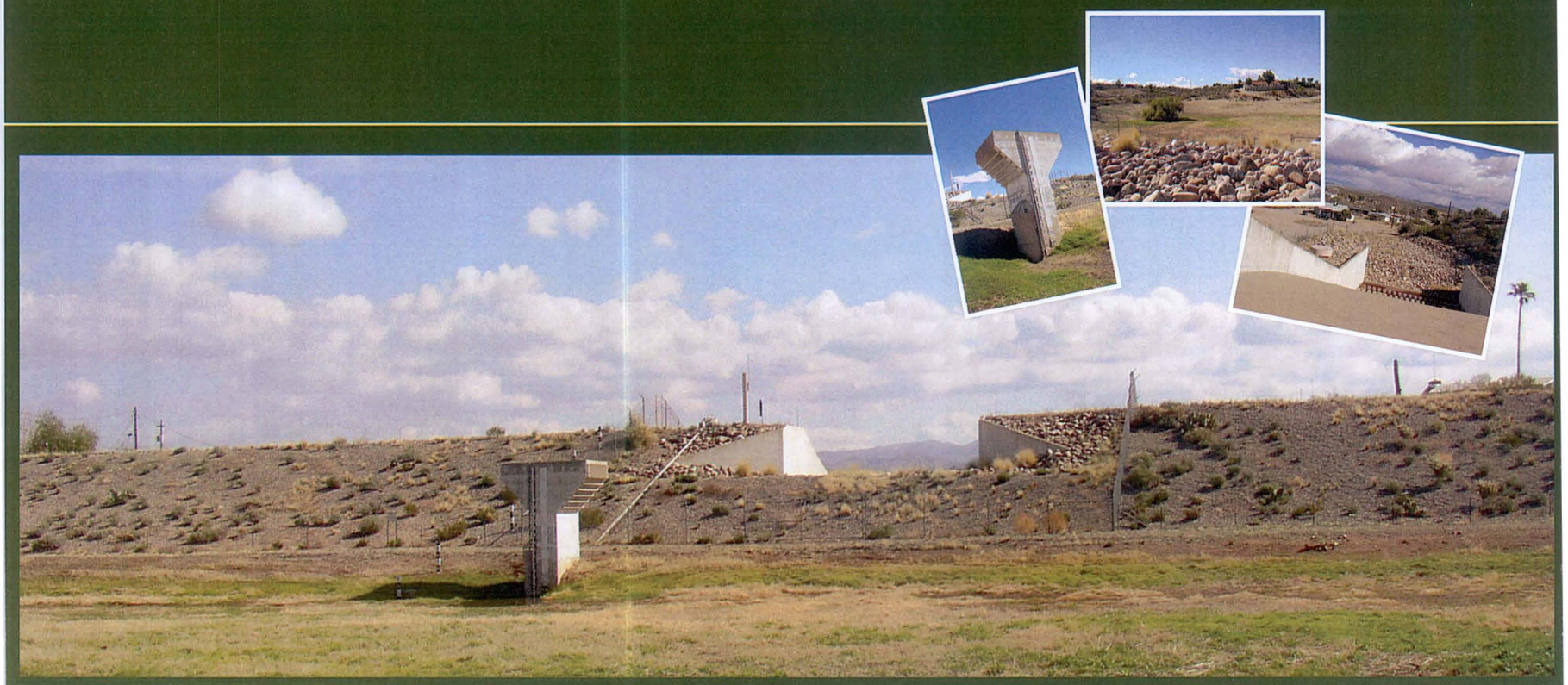

Sunset Flood Retarding Structure
Individual Structure Assessment

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

Kimley-Horn
and Associates, Inc.

On-Call Phase I Assessment

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June 2004

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INDIVIDUAL STRUCTURE ASSESSMENT
for
SUNSET FLOOD RETARDING STRUCTURE

TABLE OF CONTENTS

Section No. **Page No.**

List of Tables

Table No.

List of Figures

Figure No.

List of Appendices

Appendix.

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 **Sunset 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 Sunset 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

Sunset FRS is located in the eastern portion of the Town of Wickenburg, less than a mile from the center of the Community. The Sunset FRS is located on Sunset Wash and is a tributary to the Hassayampa River in the northeast corner of Section 11, Township 7N, and Range 5W. The project consists of the FRS embankment structure, principal spillway, and an emergency spillway.

The reservoir behind the FRS is 8 acres with a capacity of 55 acre-feet. A permanent pool will not be retained in the reservoir, instead, the FRS and reservoir are designed to trap floodwater and store it only for as long as it takes to release it slowly and safely downstream. The reservoir capacity is then restored to handle a future flood.

The emergency spillway is a reinforced concrete structure constructed on the top of the dam embankment. It consists of entrance control structure, a straight chute and a stilling basin with baffle blocks. Construction of the FRS and appurtenant structures was completed in April 1975.

The Sunset FRS is an earthfill structure with a central chimney drain/filter. The length of the FRS is 488 ft with a maximum height of 30.5 feet and crest width of 14 feet. The reservoir capacity is approximately 55 acre-feet at emergency spillway crest elevation 2131 feet. Sunset FRS is located in the Town of Wickenburg, Arizona south of the intersection of U.S. 60 and east of Mariposa Drive. The maximum recorded

impoundment for Sunset FRS is 34 acre-feet with a stage of 12.27 feet at the FRS on September 26, 1997(ADWR, http://156.42.96.39/alert/Flow/5233.htm).

Watershed
The dam was constructed across Sunset Wash, a tributary to Sols Wash and the Hassyampa River. The drainage area contributing to the dam is 0.6 square miles (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]. The elevation in the drainage area ranges from 2420 feet above mean sea level to 2116 feet above mean sea level at the dam site (AWC, 1979).

Flood Pool
The spillway crest was constructed at an elevation of 2131 feet in order to contain the 100-year flood. The as-built data show at an elevation of 2131 feet, the reservoir has a capacity of 55 ac-ft. The peak inflow into the reservoir during the 100-year flood and the PMF were estimated to be 857 cfs (SCS, 1970) and 7093 cfs (Phase I Inspection Report). The design allowed for a reservoir capacity of 55 acre feet to the spillway.

Dam Embankment
The dam was designed as a homogeneous dam with a vertical drain zone just downstream of the centerline. Zone I, forming the bulk of the dam, was constructed of clayey sands and silty sands with some gravelly clayey sand. A 6-foot wide vertical drain extends from four feet above emergency spillway level to the top of the outlet conduit. The embankment was constructed with 3:1 (horizontal to vertical) upstream slope and a 2:1 downstream slope. A 12-foot wide berm of sandy, gravelly material was added to the upstream toe to increase stability. Six inch asbestos-cement pipes were placed at the bottom of the vertical drain running from each abutment to the outlet conduit, where they connect to six inch non-perforated outfall pipe.

Principal Outlet Works
The principal spillway consists of a reinforced concrete intake tower with an uncontrolled overflow at elevation 2131 feet. The design outflow from the principal spillway is controlled by a 9-in by 9-in orifice at elevation 2120 ft.

Flow into the pipe is controlled by a reinforced concrete intake tower with the screened intake approximately at spillway crest level, 2131 feet. Discharge through the conduit enters a manhole at the downstream toe of the dam from whence flow is conveyed via the Sunset/Sunnycove Pipeline to an outfall in the Hassayampa River, approximately 1.5 miles away. The outlet works is designed to empty the flood control pool, accumulated during a 100-year storm in less than 10-days.

Emergency Spillway
The emergency spillway was designed to pass the PMF with 0.3 ft of freeboard. As-built data specify the emergency spillway will discharge 3907 cfs for the predicted PMF water surface elevation of 2141.2 feet. The emergency spillway is a reinforced concrete structure constructed on top of the dam embankment. The spillway is 40 feet wide with a

discharge capacity of 3400 cfs at maximum water surface elevation of 2140.48 feet (SCS, 1975b). The emergency spillway crest elevation is 2131 feet.

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 Sunset 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 0.3 feet of residual freeboard.

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 Sunset Wash drainage area and Sunset FRS. The hydrologic study included a routing routine to route the 100-year flows through the Sunset 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 740 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 2126.93 ft, which is approximately 4 ft below the emergency spillway elevation (2131 ft).

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 740 cfs with no outflow from the dam. The 100-year 6-hour inflow into the structure for the SCS study was 857 cfs with outflow in the principal spillway.

1.3 Geologic and Geotechnical Considerations

Geologic Setting. Sunset 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' 55" N and 112° 44' 30" 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.

Seismicity. No seismicity or earthquake evaluation was conducted for the Sunset 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 Sunset 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 Sunset FRS site.

According to Staedicke (1995) because there is no history of extensive groundwater pumping or subsidence, the NRCS has never surveyed the Sunset FRS structure. Although land subsidence is not expected to affect Sunset FRS, Kimley-Horn recommends that the structure be periodically surveyed. The last survey was completed

by the Flood Control District of Maricopa County in 2003, the results of this survey are located in Appendix B.

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

Foundation Conditions. The foundation soils at the site are described as Tertiary alluvial deposits of dense silty, gravelly sand which are cemented to various degrees. Geologists refer to this older, cemented alluvium as fanglomerate. The fanglomerate is overlain by a thin mantle of recent alluvium in the main channel floodplain area. The low density, recent alluvial materials were removed to depths up to 12 feet from the dam foundation during construction. The dam is reportedly founded on competent fanglomerate bedrock in the main channel and right abutment sections.

The mid to upper left abutment consists of variably cemented, stratified alluvial materials and manmade fill and debris. This area was highlighted by the geologic and design reports as presenting a potential seepage problem (SCS 1974, 1975a, 1975b). The natural geologic materials in the left abutment are described as stratified deposits of gravelly, fine to coarse-grained sand, with variable amounts of silt (generally 10 to 20% silt). The left abutment native soils contain zones with little to no cementation and some clean sand lenses. Fill materials found in the left abutment were described as a loose mixture of sand, silt and gravel containing broken concrete blocks and wood debris. The geologic investigation concluded that the fill had been loose dumped into in a gully just upstream from the originally proposed dam alignment. The dam alignment was shifted about 25 feet downstream to avoid the loose fill and debris, and the original plans indicated that the loose fill materials were to be completely removed to "firm residual soil" as directed by the engineer, and backfilled with Zone 1 compacted fill to restore grade to the original ground surface. The extent of over-excavation and replacement of the waste fill was anticipated to be approximately 250 feet in the upstream direction, and "as directed by Engineer" in the downstream direction. However, the extent of removal of the loose fill laterally (northward) into the abutment was constrained by the presence of an existing building at the top of the slope and it is not known for certain that "firm residual soils" were reached all along the left abutment contact

Embankment Materials. The SCS designers concluded that selective borrowing to construct a zoned embankment would be very difficult because of the stratified nature of the alluvial deposits in the borrow areas. Based on that assessment, the embankment was designed as a homogeneous section, with a vertical chimney drain zone. An upstream berm was incorporated in the design to improve stability during rapid drawdown. A typical cross section of the embankment is shown as Appendix H.

Laboratory testing of representative borrow soils was reported in the geologic report (SCS, 1974). The primary source for borrow materials was the recent alluvial deposits located within the sediment pool and dam foundation. The geologic report described these deposits as inter-layered, inter-fingering, lenticular layers of predominantly silty

sands (SM) and slightly silty, well-graded sands (SW-SM), with minor quantities of slightly clayey poorly graded sands (SP-SC). Three borrow areas were identified as follows: Borrow Area I located in the north arm of the sediment pool immediately upstream from the dam, Borrow Area IA located in the north arm of the sediment pool upstream from Area I, and Borrow Area II located in the south arm of the sediment pool.

Borrow Area I was indicated as the primary source for the cutoff trench and central portion of the Zone I fill. Once the SC materials were depleted, the remainder of Zone I was to be derived from Borrow Areas IA and II. A small zone of clean gravel materials was encountered during the site investigation in a specified area in the west portion of Borrow Area II. The draft design memorandum recommended these clean gravel materials be used in the Zone III berm. It was also recommended that other clean sands and gravels, if encountered, be routed to Zone III.

Original Slope Stability Analysis. The designers evaluated the upstream slope for a rapid drawdown condition under the assumptions that a full phreatic line could develop up to the emergency spillway elevation, and that no dissipation of pore pressures would occur following drawdown. These assumptions resulted in a factor of safety < 1 for a 3H:1V upstream slope during drawdown. A 12 ft wide, "free-draining" berm was included in the design to achieve adequate factors of safety under the assumed drawdown loading condition.

Downstream slope stability initially was evaluated for an assumed steady seepage condition without the internal drain (phreatic line emerging on the downstream slope). Computed factors of safety were unacceptably low, even for slopes as flat as 3H:1V with this assumed steady seepage condition. Slope stability analysis for the downstream slope under the dry condition (with the drain) was not documented except to evaluate the infinite slope factor of safety = 1.35 for the assumed shear strength of $\varphi = 34°$, $c = 0$.

The design intent was for the upstream berm to be highly pervious and serve as a free-draining, stabilizing buttress to improve stability during drawdown. The specifications limited fines content in Zone III to 10% to accommodate materials from the available local borrow sources. This fines content may 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 the dam, and used this as primary justification for incorporating the upstream berm and an internal drain zone (Zone II). A more critical purpose for Zone II is actually as a filter to protect against internal erosion and piping. Supplemental geotechnical analyses were performed as part of this Phase I Structures Assessment to document the slope stability and filter compatibility based on current criteria and our understanding of the structure and zoning.

1.4 Land Use

Sunset 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

Sunset 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 Sunset 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 Sunset FRS.

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

- Adverse Consequences Resulting from Emergency Spillway Discharges During Major Rainfall Events (Category I).
- Failure From Overtopping Of Sunset FRS (Category II)
- Slope Failure on Left Abutment (Category II)

The potential failure modes range from a low likelihood of occurrence, high consequence to a high likelihood, low 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. The hydraulic capacity and operation of the Saint Anthony Falls outlet and emergency spillway be evaluated under a full freeboard discharge of 3,400 cfs.
2. A spillway inundation study should be conducted for Sunset FRS following District methods for such studies.
3. An updated dambreak analysis and inundation mapping should be prepared for Sunset FRS.

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

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

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

7. Monitoring at Left Abutment during impoundment. Kimley-Horn recommends monitoring for seepage through the left abutment during impoundment events, and regular visual inspection of the clay blanket that extends approximately 400 feet upstream from the left abutment to ensure it is being properly maintained.

8. 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.

9. Downstream Slope Stability Under Steady Seepage: The original minimum factor of safety that was computed for the dry downstream slope (1.35) does not achieve the minimum criteria of 1.5. However, the KHA team does not consider the infinite slope analysis that was done in the original design as representative of a "critical" failure scenario. Preliminary analysis evaluated more substantial failure surfaces which resulted in a minimum factor of safety of 1.7.

10. Upstream slope stability under steady seepage, partial pool: The original analysis evaluated upstream slope stability under steady seepage for the maximum pool elevation, resulting in a minimum factor of safety of 1.99. 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 that result in a lower factor of safety than the steady state condition under maximum pool.

11. 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 an analysis should be conducted.

12. Phase II Slope Stability and Seepage Analysis of Left Abutment. Additional seepage and slope stability analysis could be performed to evaluate the candidate failure mode associated with potential slope instability in the left abutment.

13. Develop geologic cross sections through the abutment, beyond the extent of the Zone I/Zone II fill. Sections should be drawn for both the upstream and downstream abutment slopes.

14. Conduct seepage analysis to estimate the range or extent of saturation and seepage through the abutment during impoundment events.

15. Phase II Additional Evaluation of Zone II Filter/Drain. These preliminary analyses indicate that the Zone II filter/drain does not strictly meet filtering requirements for the finest materials that may have been used in the Zone I fill. Additional analyses could be done to evaluate the effectiveness of the Zone II using a methodology developed from recent research at the University of New South Wales, Australia (Foster and Fell, 2001).

16. 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 Sunset 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 and includes the towns of Wickenburg and Congress.

2.1 Purpose of Dam

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

2.2 Dam Location and Features

Sunset FRS is located in the eastern portion of the Town of Wickenburg, less than a mile from the center of the Community. The Sunset FRS is located on Sunset Wash and is a tributary to the Hassayampa River in the northeast corner of Section 11, Township 7N, and Range 5W. **Figure 1** provides a location map of Sunset FRS. The project consists of the FRS embankment structure, principal spillway, and an emergency spillway.

The reservoir behind the FRS is 8 acres with a capacity of 55 acre-feet. A permanent pool will not be retained in the reservoir, instead, the FRS and reservoir are designed to trap floodwater and store it only for as long as it takes to release it slowly and safely downstream. The reservoir capacity is then restored to handle a future flood.

The emergency spillway is a reinforced concrete structure constructed on the top of the dam embankment. It consists of entrance control structure, a straight chute and a stilling basin with baffle blocks. Construction of the FRS and appurtenant structures was completed in April 1975.

2.3 Physical Features

The Sunset FRS is an earthfill structure with a central chimney drain/filter. The length of the FRS is 488 ft with a maximum height of 30.5 feet and crest width of 14 feet. The reservoir capacity is approximately 55 acre-feet at the emergency spillway crest elevation 2131 feet. Sunset FRS is located in the Town of Wickenburg, Arizona south of the intersection of U.S. 60 and east of Mariposa Drive. The maximum recorded

impoundment for Sunset FRS is 34 acre-feet with a stage of 12.27 feet at the FRS on September 26, 1997 (ADWR; 2004c).

Watershed
The dam was constructed across Sunset Wash, a tributary to Sols Wash and the Hassyampa River. The drainage area contributing to the dam is 0.6 square miles (AWC, 1979). The main wash in the watershed is well defined and contains a substantial amount of bed load material. The sediment yield rate was determined to be 0.135 acre-feet per square mile per year delivered to the reservoir. With a trap efficiency of 100 percent, the sediment storage requirement is 8.1 acre-feet for the 100-year life of the structure (SCS, 1974).

Flood Pool
The spillway crest was constructed at an elevation of 2131 feet in order to contain the 100-year flood. The as-built data show that an elevation of 2131 feet, the reservoir has a capacity of 55 ac-ft. The peak inflow into the reservoir during the 100-year flood and the PMF (correspondingly) were estimated to be 857 cfs (SCS, 1975b) and 7093 cfs (AWC, 1979). The design allowed for a reservoir capacity of 55 acre feet to the spillway.

Dam Embankment
The dam was designed as a homogeneous dam with a vertical drain zone just downstream of the centerline. Zone I, forming the bulk of the dam, was constructed of clayey sands and silty sands with some gravelly clayey sand. A 6-foot wide vertical drain extends from four feet above emergency spillway level to the top of the outlet conduit. The embankment was constructed with 3:1 (horizontal to vertical) upstream slope and a 2:1 downstream slope. A 12-foot wide berm of sandy, gravelly material was added to the upstream toe to increase stability. Six inch asbestos-cement pipes were placed at the bottom of the vertical drain running from each abutment to the outlet conduit, where they connect to six inch non-perforated outfall pipe.

Principal Outlet Works
The principal spillway consists of a reinforced concrete intake tower with an uncontrolled overflow at elevation 2131 feet. The design outflow from the principal spillway is controlled by a 9-in by 9-in orifice at elevation 2120 ft.

Flow into the pipe is controlled by a reinforced concrete intake tower with the screened intake approximately at spillway crest level, 2131 feet. Discharge through the conduit enters a manhole at the downstream toe of the dam from whence flow is conveyed via the Sunset/Sunnycove Pipeline to an outfall in the Hassayampa River, approximately 1.5 miles away. The outlet works is designed to empty the flood control pool, accumulated during a 100-year storm in less than 10-days. Under normal operating conditions, the outlet conduit will not flow full as it is control by an 18-inch slide gate on the intake tower which limits the flow until a flood greater than the 100-year storm occurs (AWC, 1979).

Emergency Spillway

The emergency spillway was designed to pass the PMF with 0.3 ft of freeboard. As-built data specify the emergency spillway will discharge 3907 cfs for the predicted PMF water surface elevation of 2141.2 feet (AWC, 1979). The emergency spillway is a reinforced concrete structure constructed on top of the dam embankment. The spillway is 40 feet wide with a discharge capacity of 3400 cfs at maximum water surface elevation of 2140.48 feet. The emergency spillway crest elevation is 2131 feet (SCS, 1975b).

Table 1 provides a summary of the physical data for Sunset FRS.

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 also 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 Sunset FRS.

3.1 Dam Design Criteria

Sunset 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 Sunset 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 Sunset FRS Inflow Design Flood (IDF) is the ½ probable maximum flood (PMF). The NRCS, in their hydrologic study of Sunset 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 Sunset 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 Sunset 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 Sunset 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 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 Sunset FRS will require a straight inlet reinforced concrete chute spillway because no adequate earth spillway could be located. The Sunset

principal spillway is designed to outlet to a 12-inch pipeline. 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".

The Watershed Work Plan was updated by the SCS in their Design Report for Sunset 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 Sunset 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]
75	0.33	0.6	4.20	6.50	2120.0	9-in × 9-in	2130.9

The computer routings for the Sunset FRS agreed well with that used in the Work Plan. In this routing the flood pool is emptied in less than 10 days.

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, and 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]
2131.0	2120.0	40	vert	6.37	14.95	3400	2140.48

A maximum top-of-dam elevation of 2140.5 ft could not be exceeded at the Sunset FRS site because of topographic limitations. Principal spillway discharge was included in the routing in order to meet the maximum top-of-dam elevation for this site.

The reinforced concrete chute spillway over the embankment of Sunset FRS was designed using the freeboard discharge (3,400 cfs) for the inlet, vertical curve, and channel sections. The Saint Anthony Falls outlet was proportioned for two-thirds of the freeboard discharge (2,300 cfs). Kimley-Horn could not ascertain from the Design Report or other Sunset FRS documentation the reasoning for proportioning the Saint Anthony Falls outlet to two-thirds the freeboard discharge. Kimley-Horn recommends that the hydraulic capacity and operation of the Saint Anthony Falls outlet and emergency spillway be evaluated under a full freeboard discharge of 3,400 cfs. (FHA, 1983)

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

Item	Elevation [Ft]	Area [Ac]	Sum Storage [Af]
Bottom of Pool	2110.5	0	0
Top of Sediment Pool	2120.0	2.1	8.1
Crest of Principal Spillway			
Low Stage	2120.0	2.1	8.1
High Stage	2131.0	8.6	55.0
Crest of Emergency Spillway	2131.0	8.6	55.0
Crest of Dam (w/o camber)	2140.5	12.3	160.0

The NRCS further updated the design hydraulics for Sunset FRS and Sunnycove FRS 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. No changes were made to the hydraulic design of Sunset FRS.

The principal spillways for both Sunset FRS and Sunnycove FRS discharge into a common downstream pipeline. This pipeline carries the combined flows to outfall to the Hassayampa River. Supplement No. 1 outlines the assumptions used in the hydraulic design of the pipeline, several of which are listed as follows:

- The available entrance head at the first manhole below the FRS has been limited to the invert elevation of the 6-inch diameter asbestos cement drain pipe outlets from the embankments. This is to prevent back saturation of the drain fill in the downstream to of the embankments.

Flood Retarding Structure	Water Surface Elevation [Ft]
Sunset	2104.4
Sunnycove	2124.6

- Manholes are provided at about 1000-foot intervals along the outlet pipeline for sediment traps and for alignment/grade changes.
- Manholes are non-pressurized at design flows as follows:

Location	Design Flow
Sunset to junction	8.7 cfs
Sunnycove to junction	9.1 cfs
Junction to outlet	17.8 cfs

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 0.3 feet of residual freeboard.

The Phase I study used a drainage area of 0.6 square miles which agrees with the SCS determination of contributing watershed area. The runoff curve number used by the Phase I study is higher (78) than that used by the SCS (75). The reservoir capacity of 53.6 acre-feet was used by both the Phase I study and the Work Plan. The Corps study estimated a maximum spillway discharge of 4,100 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]	-	11.5
Time of concentration [hr]	0.33	0.33
Peak Inflow [cfs]	3393	7093
Peak Inflow*** [csm]	5655	11822
Runoff [ac-ft]	384	394
Runoff [in]	12.0	12.3
Runoff Coefficient	78	78
Routed	Yes	Yes
Peak Outflow [cfs]	2381	3907
Peak Outflow [csm]	3968	6512
Maximum Water Surface Elevation [ft]	2138.3	2141.2
Residual Freeboard	3.2	0.3

* Precipitation time distribution per SCS 6-hour 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 Sunset Wash drainage area and Sunset 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 Sunset 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 740 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 2126.93 ft, which is approximately 14.6 ft below the emergency spillway elevation (2131.0 ft).

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 740 cfs with no outflow from the dam. The 100-year 6-hour inflow into the structure for the SCS study was 857 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 Sunset FRS was conceptually evaluated in-house at the District and that back-up documentation is very limited. **Figure 4 (Figures Appendix)** illustrates the emergency spillway inundation area.

Kimley-Horn recommends that a spillway inundation study be conducted for Sunset 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 Sunset FRS and documented the analysis in their report titled Dambreak Analysis of Sunset 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]	10
Final breach width [ft]	84
Initial Water Surface Elevation [ft]	2131.0 {emergency spillway crest}
Final Breach Elevation [ft]	2113.5
Volume of Reservoir [af]	53.6
Breach Outflow [cfs]	8,900
PMF Outflow [cfs]	7,100 (Phase I Report)

The dam breach parameters do not appear to be reasonable. Using the District breach parameters the reservoir would drain in approximately 4.3 minutes. The FMEA work session for Sunset FRS estimated that a reasonable time to failure would be on the order of 30 minutes rather than 10 minutes. 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 1,000 to 5,000 acre-feet. The derivation of the breach parameters used in the District study is not documented in the report.

The EAP for Sunset FRS provides a dambreak inundation exhibit. The exhibit was developed showing dambreak inundation limits for Sunset FRS concurrent with the inundation limits from Sunnycove 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 Sunset 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 Sunset FRS should be prepared without indicating the inundation limits for Sunnycove FRS.

3.3.4. Sedimentation – The Watershed Work Plan summarizes the sedimentation investigation conducted for Sunset 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 are 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 were estimated at 8.1 acre-feet for the Sunset FRS. The sediment yield rate was determined to be 0.135 acre-feet per square mile per year or 0.081 acre-feet per year delivered to the reservoir. The NRCS used 100 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 Sunset FRS. Based on this observation, no further evaluation of sediment yield is required for Sunset 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 Sunset 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. Sunset 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' 55" N and 112° 44' 30" 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 Sunset 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).

Sunset 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 Sunset 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 Sunset 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 Sunset 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 Sunset FRS site.

According to Staedicke (1995) because there is no history of extensive groundwater pumping or subsidence, the NRCS has never surveyed the Sunset FRS structure. Although land subsidence is not expected to affect Sunset 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 Sunset FRS project area. Geological conditions in the Sunset FRS area preclude the development of earth fissures at this site.

3.4.5. Foundation Conditions. The geologic report (SCS, 1974) described the foundation soils at the site as Tertiary alluvial deposits of dense silty, gravelly sand which are cemented to various degrees. Geologists refer to this older, cemented alluvium as fanglomerate. The fanglomerate is overlain by a thin mantle of recent alluvium in the main channel floodplain area. The low density, recent alluvial materials were removed to depths up to 12 feet from the dam foundation during construction. The dam is reportedly founded on competent fanglomerate bedrock in the main channel and right abutment sections.

The mid to upper left abutment consists of variably cemented, stratified alluvial materials and manmade fill and debris. This area was highlighted by the geologic and design reports as presenting a potential seepage problem (SCS 1974, 1975a, 1975b). The natural geologic materials in the left abutment are described as stratified deposits of gravelly, fine to coarse-grained sand, with variable amounts of silt (generally 10 to 20% silt). The left abutment native soils contain zones with little to no cementation and some clean sand lenses. Fill materials found in the left abutment were described as a loose mixture of sand, silt and gravel containing broken concrete blocks and wood debris. The geologic investigation concluded that the fill had been loose dumped into in a gully just upstream from the originally proposed dam alignment. The dam alignment was shifted about 25 feet downstream to avoid the loose fill and debris, and the original plans indicated that the loose fill materials were to be completely removed to "firm residual soil" as directed by the engineer, and backfilled with Zone 1 compacted fill to restore grade to the original

ground surface. The extent of over-excavation and replacement of the waste fill was anticipated to be approximately 250 feet in the upstream direction, and "as directed by Engineer" in the downstream direction. However, the extent of removal of the loose fill laterally (northward) into the abutment was constrained by the presence of an existing building at the top of the slope and it is not known for certain that "firm residual soils" were reached all along the left abutment contact

3.4.6. Embankment Materials. The SCS designers concluded that selective borrowing to construct a zoned embankment would be very difficult because of the stratified nature of the alluvial deposits in the borrow areas. Based on that assessment, the embankment was designed as a homogeneous section, with a vertical chimney drain zone. An upstream berm was incorporated in the design to improve stability during rapid drawdown. A typical cross section of the embankment is shown as Appendix H.

Laboratory testing of representative borrow soils was reported in the geologic report (SCS, 1974). The data were compiled for this Phase I assessment, and are summarized on Table 2 of Appendix H. The primary source for borrow materials was the recent alluvial deposits located within the sediment pool and dam foundation. The geologic report described these deposits as inter-layered, inter-fingering, lenticular layers of predominantly silty sands (SM) and slightly silty, well-graded sands (SW-SM), with minor quantities of slightly clayey poorly graded sands (SP-SC). Three borrow areas were identified as follows: Borrow Area I located in the north arm of the sediment pool immediately upstream from the dam, Borrow Area IA located in the north arm of the sediment pool upstream from Area I, and Borrow Area II located in the south arm of the sediment pool.

Borrow Area I was indicated as the primary source for the cutoff trench and central portion of the Zone I fill. Once the SC materials were depleted, the remainder of Zone I was to be derived from Borrow Areas IA and II. A small zone of clean gravel materials was encountered during the site investigation in a specified area in the west portion of Borrow Area II. The draft design memorandum recommended these clean gravel materials be used in the Zone III berm. It was also recommended that other clean sands and gravels, if encountered, be routed to Zone III.

3.4.7. Original Slope Stability Analysis. Based in part on the laboratory tests as summarized on Table 2 of Appendix H (developed from data sheets attached with the geologic report, SCS, 1974), the designers assumed the parameters shown on Table 3 of Appendix H for the slope stability analyses. Slope stability analysis results were reported for the loading conditions shown on Table 4 of Appendix H.

The designers evaluated the upstream slope for a rapid drawdown condition under the assumptions that a full phreatic line could develop up to the emergency spillway elevation, and that no dissipation of pore pressures would occur following drawdown. These assumptions resulted in a factor of safety < 1 for a 3H:1V upstream slope during drawdown. A 12 ft wide, "free-draining" berm was included in the design to achieve adequate factors of safety under the assumed drawdown loading condition.

Downstream slope stability initially was evaluated for an assumed steady seepage condition without the internal drain (phreatic line emerging on the downstream slope). Computed factors of safety were unacceptably low, even for slopes as flat as 3H:1V with this assumed steady seepage condition. Slope stability analysis for the downstream slope under the dry condition (with the drain) was not documented except to evaluate the infinite slope factor of safety = 1.35 for the assumed shear strength of $\varphi = 34°$, $c = 0$.

The design intent was for the upstream berm to be highly pervious and serve as a free-draining, stabilizing buttress to improve stability during drawdown. The Specifications limited fines content in Zone III to 10% to accommodate materials from the available local borrow sources. This fines content may 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 the dam, and used this as primary justification for incorporating the upstream berm and an internal drain zone (Zone II). A more critical purpose for Zone II is actually as a filter to protect against internal erosion and piping. Supplemental geotechnical analyses were performed as part of this Phase I Structures Assessment to 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.

3.4.8. Supplemental Geotechnical Analysis

3.4.8.1 Supplemental Seepage and Slope Stability Analysis In support of the Phase I Structures Assessment, Gannett Fleming conducted preliminary supplemental seepage and slope stability analysis for Sunset FRS to document the expected stability of the structure under anticipated loading conditions. The assumptions used in the original stability analysis by SCS 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 the dam because (1) the embankment was likely to be highly stratified and have high horizontal permeability, (2) maximum release rate through the orifice-controlled low-stage inlet to the outlet works would be slow, and the inlet is vulnerable to plugging, and (3) the sediment pool drain is gate controlled and there could be an uncertain time delay in releasing impounded water from the sediment pool level below the orifice inlet.

- Revised Assumption No. 1: Development of a high-level steady state phreatic line is not likely because (1) the stratified embankment will have high horizontal permeability and will therefore drain quickly, and (2) 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 high-level steady state seepage line to develop. (3) Failure to release the sediment pool through manual operation of the gated outlet is a possibility. Gannett Fleming conducted

supplemental slope stability analyses assuming that water remains in the sediment pool for extended time periods because the low level gate remains closed.

- Assumption No. 2: Zone III is free-draining - Rapid drawdown stability was achieved for the steady-state phreatic line pore pressure assumption by incorporating a "free-draining" Zone III berm.

- Revised Assumption No. 2: Zone III is not free-draining. The Zone III materials, contain up to 10% 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, and pore pressure dissipation will occur during normal drawdown rates (see following seepage analysis), rapid drawdown is not anticipated to cause slope instability, even if the berm is not essentially free draining.

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 temporary impoundment event. A standard "Silty Sand" material type was selected from the model's database to represent the Sunset FRS embankment materials. The database provides the necessary unsaturated hydraulic parameters for use in the simulation. The vertical saturated hydraulic conductivity was assumed to be 0.3 ft/day (1×10^{-4} cm/s). This was the value used by the SCS designers for the filter/drain design, based on laboratory tests on representative sample 2102.2, as reported in the draft design documents (SCS, 1975a). The embankment was modeled as a homogeneous section, with a horizontal:vertical anisotropy (kh/kv) ratio of 10:1 for the hydraulic conductivity.

Figure 2 in 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). It was assumed that the low level outlet would remain closed, therefore the drawdown between events and following the two floods was limited to elevation 2120, at the top of the berm. This impoundment scenario was modeled to estimate a conservative phreatic line for use in evaluating slope stability during drawdown. It is evident that even following multiple storm events, the wetting front will advance to a very limited extent into the dam. Also, the model results indicate rapid dissipation of the upstream pore pressures as the pool level drops.

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 in Appendix H

show the estimated minimum factors of safety for the upstream slope above the berm at two times: (1) during drawdown after the 2nd flood impoundment (factor of safety = 2.4), and (2) after drawdown to the sediment pool level immediately following two consecutive impoundment events (factor of safety = 2.3). The factor of safety is slightly higher at the intermediate impoundment stage (during drawdown) because the pool provides additional buttressing against the slope. Note that the slope is predicted to be completely drained (low phreatic line) immediately following the two events, based on the assumptions used in the model for impoundment times, drawdown times, and hydraulic conductivity of the materials.

Slope stability for the downstream slope was also re-evaluated in order to document a factor of safety for a more meaningful failure mode than the shallow, infinite slope analysis that was completed during original design. The factor of safety shown (1.7) 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 in construction.

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

3.4.8.2. 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 positioned downstream from the dam crest. 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 in the event of transverse crack development.

Because of its critical function as a filter, the Zone II gradation was checked against current filter criteria in accordance with the NRCS, National Engineering Handbook, Chapter 26 "Gradation Design of Sand and Gravel Filters" (NRCS, 1994). Figure 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 was developed for Field Sample 2106.1 data from the geologic report (SCS, 1974). The sample was taken from Borrow Area #1, and is described as a "light brown, calcareous, gravelly silty sand, that classifies as SM according to the Unified Soil Classification system (USCS). A second base soil gradation curve is also shown (graphed with solid blue circles) to represent a more average gradation of the Zone 1. The "average" base soil gradation (blue circles) was developed from sample 2102.2, derived from Borrow Area #2, which is described as "reddish brown, calcareous, well graded silty, gravelly sand".

The base soil gradation curves (solid symbols) were adjusted for gravel content as shown by the curves graphed with open red triangular symbols (for sample 2106.1) and open blue circles (sample 2102.2). The filtering and permeability (k) criteria for the adjusted curves are shown by the solid circles and triangles on the 15% passing line. The coarse side of the Zone II specification band is too coarse to achieve the recommended filtering

limit for the finest base soils. However, Zone II does meet both filtration and permeability criteria for the "average" base soil gradation. Thus it is possible that some fines from Zone I could penetrate into Zone II under a concentrated leak through a transverse crack, if the Zone II materials were graded on the coarse band in accordance with the specified gradation limits. Considering the variability in gradation of the Zone I materials, and the fact that Zone II meets the criteria except for the finest base soil and coarsest filter possibilities, it is likely that Zone II is providing adequate filter protection. Additional analyses may be done to further evaluate the efficacy of the Zone II filter, as outlined in **Section 8** of this report.

3.5 Construction History

The January 1979 Phase I Corps of Engineers report provided the following synopsis of construction history for Sunset 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 3, 1976 which was the beginning of the foundation preparation. The only major change from the approved plans was the addition of a buttress fill along the left bank of the reservoir extending from the dam to 600 feet. Treatment of this area consisted of a cutoff trench along the abutment toe in an upstream direction that was filled with the most plastic embankment material. The same material was used to blanket the abutment in order to prevent seepage around the dam. Quality control and construction supervision for the project were done by Engineers Testing Laboratory under a contract with the Soil Conservation 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 Sunset 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 Sunset 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 Sunset FRS is displayed on the same figures as shown for Sunnycove 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 Sunset should be shown independent of Sunnycove 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. Sunset 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 9.9 feet. In addition, McDEM would be notified at an impoundment depth of 10.0 feet.

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 for notification of McDEM.

	Emergency Operations Plan	**FERM (January 2002)**
	Pool Level [ft]	**Pool Level [ft]**
District Alarm	-	6.4
Notify FCD O&M	-	9.9
Notify McDEM	10	13.4

4.0 PRELIMINARY FAILURE MODES

Kimley-Horn and Associates, Inc. (KHA) facilitated a Preliminary Failure Modes Identification workshop for Sunset 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. **Failure of Principal Outlet**: The principal outlet for the dam is a reinforced concrete pipe 30 inches in diameter.

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

5. **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).

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

7. **Failure Mechanisms Associated with Presence of Collapsible Soils in Dam Foundation:** This failure mode relates to the potential for collapse on saturation

of meta-stable soils in the dam foundation. Geologic mapping/boring logs/laboratory test data will be reviewed to assess to the extent practical the presence of potentially collapsible materials.

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

9. **Failure Mechanisms Associated with 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.

10. **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. Qualitatively assess the impact of discharge from the emergency spillway on the downstream areas.
    b. Qualitatively assess the impact of groundwater withdrawal in the vicinity of the dam.
    c. No water stops in horizontal joints of spillway chute. Reinforcing steel isn't continuous thru the floor slabs (movement of one slab either horizontal or vertical plus open joint could lead to negative pressures at joint and loss of foundation material).

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 Sunset FRS. Land use information for Sunset 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 of the dam and 500 ft upstream of the dam. Land designated as medium lot residential surrounds the south side 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 2,000 ft downstream of the dam and directly adjacent to the left abutment.
- *Public Facilities* include community centers, power sub-stations, libraries, city halls, police/fire stations, and other government facilities). There are several areas designated as public facilities located within 500 ft downstream and upstream 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 just north of Sunset FRS and runs upstream and downstream of the dam.

- Residential land surrounds the dam and is located directly downstream of the dam and within 500 ft upstream of the dam.
- There are public facilities located within 500 feet upstream of Sunset FRS and within 1500 feet down stream of the dam.
- 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 12,000 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 Sunset FRS from project files at the Flood Control District and Arizona Department of Water Resources. The reports collected from these sources date to April 14, 1980. A total of 25 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:

- Inspection reports dated July 1980, Feb 2, 1985, April 8, 1985 indicate Ford Dealership drained oil into the impoundment area.
- There is settlement of the emergency spillway wing walls, causing the walls to slightly bow.
- Approximately 2700 cubic yards of sediment was removed in 1985
- Emergency spillway joint displacement monitored by FCD staff from 1997 to 2003
- In 2003 it was noted that there are scattered rills throughout the upstream and downstream slopes.
- Significant impoundment events have occurred in 1996, 1995, 1994, 1993, 1992, and 1989. The highest impoundment of record was 14.2 ft in 1989
- The principal spillway conduit was videotaped in 2003

Summary of Dam Inspection Reports

Date	ADWR Inspector (s)	District Personnel	Other Personnel	Principal Findings
14-Apr-80		Mark Williams		Erosion Exists on the slopes of the main embankment, near the toes of the structure
30-Jul		Mark Williams Bob Pendergast		Ford Dealership is draining oil from their shop area into the impoundment area.
				Settlement of the emergency spillway wing walls, both upstream and downstream walls
				Severe erosion occurred to the slopes of the structure during the 1st heavy rain. The eroded areas should be filled with top soil from impoundment area.
2-Feb-81		Bob Pendergast		Measurements taken of the monitoring pins to observe settlement upstream and downstream.
				Ford Dealership is draining oil from their shop area into the impoundment area.
2-Jun-81		Bob Pendergast		Regular maintenance, painted and cleaning needed for access gates, pad locks, and surrounding fences
30-Nov-81		Bob Pendergast		Need hand grading and repair of erosion on both slopes of the FRS
25-Jan-82		Bob Pendergast		Erosion is occurring on the downstream slope at access gate to chain link fenced area
26-Apr-82		Bob Pendergast		Silt accumulation in the impoundment area needs to be removed.
				Stilling block has a considerable spalled area
12-Jul-82		Bob Pendergast		No maintenance is needed at this time with the exception of minor cleanup of other people's debris in the impoundment area
19-Jan-83		Bob Pendergast		Hand grade existing materials on slopes of the FRS to repair erosion.
				Hand place rock in the deeper erosion at north abutment downstream slope
				PM gated outlet works

Summary of Dam Inspection Reports

Date	ADWR Inspector (s)	District Personnel	Other Personnel	Principal Findings
19-Dec-83		Bob Pendergast		Plastic Taylor Rain Gage should be changed out
				Growth of small Mesquites need to be removed from upstream toe of FRS
				Sunset Sunny cove pipeline inspected
3-Apr-84		Bob Pendergast		A MCHD grader is scheduled to maintain and grade the impoundment area the week of April 16th-19th, the dormant grasses will be turned
				Install additional pipe beneath existing double access gated at downstream toe of FRS to prevent people from gaining access to chain link fenced area.
8-Apr-85		Bob Pendergast		Erosion exists on both slopes.
				Sediment needs to be removed from the stilling area and floor of emergency spillway
				Jones Ford continues to dump waste oil into impoundment and inflow wash area.
				Circle J-R Motel dumped 10cy of swimming pool plaster and scrap rebar debris over their fence and onto FCD acquired property.
31-Jan-85		Mark Williams		Principal Outlet valve checked, was operated, lubricated and free of debris.
Aug-85				FCD remove approximately 2700 cy of sediment from the impoundment area.
16-Oct-85				Principal Outlet valve checked, was operated, lubricated and free of debris.
15-Jun-88		Paul DiPierro Fred Fuller	Jon Hall- SCS Carmella Apodaca-SCS	Impoundment on 10/29/87 11.75 ft
				Impoundment on 11/01/87 of 13.75 ft
				Impoundment on 01/18/88 of 10.75 ft
				The crest of the dam is in good condition and the access road along the crest is graded and free of vegetation
				The downstream slope is in good condition and has been seeded and mulched since the last inspection.
				The upstream slope is in good condition and has been seeded since the last inspection

Summary of Dam Inspection Reports

Date	ADWR Inspector (s)	District Personnel	Other Personnel	Principal Findings
14-Jun-89	Ken Hussain	Ellery Biathrow Paul DiPierro	Jack Elder-SCS	Impoundment on 08/26/88 of 11 ft Impoundment on 01/04/89 of 14.2ft
		Fred Fuller		Seeding and mulching operation conducted on both slopes the past year does not appear to have been successful, mainly due to insufficient winter rainfall.
24-Jun-92		Mike Meng Bob Panasewicz		The crest of the dam is in good condition and the access road along the crest is graded and free of vegetation The downstream slope is in good condition and has been seeded and mulched since the last inspection. The upstream slope is in good condition and has been seeded since the last inspection
23-Jun-93	Ken Hussain	Ernie Hamer		Impoundment on 08/22/92 of 9ft Previous movement at expansion joints of wing walls need to be continuously checked and monitored. Dam and appurtenance are in good condition
23-Jun-94		Ernie Hamer Bob Panasewicz	John Harrington, SCS Rob Genualdi, SCS	Impoundment on 10/6/93 of 6.6ft Everything was in good condition, need to continue to monitor expansion joints of wing walls
27-Sep-95		Chuck Smith Ernie Hamer	John Harrington, NRCS (SCS) Rob Genualdi. NRCS (SCS)	Everything was in good condition, need to continue to monitor expansion joints of wing walls
2-Oct-96	Ken Hussain	Chuck Smith Ernie Hamer	Steve Smerik NRCS Richard Hansen, NRCS	Filled in voids at the wing wall and invert expansion joints with a pliable sealer. Concern about the lack of weep holes in the concrete wall of the spillway.

Summary of Dam Inspection Reports

Date	ADWR Inspector (s)	District Personnel	Other Personnel	Principal Findings
27-Oct-97	Ken Hussain	Chuck Smith Ernie Hamer		Everything was in good condition, need to continue to monitor expansion joints of wing walls
6-Nov-02		Michael Greenslade		Scattered rills throughout
				Schedule gravel mulch application on upstream and downstream slopes
				Video to inspect Principal Spillway Conduit scheduled FY 2002-2003
				Emergency Spillway shrinkage and/or temperature cracks, no structural cracks, no repairs required.
3-Nov-03		Larry Lambert	Noller Herbert - NRCS	Scattered rills throughout
				Completed gravel mulch application on upstream and downstream slopes
				Video taped the principal spillway conduit in 2003. Inspected the first manhole (cleanout) downstream of dam. It was clear with no debris present.
				Emergency Spillway shrinkage and/or temperature cracks, no structural cracks, no repairs required.
				Some joint deterioration but no repairs required at this time, need to monitor.

6.2 Field Inspection for Structure Assessment

As part of the Phase I Assessment for Sunset FRS, a visual inspection of Sunset 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 components of the dam 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 placed on upstream and downstream slopes in 2003.
- Significant animal burrows around base of prickly pear cacti. Gravel mulch does not surround the base of the prickly pears.
- From Station 12+50 to the left abutment several "patches" of finer materials were noticed near the top of the crest on the upstream slope (similar to those observed at Sunnycove). The patches were placed approximately 10 to 30 feet apart and 4 to 5 feet below crest.
- Scouring and erosion observed at the end of the grouted rip rap on left abutment.
- Wing walls at left abutment of the emergency spillway showed ½ inch off-set at the upstream and downstream sloping portion. The offset increased at the top of the wing walls.
- Energy dissipaters in emergency spillway in good condition.
- The emergency spillway showed no concrete distress, spalling or other signs of degradation.
- Very minor shrinkage cracks were observed on the emergency spillway
- No distress in downstream slope.

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 Sunset FRS and its appurtenant facilities.

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

The workshop was conducted on March 1, 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

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 were 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 Sunset 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:

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

Failure Mode Description: The Sunset FRS emergency spillway is a 40 foot wide reinforced concrete spillway located on the embankment about one third of the length of the crest away from the right abutment. Normal flood discharges from the spillway are directed into a residential area 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 under major flood events would 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 downstream housing development.

Failure From Overtopping Of Sunset FRS (Category II – Considered but not highlighted). (May move to Category IV – not credible).

Failure Mode Description: Overtopping of Sunset FRS would occur at the low point of the dam crest which from the 2003 crest monument survey indicates to be located on the

left side of the dam embankment. Flow would overtop the dam at that point and, as flows increased, erosion of the crest and downstream slope potentially would lead to an eventual breach of the dam. The top of dam crest elevation based on as-built plans is 2141.5 ft (NGVD29). The 2003 crest monument survey indicates the low spot on the left side of the dam to be 2140.0 feet. The maximum PMF (HMR-49) routing water surface elevation is 2141.2 feet based on using as-built data for dam.

Slope Failure on Left Abutment (Category II – considered but not highlighted).

Background on conditions leading to the potential failure mode: According to the geologic report, the mid-to-upper left abutment contained loose waste fill of undetermined extent. The original plans indicated that the loose fill materials were to be completely removed to "firm residual soil" as directed by the engineer, and backfilled with Zone 1 compacted fill to restore grade to the original ground surface. The extent of over-excavation and replacement of the waste fill was anticipated to be approximately 250 feet in the upstream direction, and "as directed by Engineer" in the downstream direction. However, the extent of removal of the loose fill laterally (northward) into the abutment was constrained by the presence of an existing building at the top of the slope and it is not known for certain that "firm residual soils" were reached all along the left abutment contact. It is known that, to isolate and protect the abutment, the cutoff trench was wrapped upstream along the reservoir edge as a "dog-leg" extension about 400 ft upstream (westward) from the left abutment. The area over the upstream cutoff trench extension was also blanketed with clay to minimize seepage through the abutment. The clay blanketing extended from the cutoff trench elevation to the top of the embankment fill. Also to protect the left abutment from seepage the Zone II drain was brought up to elevation 2135 feet between Station 15+00 and 15+37 within the main dam section at the abutment contact to provide additional protection at higher elevations in this area. The top of Zone II is at elevation 2127 along the remainder of the embankment.

Failure Mode Description: This failure mode considers possible slope instability in the left abutment due to the possibility of the presence of weak, loose fill materials that could not be excavated, and possible low shear strength of some of the residual materials in that abutment. For this failure mode to develop during a flood event (considering the time it would take for seepage water to ingress within the embankment) requires precursor events or conditions that would pre-moisten and soften of the postulated weak abutment materials during sequential impoundment events, or by accumulated infiltration from runoff sources associated with the structures on the slope above. The risk of saturation of the mid-to-upper abutment during sequential impoundment events would be higher if the protecting clay blanket close to the axis of the FRS is ineffective due to erosion or disturbance by shallow slope instability, significant animal burrowing activity, or inadequate placement thickness in its original construction. The Potential failure mode initiates as aslope failure in the abutment during a significant flood event leading to potential loss of a portion of the dam crest at the abutment contact, followed by accelerated seepage through the shortened seepage path, continuing erosion of the crest and slumped material, and as flows increased, eventual erosion of the crest and

downstream slope leading to concentrated overtopping at the abutment contact and eventual breach of the dam.

7.4 FMEA Limitations

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

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. The hydraulic capacity and operation of the Saint Anthony Falls outlet and emergency spillway should be evaluated under a full freeboard discharge of 3,400 cfs.

2. A spillway inundation study should be conducted for Sunset 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. An updated dambreak analysis and inundation mapping should be prepared for Sunset 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 Sunset FRS should be prepared without indicating the inundation limits for Sunnycove FRS.

4. 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 number of EAP flowchart percent levels/actions be reduced and consolidated given consideration for the time to fill the impoundment during large event storms.

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

6. 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 at Left Abutment during impoundment. In recognition of the presence of potentially weak, loose fill, and un-cemented native materials in the

upper left abutment, Kimley-Horn recommends monitoring for seepage through the left abutment during impoundment events, and regular visual inspection of the clay blanket that extends approximately 400 feet upstream from the left abutment to ensure it is being properly maintained.

2. Supplemental Geotechnical Analyses

2.1 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 Sunset 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 of 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 Gannett's preliminary (Phase I) stability analyses do not document factors of safety for all the loading conditions that would need to be evaluated under current NRCS or ADWR criteria. Table 7 in 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 temporary impoundment events, and to assess the upstream slope stability under normal drawdown rates (with the lower intake functioning at capacity). These analyses show that it is very unlikely that a steady state phreatic line would develop in the Sunset 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. In the original analysis, rapid drawdown was evaluated assuming a fully developed phreatic line from the normal high reservoir pool elevation due to a clogged outlet, followed by instantaneous drawdown (magical instant removal of the reservoir). In the original design analysis, the upstream Zone III berm was assumed to be "free-draining" providing a

high strength, fully-drained buttress under the rapid drawdown loading. It is likely that the upstream berm is not free-draining, and a more realistic, but still conservative rapid drawdown analysis would involve the following steps:

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. Realistic hydraulic conductivities can be used for Zones I and III, rather than assuming Zone I is impervious and Zone III completely drained as was done in the original drawdown analysis. 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 transient 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 original minimum factor of safety that was computed for the dry downstream slope (1.35) does not achieve the minimum criteria of 1.5 (see Table 6 of Appendix H). However, the Kimley-Horn team 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 1.7. No additional analyses for the downstream slope are considered necessary.

3. Upstream slope stability under steady seepage, partial pool: The original analysis evaluated upstream slope stability under steady seepage for the maximum pool elevation, resulting in a minimum factor of safety of 1.99. 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 that result in a lower factor of safety 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. 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:

d. 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 = 6% of the PGA, or 0.06.
e. Conduct the pseudo-static analysis on the downstream stability section with the lowest static factor of safety, and report the result.

2.2. Phase II Slope Stability and Seepage Analysis of Left Abutment

Additional seepage and slope stability analysis could be performed to evaluate the candidate failure mode associated with potential slope instability in the left abutment. The analysis is outlined as follows:

2.2.1 Develop geologic cross sections through the abutment, beyond the extent of the Zone I/Zone II fill. Sections should be drawn for both the upstream and downstream abutment slopes. Use boring log and laboratory test data from the geologic report (SCS, 1974), and the as-built construction plans to estimate the subsurface slope stratigraphy and geometry as accurately as possible. Assign material parameters (hydraulic conductivity and shear strength) for the various layers in the slope using available information and judgment.

2.2.2 Conduct seepage analysis to estimate the range or extent of saturation and seepage through the abutment during impoundment events.

a. CASE 1 – Establish a "worst case" seepage line through the abutment. A conservative analysis could assume that the outlet intake is clogged, allowing a steady-state seepage line to develop around the left end of the dam. Also, it could be assumed that the clay blanketing layer is very thin, or has been damaged, allowing reservoir seepage to quickly penetrate into coarse, loose layers of debris fill and highly stratified alluvial deposits in the abutment. Further, it could be assumed that the slope has been "pre-saturated" by vertical infiltration from the building and parking area at the top of the slope. In this worst case scenario, it is likely that seepage would emerge on the downstream left groin of the dam. The critical slope analysis for CASE 1 would be the downstream abutment slope.
b. CASE 2 – Estimate a more likely extent of saturation and pore pressure development in the abutment slope by running a seepage analysis with the clay blanketing intact, and assuming limited detention time in the reservoir following an impoundment event or events. In this case the seepage line would likely only partially penetrate the upstream slope and may not emerge on the downstream slope. The critical slope stability analysis for this case would be the upstream abutment slope under drawdown conditions.

2.2.3 Conduct slope stability analysis for CASE 1 and CASE 2 pore pressure conditions and report the factors of safety for the worst case and expected case conditions.

2.2.4 Evaluate whether the factors of safety against this failure mode are acceptable, and, if not, recommend remedial actions that could be taken to improve the conditions in the left abutment of the dam.

