ft before galvanizing.
4. 2½" dia. std. pipe shall be a minimum weight of 5.79 lbs per lin ft before galvanizing.
5. Steel tubing may be used in lieu of standard pipe, provided that: it shall be electric welded or seamless; the weight per lin ft shall be not less than the weights shown on the drawings; the O.D. of the tubing shall not exceed that of standard pipe by more than ⅛ inch; and the weight of galvanizing by the hot dip process shall be not less than 2 oz per lin foot total coated surface.
6. All gate hinges shall be heavy duty type, malleable iron or steel, industrial service type 270 degree swing of approved quality and design.
7. ℄ of fence posts or gate posts shall be a minimum of 6 inches inside of Rights of way lines.

AS BUILT
CONSTRUCTION COMPLETED
9/15/76

FENCING DETAILS
SUNNYCOVE F.R.S.
WICKENBURG - W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed K. J. Brust	5-75	Approved by	
Drawn J. D. Land	6-75	Title	
Traced		Title	
Checked		Sheet No. 14 of 18	Drawing No. 7-E-23090

SCS-ENG-313-B Rev. 7-69

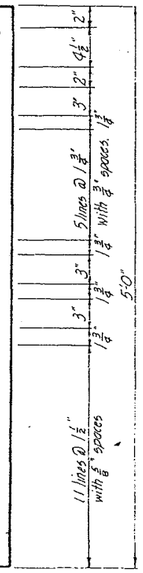
WICKENBURG WATERSHED PROJECT

FLOODWATER RETARDING DAM

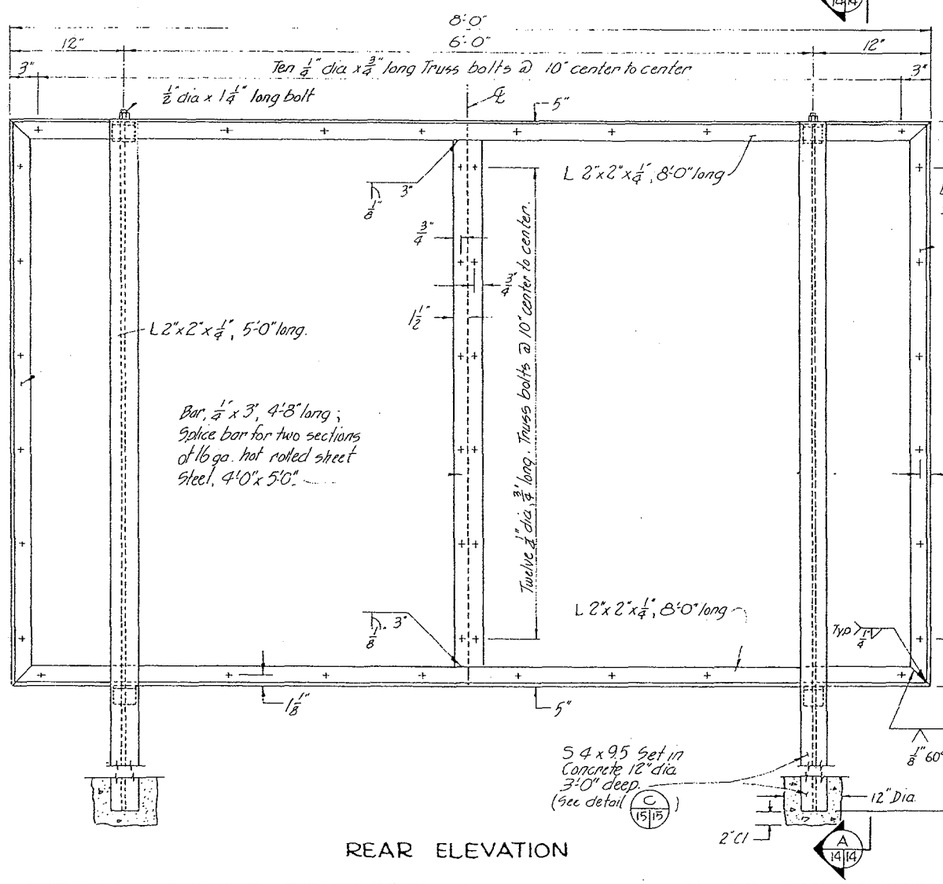
DRAINAGE AREA	864.0 ACRES
FLOOD WATER RETARDING STORAGE	218.7 ACRE FT.
WATER SURFACE AREA	18.0 ACRES
HEIGHT OF DAM	50.5 FEET
VOLUME OF FILL	111,500.0 CUBIC YD.

BUILT UNDER THE WATERSHED PROTECTION AND FLOOD PREVENTION ACT

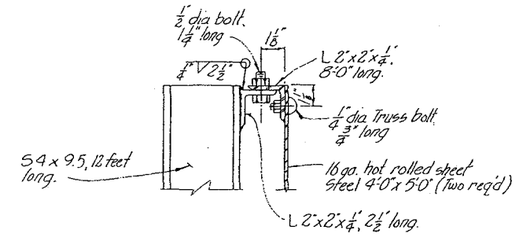
BY
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
TRIANGLE NATURAL RESOURCE CONSERVATION DISTRICT
WICKENBURG NATURAL RESOURCE CONSERVATION DISTRICT
YAVAPAI COUNTY BOARD OF SUPERVISORS
TOWN OF WICKENBURG
WITH THE ASSISTANCE OF
SOIL CONSERVATION SERVICE
OF THE
U.S. DEPARTMENT OF AGRICULTURE
1975



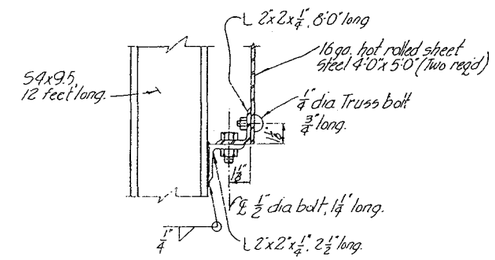
LETTERING LAYOUT



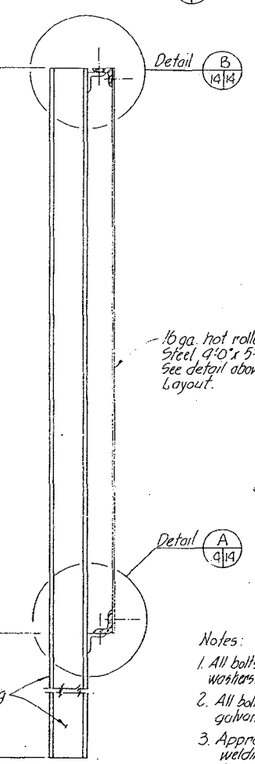
REAR ELEVATION



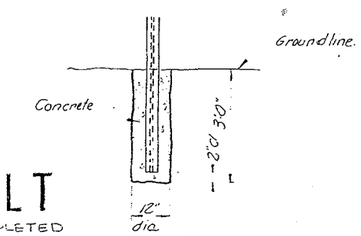
DETAIL B 14|14



DETAIL A 14|14



SECTION A 14|14



DETAIL C 15|15

AS BUILT
CONSTRUCTION COMPLETED
9/15/76

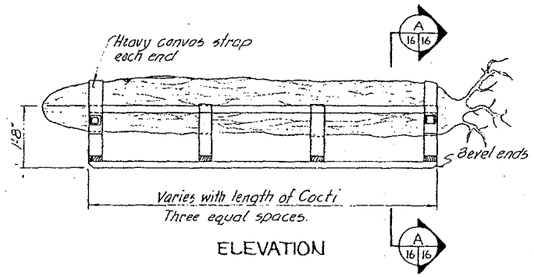
Notes:
1. All bolts shall be installed with lock washers.
2. All bolts, nuts and washers to be galvanized steel.
3. Approved spot welding or tack welding may be substituted for truss bolts in securing sheet steel sign sections to frame.

SIGN DETAILS
SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

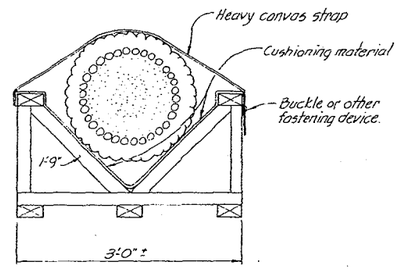
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust	Date 3-75	Approved by:	
Drawn J.D. Lond	3-75	Title	
Traced		Sign	
Checked		Sheet 15 of 18	Drawing No. T-E-23090

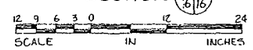
SCS-ENG-313-B Rev. 7-69

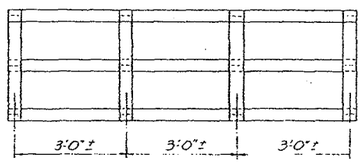


ELEVATION



SECTION A 16/16





PLAN

Notes:

1. All wood members shall be 2"x4"
2. Fasten all wood members together with wood screws 3½" in length or greater.
3. Canvas straps shall be 4" wide, fastened one side by means of wood screws. Provide buckle or other suitable fastening device so straps may be adjusted to various cacti diameters.

CACTUS CARRYING DEVICE

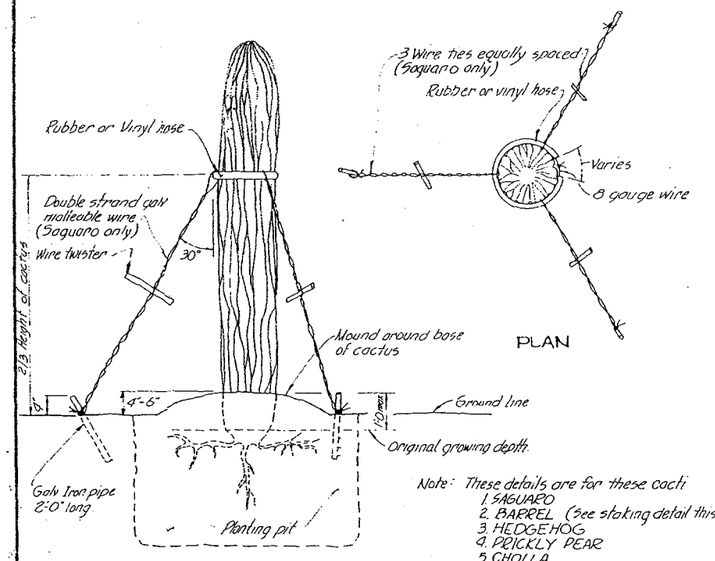
TRANSPLANTING CACTUS AND LISTED NATIVE MATERIAL

SCIENTIFIC NAME	COMMON NAME	ROOT PRUNING	6 MONTHS WATER SCHEDULE	WATER BASIN 3'-0" DIA. 6" DEEP
Cereus giganteus	Saguaro	Reduce to 1" per foot of plant height, measured from base of plant	No irrigation needed	No
Enhinocereus species	Hedgehog	Retain sufficient 6" long to anchor plant.	when planted only	No
Echinocactos	Barrel cactos	Retain sufficient 6" long to anchor plant.	When planted only	No
Opuntia species	Prickly-pear	Retain sufficient 6" long to anchor plant.	When planted only	No
Opuntia species	Cholla	Retain sufficient 6" long to anchor plant.	When planted only	No
Fouquieria Splendens	Ocotillo	Prune roots to 1" per foot of plant height.	When planted - Weekly in hot weather; Every two weeks cool weather; None October - April.	Yes
Agave species	Century, Maguey etc.	Retain all strong roots, keep roots damp until planted.	When planted - Weekly in hot weather; Every two weeks cool weather; None October - April.	Yes
Dasylirion Species	Sotol, Desert Spoon	Retain sufficient 6" long to anchor plant	When planted only	Yes
Nolina Bigelovii	Bear Grass	Retain sufficient 6" long to anchor plant	When planted only	Yes
Yucca Brevifolia	Joshua Tree	Retain sufficient 6" long to anchor plant.	When planted only	Yes
Yucca Elata	Soaptree Yucca	Retain sufficient 6" long to anchor plant	When planted only	Yes

TABLE I

PLANTING NOTES

1. Ocotillo may be planted 6" lower than original growing depth to furnish support.
2. Planting pits shall be a minimum of 1'-0" deeper than tap roots, and 1'-0" wider than lateral roots.



PLAN

ELEVATION

PLANTING NOTES

1. Planting pits shall be a minimum of 1'-0" deeper than tap roots, and 1'-0" wider than lateral roots.
2. Build a mound around base of cacti 4" to 6" in height and at least 1'-0" greater in diameter than base of plant.
3. Cactus may be set a max of 1'-0" lower than Original growing depth if required for support.

GUYING NOTES (Saguaro only)

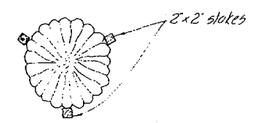
1. Rubber or vinyl hose shall be ¾" I.D. and shall be of sufficient length to protect Saguaro, But shall allow room for wire to be tightened in case of plant shrinkage.
2. Guys shall be 8 gauge galvanized malleable wire, double strand, Tightened by twisting the two wires together.
3. Stakes shall be 1" I.D. galvanized iron pipe set at an angle as shown on detail.

Note: These details are for these cacti

1. SAGUARO
2. BARREL (See staking detail this sheet.)
3. HEDGEHOG
4. PRICKLY PEAR
5. CHOLLA

CACTI PLANTING DETAILS

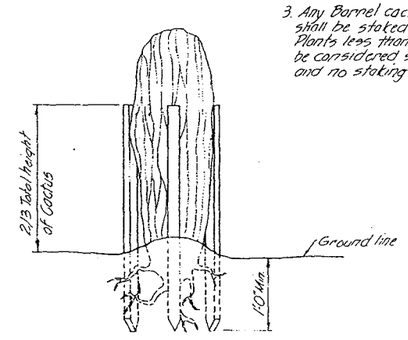
AS BUILT
CONSTRUCTION COMPLETED
9/15/76



PLAN

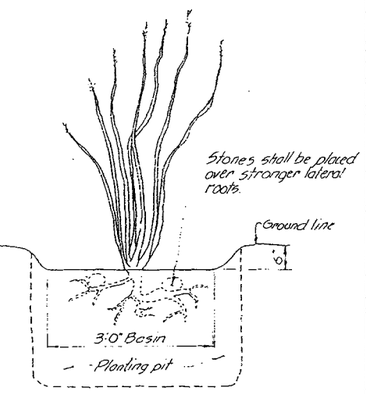
STAKING NOTES

1. Stakes shall be stained with a dark green penetrating stain
2. Stakes shall be sound Douglas fir 2"x2" set vertically against the sides of the cactus as per detail
3. Any Barrel cactus over 3'-0" tall shall be staked when planted. Plants less than 3'-0" tall shall be considered self supporting and no staking shall be required



ELEVATION

BARREL CACTUS STAKING DETAIL



ELEVATION

OCOTILLO PLANTING DETAIL

Not to scale

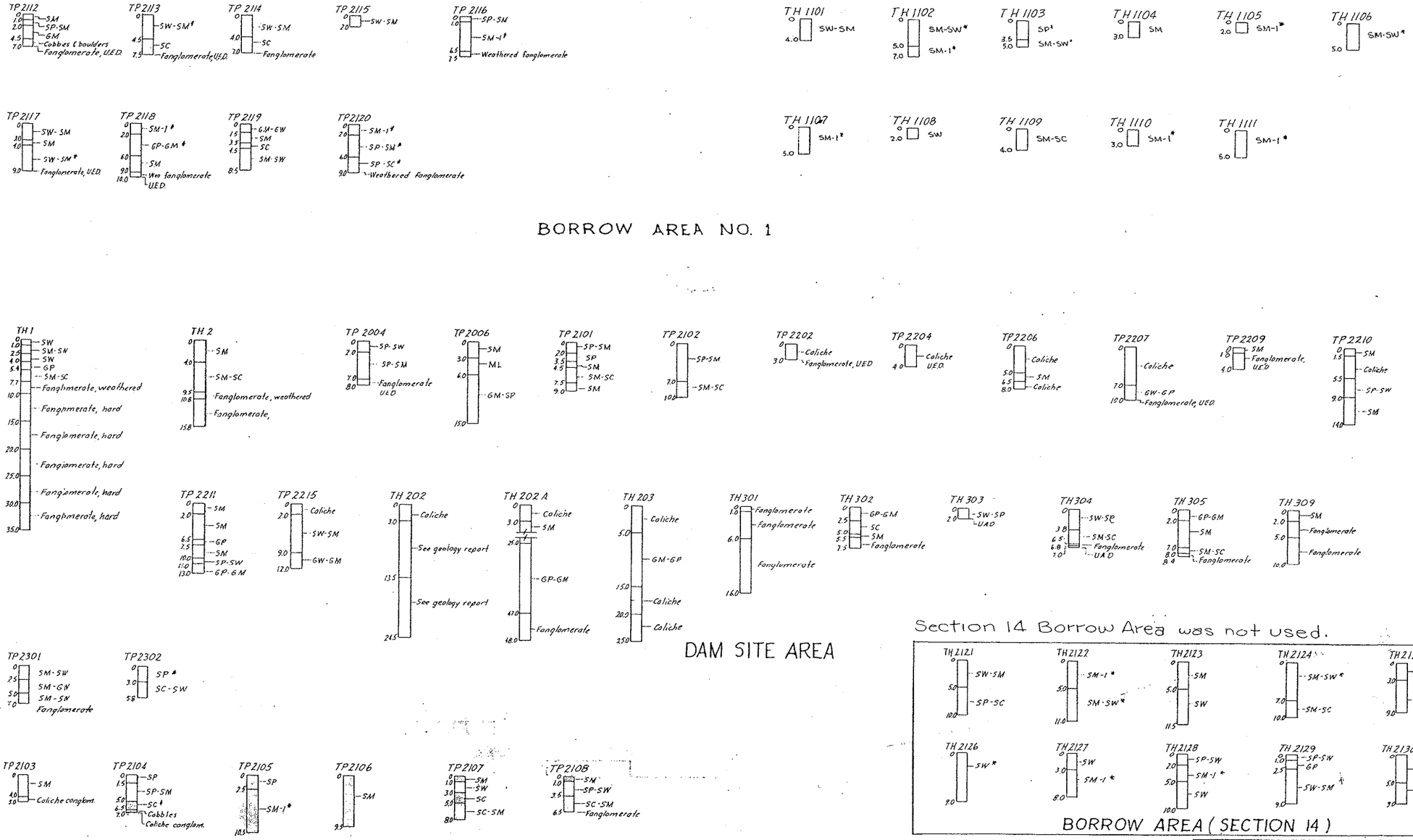
CACTUS SALVAGING DETAILS

SUNNYCOVE F.R.S
WICKENBURG W.S
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed Biddulph	Date 5-66	Approved by:	
Drawn Jack Land	6-75	Title	
Traced		Sheet No. 16 / 18	Drawing No. 7-E-23090

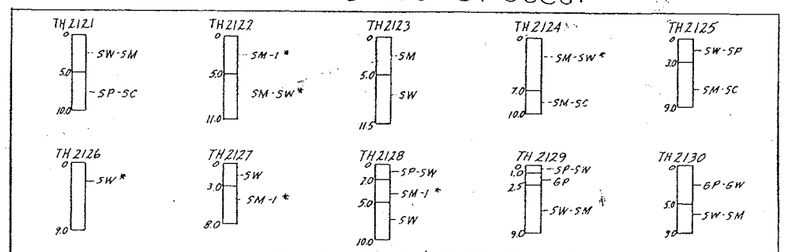
SCS ENG-413-B Rev. 7-69



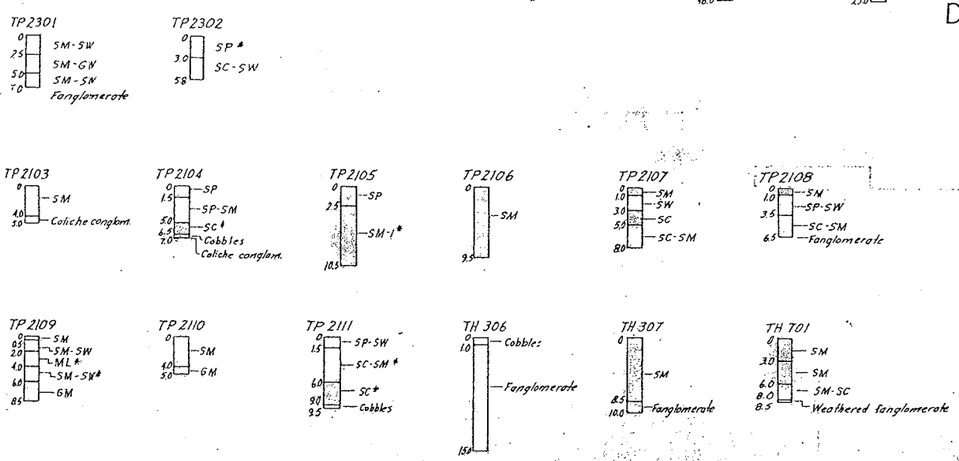
BORROW AREA NO. 1

DAM SITE AREA

Section 14 Borrow Area was not used.



BORROW AREA (SECTION 14)



BORROW AREA NO. 2

AS BUILT
CONSTRUCTION COMPLETED
9/15/76

Notes:

1. Unified soils descriptions are based on field identifications except where an asterisk is shown the classification has been based on laboratory analysis.
2. Complete field logs, laboratory test data and geologic report are available for review in the project office
3. For locations of test pits and test holes see sheets 1, 2, and 18.

SOIL LOGS

SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Logged by A. Sanders	Date 4-75	Approved by	
Drawn L.W. McClintock	4-75	Title	
Traced		Title	
Checked		Sheet No. 17 of 18	Drawing No. 7-E-23090

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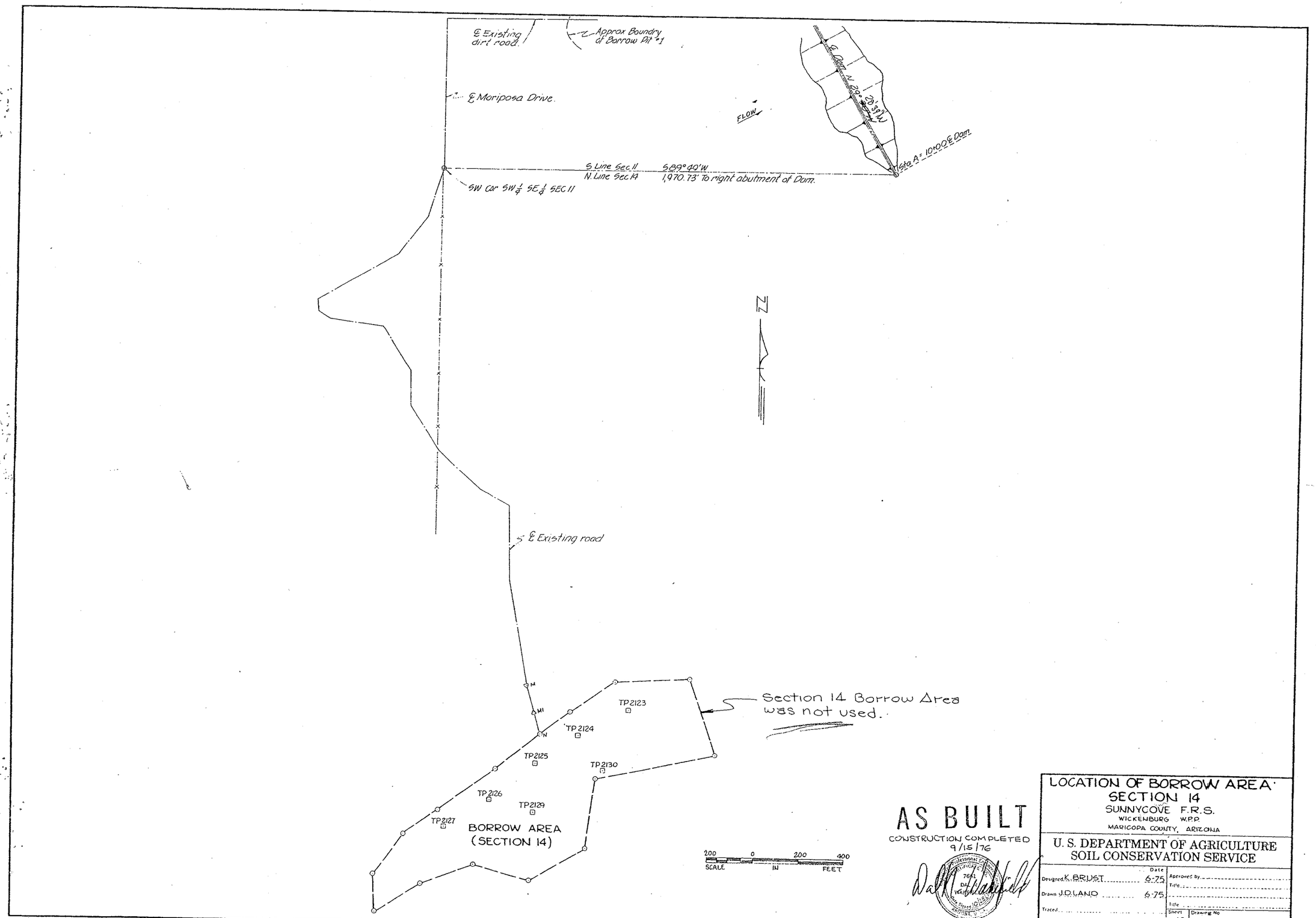


SCS--ENG--313-B Rev. 7-69

Table 1 below, compares the 2003 crest settlement monument elevations with the Adjusted Design crest elevations. The settlement monuments are physically located offset from the dam centerline on the downstream edge of the dam crest. The Design Crest elevations are referenced to NGVD 1929 and must be adjusted for comparison with 2003 elevation survey data referenced to a different vertical datum: NAVD 1988. No benchmarks were referenced prior to the 2003 survey, therefore the Datum Shift utilized in the 2003 *Casandro Wash Subsidence Survey Data Review* report was assumed for the Sunnycove FRS. Details of the adjustment calculations for this datum shift can be found in "Reference Marks", page 7, of that report.

Figure 1-1 illustrates the comparison of crest settlement monument elevation between the Adjusted Design crest elevations and the 2003 survey data listed in **Table 1.**

Figure 1-2 displays the relative change in crest settlement monument elevations obtained by subtracting the Adjusted Design crest elevation data from the 2003 survey crest elevations as calculated in **Table 1**. 2003 elevation data is the first subsidence survey data available and therefore references the Adjusted Design crest elevations as the baseline elevation. Subsequent surveys should use 2003 elevations as the baseline.

Figure 2-1 on page 5 shows the location of the crest settlement monuments.

Crest Monument Survey Data				
Marker	**Station**	**Dsgn Crest (NGVD29)**	Adj. Design (NAVD88)	2003 (NAVD88)
SNYCV11	11+1.66	2178.5	2180.73	2180.95
SNYCV12	12+3.88	2178.5	2180.73	2181.453
SNYCV13	13+4.46	2178.5	2180.73	2181.606
SNYCV14	14+5	2178.5	2180.73	2181.374
SNYCV15	15+7.06	2178.5	2180.73	2181.693
SNYCV16	16+7	2178.5	2180.73	2181.36

(Fig. 1-1 Plot Data)

2003 - Adj. Design
0.22
0.723
0.876
0.644
0.963
0.63

(Fig. 1-2 Plot Data)

Note: 1) The Datum Shift to NAVD88 elevation referenced from Casandro Wash Dam is NGVD29 elevation **plus 2.23 ft.**

2) 2003 survey data collected in-house in September 2003.

Table 1

Crest Settlement Monument Elevations

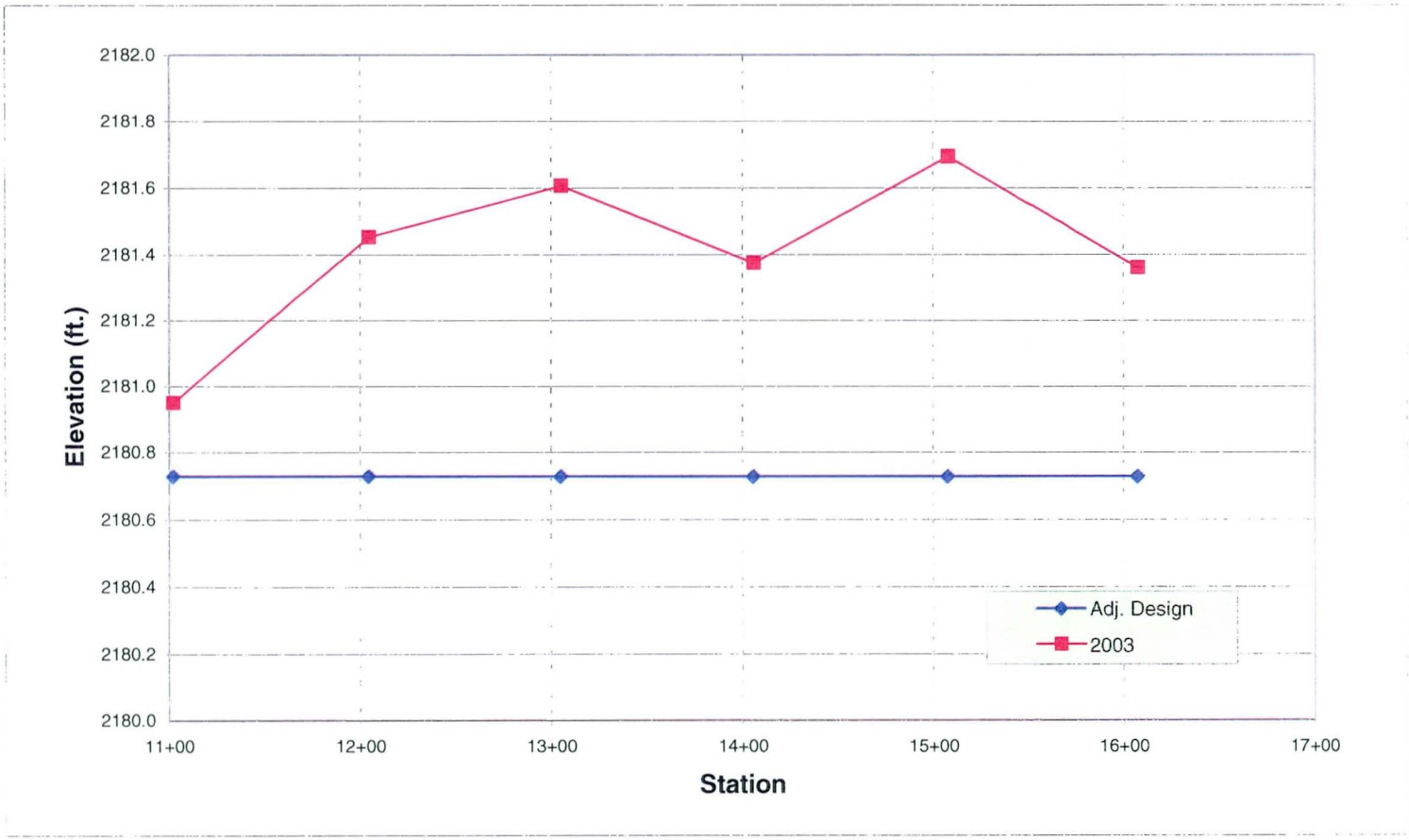


Figure 1-1

Elevation of Crest Settlement Monument Chart

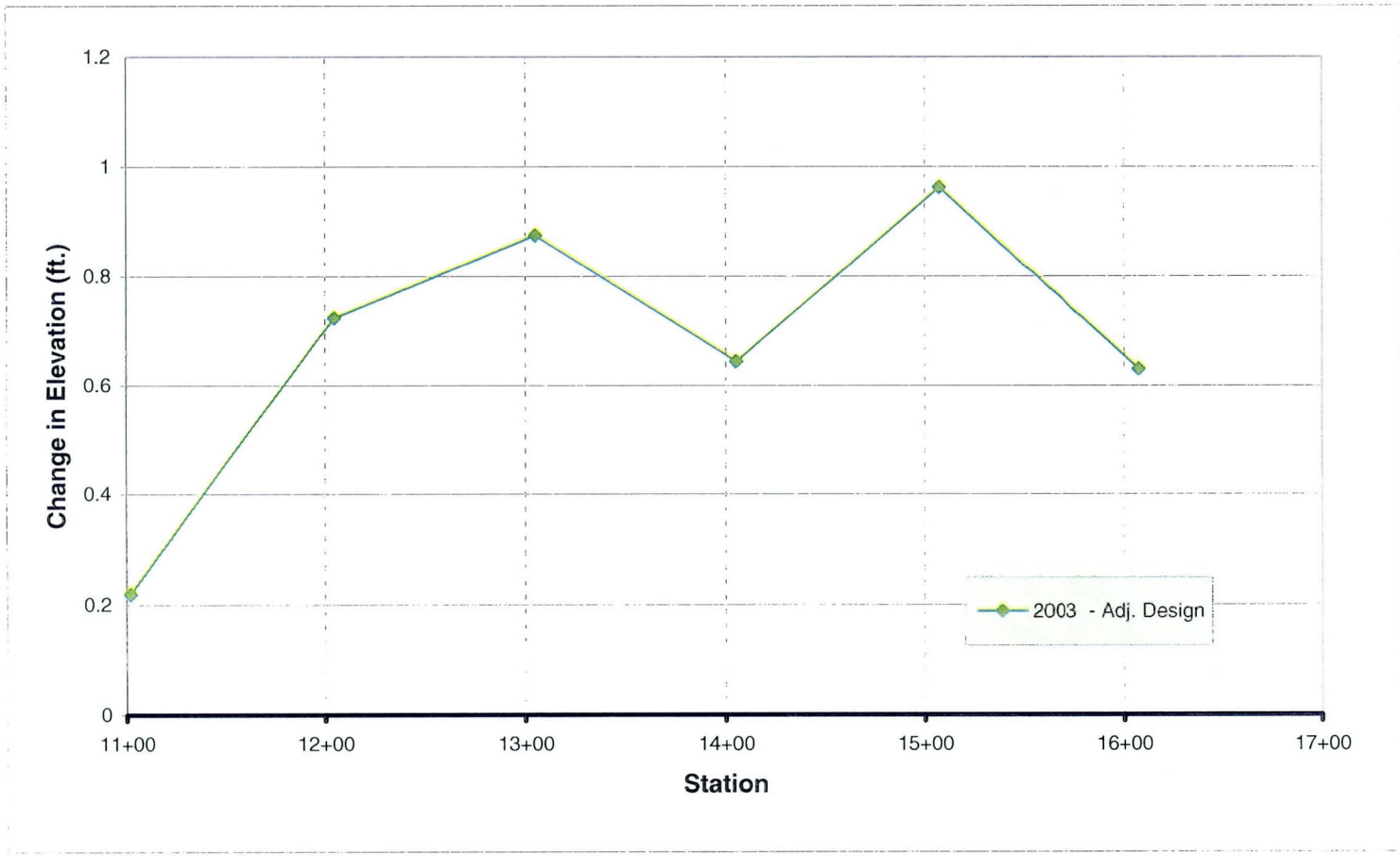


Figure 1-2

Relative Change in Dam Crest Elevation Chart, Adjusted Crest Design as Baseline Elevation Reference

Table 2-1 below, summarizes toe settlement monument elevations in the 2003 survey. This is the first set of survey elevation data collected from the toe monuments. Subsequent survey data on these toe monuments should be used for elevation comparisons and for illustrating the relative change in toe settlement monument elevations to 2003 survey with 2003 elevations as the baseline.

Table 2-2 below, displays the 2003 survey elevation at the benchmark to be referenced in later surveys.

Figure 3-1 on page 5 shows the location of the toe settlement monuments and benchmark.

Toe Monument Survey Data		
Marker	**Station**	2003
SNYCV17	13+30.79	2133.276
SNYCV18	11+8.73	2131.077
SNYCV19	15+70.91	2135.762

Table 2-1
Toe Settlement Monument Elevations

Reference Mark		
Marker	**Description**	2003
SNYCV10	CP BM FCD BC	2197.068

Table 2-2
Benchmark Elevation

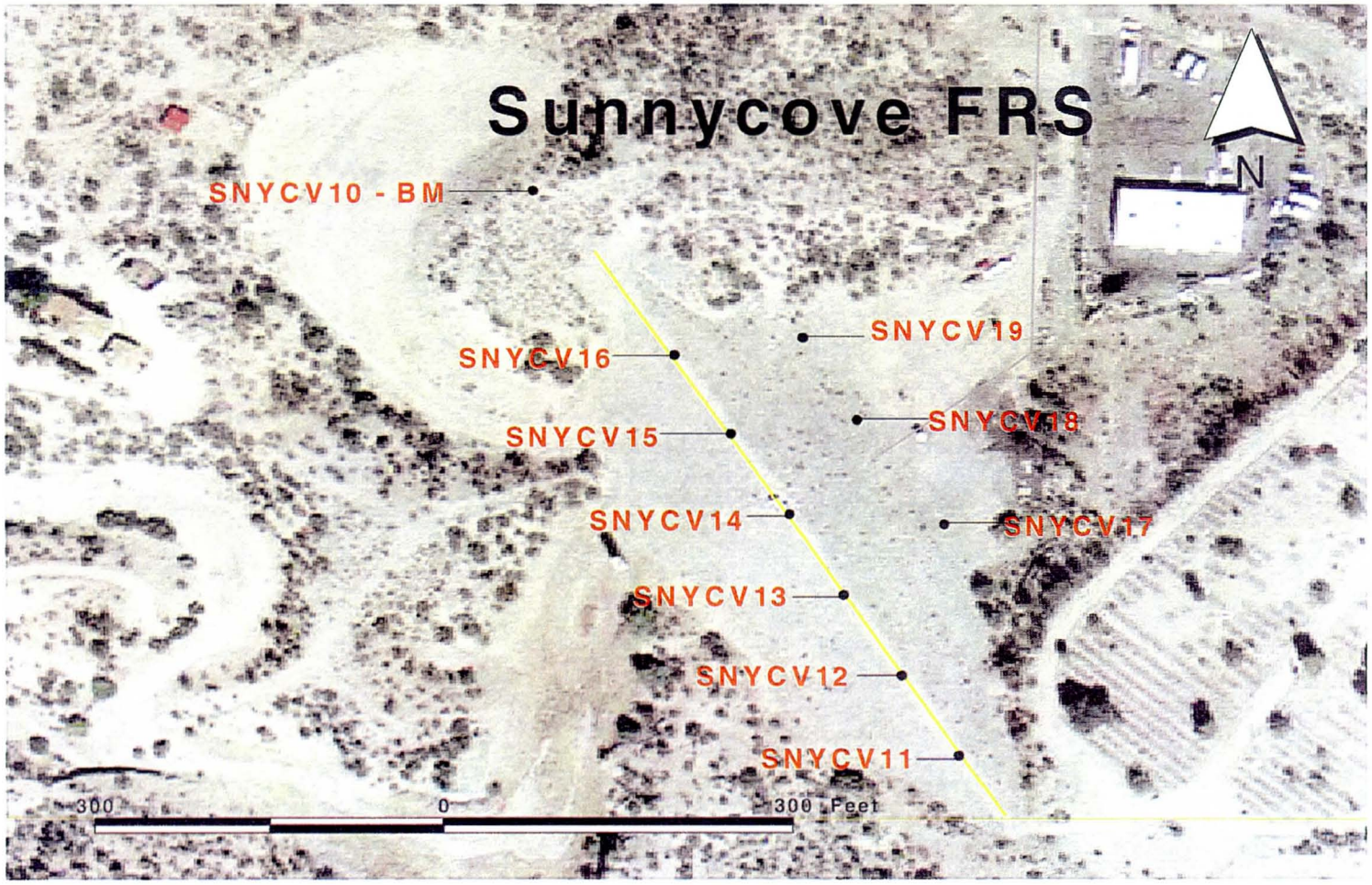
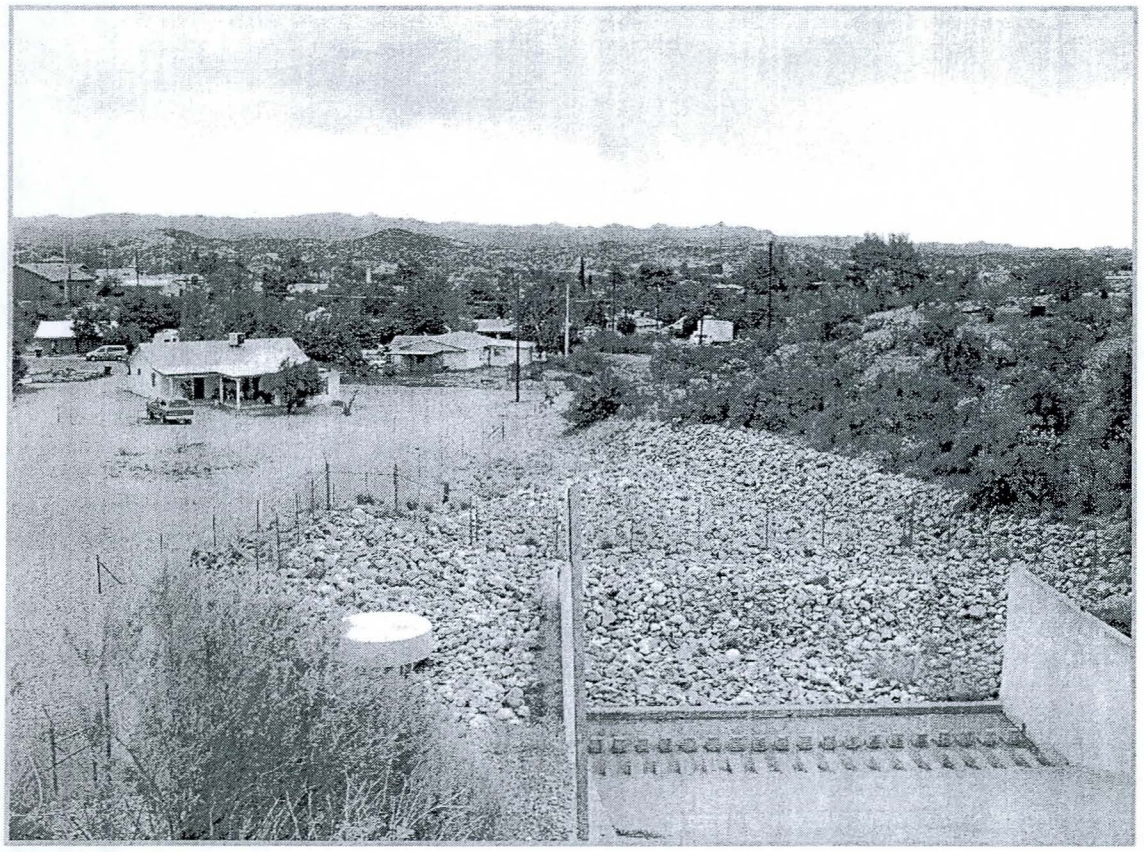


Figure 2-1
2003 Survey Monument Locations

Emergency Action Plan for Sunset FRS, Sunnycove FRS and Casandro Dam in the Town of Wickenburg, Arizona



Final - November 19, 2003

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2. Casandro Spillway 4
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2. Maricopa County Department of Emergency Management 6
3. Town of Wickenburg 6

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Wickenburg Flood Retarding Structures Emergency Action Plan

Location

The Wickenburg Structures (Sunset FRS, Sunnycove FRS and Casandro Dam) are located within the town boundaries of Wickenburg, approximately 60 miles northwest of Phoenix, Arizona. Figure 1 shows the locations of the three structures.

Description

Sunset FRS is an earthen dam nearly adjacent to highway US 60, south of the Jones Ford Dealership. Construction was completed in September, 1976 by the (then) Soil Conservation Service. It collects and stores water from Sunset Wash, draining 0.60 square miles of commercial, residential and sonoran desert land. The emergency spillway is a 40 foot-wide concrete broad-crested weir through the embankment near the center of the dam. Sunset FRS is classified as a small, high-hazard dam by the Arizona Department of Water Resources (ADWR) because of its spillway capacity (86 acre-feet) and the number of occupied structures in the path of downstream floodwaters.

Sunnycove FRS is an earthen dam located about 1 mile south of US 60 via Kellis Road. Construction was completed in September, 1976 by the (then) Soil Conservation Service. It collects and stores water from Sunnycove Wash, draining 1.35 square miles of primarily sonoran desert land with a few residences on ridge tops. The emergency spillway is a 100 foot-wide earthen channel located on the north side of the dam. Sunnycove FRS is classified as a small, high-hazard dam by ADWR because of its spillway capacity (216 acre-feet) and the number of occupied structures in the path of downstream floodwaters.

Casandro Dam is an earthen dam located about ¼ mile north of US 60 via Mariposa Road. Construction was completed in March, 1996 by the Flood Control District of Maricopa County. It collects and stores water from Casandro Wash, draining 3.0 square miles of primarily sonoran desert land with a widely-spaced residential development. The emergency spillway is an 80 foot-wide concrete broad-crested weir through the embankment near the center of the dam. Casandro Dam is classified as a small, high-hazard dam by ADWR because of its spillway capacity (143 acre-feet) and the number of occupied structures in the path of downstream floodwaters.

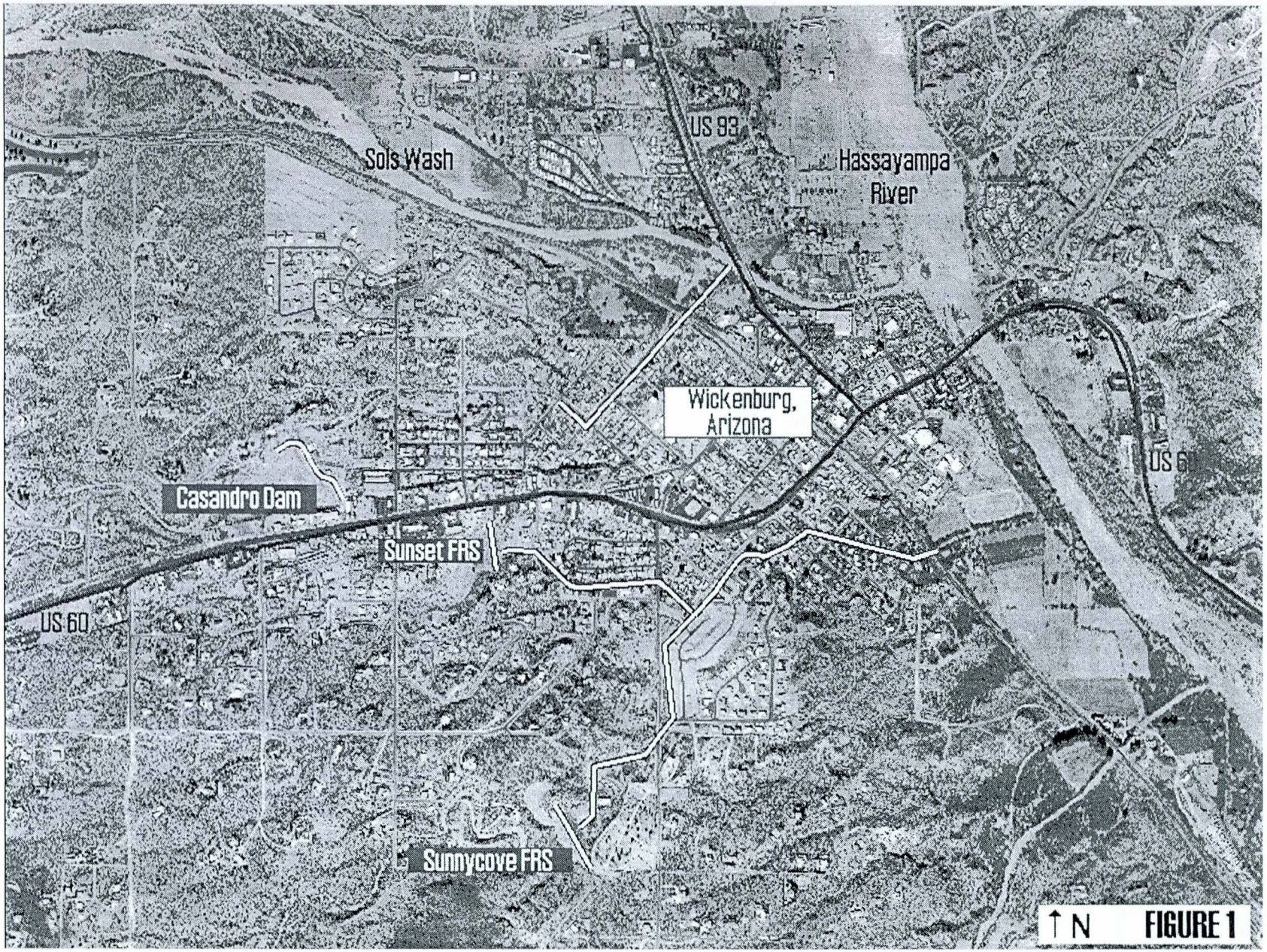

Sols Wash
US 93
Hassayampa River
Wickenburg, Arizona
US 60
Casandro Dam
Sunset FRS
US 60
Sunnycove FRS
N

FIGURE 1

Purpose of Plan

The purpose of this Emergency Action Plan (EAP) is to coordinate the prediction, detection and emergency response to a spillway or dam-failure flood downstream of Sunset FRS, Sunnycove FRS or Casandro Dam. The EAP presents rainfall, static water-level and drawdown detection criteria which could lead to a spillway flow or dam-failure event. Also presented are the lines of communication and agency actions necessary to evacuate downstream residents and others before floodwaters arrive.

Inundation Areas

See Exhibits B-1 through B-4 in **Appendix B** for maps of inundation areas.

1. **Sunset FRS and Sunnycove FRS Emergency Spillway Inundation**. During major storms, the reservoir pools can fill quickly and stormwater may discharge through the emergency spillways in a very short time. Outflow from Sunset FRS will travel in an easterly direction parallel to US 60, crossing Oxbow Drives North and South, Kellis Road, America St., Grant, Lincoln and Apache Streets, then Center and Jackson Streets. A small area along US 60 will be inundated in the area of Apache and Madison Streets. It will then flow northeast over Mesquite, Adams, Henderson and Park Streets, and fan-out to the east over Jefferson St., Fisher St., Sylvan Rd., Sylvan Dr. and Cool Water Drive. Finally it will cross the BNSF railroad tracks, Tegner and Kerkes Streets, and then enter the Hassayampa River channel. Outflow from Sunnycove FRS will travel in a northeasterly direction across Kellis Road, Desert Canyon Road and Lost Canyon Road. Near Center and Jackson Streets, the inundation joins the area described for Sunset FRS. Inundation mapping exists for spillway flows of 33, 67 and 100 percent for both dams, but the area inundated does not change significantly. The potential inundated area is shown in **Exhibit B-1**.

2. **Casandro Dam Emergency Spillway Inundation**. During major storms, the reservoir pool can fill quickly and stormwater may discharge through the emergency spillway in a very short time. Outflow from Casandro Dam will travel in an easterly direction across Mariposa Drive and Cucaracha Streets, parallel to La Paloma Drive. It turns southeast near Via Corte and Lincoln Street, flows to Jackson Street, then turns northeast along Mohave Street, crossing Madison, Jefferson and Washington Streets. The entire grounds of *My Father's Retirement Ranch* will be inundated. Flows will pond behind the BNSF railroad grade, eventually draining through the old Casandro Wash bridge to Sols Wash. Inundation mapping exists for spillway flows of 33, 67 and 100 percent for the dam, but the area inundated does not change significantly. The potential inundated area is shown in **Exhibit B-2.**

3. **Sunset FRS and Sunnycove FRS Dam-Failure Inundation**. Outflow from a failure of Sunset FRS will travel in an easterly direction parallel to US 60, crossing Oxbow Drives North and South, Kellis Road, America St., Grant, Lincoln and Apache Streets, then Center and Jackson Streets. Flow will then flow northeast over Mesquite, Adams, Henderson and Park Streets, and fan-out to the east over Jefferson St., Fisher St., Sylvan Rd., Sylvan Dr. and Cool Water Drive. Finally it will cross the BNSF railroad tracks, Tegner and Kerkes Streets, flow through the campus of *Hassayampa Upper Elementary School*, and then enter the Hassayampa River channel. Outflow from a failure of Sunnycove FRS will travel in a northeasterly direction across Kellis Road, Desert Canyon Road and Lost Canyon Road. Near Center and

Jackson Streets, the inundation joins the area described for Sunset FRS. An area along and across US 60, which includes the *Wickenburg Dialysis Center*, will be inundated from Yavapai Street to Washington Street between the highway and Apache Street. Flows will have enough momentum to cross the Hassayampa channel and inundate areas up to US 60 in the vicinity of Sullivan Street. The potential inundated area is shown in **Exhibit B-3**.

4. **Casandro Dam - Failure Inundation**. Outflow from a failure of Casandro Dam will travel in an easterly direction across Mariposa Drive and Cucaracha Streets, parallel to La Paloma Drive. It turns southeast near Via Corte and Lincoln Street, flows to Jackson Street, then turns northeast along Mohave Street, crossing Madison, Jefferson and Washington Streets. The entire grounds of *My Father's Retirement Ranch* will be inundated. Flows will pond behind the BNSF railroad grade, eventually draining through the old Casandro Wash bridge to Sols Wash. The potential inundated area is shown in **Exhibit B-4.**

Specific Tasks for Emergency Spillway Releases or Dam-Failure Floods at all Wickenburg Dams

Flood Control District of Maricopa County

a. The On-call Hydrologist (OCH) will monitor pertinent ALERT rainfall, runoff and impoundment data. An ALERT alarm will sound at 10% of spillway capacity. Perform tasks according to the Wickenburg EAP Flow Chart (Figure 2, page 7).

b. At 25% spillway capacity, the OCH will dispatch FCD O&M Team # 1 to observe the water levels and structural integrity of the dam(s) being monitored. Travel time to the dams from notification of the Team to arrival at the dam is approximately 2 hours. During this time, the Wickenburg Police Department (WPD) or Maricopa County Sheriff's Office (MCSO) will dispatch an observer to monitor conditions.

c. At 50% spillway capacity, or at the direction of an in-place observer, the OCH will notify the Maricopa County Department of Emergency Management (MCDEM).

d. At 90% spillway capacity, or at the direction of an in-place observer, the OCH will inform WPD dispatch and MCDEM that evacuations may be necessary.

e. When impounded water reaches the **spillway** elevation, **or** if an in-place observer reports an impending **failure**, **or** if ALERT data denote a falling water-level indicative of a **failure**, the OCH will notify WPD dispatch **immediately** and give clear instructions to evacuate the downstream area of the specific structure. MCDEM can then be notified to provide assistance.

f. When storm conditions have subsided and the impoundments no longer pose a threat to downstream lives or property, the OCH will issue an ALL CLEAR message to WPD and MCDEM, then contact O&M Team # 1 and instruct them to return.

Maricopa County Department of Emergency Management

a. Monitor the situation and coordinate support. Perform tasks according to the Wickenburg EAP Flow Chart (Figure 2, page 7).

b. Upon receiving notification of a 50% impoundment from FCD, activate the Emergency Operations Center. Notify MCSO, the Central Arizona Chapter of the American Red Cross and the BNSF railroad - advise them of the situation.

c. Upon receiving notification of a 90% impoundment from FCD, inform MCSO that their assistance may be needed in assisting WPD with evacuations. Inform Red Cross to begin preliminary shelter operations.

d. Upon receiving notification from FCD that evacuations have begun due to a **spillway flow** or **dam failure**, notify
   - MCSO to assist WPD with evacuations and security
   - BNSF railroad to stop all trains from passing through the Wickenburg area
   - Red Cross to establish a shelter(s) for evacuees at the Wickenburg Community Center, MacLennan School, and/or Vulture Mine School
   - Arizona Department of Water Resources and Division of Emergency Management to provide assistance in their areas of expertise

e. When storm conditions have subsided and the impoundment(s) no longer pose a threat to downstream lives or property, FCD will issue an ALL CLEAR message. Contact BNSF and tell them to inspect the track before resuming rail service.

Town of Wickenburg, Police Dispatch

The Town of Wickenburg will assume overall direction and control of emergency response operations within its jurisdiction, to include warning, evacuation and security of the affected area. The Town Manager will direct the effort, with assistance from the Chiefs of Police and Fire. The point of contact between FCD and the Town will be the WPD dispatcher.

a. Monitor the situation and coordinate support. Perform tasks according to the Wickenburg EAP Flow Chart (Figure 2, page 7).

b. Upon receiving notification of a 25% impoundment from FCD, send an officer from the Wickenburg PD or MCSO to monitor the dam(s) until an FCD crew arrives.

c. Upon receiving notification of a 90% impoundment from FCD, prepare to evacuate areas downstream of the structure(s) being monitored.

d. Upon receiving notification from FCD of an impending or in-progress **emergency spillway release** or **dam failure, immediately evacuate** areas downstream of the structure(s) being monitored.

e. When storm conditions have subsided and the impoundment(s) no longer pose a threat to downstream lives or property, FCD will issue an ALL CLEAR message. Post-flood actions can then begin.

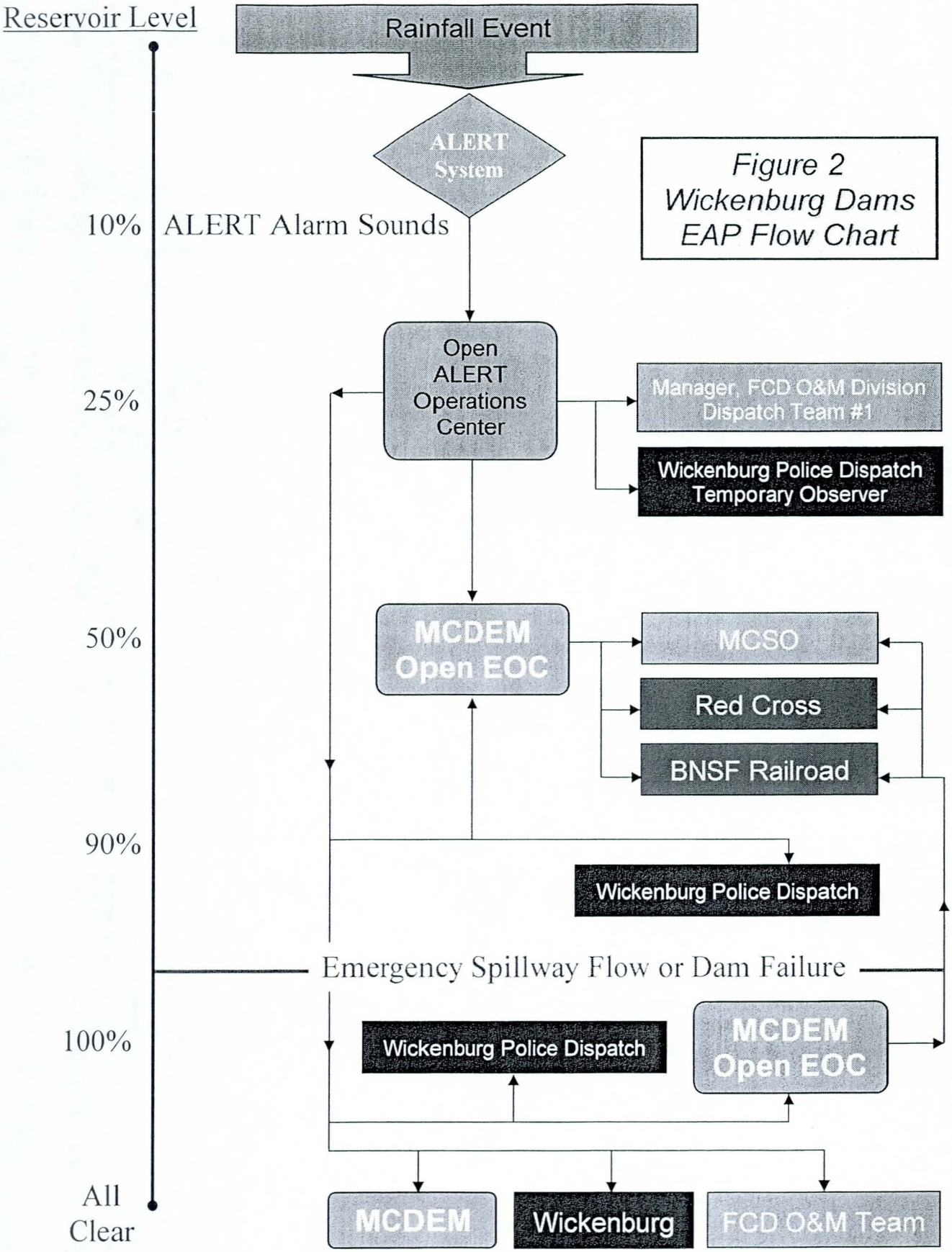


Figure 2
Wickenburg Dams
EAP Flow Chart

TABLE 1

Detailed Evacuation Instructions
(In Order of Impact by Flood Waters, Critical Facilities in Bold)

Structure / Event	Street / Drive	From	To
Casandro Dam Use the same evacuation instructions for both emergency spillway flow and dam failure	House on El Tecalote Dr.	West of Mariposa Rd.	
	House on north side of La Paloma Dr.	Mariposa Dr.	Cucuracha Dr.
	Cucuracha St.	La Paloma Dr.	El Tecalote Dr.
	Via Corte Dr.	West of Avispa St.	
	Navajo St.	Avispa St.	Adams St.
	Lincoln St.	Avispa St.	Mohave St.
	Jackson St.	Navajo St.	Santa Cruz St.
	Mohave St.	Lincoln St.	Jefferson St.
	Madison St.	Hermosa Dr. alignment	2 houses SE of Mohave St.
	Adams St.	Hermosa Dr. alignment	3 houses SE of Mohave St.
	Jefferson St.	**All of "My Father's Retirement Ranch"**	**West of Mohave St.**
Sunset FRS Emergency Spillway Flow	All of America St. and Whipple Ct.	Below the Dam	
	All of Oxbow Dr. North and South	West of Kellis Rd.	
	Kellis Rd.	America St.	Oxbow Dr. North
	Apache St.	Kellis Rd.	US 60
	Grant St.	Apache St.	Center St.
	Lincoln St.	Apache St.	Center St.
	Jackson St.	US 60	Center St.
	Center St.	America St.	Madison St.
	Madison St.	US 60	Fisher St.
	Mesquite St.	US 60	Center St.
	Adams St.	US 60	Park St.
	Structure at Adams St.	and Henderson St.	
	Park St.	Madison St.	Jefferson St.
	Jefferson St.	Park St.	Howard Ct.
	Sylvan Rd.	Park St.	Sylvan Dr.
	All of Sylvan Drive and	Cool Water Drive	
	Howard Ct.	Jefferson St.	Fisher St.
	Houses along Sunset Wash	Railroad	Tegner St.
Sunset FRS Dam Failure	All of America St. and Whipple Ct.	Below the Dam	
	All of Oxbow Dr. North and South	West of Kellis Rd.	
	America St.	Sunset FRS	Kellis Rd.
	Palo Verde Rd.	Sunny Cove Heights	America St.
	Kellis Rd.	America St.	Oxbow Dr. North
	Center St.	Kellis Rd.	Madison St.
	Apache St.	Kellis Rd.	US 60
	Grant St.	Apache St.	Center St.
	Lincoln St.	Apache St.	Center St.
	Jackson St.	US 60	Center St.
	Center St.	America St.	Madison St.

Structure / Event	Street / Drive	From	To
	Madison St.	US 60	Fisher St.
Sunset FRS	Mesquite St.	US 60	Fisher St.
Dam Failure, continued	Adams St.	US 60	Park St.
	Henderson St.	Adams St.	Jefferson St.
	Park St.	Madison St.	Jefferson St.
	Jefferson St.	US 60	Howard Ct.
	Sylvan Rd.	Park St.	Sylvan Dr.
	All of Sylvan Drive and	Cool Water Drive	
	Howard Ct.	Jefferson St.	Railroad
	Washington St.	US 60	Southeast end
	Houses on dirt road south of Howard Ct.		
	Houses along Sunset Wash	Railroad	Tegner St.
	Coconino St.	Frontier St.	Valentine St.
	Cochise St.	Frontier St.	Tegner St.
	Frontier St.	US 60	Southeast end
	Tegner St. (school)	**US 60**	**Southeast end**
	Valentine St.	US 60	Coconino St.
	Kerkes St.	US 60	Southeast end
Sunnycove FRS			
Emergency Spillway Flow	Close Kellis Road from US 60 to Cottonwood Ln.		
	Desert Canyon Rd.	Cottonwood Ln.	Cul-de-sac N of Center St.
	Lost Canyon Rd.	Kellis Rd.	Desert Canyon Rd.
	Grant St.	Apache St.	Center St.
	Lincoln St.	Apache St.	Center St.
	Close US 60 from Adams St. to Yavapai St.	Route Traffic along Adams and Yavapai Streets	
	Apache St.	Kellis Rd.	US 60
	Jackson St.	US 60	Center St.
	Center St.	America St.	Madison St.
	Madison St.	US 60	Fisher St.
	Mesquite St.	Center St.	Center St.
	Adams St.	US 60	Park St.
	Structure at Adams St.	and Henderson St.	
	Park St.	Madison St.	Jefferson St.
	Jefferson St.	Park St.	Howard Ct.
	Sylvan Rd.	Park St.	Sylvan Dr.
	All of Sylvan Drive		
	All of Cool Water Drive		
	Howard Ct.	Jefferson St.	Fisher St.
	Houses along Sunset Wash	Railroad	Tegner St.
Sunnycove FRS	Close Kellis Road from US 60 to Cottonwood Ln.		
Dam Failure	Desert Canyon Rd.	Cottonwood Ln.	Cul-de-sac N of Center St.
	Lost Canyon Rd.	Kellis Rd.	Desert Canyon Rd.
	Grant St.	Kellis Rd.	Center St.
	Lincoln St.	US 60	Desert Canyon Rd.
	Close US 60 from Washington St. to Savage St	Route traffic along Savage Washington and Yavapai St	
	Apache St.	Kellis Rd.	Washington St.
	Jackson St.	US 60	Monte Cristo Dr.
	Center St.	Kellis Rd.	Washington St.
	Madison St.	Yavapai St.	Fisher St.
	Mesquite St.	Center St.	Fisher St.
	Adams St.	Apache St.	Park St.
	Park St.	Jefferson St.	Park St.
	Jefferson St.	Apache St.	Howard Ct.