2.3 **Phase II Additional Evaluation of Zone II Filter/Drain.**

The Phase I evaluation of the Zone II as a protective filter is described in Section 2.2 of this memo. These preliminary analyses indicate that the Zone II filter/drain does not strictly meet filtering requirements for the finest materials that may have been used in the Zone I fill. Additional analyses could be done to evaluate the effectiveness of the Zone II using a methodology developed from recent research at the University of New South

Wales, Australia (Foster and Fell, 2001). The procedure outlined by Foster and Fell (2001) is a method for assessing the gradation of filters in dams that may not meet the criteria in the strict sense, but could provide a degree of protection. The methodology can be used to determine whether filters that are too coarse according to modern criteria are sufficiently fine to eventually seal, or are anticipated to allow continuous erosion that could result in piping failure.

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

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.

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AWC; 1979. "Sunset Flood Retarding – Arizona Dam No. 7-49, 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.

FHA; 1983. "Hydraulic Design of Energy Dissipaters for Culvert and Channels," HEC No. 14, US Department of Transportation Federal Highway Administration Federal Highway Administration, Sept 1983.

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Foster, M. and Fell, R. (2001). "Assessing embankment dam filters that do not satisfy design criteria," *J. Geotech. and Geoenvironmental Eng.*, ASCE, **127**(5), 398-407.

SCS; 1970. "Wickenburg WPP Arizona Sunset FRS, Sunnycove FRS, and Pipeline Hydrology," U.S. Dept. Agriculture, Soil Conservation Service, Jan 1970.

SCS; 1974. "Report of Geologic Investigation, Wickenburg Watershed, Sunset Wash Floodwater Retarding Structure," U.S. Dept. Agriculture, Soil Conservation Service, Dec.

SCS; 1975a. Supplemental package containing draft text for the Design Report, as well as documentation of the original stability analysis and drain design.

SCS; 1975b. "Design Report, Sunset and Sunnycove Floodwater Retarding Structures, Maricopa and Yavapai County, Arizona," U.S. Dept. Agriculture, Soil Conservation Service, West Technical Service Center, Engineering and Watershed Planning Unit, Portland, Oregon, Apr. 10.

Table 1. Sunset FRS
Physical Data

	As-Designed Sunset FRS and Sunnycove FRS Design Report, 1975		Sunset FRS Phase I Inspection Report, 1978		As-Builts Plans Sunnycove FRS 1975	
Item		Reference Page		Reference Page		Reference Sheet
Inflow Design Flood						
PMF						
Watershed Characteristics						
Watershed Area (acres) (square miles)			384 (0.60)	1 (Flood Estimate)	384 (0.60)	24
Maximum Elevation (ft MSL)			2420	1 (Flood Estimate)		
Minimum Elevation (ft MSL)			2116	1 (Flood Estimate)		
Reservoir or Flood Pool						
Peak Inflow Flood During 100 year, cfs	857	*See note below				
Peak Inflow Flood During PMF, cfs			7093	2 (Flood Estimate)		
Storage at Emergency Spillway Crest (ac-ft)	55	6	53.6	1	55	24
100-yr Sediment Accumulation (ac-ft)	8.1	6				
Reservoir Storage at PMF Water Level (ac-ft)			192	Structure Data Sheet by W Jenkins		
Main Embankment						
Type						
Length (ft)			488	1	470	3R
Maximum Height above Stream bed (ft)			30.5	1	30.5	24
Crest Width (ft)			14	1	14	4R
Crest Elevation (ft MSL)	2140.5	6	2141.5	1 (Information Sheet)	2141.5	4R
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	18-in Slide Gate	1	18-in Slide Gate	3	18-in Slide Gate	11
Type	RCP	1	RCP	10	RCP	12
Length (ft)					192	4R
Diameter (in).	30	1	30	10	30	12
Maximum Discharge (cfs)	8.7	4	8.7	1 (Information Sheet)		
Principal Spillway Crest (ft MSL)					2131	4R
Emergency Spillway						
Type	Concrete lined Chute	6	Concrete lined Chute	1 (Information Sheet)	Concrete lined Chute	13r
Approach Channel Length (ft)	N/A		N/A		N/A	
Crest Length (FT)	40	5	40	1 (Information Sheet)	40	13R
Crest Elevation (ft)	2131	5	2131	1 (Information Sheet)	2131	4R
PMF- Maximum Water Surface Elevation (ft MSL)	2140.48	6	2141.2	2 (Flood Estimate)		
PMF- Peak Outflow	3400	5	3907	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	
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	(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	
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	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 normally be empty.	

Item	NRCS Original Design Criteria	NRCS Current Criteria	ADWR Criteria Eff June 12, 2000	Comment/Remarks
Emergency Spillway Capacity	(a) Pass the emergency spillway hydrograph resulting from P100 at the safe velocity (b) Pass the freeboard hydrograph with the water surface elevation at or below the design top of the dam (c) Capacity must not be less than that determined from Figure F-1 on Page F-3 in EM-27	(a) Pass the emergency spillway hydrograph resulting from P100 at the safe velocity (b) Pass the freeboard hydrograph with the water surface elevation at or below the design top of the dam (c) Capacity must not be less than that determined from Figure 7-1 on Page 7-8 in TR-60	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.
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,	(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	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,
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.	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	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

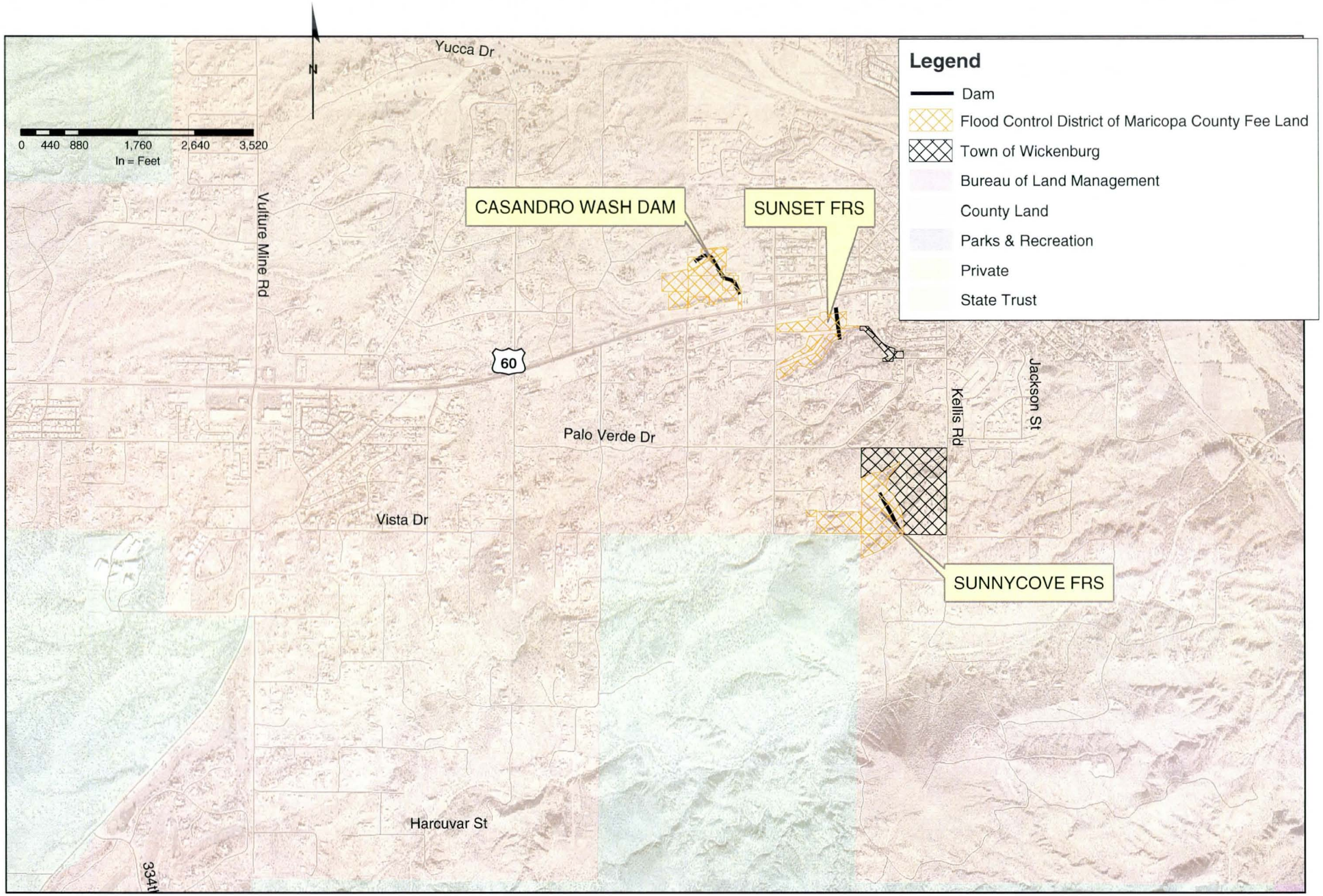

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FIGURE 2: LANDOWNERSHIP

FCD2003C015
PCN 050.36.31

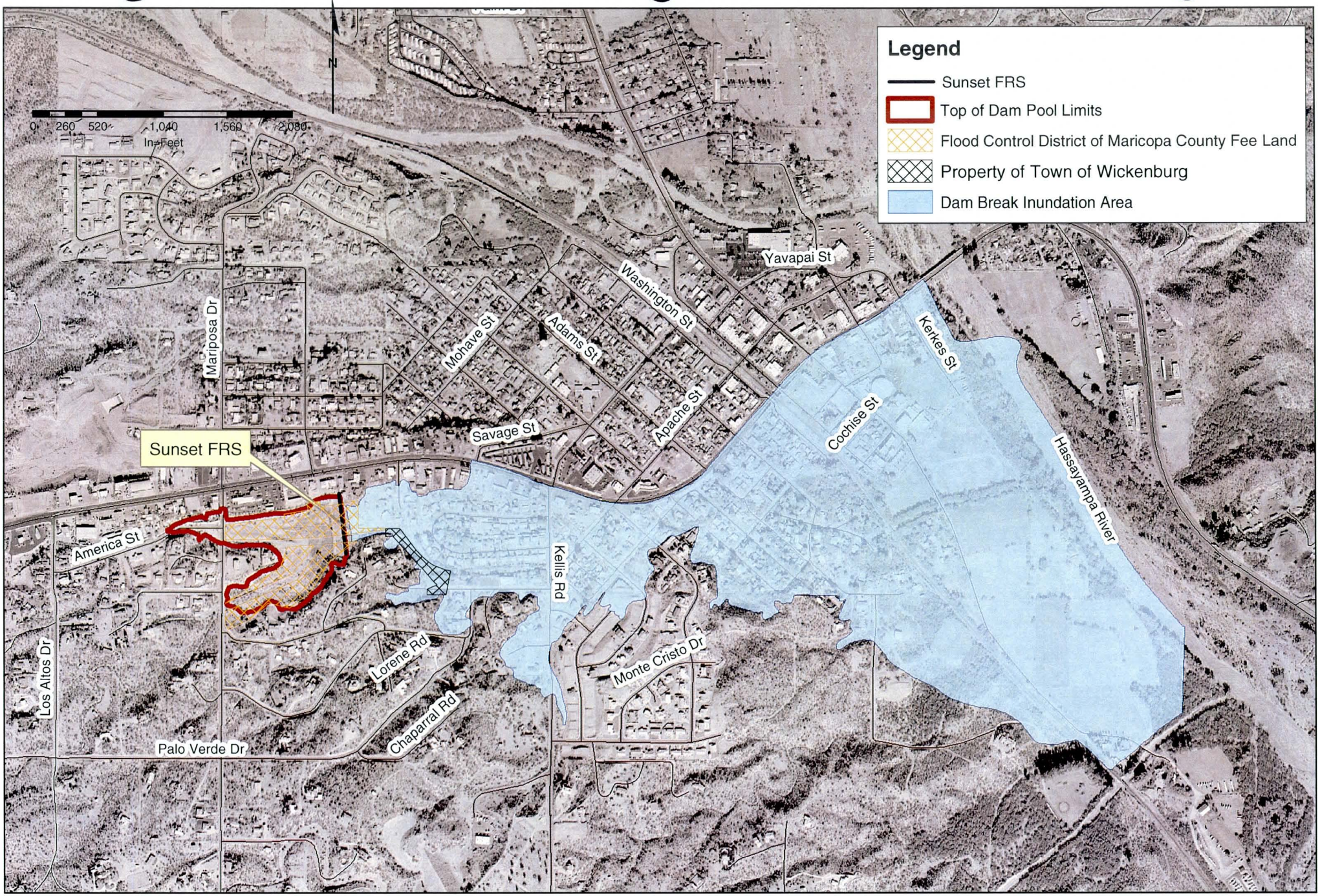

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FIGURE 3:SUNSET FRS DAM BREAK INUNDATION ZONE

FCD2003C015
PCN 050.36.31

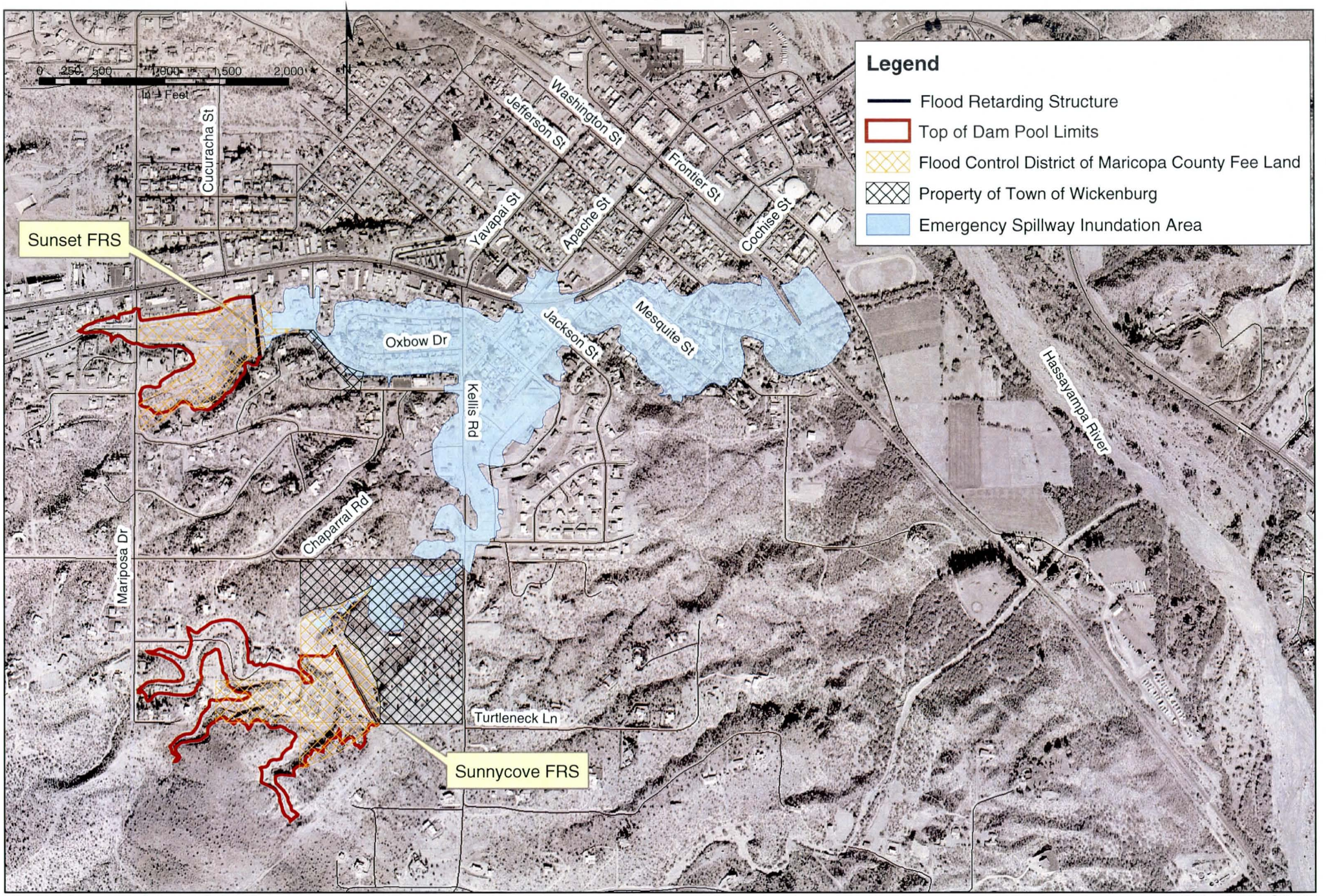

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FIGURE 4: SUNNYCOVE FRS AND SUNSET FRS EMERGENCY SPILLWAY INUNDATION AREA

FCD2003C015
PCN 050.36.31

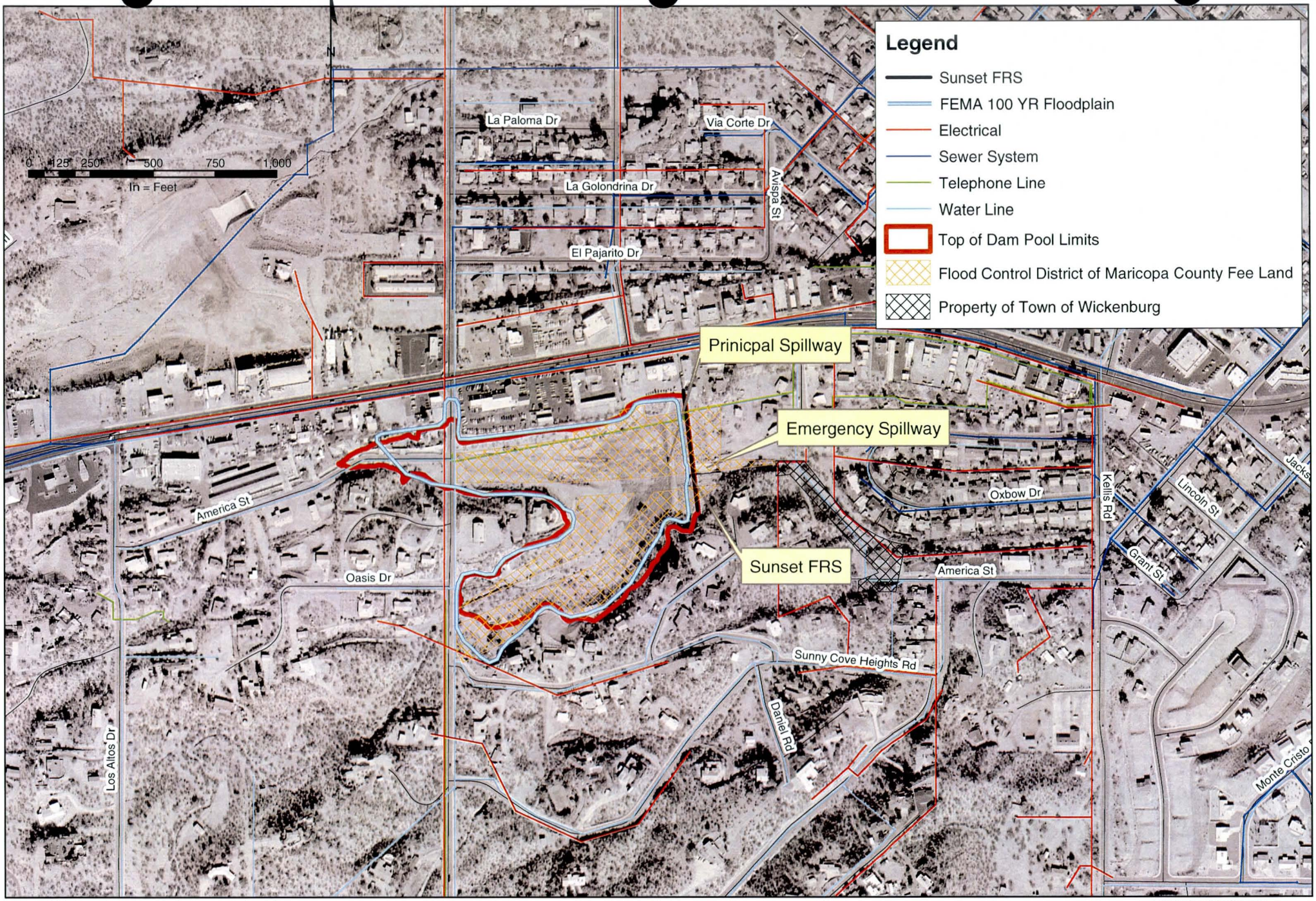

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FIGURE 5: SUNSET FRS TOP OF DAM POOL LIMITS

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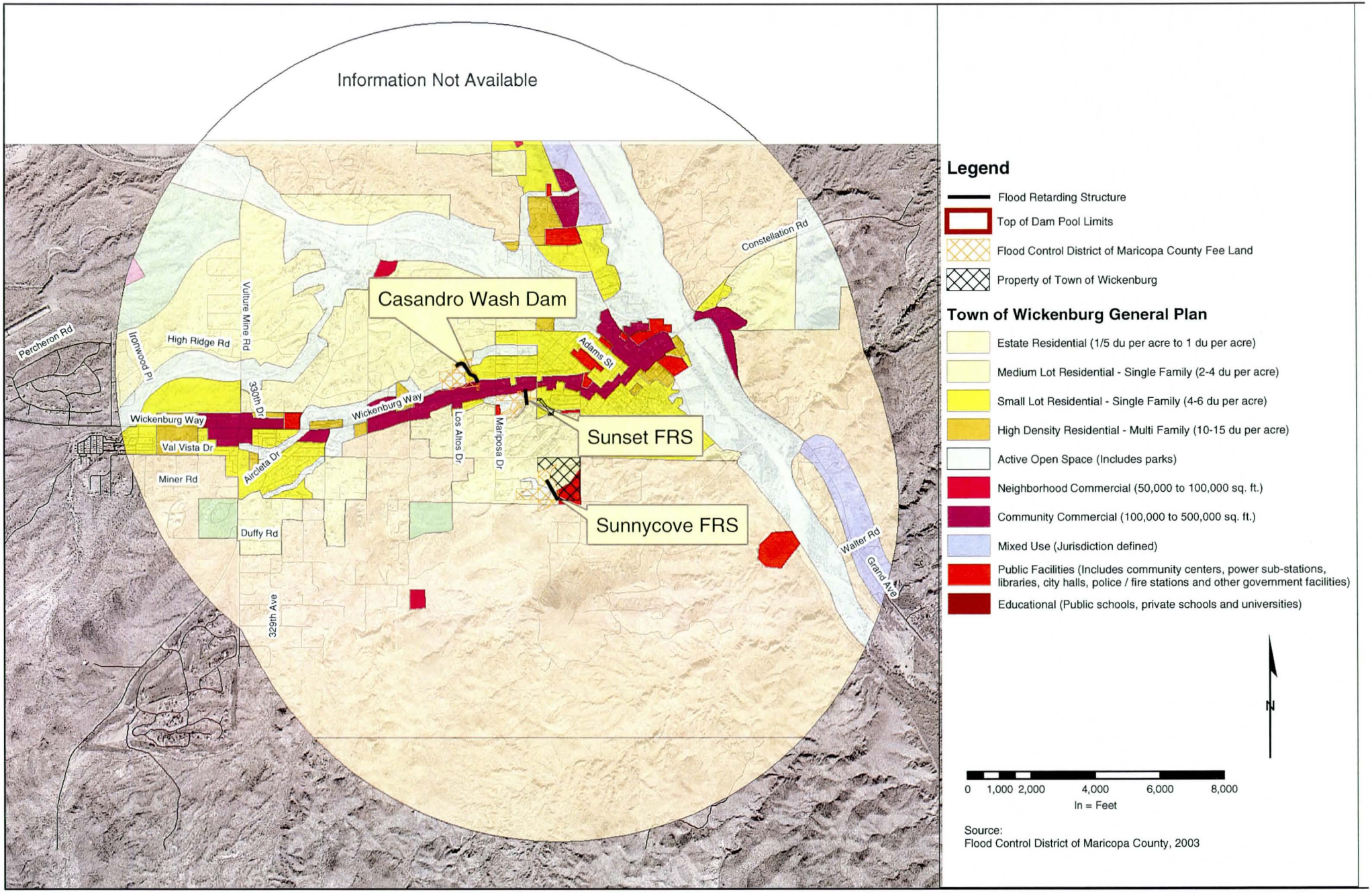

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FIGURE 6: GENERAL LAND USE

FCD2003C018
PCN 050.36.31

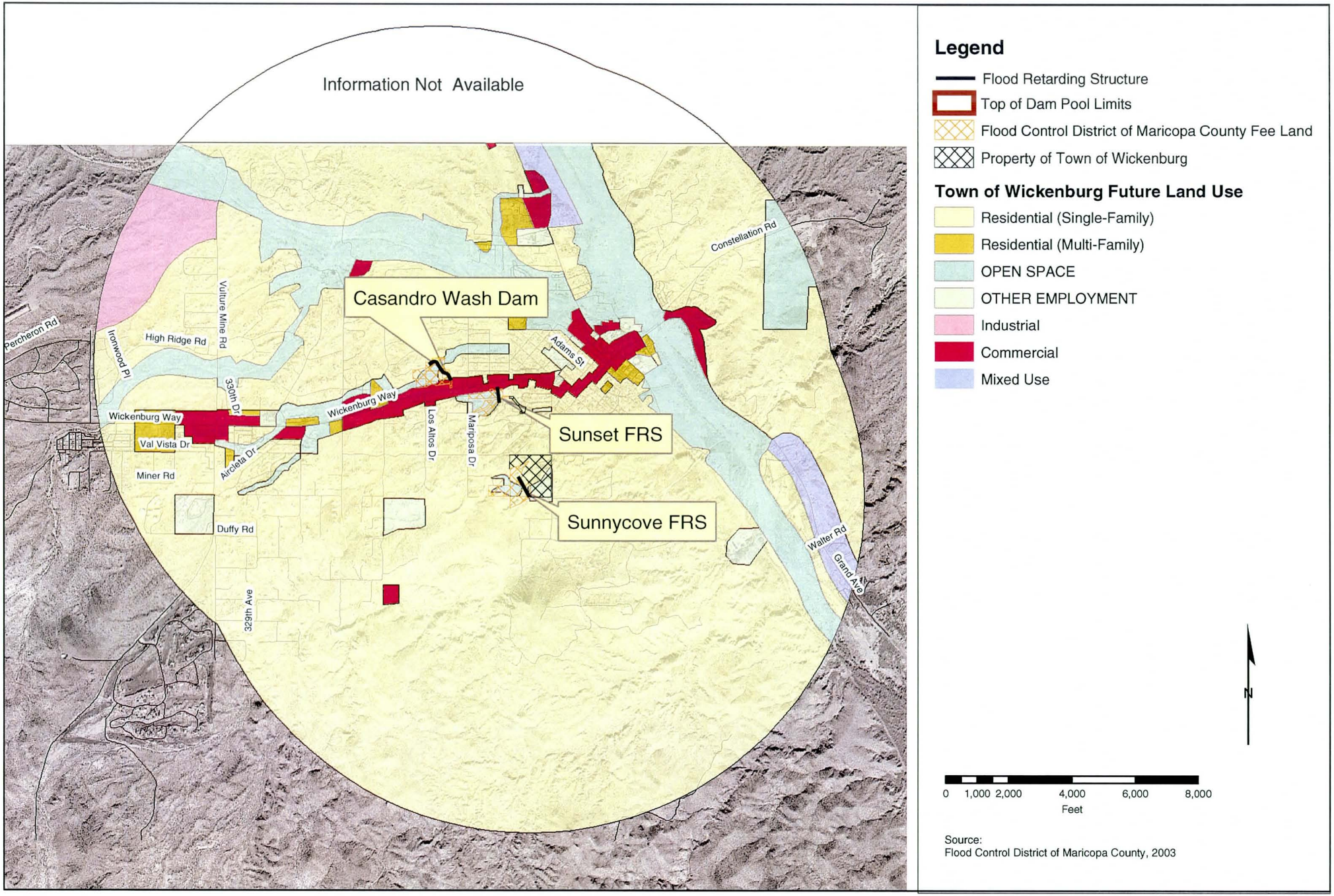

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FIGURE 7: FUTURE LAND USE

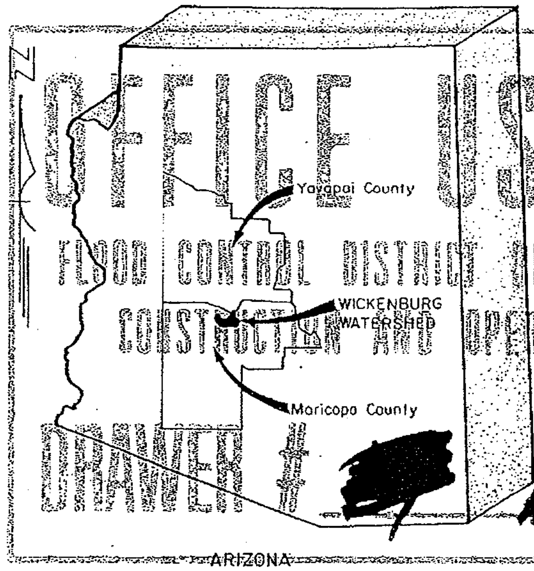
FCD2003C018
PCN 050.36.31

OFFICE USE ONLY

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

CONSTRUCTION AND OPERATIONS DIVISION

DRAWER # [redacted]



WICKENBURG WATERSHED PROTECTION AND FLOOD PREVENTION PROJECT

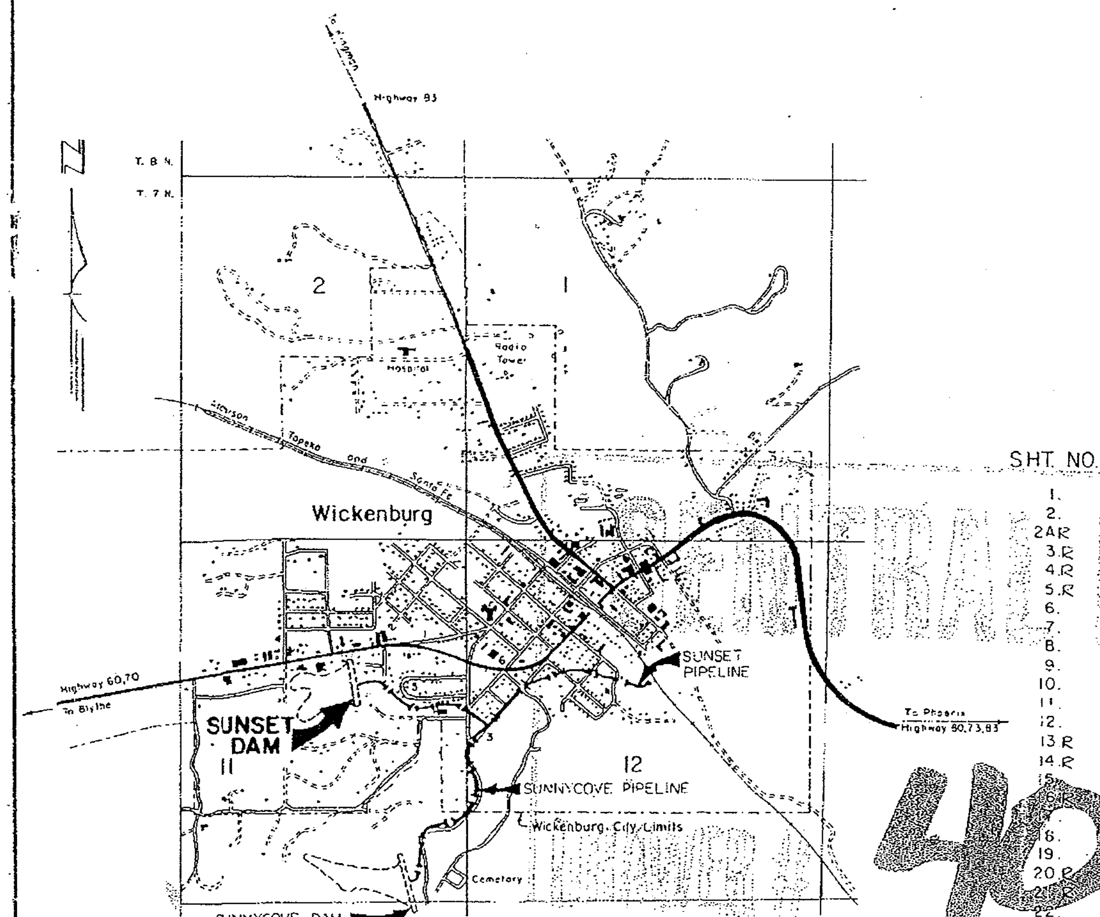
MARICOPA AND YAVAPAI COUNTIES, ARIZONA

PLANS FOR THE CONSTRUCTION OF

SUNSET 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

AS BUILT
CONSTRUCTION COMPLETED



40

INDEX OF DRAWINGS

SHT. NO.	TITLE
1.	INDEX OF DRAWINGS
2.	PLAN OF DAM AND RESERVOIR
2A R	ALIGNMENT LOCATION MAP
3. R	PROFILE ON ℄ DAM AND EXCAVATION SCHEDULES
4. R	PROFILE ON ℄ PRINCIPAL SPILLWAY & TYPICAL CROSS SECTION OF DAM
5. R	DRAIN LAYOUT DETAILS
6.	RISER LAYOUT DETAILS
7.	TRASH GUARD DETAILS
8.	RISER DETAILS
9.	RISER DETAILS
10.	RISER DETAILS
11.	GATE CONTROL DETAILS
12.	PRINCIPAL SPILLWAY CONDUIT DETAILS
13 R	EMERGENCY SPILLWAY PLAN, PROFILE AND PAYLINES
14 R	EMERGENCY SPILLWAY LAYOUT
15.	EMERGENCY SPILLWAY LAYOUT
16 R	EMERGENCY SPILLWAY LAYOUT
17.	EMERGENCY SPILLWAY INLET DETAILS
18.	EMERGENCY SPILLWAY INLET DETAILS
19.	EMERGENCY SPILLWAY CHUTE DETAILS
20 R	EMERGENCY SPILLWAY S.A.F. DETAILS
21 R	EMERGENCY SPILLWAY S.A.F. DETAILS
22.	FENCING DETAILS
23.	FENCING DETAILS
24.	SIGN DETAILS
25.	SOIL LOGS
26 R	SITE 1 [illegible]
27 R	SUNSET [illegible] EMBANKMENT FOUNDATION [illegible]
28 R	CIRCLE J-R MOTEL EMBANKMENT PROTECTION

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.

A — Section or Detail
7 | 7 — Sheet where section or detail is shown. / Sheet where section or detail is taken.

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.

SUNSET FRS
AS BUILT

7-E-23089 340

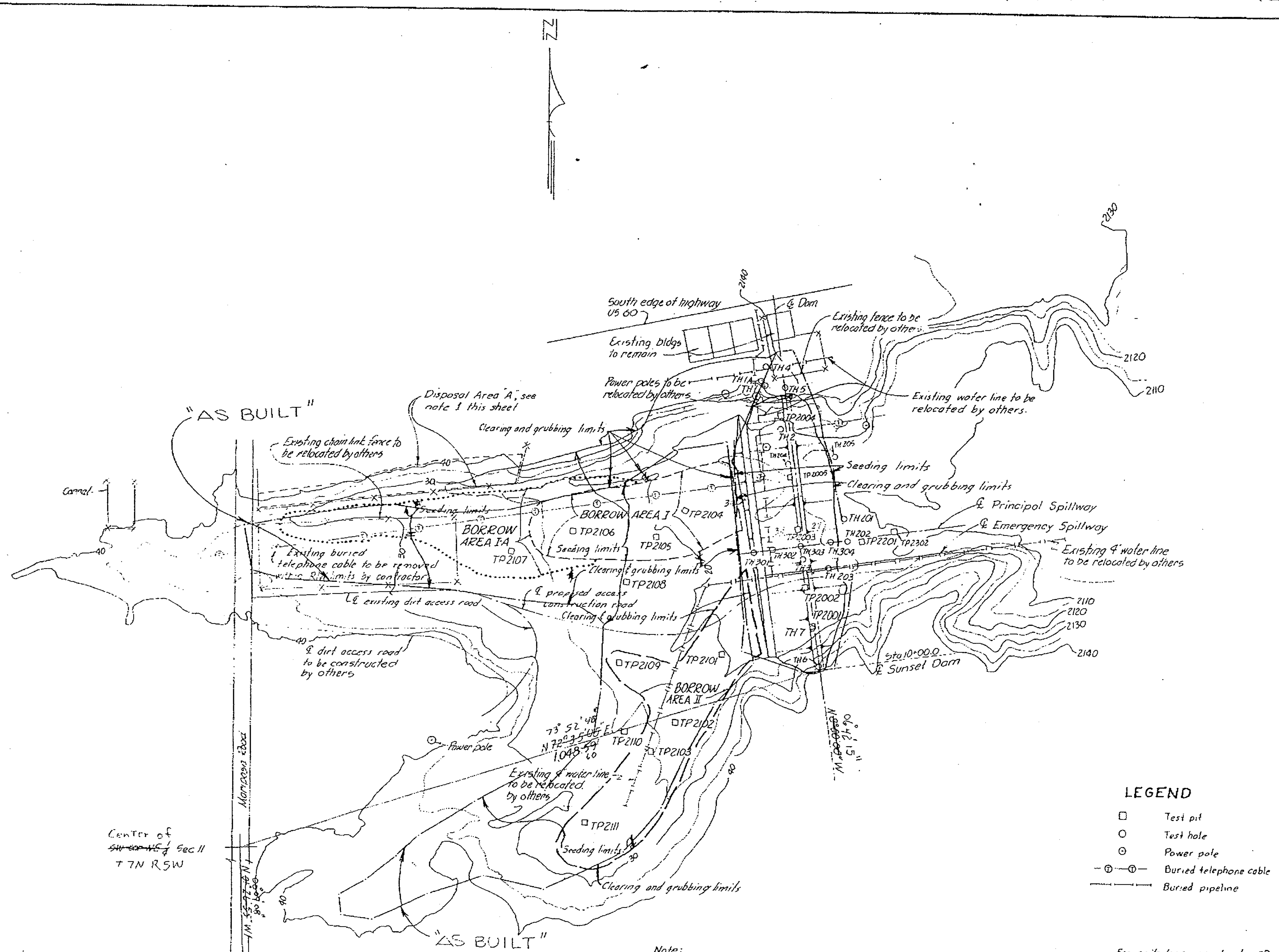
TOWN OF WICKENBURG	... COUNTY BOARD OF SUPERVISORS	WICKENBURG NATURAL RESOURCE CONSERVATION DISTRICT	TRIANGLE NATURAL RESOURCE CONSERVATION DISTRICT	MARICOPA COUNTY FLOOD CONTROL DISTRICT
APPROVED	APPROVED	APPROVED	APPROVED	APPROVED
DATE 6-16-75 [illegible] Mayor	DATE 6-16-75 [illegible] Chairman	DATE 6-12-75 [illegible] Secretary - Treasure	DATE [illegible] Curtis Ritter Chairman	DATE 6-13-75 [illegible] Chief Engineer

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date	
Designed Soil Conservation Service	3-75	Approved by Head EBWAT Unit
Drawn Soil Conservation Service	3-75	Ralph M. [illegible] State Conservation Engineer
Traced		Sheet No. 1 of 28 — Drawing No. 7-E-23089
Checked		

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



100 0 100 200
SCALE IN FEET

Note:

1. Disposal Area "B" for Sunset FRS is located approximately 1,400 feet downstream from Sunset Dam. See location on Sunset and Sunnycove Pipeline drawings No. 7-E-23091, sheet 3.

AS BUILT
CONSTRUCTION COMPLETION
8/15/76
[illegible]

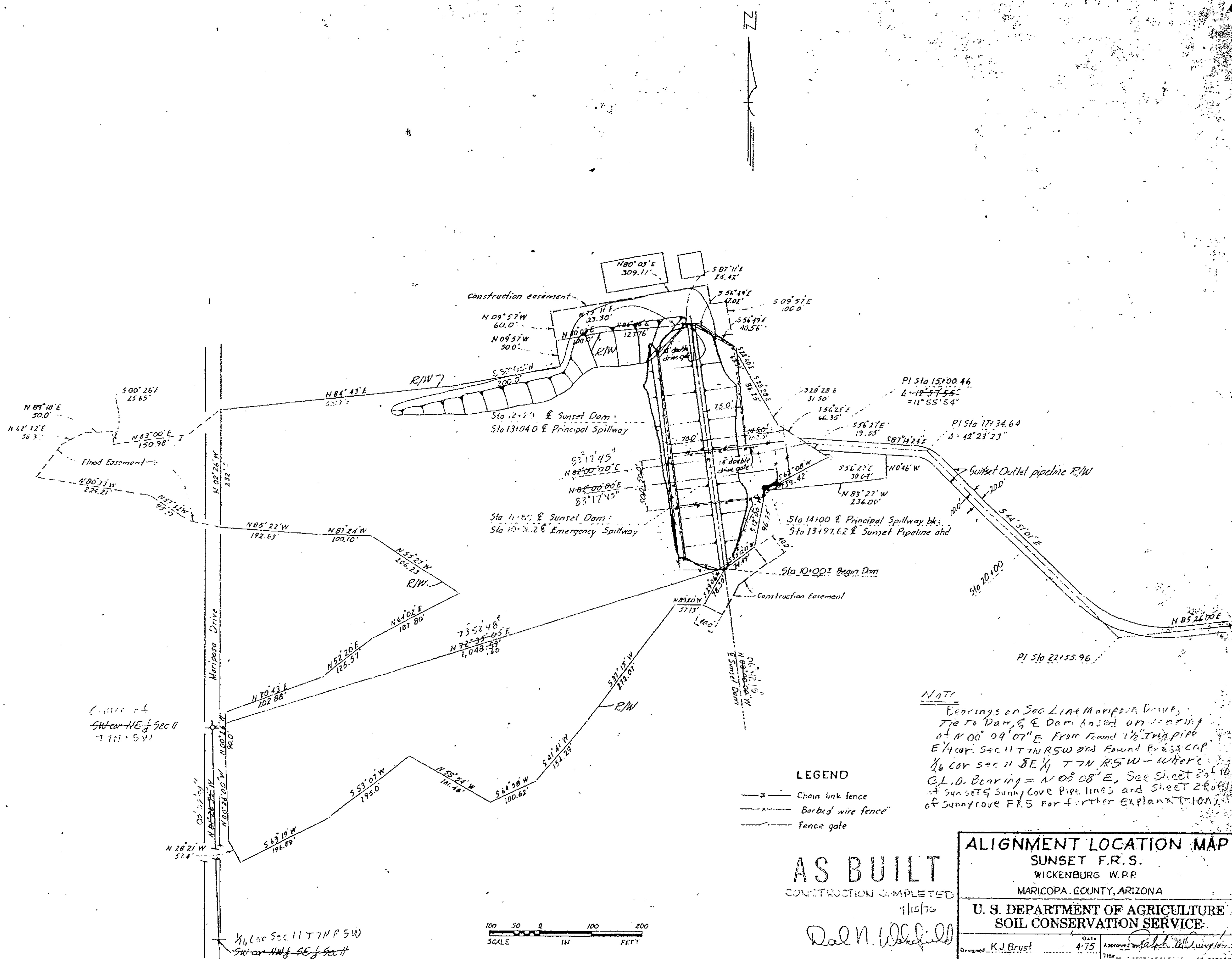
For soils logs see sheets 3, 3A and 25
For right-of-way limits see sheet 2AR

PLAN OF DAM AND RESERVOIR

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J.B. G.M.C.	Date 2-75	Approved by	
Drawn Jack D. Land	2-75	Title	
Traced		Title	
Checked		Sheet No. 2 of 28	Drawing No. 7-E-23089



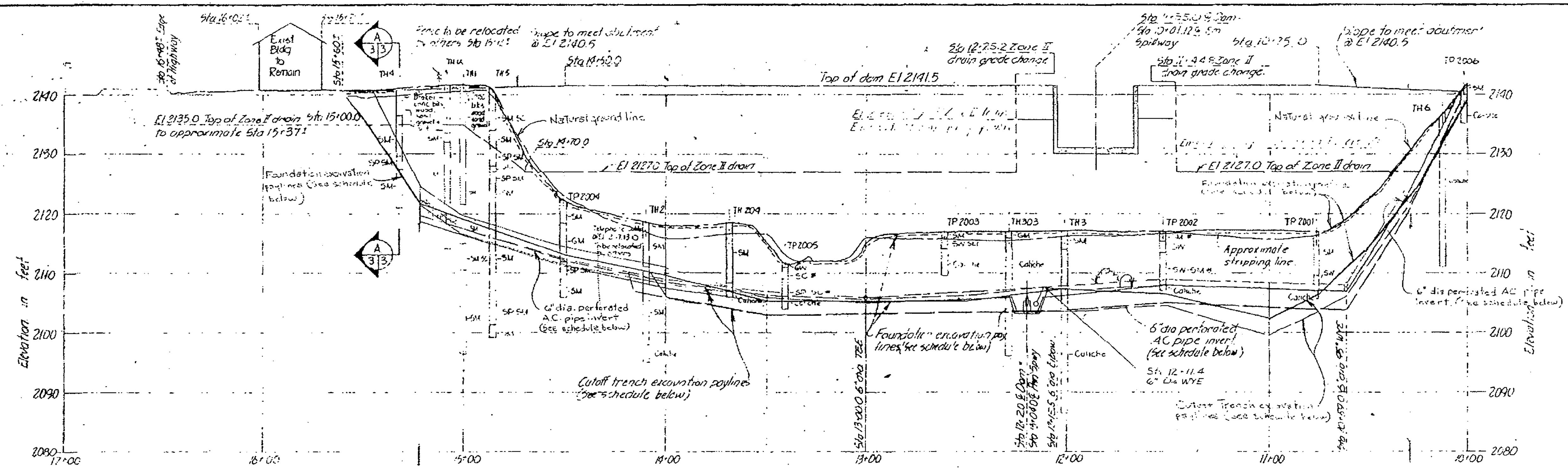
NOTE

Bearings on Sec Line Mariposa Drive, Tie To Dam, & ℄ Dam based on bearing of N 00° 09' 07" E From Found 1½" Iron pipe E¼ cor. Sec 11 T7N R5W and Found Brass cap 1/16 cor sec 11 SE ¼ T7N R5W – where G.L.O. Bearing = N 00° 08' E, See Sheet 2 of 10 of Sunset & Sunny Cove Pipe lines and Sheet 2 R of 18 of Sunny Cove F.R.S. for further Explanation.

AS BUILT
CONSTRUCTION COMPLETED
7/15/76
Dal N. Wakefield

ALIGNMENT LOCATION MAP SUNSET F.R.S. WICKENBURG W.P.P. MARICOPA COUNTY, ARIZONA		
U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE		
Designed K.J. Brust	Date 4-75	Approved by
Drawn J.O. Land	4-75	Title
Traced L.W. McClintock	4-75	Title
Revised T. Jayo	1-76	Sheet No. 2AR of 35 28 — Drawing No. 7-E-23089

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



PROFILE ON ℄ DAM

(Looking downstream)

SCHEDULES

℄ FOUNDATION EXCAVATION	
STATION	ELEVATION
10+00 ±	2140.5 1
10+30 +25	~~2123.0~~ 24.1
10+59 +50	~~2107.0~~ 13.9
11+50	~~2108.0~~ 06.3
12+06.9	~~2106.0~~ 07.0
12+09	2103.4 .0
12+21	2103.4 .0
12+23.6	~~2100.0~~ .2
~~13+00~~	2105.0
13+50	~~2106.0~~ 05.2
14+50	~~2112.0~~ 13.0
15+00 22	~~2126.0~~ 21.5
15+60 ±	~~2140.5~~ 39.1

℄ Elevation at ℄ of Dam

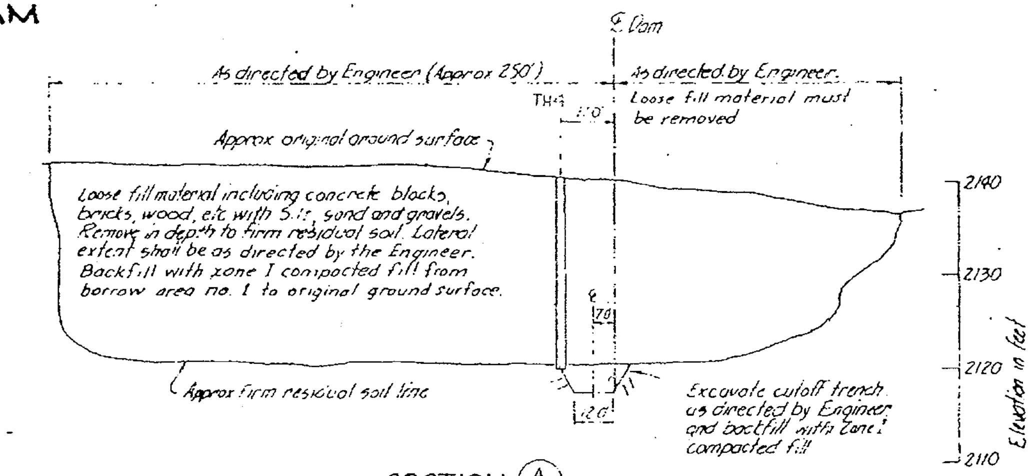
℄ CUTOFF TRENCH EXCAVATION	
STATION	ELEVATION
10+00 ±	2139.0
10+30	2121.0
10+59	~~2104.0~~ 06.0
11+50	~~2105.0~~ 04.5
12+00	2103.45
13+50	2103.0
14+00	2106.0

℄ Elevation at 70' Upstream from ℄ of Dam

℄ DRAIN PIPE PLACEMENT		
STATION	ELEVATION	REMARKS
10+30.0	~~2124.0~~ 23.0	Plug beginning end with mortar, or AC cap
10+59.0	~~2108.0~~ 07.0	CI 45° WYE, 6" dia non-perf AC outlet pipe.
11+50.0	~~2109.0~~ 08.0	Plug beginning ends with mortar (2 inch) or AC caps
~~12+00.0~~ 12+11.4	~~2107.0~~ 2108.0	~~Grade change~~ Grade change, CI WYE
12+15.5	2105.97	CI 90° Elbow to 6" dia non-perf AC outlet pipe
12+20.0	2106.0	Two pipe joints read within 1'ea of ℄ princ. spwy Sta 12+15
~~12+50.0~~ 13+00.0	~~2106.5~~ 05.5	~~Grade change~~ CI TEE to 6" dia non-perf AC outlet pipe.
13+50.0	2107.0	Grade change.
~~14+00.0~~ 14+50.0	~~2110.0~~ ~~2111.5~~ 11.8	~~Grade change~~ Grade change.
15+00.0	~~2117.0~~ 17.5	Grade change.
15+20.0	1.0' Above foundation excavation line approved by Engineer.	Plug beginning end with mortar, or AC cap

℄ Elevation at 100' downstream from ℄ of Dam

℄ All elevations and stations are approximate and may be adjusted by the Engineer during construction.



SECTION A/3 3
NOT TO SCALE

AS BUILT
CONSTRUCTION COMPLETED
1/15/76
[illegible]

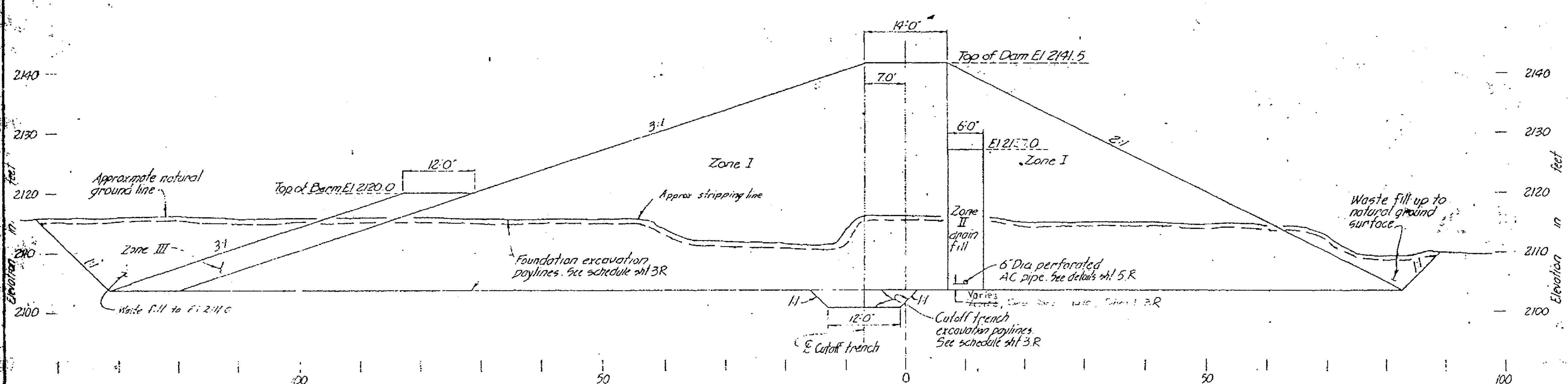
Notes:
1. Unified soil descriptions are based on field identifications except where an asterisk is shown the classification has been based on laboratory analysis.
2. Complete field logs, laboratory test data and geologic report are available for review in the project office at Wickenburg Arizona.
3. For locations of Test pits and Test holes see sheet 2.

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

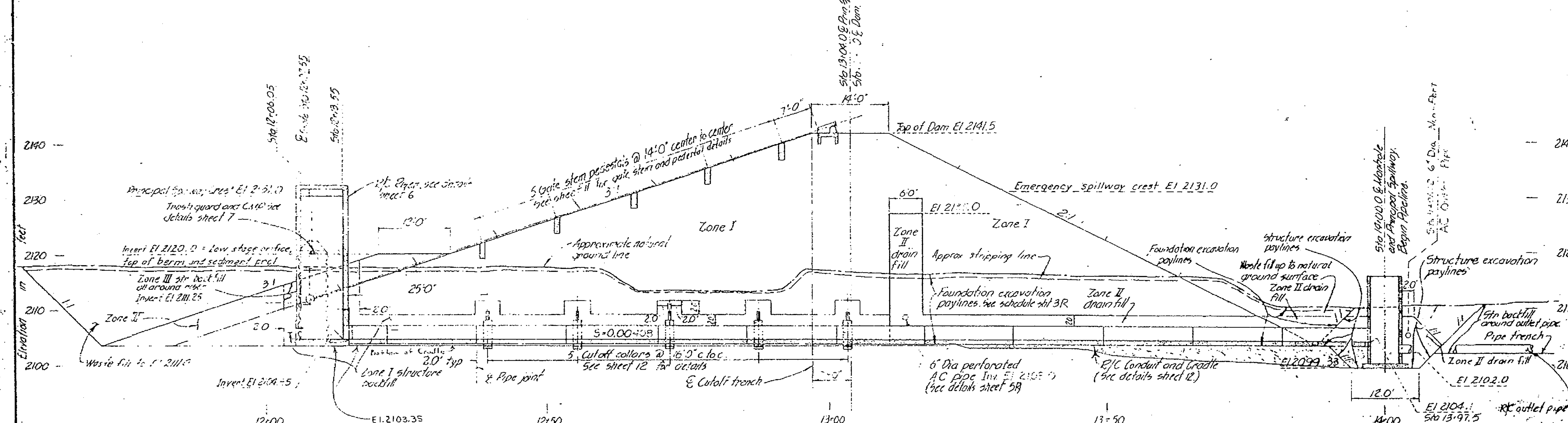
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed Greg Cunningham	3-75	Approved by [illegible]
Drawn Jack Lond	3-75	Title
Traced		Date
Revised T. Ja/o	6-76	Sheet No. 5R of 25 28 — Drawing No. 7-E-23089

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

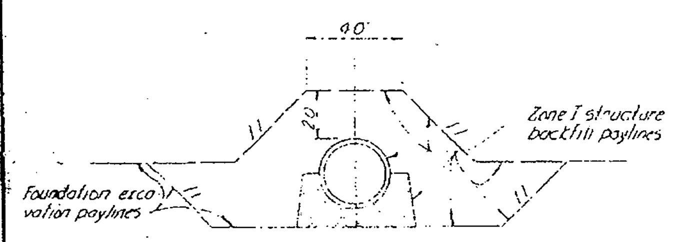


TYPICAL CROSS SECTION
OF DAM



PROFILE ON ℄ PRINCIPAL SPILLWAY

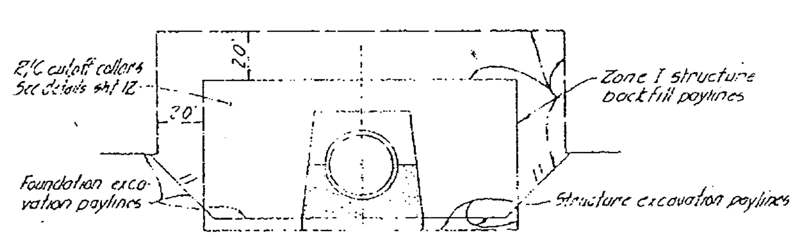
R/C manhole, Pipe trench, R/C outlet pipe and Structure backfill around outlet pipe are part of bid schedule 3.