Structure / Event	Street / Drive	From	To
Sunnycove FRS	Sylvan Rd.	Park St.	Sylvan Dr.
Dam Failure, continued	All of Sylvan Drive		
	All of Cool Water Dr.		
	Howard Ct.	Jefferson St.	Fisher St.
	Houses along Sunset Wash	Railroad	Tegner St.
	Washington St.	US 60	East end
	Frontier St.	US 60	East end
	Tegner St.	**US 60**	**East end**
	Coconino St.	Frontier St.	Valentine St.
	Cochise St.	Frontier St.	Tegner St.
	Valentine St.	US 60	Coconino St.
	Kerkes St.	US 60	East end
	Sullivan St.	US 60	East end

Appendix A – Contact Numbers

Flood Control District of Maricopa County

ALERT Room 602-506-8701
Or 602-272-0132
Hydrologist on Call (cellular) ..... 602-390-[redacted]
Steve Waters, Home ..... 480-345-[redacted]
Or Pager ..... 602-450-[redacted]
Jim Perfrement, Home ..... 602-971-[redacted]
Or Pager ..... 602-450-[redacted]

Maricopa County Department of Emergency Management

Main Number 602-273-1411

Town of Wickenburg

Police Dispatch 602-506-1563
Or 928-684-5411
Emergency Manager, Scott Bowman, Cellular ..... 928-671-[redacted]
Pager ..... 928-684-[redacted]
Town Manager, Shane Dille ..... 928-684-5451 ext. 213
Fire Marshall / Director, Bucky Walters, Home ..... 928-684-[redacted]
Fire Chief, Ed Temerowski ..... 928-684-[redacted]
Police Chief, Tony Melendez ..... 928-684-[redacted]

Maricopa County Sheriff's Office

Wickenburg Area Dispatch 1-800-352-4553

Burlington Northern & Santa Fe Railroad

Service Desk 708-995-2911
Trainmaster 602-382-5801

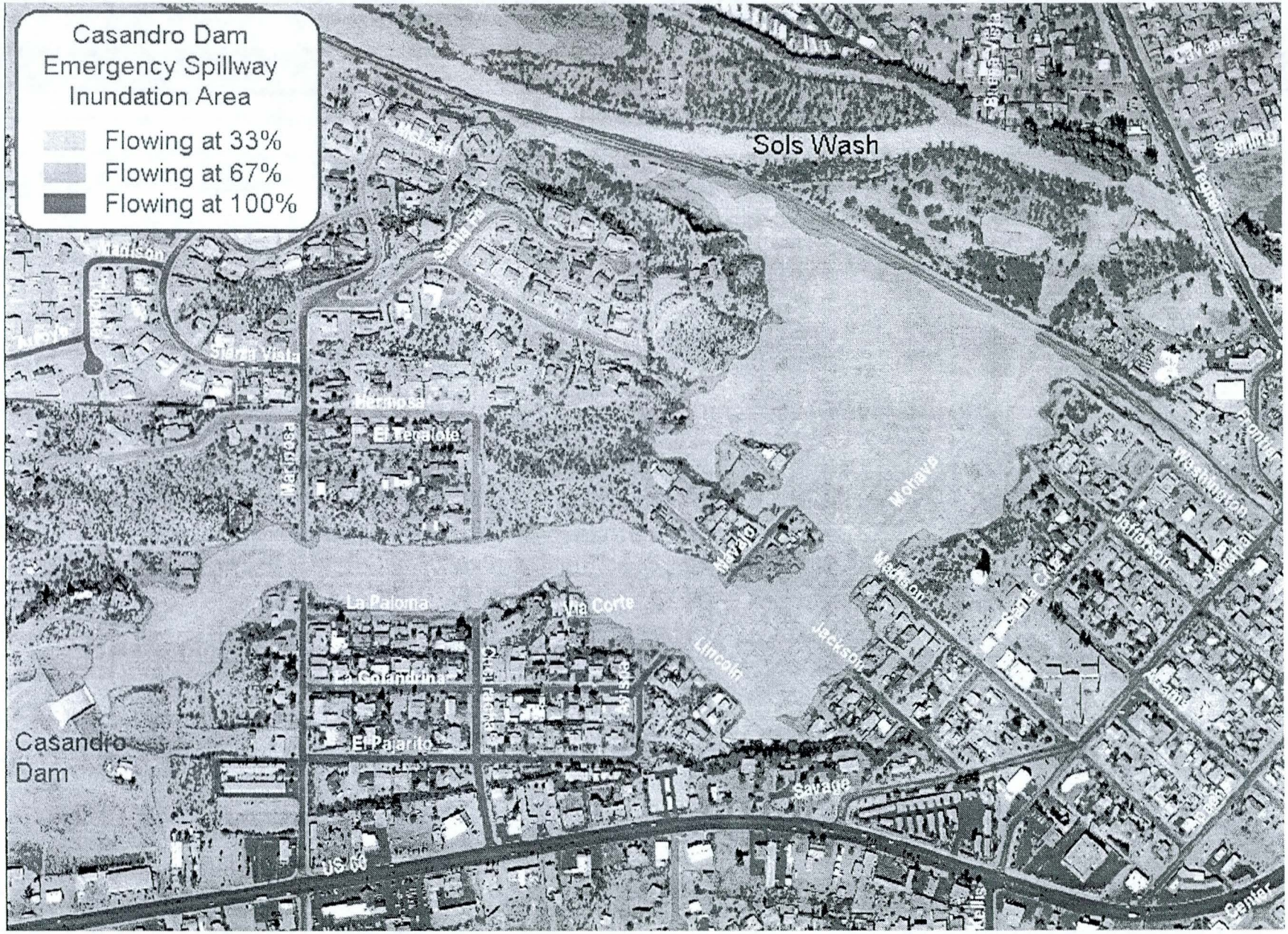
ADWR Dam Safety Division

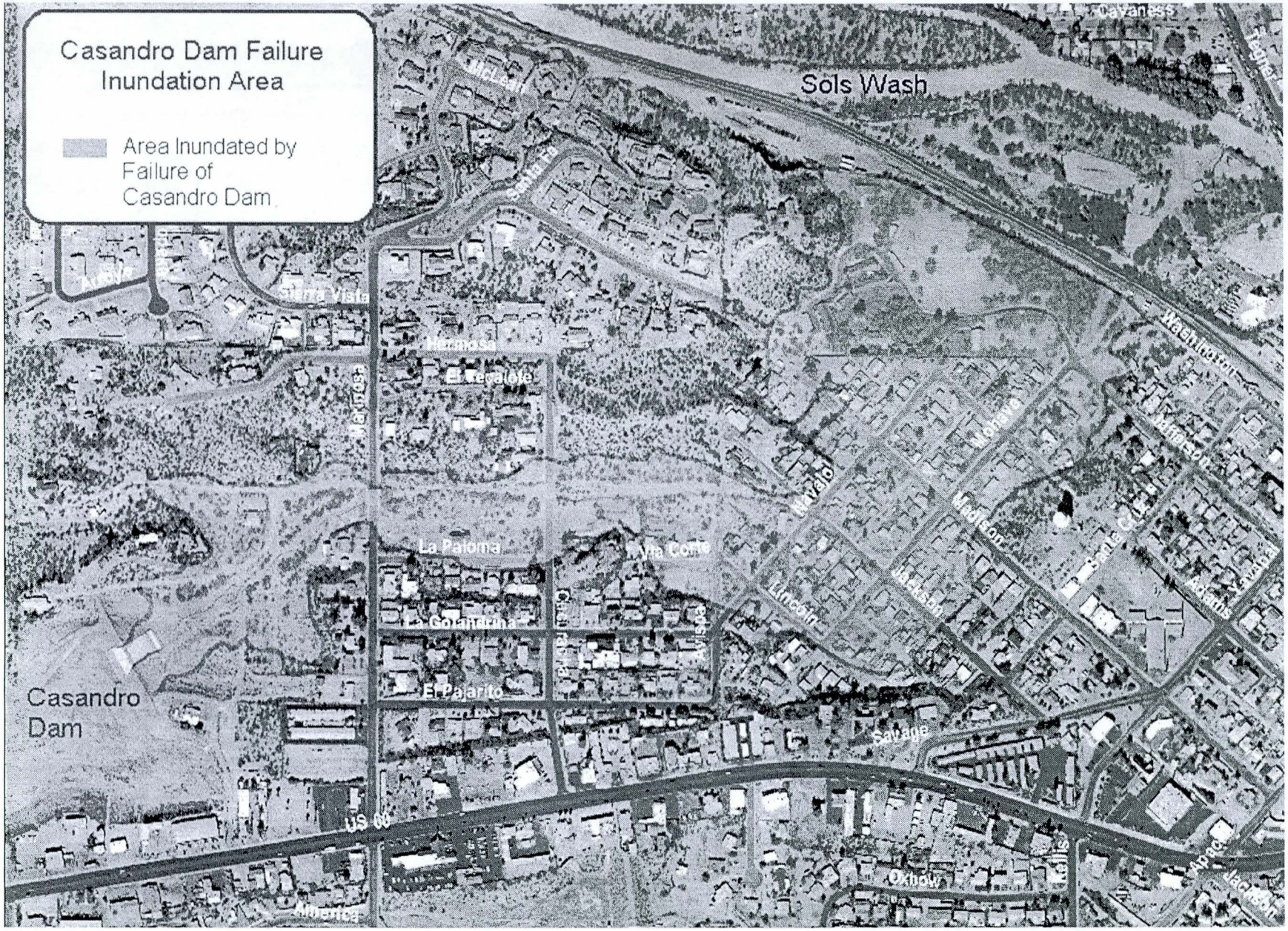
Office 602-417-2442

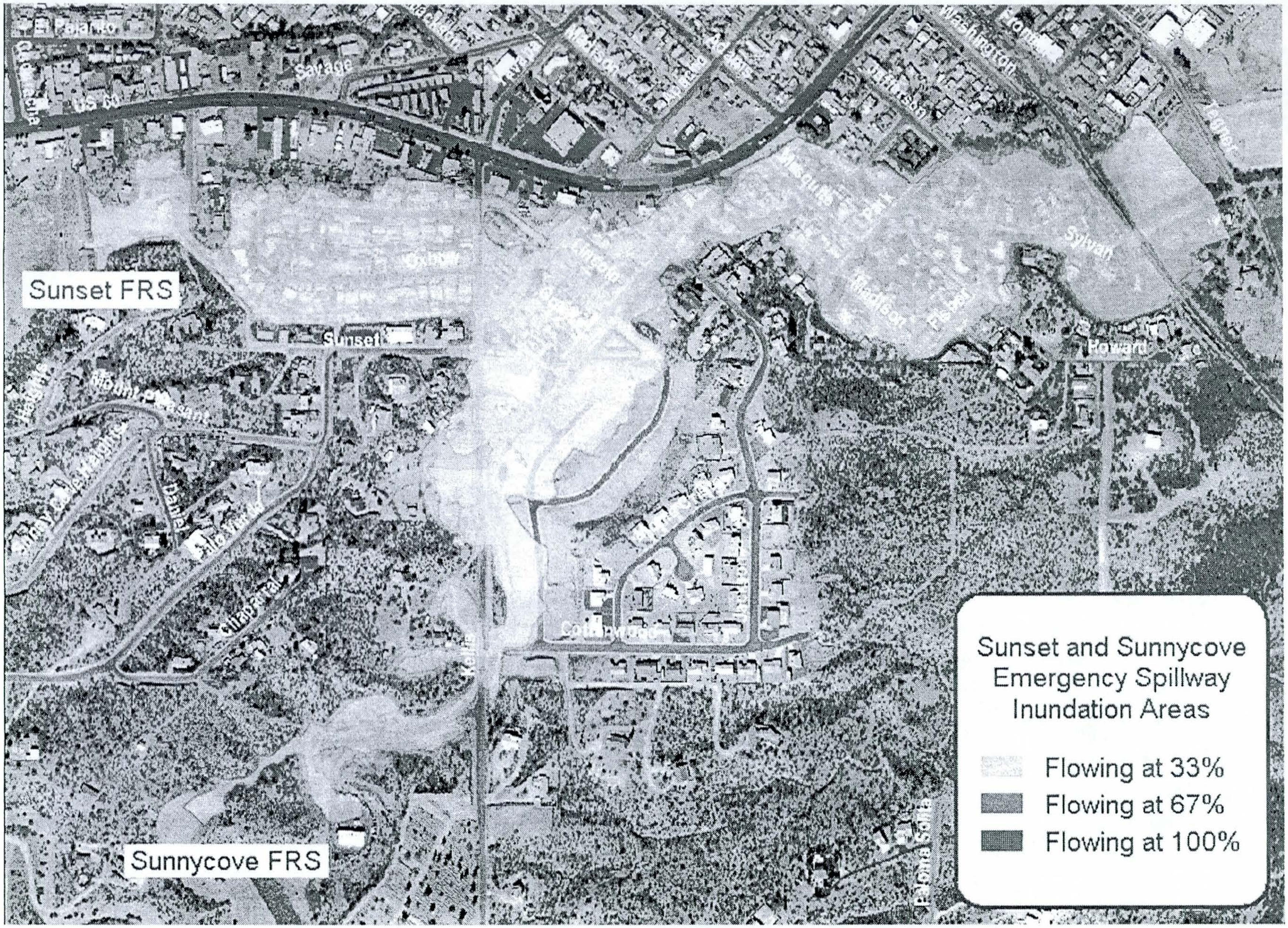
American Red Cross

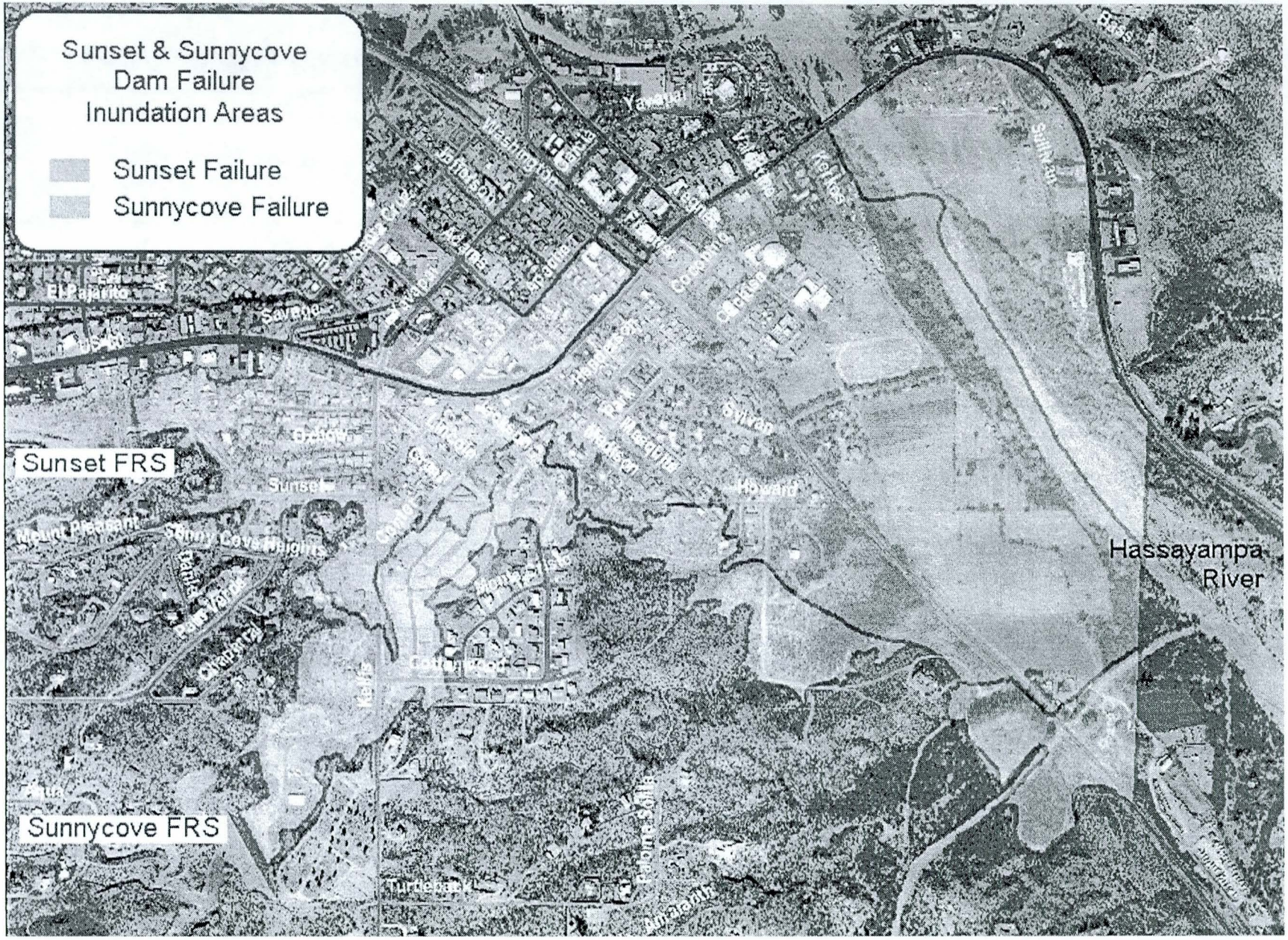
Office 602-336-6660

Appendix B – Maps of Inundation Areas


Casandro Dam
Emergency Spillway
Inundation Area
Flowing at 33%
Flowing at 67%
Flowing at 100%
Sols Wash
Sierra Vista
Mohave
Madison
Jackson
Lincoln
Savage
Casandro
Dam
US 60
La Paloma
Gotandrina
El Pajarito
Center


Casandro Dam Failure
Inundation Area
Area Inundated by
Failure of
Casandro Dam
Sols Wash
Cavaness
Tegner
McLean
Santa Fe
Arbolita
Sierra Vista
Alameda
El Regalete
Marines
Washington
Mohawk
Navajo
Madison
Jackson
Lincoln
Santa Cruz
Adams
La Maria
La Paloma
Via Corte
Casahacina
Sols
El Pajarito
Savage
Casandro
Dam
US 60
Oxbow
Kellis
Apache
Jackson
Alameda


Sunset FRS
Sunnycove FRS
Sunset
Savage
US 60
Oxbow
Sylvan
Madison
Howard
Washington
Cottonwood
Sunset and Sunnycove Emergency Spillway Inundation Areas
Flowing at 33%
Flowing at 67%
Flowing at 100%


Sunset & Sunnycove
Dam Failure
Inundation Areas
Sunset Failure
Sunnycove Failure
Sunset FRS
Sunnycove FRS
Hassayampa
River
El Pajarito
Savena
Washington
Jefferson
Yavapai
Sullivan
Kellogg
Coronado
Clarkson
Howard
Sunset
Mount Pleasant
Sunny Cove Heights
Cherry Hill
Turtle Park
Turtle Rock
Valley View
Bass

Appendix C

List of Critical Facilities by Structure Inundation Area

Structure	Facility	Location	Phone Number
Casandro	My Father's Retirement Ranch	400 N. Jefferson St.	928-684-5925
Casandro, Sunset Sunnycove	BNSF Railroad	Washington St. Alignment	602-382-5801
Sunnycove	Dell Webb Medical Care and Dialysis Center	466 W. Wickenburg Way (US 60)	623-334-3174
Sunnycove	Wickenburg Public Works Equipment Yard	Coney Orosco Rd. west of Kellis Rd.	928-684-2761
Sunset & Sunnycove	Hassayampa Upper Elementary School	251 S. Tegner St.	928-684-6750
Sunset & Sunnycove	Gate House Juvenile Recovery Center	145 W. Wickenburg Way (US 60)	928-668-1470

Settlement Monuments - Crest

Table 1 below, compares the 2003 crest settlement monument elevations with the Adjusted Design crest elevations. The settlement monuments are physically located offset from the dam centerline on the downstream edge of the dam crest. The Design Crest elevations are referenced to NGVD 1929 and must be adjusted for comparison with 2003 elevation survey data referenced to a different vertical datum: NAVD 1988. No benchmarks were referenced prior to the 2003 survey, therefore the Datum Shift utilized in the 2003 *Casandro Wash Subsidence Survey Data Review* report was assumed for the Sunnycove FRS. Details of the adjustment calculations for this datum shift can be found in "Reference Marks", page 7, of that report.

Figure 1-1 illustrates the comparison of crest settlement monument elevation between the Adjusted Design crest elevations and the 2003 survey data listed in **Table 1.**

Figure 1-2 displays the relative change in crest settlement monument elevations obtained by subtracting the Adjusted Design crest elevation data from the 2003 survey crest elevations as calculated in **Table 1**. 2003 elevation data is the first subsidence survey data available and therefore references the Adjusted Design crest elevations as the baseline elevation. Subsequent surveys should use 2003 elevations as the baseline.

Figure 2-1 on page 5 shows the location of the crest settlement monuments.

Crest Monument Survey Data				
Marker	Station	Dsgn Crest (NGVD29)	Adj. Design (NAVD88)	2003 (NAVD88)
SNYCV11	11+1.66	2178.5	2180.73	2180.95
SNYCV12	12+3.88	2178.5	2180.73	2181.453
SNYCV13	13+4.46	2178.5	2180.73	2181.606
SNYCV14	14+5	2178.5	2180.73	2181.374
SNYCV15	15+7.06	2178.5	2180.73	2181.693
SNYCV16	16+7	2178.5	2180.73	2181.36

(Fig. 1-1 Plot Data)

2003 - Adj. Design
0.22
0.723
0.876
0.644
0.963
0.63

(Fig. 1-2 Plot Data)

Note: 1) The Datum Shift to NAVD88 elevation referenced from Casandro Wash Dam is NGVD29 elevation **plus 2.23 ft.**
2) 2003 survey data collected in-house in September 2003.

Table 1
Crest Settlement Monument Elevations

Settlement Monuments - Crest

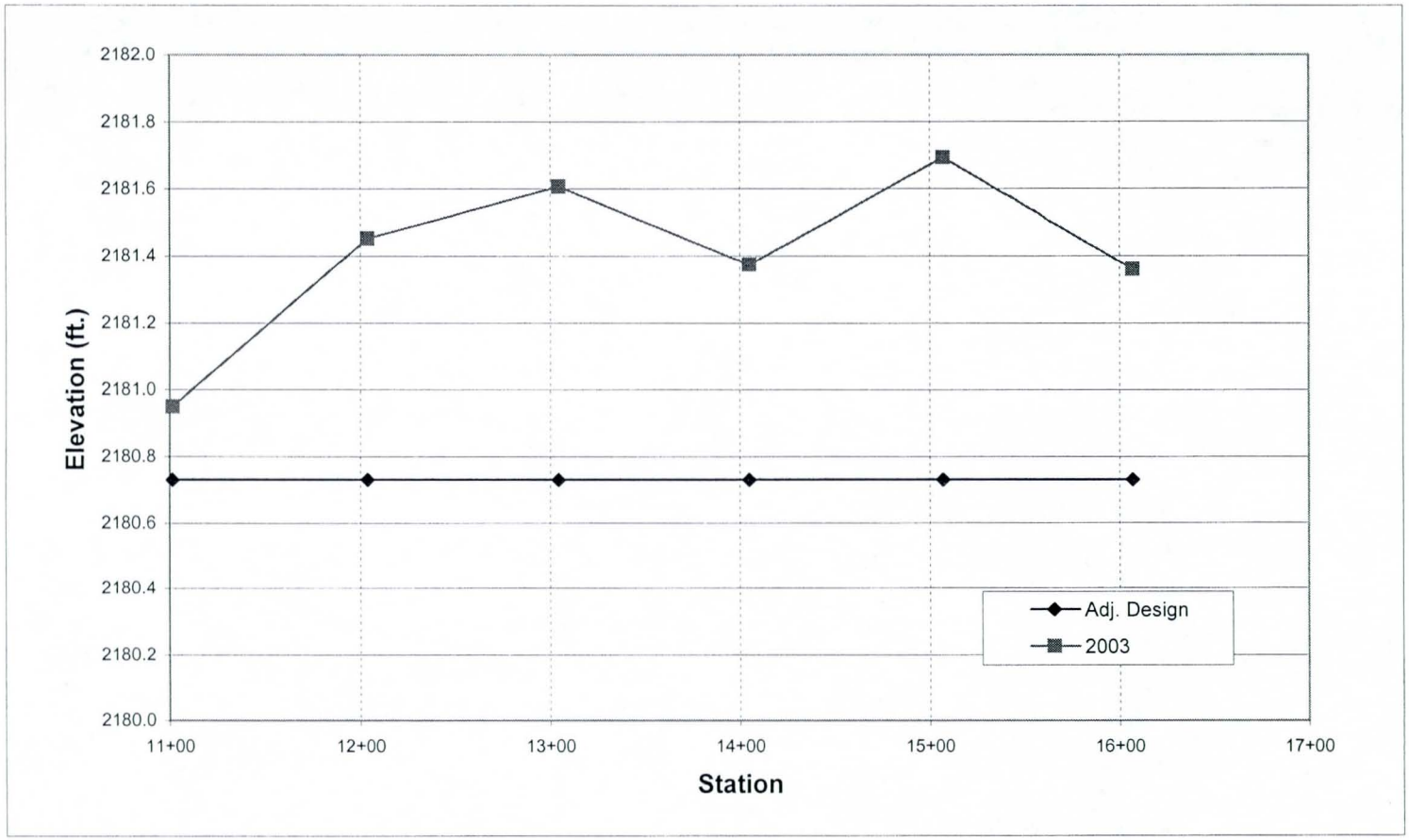


Figure 1-1

Elevation of Crest Settlement Monument Chart

Settlement Monuments - Crest

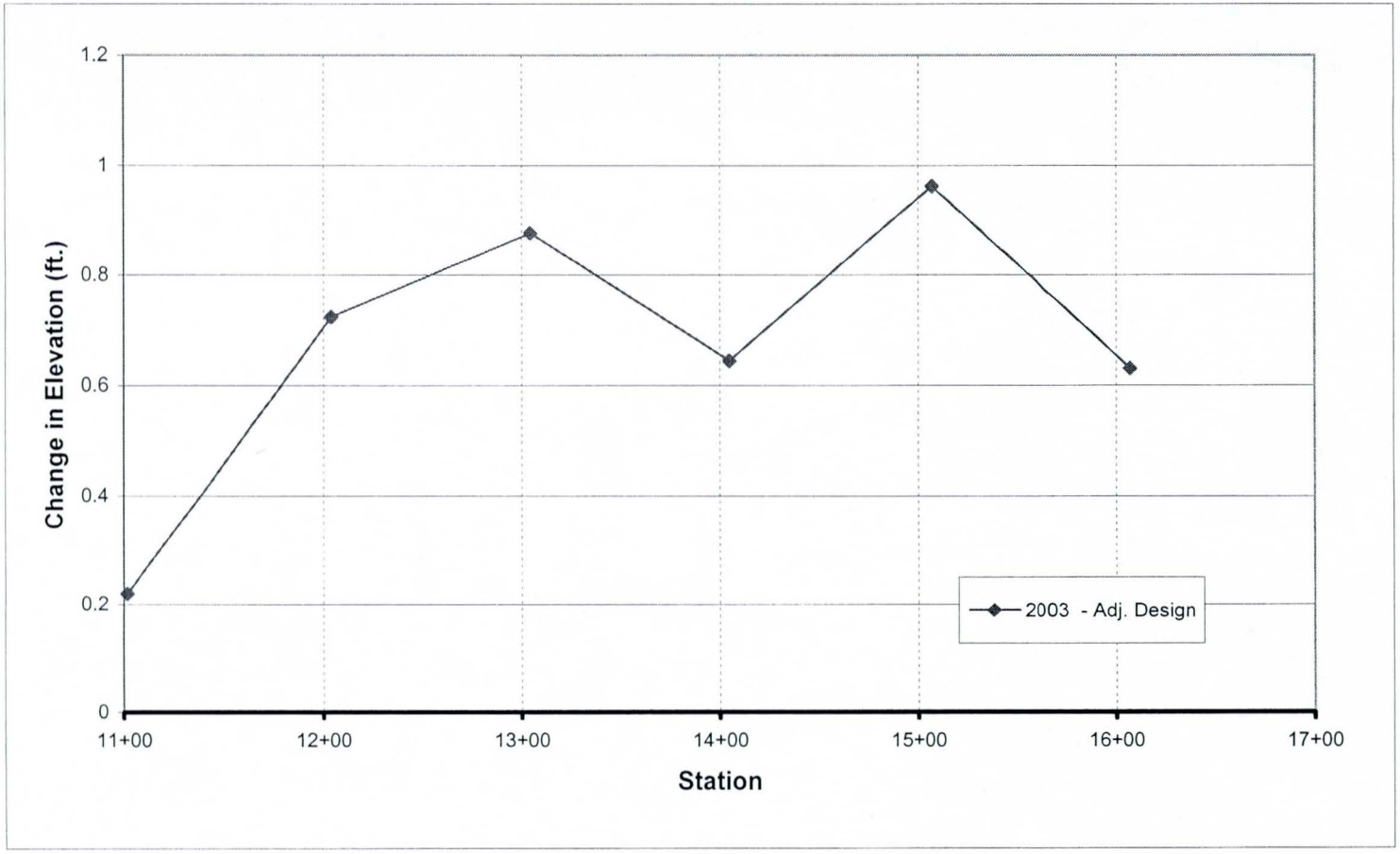


Figure 1-2

Relative Change in Dam Crest Elevation Chart, Adjusted Crest Design as Baseline Elevation Reference

Settlement Monuments - Toe
& Miscellaneous Elevations

Table 2-1 below, summarizes toe settlement monument elevations in the 2003 survey. This is the first set of survey elevation data collected from the toe monuments. Subsequent survey data on these toe monuments should be used for elevation comparisons and for illustrating the relative change in toe settlement monument elevations to 2003 survey with 2003 elevations as the baseline.

Table 2-2 below, displays the 2003 survey elevation at the benchmark to be referenced in later surveys.

Figure 3-1 on page 5 shows the location of the toe settlement monuments and benchmark.

Toe Monument Survey Data		
Marker	**Station**	**2003**
SNYCV17	13+30.79	2133.276
SNYCV18	11+8.73	2131.077
SNYCV19	15+70.91	2135.762

Table 2-1
Toe Settlement Monument Elevations

Reference Mark		
Marker	**Description**	**2003**
SNYCV10	CP BM FCD BC	2197.068

Table 2-2
Benchmark Elevation

Sunset FRS - Floodplain View

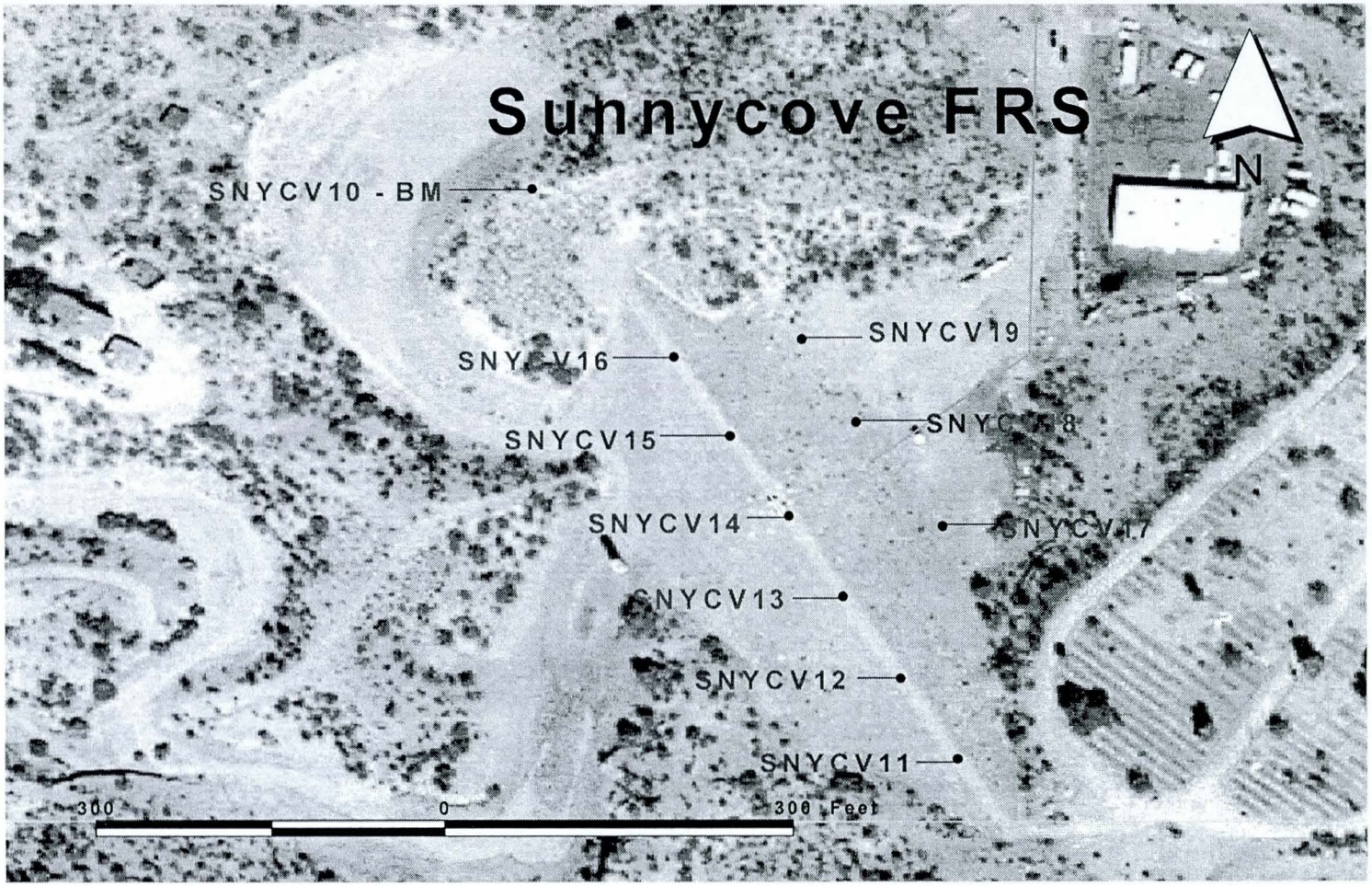


Figure 2-1
2003 Survey Monument Locations

Kimley-Horn and Associates, Inc.

PRELIMINARY FAILURE MODES IDENTIFICATION REPORT
SUNNYCOVE FLOOD RETARDING STRUCTURE
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY, ARIZONA
FEBRUARY 24, 2004

1.0 Introduction

Kimley-Horn and Associates, Inc. (KHA) has prepared this report to document discussions related to the Preliminary Failure Modes Identification workshop for Sunnycove FRS conducted on February 24, 2004. The overall objective of the workshop was to develop a comprehensive list of potential failure modes for the structure and appurtenances. The workshop was conducted at the offices of the Flood Control District of Maricopa County. The following individuals participated in the workshop:

Tom Renckly, P.E.	Flood Control District
Bob Eichinger, P.E., CFM	Kimley-Horn and Associates, Inc.
Kelli Blanchard, EIT	Kimley-Horn and Associates, Inc.
Debora Miller, Ph.D, P.E.	Gannett Fleming, Inc.
Ken Euge, R.G.	Geological Consultants, Inc.

2.0 Facility Descriptions

Sunnycove is a multi-zoned earthfill dam has a crest length of 714 ft, a crest width of 14 ft, upstream slope of 3:1 and downstream slope of 2:1 and has a maximum height of 48.5 ft. The earthlined emergency spillway is excavated through a saddle adjacent to the left abutment of the dam. The ungated outlet works consist of a multilevel, reinforced concrete tower at the upstream toe of the dam and a 30-inch reinforced concrete pipe conduit constructed through the dam at the base of the left abutment. The dam is a flood retarding structure to protect the community of Wickenburg. It was designed to never have a permanent storage pool.

3.0 Summary of Inspection Reports

Inspection reports dating from 1980 to 2003 were located and reviewed. From 1989 to 2003 there is a record of impoundments with dates and depths. In 2003 the principal spillway conduit was videotaped by the District. Gravel mulch was applied in 2003 for control of erosion rills. Before 2003 erosion was occurring along the emergency spillway approach channel, erosion gullies were forming on the downstream slope in 1997, the upstream and downstream slope experience minor erosion rills in 1996.

4.0 Preliminary Failure Modes

1. **Embankment Overtopping**: The embankment crest and downstream slope are protected against erosion. Overtopping of the embankment could lead to erosion and formation of a breach. In assessing the probability of occurrence of this failure mode, the following items need to be reviewed:

a. Review and document the freeboard available when routing the Inflow Design Flood (IDF) through the emergency spillway. The IDF for the dam is currently the ½ PMF. Check full PMF. It appears the PMF does not overtop the dam according to the Phase I Report.
b. Qualitatively assess the impact of regional subsidence on the dam crest elevation. Locate the most recent crest survey data. Initial discussion indicates that subsidence should not be a local problem.
c. Review and document the antecedent reservoir conditions for each of the spillway routings.
d. Perform a preliminary assessment to evaluate if dynamic routing of the inflow hydrograph would impact the freeboard. Apply conservative assumptions as needed. Compare "dynamic routing" approach versus "kinematic routing" or "modified-Puls" approach.
e. Review and document the most current estimate of reservoir stage capacity.
f. Review the available estimates of the Probable Maximum Precipitation (PMP). Identify the differences between each of the estimates. In particular, what factors causes a duration (6-hour or 72-hour) to become more critical?
g. Check survey crest monuments
h. Are high capacity groundwater wells having localized effect?
i. Need to check routing of IDF. (duration, depth of flow, time of impoundment)
j. Check inundation downstream of dam.

2. **Downstream Impacts**: This pertains not only to downstream impacts due to failure of one of more components of the dam, but impacts that would result from normal operations at the facility. The following are important issues that require review before the formal FMEA.
   a. Qualitatively assess downstream effects due to discharge from the emergency spillway. Qualitatively assess whether or not there would be an emergency spillway discharge during the 100-year event.
   b. Review and document the capacity of the outlet channel in light of the anticipated spillway discharges.
   c. Evaluate to the extent practical, the magnitude or frequency of storms that would result in spillway discharge.
   d. What is the EAP mapping?
   e. How fast does the reservoir fill and drain?

3. **Erosion at the Emergency Spillway (Sunnycove FRS):** There is no stabilized control section in the emergency spillway. The downstream limit of the emergency spillway terminates at a drop into a natural wash. The emergency spillway in an earth-lined channel. An erosion headcut in the emergency spillway could encroach into the reservoir pool and cause a breach to form. The following items require review:

a. During inspection, observe and document the downstream terminus of the emergency spillway into the natural wash. Observe the general condition of the earth-lined spillway.
b. Review boring logs of emergency spillway and assess to the extent practical the erosion hazard for the design flows in the spillway.
c. What is the extent of headcutting (if any) in downstream wash?

4. **Failure of Principal Outlet**: The principal outlet for the dam is a reinforced concrete pipe 30 inches in diameter. The following items require review:
   a. Review available information to assess the structural adequacy of the principal outlet.
   b. Qualitatively assess the potential for piping around the principal outlet.
   c. Inspect the intake tower of the principal outlet to assess and document is the walls have deflected due to instabilities.
   d. Review available geotechnical information to assess is the principal outlet is underlain by collapsible soils.
   e. Seepage collars around principal spillway. Question if Zone I and Zone II around pipe meets requirements for diaphragm.
   f. Visually inspect the intake tower for cracking.
   g. Does drain fill wrap fully around pipe?
   h. Is seepage out of joint possible?

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

ii. Cutoff trenches of limited depth/or no core trench - potential for concentrated seepage along base of dam/core trench
iii. Cutoff trench (near dam centerline) only extends for width of 12' (differential movement that results in cracking of core material or cracking at interface between various zones of material)
iv. Can cutoff offset in Zone 1 cause differential settlement?

d. ***Erosivity of Foundation Soils***. For dams with or without core trenches – consider erosivity of foundation soils and potential for concentrated exit gradients at unprotected toe(s) of dam(s) (under seepage during impoundment events).
e. Potential for ***earth fissures*** extending under dam(s)? Not likely.
f. ***Downstream runoff erosion***. Review and assess if discharge from natural drainages adversely impacts the downstream face or toe of the embankment.

6. **Erosion and Piping through the Embankment**: This failure mode relates to the concentrated leak piping along a transverse crack, or along a penetration through the dam (outlet pipes and utility conduits). The following are critical items that will be reviewed and assessed prior to the FMEA:
   a. ***Transverse Cracking***. Information related to identifying potential for transverse crack formation through embankment fill. Although transverse cracking has not been reported at any of the three structures, case histories on other District dams warrant the evaluation of potential failure modes related to embankment piping for all District dams.
      i. Potential for *desiccation shrinkage cracking* of clayey fill materials (review soil PI's and fines content, depth of non-clayey cover protecting clayey materials, etc).
      ii. Potential for *differential settlement-induced cracking* (transitions at cutoff trenches, collapsible soils in foundation, variability of foundation in longitudinal direction, etc.)
      iii. Discuss inability to view/inspect for transverse cracking due to rock mulch slope protection.
   b. ***Internal Filters***. Review and assess to the extent practical the level of protection against concentrated leak piping provided by internal filters. This review should also evaluate the potential for a defect through the central filter.
   c. Check for gradation data on filter/drain and core material zones. (*Filter compatibility*/filter match criteria between adjacent material zones.) Filter match of Zone I, II, III, & IV materials
   d. Check where Zone I and II start and end.
   e. Check and see if Zone II wraps fully around piping along outlet conduit.
   f. Review *internal stability* of central chimney/filter drain materials
   g. ***Penetrations through Dam***. Review drawings and information to evaluate vulnerability to piping along penetrations through dam (outlet conduits/utilities).

i. Consider outlet pipe construction methods (seepage collars, cradles, pipe bedding, etc). For example, if seepage collars were installed around principal spillway, we know that poor compaction around seepage collars has lead to piping erosion in numerous case histories.
ii. Were filter diaphragms installed, or does internal zoning around pipe meet requirements for filter diaphragms?
iii. Review utility plans

h. ***Internal zoning geometry***. Review construction details for internal zoning. Look for core/shell zones that do not extend to dam crest – if only extend to emergency spillway crest elevation – possibility of seepage "overtopping" core zone leading to erosion/loss of dam crest. (Sunset FRS) top limit (elevation vs. potential phreatic surface) for filter (Zone II) material
i. Review the characteristics of case history of FCD embankment cracking (width, spacing, depth).
j. ***Partially penetrating central filters***. Review the central filter configuration in light of maximum crack depths to evaluate the potential for piping under a partially-penetrating center filter. Zone I, II & IV materials only extend to emergency spillway crest elevation
k. Evaluate if ***animal burrows*** can serve as seepage conduits across the entire width of the embankment.

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

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

9. **Failure Mechanisms Associated with Earth Fissures**: Previous as well as current investigations by others have identified a strong potential for earth fissures at a number of FCD structures. The following issues need to be reviewed as part of the FMEA:
    a. Review current investigations to evaluate the potential for earth fissures in the vicinity of the dam.
    b. Review the geotechnical properties of the soils to assess the potential for "pipe" or "tunnel" formation through the embankment/foundation along an earth fissure.
    c. Cracking of the embankment due to one or more earth fissures. This could result in some of the failure mechanisms related to seepage and erosion piping through the embankment.
    d. Review geotechnical data to assess the stability of the upstream slope under rapid drawdown conditions. The failure mechanism is similar to that discussed in item 7(b) above, with the exception that seepage along a fissure through the foundation sild could result in loss of support due to erosion of the (as opposed to collapsible) soils.

10. **Failure Mechanisms Associated with 6-inch AC Chimney/Filter drain pipe.** The chimney/filter drain in Sunset and Sunnycove incorporates a 6-inch asbestos cement perforated drain pipe to collect seepage water. The may be a potential for failure of the drain pipe system by either clogging or structural failure by collapse. The following issues need to be reviewed as part of the FMEA:
    a. Review design and construction records for 6-inch drain pipe and drain pipe openings versus filter material size.
    b. Review 6-inch pipe strength specifications versus loading.
    c. Are there two pipes or just one?
    d. Need to verify what type of soil surrounds the chimney.
11. **Other considerations**: This section addresses issue that are not directly related to a failure of the dam or its appurtenant facilities, but which nonetheless may be relevant to the FMEA:
    a. Assess the impact of discharge from the emergency spillway on the downstream areas.
    b. Assess the impact of groundwater withdrawal in the vicinity of the dam.

5.0 Closure

The aim of the workshop on February 24, 2004 was to identify and develop a comprehensive list of failure modes for Sunnycove FRS. In addition, the participants also identified key issues that require additional review or assessment during the Individual Structures Assessment and the Field Inspections. A detailed Failure Modes and Effects Analysis (FMEA) is beyond the scope of the February 24 workshop. The FMEA for the dams is schedules as a future task of this work assignment (March 1 2004 through March 6, 2004). The list of items to be reviewed as presented in Section 4.0 above is intended to provide guidance to the risk assessment team, and does not represent

a comprehensive list of documents and information items that need to be reviewed in advance of the formal FMEA.

FIELD INSPECTION REPORT for SUNNYCOVE FLOOD RETARDING STRUCTURE

MARICOPA COUNTY, ARIZONA

Purpose

The purpose of the field examination is to provide a systematic visual field technical investigation in which the structural stability and operational adequacy of the FRS project features are analyzed and evaluated to determine if deficiencies exist at the FRS and associated project features. The examination was conducted by walking the length of the structure and visually examining the crest, upstream and downstream slopes, upstream and downstream toes, and appurtenant structures. Comments are recorded on 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 corrective measures to be taken where necessary. This examination was conducted on February 25, 2004 by the following technical examination team:

Technical Examination Team

Robert Eichinger, P.E., CFM	Project Manager, Kimley-Horn and Associates
Debbie Miller, Ph.D., P.E.	Senior Geotechnical Engineer, Gannett Fleming
Ken Euge, P.G.	Principal Geologist, Geological Consultants
Enamul Hoque, P.E.	Principal, Hoque & Associates, Inc.
Kelli Blanchard, EIT	Civil Analyst, Kimley-Horn and Associates
Mike Meng	Structures Technician, Flood Control District of Maricopa County

Operational Summary

Inspection Frequency: Sunnycove FRS is inspected on an annual basis by the Flood Control District (FCD). In addition to the annual inspections, the District conducts quarterly operation and maintenance inspections, flood related event inspections, and as-needed site inspections. The Arizona Department of Water Resources and the Natural Resources Conservation Service are invited to participate in annual inspections of Sunnycove FRS.

Maximum Water Surface Elevations: The District maintains a log of maximum water surface elevations for Sunnycove FRS. The maximum recorded impoundment for

Sunnycove reservoir is 53 acre-feet with a stage of 21.68 feet (gage height) at the FRS (August 1992).

Emergency Spillway Discharges. Based on District records, there has been no recorded emergency spillway flows at Sunnycove FRS. The spillway is an earth-lined located separate from the dam embankment off the left abutment.

Distress Observations Corrected or Operation and Maintenance Conducted Since Last Inspection: In 2003, the District has placed gravel mulch on the embankment slopes to help control slope erosion and rilling.

Past Distress Observations Not Yet Corrected: None

Flood Control District Operation and Maintenance Responsibilities: The District is responsible for total operation and maintenance of Sunnycove FRS and associated appurtenances.

Field Examination Results Summary

Embankment: The crest of the FRS is gravel plated. All crest settlement monuments were found. Station markers were located. The crest is clear of vegetation. The access gates and fences are operational. No longitudinal cracks or transverse cracks were observed on the crest of the dam.

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

Upstream Slope: There are several animal burrows on the slope face. There was no evidence of seepage, undermining, settlement or sloughing. There is gravel rock mulch protection on the slope. The inspection team noted an area of the upstream slope face that appeared to have a bulge. This area is located between Stations 11+50 and 12+50 approximately mid-height on the embankment. Review of as-built plans for the profile on dam centerline indicates a high area in foundation. There is gravel rock mulch protection on the slope.

Downstream Slope: Animal burrows are evident on this slope face primarily on the lower one-third of the slope. These burrows appear to be attributable to rodents. The slope has a low density of small shrubs and grasses. There was no evidence of seepage, undermining, settlement or sloughing. There is gravel rock mulch protection on the slope.

Principal Spillway: The approach channel was clear of debris and obstructions. The reservoir pool is clear of vegetation and debris. However, a soil material stockpile (40 ft diameter at the base and approximately 15 ft in height) was dumped in the channel approximately 300 feet upstream of the intake tower. Discussions with FCD operation

and maintenance regarding the soil stockpile indicated that the stockpile was dumped illegally and not attributable to FCD O&M for sediment removal activities.

The exterior of the inlet tower was clean. The concrete for the inlet tower structure showed no signs of structural distress. The trash rack was clear of debris and obstructions. The interior of the principal spillway conduit was not inspected visually. However, the District has videotaped the interior of the conduit. A review of the video indicates that the conduit is clear of debris and obstructions. The conduit was clean and there were no apparent signs of seepage.

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

Emergency Spillway: The emergency spillway is located off the left abutment. The FRS emergency spillway is an earth-lined spillway that terminates into a natural wash. The spillway is clear of any obstructions.

Instrumentation: Sunnycove FRS has six crest settlement monuments. The crest monuments are located just off the downstream crest of the structure. The crest monuments appeared to be undamaged. The structure also has three downstream toe monuments. An FCD benchmark monument is located on the hilltop between the left abutment and emergency spillway. All monuments were found and in good condition.

A staff gauge located on the upstream slope at the principal spillway is used to indicate the level of water impounded in the reservoir. A pressure transducer is located at the inlet structure of the principal spillway. The transducer works in combination with a flood warning telemetry system, which allows signals to be sent to a centralized receiver at the District indicating water levels at the reservoir.

Conclusions

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

Recommendations

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

a. Remove soil stockpile from upstream channel.
b. Monitor and inspect bulge in upstream slope between Stations 11+50 to 12+50.

Next Annual Inspection

The next annual inspection by FCD is scheduled for November 2004.

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

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

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

ADWR NO.: **07.48** FCD NO. **341**	DAM NAME: **Sunnycove FRS**		TYPE: **Zoned Earthfill with Chimney Drain**	NOT APPLIC.	NO	YES	MONITOR	REPAIR	INVESTIGATE
CONTACTS:		REPORT DATE: **March 24, 2004**							
INSPECTED BY: **KHA Team (See IGA Report)**		INSPECTION DATE: **February 25, 2004**							
REVIEWED BY: **Bob Eichinger**		REVIEW DATE: March 24, 2004	PAGE **1 of 6**						
SPILLWAY CREST ELEVATION: **2170 ft. (NGVD 29)**	HAZARD CLASS: **High**		SIZE: **Small**						
INFLOW DESIGN FLOOD: **½ PMF**	SPILLWAY CREST WIDTH: **100 ft.**		ADWR DAM HEIGHT: **40 ft.**						
DAM CREST LENGTH: **709 ft.**	DAM CREST WIDTH: **14 ft.**		DAM CREST ELEV.: **2178.5 ft. (NGVD 29)**						
CURRENT RESERVOIR LEVEL: **Empty**	TOTAL FREEBOARD: **8.5 ft.**		PHOTOS: **Yes (8)**						
Item	Comments								

1. CREST - **Width = 14 ft., Length = 709 ft., Dam Crest Height = 48.5 ft.; Elevation = 2178.5 ft. (NGVD 29) [Ref. 1976 As-builts]**						
a. Settlements, slides, depressions?		✓				
b. Misalignment?		✓				
c. Longitudinal/Transverse cracking?		✓				
d. Animal burrows? Minor			✓			
e. Adverse Vegetation?		✓				
f. Erosion? **Gravel mulch applied to upstream face.**		✓				
2. UPSTREAM SLOPE - **3 Horizontal : 1 Vertical Zone III (minus 9-inch random fill) shell over 1.5H:1V Zone IV (minus 6-inch random fill) and upstream sloping 0.5H:1V clay core. The clay core has a shallow 12 ft. wide cutoff trench.**						
a. Erosion? Small rills –		✓				
b. Inadequate ground cover? **Approximately 2/3 down the slope mulch stops**		✓				
c. Adverse vegetation?		✓				
d. Longitudinal/Transverse cracking?		✓				
e. Inadequate riprap? **Gravel mulch applied in 2003.**	✓					
f. Stone deterioration?	✓					
g. Settlements, slides, depressions, bulges? **Minor Bulge, not a cause of mulch build-up.**			✓	✓		✓
h. Animal burrows?		✓				
3. DOWNSTREAM SLOPE - **2 Horizontal : 1 Vertical**						
a. Erosion? **Gravel mulch applied in 2003. Minor erosion on left abutment groin**		✓	✓			
b. Inadequate ground cover?		✓				
c. Adverse vegetation?		✓				
d. Longitudinal/Transverse cracking?		✓				
e. Animal burrows?		✓				
f. Settlements, slides, depressions, bulges?		✓				
g. Soft spots or boggy areas?		✓				

Item	Comments	N/A	NO	YES	MON	REP	INV
h. Movement at or beyond toe?			✓				
4. DRAINAGE-SEEPAGE CONTROL – **6 ft. wide chimney filter/drain composed of Zone II drain fill downstream of the clay core for the full width of the dam. Zone II drain fill extends from elevation 2165 ft. to the foundation and from the chimney filter/drain around the principal spillway outlet. A 6 inch diameter perforated AC pipe is located at the base of the chimney filter/drain. Two drain outlets are located within the Zone II drain fill around the principal spillway and one is located about 70 ft. left of the principal spillway.**							
a. Internal drains flowing?	Est. Left ___gpm; Est. Right ___gpm		✓				
b. Boils at or beyond toe?			✓				
c. Seepage at or beyond toe?	Estimated ___gpm		✓				
d. Does seepage contain fines?		✓					
5. ABUTMENT CONTACTS							
a. Erosion? Very minor erosion at left abutment				✓			
b. Differential movement?			✓				
c. Cracks?			✓				
d. Settlements, slides, depressions, bulges?			✓				
e. Seepage?	Est. Left ___gpm; Est. Right ___gpm		✓				
f. Animal burrows?			✓				
6. PRINCIPAL SPILLWAY - APPROACH CHANNELS – **Not applicable (intake structure in pool area)**							
a. Eroding or backcutting?		✓					
b. Sloughing?		✓					
c. Restricted by vegetation?		✓					
d. Obstructed with debris?		✓					
e. Silted in?		✓					
7. PRINCIPAL SPILLWAY - INLET STRUCTURE - **43 ft. high concrete intake tower (Standard NRCS inlet structure) with a 2½ ft. x 7½ ft. internal dimension. The top of the opening is at elevation 2169.5 ft. An 18-inch slide gate is located on the left side of the intake tower at the base.**							
a. Seepage into structure? **Unknown. Reservoir empty and there is no indication of seepage observed.**			✓				
b. Debris or obstructions?			✓				
c. If concrete, do surfaces show:							
1. Spalling or Scaling?			✓				
2. Cracking? **Minor cracking (non-structural), no repairs required.**			✓				
3. Erosion?			✓				
4. Exposed reinforcement?			✓				
d. If metal, do surfaces show: **Corrugated metal pipe is part of the trash rack system. (see Section f. below)**							
1. Corrosion?		✓					
2. Protective coating deficient?		✓					
3. Misalignment or spilt seams?		✓					
e. Do the joints show:							

INSPECTED BY: **KHA Team (see ISA Report)** | DATE: **February 25, 2004**

Item	Comments	N/A	NO	YES	MON	REP	INV
1. Displacement or offset?		✓					
2. Loss of joint material?		✓					
3. Leakage?		✓					
f. Are the trash racks:							
1. Broken or bent?			✓				
2. Corroded or rusted? **Minor – spot rusting, no repairs required.**			✓				
3. Obstructed?			✓				
g. Principal Spillway Gate(s): **18 inch flat back slide gate**							
1. Broken or bent?			✓				
2. Corroded or rusted? **Minor rusting present**			✓				
3. Leaking? **Unknown Visual appearance indicates gate is not leaking.**			✓				
4. Not seated properly? **Visual appearance indicates gate is properly seated.**			✓				
5. Not operational?			✓				
6. Not periodically maintained?			✓				
7. Date last operated? **Gates operated quarterly**							
8. PRINCIPAL SPILLWAY – CONDUIT – **30-inch diameter RCP – discharges to hooded overflow with 15-inch diameter RCP. Pipeline diameter increases in size to 18-inches until it joins with the Sunset pipeline.**							
a. Seepage into conduit? **No visual indication of seepage into conduit.**			✓				
b. Debris present?			✓				
c. If concrete, do surfaces show: **Could not inspect. Video taped the pipe (2003).**							
1. Spalling or scaling?			✓				
2. Cracking?			✓				
3. Erosion?			✓				
4. Exposed reinforcement?			✓				
5. Other?			✓				
d. If Metal, do surfaces show: **Could not inspect.**							
1. Corrosion? Minor – spot corrosion.		✓					
2. Protective coating deficient?		✓					
3. Misalignment or spilt seams?		✓					
e. Do the joints show: **Did not inspect. Pipe video taped in 2003. See 8.c above.**							
1. Displacement or offset?			✓				
2. Loss of joint material?			✓				
3. Leakage?			✓				
9. PRINCIPAL SPILLWAY - STILLING BASIN/POOL							
a. If concrete, do surfaces show:							
1. Spalling or Scaling?		✓					

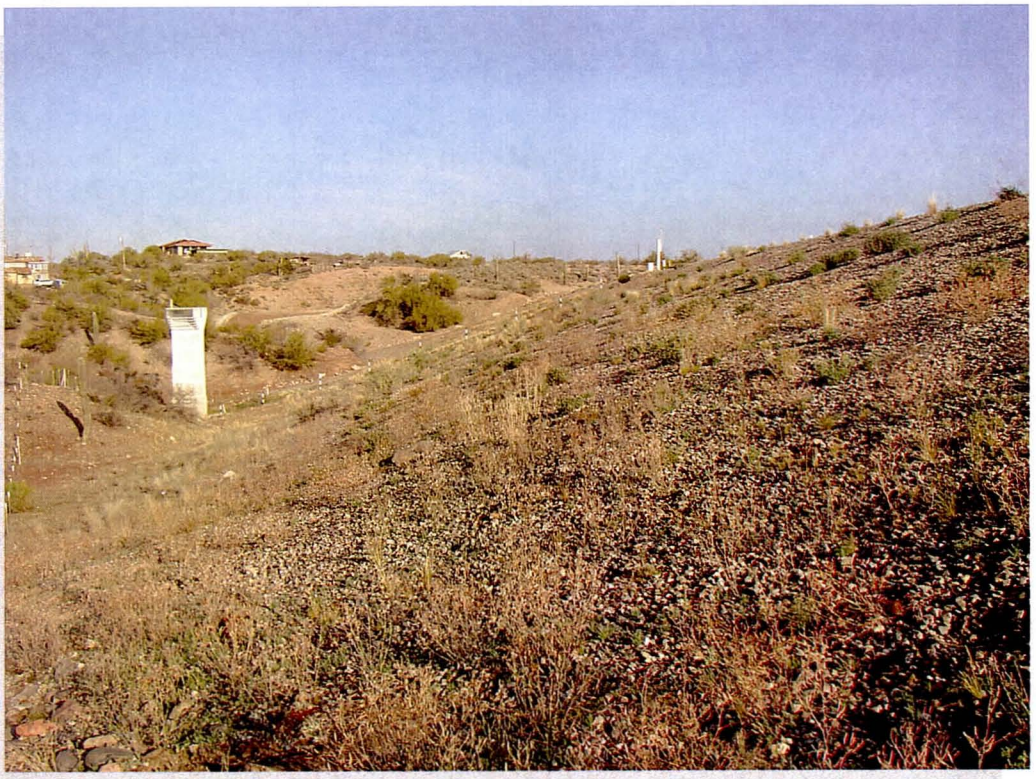
Item	Comments	N/A	NO	YES	MON	REP	INV
2. Cracking?		✓					
3. Erosion?		✓					
b. If concrete, do joints show:							
1. Displacement?		✓					
2. Loss of joint material?		✓					
3. Leakage?		✓					
c. Do the energy dissipaters show:							
1. Signs of deterioration?		✓					
2. Covered with debris?		✓					
3. Signs of inadequacy?		✓					
10. PRINCIPAL SPILLWAY – OUTLET CHANNEL - **Pipe joins with principal spillway pipe from Sunset and discharges into natural channel. Overflow discharges through hooded overflow at the downstream toe of the dam.**							
a. Eroding or backcutting?			✓				
b. Sloughing?			✓				
c. Obstructed?			✓				
d. Poorly riprapped?		✓					
e. Tailwater elevation and flow condition: **Unknown.**							
11. EMERGENCY SPILLWAY-APPROACH CHANNEL - **Unlined 100 foot wide channel left of the dam (not contiguous with dam).**							
a. Eroding or backcutting? **There is no indication of any past discharges through the emergency spillway.**			✓				
b. Sloughing?			✓				
c. Restricted by vegetation?			✓				
d. Obstructed with debris?			✓				
e. Silted in?			✓				
12. EMERGENCY SPILLWAY-CONTROL STRUCTURE – **None (Uncontrolled graded channel)**							
a. If concrete, do surfaces show:							
1. Spalling or scaling?		✓					
2. Cracking?		✓					
3. Erosion?		✓					
4. Exposed reinforcement?		✓					
b. If concrete, do joints show:							
1. Displacement or offset?		✓					
2. Loss of joint material?		✓					
3. Leakage?		✓					
c. If spillway is unlined:							
1. Are slopes eroding?			✓				
2. Are slopes sloughing?			✓				

Item	Comments	N/A	NO	YES	MON	REP	INV
3. Is crest eroding?			✓				
d. Is weir in poor condition?		✓					
e. Where is control structure? **Open channel. No concrete control section.**							
13. EMERGENCY SPILLWAY – CHANNEL - **Unlined.**							
a. Obstructions or restrictions?			✓				
b. If concrete, do surfaces show:							
1. Spalling or scaling?		✓					
2. Cracking?		✓					
3. Erosion?		✓					
4. Exposed reinforcement?		✓					
c. If concrete, do joints show:							
1. Displacement or offset?		✓					
2. Loss of joint material?		✓					
3. Leakage?		✓					
d. If an unlined channel, does it show:							
1. Erosion?			✓				
2. Slopes sloughing?			✓				
3. Poorly protected w/ vegetation/riprap? **Bedrock excavation.**		✓					
14. EMERGENCY SPILLWAY-TERMINAL STRUCTURE - **None**							
a. If concrete, do surfaces show:							
1. Spalling or scaling?		✓					
2. Cracking?		✓					
3. Erosion?		✓					
4. Exposed reinforcement?		✓					
b. If concrete, do joints show:							
1. Displacement or offset?		✓					
2. Loss of joint material?		✓					
3. Leakage?		✓					
c. Do the energy dissipaters show:							
1. Signs of deterioration?		✓					
2. Covered with debris?		✓					
3. Signs of inadequacy?		✓					
15. EMERGENCY SPILLWAY – OUTLET CHANNEL **Unlined**							
a. Eroding or backcutting?			✓				
b. Sloughing?			✓				

INSPECTED BY: **KHA Team (see ISA Report)** | DATE: **February 25, 2004**

Item	Comments	N/A	NO	YES	MON	REP	INV
c. Obstructed or restricted?			✓				
16. RESERVOIR							
a. High water marks? Evident on intake tower				✓			
b. Erosion/Slides into pool area?			✓				
c. Sediment accumulation? Amount unknown – would require a silt survey.				✓			
d. Floating debris present? **Minor amount – would not affect spillway operation.**				✓	✓		
e. Depressions, sinkholes or vortices?			✓				
f. Low ridges/saddles allowing overflow?			✓				
g. Structures below dam crest elevation?			✓				
17. INSTRUMENTATION							
a. List type(s) of instrumentation: **Reservoir gage, alert gage and settlement monument points recently installed.**							
b. Any repair or replacement required?			✓				
c. Last monitoring report: **Surveyed in 2003.**							
18. CONDITION SUMMARY / EAP / MAINTENANCE RECOMMENDATIONS / NEXT INSPECTION							
a. Any safety deficiencies? **None.**							
b. Safe storage level on License: **Emergency spillway crest (temporary only – until reservoir can be emptied).**							
c. Date of current ADWR License: **July 23, 1986**							
d. Any ADWR Actions Outstanding? Describe and list required action:			✓				
e. Recorded size: **Small** Should size be revised?			✓				
f. Recorded downstream hazard: **High** Should hazard be revised?			✓				
g. Date of last Emergency Action Plan revision **Latest MCDEM Emergency Response Manual dated November 19, 2003** Should EAP be revised? **Blue Alert page of the on-line Wickenburg Response Plan needs to be revised to include the reference to MCDEM's Emergency Action Plans Manual (Note: Current phone numbers maintained by MCDEM). EAP updated December 2003.**				✓		✓	
h. Normal inspection frequency: **ANNUAL** Should inspection frequency be revised?			✓				
i. Maintenance Recommendations: **1) Monitory buldge in upstream slope face at approximate station 10+04 to 11+00.**							
j. Is Supplemental Inspection required: **None**							
k. Recommended date for next inspection: **November 2004.**							
l. Status of Structures Assessment Program: **Scheduled for FY 2003-2004.**							

ATTACHMENTS: Photos (9)

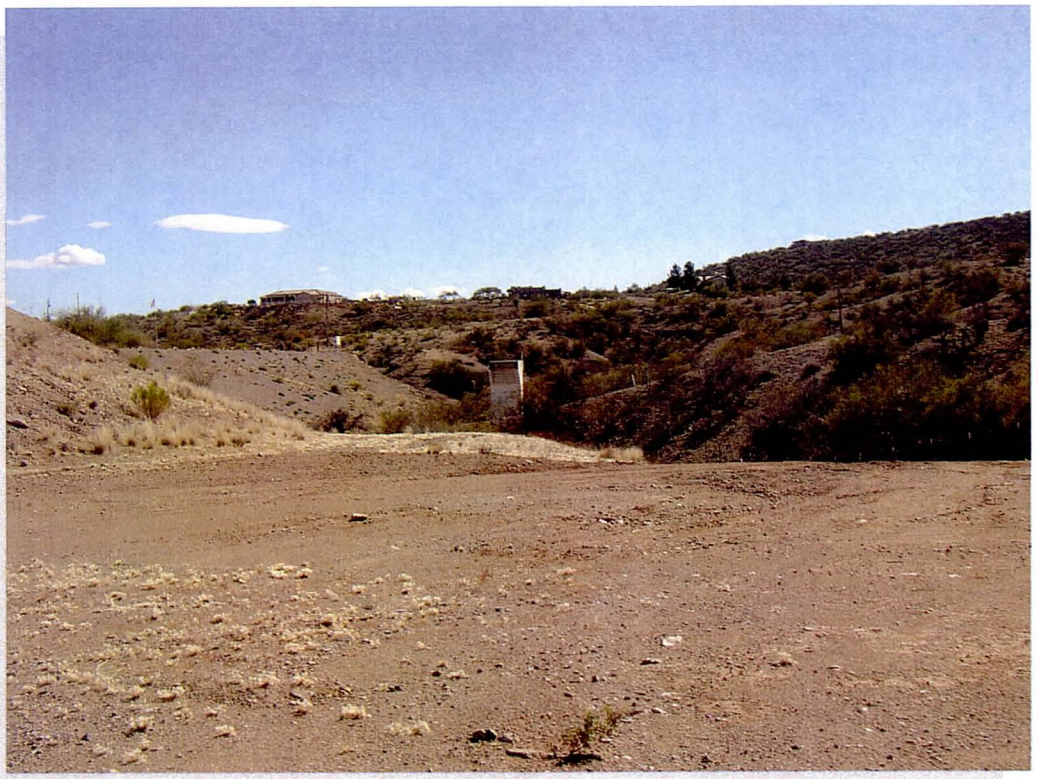


Sunnycove FRS Upstream Slope Facing West. Photo Date: 2/25/04

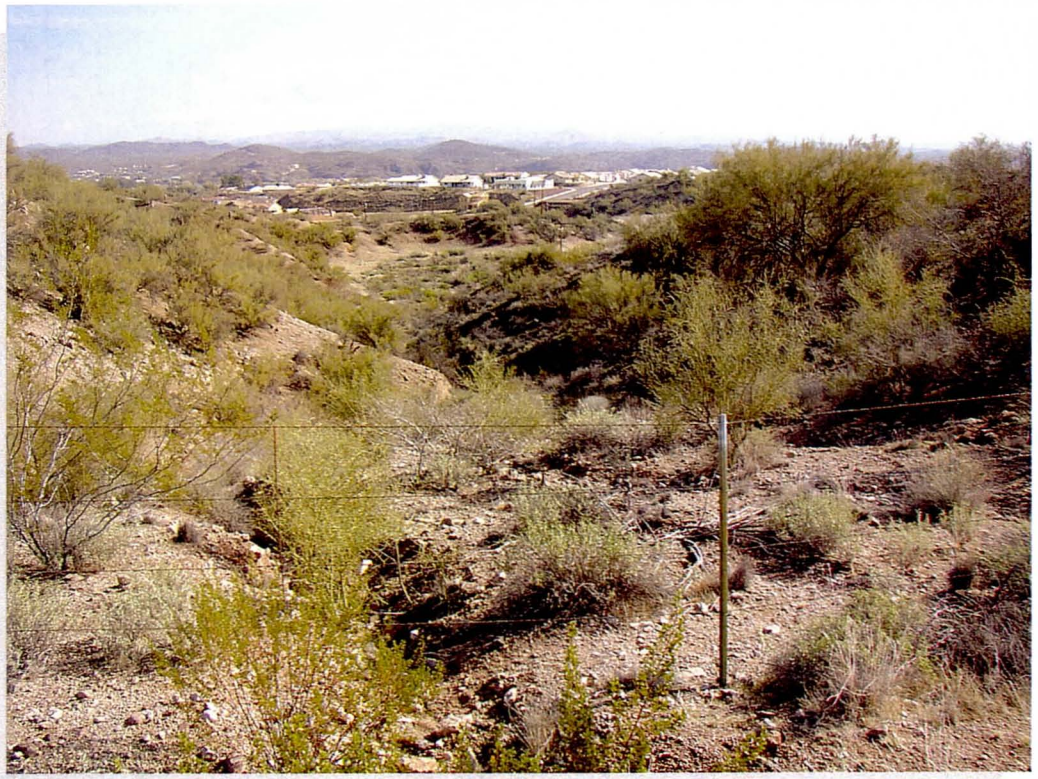
Sunnycove FRS Downstream Slope Facing West. Photo Date: 2/25/04

Sunnycove FRS Principal Spillway Overflow Outlet Facing North. Photo Date: 2/25/04

Sunnycove FRS Principal Spillway Inlet Tova and Staff Gages Facing South. Photo Date: 2/25/04



Sunnycove FRS Emergency Spillway Facing Southeast. Photo Date: 2/25/04



Sunnycove FRS Emergency Spillway End of Channel Facing North. Photo Date: 2/25/04

Sunnycove FRS Downstream Slope Facing East. Photo Date: 2/25/04

Sunnycove FRS Crest Facing East. Photo Date: 2/25/04

FAILURE MODE AND EFFECTS ANALYSIS for SUNNYCOVE FLOOD RETARDING STRUCTURE

MARICOPA COUNTY, ARIZONA

June 30, 2004

1.0 INTRODUCTION

General Description

Sunnycove FRS is located in the Town of Wickenburg, Arizona south of U.S. 60 and west of Kellis Road. The dam project consists of the FRS embankment and central chimney drain/filter carried to elevation 2165 (13.5 feet below the crest), an earth-lined emergency spillway located beyond the left abutment of the dam, an upstream intake tower that provides the inlet to the principal spillway, and a downstream outlet storm drain system with overflow structure. The principal spillway discharge is conveyed in a pipe system that is a joint system with the Sunset FRS principal outlet. The project is part of the Wickenburg Watershed Protection and Flood Prevention Project.

Sunnycove FRS is currently classified as small sized, high hazard dam. The reservoir behind the dam is 18 acres with a capacity of 219 acre-feet to the emergency spillway. Design of Sunnycove FRS was completed by the Soil Conservation Service. Construction of the FRS and appurtenant structures was completed by US Soil Conservation Services in September 1976.

The dam has performed satisfactorily to date. The dam has experienced several impoundments of various depths since construction with the largest rising to within 5 feet of emergency spillway crest. There has been no discharge through the emergency spillway to date.

Dam Data

Dam type: Zoned Earthfill with a partial height chimney drain.
Dam height: 48.5-ft
Dam length: 714-ft
Dam crest: width 14-ft; elevation: 2178.5-ft (NGVD29)
Spillways: Principal - 30-inch RCP; inlet elevations: low stage orifice invert: 2148 ft, slide gate invert elevation 2128 ft; principal spillway crest elevation 2169.5-ft;
Emergency spillway: 100-ft wide earth-lined earth channel left of dam, not contiguous with dam; emergency spillway crest elevation: 2170-ft
Freeboard: 0.7-ft
Reservoir Surface: 18-ac at emergency spillway crest
Storage: 219 ac-ft at emergency spillway crest, 375 acre-feet at dam crest

Hazard Classification: High

Hydrology Data (elevations in NGVD29 datum)

- Probable Maximum Precipitation: 14.9 inches in a 6-hour period (11.5 inches occurs in one hour) (Corps of Engineers Phase I Inspection)
- 100-year 6-hour: 3.4 in, and 100-year 24-hour: 4.2 in
- SCS Inflow Estimate: 5168 cfs Outflow Estimate: 4858 cfs
- HMR 49 Inflow Estimate: 7709 cfs, Outflow Estimate: 7188 cfs
- Emergency Spillway Design: 6300 cfs at 2177.3 ft; Corps shows: 8250 cfs (at 2180.0 ft or 2178.5 ft)
- Emergency Spillway Crest elevation: 2170.0 ft (NGVD29)
- Design Dam Crest elevation: 2178.5 ft (up to 2180.0 ft including camber over most of central portion of the FRS)
- Estimated Minimum Dam Crest Elevation based on Survey of Monuments: 2178.7 ft; most of the crest is at: 2179.0 ft
- Estimated Freeboard under PMF: Approximately 1 ft
- Reservoir Volume: 218 ac-ft at spillway crest; 375 ac-ft to top of dam
- Flood Routing Conditions– Corps Phase I: assumed full pool at spillway crest when PMF routed and outlet assumed plugged; SCS routing starts at 10-day drawdown pool (elev. 2154) and allows principal outlet to contribute.
- Outlet Capacity: Outlet orifice discharge is 9.1 cfs at elevation 2148.0 ft out of discharge pipe.
- Peak reservoir elevation under PMF routings: SCS routing: 2177.3 ft; HMR-49 Flood Routing: 2177.8 ft
- SCS 10-day drawdown requirement was to elevation 2148.0 ft (invert elevation of the 8-in by 8-in orifice)
- Orifice: SCS design draw down limits which established the 8-in by 8-in orifice opening were based on a 100-year event with a 1-day rainfall of: 4.2 in; and a 10-day rainfall of : 6.5 in.