R/C CRADLE PAYLINES

STA 12+16.5 to STA 13+11.0

(See sheet 5R for Sta 13+11 to Sta 14+00)



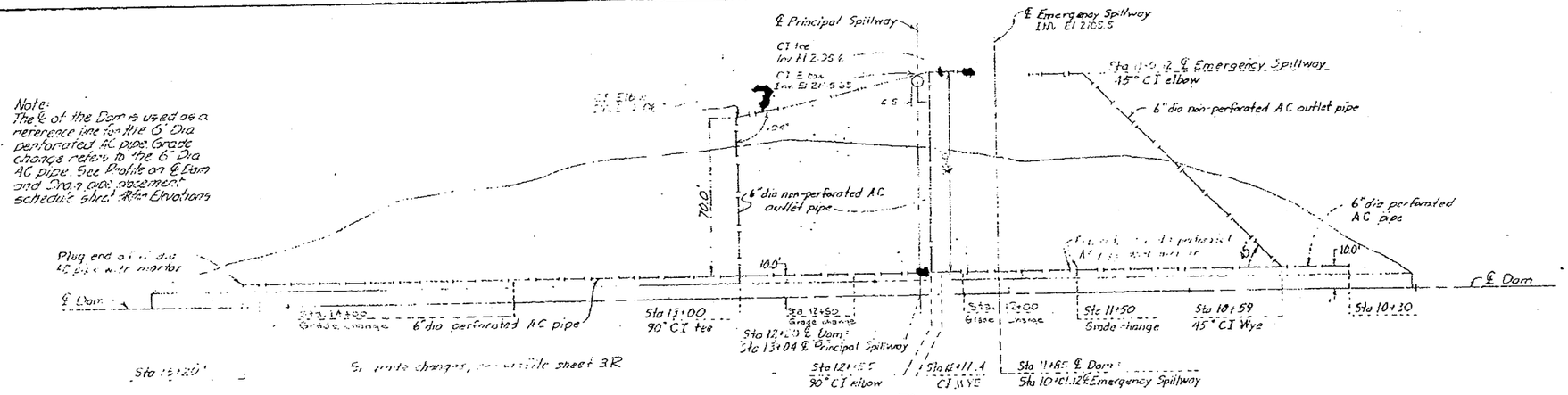
CUTOFF COLLAR PAYLINES

AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Dal N. Wakefield

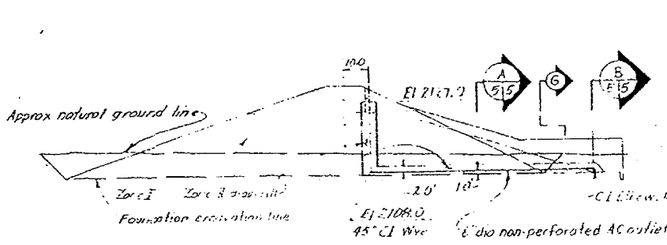
PROFILE ON ℄ PRINCIPAL SPILLWAY AND TYPICAL CROSS SECTION OF DAM SUNSET F.R.S WICKENBURG W.P.P MARICOPA COUNTY, ARIZONA	
U. S. DEPARTMENT OF AGRICULTURE SOIL CONSERVATION SERVICE	
Designed Greg Cunningham Date 3-75	[illegible]
Drawn Jack Land 3-75	
Traced	
Checked T. Jayo 6-76	Sheet No. 4R of 28 — Drawing No. 7-E-23089

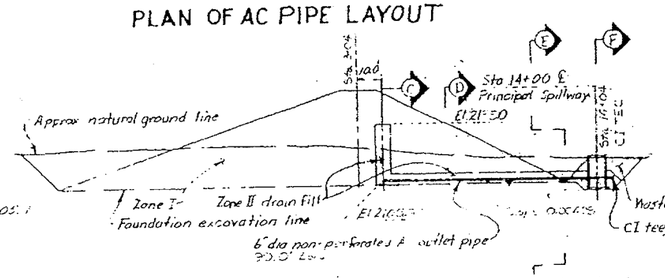
SCS-ENG-313-B Rev. 7-64

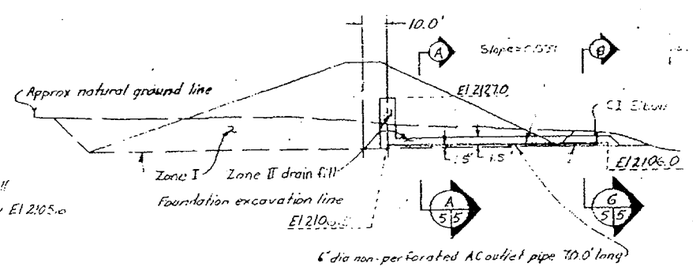
Note:
The ℄ of the Dam is used as a reference line for the 6" Dia perforated AC pipe. Grade change refers to the 6" Dia AC pipe. See Profile on ℄ Dam and Drain pipe assignment schedule sheet 3R for Elevations



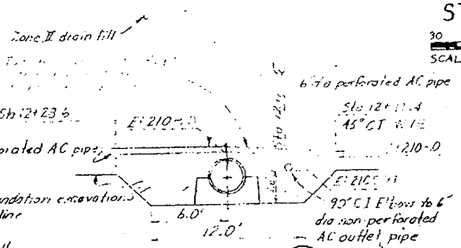
PLAN OF AC PIPE LAYOUT





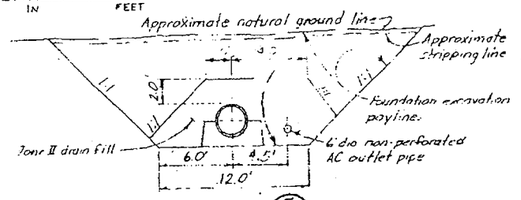


PROFILE ON ℄ AC OUTLET PIPE STA 10+59 ℄ DAM

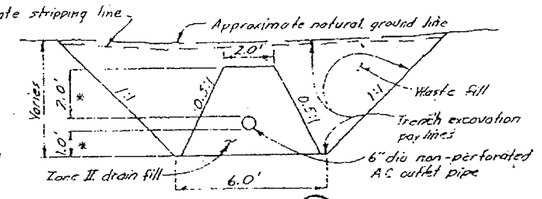


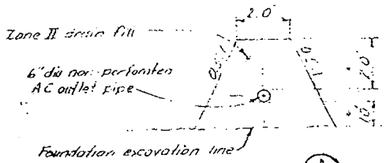
PROFILE ON ℄ AC OUTLET PIPE STA 12+15.5

SCALE 30 0 30 60 IN FEET

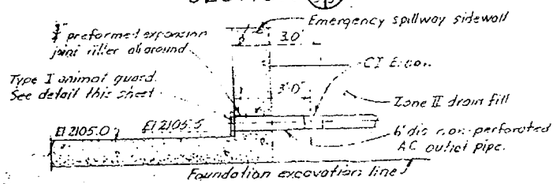


PROFILE ON ℄ AC OUTLET PIPE STA 13+00

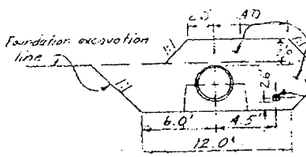




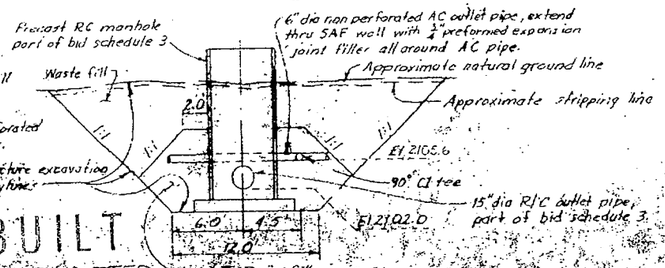
SECTION A 5/5



SECTION B 5/5

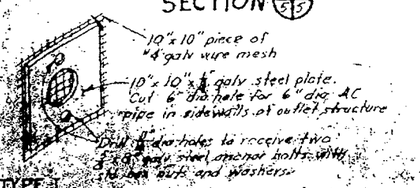


SECTION D 5/5



SECTION F 5/5

In Sections A 5/5 and G 5/5 these dimensions shall be 1.5' each for AC outlet pipe @ Sta 13+00



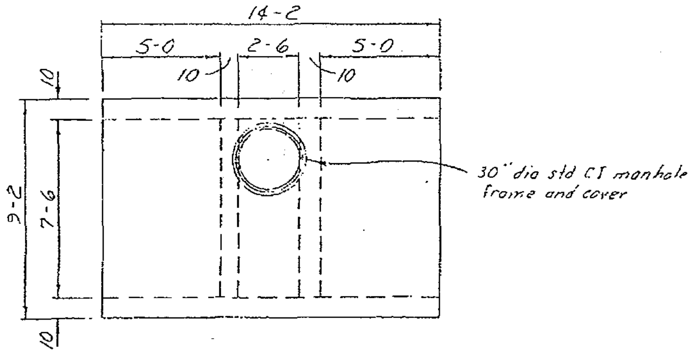
TYPE I (2 req'd)

ANIMAL GUARD DETAILS
(NOT TO SCALE)

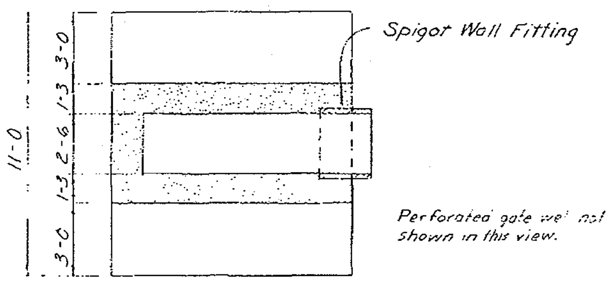
AS BUILT
CONSTRUCTION COMPLETED 9/15/70

DRAIN LAYOUT AND DETAILS
SUNSET F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

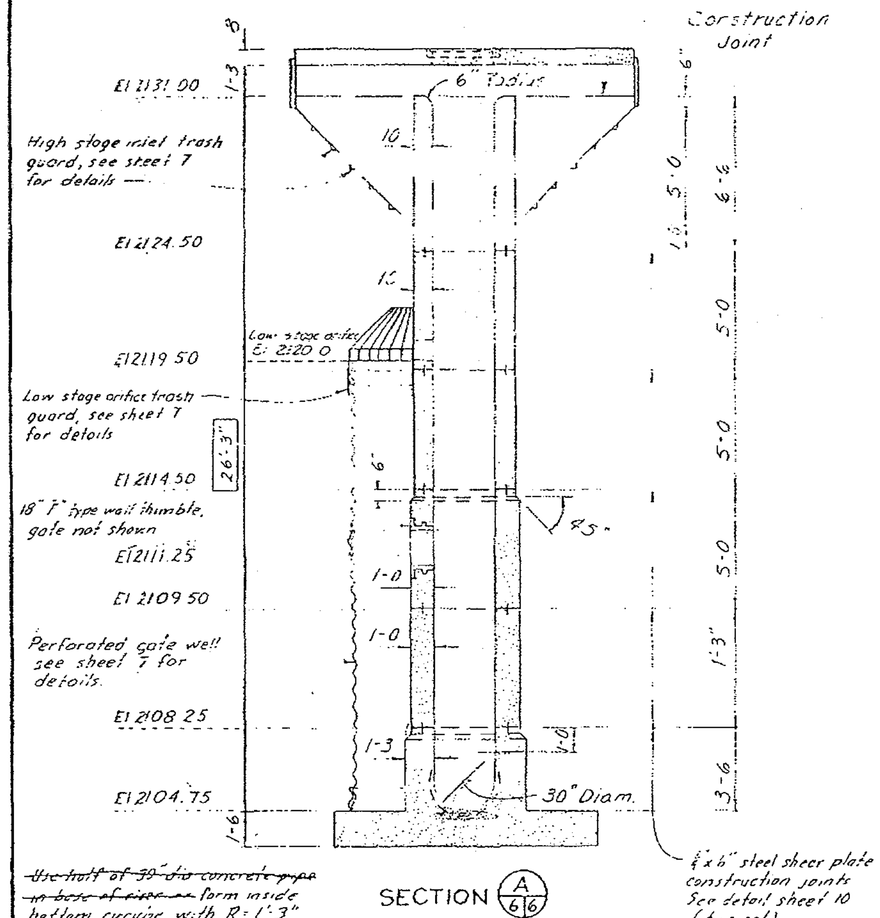
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE



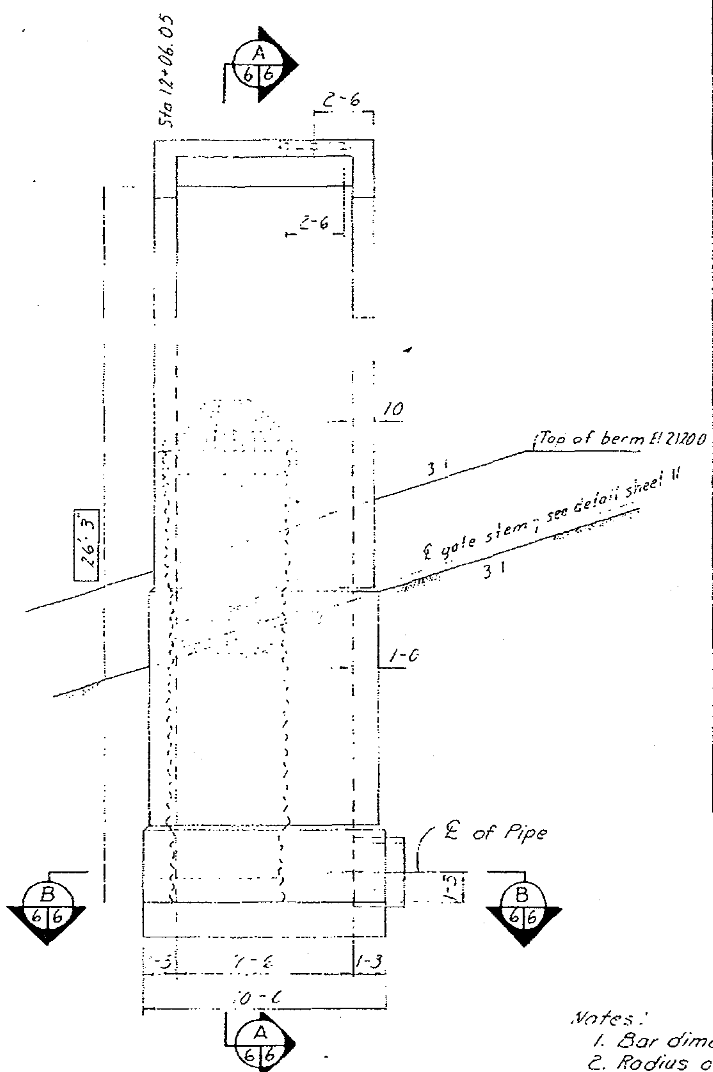
PLAN - TOP



SECTION B 6|6



SECTION A 6|6

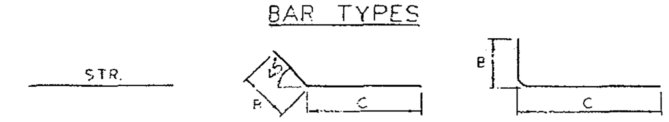


SIDE ELEVATION

STEEL SCHEDULE

Mark	Size	Quantity	Length	Type	B	C	Total Length	Mark	Size	Quantity	Length	Type	B	C	Total Length
B 1	# 7	11	10-6	1			115-6								
B 2	# 6	12	9-6	1			114-0								
B 3	# 6	38	7-6	21	3-5	4-1	285-0								
B 4	# 6	12	9-6	1			114-0								
B 5	# 6	10	10-6	1			105-0								
B 6	# 6	2	4-6	1			9-0								
B 7	# 6	6	5-3	21	[illegible]	4-1	31-6	T 1	# 5	18	6-0	1			108-0
B 8	# 6	19	7-3	21	1-1	6-2	137-9	T 2	# 5	6	8-0	1			48-0
B 9	# 5	10	8-6	1			85-0	T 3	# 5	4	4-9	1			19-0
B 10	# 6	5	3-6	1			17-6	T 4	# 5	4	3-6	1			14-0
B 11	# 6	3	2-3	1			6-9	T 5	# 5	4	2-3	1			9-0
B 12	# 6	2	2-3	1			4-6	T 6	# 5	4	9-0	19	2-0	7-0	36-0
B 13	# 6	24	4-0	1			96-0	T 7	# 5	12	8-3	1			99-0
B 14	# 5	10	6-6	21	0-10	5-8	[illegible]	T 8	# 5	2	3-3	1			6-6
B 15	# 5	14	5-9	21	3-1½	5-7½	[illegible]	T 9	# 5	2	5-9	1			11-6
B 16	# 5	4	8-3	21	2-10½	5-4½	33-0	T 10	# 5	2	10-9	1			21-6
								T 11	# 5	2	13-3	1			26-6
								T 12	# 5	14	6-3	1			87-6
								T 13	# 5	6	8-0	1			48-0
								T 14	# 5	4	6-0	1			24-0
								T 15	# 5	4	4-9	1			19-0
								T 16	# 5	4	3-6	1			14-0
								T 17	# 5	4	2-3	1			9-0
								T 18	# 5	4	9-0	19	2-0	7-0	36-0
								T 19	# 5	24	8-0	21	2-9	5-3	192-0
								T 20	# 5	2	3-3	1			6-6
R 1	# 5	22	8-0	1			176-0	T 21	# 5	2	5-9	1			11-6
R 2	# 5	6	8-6	1			51-0	T 22	# 5	2	8-3	1			16-6
R 3	# 5	4	3-6	1			14-0	T 23	# 5	2	10-9	1			21-6
R 4	# 5	26	5-9	1			149-6	T 24	# 5	2	13-3	1			26-6
R 5	# 6	~~6~~ 12	8-9	21	3-1	5-8	105-9 ~~52-6~~	T 25	# 5	4	13-9	1			55-0
R 6	# 4	20	8-3	1			165-0	T 26	# 5	4	13-9	1			55-0
R 7	# 5	10	3-6	1			35-0	T 27	# 4	14	8-3	1			115-6
R 8	# 5	20	3-8	1			73-4	T 28	# 4	2	4-9	1			9-6
R 9	# 5	36	8-3	21	[illegible]	5-4½	297-0	T 29	# 4	7	13-9	1			96-3
R 10	# 5	4	[illegible]	21	2-9	5-3	[illegible]	T 30	# 4	4	5-3	1			21-0
R 11	[illegible]	[illegible]	[illegible]	1			[illegible]	T 31	# 5	24	6-9	21	1-6	5-3	162-0
R 12	5	[illegible]	8-3	1			[illegible]	T 32	# 5	2	6-6	21	1-6	5-0	13-0
R 13	[illegible]	8	[illegible]	1			26-0	T 33	# 5	2	2-6	21	1-6	1-0	5-0
R 14	5	[illegible]	11-9	1			[illegible]	T 34	# 4	7	13-9	1			96-3
R 15	# 6	[illegible]	8-3	21	2-10½	5-4½	[illegible]	T 35	# 4	4	5-3	1			21-0
R 16	# 5	[illegible]	[illegible]	1			[illegible]								
R 17	# 5	8	3-3	1			26-0								
R 18	# 5	20	6-0	21	2-9	5-3	[illegible]								

BAR TYPES



TYPE 1 TYPE 19 TYPE 21

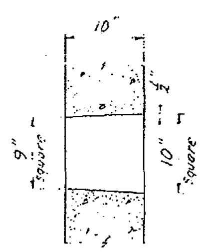
Notes:
1. Bar dimensions are out to out of bar.
2. Radius of bends equals 3 bar diameters for sizes equal to or less than #7.
3. The 2" and 3" dimensions from face of concrete to steel are clear distances.

0 2 4 6
Scale in Feet

1 0 5 10
SCALE IN FEET

AS BUILT
CONSTRUCTION COMPLETED 9/15/76
[illegible]

Low stage orifice
Invert El 2120.0



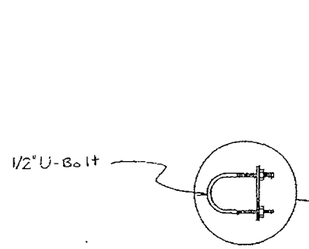
ORIFICE DETAIL
(NOT TO SCALE)

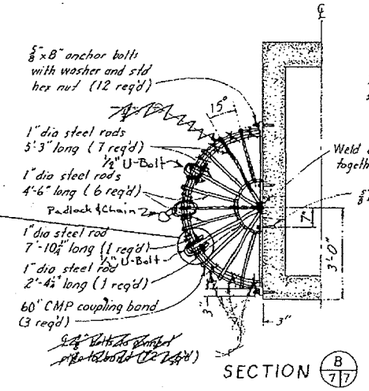
RISER LAYOUT DETAILS
SUNSET F.R.S.
WICKENBURG WPP
MARICOPA COUNTY, ARIZONA

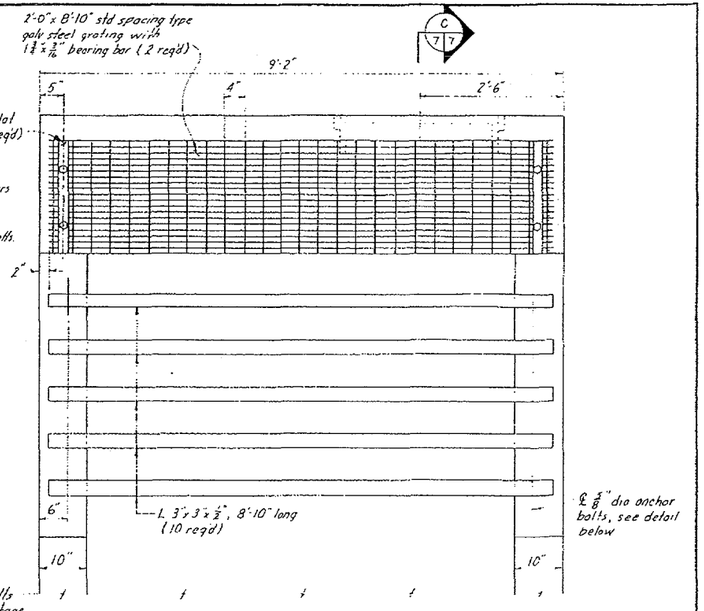
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

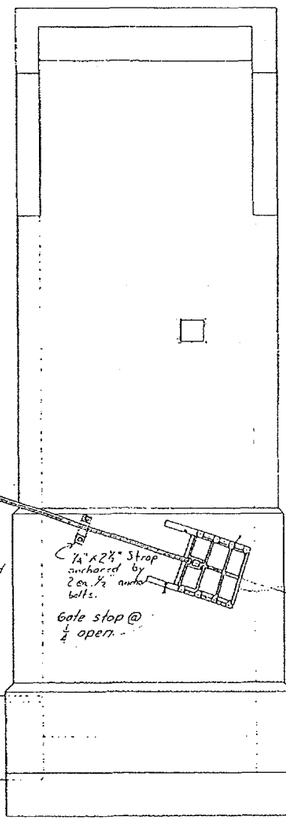
Designed K J Brust	Date 3-75	Approved by	
Drawn L.W. McClintock	3-75	Title	
Traced		Title	
Checked		Sheet No. 6 of ~~25~~ 28	Drawing No. 7-E-23089

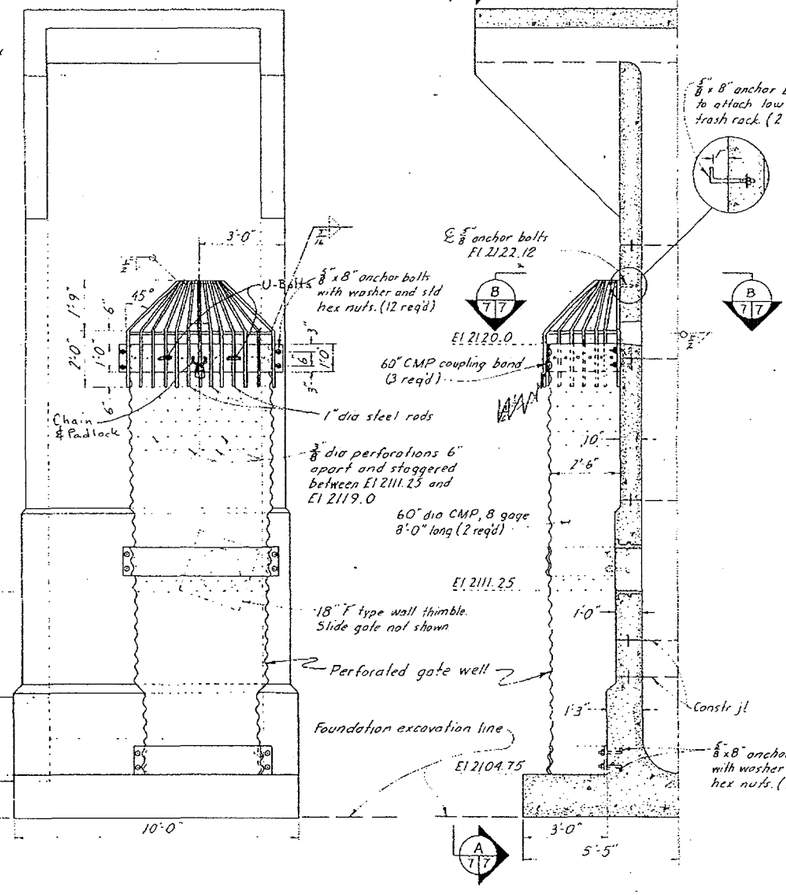
FORM SCS 313 (APRIL 1968)

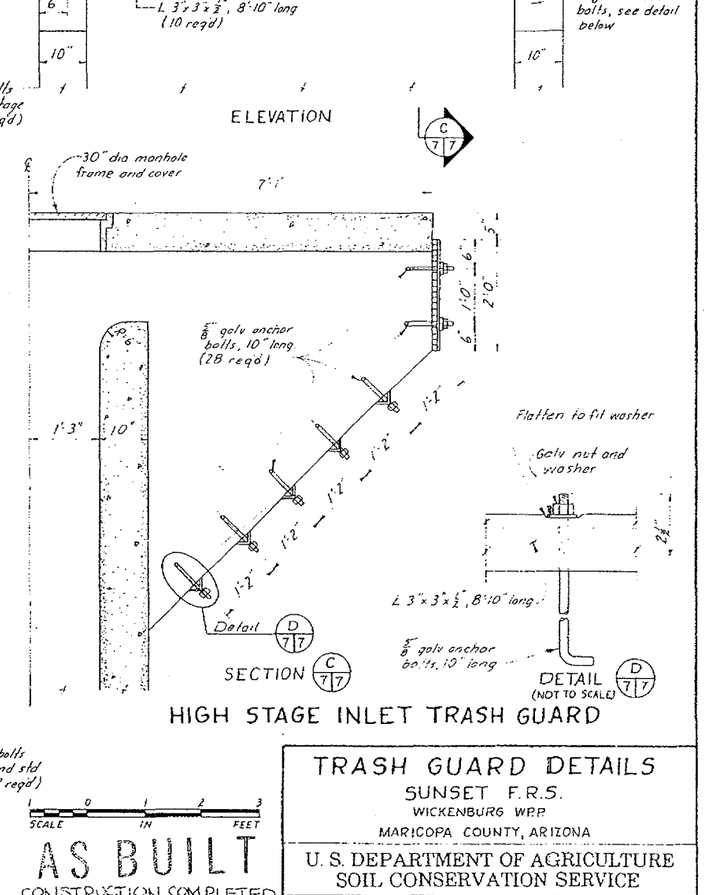












SCALE 0 1 2 3 IN FEET

AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Dale M. Wakefield

TRASH GUARD DETAILS

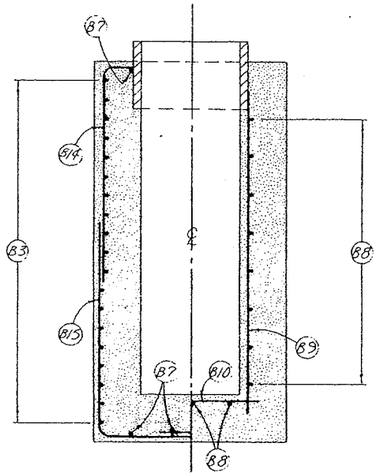
SUNSET F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date
Designed KJ Beust	3-75
Drawn LW. McClintock	3-75
Traced	
Checked	

Sheet 7 of 25 sheets
7-E-23089

SCS-ENG. 413-B Rev. 7-69

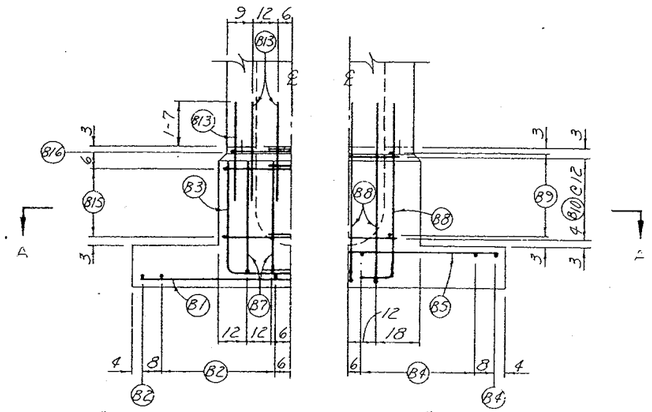


Outside Steel Inside Steel

SECTION A-A

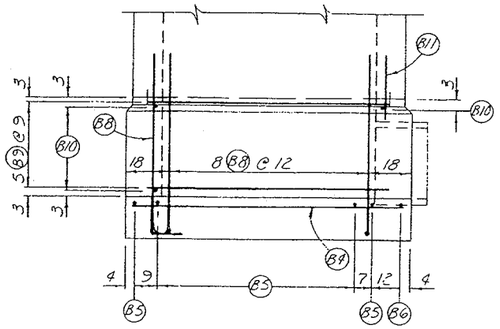
0 1 2 3

Scale in Feet



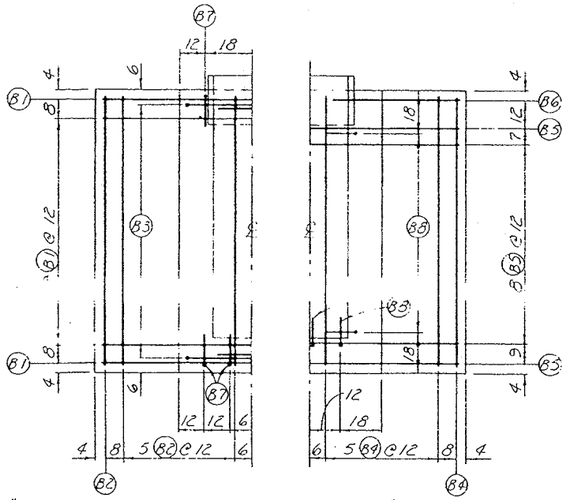
Steel 2" From Outside Face Steel 2" From Inside Face

UPSTREAM ELEVATION



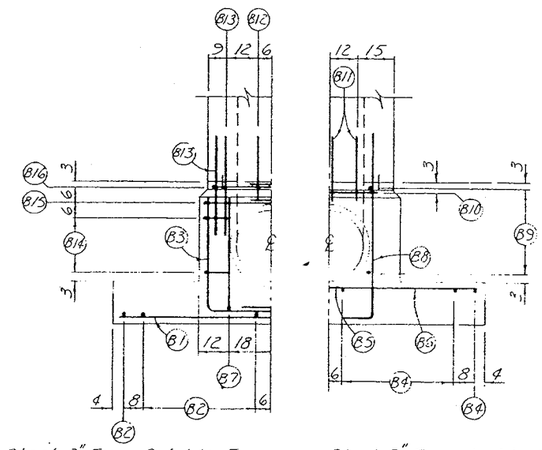
Steel 2" From Inside Face and 2" From Top of Footing

SIDEWALL ELEVATION



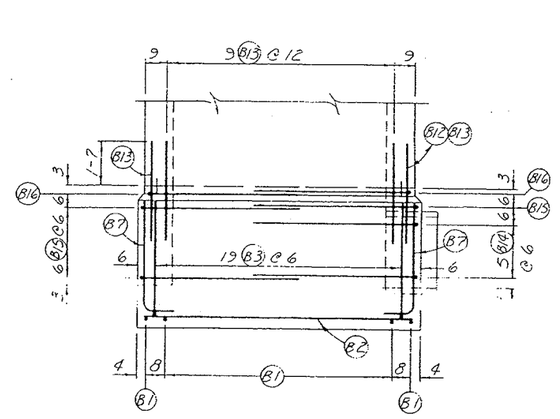
Steel 3" From Bottom of Footing Steel 2" From Top of Footing

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

0 2 4

Scale in Feet

Unless Otherwise Shown

AS BUILT

CONSTRUCTION COMPLETED

9/15/76

Dale N. Wakefield

RISER DETAILS

SUNSET F.R.S.

WICKENBURG W.P.P

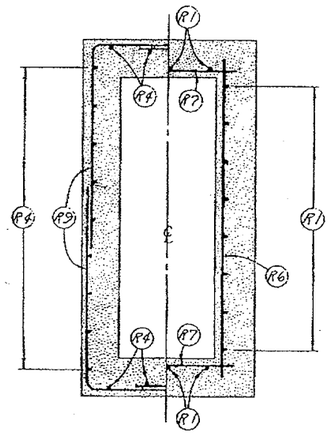
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

Designed K.J. Brust	Date 3-75	Approved By	
Drawn		Title	
Traced		Date	
Checked		Sheet No. 8 of 28 sheets	Drawing No. 7-E-23089

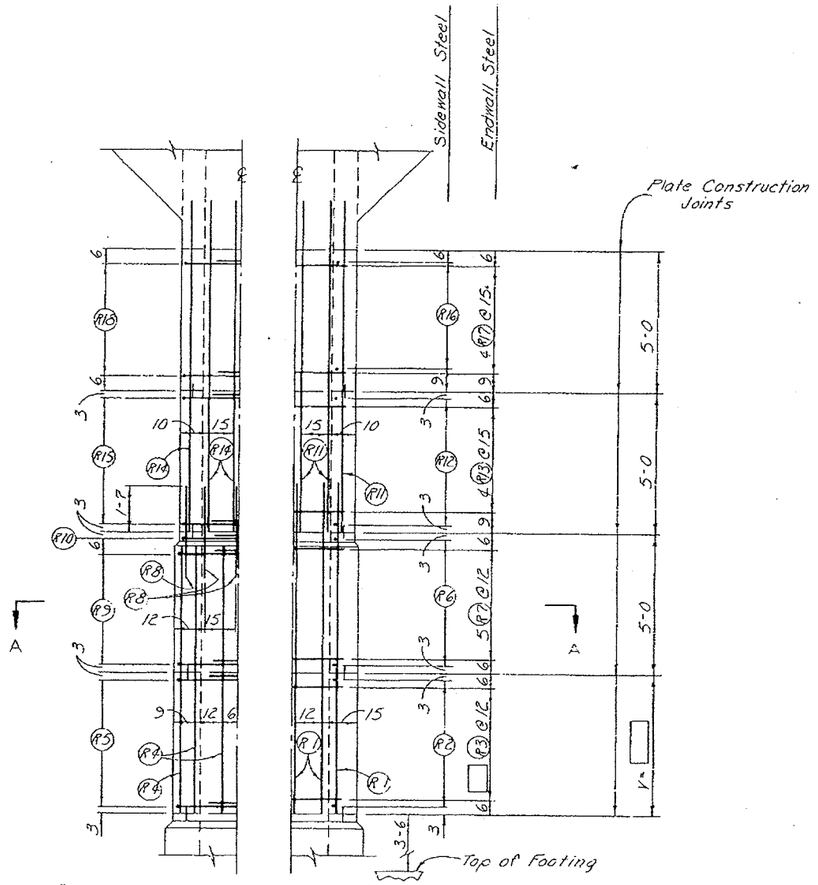
FORM SCS-ES-1 APRIL 1963



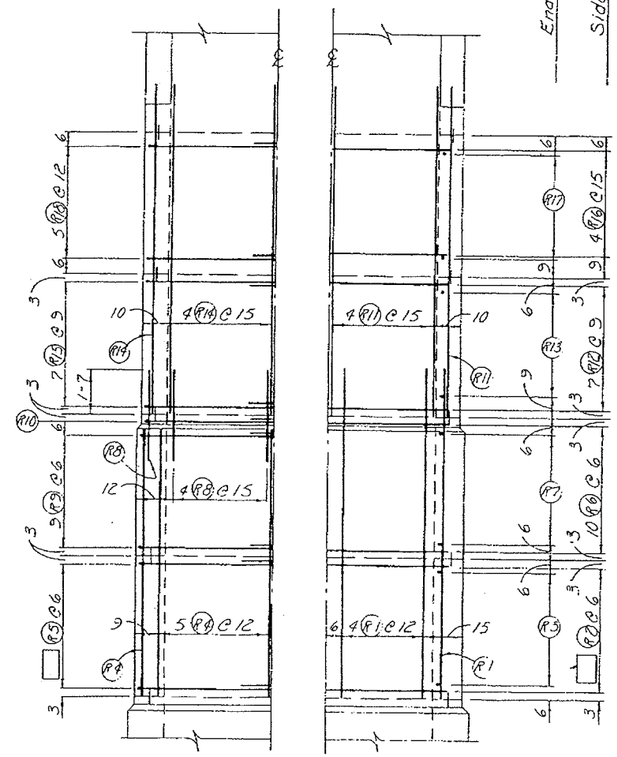
Outside Steel Inside Steel

SECTION A-A

0 1 2 3
Scale in Feet



ENDWALL ELEVATION



SIDEWALL ELEVATION

0 2 4
Scale in Feet
Unless Otherwise Shown

AS BUILT
CONSTRUCTION COMPLETED
9/6/79
Dale M. Wakefield

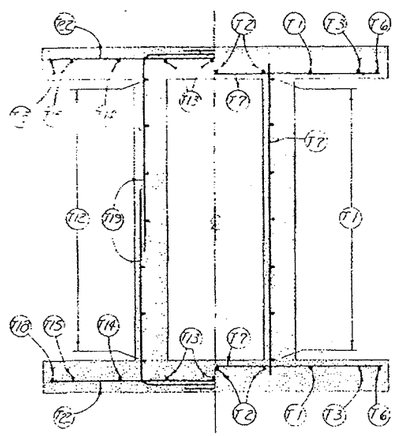
RISER DETAILS

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

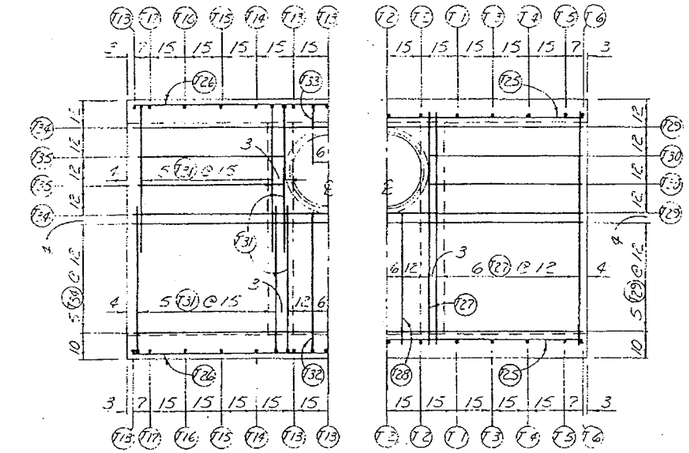
Designed K.J. Brust Date 3-75 Approved by
Drawn
Traced
Checked
Title
Sheet No. 9 of 28 Drawing No. 7-E-23089

FORM SCS 313 (APRIL 1963)



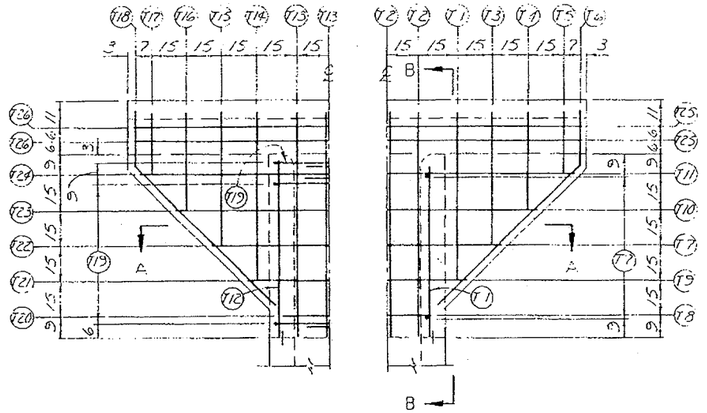
SECTION A-A

0 1 2 3
Scale in Feet



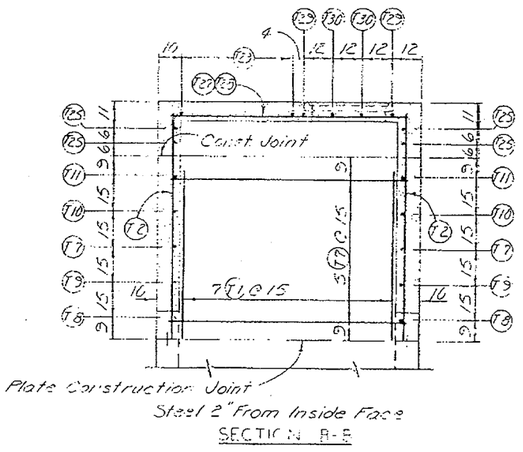
PLAN-COVER SLAB

Riser Sidewall Steel Not Shown



ENDWALL ELEVATION

Cover Slab Steel Not Shown



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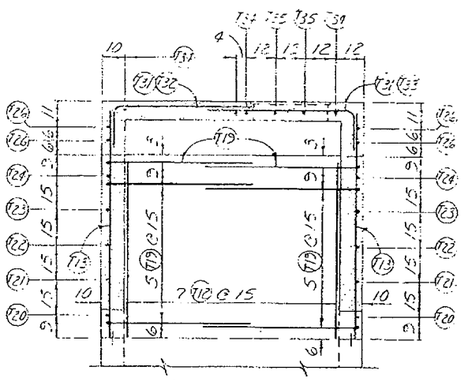

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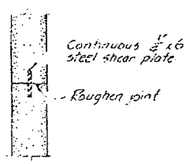


TYPICAL CONST JOINT DETAIL

(Not to scale)

0 2 4
Scale in Feet
Unless Otherwise Shown

AS BUILT

CONSTRUCTION COMPLETED 1-15-76

Walter W. Whitfield

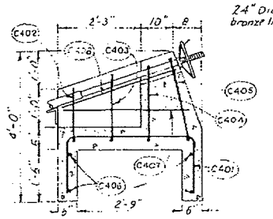
RISER DETAILS

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

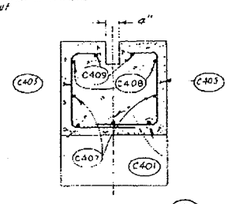
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust	Date 3-75	Approved by	
Drawn		Title	
Traced		Sheet No. 10 of 28 28	Drawing No. 7-E-23089
Checked			

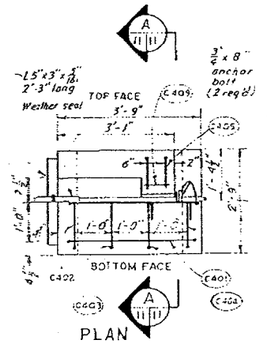
FORM SCS-ES (APRIL 1963)



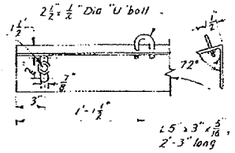
SECTIONAL ELEVATION



SECTION (A)



PLAN

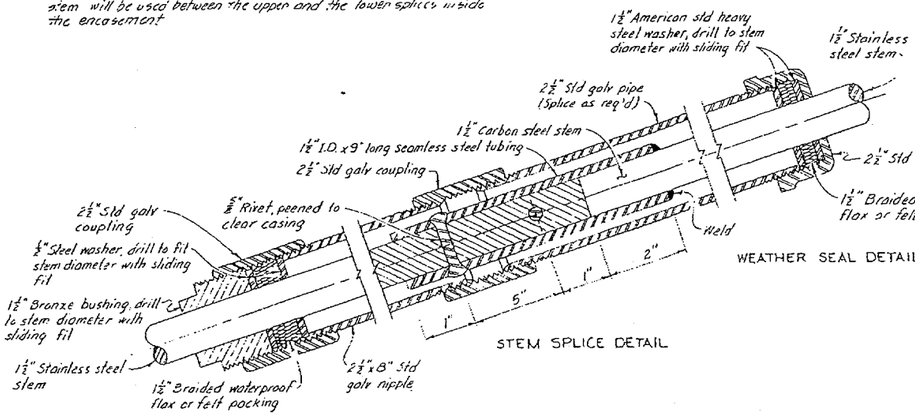


SUPPORT ANGLE DETAIL

GATE LIFT PEDESTAL DETAILS

Notes
Fill stem encasement with approximately 12 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



OIL SEAL DETAIL

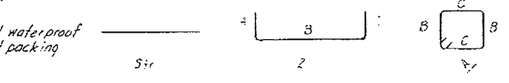
STEM SPLICE DETAIL

WEATHER SEAL DETAIL

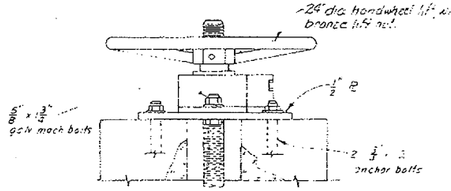
GATE STEM SPLICE

STEEL SCHEDULES

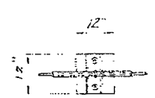
LOCATION	MARK	SIZE	QUAN	LENGTH	TYPE	A	B	C	TOTAL LENGTH
GATE STEM PEDESTALS (5 req'd)									
	P401	4	15	3'-0"	T1	0'-2"	0'-8"	0'-8"	45'-0"
	P402	4	10	2'-3"	Str				22'-6"
	P403	4	10	2'-9"	Str				27'-6"
GATE LIFT PEDESTAL									
	C401	A	3	6'-3"	2	1'-6"	3'-3"	1'-6"	18'-9"
	C402	A	1	4'-6"	2	1'-2"	2'-2"	1'-2"	4'-6"
	C403	A	1	5'-0"	2	1'-5"	2'-2"	1'-5"	5'-0"
	C404	A	2	4'-0"	2	1'-6"	0'-9"	0'-9"	8'-0"
	C405	A	2	4'-3"	2	1'-6"	2'-0"	0'-9"	8'-6"
	C406	A	4	2'-3"	Str				9'-0"
	C407	A	2	3'-0"	Str				6'-0"
	C408	A	2	2'-9"	Str				5'-6"
	C409	A	2	0'-6"	Str				1'-0"



BAR TYPES

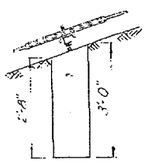


ELEVATION

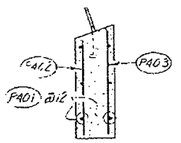


PLAN

CROSS SECTION

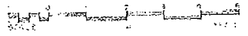


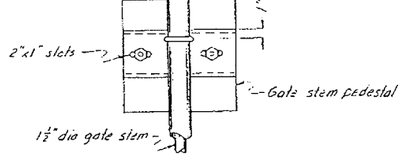
SIDE ELEVATION



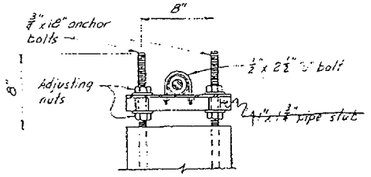
SECTIONAL ELEVATION

GATE STEM PEDESTAL

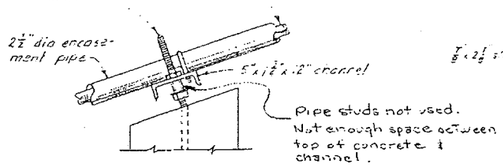




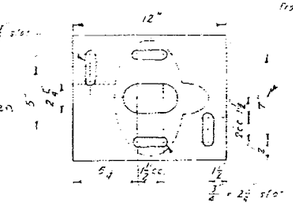
PLAN



ELEVATION



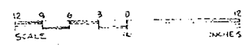
SIDE VIEW



PLAN

BASE PLATE
(For gate lift pedestal)
Not to scale.

GATE STEM GUIDE



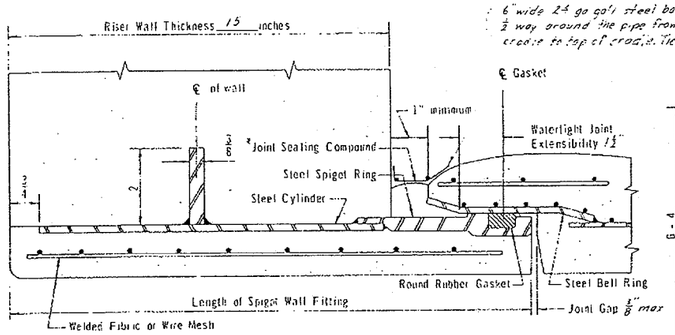
AS BUILT
CONSTRUCTION COMPLETED
5/15/76

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date	
Designed K.J. Brust	3-75	Approved by
Drawn L.W. M.S. Clintock	3-75	Title
Traced		Date
Checked		Sheet No. 11 of 28 sheets; Drawing No. 7-E-23089

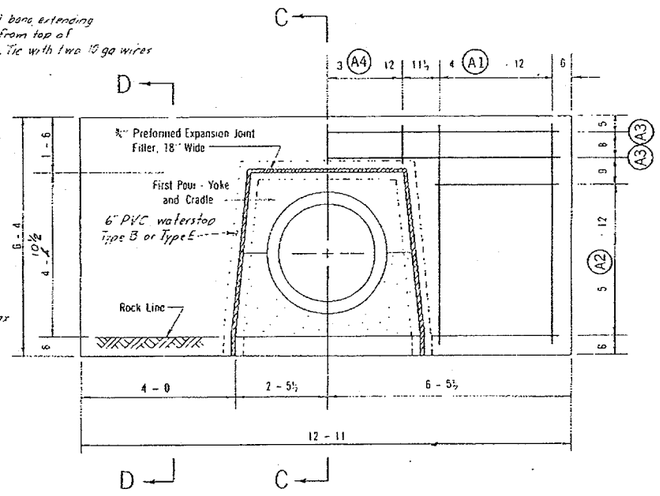
SCS ENG 313-B Rev. 7-69



DETAIL A

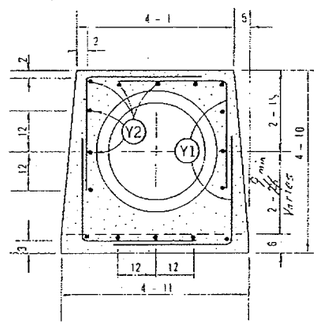
* ¾" dia circular cross section, Butyl rubber sealer, Federal Specification SS-S-00210 applied over manufacturers recommended primer.

6" wide 24 ga galv steel band extending ¾ way around the pipe from top of cradle to top of cradle. Tie with two 10 ga wires



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	6	5 - 10	1	40	~~233-4~~
A2	4	10	3 - 5	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	2†	60	340-0
Y2	4	16	1 - 2	1	80	93-4

QUANTITIES

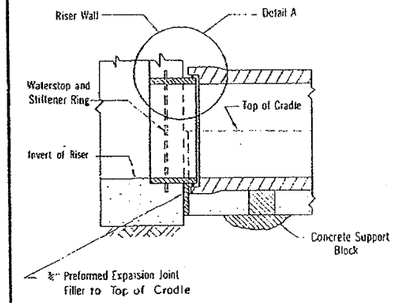
Concrete	Cu. Yds.
Anti-seep Collar including Yoke	
* Each, Additional to Cradle	1.91
Total	9.55
Cradle	
** Per Lineal Foot of Cradle	0.26
Total	47.6
Steel	**Pounds**
Anti-seep Collar including Yoke	662
Cradle	—

Concrete quantities are based on an outside diameter of pipe of 35 inches.
Steel quantities do not change with outside diameter of pipe.

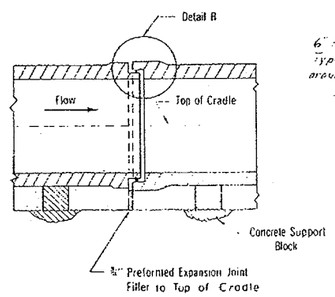
* This quantity is given by
1.875 − 0.0001515 (D$_f$ + 38) (D$_f$ − 36) cu yds

** This quantity is given by
0.2392 − 0.0001010 (D$_f$ + 38) (D$_f$ − 36) cu yds

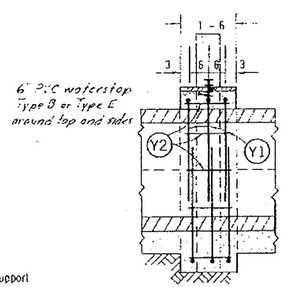
D$_f$ = outside diameter of pipe furnished, inches.



DETAIL OF SPIGOT WALL FITTING

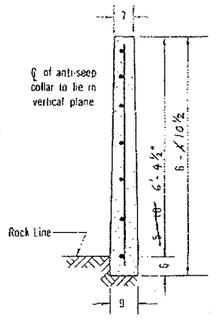


DETAIL OF PIPE JOINT

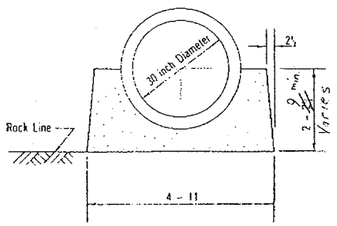


SECTION C-C

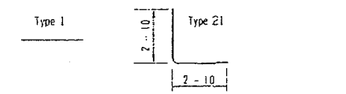
Anti-seep collar steel not shown.