Purpose and Scope

In general, the purpose of the Failure Mode and Effects Analysis (FMEA) exercise was to:

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

(Note: In this phase, the FMEA team examined the general nature of the "consequences" for the failure modes identified. Greater detail on the estimate of

the magnitude of the "consequences" for each significant failure mode may need to be addressed at some future time.)

Team Members

Tom Renckly, P.E., Flood Control District of Maricopa County, Project Manager,
Larry Lambert, P.E., Flood Control District of Maricopa County, Dam Safety Engineer
Bob Eichinger, P.E., CFM, Kimley-Horn and Associates, Inc., Project Manager
Larry Von Thun, P.E, Dam Consultant and FMEA Facilitator
Debbie Miller, P.E., PhD, Gannett Fleming, Inc. Geotechnical
Ken Euge, R.G., Geological Consultants, Geology
Kelli Blanchard, E.I.T, Kimley-Horn and Associates, Inc, Session Recorder
Nicole Spence Gibson, Arizona Department of Water Resources, Office of Dam Safety

2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

The following is a summary of the major findings and understandings for Sunnycove Flood Retarding Structure as a result of the Failure Mode and Effects Analysis (FMEA). Sunnycove FRS is one of three dams located in relatively close proximity of each other in Wickenburg, Arizona (the other two are Sunset FRS and Casandro Wash Dam).

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 Sunset FRS are given. Findings related to failure modes or adverse consequences for overtopping and spillway discharge are given next. Findings related to consequences are given next followed by action items (risk reduction and investigations). Finally, general findings that are informational for the dam is provided.

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

1. Geologic Investigation Adequate. The geological investigation completed for this dam was adequate with supporting documentation available to review. These investigations did not reveal any fundamental, unaddressed, weaknesses or flaws in the foundation that would suggest a potential failure mode.
2. Zone II Materials Provides Protection. The Zone II material is providing significant filtering protection against piping/seepage erosion up to elevation 2165 ft, essentially eliminating the possibility of any potential failure mode related to any "normally developed" phreatic surface developed through the FRS.
3. The Dam Foundation is Sound. The entire dam foundation and cutoff are founded in sound fanglomerate. Therefore no potential failure mode related to compressibility of foundation material is reasonable to postulate.
4. The Cutoff was Well Shaped but the Foundation Outside of the Cutoff had Some Irregularities. The cutoff trench was well shaped but natural irregularities existed in the foundation and these were not smoothed. The team was initially quite concerned

about the potential for cracking above these discontinuities but then realized that the abrupt appearance on the drawings was due to use of an exaggerated vertical scale. Further the most abrupt changes in slope were deep within the embankment.

5. No Fissures, Collapsible Soils, or Subsidence. Evidence and reports do not indicate the possibility for fissures, collapsible soils, or subsidence at the FRS. The possibility of the presence of earth fissuring or subsidence in the vicinity of the FRS or region around the FRS is considered to be very low due to the geologic subsurface conditions in the region (bedrock and shallow alluvium materials dominate the area). The lack of studies concerning subsidence or experience with subsidence in this region also illustrates the very low potential for fissuring or subsidence.
6. The Embankment is Constructed Using Material Zones but the Soil Materials Comprising the Zones do not Have Drastically Different Properties. The different Zoned soils may not be greatly different from each adjacent Zone and therefore are not likely to be prone to or contribute to differential settlement and potential cracking, but provide only a small amount of defense against seepage erosion progression. Therefore the dam is less vulnerable to differential settlement and cracking across zone transitions, but is not defended against progression of seepage erosion at elevations above the Zone II filter.
7. Zone II "Drain Fill" Serves as a Filter for the Zone I. Zone II was designed as a drain but serves as a filter. The SCS design intent for the Zone II "drain fill" which extends to elevation 2165 ft was to perform as a drain to control/collect seepage through the Zone I material and to establish a favorable (low) phreatic surface within the embankment. The Zone II fulfills this intended design function, however, the Zone I actually produces very little seepage (it is more impervious than the SCS probably expected) and thus the need for a drain is limited. On the other hand the Zone II meets filter criteria and fulfills a very important role in defending against internal erosion potential failure modes.
8. Zone II Does Not Extend All The Way To The Crest. Zone II extends to elevation 2165.0 ft which is 5 ft below emergency spillway crest and about 14 feet below the crest elevation. The fact that the Zone II does not extend to near the crest leaves the potential for flow through and erosion along potentially open cracks during very high water levels (above the spillway level). Is not likely to be a concern with seepage through cracks for water levels at or below the spillway crest level because it is reasonable to expect that the free water surface in the crack would intersect at or below the top of the Zone II materials and thus the Zone II would provide the required filter protection.
9. Dam Inspection Found A Bulge on Upstream Slope Face. The detailed site inspection conducted as part of the Structure Assessment found a low height bulge in upstream dam slope (lower right dam face, station 12+50 to 13+25) identified during field inspection (February 25, 2004). This may be due to bedrock high. We recommend this area be monitored and surveyed.
10. Potential for Transverse Cracking. There is a "generalized" potential for transverse cracking to develop at this site due to the primary factor of continued drying out of the embankment (leading to cracking due to drying shrinkage) aided and abetted by a crest of considerable length along with subtle foundation irregularities. However, to date there has been no evidence of or indication of transverse cracking on the FRS.

The potential for drying shrinkage appears to be lower at Sunnycove FRS, than at lower elevation sites in the valley where shrinkage cracking has occurred, apparently due to materials that are less susceptible to drying shrinkage and, by a greater amount of rainfall (thus less severe drying at this site). There have been several noted impoundments to date without evidence of cracking being observed (maximum depth of impoundment was 21.2 ft in August 1992). The FMEA team recommends that specific visual monitoring for/inspections of transverse cracks take place after significant future impoundments (depths should be established by the owner and included in the Standard Operation Procedures). Backhoe trenching to search more completely for cracking that may currently exist but be hidden may be an option as a geotechnical investigation.

11. Shift in Dam Alignment. Based on the geologic evaluation of the foundation exploration, the alignment of the FRS was shifted to the south to avoid founding a portion of the left abutment on a high terrace deposit (pervious zone) and to found the entire left abutment on fanglomerate. The success of this shift was verified by the foundation materials found in the foundation excavation.
12. Stability Analysis. Stability appears to meet design criteria but should be documented for record.
13. Dam Crest Settlement. No unusual settlement documented to date.
14. No Significant Seismic Impacts. The low height of the dam, firm foundation, and low seismicity in the area has a low impact/effect on the dam.

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

15. Antecedent Storm Used In Corps Routing. The Corps of Engineers Phase I investigation flood routing considered an antecedent storm in their PMF routing and assumed that the principal outlet was plugged. The corps assumed a full pool (100-year pool) starting water surface elevation of 2170 ft. The SCS apparently used the 10 day drawdown pool elevation (Elev. 2148 ft) to initiate PMF routing.
16. The Discharge Rating Curves For The Principal Outlet Need To Be Established/Verified For Different Assumptions On Inlet Capacities and Then Routings Performed For Several Conditions. Re-examine rating curves and document elevation vs. discharge.
17. Dambreak Analysis is Conservative. The following table provides the breach parameters used in the dambreak study. May not be able to develop the size of breach evaluated with the pool that is available.

Time To Failure [min]	24
Final breach width [ft]	140
Initial Water Surface Elevation [ft]	2170.0 {emergency spillway crest}
Final Breach Elevation [ft]	2131.5
Volume of Reservoir [af]	218.0
Breach Outflow [cfs]	17,000
PMF Outflow [cfs]	7,700

18. Intake To Principal Outlet Currently Controlled/Limited By A Small Orifice. The "at least" 10-day SCS drawdown design was established on the basis of using an 8-in by 8-in rectangular orifice with invert elevation at 2148 ft in the intake tower as the primary means to drawdown the reservoir pool below the spillway crest after a flood event. Under current operating procedures this provides the only discharge from the reservoir during the rising limb of the inflow and this discharge is not visible to downstream residents. However, the 10-day drawdown is only to elevation 2155 ft. This relatively small intake and associated discharge flow was apparently established "by design" both as a means to ensure that pool drawdown took at least 10 days (to elevation 2155 ft) and that outflow was minimal (i.e. - within the capacity of the drain pipe outlet). It is not clear whether the "at least 10 days for drawdown" requirement was established as a general upstream slope stability consideration or due to a general need to limit discharge from the FRS. However, it is now clear that a potential risk reduction alternative is to make the orifice larger, and/or add additional orifices, or use the slide gate opening early in the flood event to enhance the flood protection afforded by the dam (allowing it to be larger than the 100-year storm) and reduce the potential for discharge in the emergency spillway. It is also clear that providing a controlled, visible discharge from the FRS in advance of spillway operation could provide warning of an impending serious event and potentially reduce the risk of life loss. Further it is apparent that more rapid drawdown does not present a dam safety concern and allows risk reduction relative to "back to back" floods. The orifice only draws down to elevation 2154.8 which is 7 ft. higher than sediment pool. The lower gated outlet is not utilized but could be.
19. 30 inch Diameter Outlet Pipe Size Was Based on Standard Design. The design size of the principal spillway outlet pipe from the intake tower appears to be based on standard design instead of site specific design requirements which limited the outlet discharge. The greater size is fortunate with respect to the potential opportunities to provide risk reduction, as discussed below.
20. Use Greater Portion of the Capacity of the Outlet Works. Using a greater portion of the available capacity of the outlet works during the rising limb of the hydrograph would provide advantages in flood routing and advance warning without hindering the 10-day drawdown design from full pool after the storm has passed. This would require modification of the size or operation of the openings into the inlet tower. Currently, the lower gated outlet is not used at all and if it were to be opened it could not open beyond the ¼ stop. Under the scenario of using a greater portion of the outlet capacity, a non-damaging amount of water may be passed downstream and a significantly larger flood could be routed without damage and without violating the original design drawdown intent. Further use of a visible discharge flow could provide a great risk reduction impact by advance warning of potential flooding to downstream residents.
21. The FRS Can Safely Pass the IDF and the PMF. The FRS spillway can safely pass the Design Flood (1/2 PMF) and it can pass the PMF with 1 – 1.5 ft of freeboard.

22. Land Rights Adequate. It appears that the District holds adequate land rights up to the PMF pool. A plot of the maximum reservoir level under the PMF routing did not indicate any private residences or public roadways within the flood pool. (A few homes abutted the PMF flood pool.)
23. EAP Upstream Not Needed. Because there are no homes in the upstream pool limits of the PMF or 100-year pool. There is no need to prepare an upstream EAP at this time.
24. Dam Has Been Tested. The dam has had several recorded impoundment events. There have been 5 events with impoundment over 15 ft. in depth and the maximum impoundment event is recorded as 24 ft. in depth. The reference or zero gage point for these impoundment depth is believed to be the invert elevation of the slide gate at the intake structure (elevation 2133.0 ft). However, this should be verified and the actual elevation of the top of each past significant impoundment and the date of the impoundment should be verified. All future impoundment events should be converted from gage height and documented in terms of actual elevation of the top of the impoundment in reports on the flooding event.
25. Emergency Spillway Well Designed. The emergency spillway was well designed both in terms of capacity and structural adequacy. The spillway is located in sound fanglomerate. Deep erosion gullies that have formed in the outfall channel below the emergency spillway crest appear to be in loose-dumped spoil materials derived from the spillway excavation.
26. Emergency Spillway Operation Appears Straight-Forward. There are no significant factors that hinder discharge capability from the emergency spillway. There was no consideration of the use of the principal spillway in major flood routing. Under current operations this is reasonable as the 8 in by 8 in orifice could be blocked with debris. However modifications in size of the orifice and operation of the low level gated inlet to the intake structure could allow this restriction to be eliminated.
27. Short Impoundment Time. The duration of impoundment is short. Above the spillway crest the impoundment is but a few hours and below the spillway crest to the inlet of the orifice is designed to be less than 10-days (and could easily be designed/controlled to be less than that amount – say 5 days without concern to stability of the structure).
28. Greater Than 100-year Flood Protection. A review of the 1992 Area Drainage Master Study indicates that there may be greater than 100-year flood protection based on ADMS storage estimate. Further as noted above a revision in inlet capacity and/or operations could also provide greater protection with existing facilities.
29. No Potential Overtopping. There is no realistic dam failure mode postulated due to overtopping based on currently estimated hydrology.
30. No Failure Mode for Principal Spillway. No failure mode was noted by the FMEA team for principal spillway
31. Dam Crest is Level. The dam crest is generally level but may have a low spot which should be raised to make the crest at that location equal to the remainder of the crest. The crest should continue to be monitored to ensure that a level crest is maintained.
32. Flood Regulations Tied To Public Protection For A 100-Year Flood. Government "public safety" regulations are tied to a 100-year storm event. The only safety regulation for a storm greater than a 100-year flood event relates to the maximum

event that the structure must be able to pass without failure of the structure. There are specific considerations related to public safety for floods greater than the 100 year storm that are safely discharged from the structure.

33. Discharge in Emergency Spillway Occurs for Storm Events Greater Than 100-year Storm. The dam and reservoir were designed to control and detain the 100-year storm. The emergency spillway will not discharge unless there is a storm greater than the 100-year storm or from multiple storms, under normal operating conditions.
34. Multiple PMF Analyses. There are two PMP/PMF analyses. The most recent analysis was conducted by the Corps of Engineers in the Phase I Assessment using HMR 49 guidelines. The duration of the PMF analyzed in both studies was the 6-hour storm. The impact and scenario related to FRS response and performance related to longer duration storms and "back to back" storms should be evaluated, described and documented so that a clear understanding of potential effects and potential responses is provided.

Consequence Evaluation

35. Dambreak Analysis Was Not Realistic. Dambreak analysis used conservative values (time and width of breach appear reasonable, however, not for volume behind reservoir). The dambreak should be analyzed with reasonable assumptions. (See note under #17)
36. Well Designed Dam. The structure was well designed (used zoning and good spillway configuration). This enhanced design may have been related to the recognition of the close proximity of the downstream population at risk.
37. Warning Time Very Short. Minimal warning time to implement EAP. It is recommended that the warning time be increased and warning improved. Evaluate means to improve warning time. Local control of EAP implemented locally by Town of Wickenburg. The dam is monitored locally by the Town.
38. Consequences of Spillway Discharge. Spills from major flood events (e.g. - exceeding 500 year return period) could potentially result in life loss downstream without any dam failure. The likelihood is relatively high for spillway discharges on the order of a 500-year event and relatively low for discharges associated with a PMF event. Note however that the risk for potential loss of life existed prior to construction of the dam.
39. Consequence for Seepage-Erosion. A potential failure due to seepage erosion through cracks has a low likelihood of occurrence but would have high adverse consequences.
40. Downstream Inundation Mapping. Need to prepare downstream inundation mapping to get an improved understanding of consequences versus flood frequency relationship. This evaluation should include consideration of the Town of Wickenburg maintenance facility located just downstream of dam.
41. Downstream Discharge Configuration. A review of the topographic mapping downstream of the dam from the emergency spillway indicates that the topography in the discharge zone downstream from the dam initially widens out but then is constricted approximately 0.6 miles downstream. This constriction will maintain the depth of flow at a higher level for a longer time than if the discharge area was

uniformly widening below the FRS and this will affect the incremental consequences/damages downstream.

Action Items – Risk Reduction Measures or Investigations

42. Use Greater Capacity of Outlet Works. By using the capacity of the outlet works during the rising limb of the hydrograph advantages in routing and warning without hindering the 10-day drawdown design are provided. The intake inflow capacity of the principal outlet will provide multiple benefits. Increasing the size of the orifice is a relatively straight forward structural modification. Utilization of the slide gate is an operational change. The benefits of an increased intake capacity include:
    - Drains the impoundment quicker – makes volume available for subsequent storms
    - Less likelihood of discharge in emergency spillway
    - Lower hydraulic head loading acting on embankment
    - Route PMF at lower starting pool water surface elevation.
    - Helps prevent clogging

    Under the scenario of using a greater portion of the outlet capacity, a non-damaging amount of water may be passed downstream and a significantly larger flood could be routed without damage and without violating the original design drawdown intent. Further use of a visible discharge flow could provide a great risk reduction impact by advance warning of potential flooding to downstream residents.
43. Zero Gage Reference. Verify and document zero gage height at base of dam and relate to historic record of impoundment depths. The gauge invert is at elevation 2133.0 ft. Verify inspector observations of impoundment depths versus current gauge height.
44. Stockpile Of Sediment In Pool Area. At the time of the detailed site inspection and the FEMA workshop there was a stockpile of soil placed in the reservoir pool. This stockpile will eventually get washed down to the upstream toe of the dam and inhibit drawdown of the reservoir. Discussions with District Operations & Maintenance indicate the stockpile was illegally placed and dumped in the reservoir pool area.
45. Visually Monitor Cut Slopes of South Side of Spillway. The FMEA team recommends that the south cut slopes of the spillway be monitored during impoundment events due to the presence of high-level, pervious channel deposits that could provide seepage pathways.
46. Put Flap Gate On 6-Inch Asbestos Cement Pipe Outlet. The 6-inch AC drain pipe discharges into the downstream overflow manhole. Hydraulic grade lines above the 6-inch pipe invert may back into the pipe and up into the central drain of the dam.
47. Examine Derivation of Rating Curves. Verify and document the rating curves for the principal outlet and the emergency spillway, then compute and document the stage-discharge relations for current operating conditions for various, representative frequency floods (including the PMF and the ½ PMF). Postulate with the District a range of other operating conditions that would better utilize the capacity of the outlet and run the stage-discharge relations for those operating conditions to determine the change in effect.

48. Prepare New Dambreak Analysis. See items 17 and 35.
49. Verify Operational Opening Of Lower Gate At Intake. The as-built plans indicate that a stop was placed on the gate to limit capacity and allow gate to open only ¼ of full capability. Verify that that is the case and if what is involved in removing the stop to allow full use.
50. Probable Maximum Precipitation. Prepare PMP/PMF using 24-hr and 72-hour durations. Compare routings of these events to PMP 6-hr duration flood to verify that they are less critical (or determine that they are more critical).
51. Stage-Frequency Analysis. Conduct a stage-frequency (e.g., 200-year, 500-year, 1000-year storms) analysis with and without adjustments on discharge. Evaluate outflow and downstream inundation.
52. Improve Warning Times. Add more rain gages in watershed and adjacent region. Add stream gage. Use satellite technology or meteorological forecasting.
53. Evaluate Modifying Principal Outlet. Consider the following:
    - Sedimentation
    - Downstream Discharge
    - Various Operational Options and/or Orifice Opening Sizes
    - Improved/Advanced Warning times
54. Evaluate And Explore Difference Between SCS and Wickenburg ADMS Hydrology Study. Run both the 24-hour and/or 72-hour storm to illustrate PMF differences. (See #50)
55. Verify that District Land Rights Extend Up To PMF Pool. Also check easement of pipeline in relation to Wickenburg maintenance facility.
56. Video of Outfall Pipeline. A video inspection of the outfall pipeline has been conducted for a certain distance. Continue video from point where the inspection left off to the outfall at the Hassayampa River.
57. Investigate for the Possibility of "Hidden" Crest Cracking – Recent investigations at Spook Hill revealed that a considerably greater amount of cracking was observable in the crest of the dam when shallow test trenches were used to investigate. At Sunny Cove there has been no visual evidence of cracking on the surface and now that surface is covered with gravel mulch which would further restrict the possibility of observing any cracking. The possibility of using some shallow test trenches (12 in deep should be adequate for this purpose) to verify that cracking is not occurring at Sunnycove was suggested. The best place for the shallow test trenches at the dam crest would be between about stations 14+00 and 15+50, and between stations 11+50 and 12+50 which are locations of irregular foundation rock profile. The purpose of these trenches is to investigate the possibility of embankment cracking in general and across these transitions in particular. It is also suggested that station markers for identification of these subsurface areas be installed at the locations to aid in awareness and observation by inspectors and reviewers of the FRS performance.
58. Observations During Inspection Of Fanglomerate Ridge Adjacent To Principal Spillway. The as-built profile of the dam indicates a remnant ridge exists adjacent to the principal spillway conduit. Check records for observations/inspections of conduit placement and foundation preparation for the conduit.
59. Conduct And Document Post Flood Observations. Identify the impoundment elevations and the dates and include any observations on FRS performance.

60. Address Principal Outlet Operation Related To Failure Mode and Height Of Filter.

61. Bulge On Upstream Face of Embankment North of Right Abutment. Monitor and possibly run cross-sections across bulge.
62. Analyze Slope Stability. Update the stability analysis.
63. Investigate Patches. The detailed site inspection observed clear patches in the gravel mulch on the upstream slope face of Sunnycove FRS. The patches are located just below dam crest and are spaced at a somewhat regular interval. Investigation may include test pit with shovel. It appears that gravel mulch has been displaced or pushed aside purposefully. The patches range in diameter from 1 foot to 2 feet.

General Findings

64. One Category I Failure Mode. Only one Category I failure mode due to adverse consequences from normal operation of emergency spillway and only two Category II failure modes.
65. Relationship Between the Intake Orifice and the 6-inch Asbestos Cement Drain Pipes. The low level intake orifice (8-in by 8-in orifice at elevation 2148.0 ft) was designed to limit discharge of the impoundment pool such that the hydraulic grade line in the downstream overflow manhole was restricted in elevation to lower than the 6-in AC drain pipe outlet. The 6-in AC drain pipe outlets into the downstream manhole as shown on as-built plan sheet 4R. This provision does not seem necessary as the small, transient inflow of water into the drain material should not be detrimental; however use of a flap valve to prevent the inflow should take care of that possibility anyway.
66. Gravel Mulch. The District has placed gravel mulch on the upstream and downstream slope of the dam embankment. With dams that have the potential for transverse cracking would it still be prudent for the placement of mulch as cracks tend to express surficially on embankments.
    The District's Application of Gravel Mulching Should Be Evaluated on the basis of Site Specific Conditions? The District has applied gravel mulch to the face of some dams. The question was raised as to whether gravel mulch should be applied if the potential for transverse cracking at a dam exists. Gravel mulch application would tend to hide transverse cracks. Note that there have not been any reported transverse cracks at Sunset FRS. An evaluation matrix could be developed to address this question on a site by site basis.
67. Right-Of-Way For Outfall/Outlet Pipeline – Sunnycove FRS and Sunset FRS utilize a common outfall/storm drain to discharge flows from their respective principal spillways. The FMEA team recommended that the right-of-way for the pipeline from the dams to the Hassayampa River needs to be verified and that any structures placed over or adjacent to the pipeline will not inhibit access or maintenance. The entire length of the pipeline should be inspected using remote video camera.
68. Minimal Public Awareness. There is very little public awareness of the presence of the dam and the potential hazard associated with impoundment of floodwaters. This is evident by the continuing residential encroachment downstream of the dam. Public input may be needed to change operations of dam (to change principal outlet). The

District has an upcoming project called the Wickenburg Area Drainage Master Plan that may be a good mechanism to include public awareness regarding the dam.

69. Construction Quality Control. The FMEA team was of the opinion that it was likely that the construction quality control was reasonably good.
70. Design Documentation and Construction Photography Minimal. The FMEA team could not locate final design reports/manuals nor photographs of the construction of the dam.
71. No Need to Do Dynamic Routing. Short dam with small impoundment. Level pool routing is sufficient.

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 – Failure modes of greatest significance
- Category II – Failure modes of lesser significance (but not inconsequential).
- Category III – Failure modes for which insufficient information is presently available to make a judgment on the significance of the failure mode. The development of additional data and information is warranted. Additional records research may be justified.
- Category IV – Failure modes which are not physically possible or which are clearly not credible.

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 - FAILURE MODES OF GREATEST SIGNIFICANCE

1. Adverse Consequences Resulting from Emergency Spillway Discharges During Major Rainfall Events (Category I).

Failure Mode Description: The Sunnycove FRS emergency spillway is a 100 foot wide unlined earth and rock cut channel, located off the left abutment. The spillway channel is separated from the dam embankment by a ridge in the native rock and soil materials. The emergency spillway was constructed into sound fanglomerate as indicated in the as-built plan. Normal flood discharges from the spillway are directed into a natural wash. The flows continue in the wash and are directed toward residential development within the Town of Wickenburg. This potential "failure mode" does not "fail" the dam or emergency spillway but could result in severe adverse consequences for major flooding events. This potential failure mode was rated as a Category I failure mode because normal "successful" operation of the emergency spillway can produce discharges that could have significant adverse consequences and the likelihood of occurrence of these adverse consequences is associated with floods of reasonably probable frequency. The

floodwaters will pass through the emergency spillway. From that point the water will flow into a large housing development downstream away from the dam.

Adverse Factors:

(1) Emergency spillway flows are directed into residential homes approximately ½ mile downstream from the dam. New residential construction in the floodway is presently underway. There is a significant number of people at risk.
(2) The design discharges from the emergency spillway range from 6300 cfs to 7000 cfs.
(3) A channel constriction exists approximately 0.6 mile from dam. Note: this "constriction" does not appear to be narrower that the spillway discharge channel but does appear capable of maintaining the flow at greater depths than what would be anticipated if the topography continued to level/widen out below the FRS.
(4) No dedicated downstream easements for spillway operation.
(5) No regulation (development, density, land use) on downstream spillway discharge areas.
(6) 72-hour or 24-hour PMF may be more critical.
(7) Land use is changing in upstream watershed. Less urban development than compared to Sunset FRS.
(8) No advance flow discharge from dam to warn downstream residents.
(9) Very little warning time for discharges from emergency spillway.
(10) Very small watershed and small reservoir. Quick rainfall/runoff response and quick fill time of reservoir (approximately less than one hour of fill time).
(11) Floodwaters are not released until pool is at emergency spillway crest.
(12) Impoundment history indicates pool elevation has reached to within 5 feet of spillway crest.
(13) Potentially damaging spillway discharges will begin to occur at an estimated flood frequency of greater than the 100-yr event but less than the 500-yr event. This is a relatively frequent recurrence interval in comparison with the typically conservative design criteria that are standard for high hazard dams.

Positive Factors:

(1) It will take greater than the 100-year event for spillway to discharge (less if principal spillway/outlet is plugged or multiple storms) and somewhat greater floods to result in significant adverse consequences (perhaps the 200 to 500 year frequency event). However, even these "infrequent" events do in fact have a relatively frequent recurrence interval in comparison with the typically conservative design criteria that are standard for high hazard dams. (see #6 below and 13 above)
(2) Dam is instrumented as part of the District Alert System. This would provide warning of impending inflows and discharges.
(3) Spillway discharge exit is normal operation and straight forward. No issues related to spillway debris/capacity/blockage.
(4) Short spillway discharge duration for major floods up to and including PMF.

(5) Spillway founded in fanglomerate – good erosion control.
(6) Likelihood of spillway flows that could cause potential loss of life may be somewhat remote (e.g.- on the order of 500 year return period – to be determined based on stage- discharge - inundation routings recommended by FMEA team).
(7) Have downstream Emergency Action Plan. EAP implemented locally by Town of Wickenburg.
(8) Storm itself would provide warning of heavy rainfall.
(9) Minimal headcut erosion resulting from discharges. Due to the competent fanglomerate materials that form the spillway bottom and sidewalls, and short duration of spillway discharges. Severe erosion in the spoil fill that was placed in the ravine below the spillway crest is not representative of the expected erosional resistance of the spillway crest.

Potential Actions for Risk Reduction:
(1) Increase/improve orifice outlet capacity/capability.
(2) Provide visible discharge flows downstream during rising limb of flood
(3) Increase basin storage to reduce the frequency and magnitude of potentially damaging events

CATEGORY II – FAILURE MODES OF LESSER (BUT NOT INCONSEQUENTIAL) SIGNIFICANCE

2. Potential for Cracking due to Foundation Irregularities – Seepage Erosion Through Crack. (Category II)

As-built plans indicate potential foundation bedrock surface irregularities along the axis of dam outside of the cutoff trench. In particular, a bedrock ridge is evident crossing the dam alignment on the lower left valley section. The ridge extends upstream to downstream (east to west) across the dam axis, and is skewed to the NW-SE oriented axis of the dam. The ridge crosses the centerline of the dam at about Station 14+30 near the outlet works alignment. The as-built drawings indicate that the ridge was substantially removed and shaped. However, a trapezoidal trench was cut through the remnant, shaped ridge to install the outlet pipe. The most notable foundation irregularity is left of principal outlet conduit where as-built plans show a high point in foundation. Based on the profile on dam centerline (Sheet No. 3 in the as-built drawing set), the trench cut slope was approximately 2H:1V and about 6.5 feet high at dam centerline. Another subtle bedrock ridge is evident at the base of right abutment near Station 12+00. Transverse cracks could potentially form over these surface irregularities. The mechanism for potential transverse crack formation is tension cracking over the irregularity due to differential settlement of the embankment locally.

Failure Mode Description: The potential failure mode development sequence is: Transverse crack forms prior to or at onset of impoundment, impoundment occurs, seepage flows through crack, seepage erosion occurs along a crack, crack widens flow/erosion rate increase and breach forms.

Adverse Factors:

(1) Zone II Filter does not extend up to spillway crest elevation.
(2) Because the uninterrupted length of the FRS is relatively long (e.g. much longer than Sunset FRS) the crest has the propensity to crack somewhere along longitudinal axis to relive the drying shrinkage stress.
(3) Because this is a dry area cracking of the FRS is possible.

Positive Factors:

(1) Actual irregularities are really not as irregular as what appears on the drawings due to exaggerated scale (vertical to horizontal) used on the as-built. Recommend plotting profile at a normal scale. Worst case irregularity observed on the centerline profile was 2H:1V trench cut slope north of the outlet pipe, with a height of only 6 feet.
(2) Protected at 5 feet below spillway elevation by filter.
(3) The amount of time that the water would be available to fill/seep/through/erode/breach the cracks above the spillway crest is minimal (only a few hours) – this amount of time is considered highly unlikely to be adequate to produce a failure.
(4) Area is not as dry as valley where District has experienced extensive cracking of other Flood Retarding Structures.
(5) The foundation irregularities occur at the base of the abutment slopes, within the valley section, and are not in the steep part of abutments where differential stresses and arching could be more severe.
(6) Foundation irregularities are deep in section. Discontinuity must span all the way across dam section.
(7) No compressible zones. Dam founded on fanglomerate.
(8) Cracking induced by long term impoundments. Pool drains in less than 10-days.
(9) Shrinkage cracking potential is mitigated due to higher annual moisture. This dam is at higher elevation than other dams that have exhibited severe drying shrinkage cracks

CATEGORY III – FAILURE MODES FOR WHICH INSUFFICIENT INFORMATION IS PRESENTLY AVAILABLE FOR MAKING ENGINEERING JUDGEMENTS

No Category III potential failure modes were identified by the FMEA team for Sunnycove FRS at this time.

CATEGORY IV – FAILURE MODES WHICH ARE NOT PHYSICALLY POSSIBLE OR WHICH ARE NOT CLEARLY CREDIBLE

Other Considerations: These candidate potential failure modes (or other issues as noted) 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) **Spillway Erosion Leading to Reservoir Loss**: The spillway is founded in fanglomerate. The potential headcut path is very long to the reservoir pool. The time of duration of spillway flow is short.
(2) **Spoil Fill Placed Below The Spillway Crest**. Fill placed below the spillway crest shows spectacular erosion gullies but this erosion is not a concern to discharge capability. (It is a concern relative to the turbidity of the discharge). Thus this mode was considered too remote to be credible and was not carried forward.
(3) **Pipeline Failure Might Result in Discharge from Principal Outlet**: The amount of flow appears not to be a problem warranting consideration.
(4) **Slope Instability.** The factor of safety of 1.25 reported for the downstream slope was based on an infinite slope stability analysis. This analysis used the lowest shear strength from all tests to establish the friction angle of the material (φ 32 degrees). No evidence of problem on downstream slope. Upstream slope stability is adequate.
(5) **Piping Along Embankment/Foundation Contact.** Based on the geologic information and the construction information available no layer capable of being piped exists in the foundation.
(6) **No Compressible Soils.**
(7) **No Fissuring.**
(8) **No Unusual Settlement.**
(9) **No Identified Cracking.**
(10) **No Subsidence.**
(11) **Piping Through Embankment.** No potential piping through the FRS is considered a credible failure mode. Because no significant phreatic surface can develop and the filter drain meets filter criteria.
(12) **Left abutment terrace deposit considered in design and construction.**
(13) **No Failure Mode Identified for Principal Spillway.**
(14) **No Failure Mode Identified for 6-inch AC drain pipe.**

4.0 LIKELIHOOD AND CONSEQUENCE CATEGORIES

The likelihood of occurrence of each identified failure mode has been assigned to one of three categories according to the FMEA team professional judgment. This adopts a subjective, degree-of-belief approach to the expression of uncertainty, as opposed to relative-frequency statistics of observed occurrences. These likelihood judgments express degrees of uncertainty but are not quantified in the probability matrix. They recognize simply that the occurrence of some failure modes is believed to be more likely than others for this particular dam. This relative measure of likelihood is contained in the categories defined in Table 1.

Table 1. Likelihood Categories.

Category	Description
High	Highest likelihood of occurrence for Sunnycove FRS
Medium	Intermediate likelihood of occurrence for Sunnycove FRS
Low	Lowest likelihood of occurrence for Sunnycove FRS

In assigning likelihoods during the FMEA workshop, failure modes representative of the most likely and the least likely categories were evaluated.