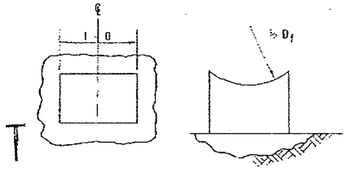
SECTION D-D



DETAIL OF CRADLE



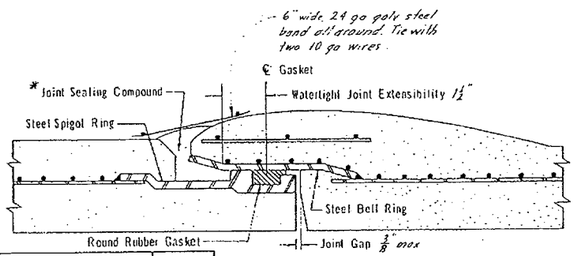
BAR TYPES



AS BUILT
CONSTRUCTION COMPLETED
9/15/76

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.



DETAIL B

Joint length equals watertight joint extensibility plus joint gap.

The pipe shall be drawn together so that the maximum joint gap does not exceed ½ 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 ½ inch.

JOINT REQUIREMENTS

Length of Pipe Section	Minimum Joint Length	Minimum Joint Limiting Angle	
feet	inches	radians	degrees
16.0	1⅜		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		
30.0	37	6200	8200

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.

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

STANDARD CONDUIT DETAILS
FOR
REINFORCED CONCRETE PRESSURE PIPE
PRINCIPAL SPILLWAY

STANDARD DWG. NO. ES-5030-CR

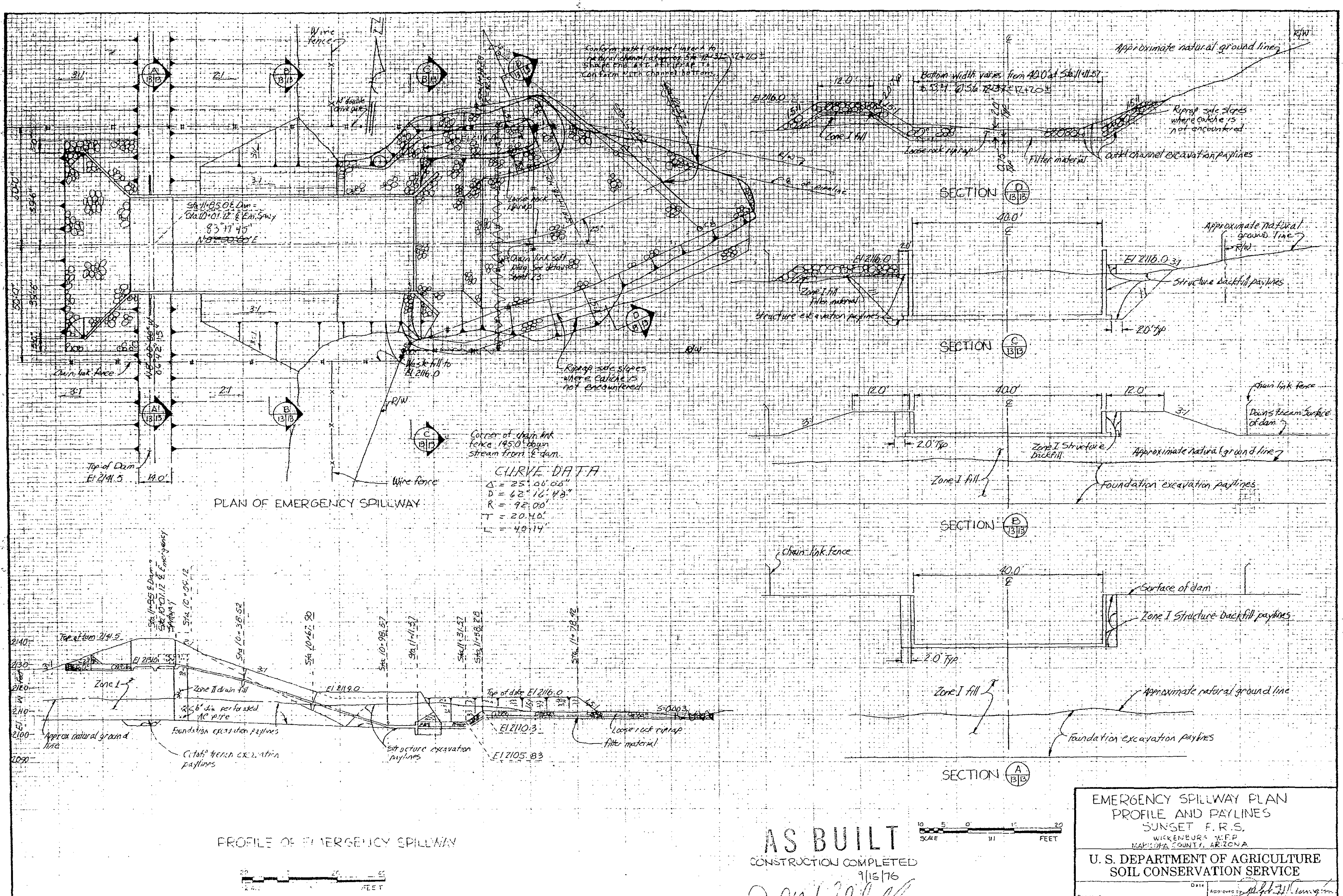
DATE 2-70 SHEET 1 OF 1

PRINCIPAL SPILLWAY CONDUIT DETAILS
SUNSET F.R.S
WICKENBURG WRP
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date	Approved by
Designed K.J. Brust, CMC	3-75	Title
Drawn L.W. McClintock	3-75	Title
Traced		Title
Checked GMC	4-75	Sheet No. 12 of 25 sheets

Drawing No. 7-E-23089



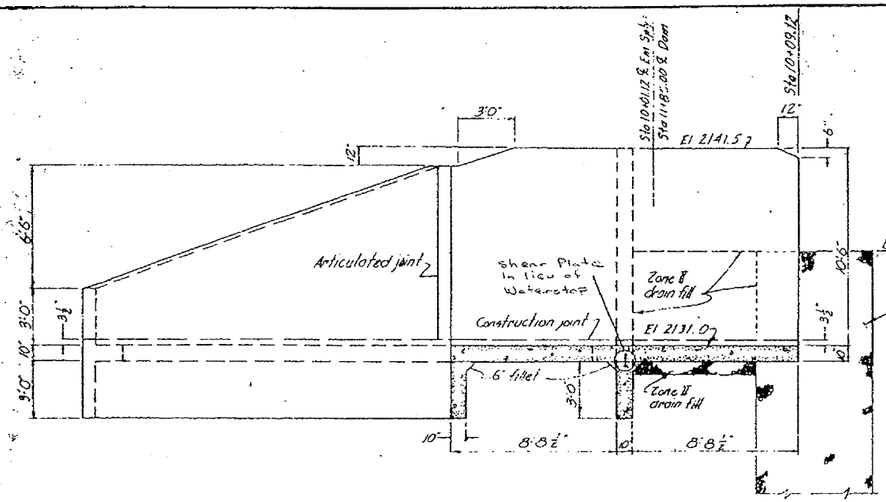
AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Dal N. Wakefield

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

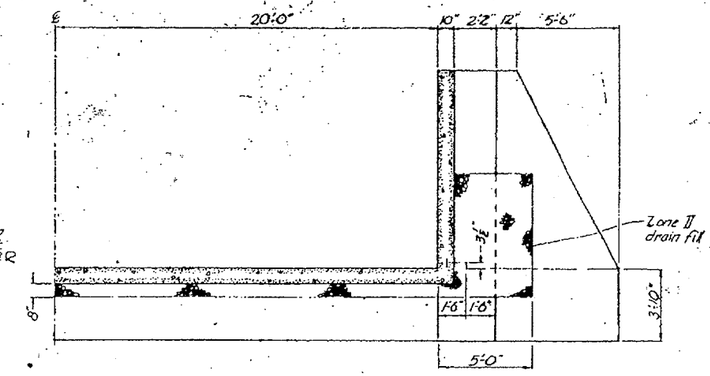
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed	Approved by [illegible]
Drawn	Title
Traced Richard A Carter 5-76	Title
Checked [illegible]	Sheet / Drawing No. [illegible]

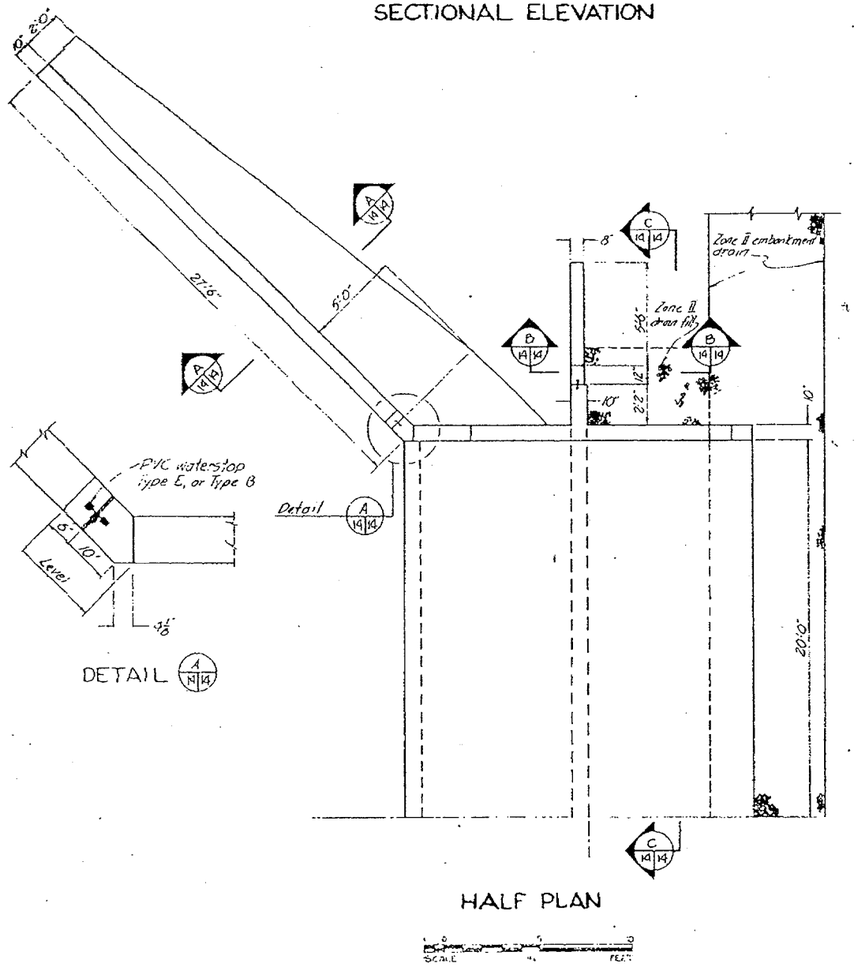
FORM SCS-315 (APRIL 1963)



SECTIONAL ELEVATION



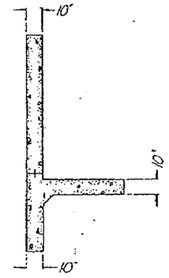
SECTION C 14 14



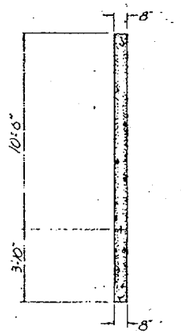
DETAIL A 14 14

HALF PLAN

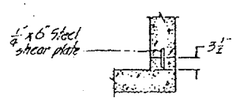
SCALE 0 4 8 FEET



SECTION A 14 14



SECTION B 14 14



TYPICAL CONSTRUCTION JOINT DETAIL

AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Dale N. Wakefield

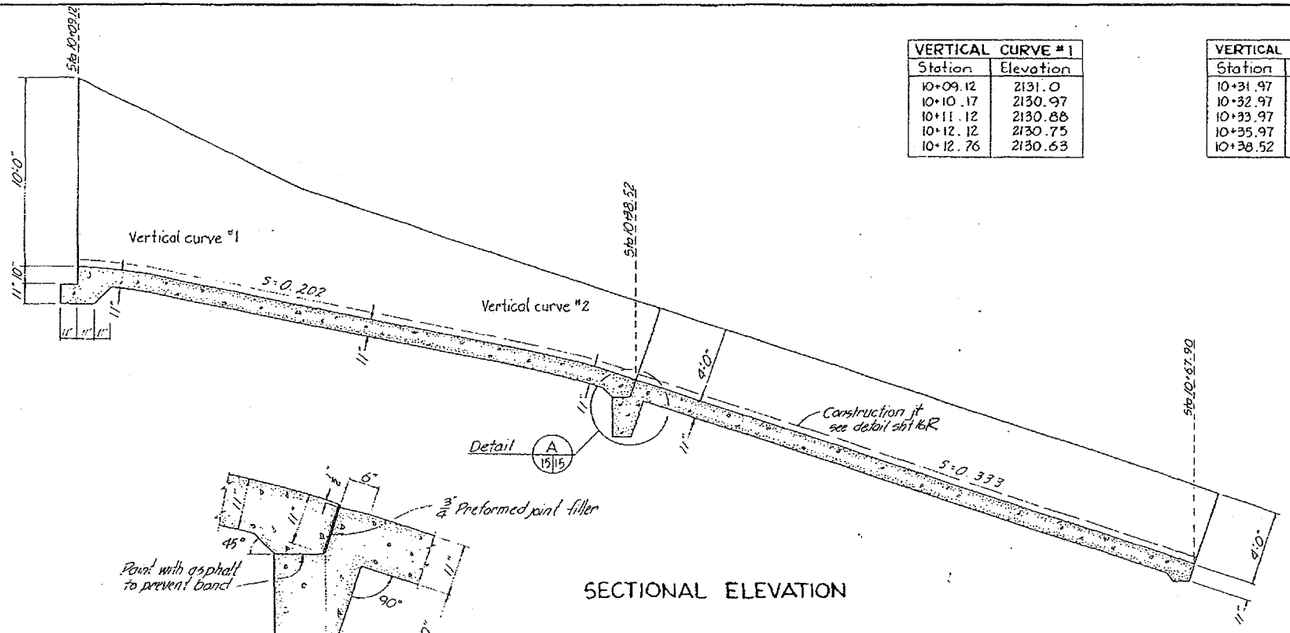
EMERGENCY SPILLWAY LAYOUT
SUNSET F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA
U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed Rick Helm	4-75	Approved	
Drawn Jack Land	4-75	Title	
Traced		Title	
Revision Checked T.E. Jayo	6-76	Sheet No 14 of 24 Sheets	Drawing No 7-E-23089

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

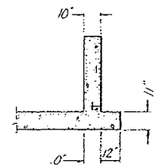
VERTICAL CURVE # 1	
Station	Elevation
10+09.12	2131.0
10+10.17	2130.97
10+11.12	2130.88
10+12.12	2130.75
10+12.76	2130.63

VERTICAL CURVE # 2	
Station	Elevation
10+31.97	2126.75
10+32.97	2126.54
10+33.97	2126.31
10+35.97	2125.78
10+38.52	2125.0



SECTIONAL ELEVATION

DETAIL (A 15/15)



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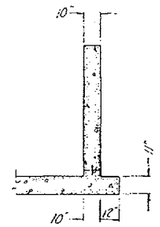

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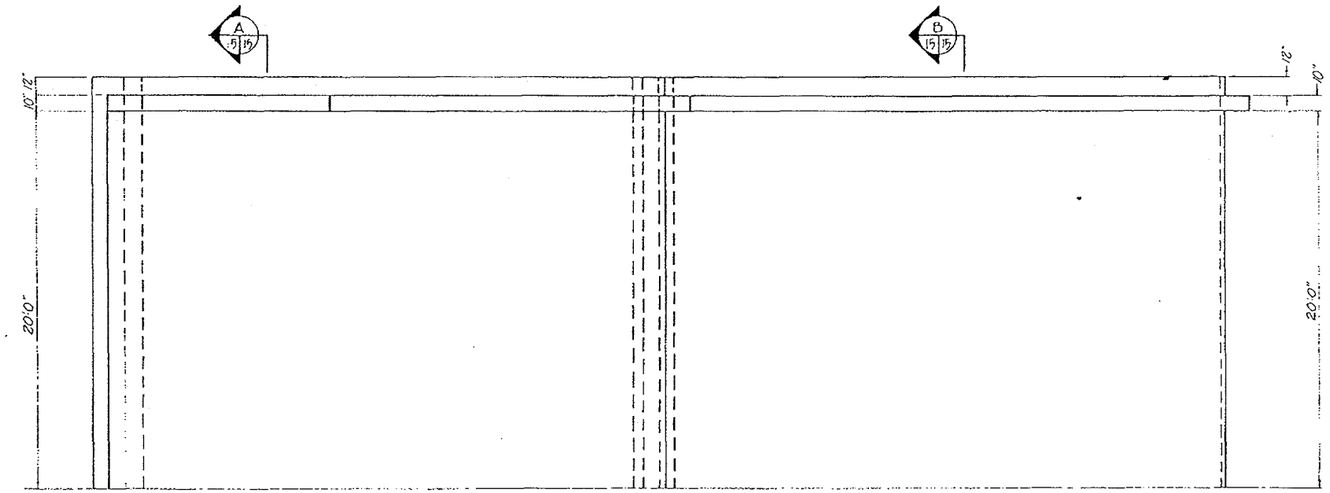


HALF PLAN

SCALE 0 5 10 FEET

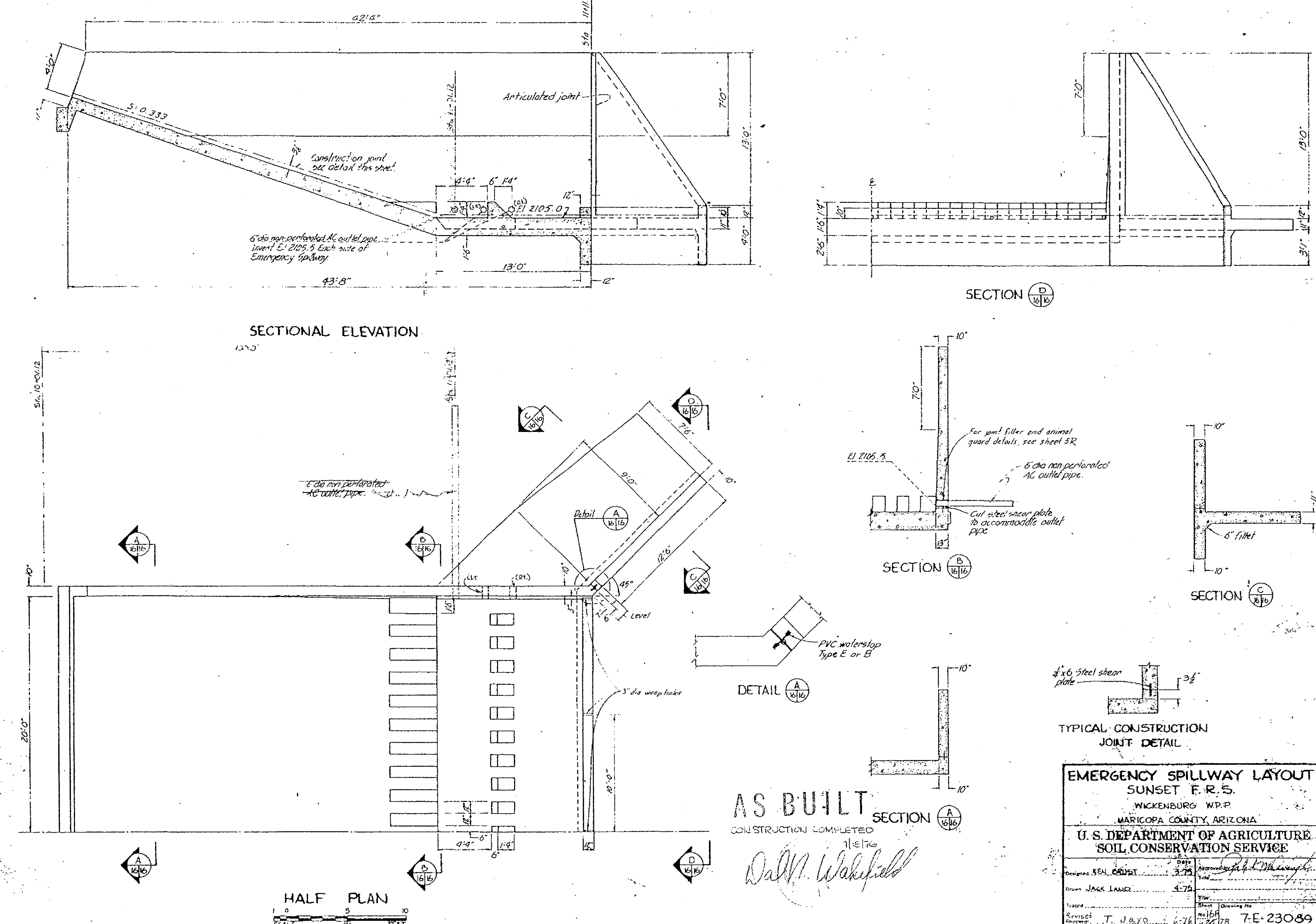
AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Dale N. Wakefield

EMERGENCY SPILLWAY LAYOUT
SUNSET F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed KEN BRUSI	4-75	Approved by	
Drawn JACK LAND	4-75	Title	
Traced		Sheet No. 15 of 28	Drawing No. 7-E-23089
Checked			

SCS ENG-313-B Rev. 7-69



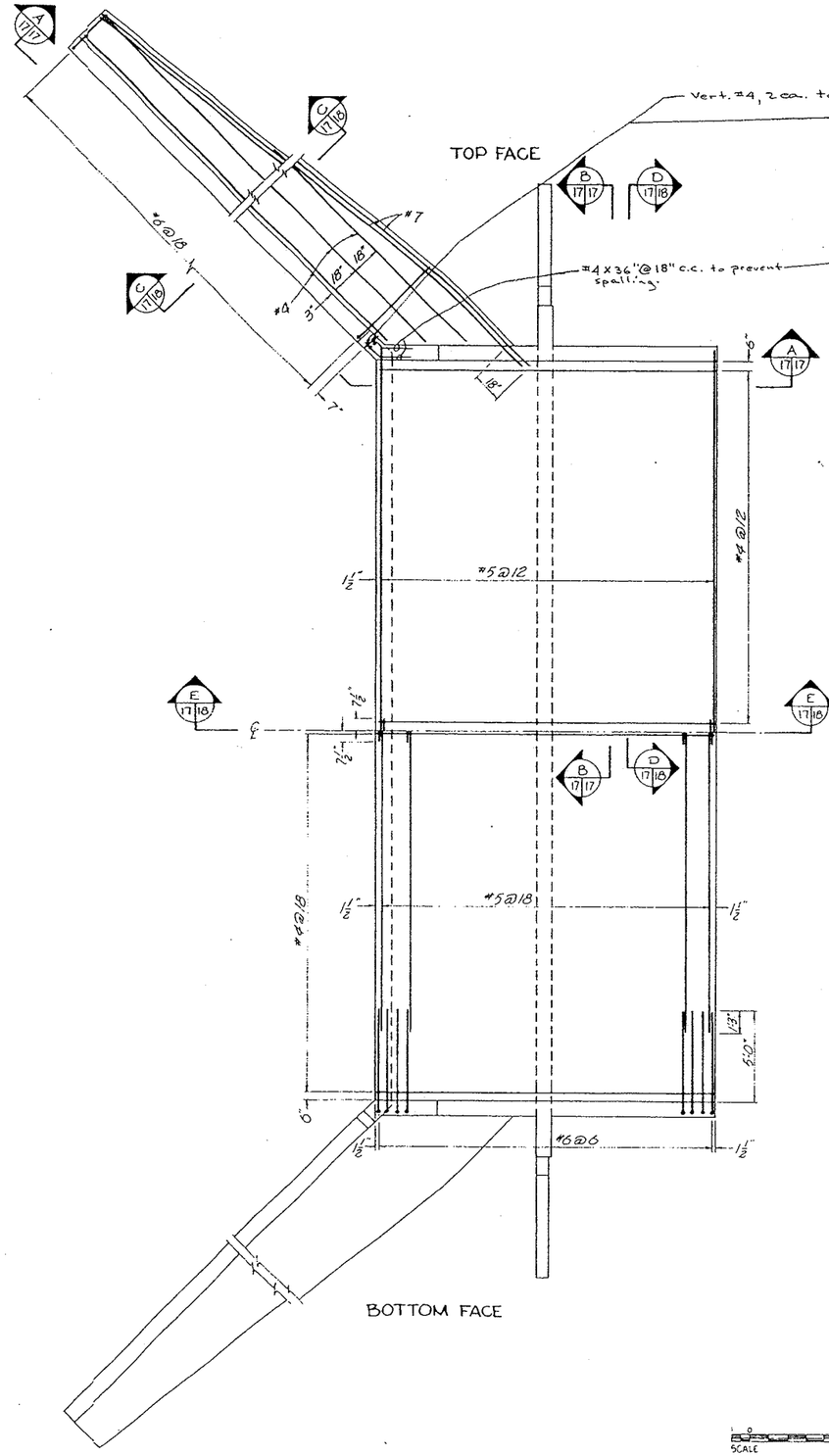
AS BUILT
CONSTRUCTION COMPLETED
7/15/76
[illegible]

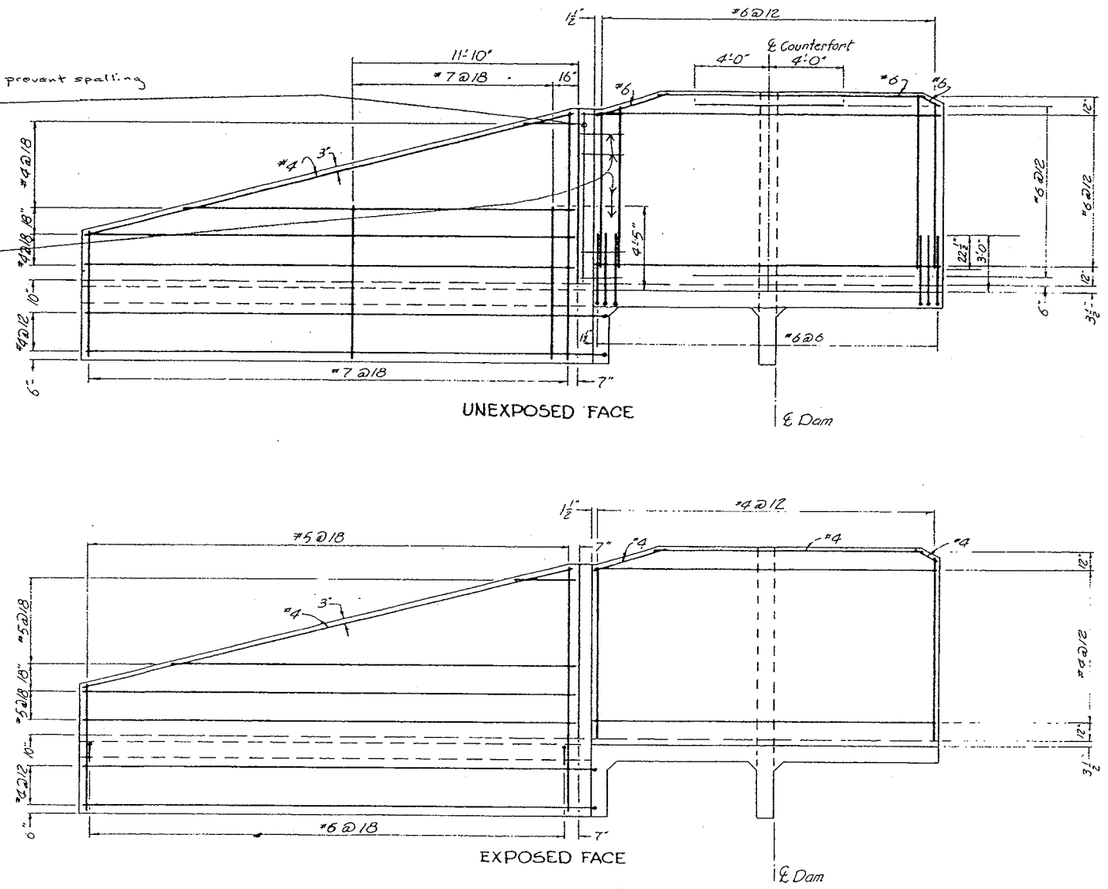
EMERGENCY SPILLWAY LAYOUT
SUNSET F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

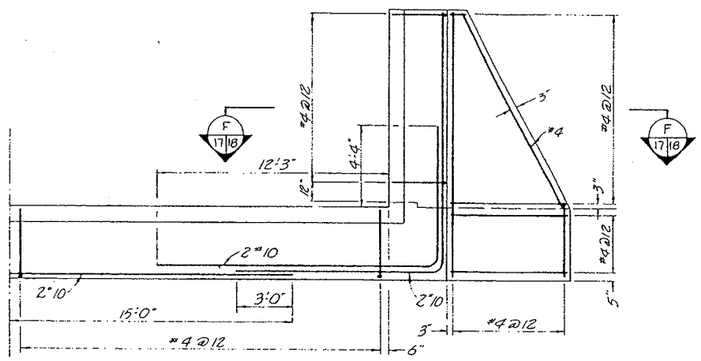
Designed KEN BRUST	Date 3-75	Approved by [illegible]
Drawn JACK LAND	4-75	Title
Traced		Title
Revised T. J.a.y.o	6-76	Sheet No 16R of 28 — Drawing No 7-E-23089

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





SECTION A 17|17



SECTION B 17|17

SCALE 0 5 10 IN FEET

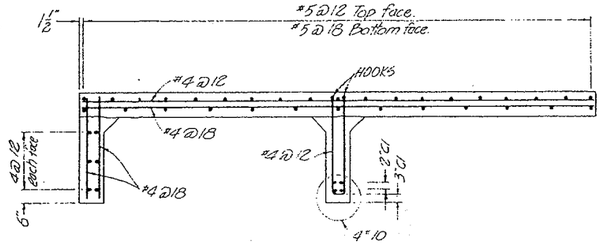
AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Walt M. Wakefield

EMERGENCY SPILLWAY
INLET DETAILS
SUNSET F.R.S
WICKENBURG W.R.P.
MARICOPA COUNTY, ARIZONA

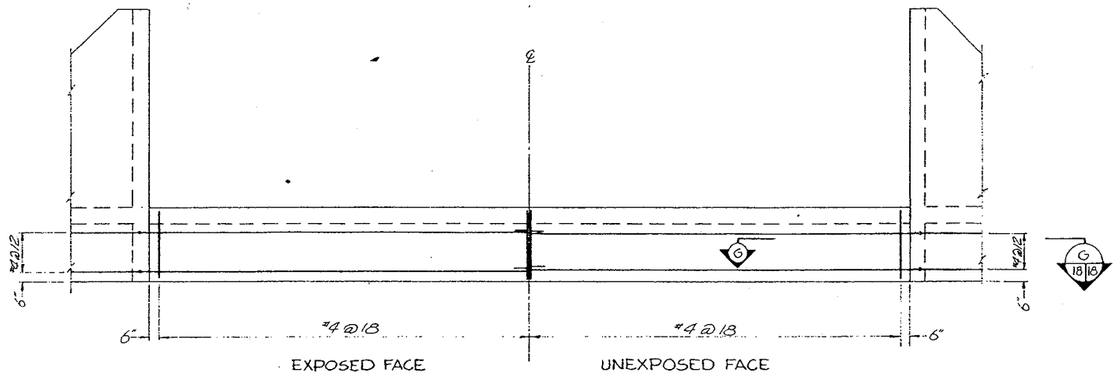
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed R.L. HELM	4-75	Approved by	
Drawn J.D. LAND	4-75	Title	
Traced		Title	
Checked		Sheet No. 17 of 28	Drawing No. 7-E-23089

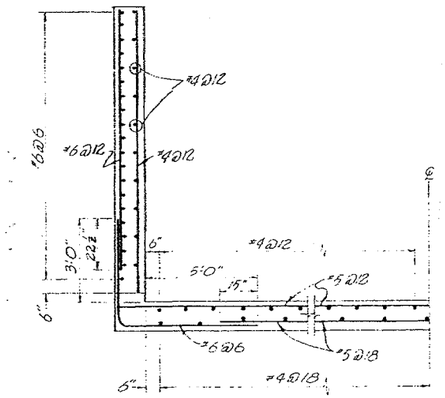
SCS-ENG-313-B Rev. 7-69



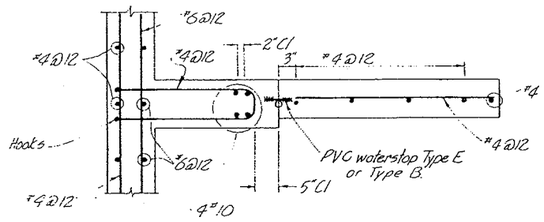
SECTION E 17/18



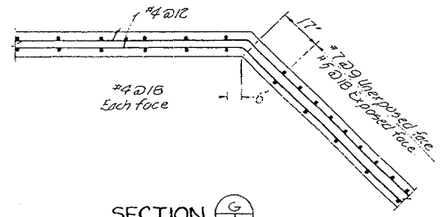
UPSTREAM ELEVATION



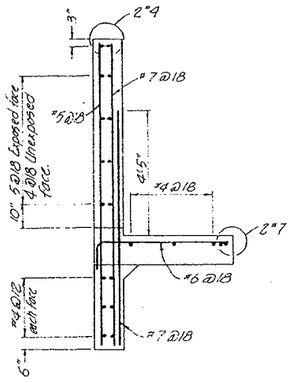
SECTION D 17/18



SECTION F 17/18



SECTION G 18/18



SECTION C 17/10

SCALE 0 5 IN FEET

SCALE 12 6 0 12 24 36 IN INCHES

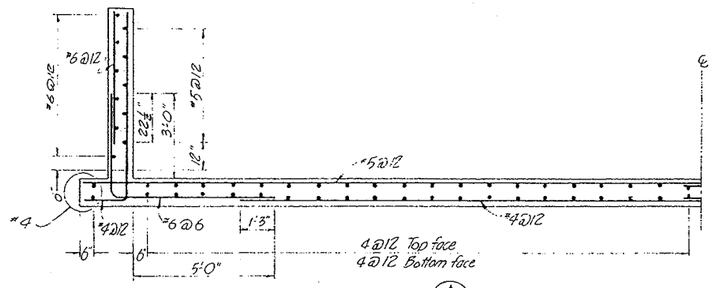
AS BUILT
CONSTRUCTION COMPLETED
9/15/76

EMERGENCY SPILLWAY
INLET DETAILS
SUNSET F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

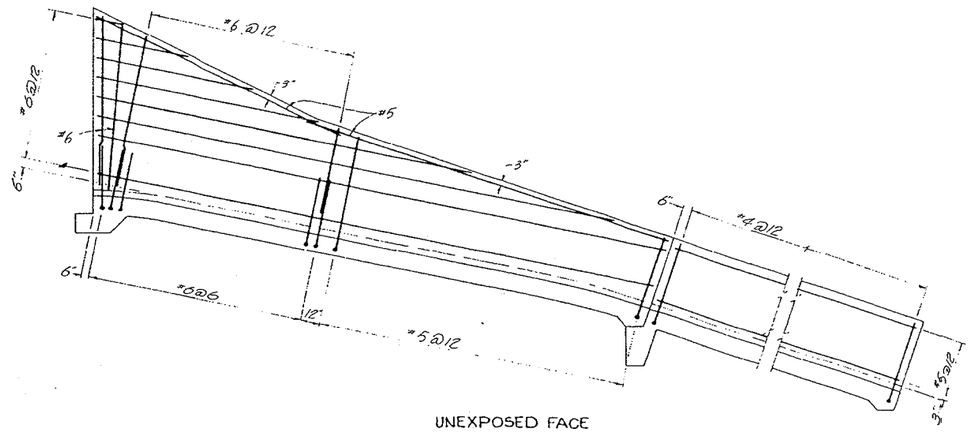
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed R.L. Helm	4-75	Approved by	
Drawn J.D. Land	4-75	Title	
Traced		Title	
Checked		Sheet No 18 of 28	Drawing No 7-E-23089

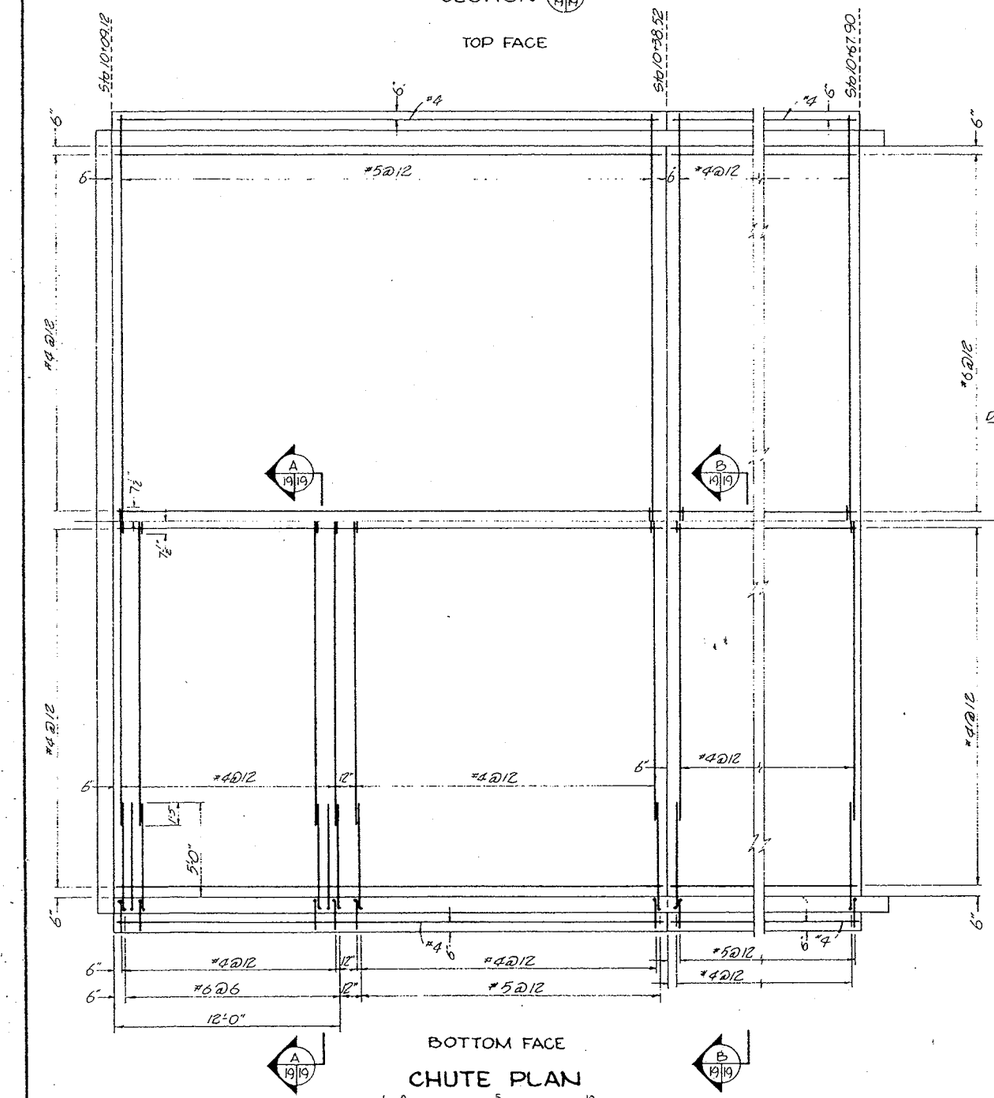
SCS-ENG-313-B Rev. 7-69



SECTION (A 19|19)

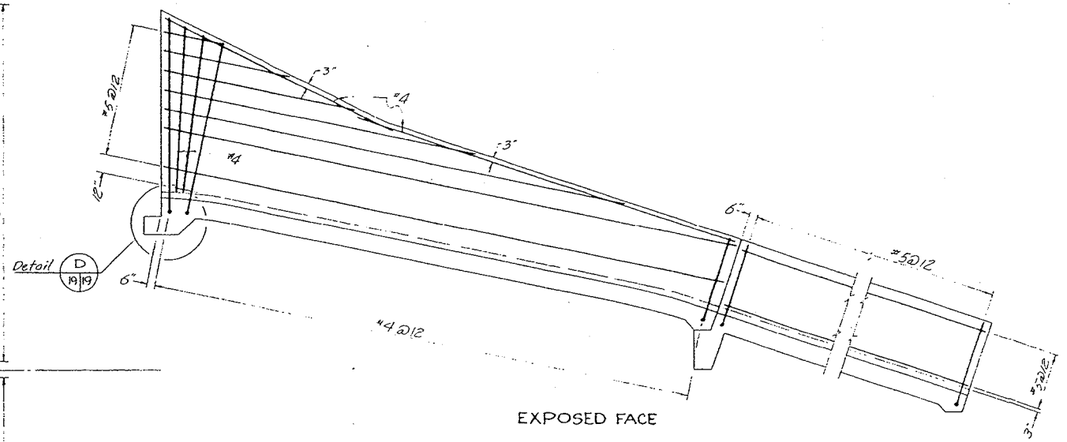


UNEXPOSED FACE



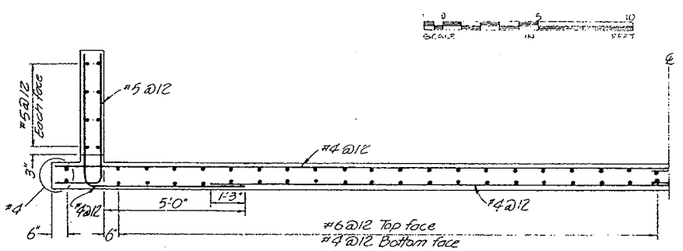
BOTTOM FACE

CHUTE PLAN



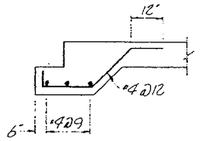
EXPOSED FACE

SIDEWALL ELEVATION



SECTION (B 19|19)

AS BUILT
CONSTRUCTION COMPLETED
7/16/76



DETAIL (D 19|19)

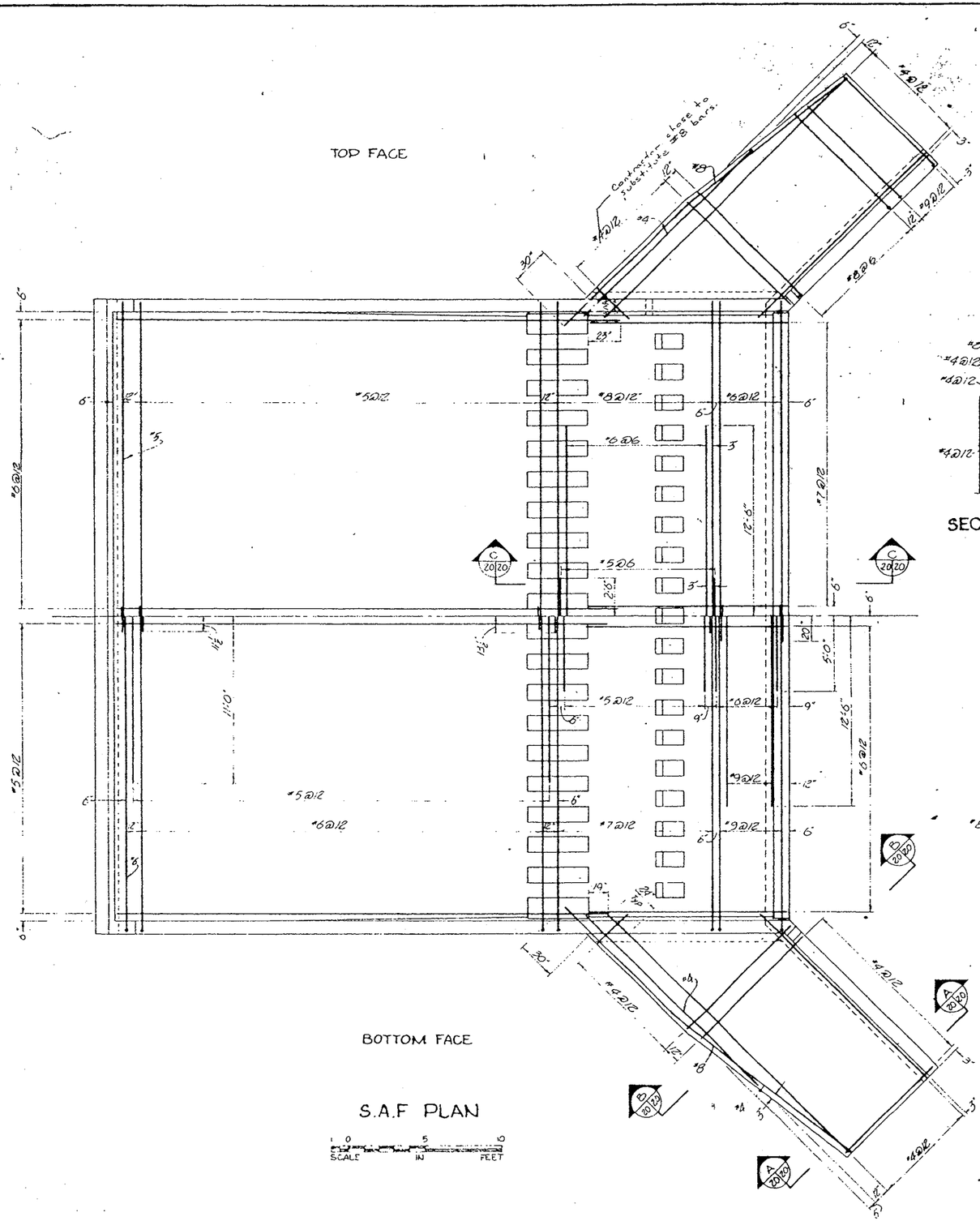
EMERGECY SPILLWAY
CHUTE DETAILS
SUNSET F.R.S.
WICKENBURG W.D.D.
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed KEN BRUST	4-75	Approved by	
Drawn JACK LAUD	4-75	Title	
Traced		Title	
Checked		Sheet No. 19 of 28 sheets	Drawing No. 7-E-23089

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

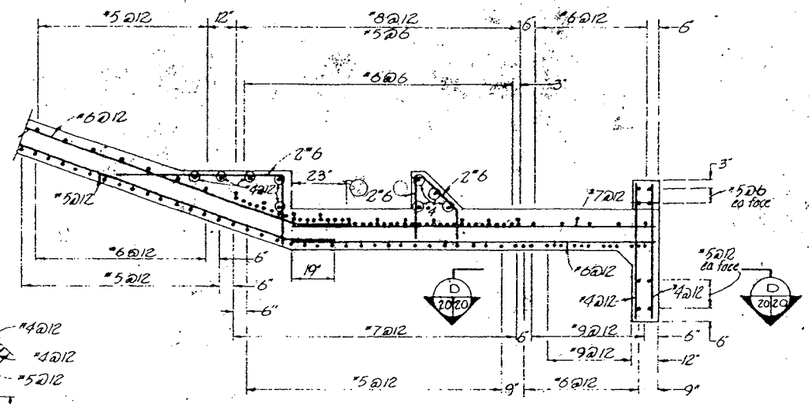
TOP FACE



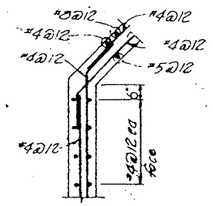
BOTTOM FACE

S.A.F PLAN

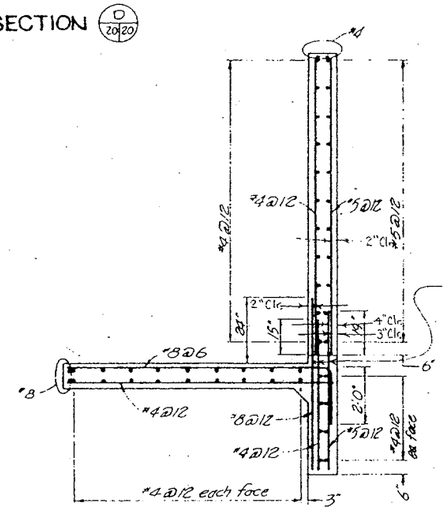
SCALE 1" 0 5 10 FEET



SECTION C 20|20

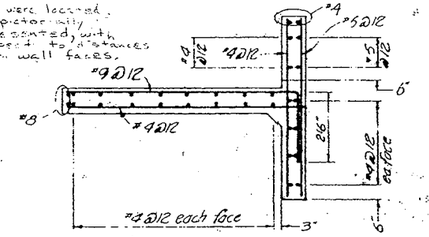


SECTION D 20|20



SECTION B 20|20

Bars were located as pictorially represented, with respect to distances from wall faces.



SECTION A 20|20

SCALE 1" 0 5 FEET

Note: Cut all steel 2" clear of weepholes and 6" dia AC outlet pipe.

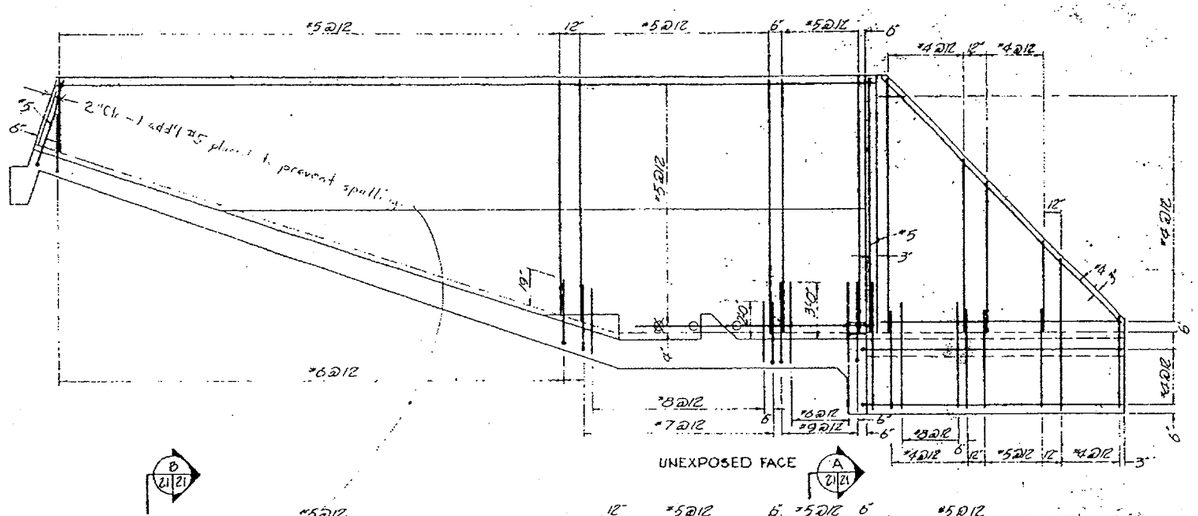
AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Dale M. Wakefield

EMERGENCY SPILLWAY S.A.F. DETAILS
SUNSET F.R.S.
WICKENBURG W.P.P.
MARICOPA COUNTY, ARIZONA

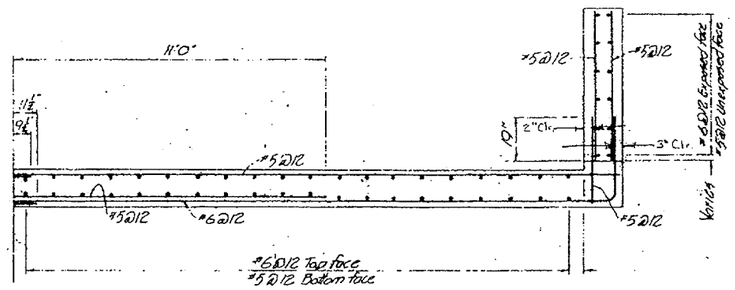
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed KEN BRUST	Date 4-75	Approved	
Drawn JACK LAND	4-75	Title	
Traced		Title	
Revised Checked T. UZYO	6-76	Sheet No 20 of 28	Drawing No 7-E-23009

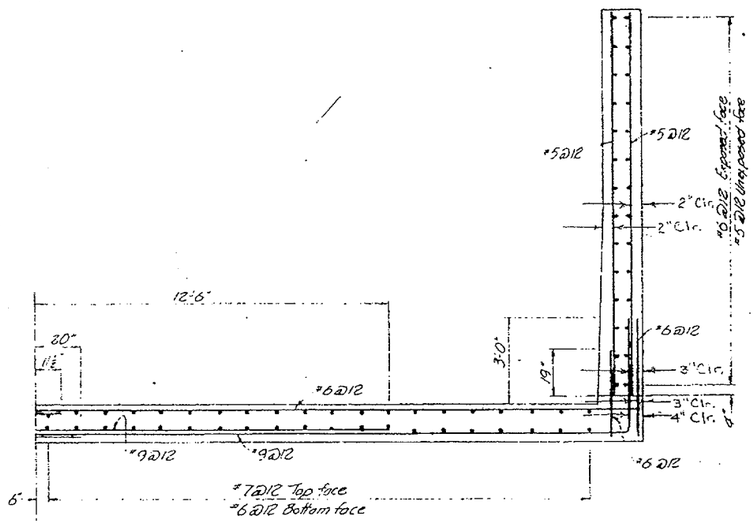
SCS-ENG-313-B Rev. 7-69

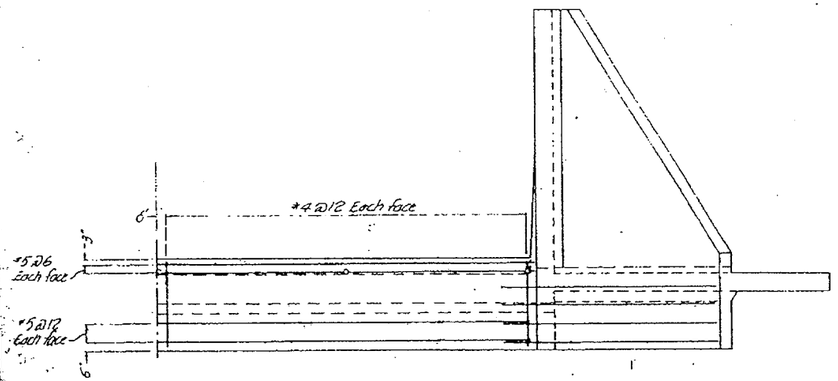


SECTIONAL ELEVATION

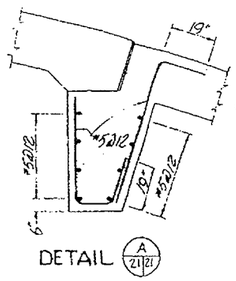


SECTION (B 21|21)





CUTOFF WALL ELEVATION



DETAIL (A 21|21)

AS BUILT
CONSTRUCTION COMPLETED
11-5-76
Walt Wakefield

SECTION (A 21|21)

SCALE 0 5 10 IN FEET

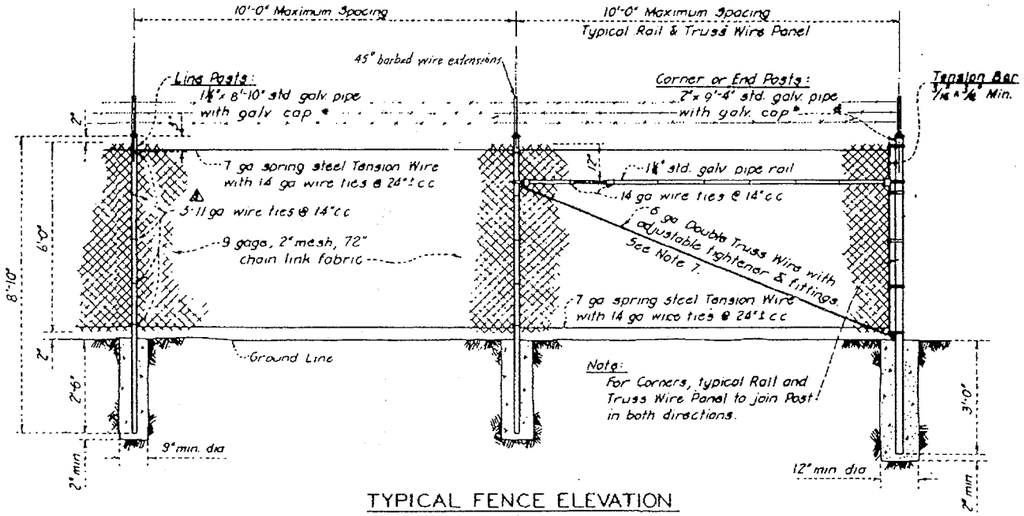
Note Cut all steel 2" clear of weepholes and 6" dia AC outlet pipe.

EMERGENCY SPILLWAY
S.A.F. DETAILS
SUNSET F.R.S.
WICKENBURG W.P.D.
MARICOPA COUNTY, ARIZONA

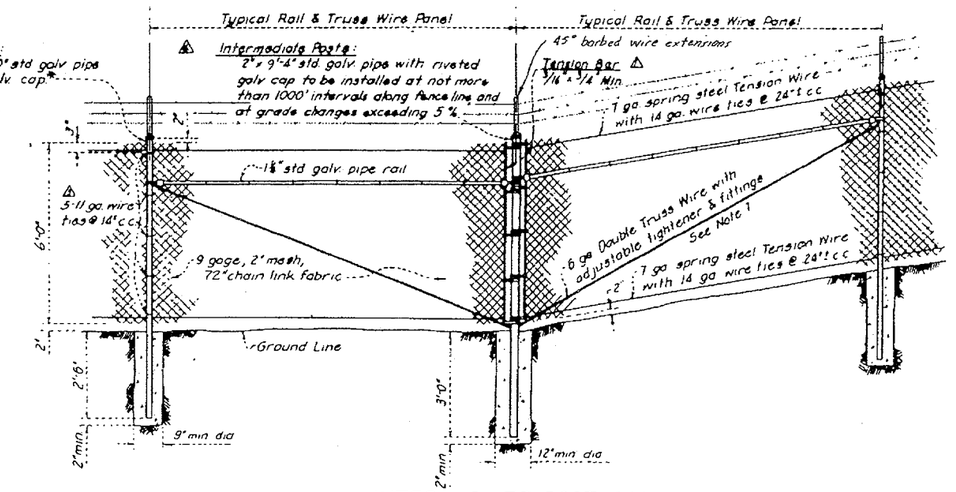
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed KEN BRUST	4-75	Approved	
Drawn JACK LAND	4-75	Title	
Traced		For	
Revised T. Sayo	6-76	Sheet No. 21B of 28	Drawing No. 7-E-23089

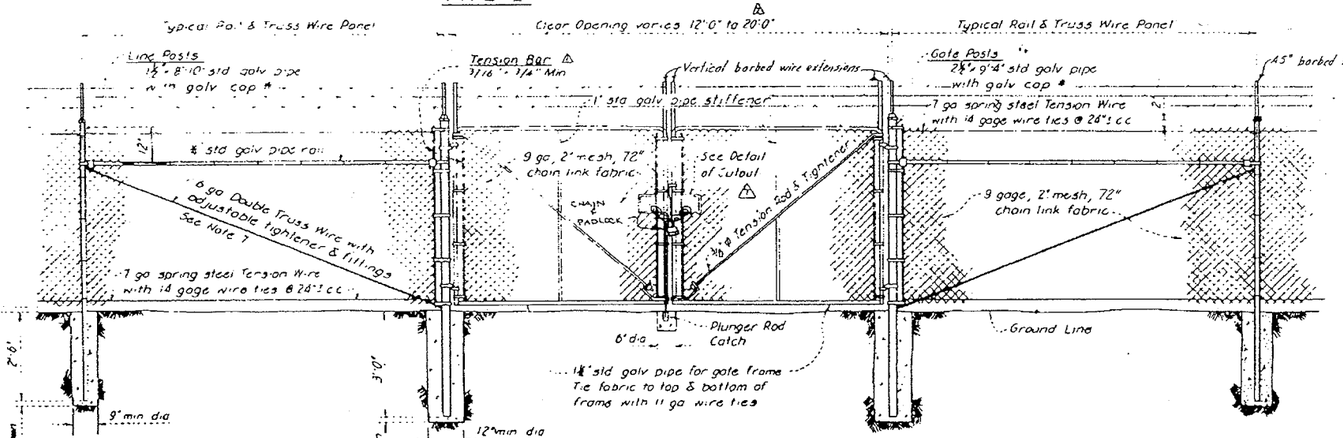
SCS-ENG-313-B (Rev. 1-69)

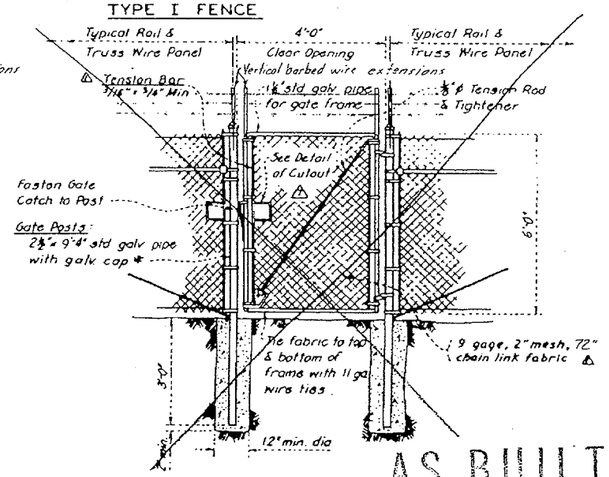


TYPICAL FENCE ELEVATION
TYPE I



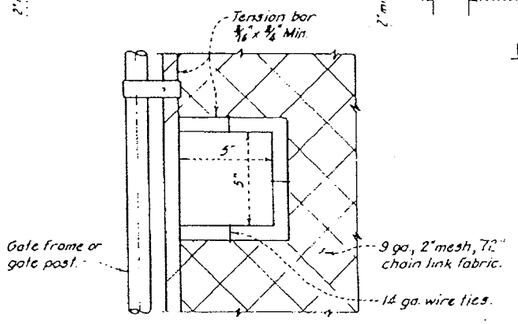
INTERMEDIATE POST DETAIL
TYPE I FENCE





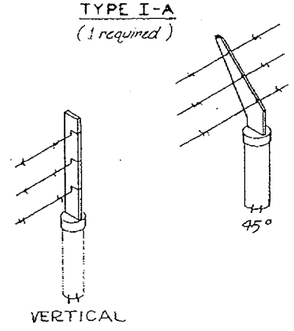
~~TYPICAL WALK GATE~~
~~TYPE I-B~~

AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Dan W. Whitfield



⚠ DETAIL OF CUTOUT FOR CHAIN AND LOCK
SCALE: NONE

DOUBLE DRIVE GATE DETAIL
TYPE I-A
(1 required)



BARBED WIRE EXTENSION ARM DETAILS

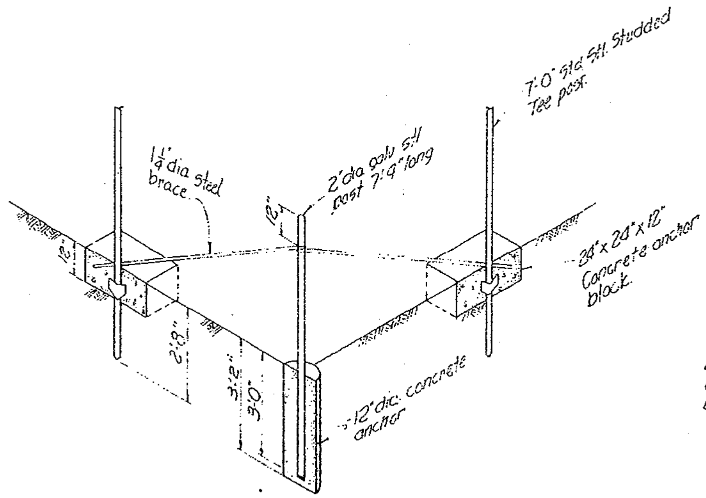
GENERAL NOTES

1— 1" std pipe shall be of minimum weight of 1.68 lbs. per lin ft before galvanizing
2— 1⅝" std pipe shall be of minimum weight of 2.27 lbs per lin ft before galvanizing
3— 1½" std pipe shall be of minimum weight of 2.72 lbs per lin. ft. before galvanizing
4— 2" std pipe shall be of minimum weight of 3.65 lbs per lin. ft before galvanizing
5— 2½" std pipe shall be of minimum weight of 5.79 lbs per lin ft. before galvanizing
6— Steel tubing may be used in lieu of standard pipe, provided that: it shall be electric welded or seamless; the weight per foot shall be not less than weights shown on the drawing; the OD 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 sq ft total coated surface.
7— Adjustable tighteners shall be turnbuckle or equivalent, having 6-inch min. take up
8— All gate hinges shall be heavy duty, malleable iron or steel, industrial service type 270 degree swing of approved quality and design.
9 — Secure galv cap to Post with ⅛-inch round head rivets.

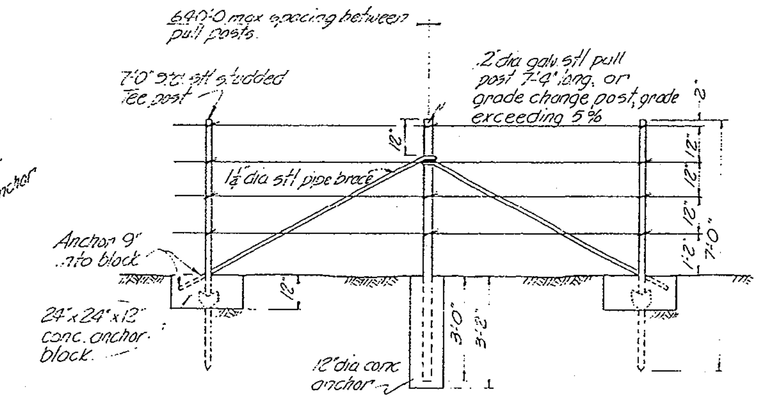
FENCING DETAILS
SUNSET F.R.S.
WICKENBURG WPP
MARICOPA COUNTY, ARIZONA

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

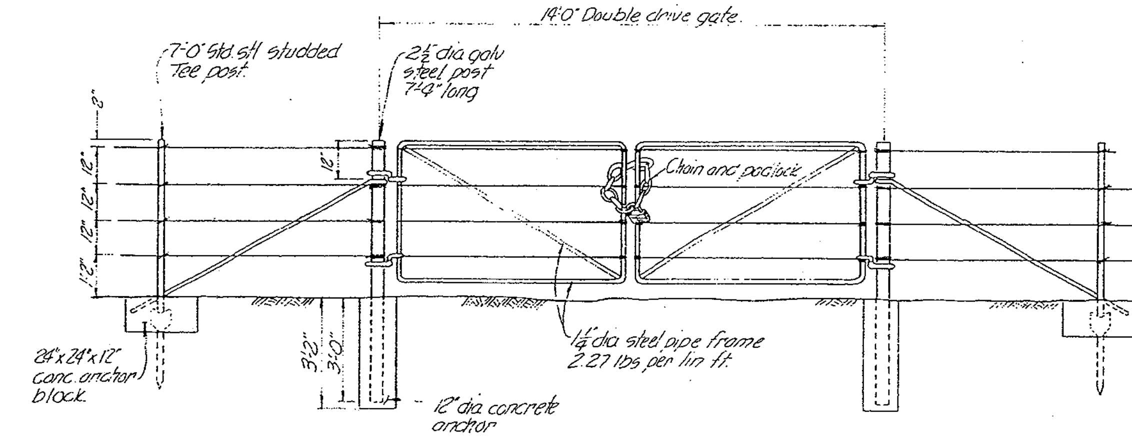
Designed K.J. Brust	Date 3-75	Approved by	
Drawn		Title	
Traced		Title	
Checked		Sheet No. 22 of 25 28	Drawing No. 7-E-23089



END OR CORNER POST ASSEMBLY

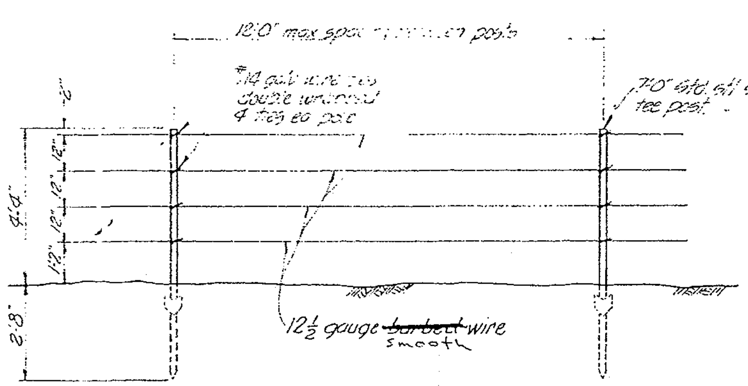


PULL POST OR CHANGE IN GRADE ASSEMBLY

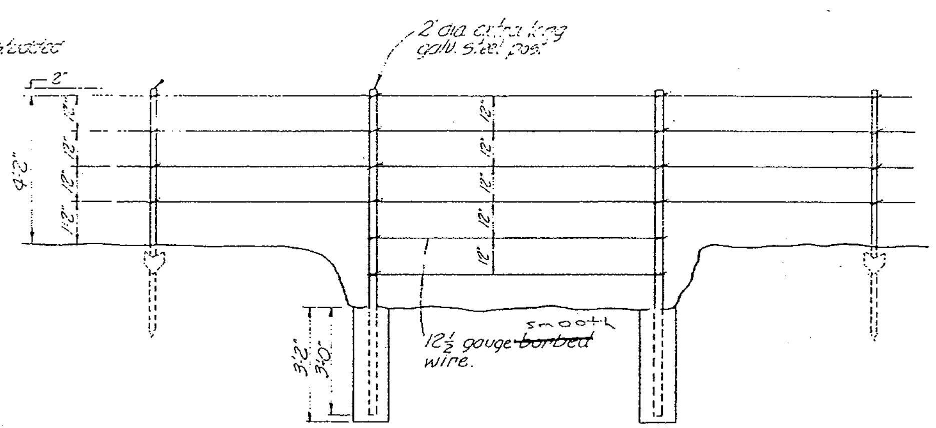


GATE AND POST ASSEMBLY

(One Gate Required)

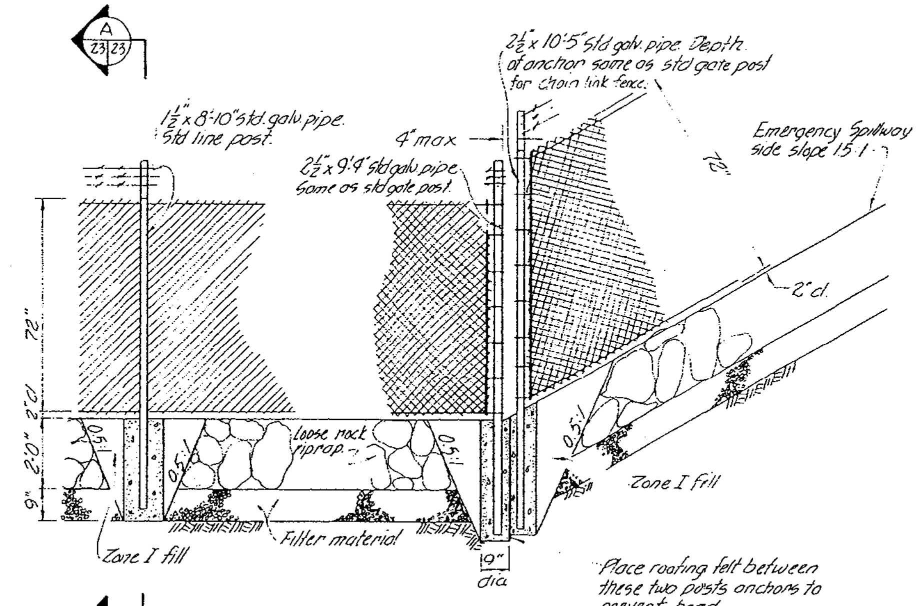


LINE POST ASSEMBLY



WATERCOURSE OR DEPRESSION CROSSING ASSEMBLY

Drawings on this sheet not to scale.



A 23 23 CHAIN LINK FENCE SOFT PLUG ASSEMBLY

Notes:

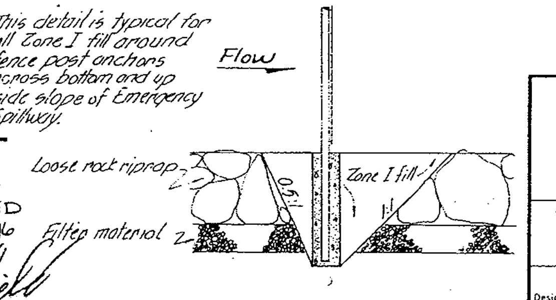
1. 1¼" dia std pipe shall be a minimum weight of 2.27 lbs per lin ft before galvanizing.
2. 1½" dia std pipe shall be a minimum weight of 2.72 lbs per lin ft before galvanizing.
3. 2" dia std pipe shall be a minimum weight of 3.65 lbs per lin ft before galvanizing.
4. 2½" dia std pipe shall be a minimum 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

[illegible]

Professional Engineer Certificate No. 7661 DALE WAKEFIELD Arizona, U.S.A.

This detail is typical for all Zone I fill around fence post anchors across bottom and up side slope of Emergency Spillway.



SECTION A 23 23

FENCING DETAILS			
SUNSET 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. 23 of ~~25~~ 28	Drawing No. 7-E-23089

USDA-SCS-PORTLAND, OREG. 1975

U.S. GOVERNMENT PRINTING OFFICE

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

WICKENBURG WATERSHED PROJECT

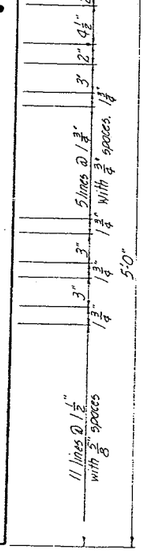
FLOODWATER RETARDING DAM

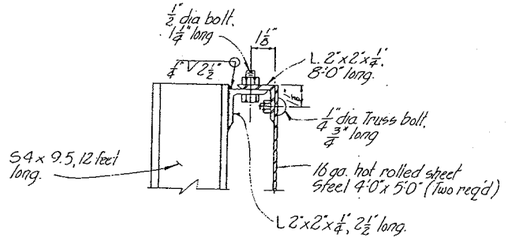
DRAINAGE AREA	384.0 ACRES
FLOOD WATER RETARDING STORAGE	55.0 ACRE FT.
WATER SURFACE AREA	8.6 ACRES
HEIGHT OF DAM	30.5 FEET
VOLUME OF FILL	67,800.0 CUBIC YD.

BUILT UNDER THE WATERSHED PROTECTION AND FLOOD PREVENTION ACT

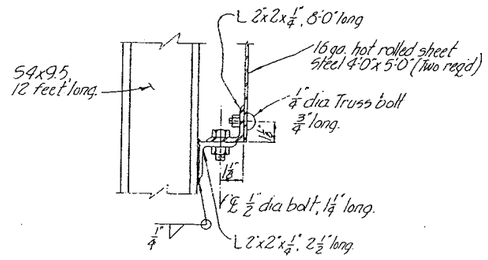
BY
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
TRIANGLE NATURAL RESOURCE CONSERVATION DISTRICT
WICKENBURG NATURAL RESOURCE CONSERVATION DISTRICT
YAVAPAI COUNTY BOARD OF SUPERVISORS
TOWN OF WICKENBURG
WITH THE ASSISTANCE OF
SOIL CONSERVATION SERVICE
OF THE
U.S. DEPARTMENT OF AGRICULTURE
1975

LETTERING LAYOUT

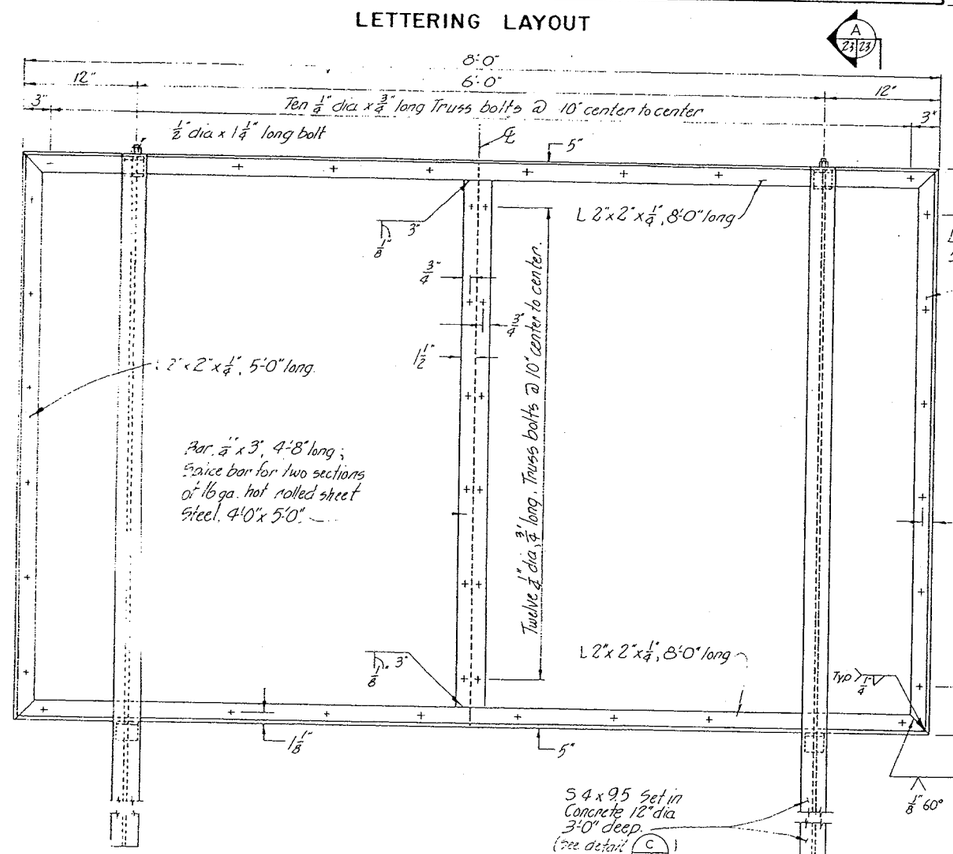




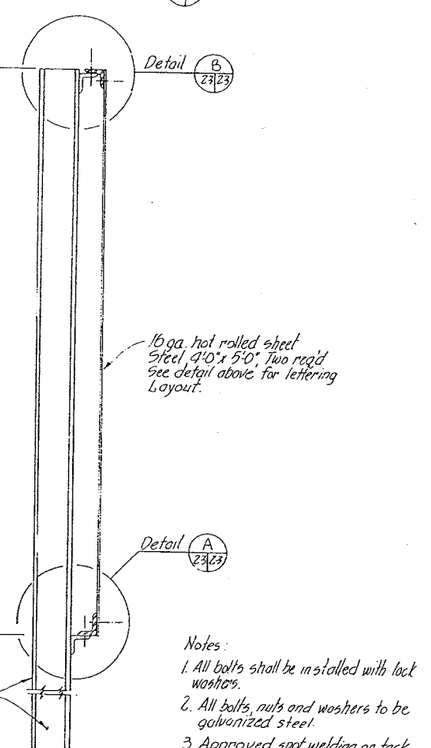
DETAIL (B 23|23)



DETAIL (A 23|23)

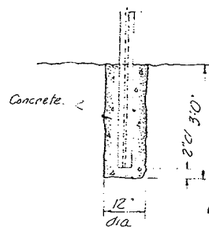


REAR ELEVATION



SECTION (A 23|23)

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.



DETAIL (C 24|24)

AS BUILT
COUSTRUCTION COMPLETED
9/15/76
Walt R. Wakefield

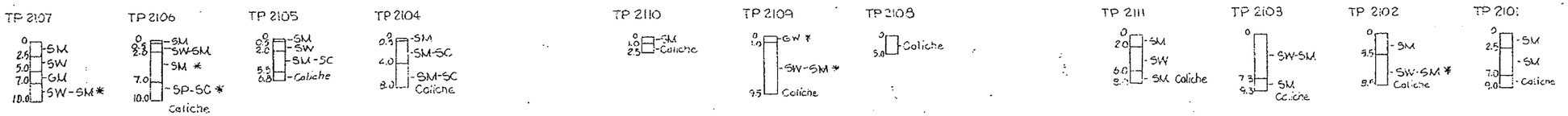
SIGN DETAILS

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

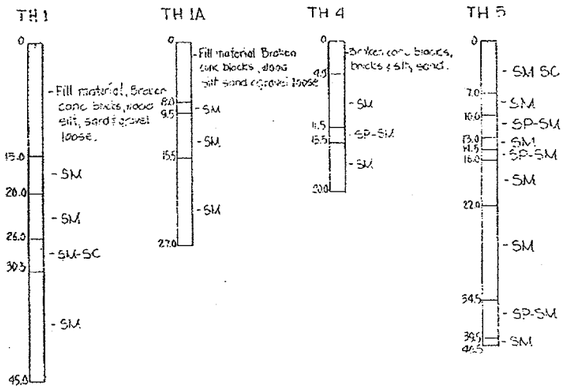
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J. Brust	Date 3-75	Approved by	
Drawn J.D. Land	3-75	Title	
Traced		Sheet No. 24 of 28 sheets	Drawing No. 7-E-23089
Checked			

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



BORROW AREA



DAM SITE AREA

AS BUILT
CONSTRUCTION COMPLETED
9/15/76
Dallas Wakefield

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 Testholes see sheet 2.

SOIL LOGS

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Logged A. Sanders	9-74	Approved by	
Drawn Jack. D. Land	3-75	Title	
Traced		Title	
Checked G.M.C.	4-75	Sheet 25 of 28	Drawing file 7-E-23089

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

BLDG.

BLDG.

BLDG.

EXCAVATION LIMIT BOUNDRY

CIRCLE J-R
~~CRESTVIEW~~ MOTEL

ORIG. GROUND

CUTOFF TRENCH

12+00

11+00

15+00

ORIG. GROUND

STOP OF CUTOFF TRENCH TO NORTH & START OF CUTOFF TRENCH TO WEST STA 14+15

14+00

14+00

13+00

CUTOFF TRENCH

END OF CUTOFF TRENCH TO WEST

13+00

℄ SUNSET DAM

AS BUILT
CONSTRUCTION COMPLETED
9/15/76

2145
2140
2135
2130
2125
2120
2115

TOP OF FILL

STA 10+75

ZONE III MATERIAL

2.0

ZONE I MATERIAL

DAM SLOPE 1:3

FOUNDATION EXCAVATION LIMIT

STA 11+50

STA 15+00
℄ DAM

SECTION (B/III)

STA 10+75

ZONE I MATERIAL

2125
2120
2115
2110

ZONE III MATERIAL

DAM SLOPE 1:3

ZONE I MATERIAL

FOUNDATION EXCAVATION LIMIT

11+50

STA 14+50
℄ DAM

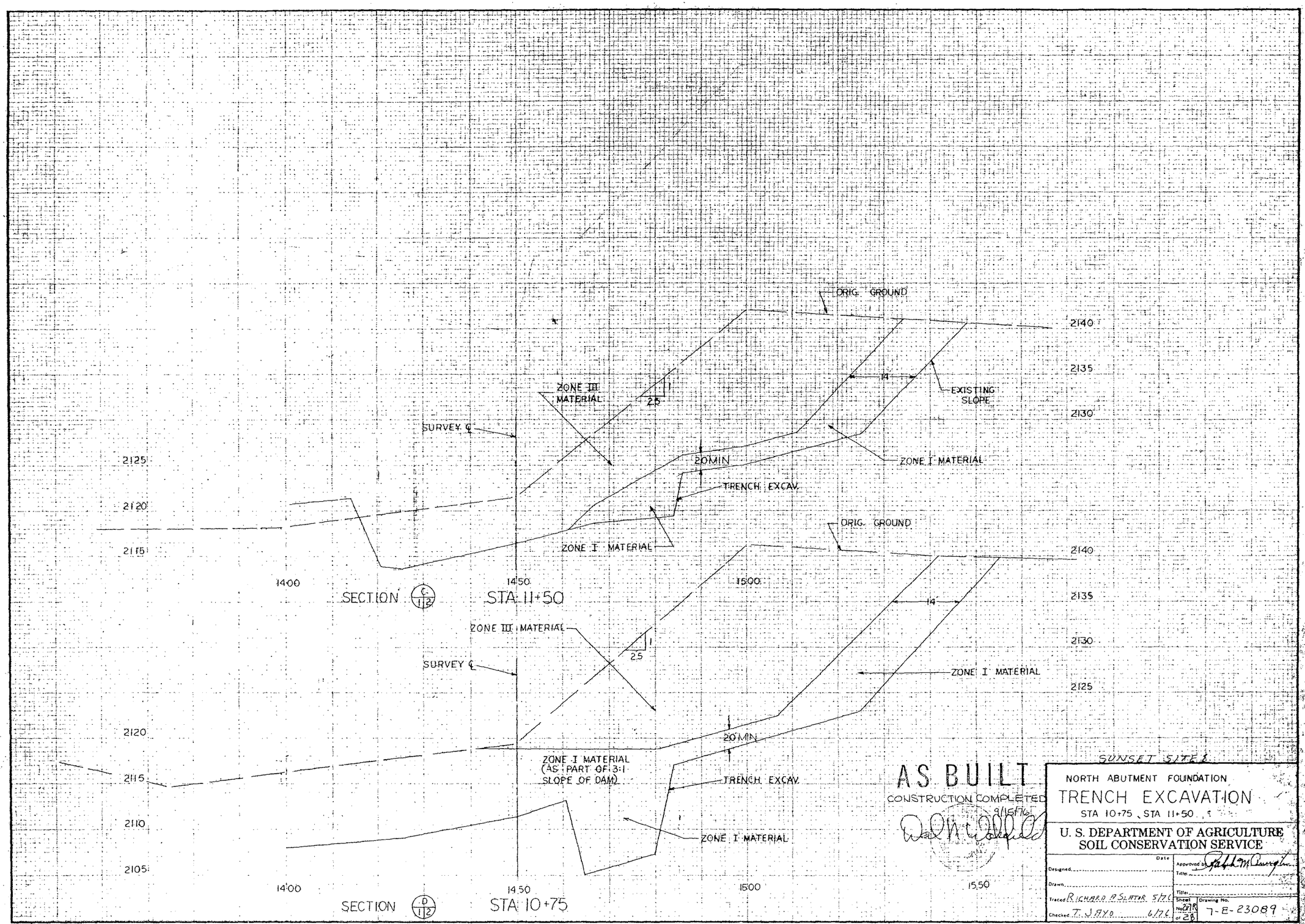
10+75

SECTION (A/III)

SUNSET DAM
WICKENBURG W.R.P.
SITE 1 NORTH BANK
MARICOPA COUNTY

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed	Date	
Drawn N. DUNCAN	5/13/76	
Traced		Title
Checked T. JAYO	6/17/76	Sheet No. 28 of Drawing No. 7-E-23089



AS BUILT
CONSTRUCTION COMPLETED
9/15/76

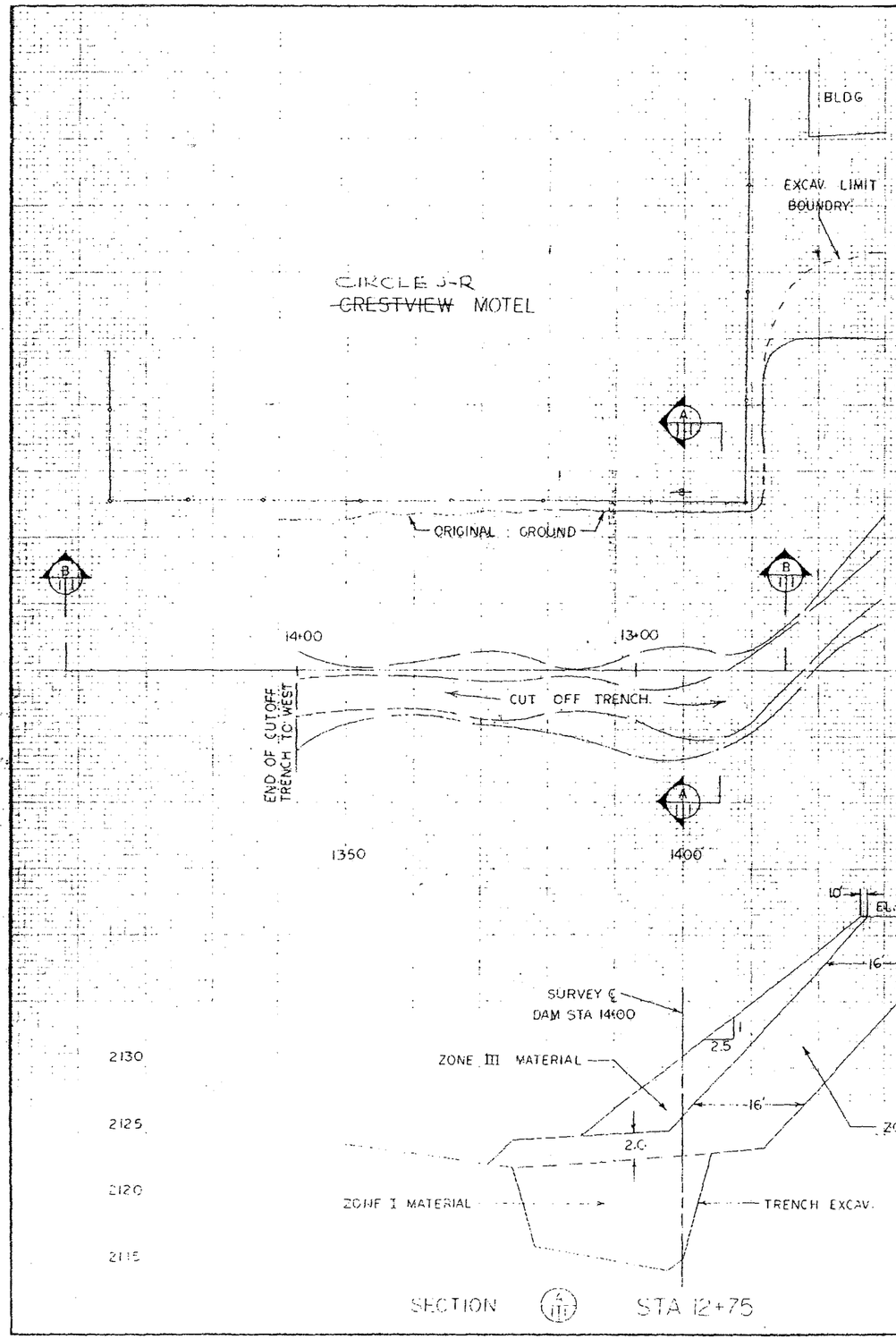
SUNSET SITE 1

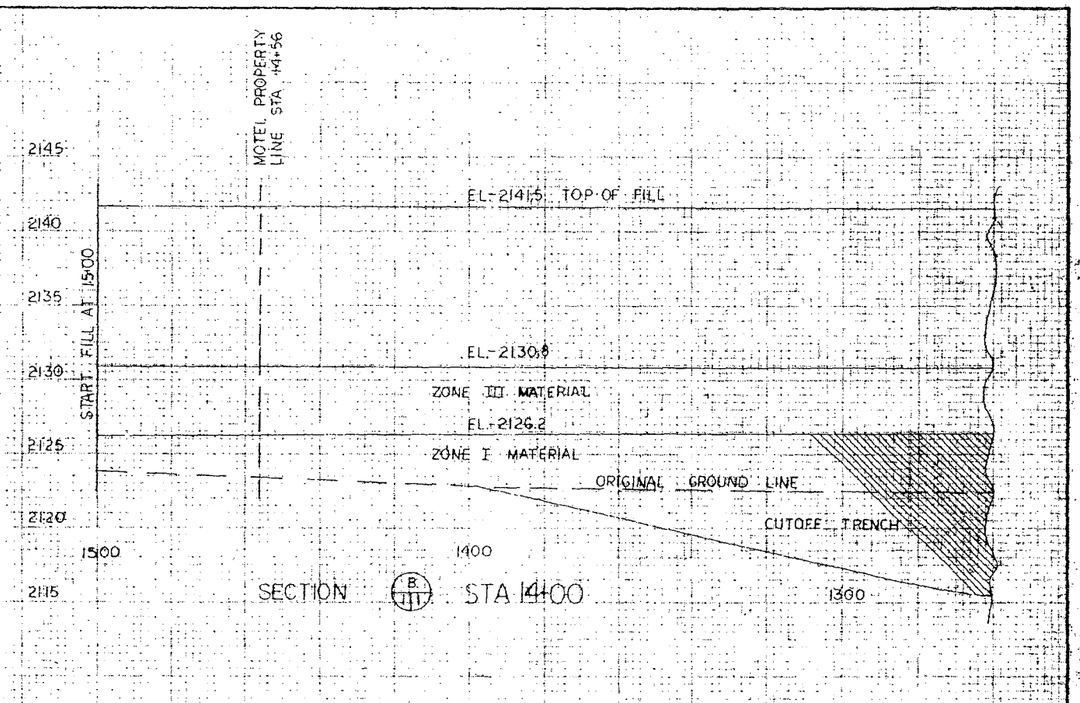
NORTH ABUTMENT FOUNDATION
TRENCH EXCAVATION
STA 10+75, STA 11+50

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed
Drawn
Traced RICHARD A SLATER 5/76
Checked T. J AYD 6/76
Date
Approved by Gaylord M Ewright
Title
Title
Sheet No. 27R of 28
Drawing No. 7-E-23089

FORM SCS-315 (APRIL 1963)





WICKENBURG W.P.P.
SUNSET DAM - SITE 1
MARICOPA COUNTY

AS BUILT
CONSTRUCTION COMPLETED
9/15/76
W.M. Wakefield

~~CIRCLE J-R~~ ~~CRESTVIEW~~ MOTEL
EMBANKMENT PROTECTION

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed
Drawn K. SLATER 5/80
Traced
Checked J. WARD
Approved Ralph W. Lungren
File
Title
Drawing No. T E 25089

FORM SCS 315 APRIL 1963

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 Sunset FRS. Details of the adjustment calculations for this datum shift can be found in the "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 year-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)
SNST11	14+83	2141.5	2143.73	2143.215
SNST12	13+95.66	2141.5	2143.73	2143.105
SNST13	12+96.2	2141.5	2143.73	2143.13
SNST14	10+95.0	2141.5	2143.73	DAMAGED

(Fig. 1-1 Plot Data)

2003 - Adj. Design
-0.515
-0.625
-0.6
-

(Fig. 1-2 Plot Data)

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

2) SNST14 was found destroyed in the 2003 survey. This location's elevation is invalid data and the monument needs to be replaced.

3) 2003 survey data collected in-house in September 2003.

Table 1

Crest Settlement Monument Elevations

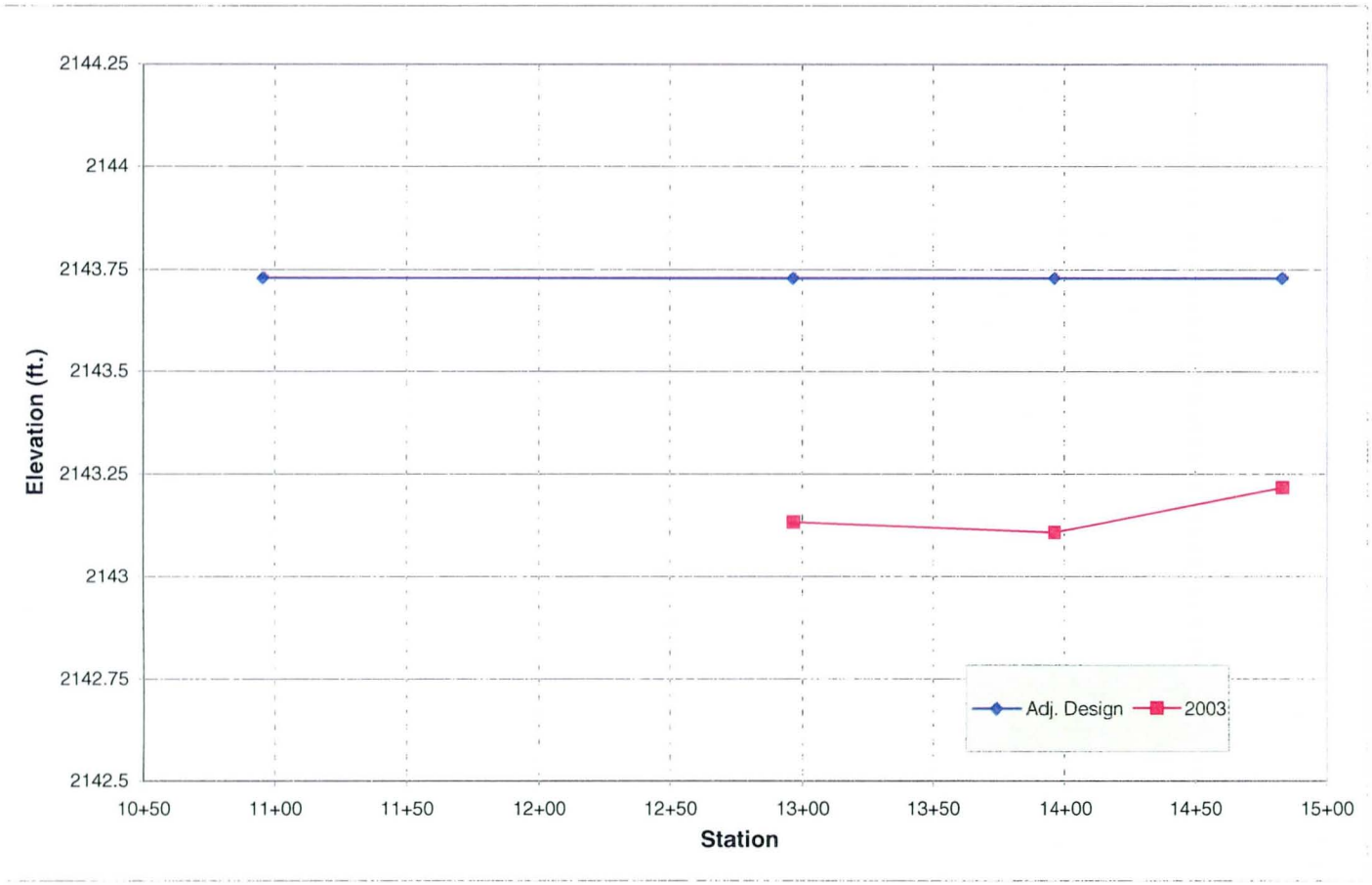


Figure 1-1

Elevation of Crest Settlement Monument Chart

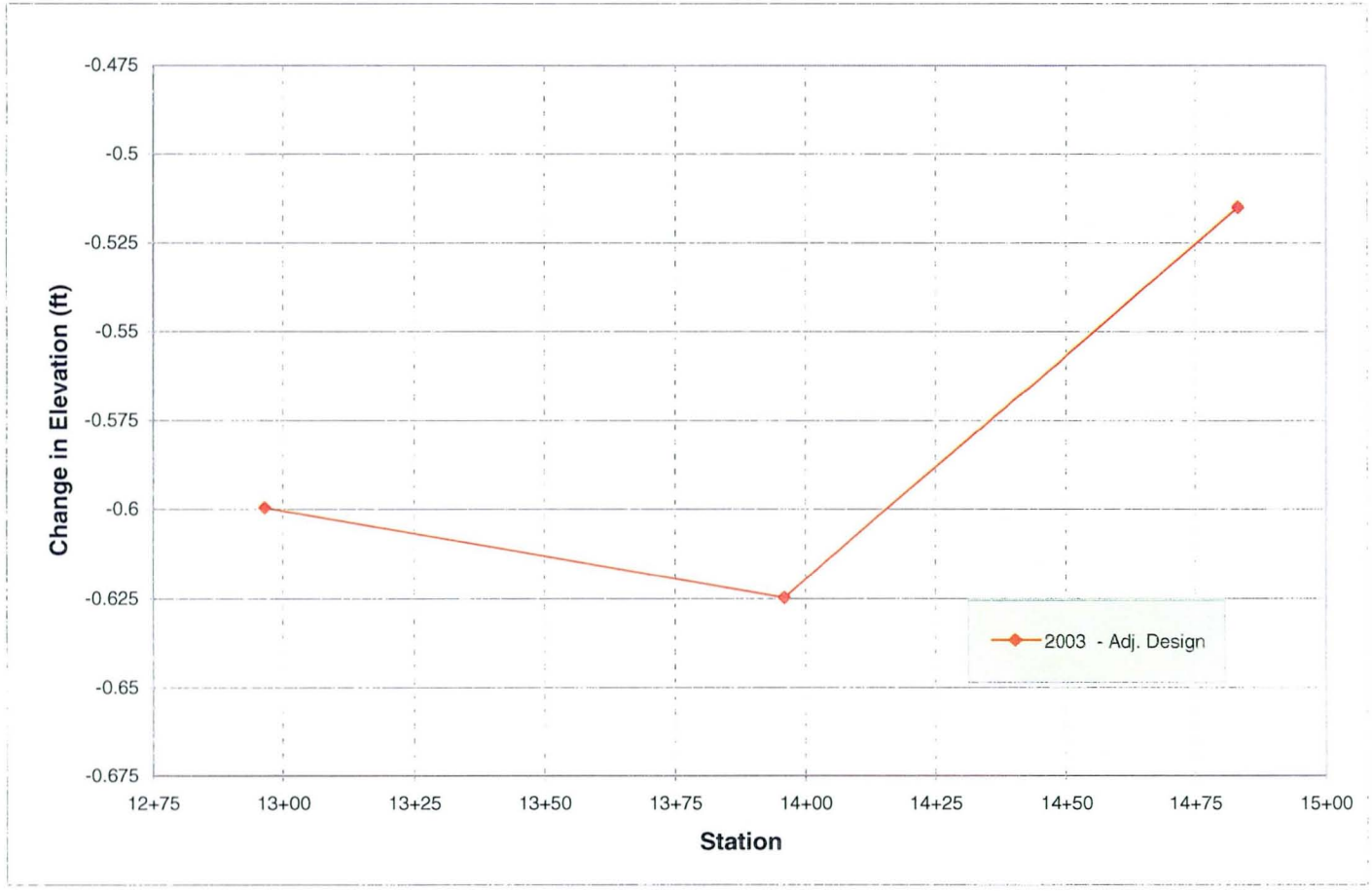


Figure 1-2

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

Table 2-1 below, summarizes the 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 the benchmark.

Toe Monument Survey Data		
Marker	**Station**	2003
SNT15	10+93	2121.842
SNT16	12+96.6	2114.776
SNT16	13+91.8	2122.716

Table 2-1
Toe Settlement Monument Elevations

Reference Mark		
Marker	**Description**	2003
SNST10	ADOT BC BM	2144.166

Note: BM SNST 10 is set at the intersection of Cucaracha and Wickenburg Way (US 60).

Table 2-2
Benchmark Elevation

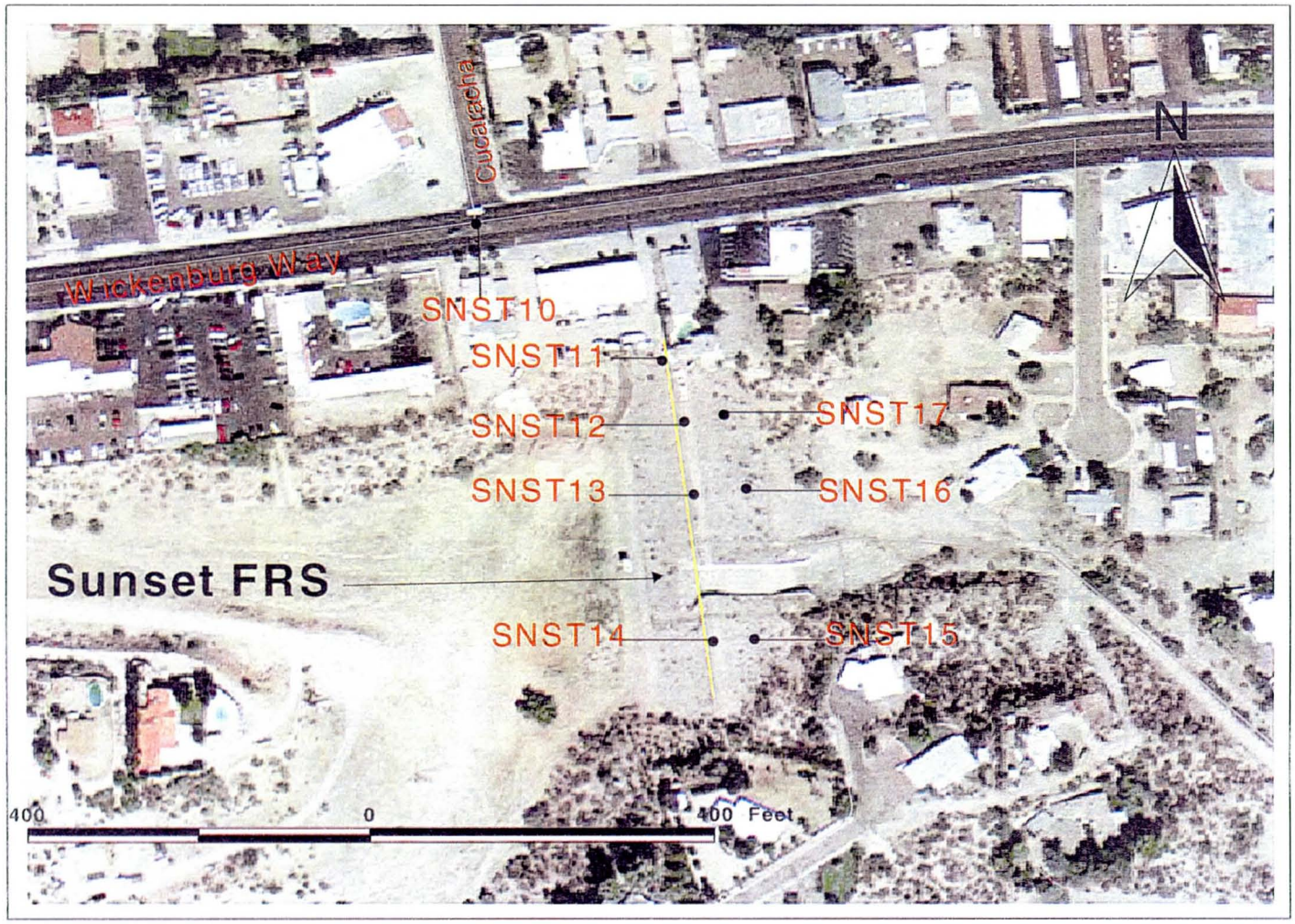
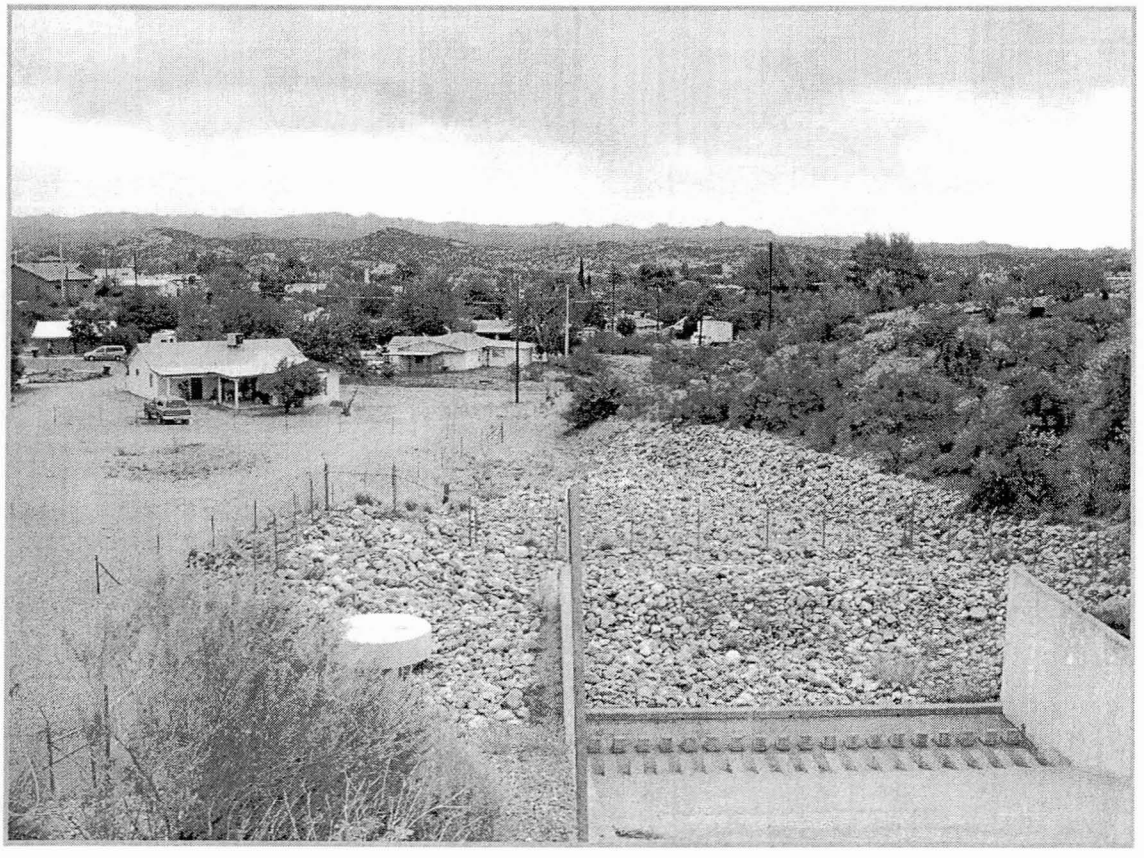


Figure 2-1
2003 Survey Monument Locations

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



Final - November 19, 2003

Table of Contents

Wickenburg Flood Retarding Structures Emergency Action Plan

Wickenburg Flood Retarding Structures Emergency Action Plan

Location

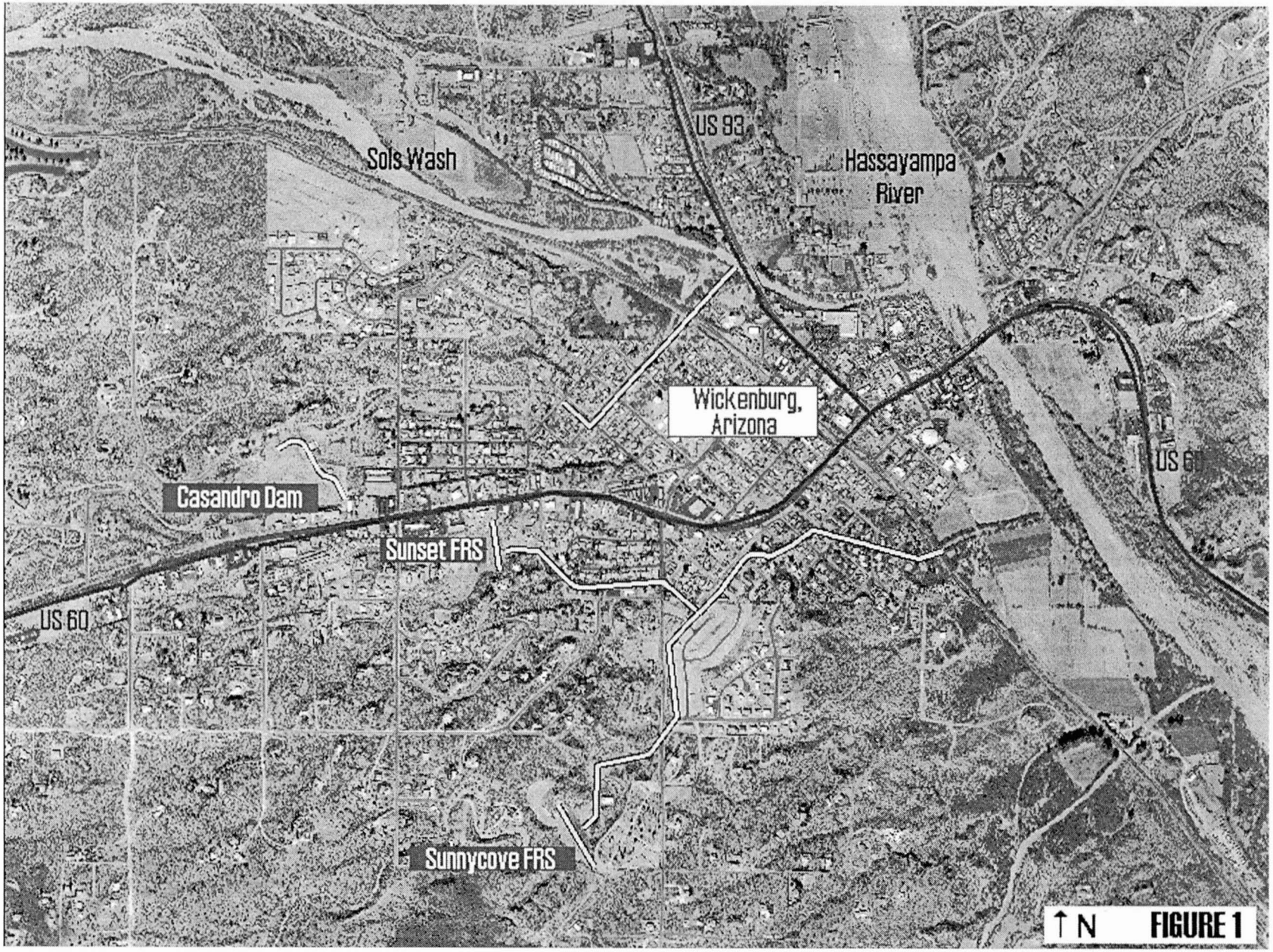
The Wickenburg Structures (Sunset FRS, Sunnycove FRS and Casandro Dam) are located within the town boundaries of Wickenburg, approximately 60 miles northwest of Phoenix, Arizona. Figure 1 shows the locations of the three structures.