Consequence categories follow along similar lines as likelihood categories in reflecting the relative severity of failure effects specific to the dam. The actual magnitude of the downstream consequences depends on such factors as economic losses, population at risk, and the effectiveness of the warning and evacuation. These were not evaluated directly 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 Sunnycove FRS
Medium	Intermediate inundation effects for Sunnycove FRS
Low	Lowest inundation effects for Sunnycove FRS

5.0 FAILURE MODE AND EFFECTS TABLE

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

- Failure Mode – identifies the primary failure mechanism
- Initiating Condition – condition(s) giving rise to initiation of the failure mode/sequence
- Effects – distinguishes dam breach and spillway discharge failure types
- Likelihood – likelihood category form Table 1
- Consequences – consequence category from Table 2
- Information Needs – summary of important additional information that could support or modify the failure mode assessment provided
- Existing Risk Reduction Factors – conditions or measures in place that have acted to reduce likelihood and/or consequences assigned
- Potential Risk Reduction Measures – action, studies, or features that might reduce the assigned likelihood and/or consequences
- Comments – supplemental remarks

Table 3. Summary of Failure Mode and Effects Analysis
Sunnycove Flood Retarding Structure
Maricopa County, Arizona

FAILURE MODE	INITIATING CONDITION	EFFECT	LIKELIHOOD	CONSEQUENCES	INFORMATION NEEDS	EXISTING RISK REDUCTION FACTORS	POTENTIAL RISK REDUCTION FACTORS	COMMENTS
1. Adverse Consequences for Normal Operations of Emergency Spillway	Reservoir inflow greater than 100-year flood	Discharges in emergency spillway and downstream inundation	Low (PMF Event) High (500 yr Event)	High (PMF Event) Medium (500 yr Event)	New flood routing Multiple storms 24-hr/72-hr PMP Stage Discharge-Inundation for 200-500- 1000 year floods with current / and new (to be proposed) operations	Dam is instrumented as part of District ALERT system Have Downstream EAP Discharge duration of PMF is short	Increase capacity of principal outlet	
2. Cracking Due to Foundation Irregularities – followed by Seepage Erosion through crack during an extreme flood event	Transverse crack and long duration impoundment at levels above the spillway crest	Downstream Inundation	Low	High		Have Downstream EAP	Increase capacity of principal outlet Inspect for cracking at identified stations on upstream and downstream slope	

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 Sunnycove FRS

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

Consequences

Likelihood	High	Medium	Low
High		1. 500-year event	
Medium			
Low	1. PMF Event 2		

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 only a portion of failure mode 1 is directly within the shaded region. Failure Mode 1 (adverse consequences related to normal spillway discharge) is depicted as ranging from low likelihood, high consequences (for the PMF event) to high likelihood, medium consequences (for the 500-year event). The range for failure mode 1 spans the medium likelihood, medium consequence of the shaded risk region. The determination of failure mode 1 falling within this block of the shaded region is dependent on the storm frequency, magnitude, and downstream consequences.

7.0 SUMMARY AND CONCLUSIONS

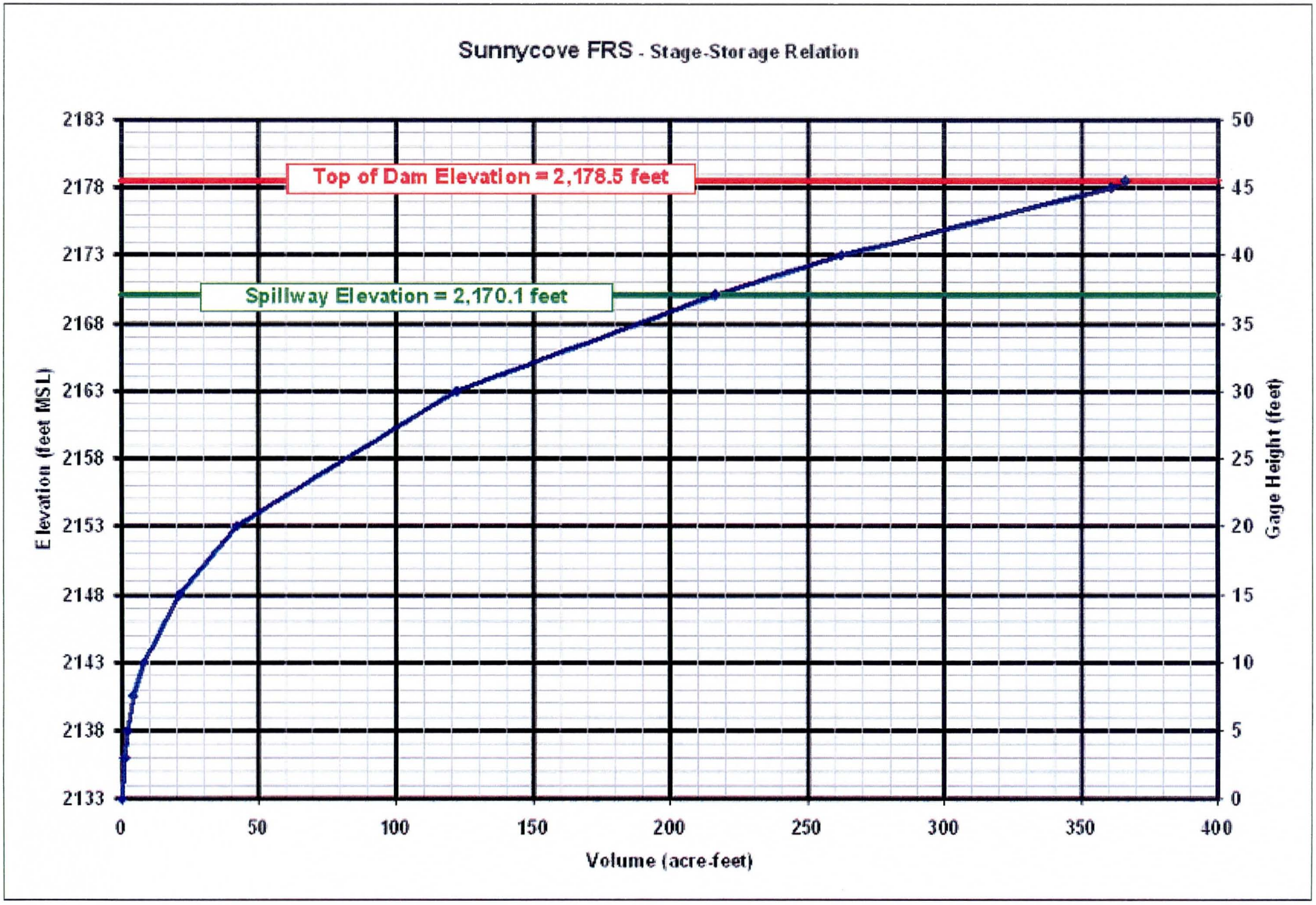
Sunnycove FRS was constructed pursuant to a relatively modern dam design. Construction appears to have been without any particular issues. The dam has performed normally and satisfactorily for 28 years. The structure is satisfactorily maintained and monitored.

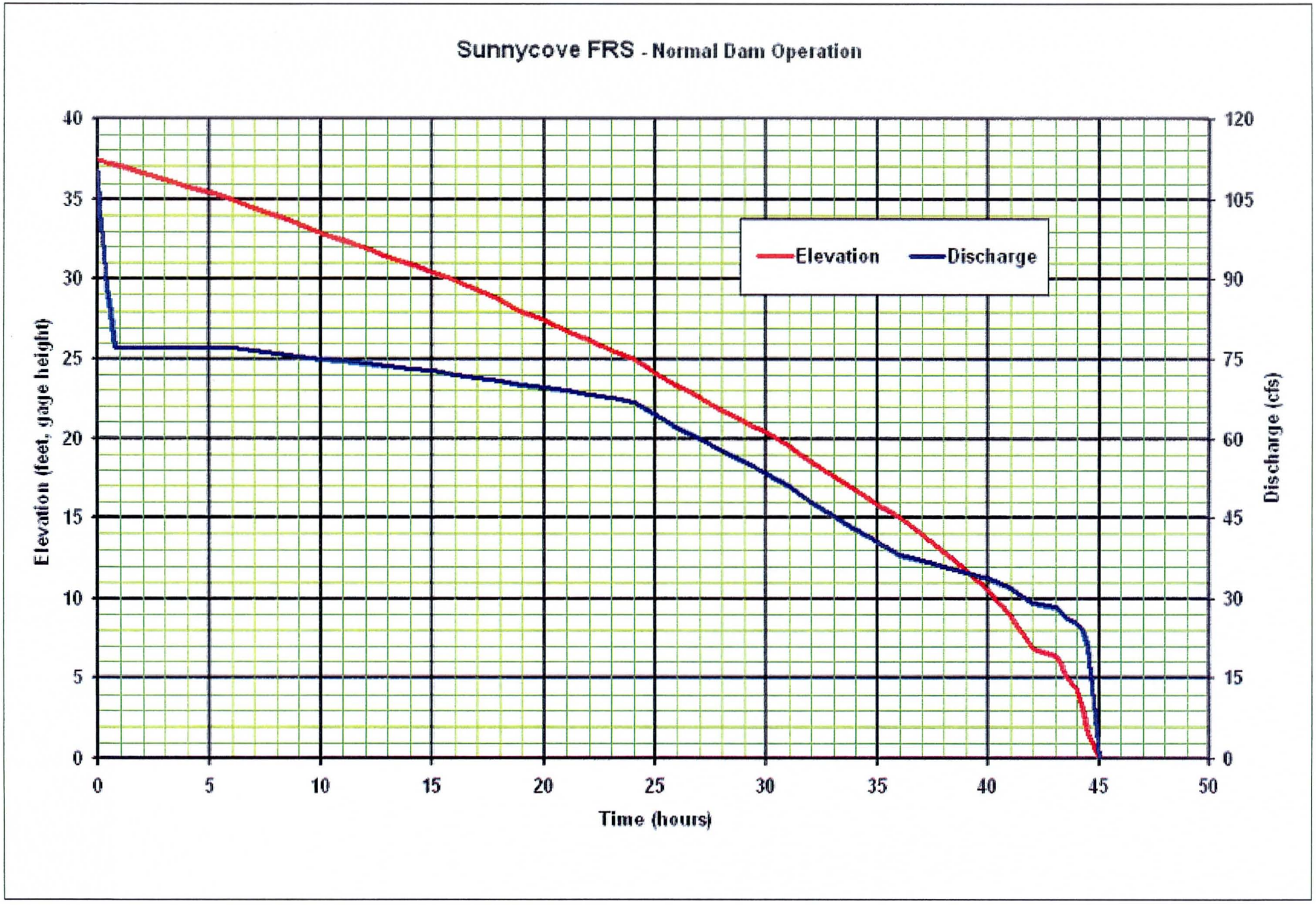
However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that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

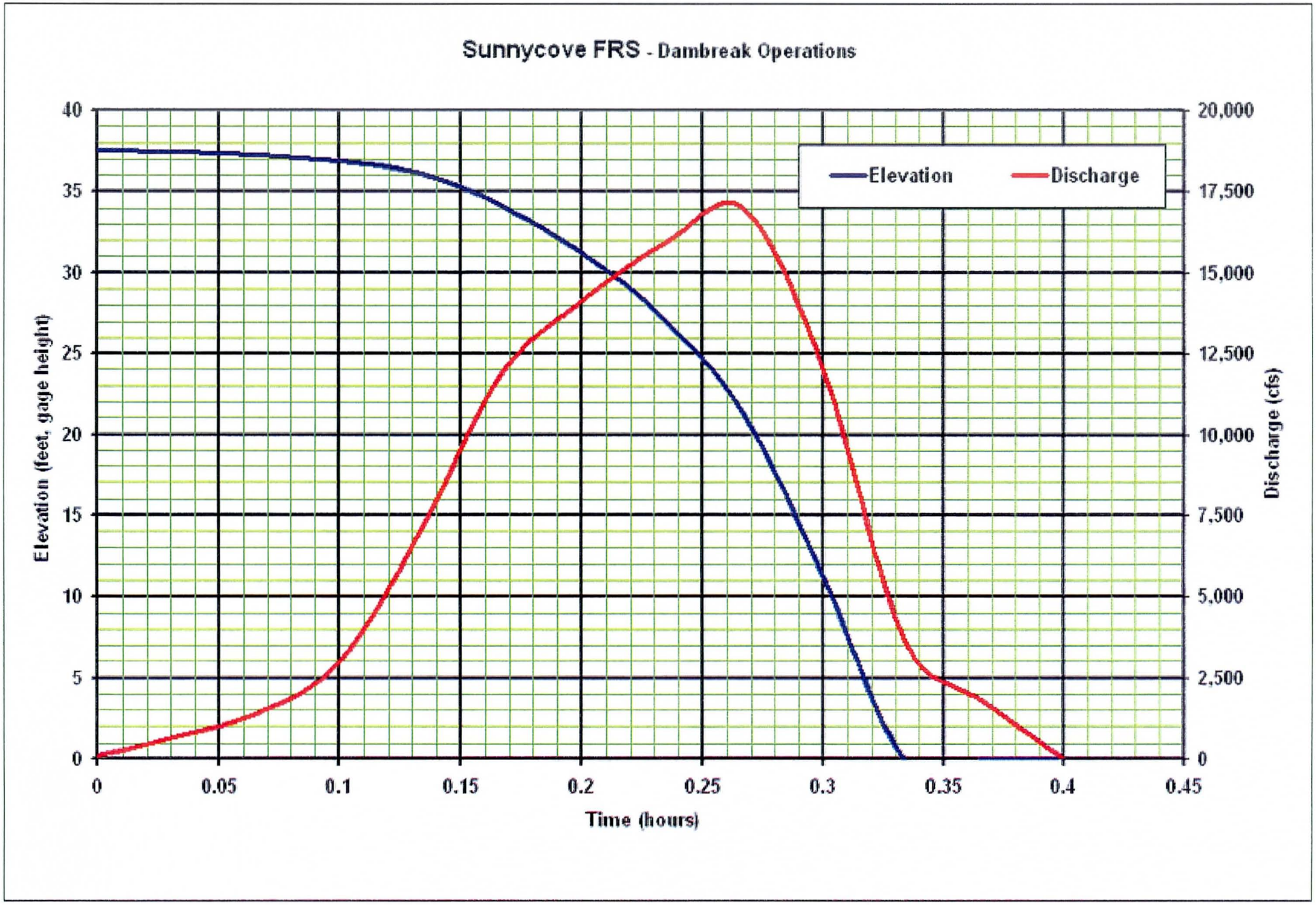
One Category I potential failure modes was identified by the FMEA team. The Category I failure mode is related to adverse consequences from normal operation of the emergency spillway during major rainfall events. The length of time the spillway flows is of short duration (a few hours). However, there are residential structures immediately downstream of the emergency spillway and there is no defined downstream channel. There are a considerable number of people and structures at risk in the flow path in the event of a spillway discharge.

A number of potential risk reduction actions were identified by the team related to monitoring, information collection and documentation and modification of operations. These are all identified in the section on Major Findings and Understandings.

SUNNYCOVE FRS APPENDIX - FMEA REFERENCE MATERIALS


Sunnycove FRS - Stage-Storage Relation
Top of Dam Elevation = 2,178.5 feet
Spillway Elevation = 2,170.1 feet
Elevation (feet MSL)
2183
2178
2173
2168
2163
2158
2153
2148
2143
2138
2133
Gage Height (feet)
50
45
40
35
30
25
20
15
10
5
0
0
50
100
150
200
250
300
350
400
Volume (acre-feet)


Sunnycove FRS - Normal Dam Operation
Elevation
Discharge
Elevation (feet, gage height)
Discharge (cfs)
Time (hours)
0
5
10
15
20
25
30
35
40
45
50
0
15
30
45
60
75
90
105
120


Sunnycove FRS - Dambreak Operations
Elevation
Discharge
Elevation (feet, gage height)
Discharge (cfs)
Time (hours)
40
35
30
25
20
15
10
5
0
20,000
17,500
15,000
12,500
10,000
7,500
5,000
2,500
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
0.45

SUNNYCOVE FRS

FCD GAGES 5247 AND 5248

STATION DESCRIPTION

LOCATION - The gage site is a dam located in the town of Wickenburg, approximately 1 mile southwest of the downtown area. From US 60, turn south of Kellis Road to the cemetery. The dam is located behind the cemetery. Latitude 33 57 25; Longitude 112 44 24. Located in the SE1/4 SE1/4 of S11 T7N R5W of the Wickenburg 7.5-minute USGS quad map.

ESTABLISHMENT - The gage was installed on July 1, 1986.

DRAINAGE AREA - 1.35 mi^2.

GAGE - The gage is a non submersible pressure transducer. The PT orifice line is at 0.60 feet gage height, or 2,133.60 feet M.S.L., levels of January 8, 2002.

There are ten staff gages at this location.

A staff plate is located near the orifice line and is viewable from the top of the dam. It reads directly in gage height, levels of January 8, 2002.

There are nine black and white staff gages located on the upstream face of the dam. The staff gages are five foot in height and are divided into one foot increments. The range is from zero to 45 feet. The staff gages read in gage height, levels of January 8, 2002.

HISTORY - PT elevation of 2,134.50 feet MSL on January 9, 1990. Survey by RWC placed PT at 2,134.02 feet MSL, or 6.28 feet gage height, where 0.00 feet gage height was the invert of the lower orifice in the intake tower. PT lowered to -0.10 feet gage height (2,127.82) on July 13, 1992. PT moved to 8.50 feet gage height on August 17, 1993. PT moved to 0.75 feet gage height on August 27, 1993. PT moved to 1.50 feet gage height on January 19, 1994. PT moved to 0.00 feet gage height on February 16, 1996. PT moved out of CMP on July 24, 1997 to escape drawdown when gates are opened. New PT elevation of 0.90 feet gage height. A non-submersible PT was installed on February 25, 1999 at 0.90 feet gage height. The SCS staff gage was replaced with nine 5-foot staff gages reading in gage height during spring 2000. Found the orifice at 0.60 feet gage height during survey of January 8, 2002. Made effective for Water Year 2002.

REFERENCE MARKS -

Zero gage height is 2,133.00 feet MSL.

RM1 - Reference 431 from the Wickenburg ADMS It is a chiseled '+' on top of east concrete curb at the south end of pavement on Kellis Road. Elevation = 2,160.71 feet MSL, gage height 27.71 feet.

RP1 - Nail in center of white '+' used for aerial survey of Wickenburg ADMS located near gate at southeast end of dam. Elevation = 2,178.33 feet MSL, gage height 45.33 feet.

Source

Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

RP2 - Top of yellow rod near fence post made of tie near RP1. Elevation = 2,179.30 feet MSL, gage height 46.30 feet.

RP3 - White paint on northwest corner of concrete for gate control. Elevation = 2,180. 44 feet MSL, gage height 47.44 feet.

CHANNEL AND CONTROL - The gage is located behind the dam. The control for flows through the principal outlet is a 30-inch diameter culvert, assuming the gate is open. For impoundments greater than 37.1 feet gage height, weir flow will begin through the auxiliary spillway.

PRIMARY / AUXILIARY OUTLET

The primary outlet is a 30 inch diameter culvert pipe of length 290 feet. The culvert invert elevation at the inlet is 2,126.50 feet MSL or -6.50 feet gage height. The culvert invert elevation at the outlet is 2,124.00 feet MSL or -9.00 feet gage height. There are two intake orifices in the intake tower. The lower orifice is at elevation 2,128.00 feet MSL, or -5.00 feet gage height. The upper orifice is at elevation 2,148.00 feet MSL, or 15.00 feet gage height. Flow begins in the intake tower at elevation 2,169.50 feet MSL or 36.50 feet gage height.

The auxiliary spillway crest is at elevation 2,170.09 feet MSL, or 37.10 feet gage height. The spillway is 100 feet wide and side walls sloping at 2:1.

Top of dam elevation is at 2,178.50 feet MSL, or 45.50 feet gage height.

Source

Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

SITE DATA

LOCATION	LOCATED IN WICKENBURG SOUTH OF US 60 ON KELLIS ROAD BEHIND THE CEMETERY
DRAINAGE AREA	1.35 MI^2
JURISDICTION	WICKENBURG, ARIZONA
WATERSHED	LOWER HASSAYAMPA
SECTION/TOWNSHIP/RANGE	SE1/4 SE1/4 S11 T7N R5W
LATITUDE	N 33 57 25
LONGITUDE	W 112 44 24
USGS QUAD MAP	WICKENBURG 7.5-MINUTE
INSTALLATION DATE	JULY 1, 1986 (WY 1986)
LENGTH OF RECORD (AS OF 10/01/03)	17.25 YEARS
STAGE GAGE TYPE	PRESSURE TRANSDUCER
STAFF GAGE	TEN
CREST STAGE GAGE	NONE
ZERO GAGE HEIGHT ELEVATION	2,133.00 FEET M.S.L.
STAGE GAGE ELEVATION	0.60 FEET GAGE HEIGHT
POINT OF ZERO FLOW	-5.00 FEET GAGE HEIGHT
SPILLWAY CREST ELEVATION	37.1 FEET GAGE HEIGHT
TOP OF DAM ELEVATION	45.0 FEET GAGE HEIGHT

Source
Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

GAGE ID HISTORY

ID	Elev of Instr. in GH	Elev of Instr. in MSL	Period
5248	0.60	2,133.60	10/1/01 - present
5248	0.90	2,133.90	10/1/97 - 10/1/01
5248	0.00	2,133.00	2/16/96 - 10/1/97
5248	1.50	2,129.50	8/10/92 - 2/16/96
3638	1.50	2,129.50	10/1/87 - 8/10/92
80	UNKNOWN	UNKNOWN	7/1/86 - 10/1/87

STAFF GAGE INFORMATION

STAFF GAGE RANGE	STAFF GAGE INFORMATION
0 - 45	NINE 5-FOOT STAFF GAGES ON FACE OF DAM. READS IN GAGE HEIGHT
NOTES:	(1) BOTTOM OF NUMBER (i.e. 54) IS 54 FEET. (2) SUBTRACT (12)49.43 FEET FROM STAFF GAGE TO GET GAGE HEIGHT LEVELS

Source
Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

RATING INFORMATION

RATING TABLE			
DISCHARGE RATING NUMBER 3, APPLIED AS OF OCTOBER 7, 1997			
CAPACITY RATING NUMBER 3, APPLIED AS OF OCTOBER 7, 1997			
GAGE HEIGHT (FEET)	ELEVATION (FEET NGVD 29)	DISCHARGE (CFS)	VOLUME (ACRE-FEET)
0.0	2,133.00	0	0
1.0	2,134.00	19	0
5.0	2,138.00	26	2
10.0	2,143.00	33	8
15.0	2,148.00	38	21
20.0	2,153.00	52	42
25.0	2,158.00	67	75
30.0	2,163.00	73	122
35.0	2,168.00	77	183
37.1	2,170.10	77	216
40.0	2,173.00	1,438	262
45.0	2,178.00	6,300	361

RATING -

The current discharge rating is rating #3 which like rating #2, is a combination of culvert flows through the principal outlet and weir flow through the auxiliary spillway. Flows through the culvert were evaluated using the FHWA HY8 program. Flows through the auxiliary spillway were evaluated using the weir equation and weir coefficient of 2.90. The two ratings were combined into a single discharge rating. Rating #2 was revised to get rating #3 by adjusting elevations. Rating #1 was developed using similar methods.

Source
Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

The current capacity rating is rating #3 which is a revision of rating #2, adjusted for movement of the gage. Rating #2 was developed from DTM from the Wickenburg ADMS. Rating #1 was taken from the HEC-1 output from the ADMS in 1992.

POINT OF ZERO FLOW - The PZF is at -5.00 feet gage height, or 2,128.00 feet MSL. This represents the lower orifice elevation of the intake tower.

DISCHARGE MEASUREMENTS - Outlet culvert is buried until it reaches the Hassayampa River. Discharge measurements are not possible. If desired, low flows through the auxiliary spillway could be measured.

Source
Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

Reservoir Operation Records

WATER YEAR PEAKS

WATER YEAR	EVENT DATE	PEAK WATER LEVEL (FEET GH)	PEAK VOLUME (ACRE-FEET)
2004			
2003	2/25/03	5.80	3
2002	10/7/01	7.02	4
2001	10/27/00	15.16	22
2000	8/29/00	10.63	11
1999	8/31/99	8.47	10
1998	8/13/98	2.95	0
1997	9/26/97	17.19	30
1996	9/11/96	7.13	4
1995	8/15/95	15.17	6
1994	10/6/93	11.56	11
1993	2/10/93	18.68	36
1992	8/22/92	21.68	53
1991	3/27/91	3.60	1
1990	7/6/90	10.10	8
1989	10/4/88	10.00	8
1988	9/18/88	15.80	24
1987	Data Unavailable	Data Unavailable	Data Unavailable
1986	Data Unavailable	Data Unavailable	Data Unavailable

Source
Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

Reservoir Operation Records

IMPOUNDMENT HISTORY

DATE OF PEAK	TIME OF PEAK	EVENT PERIOD	DURATION (HOURS)	PEAK STAGE (FEET, G.H.)	PEAK VOLUME (ACRE-FEET)	WATER YEAR
11/12/03	18:02	11/12 17:52 - 11/23 07:36	254	1.21	<1	2004
8/24/03	19:52	08/24 19:52 (single report)	0.0	1.14	<1	2003
8/15/03	03:42	08/14 22:07 - 08/18 13:32	87.4	2.83	1	2003
7/28/03	02:09	07/28 01:54 - 08/08 00:07	262	3.17	1	2003
3/21/03	17:51	03/16 14:01 - 04/12 02:41	637	3.91	1	2003
2/25/03	22:02	02/25 17:07 - 02/26 17:11	24.1	5.80	3	2003
2/14/03	11:37	02/13 17:22 - 02/18 16:12	119	3.51	1	2003
1/8/03	20:46	01/08 20:46 - 01/08 23:46	3.0	1.01	<1	2003
10/28/02	09:52	10/26 13:27 - 11/06 09:11	260	5.06	2	2003
9/9/02	08:54	09/06 19:04 - 09/18 07:47	277	6.75	4	2002
7/15/02	20:40	07/15 20:40 (single report)	0.0	1.21	<1	2002
7/9/02	23:18	07/09 23:18 (single report)	0.0	1.21	<1	2002
10/7/01	23:52	10/07 22:37 - 10/10 00:56	50.3	7.02	5	2002
7/6/01	07:57	07/06 05:27 - 07/10 05:18	95.6	8.00	6	2001
3/8/01	00:54	03/07 05:04 - 04/02 12:57	632	2.99	1	2001
11/14/00	09:50	11/14 09:25 - 11/14 21:50	12.4	1.64	<1	2001

Source

Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

10/27/00	14:11	10/27 06:15 - 11/14 08:55	435	15.16	22	2001
10/25/00	19:55	10/25 19:55 - 10/27 03:55	32.0	6.71	4	2001
10/22/00	09:50	10/21 20:30 - 10/25 09:50	85.3	6.51	4	2001
8/29/00	15:11	10/29 09:51 - 09/03 02:36	113	10.63	11	2000
8/27/00	16:16	08/27 07:06 - 08/29 08:11	49.1	8.40	6	2000
8/31/99	18:26	08/31 16:56 - 09/04 10:57	90.0	8.47	6	1999
7/15/99		07/17 22:54 (end)		10.97	10	1999
10/30/98	19:45	10/30 19:13 - 10/31 00:55	5.7	4.90	2	1999
8/13/98	18:46	08/12 19:40 - 08/13 23:30	27.8	2.95	<1	1998
9/26/97	04:44	09/25 19:39 - undetermined		17.19	30	1997
9/5/97	21:42	09/05 21:22 - 09/14 09:21	204	1.66	<1	1997
9/11/96	03:00	09/10 23:30 - 09/22 16:44	281	7.13	4	1996
8/15/95	00:53	08/14 22:46 - 08/16 10:11	35.4	15.17	6	1995
7/14/95	20:06	07/14 17:56 - 07/31 23:21	413	13.12	5	1995
3/12/95	13:43	03/12 13:10 - 03/13 00:33	11.4	6.29	3	1995
2/14/95	23:56	02/14 20:14 - 02/18 19:00	94.8	14.20	6	1995
1/26/95	05:11	01/25 18:27 - 02/06 19:31	289	7.07	1	1995
1/5/95	03:45	01/04 23:06 - 01/12 17:12	186	5.02	<1	1995
12/25/94	22:39	12/25 15:27 - 01/01 06:38	159	5.41	<1	1995
9/20/94	20:58	09/20 20:39 - 09/22 22:14	49.6	4.97	<1	1994
10/6/93	13:20	10/06 11:45 - 10/17 22:33	275	11.56	<1	1994
2/9/93	04:24	02/08 09:21 - 02/12 15:38	102	18.68	36	1993

Source
Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

1/19/93	05:19	01/19 05:19 (single report)		10.28	9	1993
1/10/93	20:26	No Information		7.88	5	1993
1/8/93	08:40	No Information		7.08	4	1993
8/22/92	23:23	08/22 21:15 - 09/01 14:06	233	21.68	53	1992
8/12/92	13:28	08/11 19:16 - 08/13 20:07	48.9	10.98	10	1992
3/9/92	12:51	03/09 02:54 - 03/10 14:50	35.9	7.58	5	1992
2/13/92	14:12	02/13 10:30 - 02/25 01:39	279	10.98	10	1992
2/7/92	14:56	02/07 09:39 - 02/12 23:24	134	11.38	11	1992
1/6/92	12:43	01/06 04:54 - 01/09 06:05	73.2	5.50	3	1992
8/11/91	09:37	08/11 05:48 - 08/12 09:06	27.3	3.00	1	1991
3/27/91	16:10	03/26 08:37 - 03/29 20:58	84.4	3.60	1	1991
8/14/90	13:38	08/14 12:47 - 08/14 20:37	7.8	1.20	<1	1990
7/6/90	18:37	07/06 15:53 - 07/23 12:50	405	10.10	9	1990
12/16/88	12:57	12/16 12:57 (single report)	0.0	1.80	<1	1989

FLOODS / SIGNIFICANT IMPOUNDMENTS -

REGULATION - The dam itself regulates flows on Sunnycove wash.

DIVERSIONS - None known

ACCURACY - Good

JUSTIFICATION - Monitor impoundments behind Sunnycove dam for flood warning to town of Wickenburg.

UPDATE – January 17, 2002

Source
Flood Control District of Maricopa County, Sunnycove FRS ID #5248
http://156.42.96.39/alert/Flow/5248.htm, December 11, 2003.

Mar 1 2004 6:31 PM

SUNNYCOVE FRS

S E N S O R 5 2 4 8

STRUCTURE DATA

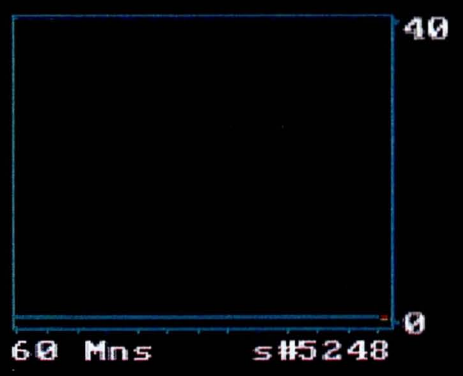
TOP OF DAM.... 2180.4 ft
SPILLWAY ELEV. 2170.1 ft
DEPTH @ SPILL... 37.1 ft
SPILLWAY CAP.. 216 ac-ft

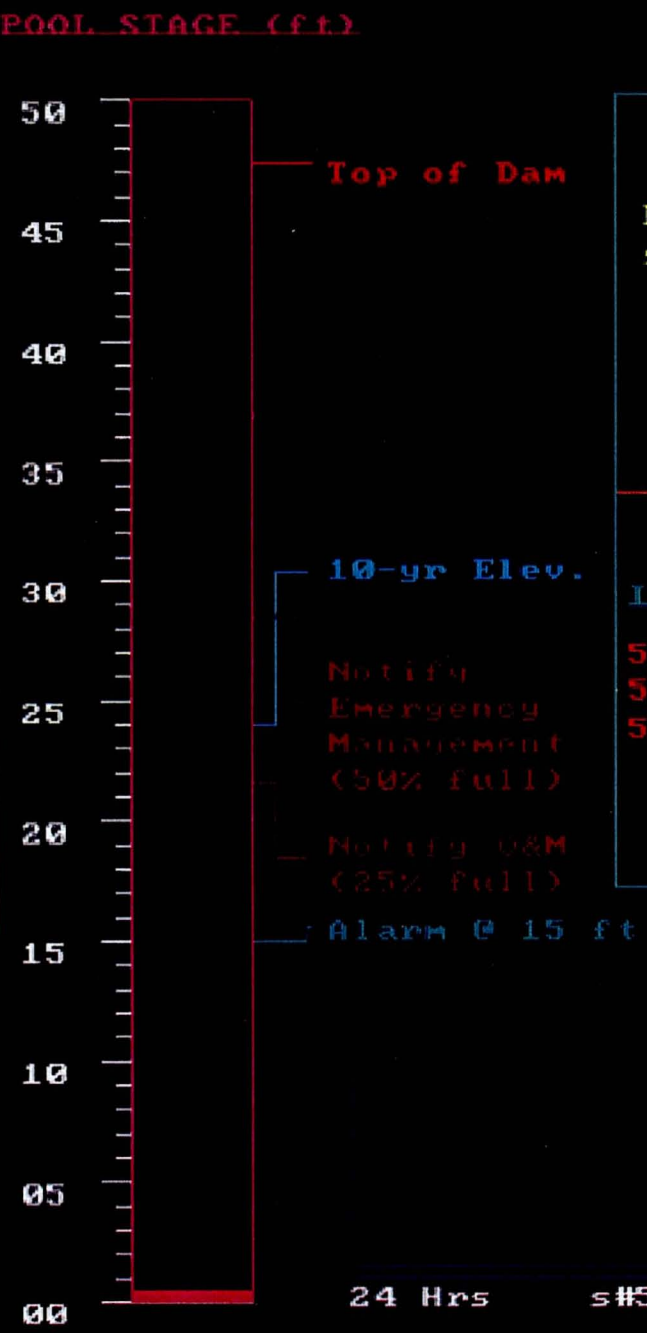
STRUCTURE DIMENSIONS

LENGTH..... 400 ft
TOP CREST..... 14 ft
DRAW DOWN..... 10 days

CURRENT CONDITIONS:

Stage (ft) ------> 0.6
Discharge (cfs) -> 0
Volume (ac-ft) --> 0.0
Percent Full (%)-> 0





PROBLEM AREAS

Downstream APS power station

BASIN RAINFALL

I.D.	15m	60m	3hr	WYtD
5245	0.00	0.00	0.00	2.56
5230	0.00	0.00	0.00	2.48
5260	0.00	0.00	0.00	2.05

PT @ 0.6 ft (ALERT Staff Gage)

Note: PT @ 2133.6 ft MSL (ADMS datum)

GEOLOGICAL CONSULTANTS INC

KENNETH M. EUGE, R.G.

Memorandum

April 20, 2004

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

From: Ken Euge, R.G.
Principal Geologist

Subject: Geological Input to Structures Assessment Reports
Sunnycove FRS, Wickenburg, Maricopa County, Arizona.
Work Order No. 1
FCDMC Contract No. 2003C015
Geological Consultants Project No. 2003-161 (SC)

In response to your request, submitted herewith are the input sections, including geology, seismicity, and ground subsidence, for the Sunnycove FRS structures assessment report. We have not numbered the report sections for the Structures Assessment Report. However, please edit as appropriate to conform to your report format.

Sunnycove FRS

Geologic Setting

Sunnycove FRS is located in hilly terrain within the northeast-central portion of the Sonoran Desert section of the Basin and Range Physiographic Province near its boundary with the Arizona Transition Zone Section. The latitude and longitude of the center part of the structure is approximately 33° 57' 27" N and 112° 44' 18" W based on NAD 27 datum. This portion of the Basin and Range is characterized by broad alluvial fans that are locally dissected and gently sloping connected valleys bounded by high, rugged northwest, north, and northeast trending mountains including the Date Creek and Weaver Mountains to the north, the Vulture Mountains to the south and the Wickenburg Mountains to the east that rise abruptly to form broad, elongated, deep, sediment-filled valleys produced by block faulting and folding during past episodes of mountain/basin bounding fault movements (Cooley, 1977). The Dam is within the town limits of Wickenburg, Arizona off the northeastern flank of the Vulture Mountains in the southeast quarter of Section 11, Township 7 North, Range 5 West (Figure 1, Site Location Map).

The geology of the project area is briefly described in the Report of Geologic Investigation, Wickenburg Watershed, Sunnycove Floodwater Retarding Structure (SCS, 1974). The geologic description provided in this memorandum is excerpted from the SCS report with the description supplemented with observations made during the site visits (February 2003).

The geology of the Sunnycove FRS area includes bedrock described as Tertiary age fanglomerate which is generally moderately to well cemented. The fanglomerate is dark yellow brown to brown, silty poorly sorted gravelly sand and sandy gravel. The course grained angular to subangular fragments are predominantly clasts of tuff, basalt, andesite and rhyolite with minor amounts of granitic and metamorphic rock fragments (Photo 1). The unit is cemented with caliche to a near rocklike consistency. Although the unit is structurally massive, the fanglomerate is moderately to well stratified as evidenced by the grading of both the coarse and fine-grained materials within individual beds. Structurally, the strata are tilted at a very low angle to dip toward the northeast. No structural discontinuities such as faults, joints, or fractures were observed in the exposures of the fanglomerate at this site.

Photo 1: Outcrop of Tertiary age fanglomerate exposed downstream of right abutment. Note layered stratigraphy and angular rock fragments held in finer-grained matrix.

Groundwater

No groundwater was encountered in any of the test holes or exploratory excavations made during the site subsurface investigation (SCS, 1974).

Geology and Soils of the Dam and Outlet Works.

Sunnycove FRS abutments are founded on moderately to well cemented, dense fanglomerate which is weathered at the surface exposures and tends to disaggregates in response to weathering. The weathered zone is relative shallow, generally ranging between one and three feet. A 12-foot wide cutoff trench is excavated into the fanglomerate upstream of the dam centerline. Figure 2 depicts a typical cross section of the dam showing the excavation depth into the fanglomerate.

Young alluvial terrace deposits are present in the upper portion of the left abutment where the younger deposits overly the fanglomerate. The young terrace deposits are poorly graded, slightly silty gravel (GM-GP) with some silty sand (SM) and silt (ML) layers. This deposit is medium dense to dense with some very loose pockets. At station 16+85, these deposits extend to depth in excess of fifteen feet. Because of the loose consistency of the younger terrace deposits at the left abutment, the left abutment was over-excavated to remove the loose soils and found the embankment in the fanglomerate.

A narrow ridge of fanglomerate bisects the floodplain of the main wash. This ridge spur has a base width of about 70 feet and extends about nine feet above the elevation of the main wash at the dam centerline at about station 14+50 (Photo 2, Figure 3) (SCS Drawing 7-E-23090, Sheet 3 of 18).

Photo 2: Fanglomerate ridge spur upstream of FRS embankment. Main channel to left of ridge spur.

Flood plain deposits include variable and stratified, lenticular alluvial deposits that are composed of clean well-graded sand (SW) to silty, clayey sand (SC). The lenticular deposits are not laterally extensive. These deposits extend to depths ranging from two to eight feet below grade and unconformably overlie the fanglomerate.

Emergency Spillway

The emergency spillway centerline for Sunnycove FRS is located about 225 feet northwest of the left abutment. The entire bottom of the spillway is excavated into the cemented fanglomerate with the exception of relative short sections at the spillway approach and at the outlet section where it discharges into a natural channel. With the exception of a relatively thin capping of alluvial terrace deposits, the emergency spillway side slopes are composed of cemented fanglomerate. Minor headward erosion is occurring at the emergency spillway outlet (Photo 3). The curved spillway, which is trapezoidal in cross section, has 2:1 H:V side slopes. The maximum depth of cut is about 35 feet. The spillway has an average width of about 100 feet. The spillway is a broadcrested, unlined channel (Figure 4) (SCS Drawing 7-E-23090, Sheet 4 of 18.

Photo 3: View of gully erosion feature at downstream outlet of emergency spillway channel.

Photo 4: Emergency spillway channel looking downstream.

Site Investigation

The Soil Conservation Service initiated its dam site investigation in

February 1970 with a supplemental borrow area investigation in May 1975. The geological characterization of the site was defined using several techniques including: geologic reconnaissance and mapping, auger drilling, and backhoe trenching. Test pit and drill hole location are depicted on Figure 2 (Site Geologic Map). They are also depicted along the dam centerline on Figure 3. A summary of the explorations is provided in the following table.

Table 1. Summary of Geologic Testing
Sunnycove FRS

Structure	Exploration Techniques	
	Exploratory Borings	Test Trenches/ Pits
Dam	2	5
Principal Spillway	9	2
Emergency Spillway	4	16
Borrow Site	-	20

Seismicity

No seismicity or earthquake evaluation was conducted for the Sunnycove FRS dam design based on a review of the project files. However, a seismicity evaluation for all of the FCDMC dam structures was conducted in 2002. The report entitled "Seismic Exposure Evaluation, Dam Safety Program, Flood Control District of Maricopa County" describes the various seismotectonic zones, fault zones, design earthquake, and characteristic ground motion affecting FCDMC structures (AMEC, 2002).

Sunnycove FRS is situated within the Southern Basin and Range (SBR) Source Zone as defined by AMEC (2002) which includes the Sonoran Seismic Source Zone defined by ADOT (1992). The SRB source zone appears to be tectonically quiescent, with a low level of seismicity and few neotectonic faults that would be considered active or potentially active sources of earthquakes (Bausch and Brumbaugh, 1994; ADOT, 1992). The large historic earthquake within this zone was a magnitude 5.0 that occurred in the southern part of the source zone in 1965. Only a few

minor faults occur in the SBR (AMEC, 2002; ADOT, 1992). Earthquake epicenters and Quaternary faults are shown in Figure 3 of the AMEC (2002) report.

The deterministic and probabilistic analysis of seismic hazard affecting the Sunnycove FRS area was conducted by AMEC (2002) to establish seismic attenuation relationships and the maximum probable earthquake. The closest Quaternary age fault is the Sand Tank Fault located about 77 miles south of the site. According to AMEC (2002) the maximum credible earthquakes for this fault source ranges between M6.2 and M6.6. The background earthquake, which is estimated to have a higher maximum magnitude of M7.2, was applied to the regression relationship to derive the horizontal ground acceleration. The recommended peak ground acceleration calculated for the Sunnycove FRS area, based on the background seismic source, is 0.10 g (10 percent of gravitational acceleration) (AMEC, 2002).

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. However, in the Wickenburg area, there is no documented evidence of excessive groundwater withdrawal nor land subsidence.

No unconsolidated, compressible basin fill soils are believed to be present beneath the Sunnycove FRS. The subsurface geological conditions in the embankment dam area (Figures 2, 3, and 4), consists of relatively hard, cemented Tertiary age fanglomerate (at the surface) and in the subsurface deposited on crystalline bedrock indicate the potential for land subsidence due to groundwater withdrawal does not exist at the Sunnycove FRS site.

According to Staedicke (1995), because there is no history of extensive groundwater pumping or subsidence, the NRCS (formerly the SCS) has never surveyed the Sunnycove FRS structure. Although land subsidence is not expected to affect the Sunnycove FRS, we recommend the structure be surveyed periodically (say at 5-year intervals). The data should be compiled in the FCDMC structures subsidence monitoring program

Earth Fissures

No earth fissures, related to land subsidence, are documented nor reported as occurring within the Sunnycove FRS project area. Geological conditions in the Sunnycove FRS area preclude the development of earth fissures at this site.

Bibliography-Sunnycove FRS

ADOT; 1992; Development of Seismic Acceleration Contour Maps for Arizona; ADOT Report No. AZ92-344; prepared by K.M. Euge and B.A. Schell, Geological Consultants, Phoenix, Arizona and I.P. Lam, Earth Mechanics Inc. Fountain Valley, CA; September 1992; 328 p.

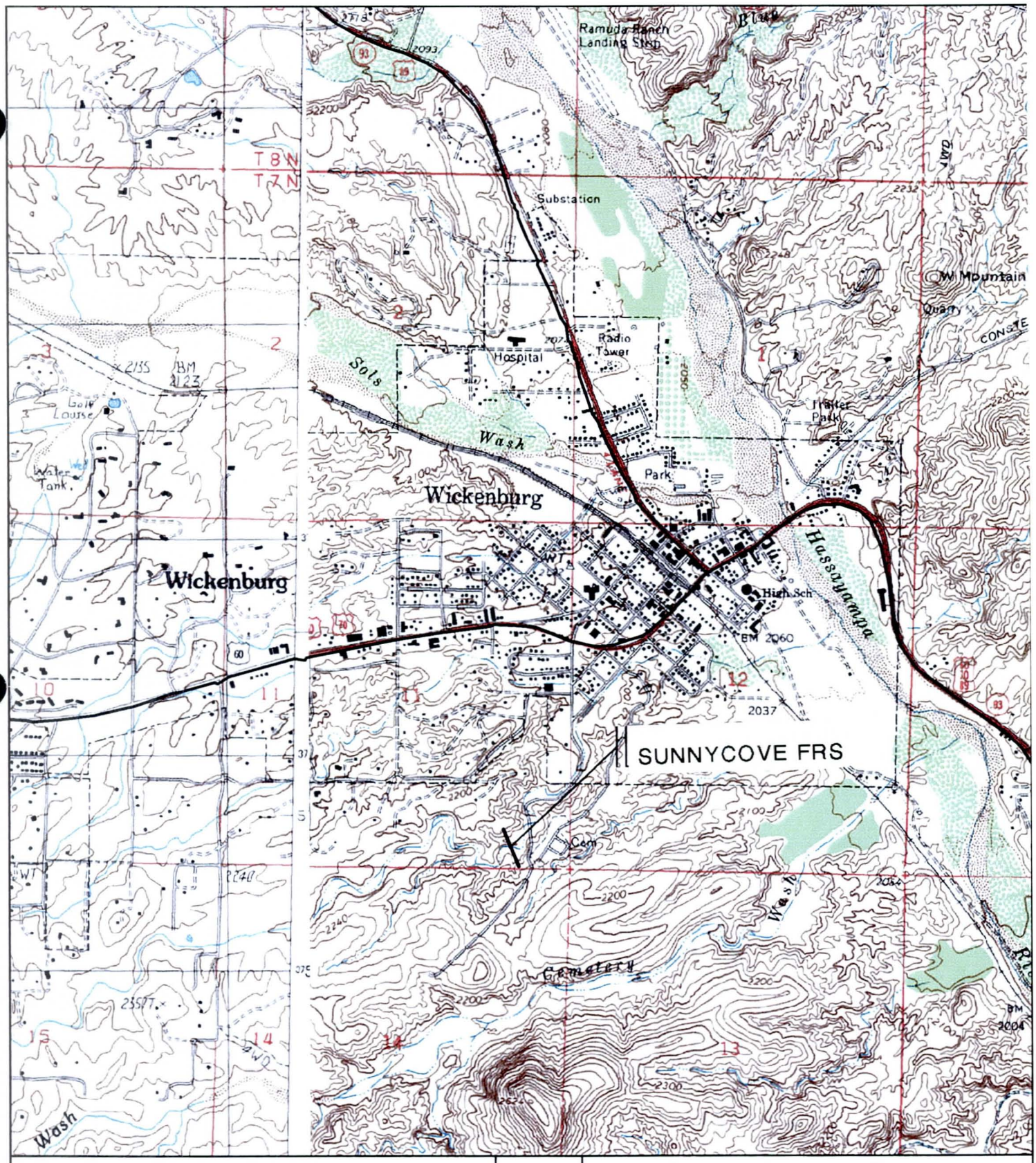
AMEC; 2002; consultants report entitled "Seismic Exposure Evaluation, Dam Safety Program, Flood Control District of Maricopa County"; prepared for Flood Control District of Maricopa County, Phoenix, Arizona; AMEC Job No. 0-117-001122, Task 2; May 2, 2002; 22 p.

Cooley, M.E.; 1973; Map Showing Distribution and Estimated Thicknesses of Alluvial Deposits in the Phoenix Area, Arizona; U.S. Geological Survey Miscellaneous Investigation Series Map I-845-C; 1 sheet; Scale 1:250,000.

Cooley, M.E.; 1977; Map Showing Selected Alluvial Structures and Geomorphic Features; U.S. Geological Survey Open-File Report 77-343; 29 p.

Soil Conservation Service; 1974; Report of Geological Investigation, Wickenburg Watershed, Sunnycove Flood Retarding Structure; prepared by Aubrey Sanders; December 1974; 10 p.

Staedicke, J.M.; 1995; Settlement Monitoring of Earthen Dams Operated by the Flood Control District of Maricopa County; prepared by Jan M. Staedicke, FCDMC; June 1, 1995;



Base map from USGS Wickenburg 7.5 minute topographic quadrangle (1964, photoinspected 1978)

0 0.25 0.5 1 Miles

1:24,194

Structures Assessment Program
Sunnycove FRS
Site Location Map
Figure 1

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

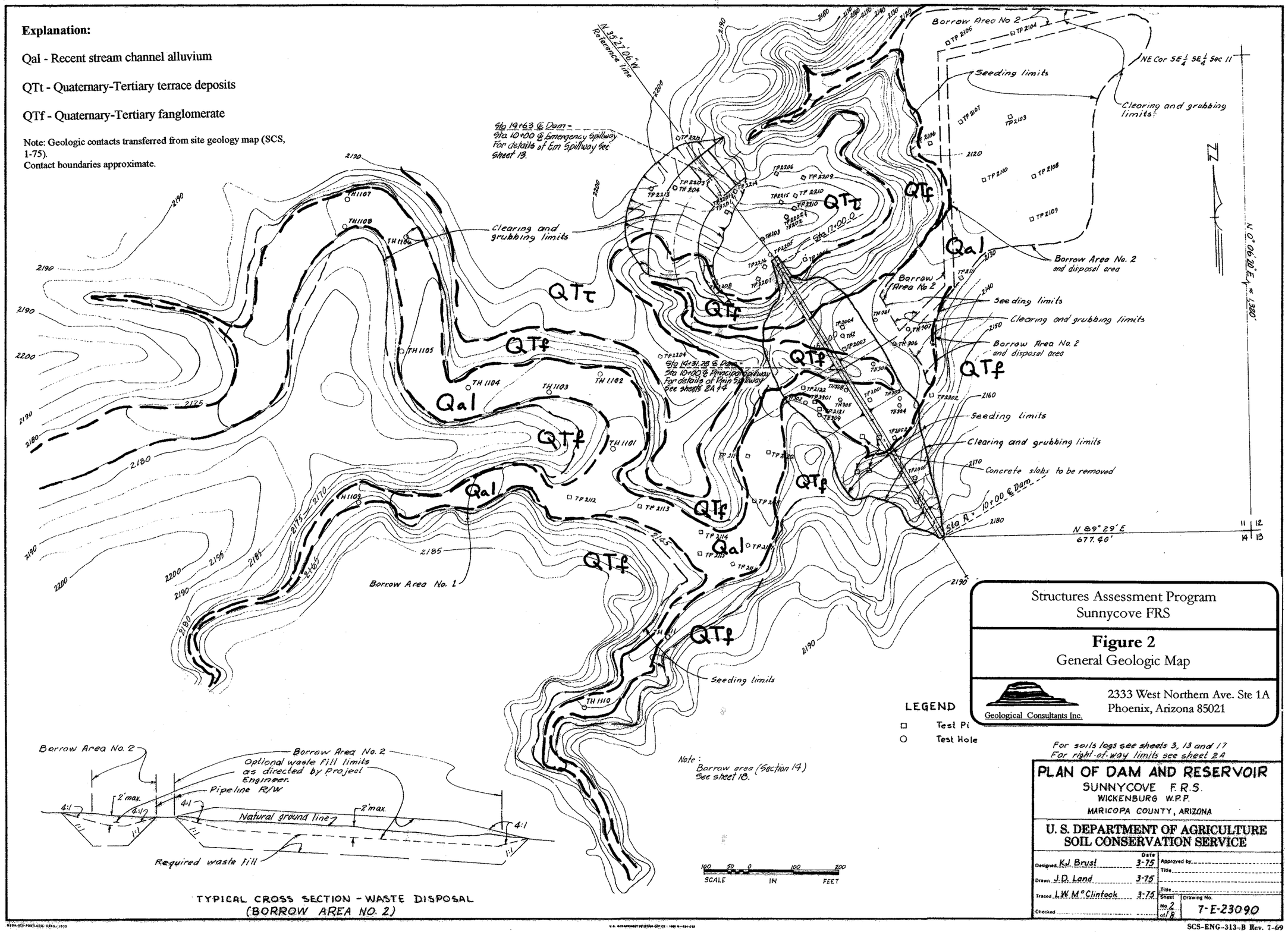
Explanation:

Qal - Recent stream channel alluvium

QTt - Quaternary-Tertiary terrace deposits

QTf - Quaternary-Tertiary fanglomerate

Note: Geologic contacts transferred from site geology map (SCS, 1-75).
Contact boundaries approximate.



Structures Assessment Program
Sunnycove FRS

Figure 2
General Geologic Map

Geological Consultants Inc.
2333 West Northern Ave. Ste 1A
Phoenix, Arizona 85021

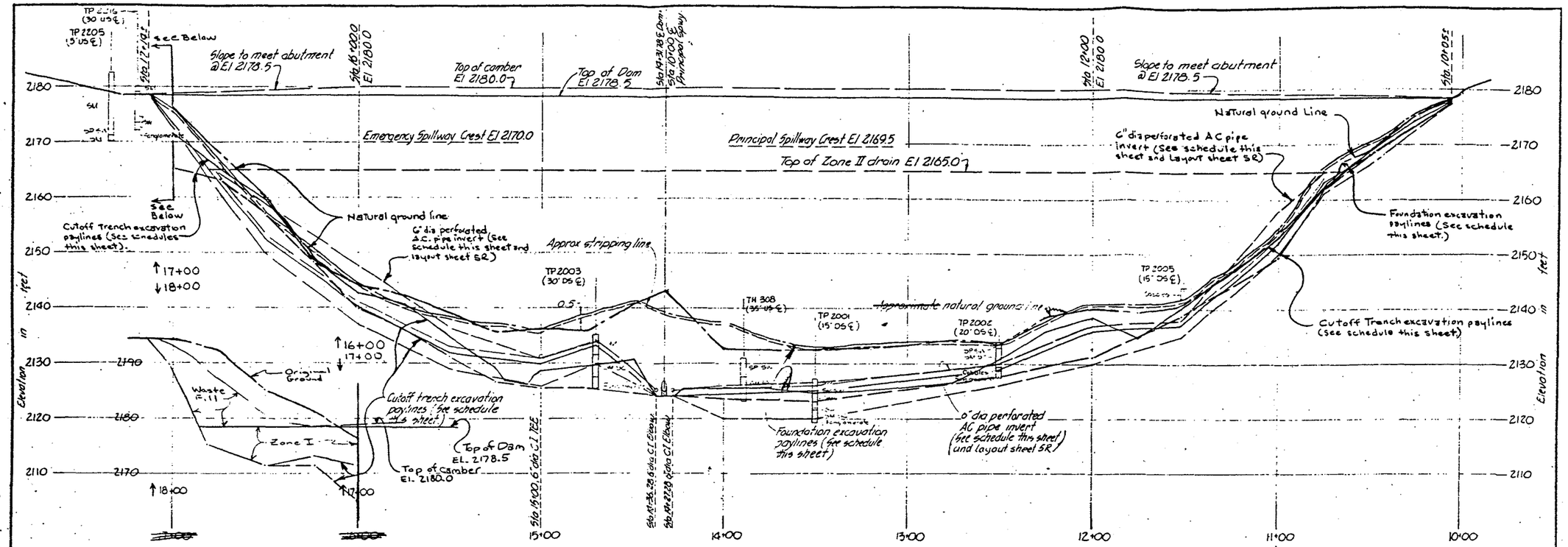
For soils logs see sheets 3, 13 and 17
For right-of-way limits see sheet 2A

PLAN OF DAM AND RESERVOIR
SUNNYCOVE F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust	Date 3-75	Approved by	
Drawn J.D. Land	3-75	Title	
Traced L.W. McClintock	3-75	Title	
Checked		Sheet No. 2 of 8	Drawing No. 7-E-23090

SCS-ENG-313-B Rev. 7-69



PROFILE ON ℄ DAM
(Looking downstream)

SCHEDULES

FOUNDATION EXCAVATION		
STATION	ELEVATION	
10+05 ±	~~2177.5~~	2177.7
10+50	~~2168.0~~	2168.9
10+75	~~2163.0~~	2162.1
11+00	~~2152.0~~	2153.1
11+50	~~2137.0~~	2139.8
12+00	~~2136.0~~	2138.5
12+50	~~2129.0~~	2131.1
——	——	
13+50	~~2125.0~~	2124.7
14+27.28	2124.1	
14+36.28	2124.1	
14+70	~~2133.0~~	2130.6
15+00	~~2130.0~~	2127.9
15+50	~~2132.0~~	2134.9
16+00	~~2140.0~~	2140.0
16+50	~~2153.0~~	2159.7
17+00	~~2174.0~~	2171.4
17+14 ±	~~2177.5~~	2172.4

℄ Elevations at ℄ Dam Excavated surface in cross sections shall be to firm conglomerate except from Sta. 3+50 to Sta 14+50 where foundation excavation shall be uniformly shaped to the foundation excavation lines required for Principal Spillway. See Profile on ℄ Principal Spillway sheet 4R

CUTOFF TRENCH EXCAVATION		
STATION	ELEVATION	
10+05 ±	~~2177.5~~	2177.4
10+50	~~2166.0~~	2168.0
~~10+75~~	~~2161.0~~	
11+00	~~2150.0~~	2149.4
11+50	~~2135.0~~	2139.0
12+00	~~2130.0~~	2131.5
12+50	~~2126.0~~	2127.7
13+50	~~2120.0~~	2123.3
14+00	~~2120.0~~	2123.3
14+27.28	2124.1	
14+36.28	2124.1	
14+70	~~2130.0~~	2124.5
15+00	~~2126.0~~	2125.5
15+50	~~2129.0~~	2131.0
16+00	~~2137.0~~	2139.9
16+50	~~2150.0~~	2157.0
17+00	~~2172.0~~	2165.1
17+75	~~2177.5~~	2171.3

℄ Elevation at 7.0' upstream from ℄ of Dam.

CUTOFF TRENCH EXCAVATION	
17+50	2171.3
17+80	2175.2
18+01	2189.7
18+17	2195.0

DRAIN PIPE PLACEMENT		
STATION	ELEVATION	REMARKS
10+75	~~2164.0~~ 2165.0	Plug beginning end with mortar, or AC cap
11+00	~~2153.0~~ 2158.0	Grade change
11+50	2138.0	Grade change
12+00	2137.0	Grade change
12+50	2130.0	Grade change
~~13+00~~	~~2128.0~~	~~Grade change~~
13+50	~~2126.0~~ 2126.0	Grade change
14+27.28	2125.1	C1 Elbow to 6" dia non-perforated AC outlet pipe
14+36.28	2125.1	C1 Elbow to 6" dia non-perforated AC outlet pipe
14+70	2134.0	Grade change. Plug beginning ends with mortar or two AC caps.
15+00	2131.0	C1 Tee to 6" dia non-perforated AC outlet pipe.
15+50	~~2133.0~~ 2136.5	Grade change
16+00	~~2141.0~~ 2147.5	Grade change
16+50	~~2154.0~~ 2158.5	Grade change
16+75	2164 ±	Plug beginning end with mortar or AC cap

℄ Elevation at 7.0' Downstream from ℄ of Dam.

℄ All Elevations and Stations are approximate and may be adjusted by Engineer.

Notes:

1. Unified soil descriptions are based on field identifications except where an asterisk is shown the classification has been based on laboratory analysis.
2. Complete field logs, laboratory test data and geologic report are available for review in the project office
3. For locations of test pits and test holes see sheet 2R
4. Test pit or test hole location DS = Downstream from ℄ Dam and US = Upstream from ℄ Dam.

Structures Assessment Program
Sunnycove FRS

Figure 3
Centerline Profile

Geological Consultants Inc.
2333 West Northern Ave. Ste 1A
Phoenix, Arizona 85021

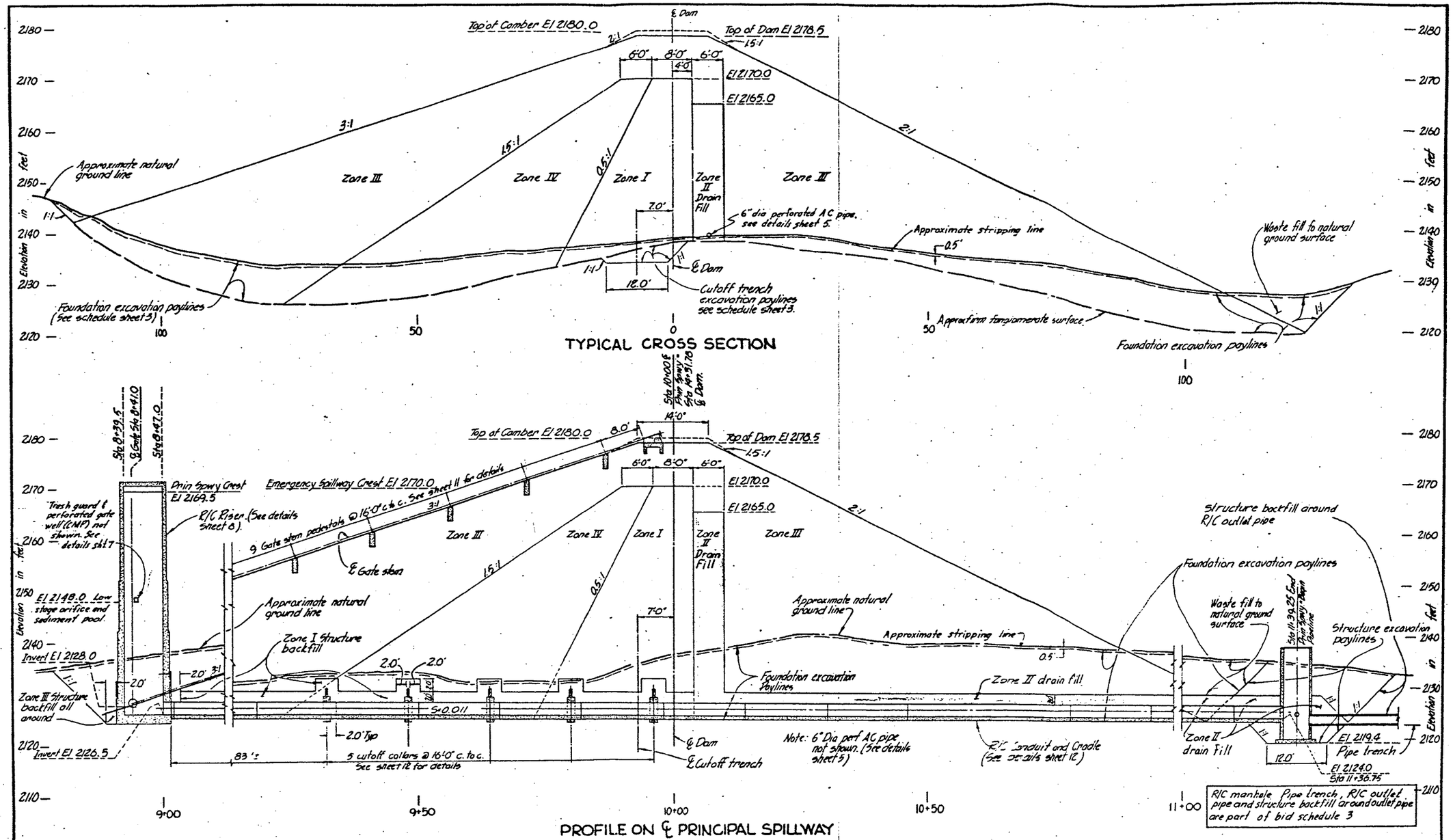
AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Don Wakefield

PROFILE ON ℄ DAM AND EXCAVATION SCHEDULES
SUNNYCOVE F.R.S.
WICKENBURG, W.P.D.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date	
Designed Greg Cunningham	3-75	Approved: by
Drawn Jack Long	4-75	Title
Traced		Title
Checked		Sheet No. 3 of 18 — Drawing No. 7-E-23090

SCS-ENG-313-B Rev. 7-69



Structures Assessment Program
Sunnycove FRS

Figure 4
Dam Cross Section & Spillway Centerline Profile

Geological Consultants Inc.
2333 West Northern Ave. Ste 1A
Phoenix, Arizona 85021

PROFILE ON ℄ PRINCIPAL SPILLWAY AND TYPICAL CROSS SECTION OF DAM
SUNNYCOVE F.R.S
WICKENBURG · W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed Greg Cunningham 3-75
Drawn Jack Land 3-75
Traced
Checked
Approved by
Title
Sheet No. 4 of 18
Drawing No 7-E-23090

SCS-ENG-313-B Rev. 7-69

GEOTECHNICAL MEMORANDUM
Sunnycove FRS FMEA – Phase I Structures Assessment

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

Submitted by:

Gannett Fleming, Inc.
3001 E. Camelback Road
Phoenix, AZ 85016

Registered Professional Engineer (Civil)
CERTIFICATE NO.
35654
DEAN B.
DURKEE
Date Signed 6/30/04
ARIZONA, U.S.A.

for Debora J. Miller, Ph.D., P.E.
Senior Project Manager

Dean B. Durkee, Ph.D., P.E.
Manager, Earth Science & Hydraulics

June 30, 2004

043394

1.0 REVIEW OF PREVIOUS GEOTECHNICAL DOCUMENTATION

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

- Report of Geologic Investigation, Wickenburg Watershed, Sunnycove Floodwater Retarding Structure (SCS, 1974)
- Design Report, Sunset and Sunnycove Floodwater Retarding Structures, Maricopa and Yavapai County, Arizona (SCS, 1975)
- Construction Plans and Specifications, Sunnycove FRS, Wickenberg Watershed Protection and Flood Prevention Project (SCS, 1975a)
- As-built Construction Plans, Sunnycove FRS (SCS, 1976)
- Sunnycove Flood Retarding Structure – Arizona Dam No. 7-48, Maricopa County, Arizona, Phase I Inspection Report, National Dam Safety Program (AWC, 1979)

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

1.1 Foundation Conditions

The dam is founded on competent, generally well-cemented fanglomerate bedrock which is generally free of jointing. Recent alluvial and colluvial deposits, and weathered bedrock overlying the unweathered bedrock (fanglomerate) were removed as part of foundation preparation. A high gravel terrace deposit was identified in the upper left abutment on the original dam alignment. The terrace deposit consisted of uncemented, silty gravels and sands containing some very loose pockets of material. The terrace deposit was recognized as a potential seepage problem during the design process, and the dam alignment was shifted upstream to avoid founding the left abutment on these materials. The cutoff trench was excavated into competent fanglomerate in the upper left abutment along the revised alignment.

1.2 Foundation Surface Irregularities and Embankment Settlement

As-built plans indicate that the excavated cutoff trench was shaped to a smooth profile. However, potential foundation bedrock surface irregularities are indicated on the drawings along the dam axis outside of the cutoff trench. In particular, a bedrock ridge is evident crossing the dam alignment on the lower left valley section. The ridge spur can be seen upstream from the dam (see Photo 2 in Geologic Section provided by Ken Euge). The ridge extends upstream to downstream (east to west) across the dam axis, and is skewed to the NW-SE oriented axis of the dam. The ridge crosses the centerline of the dam at about Station 14+30 near the outlet works alignment. The as-built drawings indicate that the ridge was substantially removed and shaped. However, a trapezoidal trench was cut through the remnant, shaped ridge to install the outlet pipe. The most notable foundation irregularity is left of the principal outlet conduit where as-built plans show a high point in the foundation. Based on the profile on dam centerline (Sheet No. 3 in the as-built drawing set), the trench cut slope was approximately 2H:1V and about 6.5 feet high at dam centerline. Another subtle bedrock ridge is evident at the base of right abutment, crossing the dam axis at centerline near Station 12+00. These subtle foundation surface irregularities occur under the deeper sections of the embankment, and are not considered to be problematic with regard to transverse cracking. The mechanism for potential transverse crack formation is tension cracking over the irregularity due to differential settlement of the

embankment locally. Monitoring for transverse tension cracking in the dam crest above these foundation irregularities has been recommended as part of the FMEA.

1.3 Embankment Soils

The dam was constructed as a zoned embankment, with material zones as shown on Table 1. A typical cross-section of the embankment is shown as Figure 1.

Table 1. Sunnycove FRS - Embankment Material Zones

Zone	Description	USCS	Properties	
I	Central core –sandy clay, clayey sand, and sandy silt	SC, SM, SC-SM	20% fines min., 6" max. rock size	
II	Chimney drain – clean, coarse sandy gravel from imported source	SP, GP	Sieve	% Passing
			3-inch	100
			1½-inch	85-100
			¾ - inch	70-95
			No. 4	40-60
			No.10	20-40
			No. 20	0-20
			No. 40	0-5
III	Shells – gravelly sands or sandy gravels from designated borrow	variable	Fines limited to 15% max., 9" max. rock size	
IV	Transition – similar to Zone III but higher fines content	variable	No fines limitation, up to 35% gravel, 6" max.	

The materials used to construct Zones I, III, and IV were derived from local borrow sources consisting of alluvial deposits in the vicinity of the dam. The Geologic Report (SCS, 1974) provides logs for test pits and borings, and laboratory test results for bulk samples obtained from two designated borrow areas: Borrow Area 1 located within the limits of the sediment pool, and Borrow Area 2 located within the floodplain downstream from the dam. Materials in the borrow areas consisted primarily of silty sand (SM), clayey sand (SC), and clean to slightly silty sand (SP, SW-SM, SP-SM). Materials in Borrow Area 2 appeared to have higher fines content, and this area was designated as the primary source for the core section (Zone I). Supplemental borrow investigations were also performed in two tributary washes upstream from the dam, and in the main Sunnycove wash above the sediment pool elevation. The main purpose for these supplemental investigations was to locate additional quantities of granular materials for Zone III.

Laboratory testing of representative borrow soils was reported in the Geologic Report (SCS, 1974). Table 2 provides a summary of the laboratory testing results for representative samples from Borrow Areas 1 and 2. The results on Table 2 show that materials within the borrow areas were variable, and that segregating materials by fines content and plasticity would have been challenging.