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.


US 93
Sols Wash
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.

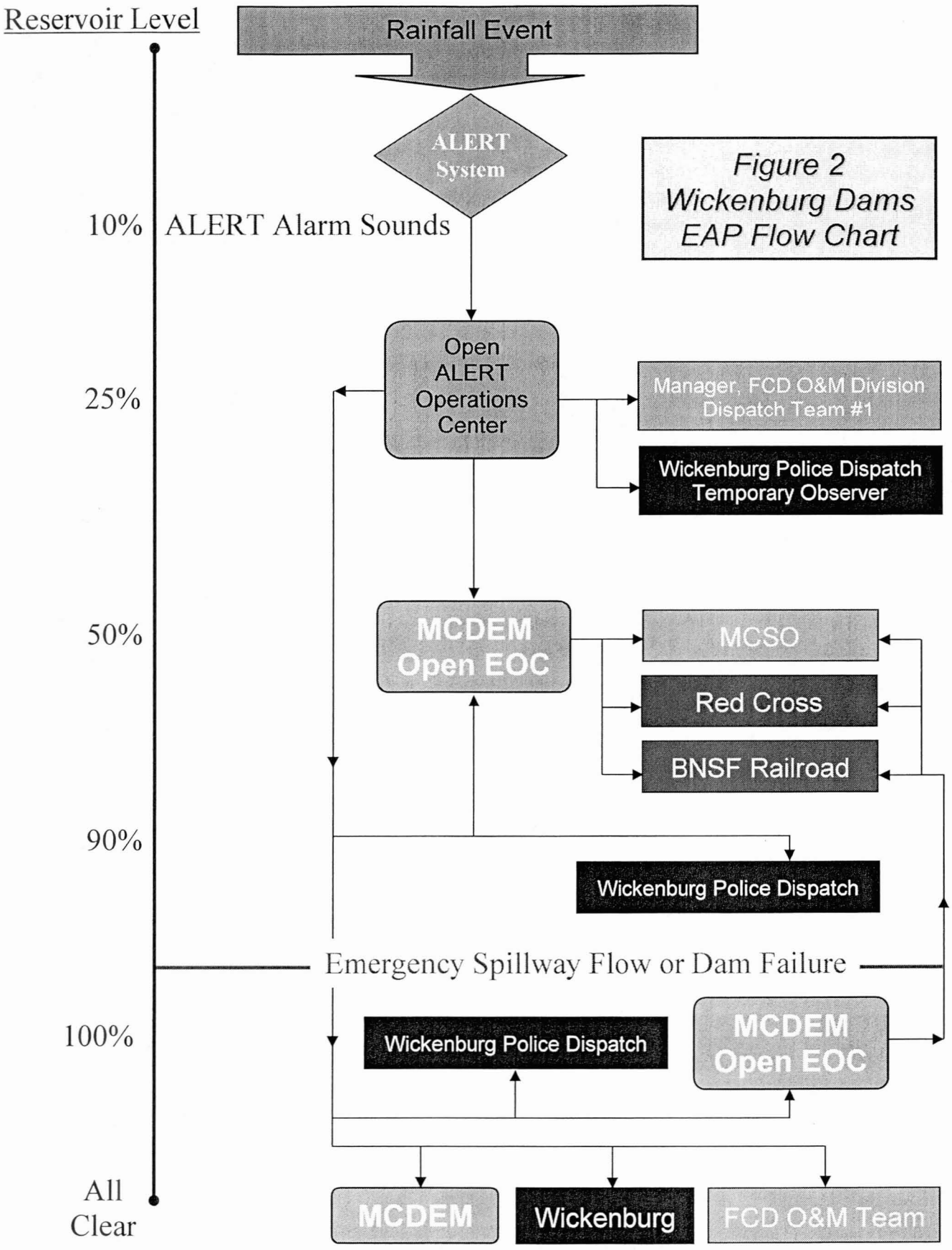

Reservoir Level
Rainfall Event
ALERT System
Figure 2
Wickenburg Dams
EAP Flow Chart
10%
ALERT Alarm Sounds
Open ALERT Operations Center
25%
Manager, FCD O&M Division Dispatch Team #1
Wickenburg Police Dispatch Temporary Observer
MCDEM Open EOC
50%
MCSO
Red Cross
BNSF Railroad
90%
Wickenburg Police Dispatch
Emergency Spillway Flow or Dam Failure
100%
Wickenburg Police Dispatch
MCDEM Open EOC
All Clear
MCDEM
Wickenburg
FCD O&M Team

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 Howard Ct.	Cool Water Drive 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

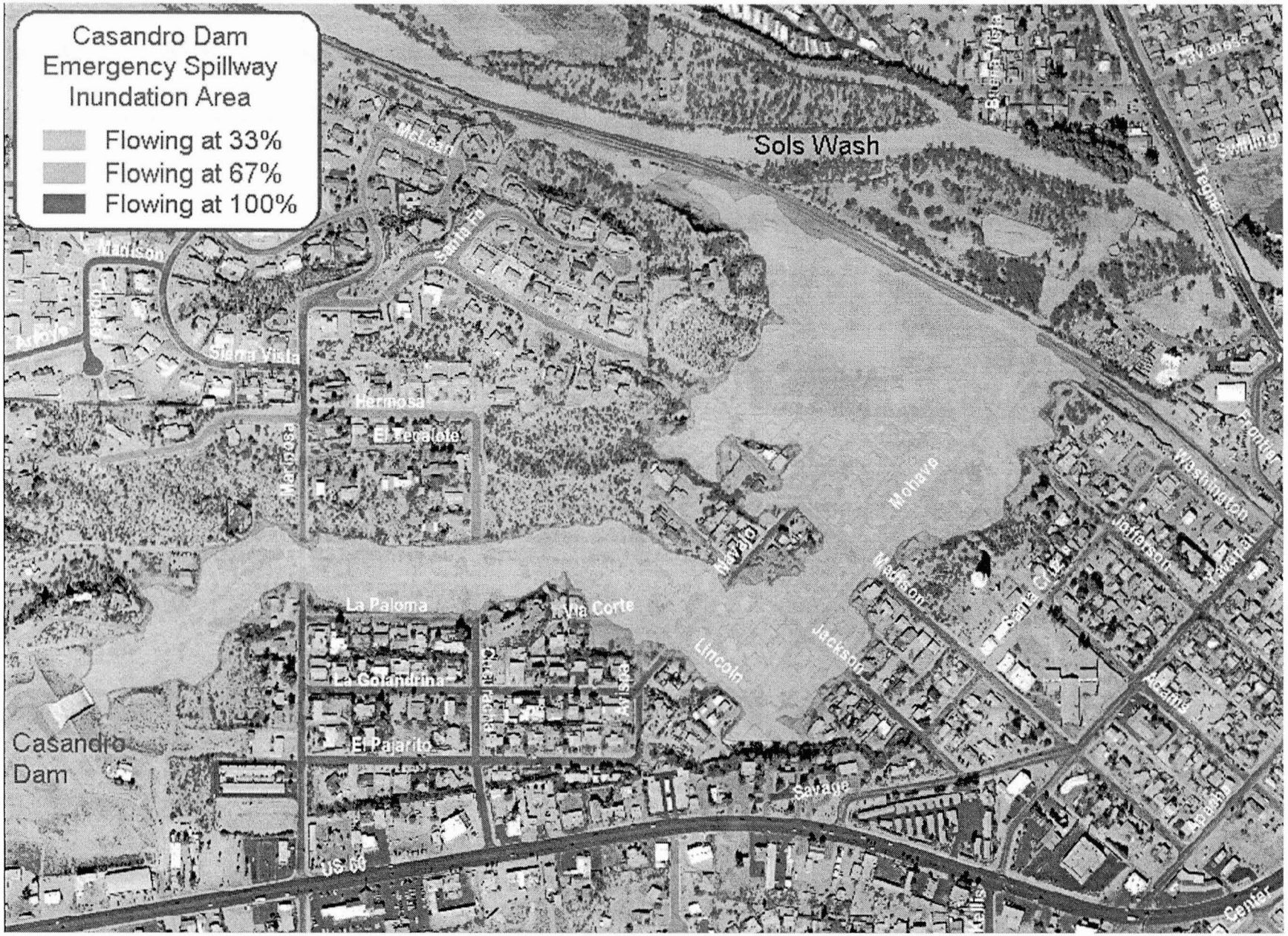
ADWR Dam Safety Division

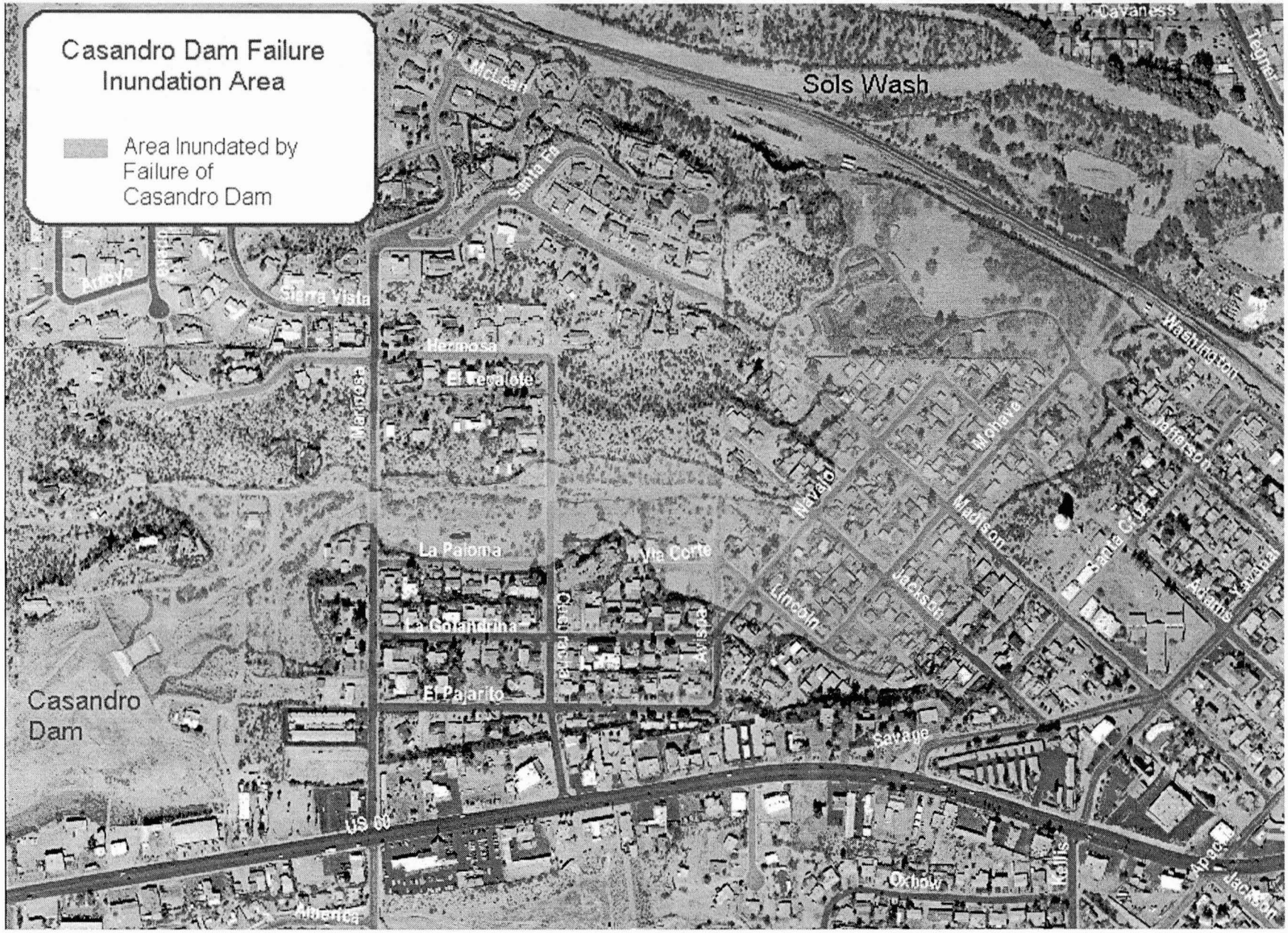
Office 602-417-2442

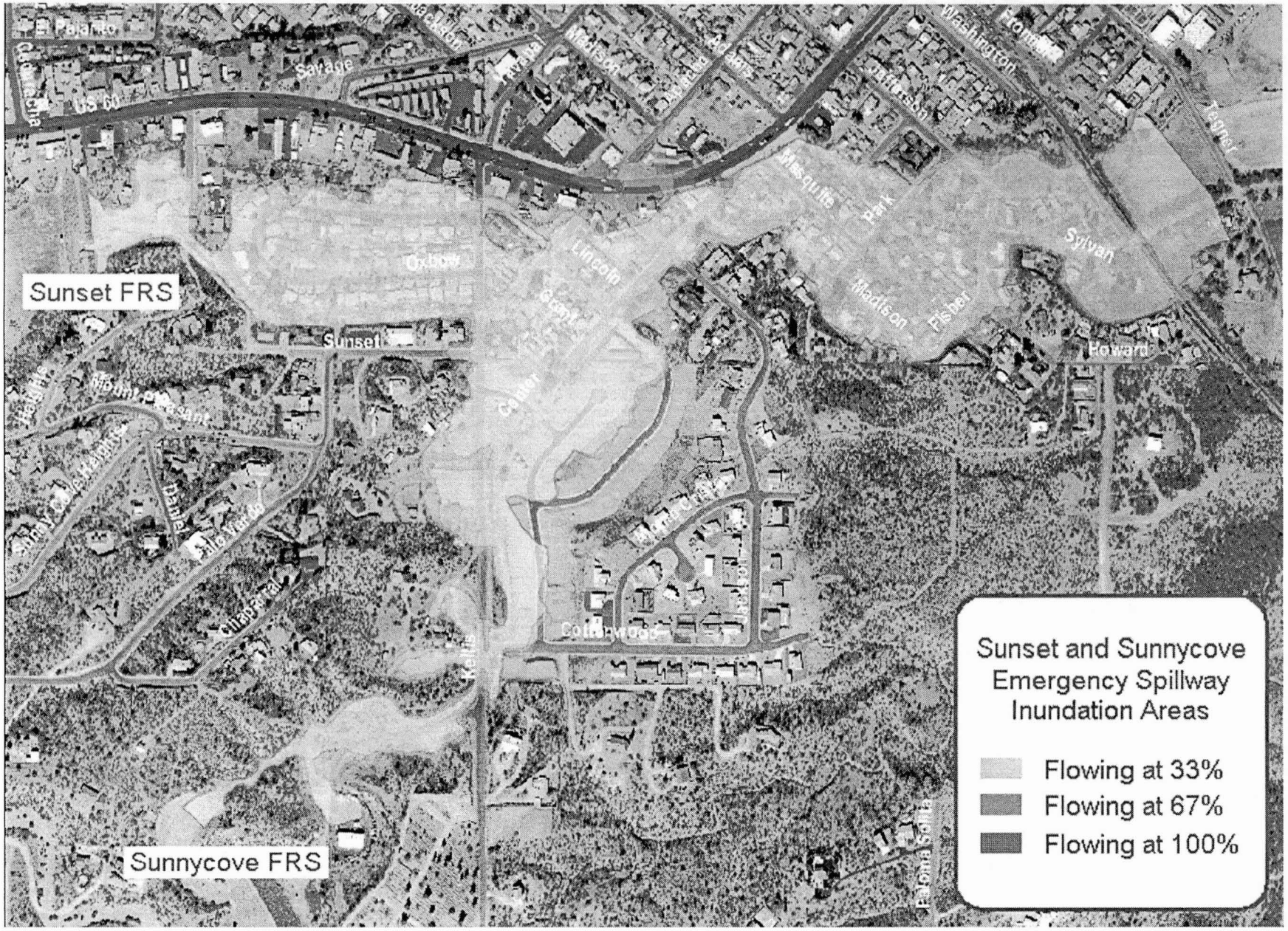
American Red Cross

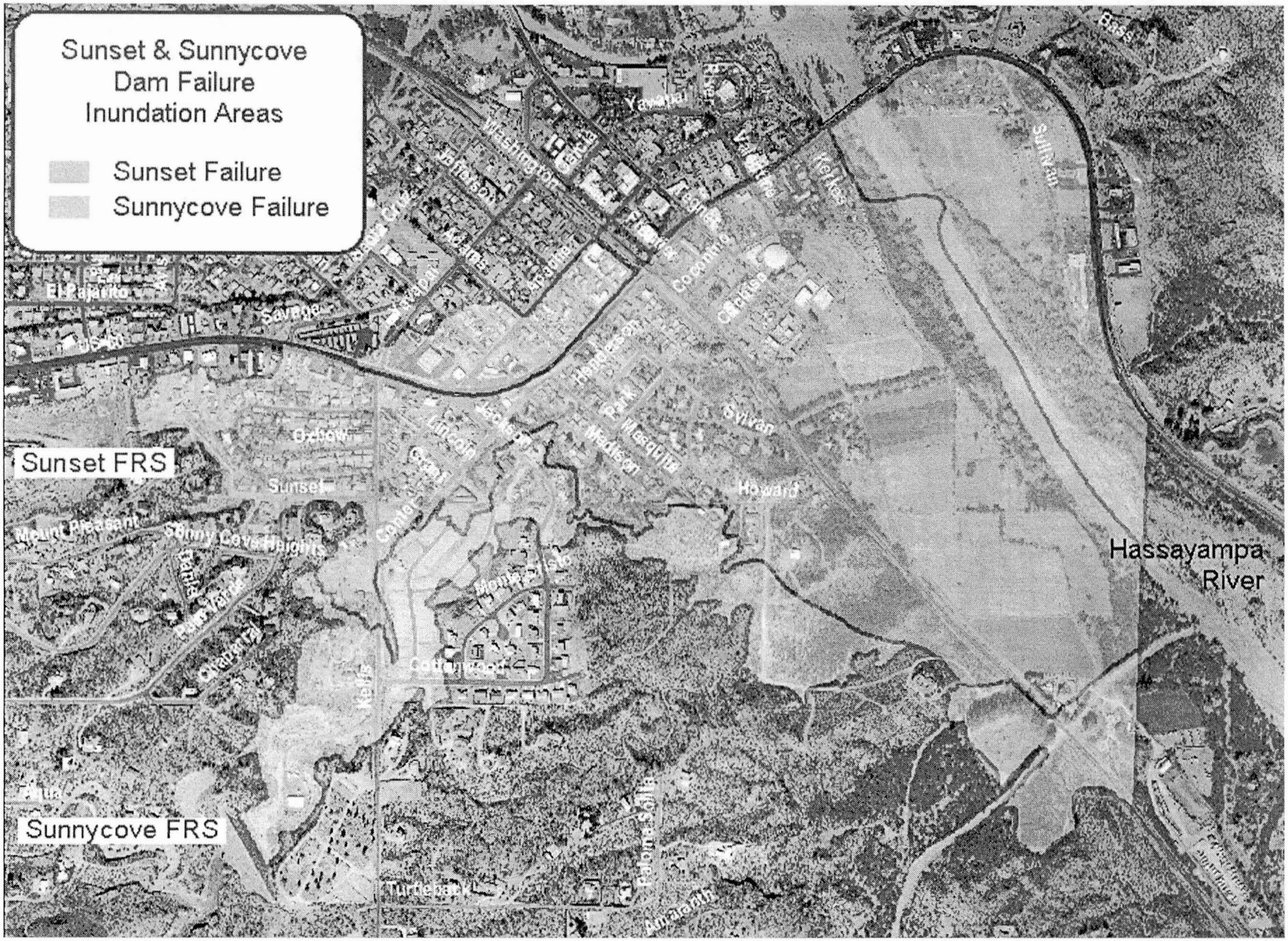
Office 602-336-6660

Appendix B – Maps of Inundation Areas


Casandro Dam
Emergency Spillway
Inundation Area
Flowing at 33%
Flowing at 67%
Flowing at 100%
Sols Wash
McLean
Santa Fe
Madison
Arroyo
Sierra Vista
Hermosa
El Recalote
Marigold
Mohave
Marigold
Madison
La Paloma
Via Corte
Lincoln
Jackson
La Colandrina
El Pajarito
Casandro
Dam
Savage
US 60
Center
Frontier
Washington
Jefferson
Acoma
Tegner


Casandro Dam Failure
Inundation Area
Area Inundated by
Failure of
Casandro Dam
Sols Wash
McLean
Santa Fe
Sierra Vista
Hermosa
El Revalote
Navajo
Mohave
Madison
Jackson
Lincoln
La Paloma
Via Corte
La Golondrina
El Pajarito
Casandro
Dam
Savage
US 60
Oxbow
Davaness
Tegner
Washington
Adams
Jackson


Sunset FRS
Sunnycove FRS
Sunset and Sunnycove
Emergency Spillway
Inundation Areas
Flowing at 33%
Flowing at 67%
Flowing at 100%
US 60
Savage
Madison
Washington
Sylvan
Mesquite
Park
Lincoln
Grant
Fisher
Howard
Oxbow
Sunset
Cottonwood
Tegner


Sunset & Sunnycove
Dam Failure
Inundation Areas
▨ Sunset Failure
▨ Sunnycove Failure
Sunset FRS
Sunnycove FRS
Hassayampa
River
Yavapai
Washington
Jefferson
Adams
Apache
Frontier
Tegner
Coconino
Cochise
Kerkes
Sullivan
Bass
El Pajarito
Savage
US 60
Oxbow
Sunset
Lincoln
Jackson
Henderson
Park
Madison
Mesquite
Sylvan
Howard
Grant
Center
Mount Pleasant
Monte Cristo
Cottonwood
Kellis
Palm Verde
Chaparral
Turtleback
Amaranth

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 Sunset FRS. Details of the adjustment calculations for this datum shift can be found in the "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 year-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)
SNST11	14+83	2141.5	2143.73	2143.215
SNST12	13+95.66	2141.5	2143.73	2143.105
SNST13	12+96.2	2141.5	2143.73	2143.13
SNST14	10+95.0	2141.5	2143.73	DAMAGED

(Fig. 1-1 Plot Data)

2003 - Adj. Design
-0.515
-0.625
-0.6
-

(Fig. 1-2 Plot Data)

Notes: 1) The Datum Shift to NAVD88 elevation referenced from Casandro Wash Dam is NGVD29 elevation **plus 2.23 ft**.
2) SNST14 was found destroyed in the 2003 survey. This location's elevation is invalid data and the monument needs to be replaced.
3) 2003 survey data collected in-house in September 2003.

Table 1
Crest Settlement Monument Elevations

Settlement Monuments - Crest

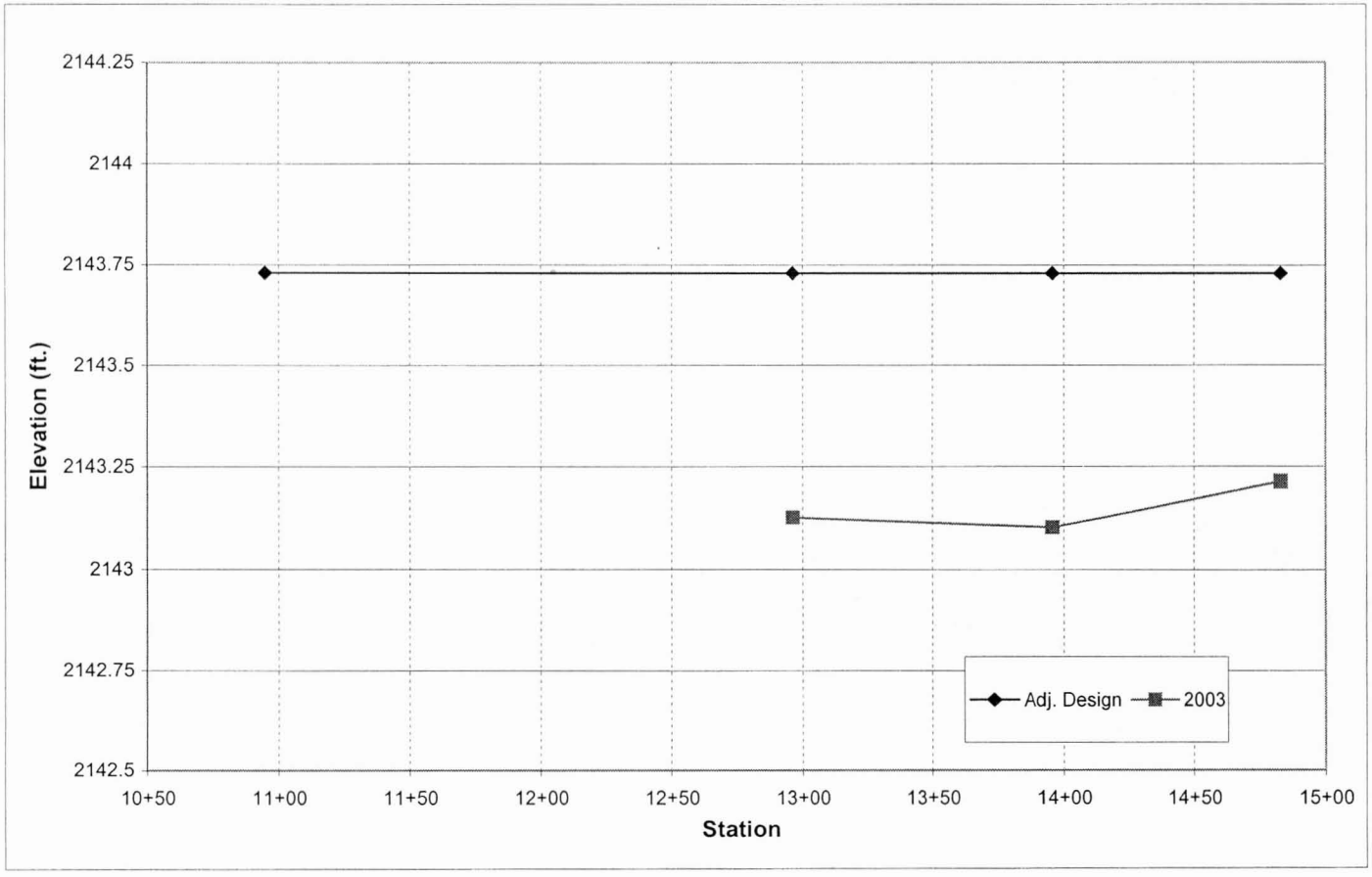


Figure 1-1

Elevation of Crest Settlement Monument Chart

Settlement Monuments - Crest

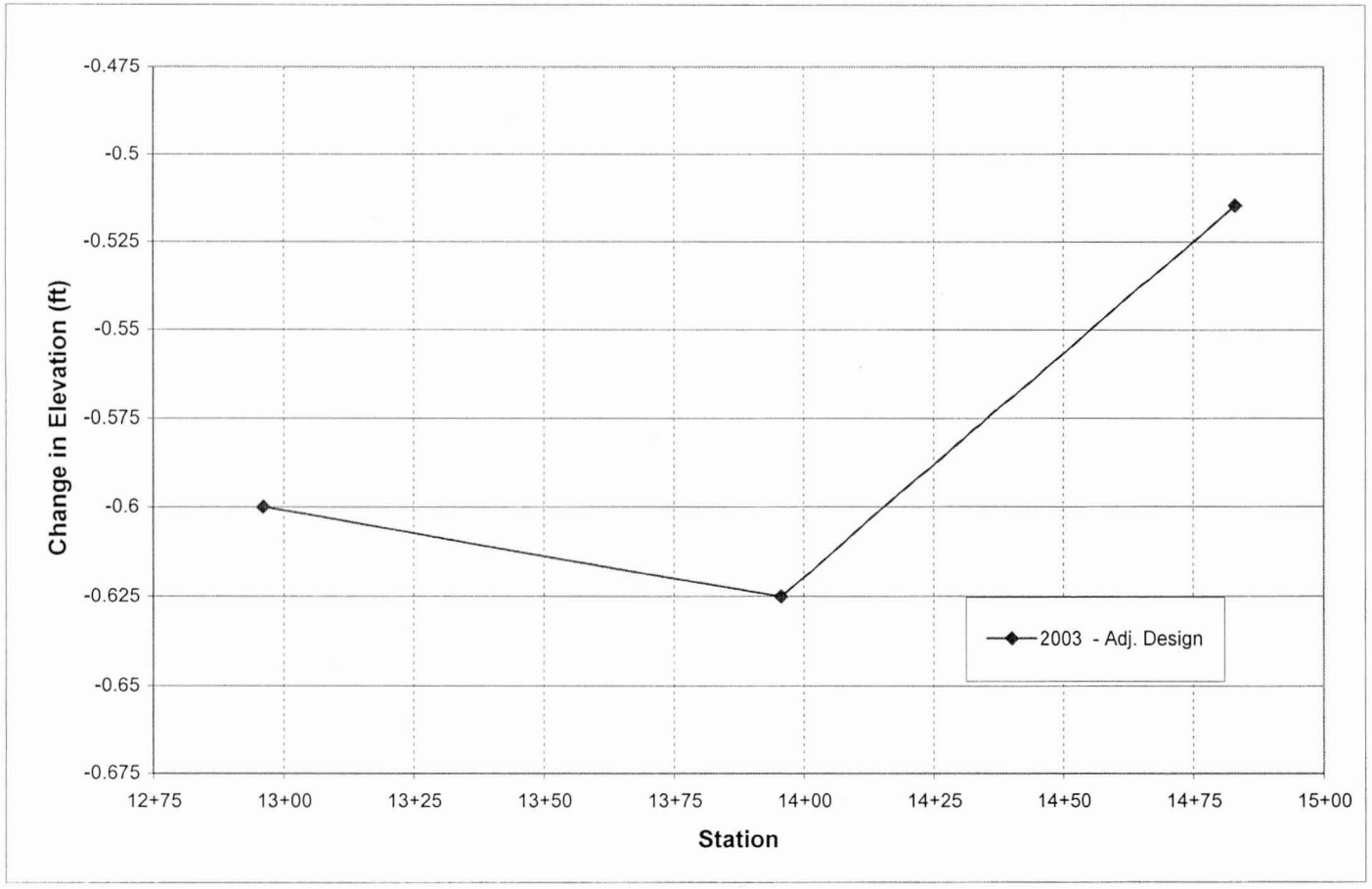


Figure 1-2

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

Settlement Monuments - Toe & Benchmark

Table 2-1 below, summarizes the 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 the benchmark.

Toe Monument Survey Data		
Marker	**Station**	**2003**
SNT15	10+93	2121.842
SNT16	12+96.6	2114.776
SNT16	13+91.8	2122.716

Table 2-1
Toe Settlement Monument Elevations

Reference Mark		
Marker	**Description**	**2003**
SNST10	ADOT BC BM	2144.166

Note: BM SNST 10 is set at the intersection of Cucaracha and Wickenburg Way (US 60).

Table 2-2
Benchmark Elevation

Sunset FRS - Floodplain View

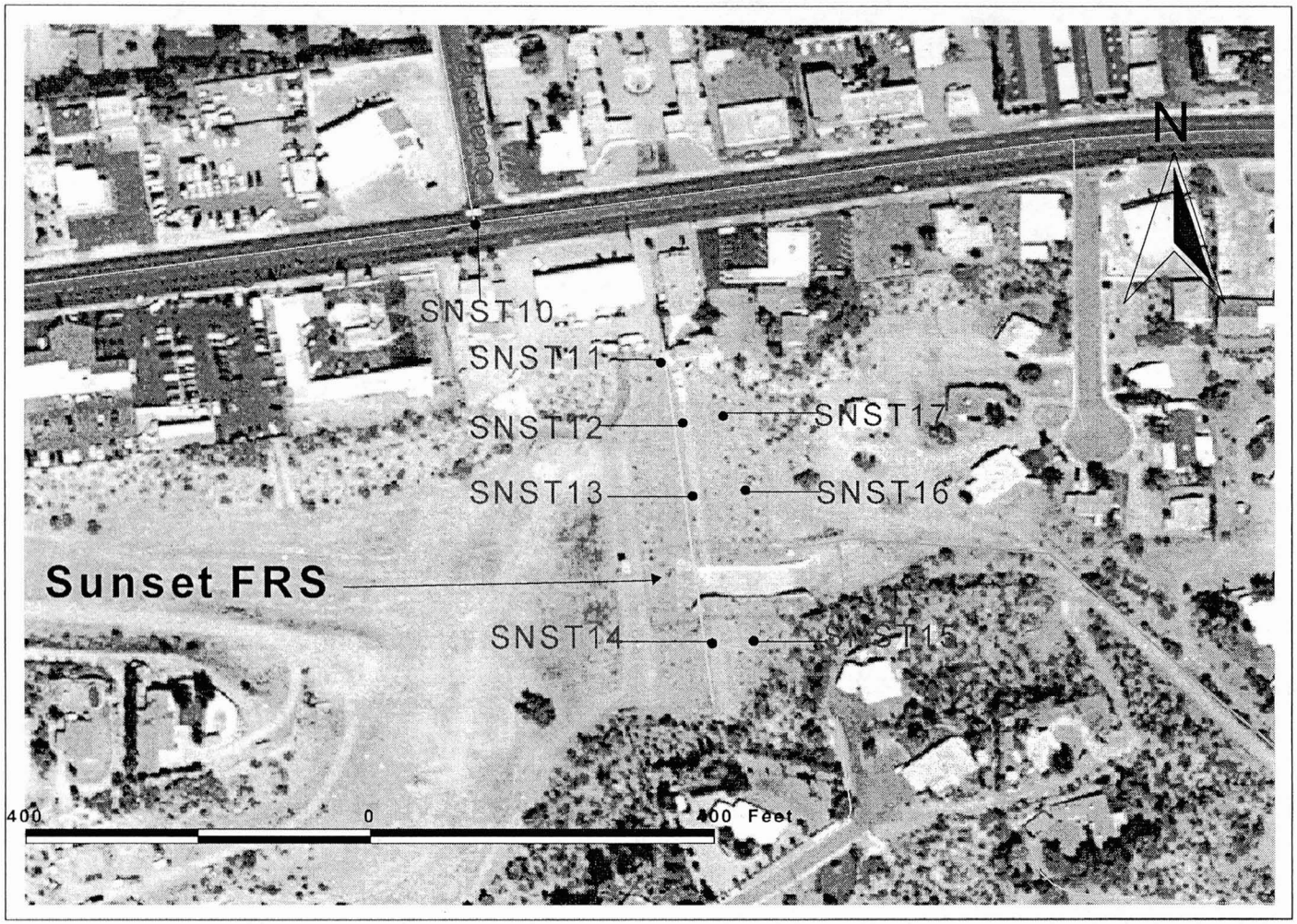


Figure 2-1
2003 Survey Monument Locations

PRELIMINARY FAILURE MODES IDENTIFICATION REPORT SUNSET 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 Sunset 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
Andrew Dziobek	Natural Resources Conservation Service
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

Sunset FRS is a small earthfill structure with a crest length of 488 ft, a crest width of 14 ft, upstream slope of 3:1, downstream slope of 2:1, a dam height of 30.5 ft and reservoir capacity of 55 ac-ft to the spillway crest. It is a homogeneous dam with an internal chimney drain with an internal drainage collection pipe. A reinforced concrete conduit through the central portion of the dam at streambed level conveys reservoir discharges to an overflow manhole structure at the downstream toe. An intake structure/tower at the upstream end of the conduit permits discharge through a slide gate protected by a trash screen. The intake structure also functions as a drop inlet with an open top to permit discharge for flows in excess of normal reservoir operating storm. A reinforce concrete spillway structure has been constructed over the dam embankment.

3.0 Summary of Inspection Reports

Inspection reports from 2003 to 1980 were located and reviewed. In 2003 the principal spillway conduit was videotaped by the District. From 1989 to 2003 there is a record of impoundments with dates and depths. Gravel mulch placed on upstream and downstream slopes in 2003 to control minor erosion rills. Inspection reports from 1980 to 1985 indicate the adjacent Ford Dealership was draining waste oil into the pool area. This practice has since been stopped. Inspection reports since 1980 indicate displacement of the emergency spillway training wingwalls. The District has set monitoring pins on the joints of the wing walls. Monitoring has been discontinued as no further displacement has been noted.

4.0 Preliminary Failure Modes

1. **Embankment Overtopping**: The embankment crest and downstream slope are protected against minor 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. Check current criteria vs. analysis regarding PMF
    e. Are high capacity groundwater wells having localized effect?
    f. What are the current warning times and durations?
    g. 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.
    h. Review and document the most current estimate of reservoir stage capacity.
    i. 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?

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 are current construction and development plans in the area?

3. **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. Seepage collars around principal spillway – not sure of detail how Zone II installed around pipe (could act as diaphragm)
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. Visually inspect the intake tower for cracking.
f. What will happen if outlet pipe breaks?
g. Is principal outlet underlined by collapsible soil?
h. What is the seismic effect on the tower?
i. Is the piping asbestos cement? Yes.
j. How was Zone II placed around pipe?

4. **Piping Involving Foundation and Abutments**: Relates to potential piping erosion of soil materials from the embankment fill into the foundation and/or developing through the foundation under the embankment. 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. ***Piping Involving Foundation/Abutments***
      i. Look for sharp transitions and high points
      ii. Look for buried alluvial channels and seepage pathways
   c. ***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?)
   d. ***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

e. ***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).
f. Potential for ***earth fissures*** extending under dam(s)? Not likely.
g. ***Downstream runoff erosion***. Review and assess if discharge from natural drainages adversely impacts the downstream face or toe of the embankment.

5. **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.
      i. Check for gradation data on filter/drain and core material zones. (*Filter compatibility*/filter match criteria between adjacent material zones.)
      ii. Review *internal stability* of central chimney/filter drain materials and filter match of Zone I to Zone II
   c. ***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

d. ***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
e. Review the characteristics of case history of FCD embankment cracking (width, spacing, depth).
f. ***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 II material only extends to spillway crest elevation except at spillway (approximately 4' wide Zone II on either side of spillway to elevation 2135 NGVD, which is 4' above spillway crest elevation)
g. Evaluate if ***animal burrows*** can serve as seepage conduits across the entire width of the embankment.

6. **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.

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

8. **Failure Mechanisms Associated with Earth Fissures**: Previous as well as current investigations by others have identified a strong potential for earth fissures at a number of FCD structures. This potential failure mode is an unlikely scenario. 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 soils could result in loss of support due to erosion of the (as opposed to collapsible) soils.

9. **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.

10. **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. Qualitatively assess the impact of discharge from the emergency spillway on the downstream areas.
    b. Qualitatively assess the impact of groundwater withdrawal in the vicinity of the dam.
    c. No water stops in horizontal joints of spillway chute. Reinforcing steel isn't continuous thru the floor slabs (movement of one slab either horizontal or vertical plus open joint, could lead to negative pressures at joint and loss of foundation material)

5.0 Closure

The aim of the workshop on February 24, 2004 was to identify and develop a comprehensive list of failure modes for Sunset 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 Februarys 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
SUNSET 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: Sunset 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 Sunset FRS.

Maximum Water Surface Elevations: The District maintains a log of maximum water surface elevations for Sunset FRS. The maximum recorded impoundment for Sunset reservoir is 34 acre-feet with a stage of 12.27 feet (gage height) at the FRS (September 1997).

Emergency Spillway Discharges. Based on District records, there has been no recorded emergency spillway flows at Sunset FRS. The spillway is a reinforced concrete chute structure constructed integral with the dam embankment. The downstream end of the spillway terminates in an energy dissipator.

Distress Observations Corrected or Operation and Maintenance Conducted Since Last Inspection: The District has placed gravel mulch on the embankment slopes to help control erosion 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 Sunset 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 some animal burrows on the slope face. The upstream toe shows very minor signs of erosion. There was no evidence of seepage, undermining, settlement or sloughing. 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.

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 was no signs of seepage.

Emergency Spillway: The emergency spillway is located at embankment Station 11+85 and was designed and constructed on the dam embankment. The FRS emergency spillway is a reinforced concrete drop spillway structure with downstream energy dissipater. The downstream channel is lined with rip-rap and was clear of any obstructions. There is a chain link fence spanning the discharge channel that was designed and constructed as a break- away fence.

Instrumentation: Sunset FRS has four 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 in the intersection of Wickenburg Way and Cucaracha Road. 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 Sunset 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. Consider how to access to right abutment during flood events having discharges from the emergency spillway.
b. Repair slope erosion rills, as needed, on downstream and upstream slopes.

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.

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

ADWR NO.: **07.49** FCD NO. **340**	DAM NAME: **Sunset FRS**		TYPE: **Earthfill with chimney drain**	NOT APPLIC.	NO	YES	MONITOR	REPAIR	INVESTIGATE
CONTACTS:		REPORT DATE: **March 24, 2004**							
INSPECTED BY: **KHA Team (see ISA Report)**		INSPECTION DATE: **February 25, 2004**							
REVIEWED BY: **Bob Eichinger**		REVIEW DATE:March 24, 2004	PAGE **1 of 6**						
SPILLWAY CREST ELEVATION: **2131 ft.**	HAZARD CLASS: **High**		SIZE: **Small**						
INFLOW DESIGN FLOOD: **½ PMF**	SPILLWAY CREST WIDTH: **40 ft.**		ADWR DAM HEIGHT: **20 ft.**						
DAM CREST LENGTH: **560 ft.**	DAM CREST WIDTH: **14 ft.**		DAM CREST ELEV.: **2141.5 ft.**						
CURRENT RESERVOIR LEVEL: **Empty**	TOTAL FREEBOARD: **10.5 ft.**		PHOTOS: Yes (8)						
Item	Comments								
1. CREST - **Width = 14 ft., Length = 560 ft., Dam Crest Height = 30.5 ft.; Elevation = 2141.5 ft. (NGVD 29) [Ref. 1976 As-builts]**									
a. Settlements, slides, depressions?					✓				
b. Misalignment?					✓				
c. Longitudinal/Transverse cracking?					✓				
d. Animal burrows?					✓				
e. Adverse Vegetation?					✓				
f. Erosion?					✓				
2. UPSTREAM SLOPE - **3 Horizontal : 1 Vertical**									
a. Erosion? Small rills –.					✓				
b. Inadequate ground cover?					✓				
c. Adverse vegetation? The Prickly Pears are adding to animal habitat along the upstream and downstream slopes					✓		✓		
d. Longitudinal/Transverse cracking?					✓				
e. Inadequate riprap? **None The lower portion of zone 3 has no rock mulch**						✓			
f. Stone deterioration?				✓					
g. Settlements, slides, depressions, bulges?					✓				
h. Animal burrows?					✓				
3. DOWNSTREAM SLOPE - **2 Horizontal : 1 Vertical**									
a. Erosion? **Right groin and toe of embankment**					✓		✓		
b. Inadequate ground cover? Open patches along fence					✓		✓		
c. Adverse vegetation? The Prickly Pears are adding to animal habitat along the upstream and downstream slopes						✓			
d. Longitudinal/Transverse cracking?					✓				
e. Animal burrows? Yes, well established around Prickly Pear, possibly rabbits.						✓			
f. Settlements, slides, depressions, bulges?					✓				
g. Soft spots or boggy areas? Grassy area lower right wing wall					✓		✓		
h. Movement at or beyond toe?					✓				

Item	Comments	N/A	NO	YES	MON	REP	INV
4. DRAINAGE-SEEPAGE CONTROL – **6 ft. wide chimney filter/drain located downstream of the dam crest that extends from elevation 2127 ft. to the foundation and from the chimney filter/drain around the principal spillway outlet RCP. At the base of the chimney filter/drain is a 6-inch diameter perforated AC (Asbestos-Cement) pipe with three connected 6-inch outlets consisting of solid AC pipe.**							
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?			✓				
b. Differential movement?			✓				
c. Cracks?			✓				
d. Settlements, slides, depressions, bulges?			✓				
e. Seepage? Est. Left ___gpm; Est. Right ___gpm			✓				
f. Animal burrows? Minor on Left and right downstream slopes				✓	✓		
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 - **29 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 2131 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:							
1. Displacement or offset?		✓					

Item	Comments	N/A	NO	YES	MON	REP	INV
2. Loss of joint material?		✓					
3. Leakage?		✓					
f. Are the trash racks:							
1. Broken or bent?			✓				
2. Corroded or rusted?			✓				
3. Obstructed?			✓				
g. Principal Spillway Gate(s): **18-inch flat back slide gate**							
1. Broken or bent?			✓				
2. Corroded or rusted?			✓				
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 Sunnycove pipeline. Downstream from this junction the pipeline increases in size to 21, 24, and 27-inches.**							
a. Seepage into conduit? **No visual indication of seepage into conduit.**			✓				
b. Debris present?			✓				
c. If concrete, do surfaces show: **Video taped the pipe in 2003.**							
1. Spalling or scaling?			✓				
2. Cracking?			✓				
3. Erosion?			✓				
4. Exposed reinforcement?			✓				
5. Other?			✓				
d. If Metal, do surfaces show:							
1. Corrosion? Minor – spot corrosion.		✓					
2. Protective coating deficient?		✓					
3. Misalignment or spilt seams?		✓					
e. Do the joints show: **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 Sunnycove and discharges into natural channel. Overflow discharges in unlined channel left of the emergency spillway.**							
a. Eroding or backcutting?			✓				
b. Sloughing?			✓				
c. Obstructed?			✓				
d. Poorly riprapped?		✓					
e. Tailwater elevation and flow condition: **Unknown.**							
11. EMERGENCY SPILLWAY-APPROACH CHANNEL - **Concrete lined 40 foot wide broad crested weir over the embankment. Concrete training walls.**							
a. Eroding or backcutting?			✓				
b. Sloughing?			✓				
c. Restricted by vegetation?			✓				
d. Obstructed with debris?			✓				
e. Silted in?			✓				
12. EMERGENCY SPILLWAY-CONTROL STRUCTURE							
a. If concrete, do surfaces show:							
1. Spalling or scaling? **Minor – no repairs required.**				✓	✓		
2. Cracking? **Shrinkage and/or temperature cracks. No structural cracks noted. No repairs required.**				✓	✓		
3. Erosion?			✓				
4. Exposed reinforcement?			✓				
b. If concrete, do joints show:							
1. Displacement or offset? **Side (training) walls show offset at first joint upstream from dam crest. It appears the displacement was result of differential movement due to cantilevered wall attached to the fixed wall of the spillway structure. There has been no additional movement documented and the wall appears stable.**				✓	✓		
2. Loss of joint material?			✓				
3. Leakage?			✓				
c. If spillway is unlined:							
1. Are slopes eroding?		✓					

Item	Comments	N/A	NO	YES	MON	REP	INV
2. Are slopes sloughing?		✓					
3. Is crest eroding?		✓					
d. Is weir in poor condition?			✓				
e. Where is control structure? **Broad crested weir on dam crest (concrete lined).**							
13. EMERGENCY SPILLWAY – CHANNEL - **Concrete lined to stilling basin.**							
a. Obstructions or restrictions?			✓				
b. If concrete, do surfaces show:							
1. Spalling or scaling?			✓				
2. Cracking? **Shrinkage and/or temperature cracks only. No structural cracks noted. No repairs required.**				✓	✓		
3. Erosion?			✓				
4. Exposed reinforcement?			✓				
c. If concrete, do joints show:							
1. Displacement or offset?			✓				
2. Loss of joint material? **No repairs required at this time.**				✓	✓		
3. Leakage?			✓				
d. If an unlined channel, does it show:							
1. Erosion?			✓				
2. Slopes sloughing?			✓				
3. Poorly protected w/ vegetation/riprap? **Riprap lined downstream of stilling basin.**			✓				
14. EMERGENCY SPILLWAY-TERMINAL STRUCTURE - **Stilling Basin**							
a. If concrete, do surfaces show:							
1. Spalling or scaling?			✓				
2. Cracking? **Shrinkage or temperature cracks only. No structural cracks noted.**			✓				
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 **Riprap Lined**							
a. Eroding or backcutting?			✓				

Item	Comments	N/A	NO	YES	MON	REP	INV
b. Sloughing? The wingwalls at left abutment of the emergency spillway showed ½ inch offset at the upstream and downstream sloping portion.				✓	✓		
c. Obstructed or restricted?			✓				
16. RESERVOIR							
a. High water marks?				✓			
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? **There is a structure located upstream that could be lower than crest of dam. Will need to get elevation to determine whether within the flood pool or not.**							✓
17. INSTRUMENTATION							
a. List type(s) of instrumentation: **Reservoir gage, alert gage and settlement monument points.**							
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) Repaired grouted riprap in left upstream groin area (2003); 2) Surveyed dam in 2003; 3) Repaired riling – both upstream and downstream with gravel mulch application in 2003; 4) Video tape and inspect principal spillway conduit and pipe in 2003.**							
ii. **2003 Recommendations: 1)Complete survey and video tape reports; 2) Provide updated EAP.**							
j. Is Supplemental Inspection required: **None 2003.**							
k. Recommended date for next inspection: **November 2004.**							
l. Status of Structures Assessment Program: **Scheduled for FY 2003-2004.**							

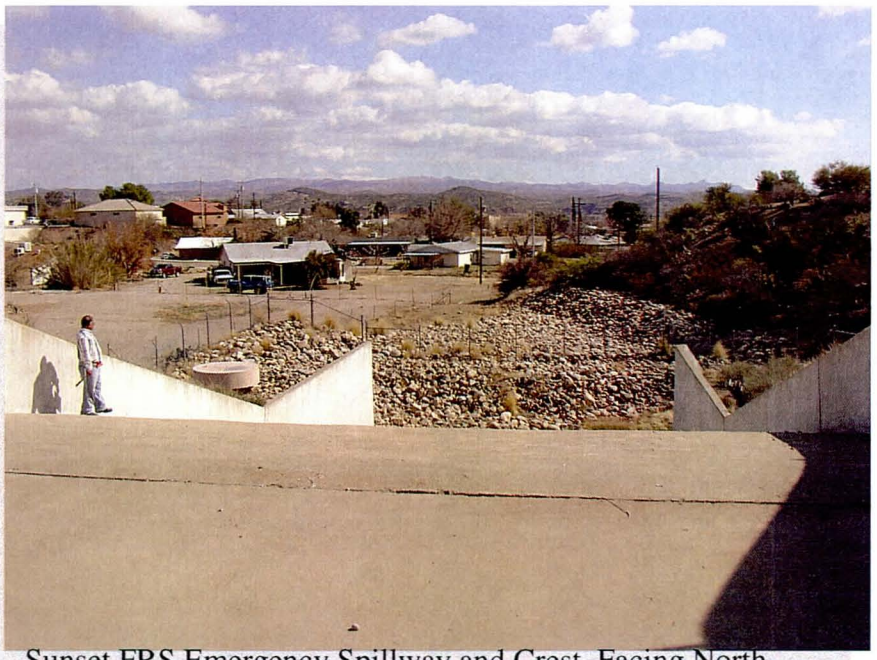
ATTACHMENTS: Photos (10)

Sunset FRS Upstream Slope Facing North. Photo Date : 2/25/04

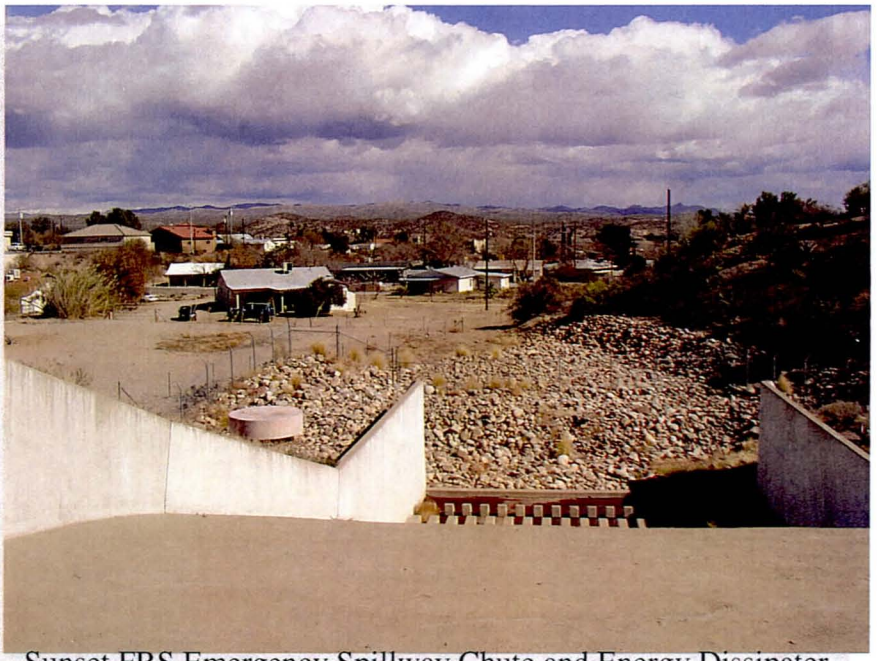
Sunset FRS Crest Facing East. Photo Date: 2/25/04

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

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

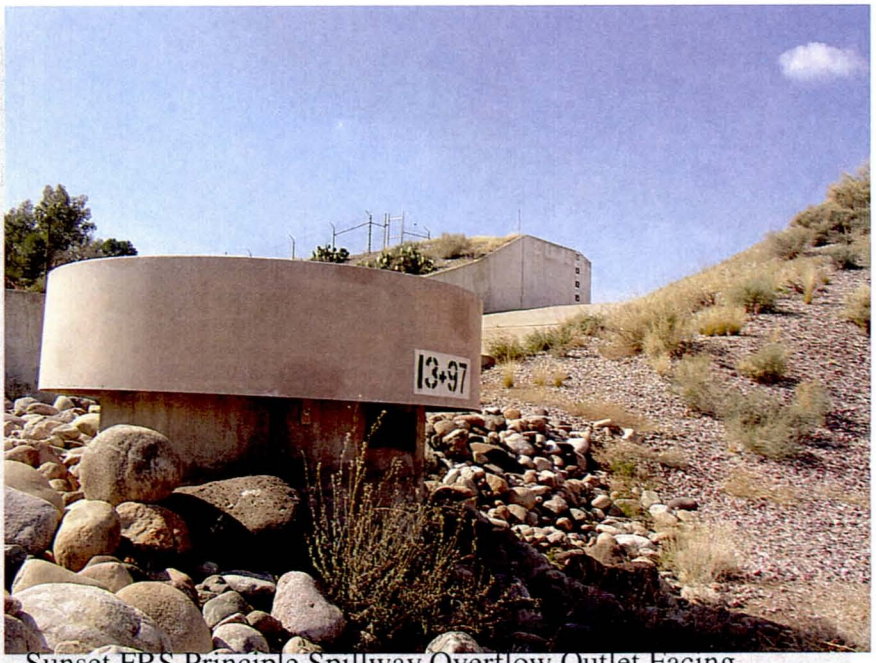


Sunset FRS Emergency Spillway and Crest Facing North.
Photo Date 2/25/04



Sunset FRS Emergency Spillway Chute and Energy Dissipater
Facing North. Photo Date 2/25/04

Sunset FRS Principle Spillway Inlet Tower and Gate Wheel/Stem Facing South. Photo Date: 2/25/04



Sunset FRS Principle Spillway Overflow Outlet Facing Southeast. Photo Date 2/25/04

FAILURE MODE AND EFFECTS ANALYSIS for SUNSET FLOOD RETARDING STRUCTURE

MARICOPA COUNTY, ARIZONA

June 30, 2004

1.0 INTRODUCTION

General Description

Sunset FRS is located in the Town of Wickenburg, Arizona south of the intersection of U.S. 60 and east of Mariposa Drive. The dam project consists of the FRS embankment with a partial height central chimney drain/filter, a concrete lined emergency spillway located centrally on the embankment, an upstream intake tower that comprises 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 Sunnycove FRS principal outlet. The project is part of the Wickenburg Watershed Protection and Flood Prevention Project.

Sunset FRS is currently classified as small sized, high hazard dam. The reservoir behind the dam is 8 acres with a capacity of 55 acre-feet. Design of Sunset FRS was completed by the Soil Conservation Service. Construction of the FRS and appurtenant structures was completed by US Soil Conservation Services in April 1975.

The dam has performed satisfactorily to date. The dam has experienced several partial impoundments of various depths since construction. There has been no discharge through the emergency spillway to date.

Dam Data

- Dam type: Homogeneous earthfill with a partial height chimney drain
- Dam height: Structural Height: 30.5 ft; Hydraulic Height: 29.5 ft
- Dam length: 488 ft
- Dam crest: width 14 ft; elevation 2141.5 ft (NGVD29)
- Spillways: Principal – 30 inch RCP; inlet elevation low stage orifice invert 2120 ft, slide gate invert 2111.25 ft, principal spillway crest 2131 ft; Emergency spillway – 40 ft wide concrete lined wide broad crested weir with concrete training walls; emergency spillway crest elevation 2131 ft
- Freeboard: 0.3 ft
- Reservoir Surface: 8 ac at emergency spillway crest
- Storage: 55 ac-ft at emergency spillway 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 Precipitation: 3.4 in and 100-year 24-hour: 4.2 in
- SCS Inflow Estimate: 3393 cfs; Maximum resulting outflow: 2381 cfs
- HMR 49 estimated Inflow (COE): 7093 cfs; Maximum resulting Outflow 3907 cfs.
- Flood Volume (Runoff): SCS: 384 ac-ft; COE: 394 ac-ft
- Spillway Design Flood: 3400 cfs at 2140.48 ft; Estimated Peak capacity: 4100 cfs at 2141.5 ft
- Spillway Crest elevation: 2131.0 ft (NGVD29)
- Current Minimum Dam Crest Elevation: 2140.9 ft (best estimate derived from 2003 crest monument survey)
- Reservoir Volume: 55 ac-ft at spillway crest; 160 ac-ft at top of dam
- Flood Routing: Corps Phase I assumed full pool at spillway crest when PMF routed and assumed outlet plugged; SCS routing starts at 10-day drawdown pool (elev. 2120 ft) and allows principal outlet to contribute.
- Outlet Capacity at spillway crest: 71 cfs; Outlet pipeline limits discharge to 8.7 cfs
- PMF Elevations: SCS: 2138.3 ft; HMR-49: 2141.2 ft
- SCS 10-day drawdown elevation: 2120.0 ft
- Drainage basin area: 0.6 sq mi (vs. 0.95 sq mi from District web data)

Purpose and Scope

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

- Identify potential site-specific failure modes for the dam.
- Discuss qualitatively the likelihood of the occurrence of 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.

(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

2.0 MAJOR FINDINGS AND UNDERSTANDINGS GAINED

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

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

1) The Dam Is Protected From Internal Erosion By Zone II Fill Materials. The SCS design intent for the Zone II material (chimney drain), which extends to elevation 2127 ft over most of the FRS (from the right abutment to station 14 +70) and then rises to elevation 3135 ft at station 15 +00 and then ties into the left abutment, was to perform as a drain to control/collect seepage through the Zone I material and to establish a favorable phreatic surface. The Zone II fulfills this function, however, the Zone I actually produces very little seepage (it is more impervious than 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 an internal erosion potential failure mode.
2) Most of the FRS is Founded On Fanglomerate. The right abutment and the foundation to about station 14+00 are founded on fanglomerate (an old alluvium cemented with caliche). This material forms a sound foundation for the FRS. From Station 14+00 to the top of the left abutment (station 15+60) the FRS was founded on "firm residual soil" after excavation of the overlying young alluvium material.
3) The Embankment Stability Is Considered To Be Adequate. The stability of the embankment appears to be adequate based on the strength of the Zone I materials used, the design slopes (2: 1 downstream and 3:1 upstream: with a 12 foot berm at

elevation 2120 ft), a very low (or non-existent) phreatic surface, and relatively little hydrostatic loading except under highly unusual conditions (and then only very briefly – a few hours).

However, the sliding stability of the FRS needs to be documented based on realistic shear surfaces and reasonable "flood event" phreatic surfaces through the FRS and for the flood drawdown pore pressure conditions on the upstream slope. The addition of an upstream stability berm was based on extremely conservative pore pressures along potential sliding surfaces based on what appears to be a totally unrealistic and inappropriate assumption for the location of the water surface (maximum level of the flood) in the fill during and after drawdown. This is illustrated by the fact that even after the addition of the berm the factor of safety against sliding remained at the same low value in the fill face as prior to the addition of the berm. Further a review of the estimated times for fill versus drawdown clearly illustrates that by design more time is allowed for drawdown than for filling, thus the embankment will drain the small amount of free water than enters the slope during the flood and will not be able to hold any substantial head (phreatic surface) in the upstream bank.

4) The Likelihood of Subsidence Induced Earth Fissures Is Very Low. The presence of earth fissuring in the vicinity of the dam or locality of the dam 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 subsidence.
5) The 14-Ft Crest Width Allows For A Future FRS Raise. The 14-ft crest width can accommodate a moderate raise without adjustment of the FRS slopes. Thus such a raise could be easily and economically accommodated if an increase in freeboard is required as a future risk reduction or mitigation action in the event of a change in flood hydrology or as the result of other site conditions.
6) The Left Reservoir Bank (Upstream of the Left Abutment) Is Protected By a cutoff and Blanket of Zone I material. The bank upstream of the left abutment is protected by a Zone I blanket to dam crest level. This blanket ties into the Zone I of the dam and into the 400 foot length of cutoff that also extends around the left side of the dam. The left bank cutoff and blanket were placed to protect against "end around seepage through the alluvium that forms the left abutment and left bank of the reservoir.
7) The Slope Stability of the Left Abutment Needs To Be Analyzed. There is no physical evidence of a stability problem with the left abutment noted to date. However, the abutment consists of natural materials and could be influenced by seepage through the dam or from the offsite area above the dam on the left side. Therefore an analysis should be performed to document the expected stability performance and associated sensitivities from water loading.
8) The Probability Of Significant Seismic Events Adversely Affecting the Site Is Low. The relatively low seismicity in the area, the low height of the FRS, the firm foundation over the bulk of the dam indicates that seismic loading has a low impact/effect on the dam.
9) Near Surface Transverse Cracks are Not Protected from Seepage Erosion By the Filter. A physically possible potential failure mode of seepage erosion through cracks

exists at the FRS during a very high water condition at the site because over most of the length of the structure the filter is 14.5 feet below the crest of the FRS and 4 feet below the spillway crest. Note that for the potential failure mode to develop the high water level must be sustained, a substantial crack must develop quickly upon loading (or be formed prior to the loading) and the Zone I must be erodible at the velocities of water that are achieved for the water flowing in the crack.

10) The Relatively Short Uninterrupted Lengths Of the Sunset Embankment Reduces Cracking Potential. Shrinkage related cracking, in general, is more likely to occur the longer the uninterrupted length of material placement (concrete or soil). The relatively short uninterrupted lengths of embankment at Sunset FRS are favorable with respect to avoiding crack formation.
11) No Water Stops in the Horizontal Joint in Spillway Chute. Potential for water to seep into joints.
12) Essentially No Potential for FRS Cracking Due to Collapsible Soils In Foundation. Most of the embankment is founded on fanglomerate and the remainder is founded on residual soils. Stream remnant deposits were removed during foundation preparation. Therefore no potentially collapsible soils are known to exist beneath the FRS.
13) Minimal Depth Design/Constructed Cutoff Trench. The cutoff trench in the fanglomerate and in the residual soils was 12 feet wide with 1:1 side slopes and relatively shallow (maximum depth of 3 feet over most of the foundation). Since there was not really a permeable layer to be cutoff the shallow cutoff trench depth is adequate and the gentle relief of this feature does not create any concern with differential settlement.

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

14) The 9-In By 9-In Orifice Is Used For 10-Day Drawdown. A 9-in by 9-in rectangular orifice (with invert elevation at 2120 ft) in the intake tower is used as the primary means to drawdown the reservoir pool below the spillway crest after a flood event and under current operating procedures provides the only discharge during the rising limb of the inflow. This relatively small flow was apparently established "by design" both as a means to ensure that pool drawdown took at least 10 days 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 due to upstream slope stability concerns (see Item 3) 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, add additional orifices, or use the 18-in 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" flood events.

15) Discrepancy in Published Reports on Basin Area and Reservoir Rating Curves. The FMEA team noted that differences were apparent in contributing drainage area, stage-storage, and stage-discharge rating curves among the various design and study reports. For future use the correct (accepted) rating curves and correct drainage area need to be identified, established and documented. The basis for their selection needs to be clarified and noted.
16) The SCS Rating For The Energy Dissipater/Stilling Basin Is Less Than The Emergency Spillway Design Capacity. The FMEA team noted that the St. Anthony's Falls energy dissipater/stilling basin is rated at (designed for) two-thirds of the emergency spillway discharge rating by design. The reason for this difference could not be ascertained from the published reports.
17) Emergency Spillway Discharges. Based on current operation, storms greater than approximately the 100-year event will result in a discharge in the emergency spillway without any advance discharge warning from a low level outlet. Development (homes and a population at risk) is located directly (within a few hundred yards) downstream from the emergency spillway. This could have serious consequences with respect to potential of life loss for floods that result in a threatening discharge level (e.g., above the 200 – 500 year level). An evaluation of the level/frequency of discharge that threatens the population at risk should be estimated.
18) A Small Contributing Watershed Such As At Sunset FRS Has A Greater Chance Of Experiencing Rainfall At Or Near The PMP Than Does A Large Basin. Based on an analysis of empirical data across the country (as well as common sense) it has been learned that the smaller the watershed the more likely it is that they will experience major rainfall events over the entire watershed. Thus it is much more probable that the Sunset Basin will experience major storms (greater than the 100-yr up to PMP events) than would a basin that has a drainage area of several hundred square miles.
19) FRS Crest Elevation. The design crest elevation of Sunset FRS was 2141.5 ft (NGVD29). Camber sloping from 0.0 at the abutments to 1 feet from sta 10+75 to 14+00 was provided.
    a) Crest survey data indicates that a maximum settlement of 0.625 ft has occurred leaving the current minimum crest elevation at 2140.9 ft.
    b) Crest survey data indicates that the dam crest is nearly level. This indicates that a generally uniform settlement of the dam has occurred. The resulting relatively level condition of the crest is favorable in that if overtopping were to occur the flow would be over entire length of crest instead of being at a localized low spot.
20) Impoundment History. Sunset FRS has a recorded history of impoundments. Three events are of considerable significance in regards to loading and performance operations (October 2000 – 8.43 ft; September 1997 – 12. 27 ft; and August 1992 – 9.44 ft). The reference or zero gage point for this impoundment depth is believed to be the invert elevation of the slide gate at the intake structure (elevation 2111.46). However, this should be verified and the actual elevation of the top of each impoundment should be reported. 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.
21) Loading Duration Short. The hydraulic loading on the upper 10.5 feet of the structure (above the spillway crest elevation 2131ft.) is estimated to be only a few hours; the

hydraulic loading on the structure below the spillway crest down to the invert of the orifice on the intake (elev. 2120 ft.) should be about 10 days under current operations but could be longer if orifice plugging takes place and the slide gate is not opened. As noted above with different operations the loading time could be significantly less.

22) IDF for the Sunset FRS Could Be Raised To PMF. The current IDF for the Sunset FRS was established as ½ the PMF based on its small size and volume and on its high hazard rating. If the District has to implement major modification of dam the IDF might be raised to the PMF based on current Arizona Dam Safety regulations (may cause dam crest elevation increase). No obvious major modification appeared to be indicted as necessary on the basis of the FMEA.

23) The 20 Foot Height of the Dam used in Various Reports Appears to be Understated. As reported in the Phase 1 inspection and in other official documents the dam height appears to be understated. The actual height of the FRS based on the excavation schedule drawing is: Structural Height 38.5-ft --- Hydraulic Height -- 29.5 ft

24) Zone III Berm And Associated Bench Was Not Required For Satisfactory Performance Of The Structure. The Zone III bench on the upstream toe may not have been required but its presence improves the stability of the structure and improves the drainage around the intake structure.

Consequence Evaluation

25) Minimum Warning Time with Emergency Spillway Discharges. The normal operation of Sunset FRS discharges impounded floodwater through the principal outlet. The principal outlet is tied to a common storm drain with Sunnycove FRS. Other District structures utilize an open channel/floodway to convey discharges from the principal outlets/spillways. Discharges into a floodway enables a visual observation by downstream residents that the reservoir is filling, that the principal (low level) outlet is discharging flood water and that there is a potential for the emergency spillway to operate. In the case of Sunset FRS, the only indication downstream will come when a discharge from the emergency spillway commences. The emergency action plan for Sunset FRS is implemented by the Town of Wickenburg. Since there is no discharge through the emergency spillway prior to a 100-year (or greater) storm event (or multiple events), the Town must be diligent in the observation of inflow into the impoundment.

26) Dam Break Analysis Unreasonable. The opinion of the FMEA team is that the existing dambreak study is overly conservative in estimating the peak discharge. The following table provides the breach parameters used in the dambreak study.

Time To Failure [min]	10
Final breach width [ft]	84
Initial Water Surface Elevation [ft]	2131.0 {emergency spillway crest}
Final Breach Elevation [ft]	2113.5
Volume of Reservoir [af]	53.6
Breach Outflow [cfs]	8,900
PMF Outflow [cfs]	7,100 (Phase I Report)

The FMEA team recommends that a new dambreak analysis be prepared for Sunset FRS. The new analysis should consider a more reasonable erosion/breach time and account for the drop in hydraulic head in the reservoir as the breach develops and flow erodes the breach. It was estimated that with the breach parameters specified the reservoir would be emptied in 4.3 minutes. The actual maximum flow from the breach would likely come before the breach is fully developed because of the small volume of water in the reservoir.

27) Emergency Action Plan. The District has prepared an EAP for the area downstream from the dam. The upstream pool should also be included in the Emergency Action Plan since there are structures impacted within the flood pool.

28) No Floodway or Outlet Channel. Sunset FRS was designed without a downstream floodway or outlet channel to accommodate discharges from the emergency spillway. Residential structures are located immediately downstream from the emergency spillway. Neither the District nor the Town has flowage easements downstream from the emergency spillway.

29) Downstream Inundation Mapping. The inundation mapping downstream from Sunset FRS was prepared assuming discharges from Sunnycove FRS as well. Although this is prudent given the close proximity of the dams and contributing watersheds, a separate inundation map for discharges from Sunset FRS should be conducted independent of discharges from Sunnycove FRS.

30) Need to Improve Warning Time. The Sunset FRS contributing watershed is relatively small in comparison to other District structures. The relatively small watershed has a very fast rainfall/runoff response. Additional watershed instrumentation may improve monitoring for earlier warning time and notices.

Action Items – Potential Risk Reduction Measures or Investigations

31) Increase The Size Of The Orifice Outlet And /Or Utilize The Existing Slide Gate. Increasing the intake inflow capacity of the principle 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 in pool 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

32) Closely Monitor Left Abutment Area. Monitor the left abutment during inflows and impoundments. Especially monitor the integrity of blanket upstream and downstream for seepage breakout.

33) Assess Sunset/Sunnycove FRS Pipeline Hydraulics. There may be capacity in the pipeline for additional flows from the principal outlets from Sunset and Sunnycove. The pipeline was designed as a non-pressurized system.

34) Flap Valve on Six-Inch Asbestos Cement Drain Pipe. The 6-inch AC drainfill pipes discharge into the stilling well of the emergency spillway. The end of the pipes are

protected with screened animal guards, however, this allows water to back-up into the drain pipe. The FMEA team recommended considering the installation of flap valves on the ends of the drain pipes.

35) Elevation of Emergency Spillway Crest. Check existing elevation of emergency spillway crest versus as-builts. Top of dam monument surveys are conducted and it is easy to obtain the spillway crest elevation at the same time.
36) Get Survey of Right Abutment. Need to establish/verify the low point elevation along the reservoir rim.
37) Check Reservoir Volume Estimate. The District estimate of the reservoir volume is 86 ac-ft to spillway crest. As-builts and Corps Phase I Inspections Report indicates 55 ac-ft.
38) Use The Lower Gate. Use the lower gated outlet to drain reservoir more rapidly
39) Use Full Capacity Of 30-Inch Outlet Pipe. The capacity of the 30-inch outlet pipe is not used to full capacity based on design. Adding additional orifices, locating orifices at lower elevations, or increasing the size of the exiting orifice to match the 30-inch pipe capacity will allow draining the pool in less time. In addition an outflow would be experienced in the downstream community during reservoir filling. Satisfactory operation of the pipe drain overflow vent should be verified as well.
40) Ten-Day Drawdown. If improvements are made to reduce the drawdown time for the dam (by implementing Items 33, 41, and/or 40) then reservoir will be assured to drain in less than 10-days. The routing for the inflow design flood may then be re-run and routed through the pool at a lower starting water surface elevation (elevation after 10-days of drawdown – per ADWR guidelines). Re-run the routing for the 100-year and Probable Maximum Flood to check drawdown times and maximum water surface elevations.
41) Chainlink Fence Located Downstream of Emergency Spillway. Is this fence a designed to be a break-away fence? There may be a potential for debris to accumulate on the fence.
42) Fix Low Spots On Dam. Review of crest monument data indicates a low area on the crest of the dam (on the order of greater than 0.3 ft).
43) Spillway Inundation Maps. The District has prepared downstream spillway inundation mapping. However, the mapping does not include potential discharge from 30-in pipe from the downstream overflow manhole. Discharge from the overflow manhole could provide warning of potential emergency spillway discharge.
44) Provide Additional Means for Warning. Add more gauges in contributing watershed, outside watershed, stream gauges. Consider use of Doppler radar and satellite imaging.
45) Basin Excavation. Excavation of additional storage volume.
46) Downstream Low-Flow Channel. Develop a downstream channel to convey low-flows. When residents see flows in low-flow channel this may assist in warning time and response/actions.
47) Ascertain District Property Rights. Confirm right-of-ways/easements for impoundment pool area/limits. Check easements for pool elevations for 100-year and PMF.
48) Confirm PMF Routing Through Emergency Spillway.

49) Conduct New Stability Analysis Of Upstream Slope. Use reasonable pore pressure. Conduct stability circle for downstream slope and model left abutment slope failure in slope stability analysis.
50) Direct Runoff From Businesses To Grouted Area In Left Upstream Groin Instead Of Down Face Of Left Abutment Slope.
51) Analyze Stability Of Blanket Abutment.
52) Monitoring Foundation of Left Side Abutment. Include monitoring of left abutment in site inspections and crest monument surveys.
53) Run Collapse Tests on Left Side Materials (Residual Soils on which the FRS was founded). Sample materials and perform collapsible test (preferably insitu)
54) Confirm Basis Of Drawdown Curve. Is the stage/discharge and stage/drawdown curve based on 9-in by 9-in orifice or 30-in principal conduit?
55) Check Gate Opening Capability. As-built plans indicate a gate stop installed on the gate. It is possible to remove the gate stop to use lower gate to assist in draining pool quicker.
56) Perform Seismic Stability Of Tower.
57) Check Stage/Frequency. Check the stage versus frequency for existing and proposed improved (increased) outflow conditions during reservoir filling.
58) Check Contributing Basin Area. District database on website indicates 0.95 sq. mi. whereas documentation indicates smaller basin 0.6 Sq. Mi.
59) Prepare System Graphic Depicting:
    a) Contributing Basin boundary,
    b) 100-year and PMF Pool Limits,
    c) Downstream Channel
    d) Outlet Pipe System
    e) Linkage to Sunnycove and Hassayampa River

General Findings

60) Only One Category I Potential Failure Mode. This Category I Failure Mode is related to normal operation of the emergency spillway on the likelihood of high consequences downstream of the dam due to normal discharges.
61) 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.

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 judgement 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 Sunset FRS emergency spillway is a 40 foot wide reinforced concrete spillway located on the embankment about one third of the length of the crest away from the right abutment. Normal flood discharges from the spillway are directed into a residential area 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 under major flood events would 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 downstream housing development.

Adverse Factors:

(1) Emergency spillway flows are directed into the Town of Wickenburg for flows greater than the 100-year event in upstream watershed.
(2) No dedicated downstream easements for spillway discharges.
(3) No regulation (development, density, land use) on downstream spillway discharge areas.
(4) 72-hour or 24-hour PMF may be more critical.
(5) Land use is changing in upstream watershed. More highly urbanized.
(6) No advance flow discharge from dam to warn downstream residents.
(7) Very small watershed and small reservoir. Quick rainfall/runoff response and quick fill time of reservoir (approximately less than one hour of fill time).
(8) Floodwaters are not released until pool is at emergency spillway crest.
(9) Residential structures in downstream channel.

(10) Impoundment history indicates pool elevation has reached to within 5 feet of spillway crest.

Positive Factors:

(1) It will take greater than the 100-year event for the spillway to discharge (less if principal spillway/outlet is plugged or multiple storms) and some what 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 a high hazard dams. Note however that the risk for potential loss of life existed prior to construction of the dam.
(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.
(4) Short spillway discharge duration for major floods up to and including PMF.
(5) Reinforced concrete spillway – good erosion control.
(6) Likelihood of spillway flows may be low.
(7) Have downstream Emergency Action Plan.

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. Failure From Overtopping Of Sunset FRS (Category II – Considered but not highlighted). (May move to Category IV – not credible).

Failure Mode Description: Overtopping of Sunset FRS would occur at the low point of the dam crest which from the 2003 crest monument survey indicates to be located on the left side of the dam embankment. Flow would overtop the dam at that point and, as flows increased, erosion of the crest and downstream slope potentially would lead to an eventual breach of the dam. The top of dam crest elevation based on as-built plans is 2141.5 ft (NGVD29). The 2003 crest monument survey indicates the low spot on the left side of the dam to be 2140.0 feet. The maximum PMF (HMR-49) routing water surface elevation is 2141.2 feet based on using as-built data for dam.

Adverse Factors:

(1) Less competent material on left abutment.
(2) Potential for erosion through backfill behind the spillway walls.
(3) Little release of floodwaters in advance of pool elevation reaching spillway crest.

(4) More likely to get PMP on smaller watershed.
(5) Potential for debris to clog orifice reducing principal outlet capacity.
(6) Downstream energy dissipater capacity only two-thirds of spillway design capacity.
(7) History of spillway training wall movement. May collapse into spillway during spillway discharge.
(8) Land use changes and density of urbanization increased since construction of dam.
(9) Only 0.3 feet of freeboard under HMR-49 PMP (based on as-built plans).
(10) Routing of multiple storms.
(11) Longer duration PMP (24-hr or 72-hr) may be more critical.

Positive Factors:
(1) PMP/PMF (6-hr) is a short duration storm
(2) Overtopping would be distributed over length of relatively level crest.
(3) Crest is 14-feet wide and gravel mulch on both upstream and downstream slopes
(4) Low spot on crest is easily filled and/or repaired.
(5) PMP is a rare event.
(6) Subsidence at dam not likely.
(7) Routing for PMF (HMR-49) started at full pool and assumed principal outlet plugged.
(8) Low potential for additional settlement of embankment

Potential Actions for Risk Reduction:
(1) Fill low spot on dam crest and maintain a level crest condition.
(2) Increase capacity of principal spillway outlet and/or improve the ability of the principal spillway to accept inflow at low reservoir levels. Add an additional orifice or increase the size of the existing orifice.
(3) Open low-level outlet gate during the rising limb of the flood.

Consequences:
(1) Population downstream from FRS is at risk.
(2) Downstream property damage
(3) 24-hour and/or multiple events may be more critical.
(4) Incremental damage assessment should be conducted.
(5) Upstream development will continue.

3. Slope Failure on Left Abutment (Category II – considered but not highlighted).

Background on conditions leading to the potential failure mode: According to the geologic report, the mid-to-upper left abutment contained loose waste fill of undetermined extent. The original plans indicated that the loose fill materials were to be completely removed to "firm residual soil" as directed by the engineer, and backfilled with Zone 1 compacted fill to restore grade to the original ground surface. The extent of over-excavation and replacement of the waste fill was anticipated to be approximately

250 feet in the upstream direction, and "as directed by Engineer" in the downstream direction. However, the extent of removal of the loose fill laterally (northward) into the abutment was constrained by the presence of an existing building at the top of the slope and it is not known for certain that "firm residual soils" were reached all along the left abutment contact. It is known that, to isolate and protect the abutment, the cutoff trench was wrapped upstream along the reservoir edge as a "dog-leg" extension about 400 ft upstream (westward) from the left abutment. The area over the upstream cutoff trench extension was also blanketed with clay to minimize seepage through the abutment. The clay blanketing extended from the cutoff trench elevation to the top of the embankment fill. Also to protect the left abutment from seepage the Zone II drain was brought up to elevation 2135 feet between Station 15+00 and 15+37 within the main dam section at the abutment contact to provide additional protection at higher elevations in this area. The top of Zone II is at elevation 2127 along the remainder of the embankment.

Failure Mode Description: This failure mode considers possible slope instability in the left abutment due to the possibility of the presence of weak, loose fill materials that could not be excavated, and possible low shear strength of some of the residual materials in that abutment. For this failure mode to develop during a flood event (considering the time it would take for seepage water to ingress within the embankment) requires precursor events or conditions that would pre-moisten and soften of the postulated weak abutment materials during sequential impoundment events, or by accumulated infiltration from runoff sources associated with the structures on the slope above. The risk of saturation of the mid-to-upper abutment during sequential impoundment events would be higher if the protecting clay blanket close to the axis of the FRS is ineffective due to erosion or disturbance by shallow slope instability, significant animal burrowing activity, or inadequate placement thickness in its original construction. The Potential failure mode initiates as aslope failure in the abutment during a significant flood event leading to potential loss of a portion of the dam crest at the abutment contact, followed by accelerated seepage through the shortened seepage path, continuing erosion of the crest and slumped material, and as flows increased, eventual erosion of the crest and downstream slope leading to concentrated overtopping at the abutment contact and eventual breach of the dam.

Adverse Factors:

(1) Surface water runoff from parking lot and rooftops of businesses above left abutment.
(2) Some evidence of low strength material.
(3) Some concern regarding seepage into left abutment area.
(4) Principal cutoff does not extend into abutment
(5) Possibility of a relatively short seepage path from the reservoir into the abutment through a flaw in the blanket near the beginning of the wrap around section.

Positive Factors:

(1) High water only for short duration
(2) Failure surface would be skewed (flatter than 2:1 slope)

(3) Area of left abutment was blanketed
(4) Over 400 feet of cutoff trench extended and around the corner along the left abutment.
(5) Blanket was carried to top of fill.
(6) Extended drain to intersect seepage.
(7) Storm event would be approaching PMF for concern.
(8) Would need to erode "slipped" material through direct or piping
(9) Mostly sand in abutment.

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 Sunset 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 issue as noted) were discussed but a potential failure mode was not identified for evaluation (descriptions of adverse and positive factors were not developed).

(1) Collapsible Soils in Left Abutment Materials Lead to Cracking – Seepage Erosion and Breach: Water during the flooding by passes the protective blanket or seeps through the FRS and saturates the soils in the left abutment. The collapsible soils assumed to be present for this candidate mode in the left abutment settle. The FRS then cracks and flow through the cracks leads to a breach. Based on a description of the materials, the presence of the blanketing, the short duration of flooding the anticipated diligence of the construction and the inspection of the construction, the team considered that there was little likelihood of collapsible soils being present and even if present there was little likelihood that enough settlement cracking and subsequent seepage erosion would be significant enough to lead to a breach. Thus this mode was considered too remote to be credible. The action called for relative to this candidate mode is to regularly visually monitor the left abutment between the end of the dam and the end of the abutment for any seepage and for any settlement. Also crest monuments should be extended beyond the left abutment contact to record rim elevation and monitor for settlement.

(2) Settlement of Downstream Portion of Principal Outlet Conduit. No evidence indicating settlement of principal outlet. Video of outlet shows no evidence of distortion. Only 5 feet of fill over downstream end of conduit. Therefore this issue was not addressed further and a candidate mode was not postulated. The FMEA team recommended continued routine monitoring of the conduit.

(3) Piping/Seepage Erosion Of A Fine Layer Adjacent To A Coarse Layer Of In-Situ Materials Around The Left Abutment Blanketing. The team estimated the flow path distance and compared it to the distance seepage water could move during a flood event. It was clear to the team that this potential mode is not

considered a single-event related failure mode, but would require multiple impoundment events to develop Hole would have to develop and collapse. Do not have enough time for seepage erosion process. Seepage erosion though "lenses" in left abutment. At this time there is no physical evidence that this failure mode is occurring, and there is no exploration evidence of pervasive coarse lenses that would provide concentrated seepage pathways. Monitor for any sign of this failure mode in left abutment during major flood and post flood operations by observation of seepage outlets.

(4) **No Water Stops in Spillway Chute Horizontal Joints**. Discharge through emergency spillway is short in duration.

(5) **Crack Exists In Dam Prior To Flood Or Develops At The Onset Of The Flood – Seepage Fills The Crack And The Flow Velocity Erodes/Expands The Crack Leading To A Breach (Root Cause - Zone II Filter Does Not Extend To Dam Crest)**. Zone II provides filter protection up to elevation 2127, but does not protect against possible seepage erosion due to transverse cracking through the dam crest.) This candidate mode was not carried further because (A) There is currently no evidence of transverse cracking on this dam, and its short crest length is a mitigating factor against cracking, and (B) for floods that rise above the spillway crest elevation there is inadequate time for seepage erosion to expand the crack to form a breach before the reservoir is drawn down by spillway flows and for flood water below the spillway crest it is highly likely that the free water surface would drop to the level of the filter protection.

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 Sunset FRS
Medium	Intermediate likelihood of occurrence for Sunset FRS
Low	Lowest likelihood of occurrence for Sunset 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 Sunset FRS
Medium	Intermediate inundation effects for Sunset FRS
Low	Lowest inundation effects for Sunset FRS

5.0 FAILURE MODE AND EFFECTS TABLE

Construction of the Failure Mode and Effects Table (Table 5) 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
Sunset 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 (200 yr Event)	High (PMF Event) Low (200 yr Event)	New flood routing Multiple storms 24-hr/72-hr PMP	Dam is instrumented as part of District ALERT system Have EAP Discharge duration of PMF is short	Increase capacity of principal outlet	
2. Overtopping	Reservoir inflow at Probable Maximum Flood	Downstream inundation	Low	Low	New flood routing Multiple storms 24-hr/72-hr PMP	PMF is short duration Gravel mulch on both slopes	Fill low spot on crest Increase capacity of principal outlet Open low level gated outlet	
3. Slope Failure of Left Abutment	Seepage erosion	Downstream inundation	Low	Low	Assess Parking lot drainage onto slope Slope protection	Left Abutment was blanketed Cutoff trench extended	Divert parking lot drainage to grouted left groin	

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 Sunset 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. 200 year event
Medium			
Low	1. PMF Event 2 3		

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 none of the three failure modes are 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, low consequences (for the 150 to 200 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 and magnitude.

7.0 SUMMARY AND CONCLUSIONS

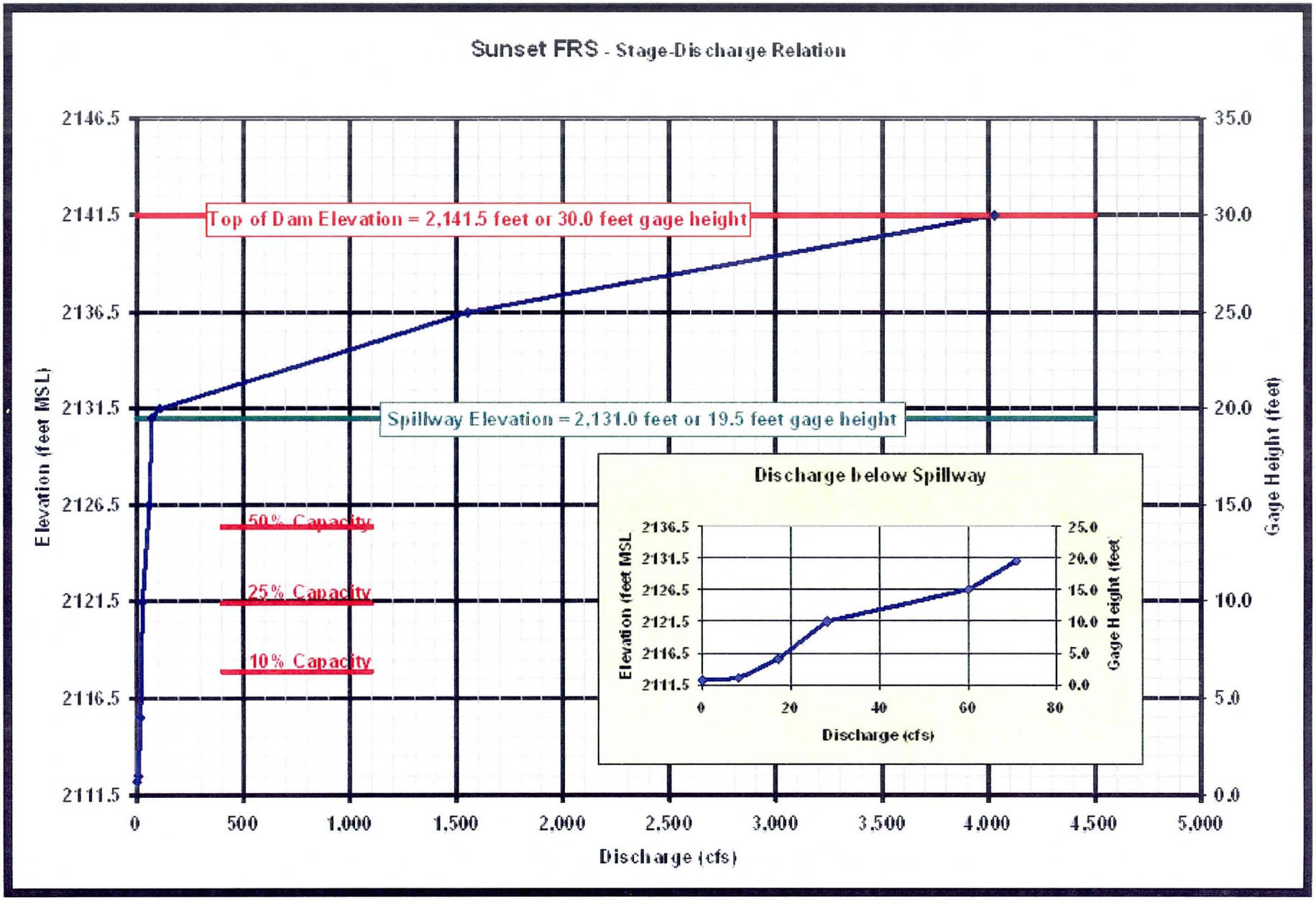
Sunset FRS was constructed pursuant to a relatively modern dam design. Construction appears to have been without any particular issues. The dam has performed normally and satisfactorily for 28 years. The structure is satisfactorily maintained and monitored.

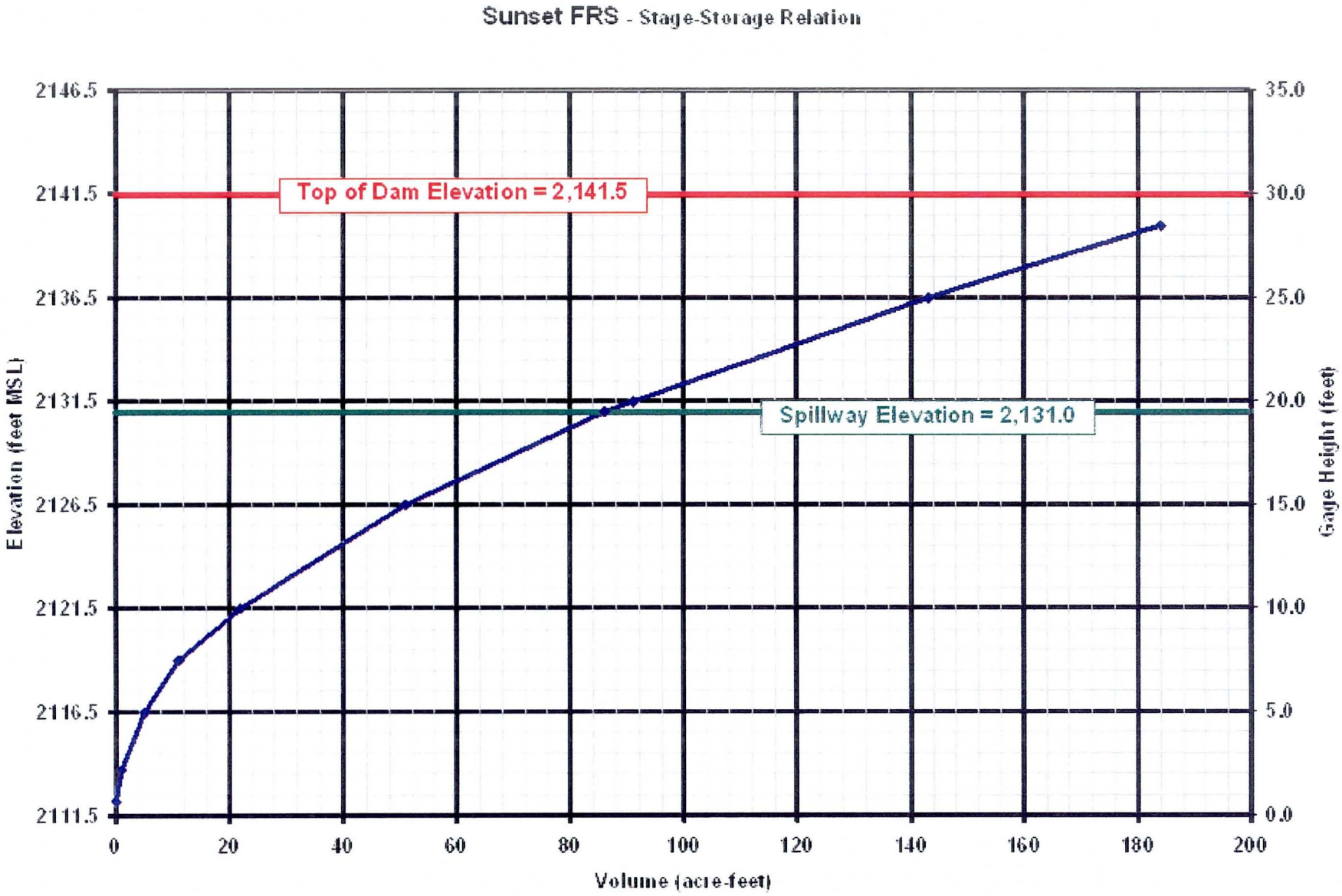
However, it is prudent to recognize that there exist for all dams specific ways that failure could come about that warrant attention and diligent monitoring. The identification of a condition or process as a "potential failure mode" does not imply that the dam is about to fail or even necessarily that 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.

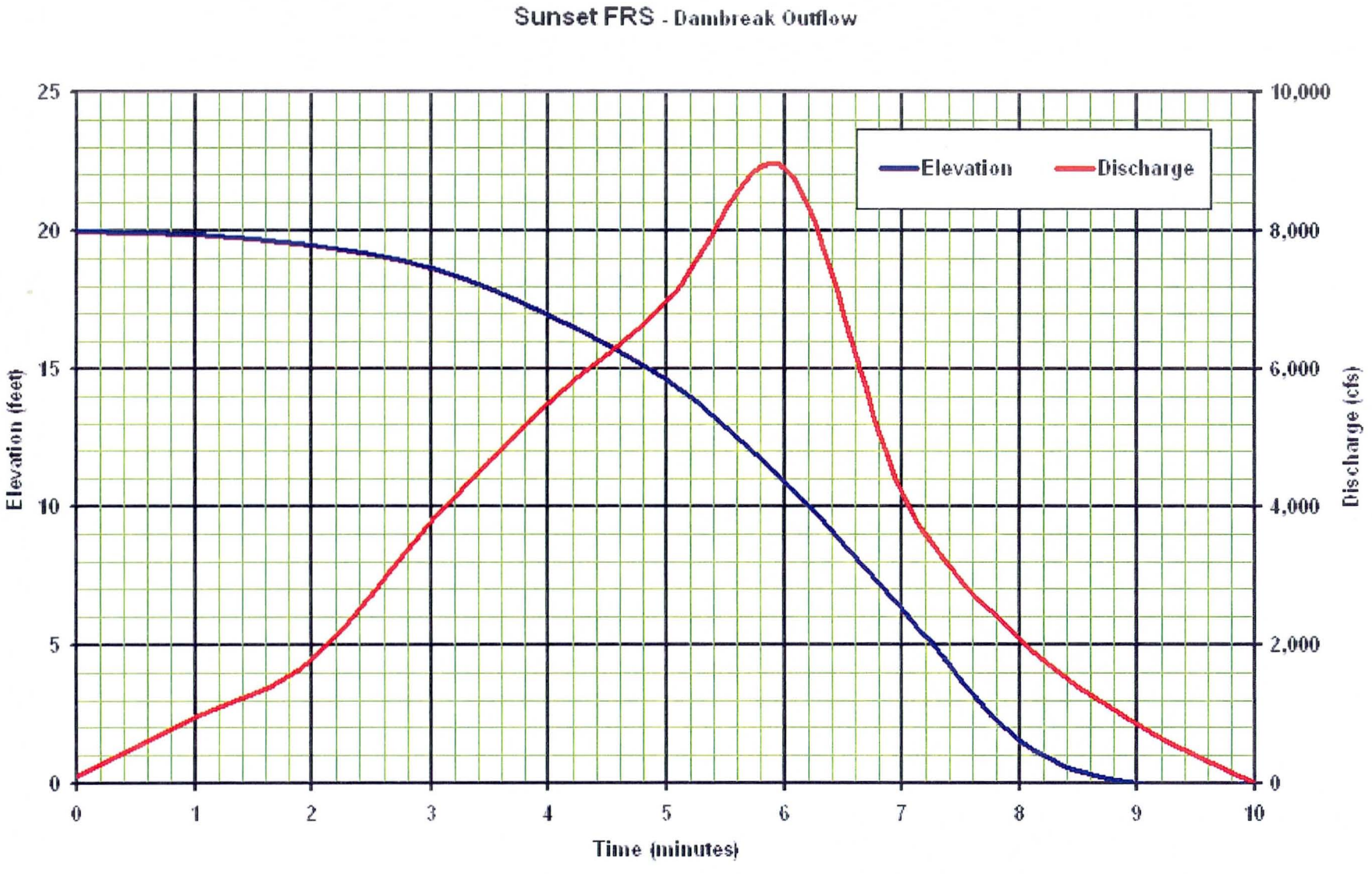
One Category I potential failure modes was identified by the FMEA team. The Category I failure mode is related to adverse consequences from normal operations of the emergency spillway during major rainfall events. The length of time the spillway flows is of short duration (a few hours). However, there 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.

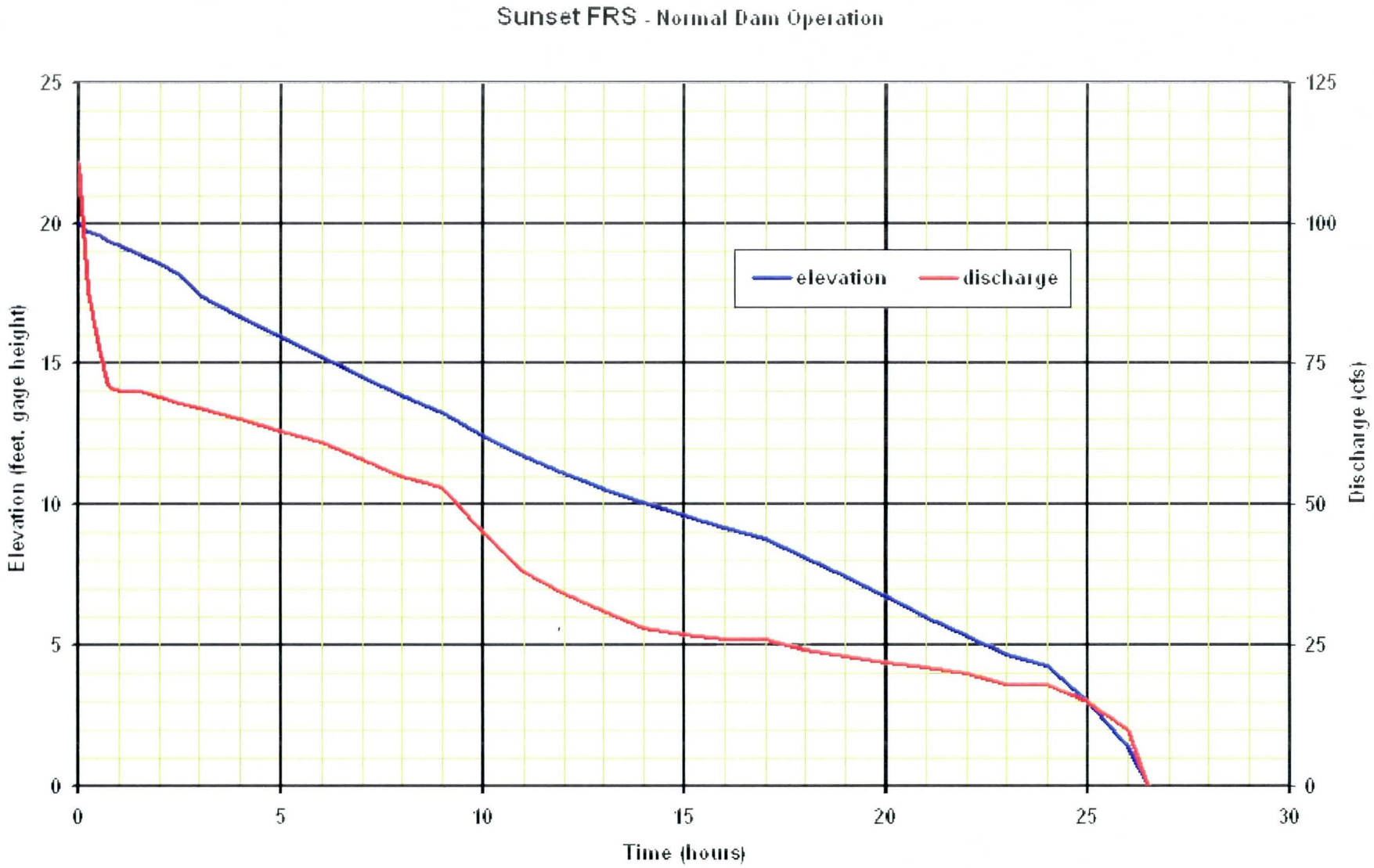
A number of potential risk reduction actions were identified by the team related to monitoring, information collection and documentation and modification of operations. These are all identified in the section on Major Findings and Understandings.

SUNSET FRS APPENDIX - FMEA REFERENCE MATERIALS


Sunset FRS - Stage-Discharge Relation
Top of Dam Elevation = 2,141.5 feet or 30.0 feet gage height
Spillway Elevation = 2,131.0 feet or 19.5 feet gage height
50% Capacity
25% Capacity
10% Capacity
Elevation (feet MSL)
Gage Height (feet)
Discharge (cfs)
2146.5
2141.5
2136.5
2131.5
2126.5
2121.5
2116.5
2111.5
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
0
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
4,500
5,000
Discharge below Spillway
Elevation (feet MSL
Gage Height (feet
Discharge (cfs)
2136.5
2131.5
2126.5
2121.5
2116.5
2111.5
25.0
20.0
15.0
10.0
5.0
0.0
0
20
40
60
80


Sunset FRS - Stage-Storage Relation
Top of Dam Elevation = 2,141.5
Spillway Elevation = 2,131.0
Elevation (feet MSL)
Gage Height (feet)
Volume (acre-feet)
2146.5
2141.5
2136.5
2131.5
2126.5
2121.5
2116.5
2111.5
35.0
30.0
25.0
20.0
15.0
10.0
5.0
0.0
0
20
40
60
80
100
120
140
160
180
200


Sunset FRS - Dambreak Outflow
Elevation
Discharge
Elevation (feet)
Discharge (cfs)
Time (minutes)
25
20
15
10
5
0
10,000
8,000
6,000
4,000
2,000
0
0
1
2
3
4
5
6
7
8
9
10


Sunset FRS - Normal Dam Operation
elevation
discharge
Elevation (feet, gage height)
Discharge (cfs)
Time (hours)
0
5
10
15
20
25
30
0
5
10
15
20
25
0
25
50
75
100
125

SUNSET FRS

FCD GAGE# 5232 AND 5233

STATION DESCRIPTION

LOCATION - The gage is located in the town of Wickenburg, approximately 1/2 mile west of downtown. From US 60 turn south on Mariposa Road (Jones Ford) to the wash. The dam is to the left. Latitude N33 57 50, Longitude W112 44 33. Located in the SW1/4 NE1/4 S11 T7N R5W in the Wickenburg 7.5-minute quadrangle.

ESTABLISHMENT - The gage was installed on February 12, 1989.

DRAINAGE AREA - 0.95 mi^2

GAGE - The gage is a pressure transducer type instrument, located at 0.13 feet gage height, or 2,111.59 feet M.S.L., levels of January 8, 2002.

There are eight staff gages at this location.

A staff plate is located near the pressure transducer. It reads in gage height, verified with levels of January 8, 2002.

Seven staff gages are located on the upstream side of the dam. The gages are in five foot segments and are subdivided into one foot increments. The gages read in gage height, verified with levels of January 8, 2002.

HISTORY - The gage had previously been at 1.14 feet gage height, 2,112.60 feet MSL, but was moved to its current location on January 28, 1994. The dam has been in place since 1976. Found PT diaphragm at 0.13 feet gage height during survey of January 8, 2002. Made effective beginning with Water Year 2002.

REFERENCE MARKS -

RM1 - This is reference 431 from the Wickenburg ADMS. It is a chiseled '+' on top of east concrete curb at the south end of pavement of Kellis Road. Elevation = 2,160.71 feet MSL, gage height 49.15 feet.

RP1 - White paint spot on northeast corner of concrete base for outlet gate control. Elevation = 2,143.35 feet MSL, gage height = 31.89 feet.

RP2 - White paint on concrete at base of first short post north of structure. Elevation = 2,120.36 feet MSL, gage height = 8.90 feet.

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233, http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

RP3 - Northernmost bolt on staff gage 0.18 feet below 14 feet. Elevation = 2,113.84 feet MSL, gage height = 2.38 feet.

RP4 - Top of angle iron on top of dam, north of gate. Tag 4USSFRS 3130. Elevation = 2,139.93 feet MSL, gage height = 28.47 feet.