Table 2. Summary of Representative Laboratory Test Results - Borrow Areas 1 and 2 Samples

Borrow Area/ (samples)	USCS	% Fines (-#200)	PI (%)	G_s	γ_d (pcf)	w_{opt} (%)	φ	c
1/2113.1	SW-SM	9	np	2.67	105	18.8	32°	0
2116.1	SM	31	2	2.70	114.5	14.3	35°	0
2117.1	SW-SM	8	np	2.65				
2118.1	SM	21	np	2.69				
2116.2	GP-GM	8	np	2.67				
2120.1	SM	19	np	2.69				
2120.2	SP-SM	8	np	2.65				
2120.3	SP-SC	11	20	2.65				
2/(2101.1	SC	17	16	2.67				
2104.1	SC	27	10	2.71				
2105.1	SM	27	np	2.67	115.1	13.5	36°	0
2109.1	ML	53	6	2.73				
2109.2	SW-SM	10	np	2.66				
2111.1	SC-SM	33	7	2.70				
2111.2	SC	33	12	2.72				

1.4 Original Slope Stability Analyses

Based in part on the laboratory tests (as summarized on Table 2, derived from SCS, 1974), the designers assumed the parameters shown on Table 3 for the slope stability analyses. Slope stability analysis results were obtained for the loading conditions shown on Table 4.

Table 3. Embankment Soil Properties Used in Stability Analysis

Property	Zone I	Zone III
Dry unit weight (γ_d) (pcf)	110	100
Moist unit weight (γ) (pcf)	126	114
Saturated unit weight (γ_{sat}) (pcf)	130	125
Angle of internal friction (φ) (degrees)	25 to 30*	32
Cohesion (c)	0	0

*Shear strength (φ) for the core (Zone I) was estimated, and was not based on laboratory tests.

Table 4. Original Slope Stability Analyses Results

Slope	Conditions	Minimum F.S.
2H:1V downstream	Dry slope – infinite slope analysis	1.25
3H:1V upstream	Steady seepage, full reservoir to emergency spillway crest	1.8

The Design Report (SCS, 1975) indicated that the factor of safety of 1.25 for the 2H:1V downstream slope was acceptable because the Zone III outer shell material was anticipated to have a higher shear strength than the $\varphi = 32^\circ$, c=0 strength assumption used in the analysis. The shear strength assumption was reported to be conservative because the materials specified for Zone III would have lower fines contents (<10%) than the materials tested in the laboratory. However, based on the laboratory tests, the materials with higher fines contents actually exhibited higher shear strengths. The shear strength assumption of $\varphi = 32^\circ$, c=0 is considered to be reasonable and conservative for the broadly graded Zone III materials, but not for the reason stated in the Design Report. The assumed shear strength is conservative because it is based on the lowest value from direct shear testing of three samples, as summarized on Table 2.

The design intent was for the core section to have higher fines content such that it served as the primary water barrier, and for Zone III to have lower fines content and provide strength and faster drainage to improve stability during drawdown. Ultimately, however, the gradational distinction between Zones I, III, and IV is subtle, and for practical purposes the differences in engineering behavior of these materials is small or negligible. The Specifications limited fines content in Zone III to 15% (revising the original Design Report assumption of 10% fines in Zone III) to accommodate materials from the available local borrow sources. This high fines content does not provide a "free draining" zone as was assumed in the stability analyses for rapid drawdown. Also, the designers assumed full development of a phreatic line within Zones I and IV, and used this as primary justification for incorporating a drain zone (Zone II) in the dam. A more critical purpose for Zone II is actually as a filter to protect against internal erosion and piping. Supplemental geotechnical analysis have been performed as part of this Phase I Structures Assessment to correctly document the slope stability and filter compatibility based on current criteria and our understanding of the structure and zoning. These analyses are described in the following section.

2.0 SUPPLEMENTAL GEOTECHNICAL ANALYSIS

2.1 Supplemental Seepage and Slope Stability Analysis

In support of the Phase I Structures Assessment, Gannett Fleming has conducted preliminary supplemental seepage and slope stability analysis for the Sunnycove FRS to document the expected stability of the structure under anticipated loading conditions. The assumptions used in the original stability analysis are suspect for the following reasons:

- **Assumption No. 1: Development of a steady state phreatic line –** The original design assumed that a steady state phreatic line would be likely to develop within Zones I and IV because of the "...slow release rate of the low stage inlet."
  - **Revised Assumption No. 1: Development of a steady state phreatic line is unlikely.** The maximum detention time for a 100-year event will be less than 10 days, assuming the outlet does not clog. In our estimation, this is insufficient time for a steady state seepage line to develop.
- **Assumption No. 2: Zone III is free-draining** - Rapid drawdown was not considered in the original design because the Zone III shell was thought to be "free-draining" in comparison to the "slow drawdown rate" of the principal spillway.
  - **Revised Assumption No. 2: Zone III is not free draining.** The Zone III materials, contain up to 15% fines content and are therefore probably not free-draining.

However, since the wetting front advance into the upstream zone during a detention event is expected to be very minimal (see following seepage analysis), rapid drawdown is not anticipated to cause slope instability.

Gannett Fleming conducted preliminary seepage analyses using a numerical model (SEEP/W) that allows simulation of the transient wetting front advance into the upstream shell of the dam during a storm detention event, or sequence of events. The results are shown on Figure 2 for a sequence of two back-to-back 100-year floods.

The SEEP/W model correctly accounts for unsaturated and saturated hydraulic conductivities and gradients within the soil to predict the rate of infiltration during a transient event. A standard "Silty Sand" material type was selected from the model's database to represent the Sunnycove embankment materials. The database provides the necessary unsaturated hydraulic parameters for use in the simulation. The vertical saturated hydraulic conductivity was assumed to be 0.6 ft/day (2×10^{-4} cm/s), which was the value used by the SCS designers for the filter/drain design as reported in the Design Report (SCS, 1975). The embankment was modeled as a homogeneous section (ignoring the subtle differences between zones), with a horizontal:vertical anisotropy (k_h/k_v) ratio of 10:1 for the hydraulic conductivity.

Figure 2 shows the simulated development of the seepage line into the embankment with time during a sequence of two consecutive 100-year events (multiple storm scenario). This impoundment scenario was modeled to estimate realistic, but conservative phreatic lines for use in evaluating upstream slope stability. It is evident that even following multiple storm events, the wetting front will advance to a very limited extent into the dam. During the first impoundment event, the phreatic line does not even reach the drain, and barely penetrates into the foundation. The model results indicate rapid dissipation of the upstream pore pressures as the pool level drops after each event.

Slope stability was analyzed using the program SLOPE/W, which imports the estimated pore pressures from the SEEP/W analysis. Stability was evaluated using the same material property assumptions that the SCS designers used except that a small cohesion intercept (c = 10 psf) was assigned for the strength estimate in order to exclude trivial, extremely shallow (infinite slope) failure surface results. Figures 3 and 4 show the estimated minimum factors of safety for the upstream slope at two times: (1) during drawdown after the 2nd flood impoundment (factor of safety = 2.1), and (2) after drawdown immediately following two consecutive impoundment events (factor of safety = 2.0). The factor of safety is slightly higher at the intermediate impoundment stage because the pool provides additional buttressing against the slope. Note that the slope is predicted to be nearly completely drained (low phreatic line) immediately following the events, based on the assumptions used in the model for impoundment times, drawdown times, and hydraulic conductivity of the materials.

Figure 5 shows the results of slope stability for the downstream slope. Downstream slope stability was modeled to document the factor of safety for a more meaningful failure mode than the shallow, infinite slope analysis that was completed during original design. The minimum factor of safety shown (1.4) is representative of a substantial slope failure that would impact a portion of the crest of the dam. As previously discussed, this factor of safety is based on a conservative shear strength assumption for the Zone III materials, which may not be representative of the materials actually used to construct the dam.

The results of the preliminary supplemental seepage and slope stability analyses are summarized on Table 5.

Table 5. Preliminary* Supplemental Slope Stability Analyses

Slope	Report Figure	Conditions	Minimum F.S.
3H:1V upstream	3	Intermediate drawdown level during sequence of two, 100-yr impoundment events	2.1
	4	Immediately following two, 100-yr impoundment events	2.0
2H:1V downstream	5	Dry slope – critical failure surface	1.4

* These results are based on preliminary analyses conducted using soil parameters from previous design reports and assumed hydraulic conductivity parameters.

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

Zone II is shown on the as-built drawings as a 6-ft wide, vertical chimney drain immediately downgradient from the Zone I core. This zone was designed to act as a drain, but its most important function is to serve as a filter to protect against potential internal erosion and piping of the core materials if transverse cracks were to develop.

Because of its critical function as a filter, the Zone II gradation was checked against current filter criteria in accordance with the NRCS, National Engineering Handbook, Chapter 26 "Gradation Design of Sand and Gravel Filters" (NRCS, 1994). Figure 6 shows what is believed to be a representative gradation curve for the finer materials used in the Zone I "Base Soil" (graphed with solid red triangular symbols). This gradation curve is re-plotted from the gradation curve for Field Sample 2104.1 from the Geological Report (SCS, 1974). The sample was taken from Borrow Area #2 downstream from the dam, and is described as a "reddish brown, calcareous, coarse clayey sand, that classifies as SC according to the Unified Soil Classification system (USCS).

The base soil gradation curve was adjusted for gravel content as shown by the curve graphed with green open triangular symbols. The filtering and permeability (k) criteria which are the basis for the required filter gradation are shown for the adjusted base soil curve by the solid circles on the 15% passing line. The Zone II specification band is shown to fit within the filtering and permeability limits. Thus Zone II is compatible as a protective filter against piping of Zone 1 materials assuming it was placed in accordance with the specified gradation limits.

3.0 RECOMMENDATIONS FOR FUTURE ACTIONS

3.1 Monitoring

In recognition of the presence of substantial zones of uncemented granular materials in the bedrock ridge that forms the upper left abutment, we recommend monitoring for seepage through

the left abutment and on the south side of the emergency spillway cut during any sustained impoundment events.

In recognition of foundation irregularities, we recommend visual monitoring for transverse tension cracking in the dam crest near Stations 12+00 and 14+30.

3.2 Phase II Documentation of Slope Stability and Seepage Analyses for Main Dam

Gannett Fleming does not anticipate any problems with slope stability under any reasonable loading conditions for Sunnycove FRS. However, we could not find adequate documentation of slope stability factors of safety for specified loading and design criteria that have been established by appropriate jurisdictional agencies. Table 6 shows the definitions of various loading conditions and a comparison between the current NRCS design criteria that are outlined in TR-60 (SCS, 1985), and the current criteria as presented in the ADWR dam safety rules and regulations for jurisdictional dams.

Table 6. Slope Stability Design Criteria

Loading Condition	TR-60 (SCS, 1985)	ADWR[1]
End of Construction (upstream and downstream slopes)	1.4	1.3[2]
Rapid Drawdown (upstream slope)	1.2	1.2
Steady seepage w/o seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	1.5	1.5
Steady seepage w/ phreatic surface developed from critical partial pool elevation (upstream slope)	n/a	1.5
Steady seepage w/seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	1.1	n/a[3]

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

The original stability analysis, and our preliminary (Phase I) stability analyses do not completely document factors of safety for all the loading conditions that would need to be evaluated under current NRCS or ADWR criteria. Table 7 summarizes the results from the original stability analysis, results from the preliminary supplemental analysis performed as part of this Phase I study, and indicates where additional analysis is required to document factors of safety under all loading conditions.

Table 7. Slope Stability Documentation to Date and Additional Analyses Required to Comply with Current Design Criteria

Loading Condition	Original Analysis	Preliminary Phase I Analysis	Recommendation (see text for discussion)
Rapid Drawdown (upstream slope)	Not evaluated	2.0	(1)
Steady seepage w/o seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	1.25	1.4	(2)
Steady seepage w/ phreatic surface developed from critical partial pool elevation (upstream slope)	1.8 (full pool) Partial pool not analyzed	Not analyzed	(3)
Steady seepage w/seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	Not analyzed	Not analyzed	(4)

(1) **Rapid Drawdown Stability (upstream slope)**: Preliminary analyses were conducted as part of this Phase I study that simulated a plausible scenario for development of the seepage line into the dam under multiple temporary impoundment events, and to assess the upstream slope stability under normal drawdown conditions. These analyses show that it is very unlikely that a steady state phreatic line will develop in the Sunnycove FRS, assuming the outlet works is operational and is not clogged for sustained periods of time following a flood event. ADWR criteria require that an "instantaneous" drawdown analysis be performed. The ADWR guidance and rules were developed for water retention dams, and the criteria are interpreted to mean that rapid drawdown stability should be evaluated assuming that a steady state phreatic line has developed from the normal high reservoir pool elevation. It is not clear what is meant in the ADWR regulations by "instantaneous" drawdown, and this may be subject to interpretation. Additional analyses would be required to assess and document rapid drawdown factors of safety under these assumptions. A recommended analysis procedure is outlined as follows:

a. Establish the steady state phreatic line and pore pressure distribution using 2-D seepage analysis. Use reasonable assumptions for hydraulic conductivity and anisotropy for the embankment materials based on available information.
b. Model the dissipation of pore pressures with time, starting from the steady state initial condition, and assuming a worst case drawdown rate. The drawdown rate should be based on the current outlet capacity, or an adjusted (higher) capacity if the outlet is modified. This is not an "instantaneous" drawdown assumption, but is much more realistic given the physical constraints on the rate of drawdown. Pore pressure dissipation with time from the steady state condition can be estimated using either a transient numerical flow analysis or a suitable analytical procedure.

c. Evaluate the upstream slope stability at various stages of the drawdown by inputting the instantaneous pore pressure grids and reservoir levels from the seepage analysis. Report the minimum value, and compare against the design criteria (minimum factor of safety = 1.2).

(2) **Downstream Slope Stability Under Steady Seepage**: The minimum factors of safety that have been computed for the dry downstream slope do not achieve the minimum criteria of 1.5 (see Table 6). Gannett Fleming does not consider the infinite slope analysis that was done in the original design as representative of a "critical" failure scenario. Our preliminary analysis evaluated more substantial failure surfaces which resulted in a minimum factor of safety of 1.4. This factor of safety was computed using the original strength assumptions of $\varphi = 32^\circ$and c=0. It is likely that this strength assumption is conservative, based on the results of three strength tests that were done on the original borrow materials (Table 2). The following additional analyses could be performed to further evaluate and document the stability of the downstream slope:

a. Conduct a search for reasonable "critical" failure surfaces on the dry downstream slope using a higher shear strength, which may be more representative of the in-place materials. The average shear strength for three tests on borrow samples is $\varphi = 34^\circ$ and c = 0.
b. Re-test the shear strength using representative samples of materials from the downstream Zone III shell. Sample representative materials from the downstream slope using a backhoe or drill rig, and evaluate gradation, plasticity, and compaction characteristics. Compact shear strength test samples to 95% maximum dry density (per the placement specifications), and test in direct shear or triaxial shear to estimate the actual shear strength of the Zone III fill. Re-compute the factor of safety based on the revised shear strength.

(3) **Upstream slope stability under steady seepage, partial pool**: The original analysis evaluated upstream slope stability under steady seepage for the maximum pool elevation only, resulting in a minimum factor of safety of 1.8. The ADWR criteria for partial pool conditions is intended for water retention dams, in which a steady state phreatic line may develop for intermediate pool elevations. The factor of safety may be lower for the intermediate pool conditions than the steady state condition under maximum pool. The following analysis could be done to document the minimum partial pool factor of safety, under the scenario that the outlet works is clogged such that the steady state phreatic line develops:

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

(4) **Pseudo-static stability analysis (critical downstream slope section)**: Seismic stability analyses were not performed as part of the original design. To document seismic stability under current design criteria, the following analysis could be conducted:

a. Based on the regional seismicity review performed for the Casandro Wash Dam, as documented the design report for that structure (CH2M Hill, 1995), a reasonable estimate for the peak ground acceleration (PGA) for the area is 0.1g. ADWR guidance (R-15-1216(B)(2)(b)(i), effective June 12, 2000) recommends using a pseudo-static coefficient = 60% of the PGA. This would result in a pseudo static coefficient = 0.06.
b. Conduct the pseudo-static analysis on the downstream stability section with the lowest static factor of safety, and report the result.

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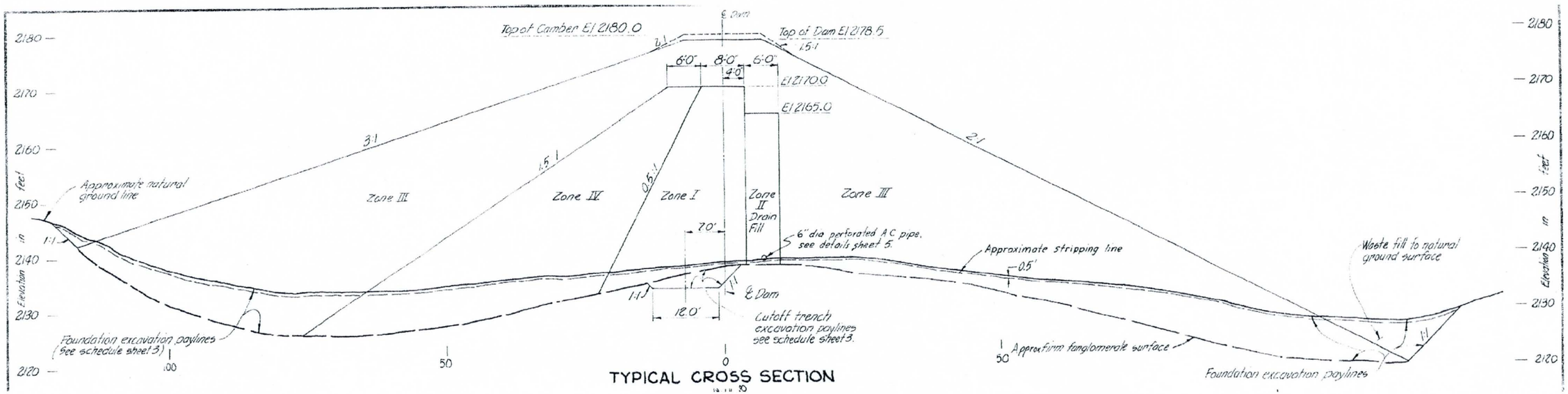
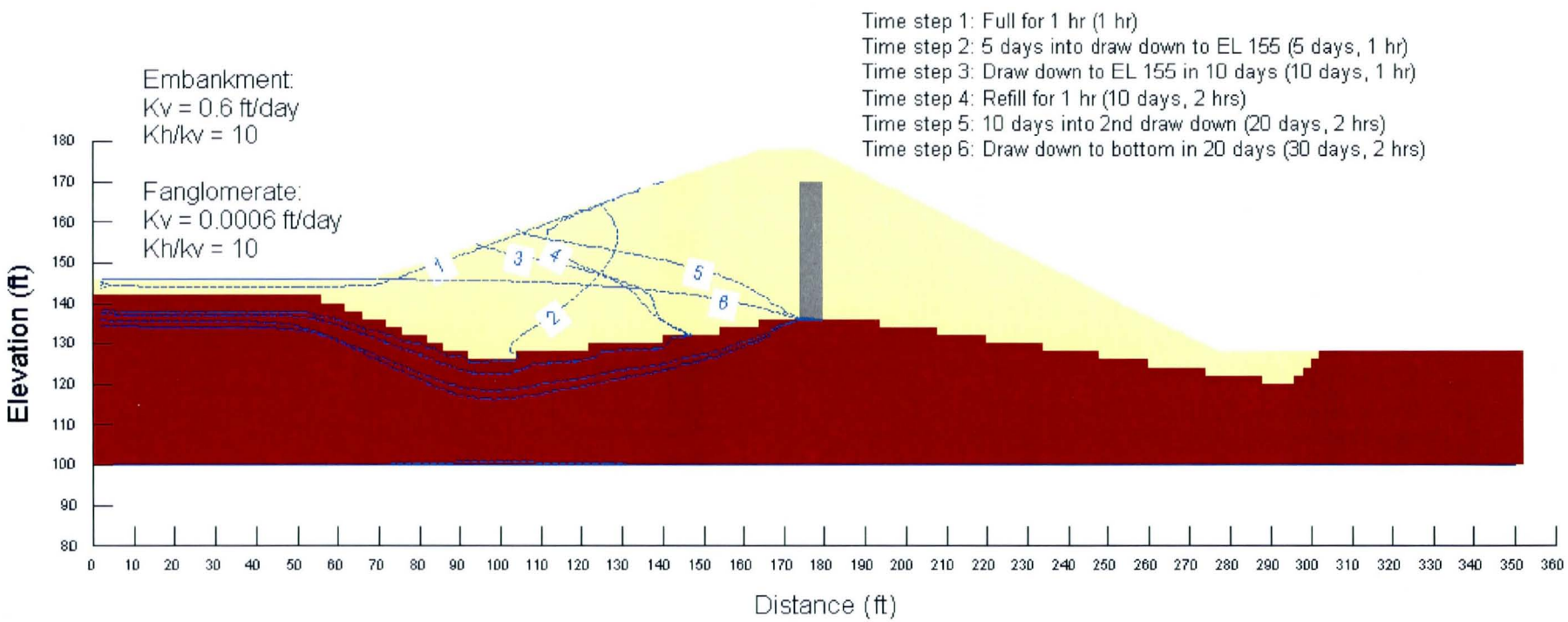


TYPICAL CROSS SECTION

Gannett Fleming

TYPICAL CROSS SECTION
SUNNYCOVE FLOOD RETARDING STRUCTURE
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

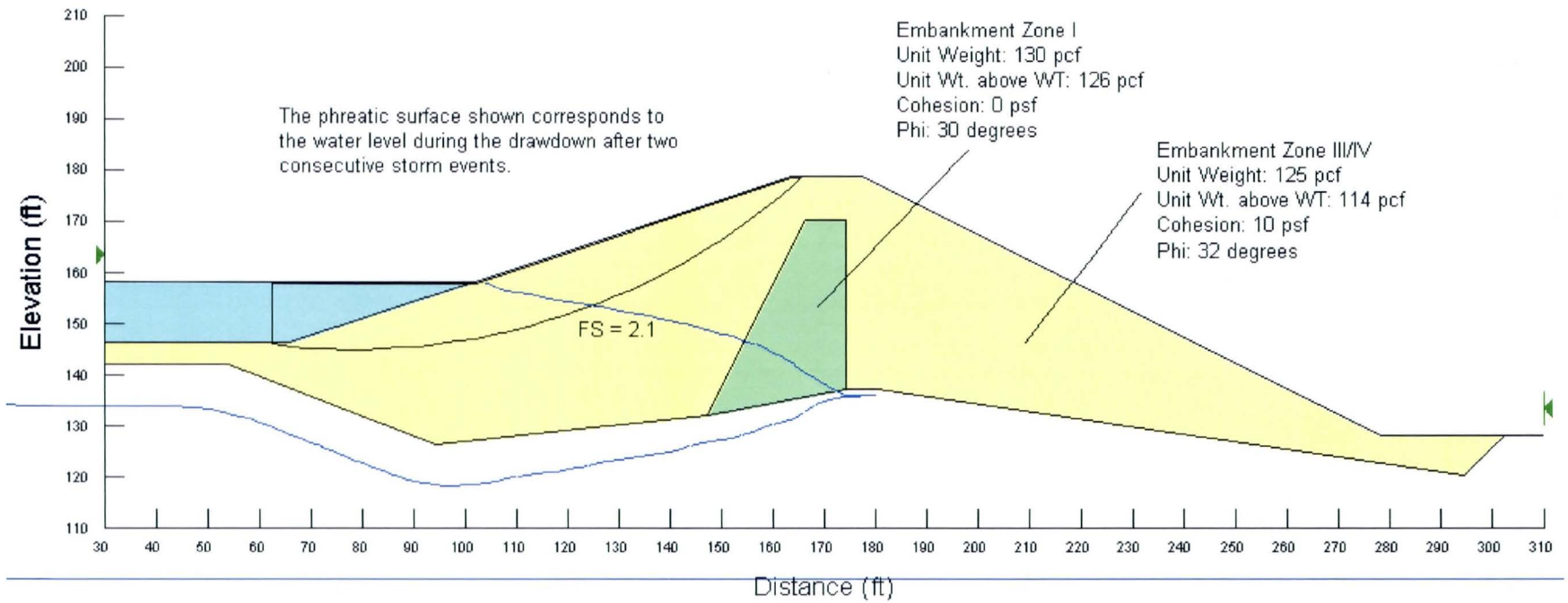
FIGURE
1



Gannett Fleming

SEEPAGE ANALYSIS
SUNNYCOVE FLOOD RETARDING STRUCTURE
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

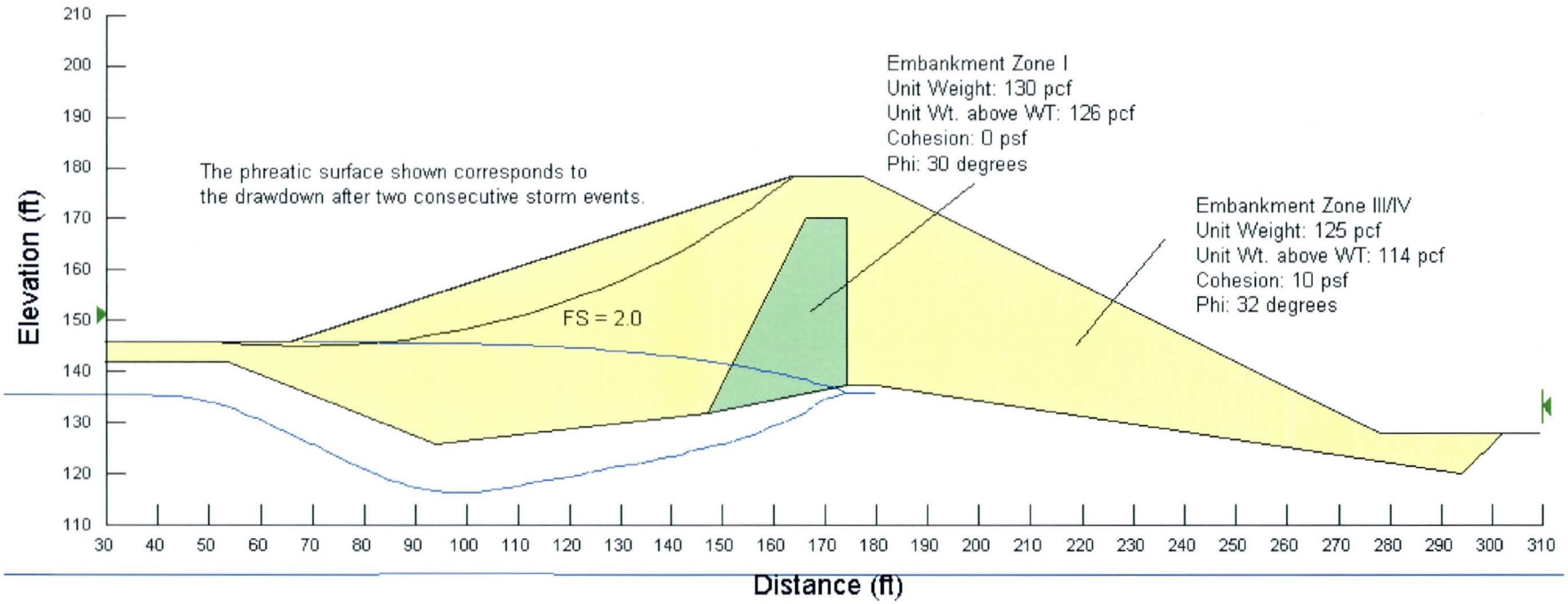
FIGURE
2



Gannett Fleming

UPSTREAM STABILITY DURING DRAWDOWN AFTER TWO CONSECUTIVE EVENTS
SUNNYCOVE FLOOD RETARDING STRUCTURE
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

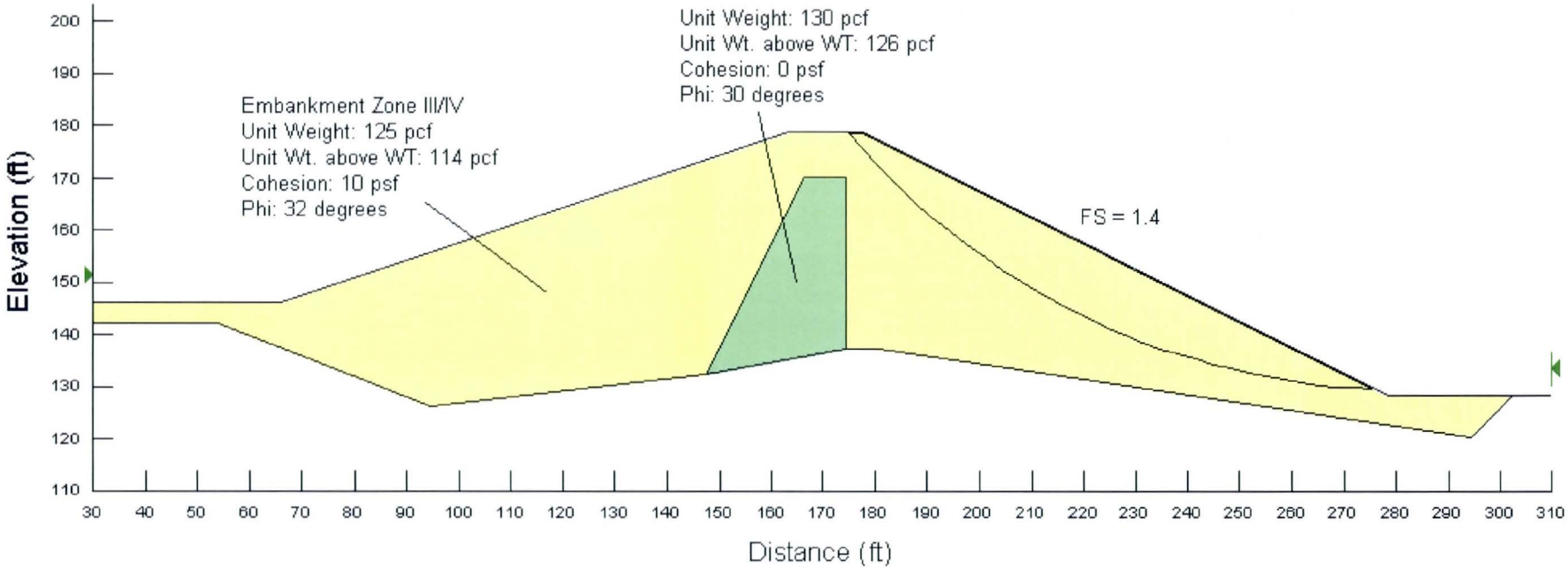
FIGURE
3



Gannett Fleming

UPSTREAM STABILITY AFTER DRAWDOWN FROM TWO CONSECUTIVE EVENTS
SUNNYCOVE FLOOD RETARDING STRUCTURE
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

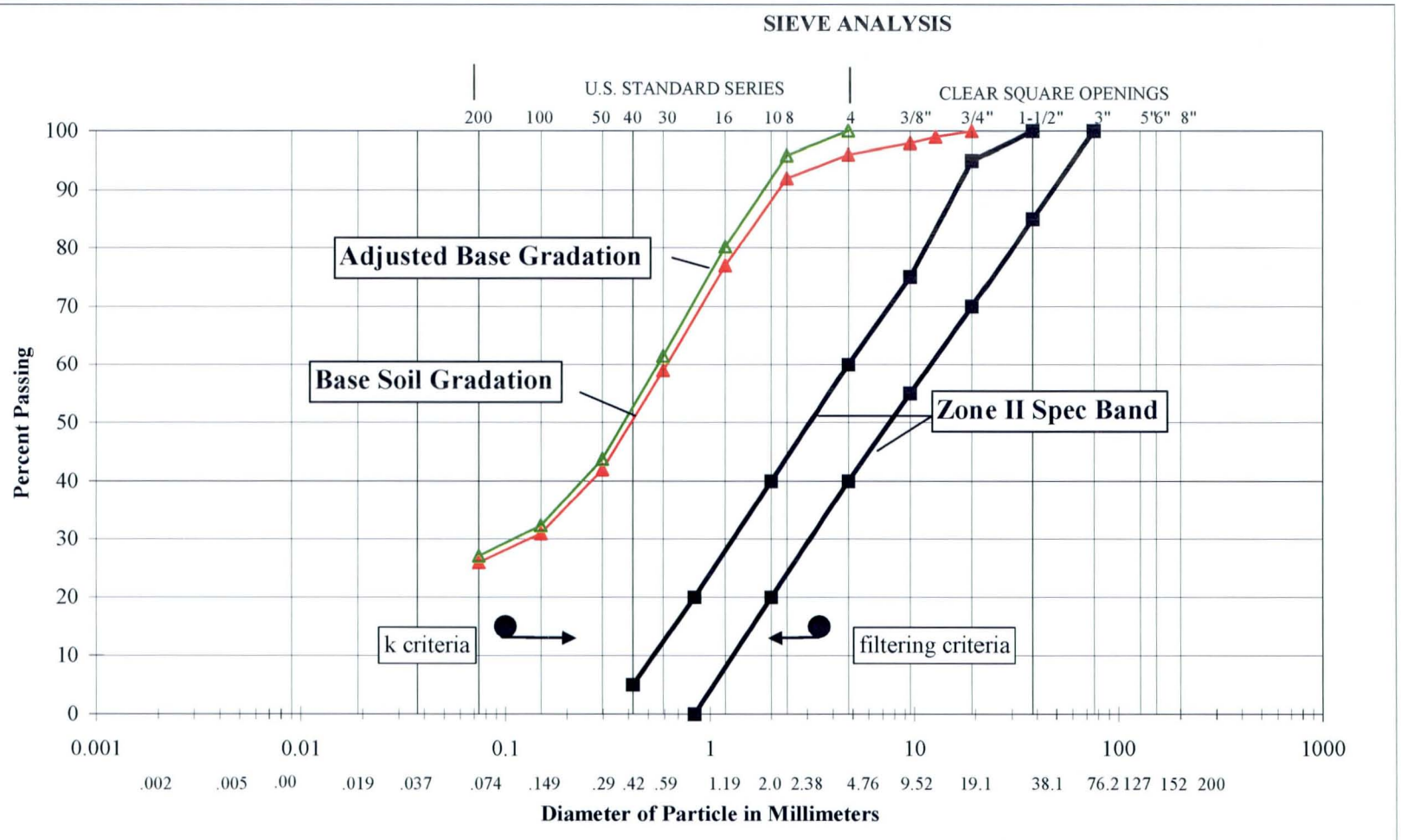
FIGURE 4



Gannett Fleming

DOWNSTREAM STABILITY
SUNNYCOVE FLOOD RETARDING STRUCTURE
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FIGURE
5



CLAY (plastic) TO SILT (non-plastic)	SAND			GRAVEL		COBBLES
	FINE	MEDIUM	COARSE	FINE	COARSE	

Gannett Fleming

FILTER COMPATIBILITY CHECK FOR ZONE II
SUNNYCOVE FLOOD RETARDING STRUCTURE
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

FIGURE
6