RP5 - Brass cap in center of roadway on Kellis Road south of RM1. Elevation = 2,080.89 feet MSL, gage height = -30.57 feet.

CHANNEL AND CONTROL - The primary outlet for the dam is a 30 inch diameter, 184 foot long culvert. The auxiliary outlet for the dam is a concrete spillway located in the center of the dam.

PRIMARY / AUXILIARY OUTLET

The primary outlet is a 30-inch diameter culvert pipe. The culvert invert elevation at the inlet is 2,104.85 feet MSL or -6.40 feet gage height. The culvert invert elevation at the outlet is 2,104.10 feet MSL or -7.15 feet gage height. There are two intake orifices in the intake tower. The lower orifice is at elevation 2,111.46 feet MSL or 0.00 feet gage height. The upper orifice is at elevation 2,120.21 feet MSL, or 8.75 feet gage height. Flow begins in the intake tower at elevation 2,131.21 feet MSL, or 19.75 feet gage height.

The auxiliary spillway crest is at elevation 2,130.94 feet MSL or 19.48 feet gage height. The spillway is 40 feet wide and 10.5 feet high.

Top of dam elevation is 2,141.50 feet MSL (from the design).

RATING -

The current discharge rating is number 2 developed by R.W. Cruff in February 1992 utilizing a combination FHWA HY8 culvert analysis for flows through the primary outlet, and the weir equation for flows through the auxiliary spillway. The weir coefficient used was 2.90. Rating number 1 was developed by S.D. Waters in 1990.

The current capacity rating is rating #2 from the Wickenburg ADMS DTM study. The previous rating was from the Wickenburg ADMS HEC-1 input.

DISCHARGE MEASUREMENTS - The primary outlet is an underground culvert running to the Hassayampa River. Flows through the auxiliary spillway may be too dangerous to attempt.

POINT OF ZERO FLOW - Flow begins at 0.00 feet gage height or 2,111.46 feet MSL through the lower orifice.

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233, http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

FLOODS / SIGNIFICANT IMPOUNDMENTS -

REGULATION - The dam is a regulation of natural flows in Sunset Wash.

DIVERSIONS - None known

ACCURACY - Good

JUSTIFICATION - Monitor impoundment behind Sunset dam for flood warning to the town of Wickenburg.

UPDATE - January 17, 2002

DE Gardner

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

GAGE ID HISTORY

ID	Elev of Instr. in GH	Elev of Instr. in MSL	Period
5233	0.13	2,111.59	10/1/01 - present
5233	0.70	2,112.16	1/29/94 - 10/1/01
5233	1.14	2,112.60	8/10/92 - 1/29/94
3633	1.14	2,112.60	2/12/89 - 8/10/92

STAFF GAGE INFORMATION

STAFF GAGE RANGE	STAFF GAGE INFORMATION
0 - 35	STAFF GAGES IN FIVE FOOT SECTIONS, READ IN GAGE HEIGHT
0 - 18	STAFF PLATE ON OUTLET TOWER

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233, http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

SITE DATA

LOCATION	LOCATED IN WICKENBURG AT MARIPOSA DRIVE BEHIND JONES FORD
DRAINAGE AREA	0.95 MI^2
JURISDICTION	WICKENBURG, ARIZONA
WATERSHED	LOWER HASSAYAMPA
SECTION/TOWNSHIP/RANGE	SW1/4 NE1/4 S11 T7N R5W
LATITUDE / LONGITUDE	N 33 57 50 / W 112 44 33
USGS QUAD MAP	WICKENBURG 7.5-MINUTE
INSTALLATION DATE	FEBRUARY 12, 1989 (WY 1989)
LENGTH OF RECORD (AS OF 10/01/03)	14.63 YEARS
STAGE GAGE TYPE	PRESSURE TRANSDUCER
STAFF GAGE	EIGHT
ZERO GAGE HEIGHT ELEVATION	2,111.46 FEET M.S.L.
STAGE GAGE ELEVATION	0.13 FEET GAGE HEIGHT
POINT OF ZERO FLOW	0.00 FEET GAGE HEIGHT
SPILLWAY CREST ELEVATION	19.5 FEET GAGE HEIGHT
TOP OF DAM ELEVATION	35.0 FEET GAGE HEIGHT

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

RATING INFORMATION

RATING TABLE			
DISCHARGE RATING NUMBER 2, APPLIED AS OF OCTOBE			
CAPACITY RATING NUMBER 1, APPLIED AS OF FEBRUARY 12, 1989			
GAGE HEIGHT (FEET)	ELEVATION (FEET NGVD 29)	DISCHARGE (CFS)	VOLUME (ACRE-FEET)
0.0	2,111.46	0	0
1.0	2,112.46	8	0.1
5.0	2,116.46	19	5
10.0	2,121.46	28	22
15.0	2,126.46	60	51
19.5	2,130.96	71	86
20.0	2,131.46	111	91
25.0	2,136.46	1,551	143
30.0	2,141.46	4,030	200

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.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, 2/14, 10/26	4.91	5
2002	9/9/02	4.93	5
2001	10/27/00	8.43	14
2000	8/29/00	7.78	12
1999	7/15/99	6.80	10
1998	10/1/97	6.62	9
1997	9/26/97	12.27	34
1996	9/11/96	7.20	11
1995	8/14/95	7.28	11
1994	10/6/93	6.64	9
1993	2/9/93	7.04	10
1992	8/22/92	9.44	
1991	8/11/91	4.84	
1990	7/6/90	8.14	
1989	7/10/89	4.14	

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.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
11/13/03	18:29	11/12 15:30 - 11/28 08:58	377	3.76	2	2004
9/4/03	20:21	09/04 18:31 - 09/09 09:36	111	1.98	<1	2003
8/27/03	12:45	08/27 09:40 - 09/04 14:26	197	1.93	<1	2003
8/20/03	14:45	08/19 11:20 - 08/27 05:15	186	1.65	<1	2003
8/14/03	23:30	08/14 22:05 - 08/18 22:50	96.8	3.81	2	2003
/28/03	02:47	07/28 01:37 - 08/04 22:50	189	3.58	2	2003
3/17/03	03:54	03/16 11:09 - 03/25 17:49	223	3.06	1	2003
2/25/03	22:05	02/25 12:25 - 02/28 17:19	76.9	4.91	5	2003
2/14/03	04:15	02/12 22:39 - 02/20 08:10	178	4.91	5	2003
1/8/03	10:37	01/08 04:42 - 01/10 04:02	47.3	0.98	<1	2003
11/30/02	04:54	11/30 04:49 - 11/30 07:54	3.1	0.95	<1	2003
10/26/02	14:17	10/26 13:27 - 10/29 18:02	76.6	4.91	5	2003
9/9/02	09:27	09/06 19:12 - 09/19 12:07	305	4.93	5	2002
7/14/02	22:54	07/14 22:49 - 07/20 18:24	140	1.48	<1	2002
7/10/02	00:24	07/09 22:34 - 07/14 12:24	110	1.58	<1	2002
10/8/01	00:09	10/07 23:15 - 10/10 13:25	62.2	4.45	5	2002
/6/01	05:59	07/06 04:55 - 07/10 03:19	94.4	4.32	3	2001

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

3/7/01	14:12	03/07 02:28 - 03/21 08:42	342	4.68	4	2001
1/15/01	18:22	01/15 17:37 - 01/15 22:00	4.4	1.60	<1	2001
10/27/00	13:47	10/27 12:21 - 10/30 09:10	68.8	8.43	14	2001
10/22/00	05:33	10/21 09:09 - 10/27 04:28	139	7.03	10	2001
8/29/00	12:07	08/29 09:35 - 09/07 06:12	213	7.78	12	2000
8/27/00	11:12	08/27 06:42 - 08/29 09:19	50.6	6.62	9	2000
3/5/00	23:57	03/05 21:12 - 03/11 21:18	144	2.42	1	2000
2/21/00	18:50	02/21 18:37 - 02/23 21:19	50.7	1.70	<1	2000
9/11/99	21:23	09/11 20:22 - 09/16 07:15	107	2.10	1	1999
8/31/99	17:46	08/28 20:05 - 09/02 22:06	122	6.70	9	1999
7/29/99		No Information		3.10	1	1999
7/15/99	05:39	07/15 03:55 - 08/14 21:22	737	6.80	10	1999
7/6/99	19:25	07/06 19:03 - 07/09 01:17	54.2	1.85	<1	1999
6/2/99	09:57	06/02 09:31 - 06/04 09:23	47.9	1.60	<1	1999
4/2/99	08:21	04/02 03:42 - 04/13 21:23	282	3.10	1	1999
12/2/98	21:35	12/02 11:36 - 12/04 13:33	50.0	1.58	<1	1999
11/29/98	21:58	11/28 21:03 - 12/02 09:35	84.5	2.17	1	1999
10/30/98	19:48	10/30 19:13 - 11/02 17:19	70.1	2.45	1	1999
10/26/98	07:48	10/26 07:36 - 10/27 09:33	26.0	1.90	<1	1999
9/4/98	09:55	09/04 08:32 - 09/04 12:06	3.6	1.90	<1	1998
9/1/98	00:23	08/31 23:59 - 09/01 09:32	9.6	2.25	1	1998
8/24/98	21:39	08/24 21:19 - 08/24 23:45	2.4	2.28	1	1998

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

8/17/98	17:49	08/17 17:43 - 08/18 09:32	15.8	1.80	<1	1998
8/13/98	18:43	08/12 18:35 - 08/14 22:25	51.8	5.38	6	1998
8/8/98	02:25	08/07 23:46 - 08/12 16:01	112	3.10	1	1998
7/21/98	18:43	07/19 21:31 - 07/25 09:32	132	2.15	1	1998
5/13/98	07:52	05/13 07:20 - 05/15 05:35	46.3	1.58	<1	1998
4/12/98	11:53	04/12 11:36 - 04/18 21:51	154	1.80	<1	1998
3/28/98	16:52	03/26 02:18 - 04/10 21:51	154	2.45	1	1998
2/17/98	22:30	02/15 00:19 - 02/14 21:51	442	4.05	2	1998
2/9/98	21:51	02/03 22:41 - 02/14 21:51	263	3.53	1	1998
1/10/98	15:26	01/10 12:14 - 01/21 13:15	265	2.65	1	1998
1/4/98	17:59	01/03 01:16 - 01/10 01:16	168	1.98	<1	1998
12/22/97	05:41	12/21 22:17 - 01/01 13:15	255	3.05	1	1998
12/7/97	03:45	12/07 01:14 - 12/16 13:15	228	2.47	1	1998
9/26/97	06:42	09/25 15:05 - 10/06 01:52	251	12.27	34	1997
9/23/97	12:29	09/23 10:49 - 09/24 22:58	36.2	2.12	1	1997
9/5/97	22:52	09/05 21:07 - 09/15 23:44	243	4.25	3	1997
8/9/97	00:28	08/08 20:46 - 08/16 10:53	182	2.67	1	1997
8/5/97	16:34	08/05 15:48 - 08/08 10:51	67.1	2.42	1	1997
2/28/97	00:50	02/27 13:30 - 03/07 23:19	202	2.67	1	1997
1/13/97	23:17	01/13 02:50 - 01/18 23:18	140	2.08	1	1997
9/11/96	02:30	09/10 20:47 - undetermined		7.20	11	1996
9/5/96	06:47	09/05 06:34 - 09/06 21:10	38.6	1.50	<1	1996

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

7/28/96	00:10	07/27 23:31 - 08/02 22:22	143	2.58	1	1996
7/25/96	22:56	07/25 22:46 - 07/27 21:51	47.1	1.75	<1	1996
3/14/96	02:24	03/14 01:06 - 03/18 18:22	1	2.08		1996
2/26/96	16:59	02/26 16:40 - 02/28 13:23	44.7	1.33	<1	1996
11/1/95	18:59	11/01 16:38 - 11/05 00:39	80.0	1.90	<1	1996
8/14/95	23:46	08/14 22:36 - 08/16 20:36	46	7.28	11	1995
7/14/95	19:30	07/14 18:04 - 07/31 16:05	406	5.95	7	1995
2/15/95	00:00	02/14 19:15 - 02/18 19:56	97.6	5.45	6	1995
1/26/95	01:25	01/25 18:50 - 02/05 19:14	264	4.38	3	1995
1/5/95	05:29	01/05 00:36 - 01/11 05:48	149	2.72	1	1995
12/25/94	22:41	12/25 19:16 - 12/29 05:05	81.8	1.70	<1	1995
9/20/94	21:21	09/20 21:11 - 09/21 02:36	5.4	1.70	<1	1994
11/15/93	11:20	11/13 11:05 - 11/19 11:36	145	2.34	<1	1994
10/6/93	12:46	10/06 11:46 - 10/29 05:15	546	6.64	9	1994
9/12/93	20:12	09/12 20:07 - 09/15 14:48	66.7	1.94	<1	1993
8/29/93	02:32	08/29 01:47 - 09/03 02:07	120	2.64	1	1993
2/28/93	04:32	02/27 22:50 - 03/01 04:08	29.3	2.84	1	1993
2/19/93	22:39	02/19 20:46 - 02/20 13:09	16.4	2.04	<1	1993
2/9/93	16:49	02/08 02:50 - 02/14 09:01	150	7.04	10	1993
1/17/93	16:50	01/16 09:35 - 01/23 03:51	162	4.34	3	1993
1/14/93	07:20	01/13 19:55 - 01/16 09:27	61.5	3.04	1	1993
1/12/93	23:57	01/12 00:36 - 01/13 12:20	61.5	3.04	1	1993

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

1/10/93	22:45	01/10 01:38 - 01/12 00:32	46.9	3.64	1	1993
1/8/93	04:40	01/06 21:03 - 01/10 01:34	76.5	3.24	1	1993
12/28/92	19:26	12/28 18:57 - 01/04 16:19	165	3.14	1	1993
12/8/92	04:55	12/08 01:16 - 12/13 06:18	125	2.24	1	1993
8/22/92	22:23	08/20 22:30 - 08/28 19:21	189	9.44	18	1992
8/11/92	19:26	08/11 17:33 - 08/13 04:45	35.2	3.54	1	1992
8/6/92	12:13	08/06 11:51 - 08/08 00:53	37.0	2.04	<1	1992
7/11/92	02:58	07/10 20:50 - 07/14 19:26	94.6	2.04	1	1992
5/21/92	11:19	05/21 10:57 - 05/21 19:34	8.6	1.64	<1	1992
5/9/92	21:37	05/09 21:30 - 05/10 00:47	3.3	1.64	<1	1992
3/31/92	10:31	03/31 10:20 - 03/31 18:17	8.0	1.64	<1	1992
3/9/92	03:42	03/08 00:27 - 03/09 19:03	42.6	1.94	<1	1992
2/13/92	12:47	02/13 06:18 - 02/15 17:07	58.8	3.84	1	1992
2/7/92	13:20	02/07 02:21 - 02/10 17:18	87.0	5.24	5	1992
1/6/92		No Information		5.04	5	1992
8/11/91	06:45	08/11 05:18 - 08/24 06:00	313	4.84	4	1991
3/27/91	17:30	03/26 08:00 - 03/28 01:59	42.0	4.74	4	1991
3/1/91	17:34	02/28 01:49 - 03/16 06:57	389	4.74	4	1991
8/14/90	14:03	08/14 12:39 - 08/16 06:50	42.2	2.94	1	1990
7/6/90	17:30	07/06 15:48 - 07/14 05:11	181	8.14	13	1990
1/17/90	16:27	01/17 14:23 - 01/21 11:39	93.3	2.54	1	1990
/26/89		No Information		2.24	1	1989

Source
Flood Control District of Maricopa County, Sunset FRS ID #5233,
http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.

7/10/89	22:43	07/10 20:32 - 07/18 05:12	177	3.00	2	1989
7/8/89	20:08	07/08 19:28 - 07/10 15:37	44.2	3.74	2	1989

Source

Flood Control District of Maricopa County, Sunset FRS ID #5233, http://156.42.96.39/alert/Flow/5233.htm, December 11, 2003.


Mar 1 2004 7:13 AM
SUNSET FRS
SENSOR 5233
STRUCTURE DATA
TOP OF DAM..... 2140.0
SPILLWAY ELEV... 2131.0
MAX.DEPTH..... 28.5 ft
SPILLWAY CAP.. 86 ac-ft
SPILLWAY DEPTH. 19.5 ft
STRUCTURE DIMENSIONS
LENGTH..... 488 ft
TOP CREST..... 14 ft
DRAW DOWN..... 10 days
CURRENT CONDITIONS:
Stage (ft) ----> 0.2
Discharge (cfs) -> 0
Volume (ac-ft) --> 0.0
Percent Full (%)-> 0
POOL STAGE (ft)
TOP @ 28.5 ft
Emergency Spillway
Notify Emergency Management (50% Full)
Notify O&M (25% Full)
ALARM @ 6.4 ft
24
21
18
15
12
9
6
3
0
PROBLEM AREAS
All residences downstream of emergency spillway
Storm sewer surge for outlet dischages > 10 cfs
Note: FCD ALERT staff gage shows depth above PT
BASIN RAINFALL
I.D. 15m 60m 3hr WY-to-D
5230 0.00 0.00 0.00 2.48
5245 0.00 0.00 0.00 2.56
5260 0.00 0.00 0.00 2.05
60 Mns s#5233
24 Hrs s#5233
7 Dys s#5233
Sensor Offset = +0.13 feet
Significant Storage Begins at 3.5 ft

GEOLOGICAL CONSULTANTS INC

KENNETH M. EUGE, R.G.

Memorandum

May 3, 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
Sunset 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 Sunset 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.

Sunset FRS

Geologic Setting

Sunset 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' 55" N and 112° 44' 30" 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 northeast quarter of Section 11, Township 7 North, Range 5 West (Figure 1, Site Location Map).

The geology (Figure 2) of the project area is briefly described in the Report of Geologic Investigation, Wickenburg Watershed, Sunset Wash 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 2004).

The geology of the Sunset FRS area includes bedrock described as "Tertiary alluvial deposits of silty, gravelly sand which is dense and which is cemented to various degrees to form a caliche type material. Within this material are some zones with little or no cementation." The alluvial deposits reportedly form portions the abutments for the dam.

Photo 1: Exposure of Tertiary age fanglomerate in the downstream right abutment area. Embankment to left of erosion gully.

Exposures in the right abutment appear to similar to the Tertiary age fanglomerate exposed at the Sunnycove FRS site located about one-half mile to the south. 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 structurally massive and it is moderately to well stratified. These deposits underlie a thin mantle of recent alluvium in the stream floodplain. The fanglomerate, as well as the variably cemented alluvial deposits, is complete covered upstream and downstream from the embankment in the left abutment area with engineered cut-off earth fill and by probably uncontrolled earth fill associated

with residential and commercial development adjacent to the site.

Photo 2: Right abutment upstream from dam. Fill area associated with the cutoff extended upstream to isolate large area of uncontrolled dumped fill from the left abutment. Commercial area parallel to U.S. Highway 60 in upper right of view.

At the time of the SCS geological investigation, topographic features in the dam site area had been "considerably modified by excavations and filling within the area." According to the site plan and profile (Figures 3 and 4), about 20 feet of loose trash fill was located at the left abutment that required a substantial modification and relocation of the cut off upstream of the actual left abutment to isolate the trash fill area (Photo 2). Apparently some of the washes had been changed by filling and were "no longer distinguishable at the surface."

Structurally, the strata underlying the Sunset FRS are believed to nearly flat lying or tilted at a very low angle to dip toward the northeast similar to the deposits exposed in the Sunnycove FRS site area. No structural discontinuities such as faults, joints, or fractures were observed in the limited exposures of the formation this site nor are any reported in the dam investigation documentation.

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 Principal Spillway

Sunset FRS abutments are founded, in part, on dense weakly to moderately cemented alluvial terrace deposits. The cutoff trench extends into the moderately to well-cemented caliche cemented deposits. These materials were very hard to drill and had standard penetration test blow count values of greater than 100 blows per foot. This material is believed to be the dense Tertiary age fanglomerate which is weathered at the surface exposures depicted in Photo 1. Based on our review of boring logs, the depth to the alluvial terrace deposit and fanglomerate boundary is quite variable ranging from about 10 feet below original ground surface at the right abutment, 9 feet to 21 feet in the central portion of the embankment, and 39 feet at the left abutment. 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 5 (SCS Drawing 7-E-23089, Sheet 4R of 28)e dam showing the excavation depth into the fanglomerate.

Young alluvial terrace deposits are present throughout the left abutment where the younger deposits consist of stratified terrace alluvium. The young terrace deposits are poorly graded, gravelly, fine to coarse-grained sand with 10 percent to 20 percent silt (SP,SM, SP-SM). A substantial area near the left abutment includes a dump fill consisting of a heterogeneous mixture of construction debris and trash. Because of the loose consistency of the dumped fill and debris mixed with younger terrace deposits at the left abutment, the left abutment was over-excavated to remove as much of the unsuitable soils as possible without jeopardizing private property adjacent to the left abutment. Also, an area extending about 225 feet upstream from the left abutment includes a dumped fill used to construct a parking lot (see Photo 2). In-place density tests taken in this apparently uncompacted embankment indicate little or no compaction. The fill has an in-place density of 86.6 pounds per cubic foot or about 73 percent of its maximum dry density. To construct the left abutment area, extensive excavation was required to remove a portion of the unsuitable fill and to construct a cutoff trench that extends upstream and parallel to the existing slope. A compacted buttress fill embankment was constructed to bury the cutoff trench and to support the uncompacted parking lot fill. The location of the extended cutoff and buttress fill slope is depicted in Figure 6 (As-Built SCS Drawing 7-E-23089, Sheet 2AR of 28).

Flood plain deposits include variable and stratified, lenticular alluvial deposits that are composed of clean well-graded sand (SW) to silty sand (SM) (Photo 3). Because of the lenticular character

of these deposits, they are not expected to be laterally extensive. These deposits extend to depths ranging from two to eleven feet below grade and unconformably the variably cemented alluvial terrace deposits. In-place density tests in these materials range from 92.8 to 100.8 pounds per cubic foot, or about 75 to 86 percent of their maximum dry density.

Photo 3: View looking upstream from Sunset FRS. Floodplain excavated for borrow material has modified the drainageway.

The Principal Spillway crosses perpendicular to the embankment at dam centerline station 12+20. The Principal Spillway is founded in the cemented alluvial sediments (fanglomerate).

Emergency Spillway

A concrete lined emergency spillway is constructed on the Sunset FRS. The centerline of the emergency spillway crosses the embankment dam centerline at station 11+85. The spillway control section is founded on the Zone 1 embankment fill. The section of the spillway shoot constructed beyond the limits of the embankment and the riprap outlet area are founded on prepared floodplain deposit soils (Photo 5).

Photo 4: Principal Spillway riser structure. North tributary inlet in background.

Site Investigation

Photo 5: Emergency spillway and riprap discharge channel looking upstream. Note principal spillway outlet to right of spillway chute, right-center of view.

The Soil Conservation Service initiated its dam site investigation in July, September, and October 1974. The geological characterization of the site was defined using several techniques including: geologic reconnaissance and mapping, auger drilling, and backhoe trenching. In-place density test and inflow permeability tests were made along the dam centerline. Test pit and drill hole location are depicted on Figure 3 (Plan of Dam and Reservoir; SCS Drawing 7-E-23089, Sheet 2 of 28). They are also depicted along the dam centerline on Figure 4 (Profile on CL Dam & Excavation Schedules; SCS Drawing 7-E-23089, Sheet 3R of 28). A summary of the explorations is provided in the following table.

Table 1. Summary of Geologic Testing
Sunset FRS

Structure	Exploration Techniques			
	Exploratory Borings	Test Trenches/ Pits	In-Place Density Tests	In-Flow Permeability Tests
Dam	8	6	4	3
Principal Spillway	4	2	-	-
Emergency Spillway	5	1	-	-
Borrow Site	11	-	-	-

Seismicity

No seismicity or earthquake evaluation was conducted for the Sunset 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).

Sunset 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 Sunset 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 Sunset 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 Sunset FRS. The subsurface geological conditions in the embankment dam area (Figures 3 and 4), consists of relatively hard, cemented Tertiary age fanglomerate and in the subsurface deposited on crystalline bedrock indicate the potential for land subsidence due to groundwater withdrawal does not exist at the Sunset 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 Sunset FRS structure. Although land subsidence is not expected to affect the Sunset 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 Sunset FRS project area. Geological conditions in the Sunset FRS area preclude the development of earth fissures at this site.

Bibliography-Sunset 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.

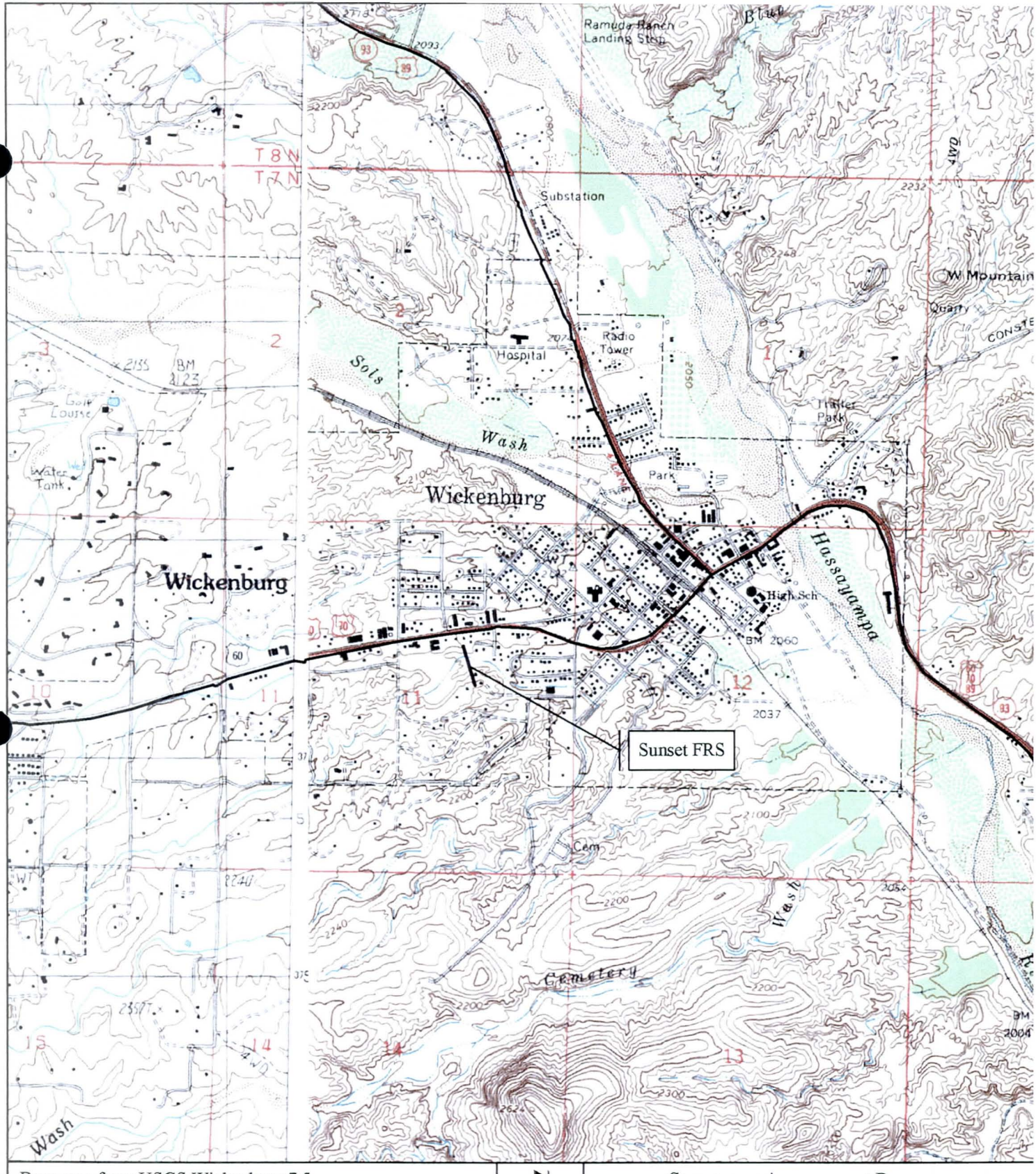
Cooley, M.E.; 1973; Map Showing Distribution and Estimated Thicknesses of Alluvial Deposits in the Phoenix Area, Arizona; U.S. Geological Survey Miscellaneous Investigation Series Map I-845-C; 1 sheet; Scale 1:250,000.

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, Sunset Wash Flood Retarding Structure; prepared by Aubrey Sanders; December 27, 1974; 7 p.

Soil Conservation Service; 1975; Sunset FRS Foundation & Embankment Design Report (Draft); includes embankment slope stability analysis and seepage analysis; 6 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;



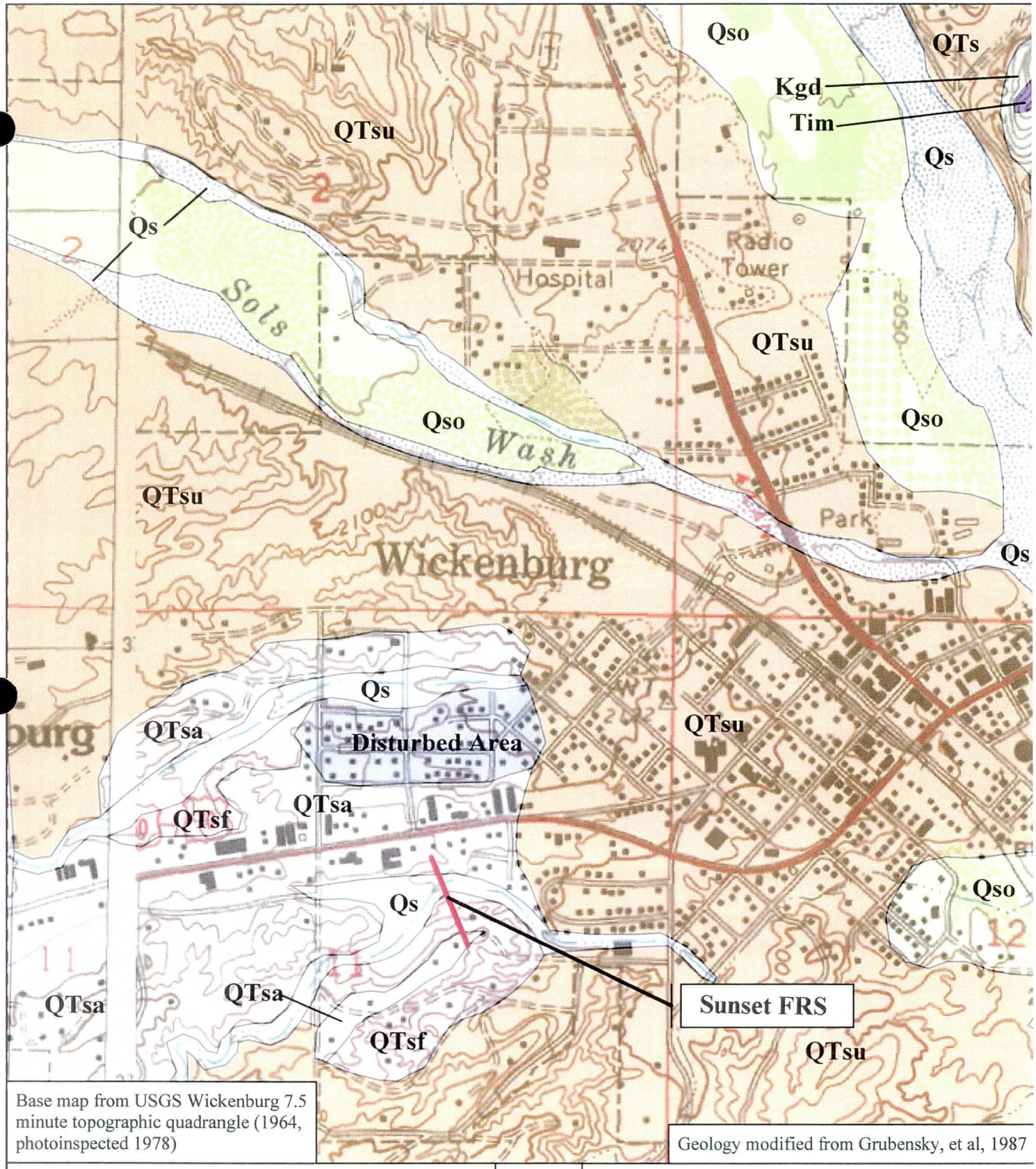
Base map from USGS Wickenburg 7.5 minute topographic quadrangle (1964, photoinspected 1978)

0 0.25 0.5 1 Miles

1:24,000

Structures Assessment Program
Sunset 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



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

Geology modified from Grubensky, et al, 1987

EXPLANATION

Qs - Younger Alluvium
Qso - Older Alluvium
QTsu - Sedimentary Rocks Udifferentiated
QTsa - Alluvial Terrace Deposits
QTsf - Fanglomerate
Tim - Mafic Dikes and Plugs
Kgd - Granodiorite

N

1:12,000

Structures Assessment Program
Sunset FRS
Geologic Map
Figure 2

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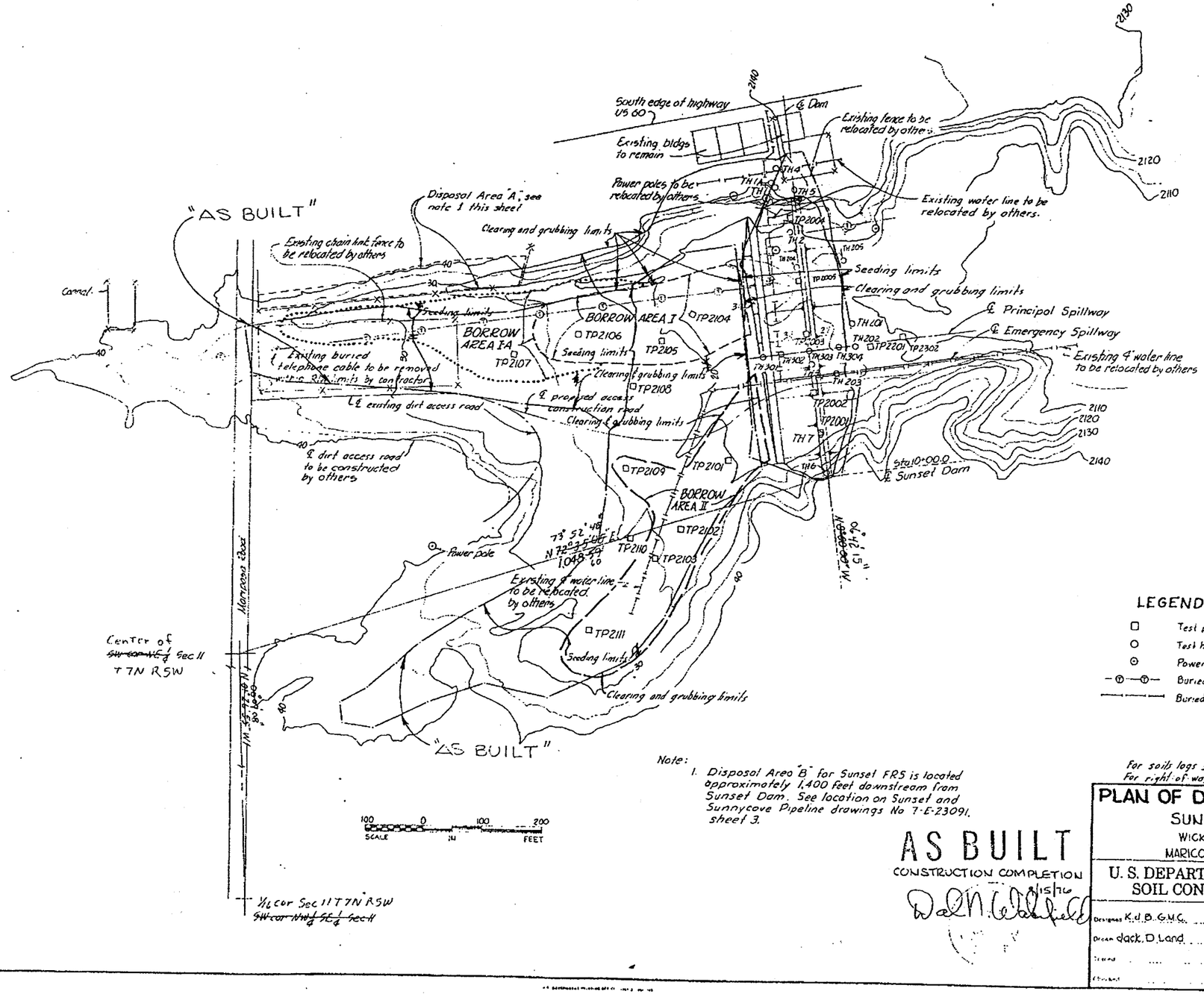
fax 602-864-1899

Structures Assessment Program
Sunset FRS

Figure 3
Plan of Dam & Reservoir

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



LEGEND

- Test pit
- Test hole
- Power pole
- Buried telephone cable
- Buried pipeline

Note:

1. Disposal Area "B" for Sunset FRS is located approximately 1,400 feet downstream from Sunset Dam. See location on Sunset and Sunnycove Pipeline drawings No 7-E-23091, sheet 3.

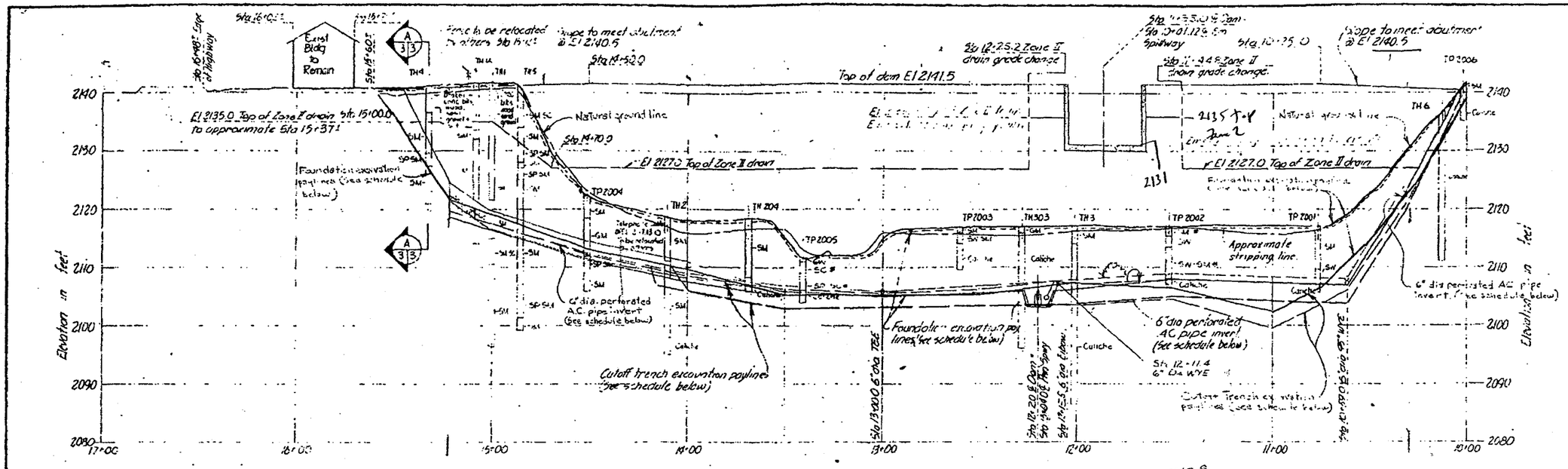
For soils logs see sheets 3 and 25
For right-of-way limits see sheet 2AR

AS BUILT
CONSTRUCTION COMPLETION
2/15/76

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

Designed K.J.B. G.M.C.	Date 2-75	Approved by	
Drawn Jack D. Land	2-75	Title	
Traced		Sheet No. 2	Drawing No. 7-E-23089
Checked			



PROFILE ON ℄ DAM

(Looking downstream)

SCHEDULES

℄ FOUNDATION EXCAVATION

STATION	ELEVATION
10+00 ±	2140.5
10+30	2123.0
10+59	2107.0
11+50	2108.0
12+00	2106.0
12+09	2103.4
12+21	2103.4
12+23.6	2100.9
13+00	2105.0
13+50	2106.0
14+50	2112.0
15+00.22	2120.0
15+60 ±	2140.5

℄ Elevation at ℄ of Dam

℄ CUTOFF TRENCH EXCAVATION

STATION	ELEVATION
10+00 ±	2139.0
10+30	2121.0
10+59	2104.0
11+50	2105.0
12+00	2103±5
13+50	2103.0
14+00	2106.0

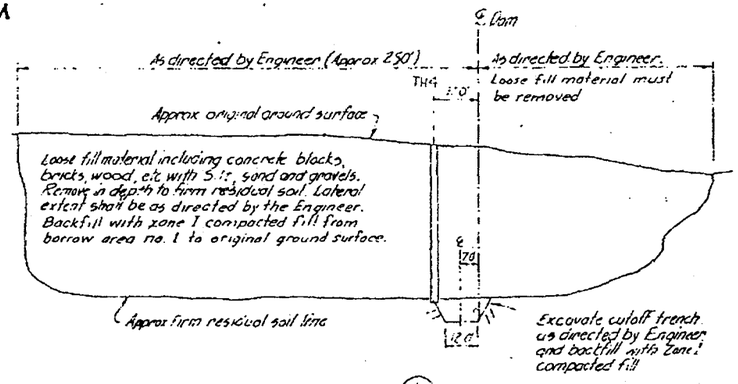
℄ Elevation at 70' upstream from ℄ of Dam

℄ DRAIN PIPE PLACEMENT

STATION	ELEVATION	REMARKS
10+30.0	2124-2123.0	Plug beginning end with mortar or AC cap
10+59.0	2108-2107.0	CI 45° WYE 6" dia non-perf AC outlet pipe.
11+50.0	2109-2108.0	Plug beginning ends with mortar (2 inch) or ℄ cap
12+00.0	2107.0	Grade change. CI WYE
12+11.4	2108.0	Grade change. CI WYE
12+15.5	2105.97	CI 90° Elbow to 6" dia non-perf AC outlet pipe
12+20.0	2106.0	Two pipe joints need within 1'ea. of ℄ prmt. spwy Sta 12+15
12+50.0	2106.5-105.5	Grade change
13+00.0	2106.5-105.5	CI TEE to 6" dia non-perf AC outlet pipe.
13+50.0	2107.0	Grade change.
14+00.0	2110-9	Grade change.
14+50.0	2111-2117.8	Grade change.
15+00.0	2117-2117.5	Grade change.
15+20.0	1.0 Above foundation excavation line approved by Engineer.	Plug beginning end with mortar or AC cap

℄ Elevation at 100' downstream from ℄ of Dam

℄ All elevations and stations are approximate and may be adjusted by the Engineer during construction.



SECTION (A 3/3) NOT TO SCALE

AS BUILT

CONSTRUCTION COMPLETED 1/15/76

Dan N. Wakefield

Structures Assessment Program
Sunset FRS

Figure 4

Profile on Centerline of Dam

Geological Consultants Inc.

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

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 at Wickenburg Arizona.
3. For locations of Test pits and Test holes see sheet 2.

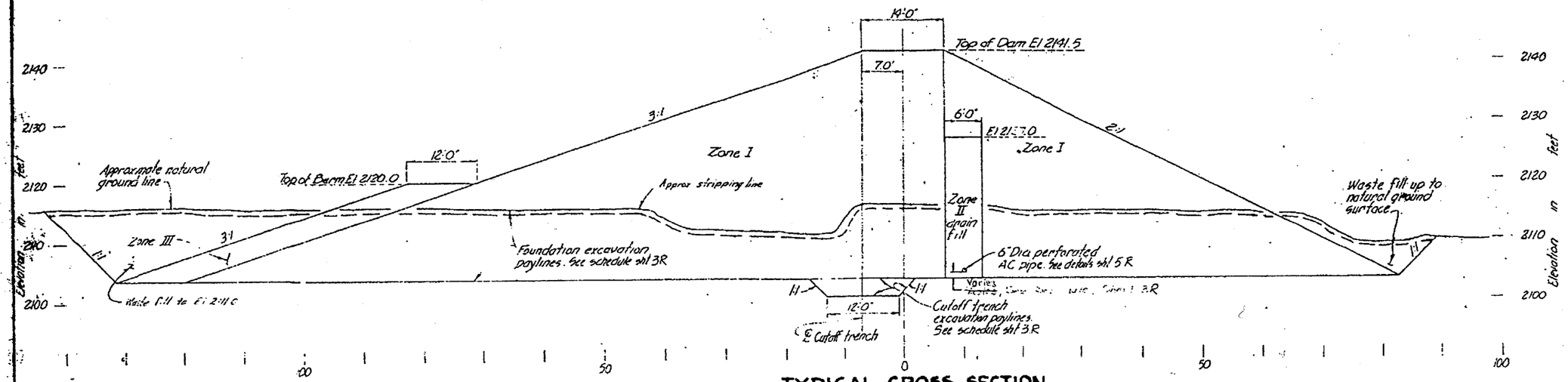
PROFILE ON ℄ DAM AND EXCAVATION SCHEDULES
SUNSET F.R.S.
WICKENBURG W.P.D.
MARICOPA COUNTY, ARIZONA.

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

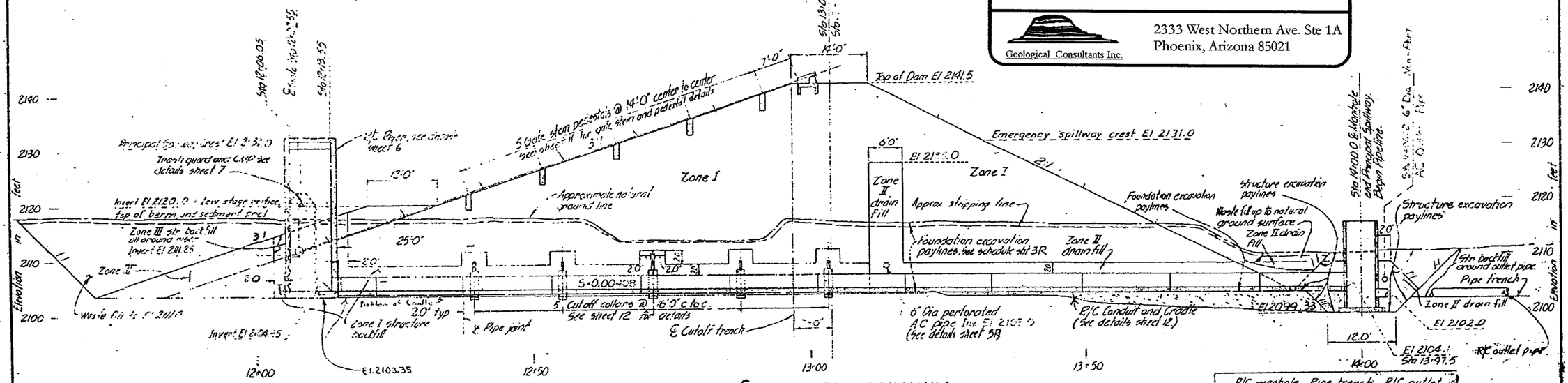
	Date
Designed Greg Cunningham	8-75
Drawn dock Land.	8-75
Traced	
Revised T. Jolo	4-76

Approved Ralph N. Cuneo
Title
Date
Sheet 9R of 28
Drawing No. 7-E-23089

SCS-ENG-311-R Rev. 7-69

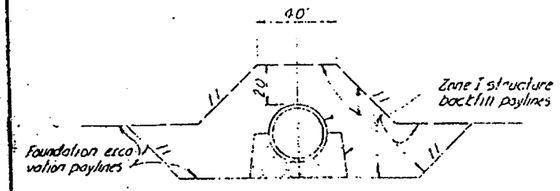


TYPICAL CROSS SECTION OF DAM

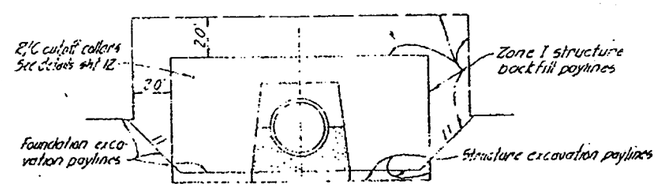


PROFILE ON ℄ PRINCIPAL SPILLWAY

RIC manhole, Pipe trench, RIC outlet pipe and Structure backfill around outlet pipe are part of bid schedule 3.



R/C CRADLE PAYLINES
STA 12+16.5 to STA 13+11.0
(See sheet 5 for Sta 13+11 to Sta 14+00)



CUTOFF COLLAR PAYLINES

AS BUILT
CONSTRUCTION COMPLETED
3/15/76
Dale N. Wakefield

Structures Assessment Program
Sunset FRS

Figure 5
Typical Cross Section of Dam

Geological Consultants Inc.
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Phoenix, Arizona 85021

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

U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

	Date		
Designed Greg Cunningham	3-75	Approved	
Drawn Jack Land	3-75	Title	
Traced		Date	
Revised T. Jayo	4-76	Sheet No. 4R of 28	Drawing No. 7-E-23089

SCS-ENG-313-B Rev. 7-64

Structures Assessment Program
Sunset FRS

Figure 6
Upstream Buttress Slope and Cutoff Area

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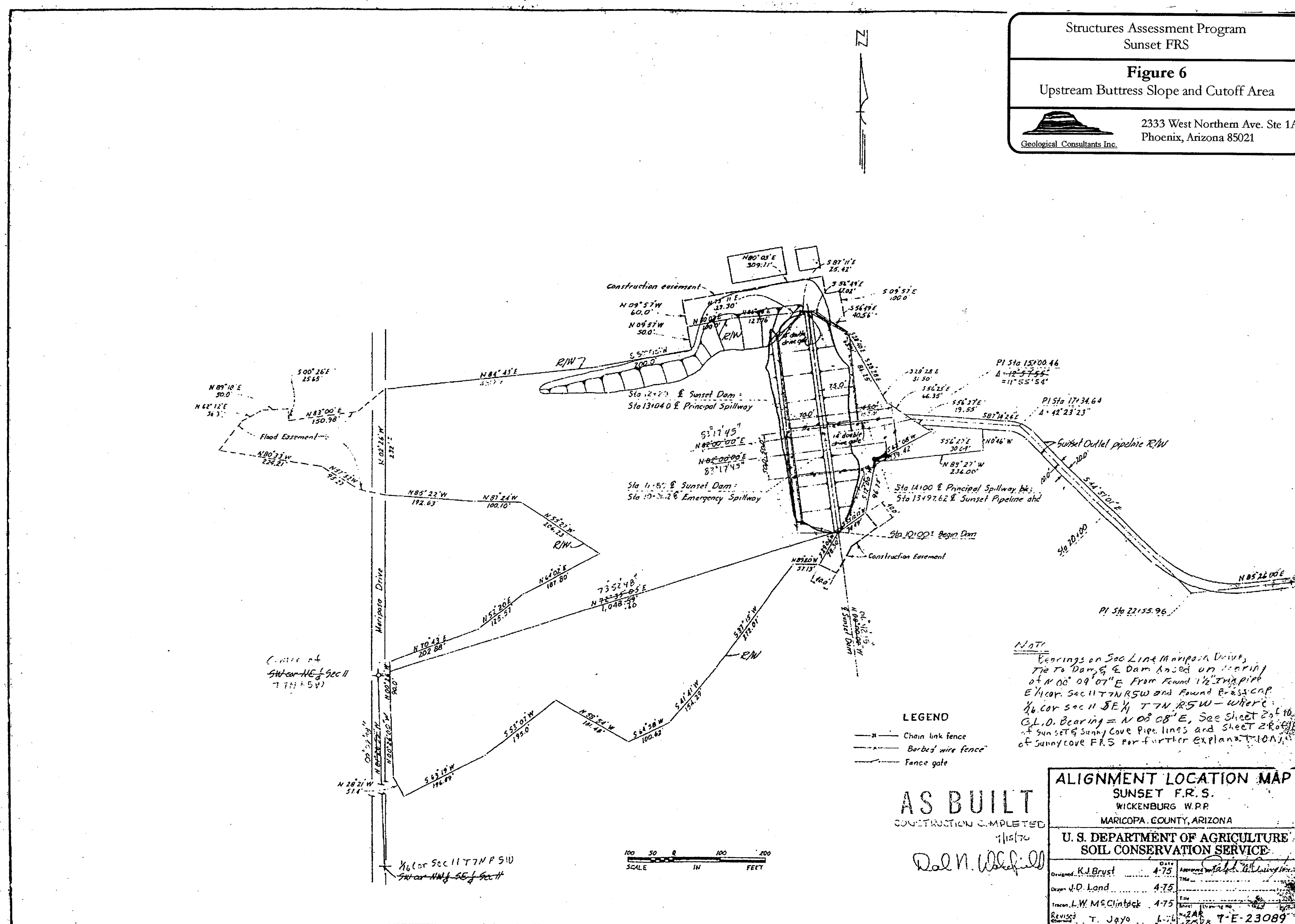
Phoenix, Arizona 85021



GEOTECHNICAL MEMORANDUM
Sunset FRS FMEA – Phase I Structures Assessment

Submitted to:
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7878 N. 16th Street
Phoenix AZ 85020

Submitted by:

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

for RD Silva
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, Sunset Wash Floodwater Retarding Structure (SCS, 1974)
- Supplemental package containing draft text for the Design Report, as well as documentation of the original stability analysis and drain design (SCS, 1975a)
- Design Report, Sunset and Sunnycove Floodwater Retarding Structures, Maricopa and Yavapai County, Arizona (SCS, 1975b)
- Construction Plans and Specifications, Sunset FRS, Wickenberg Watershed Protection and Flood Prevention Project (SCS, 1975c)
- As-built Construction Plans, Sunset FRS (SCS, 1976)
- Sunset Flood Retarding Structure – Arizona Dam No. 7-49, 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 geologic report (SCS, 1974) described the foundation soils at the site as Tertiary alluvial deposits of dense silty, gravelly sand which are cemented to various degrees. Geologists refer to this older, cemented alluvium as fanglomerate. The fanglomerate is overlain by a thin mantle of recent alluvium in the main channel floodplain area. The low density, recent alluvial materials were removed to depths up to 12 feet from the dam foundation during construction. The dam is reportedly founded on competent fanglomerate bedrock in the main channel and right abutment sections.

The mid to upper left abutment consists of variably cemented, stratified alluvial materials and manmade fill and debris. This area was highlighted by the geologic and design reports as presenting a potential seepage problem (SCS 1974, 1975a, 1975b). The natural geologic materials in the left abutment are described as stratified deposits of gravelly, fine to coarse-grained sand, with variable amounts of silt (generally 10 to 20% silt). The left abutment native soils contain zones with little to no cementation and some clean sand lenses. Fill materials found in the left abutment were described as a loose mixture of sand, silt and gravel containing broken concrete blocks and wood debris. The geologic investigation concluded that the fill had been loose dumped into in a gully just upstream from the originally proposed dam alignment. The dam alignment was shifted about 25 feet downstream to avoid the loose fill and debris, and the original plans indicated that the loose fill materials were to be completely removed to "firm residual soil" as directed by the engineer, and backfilled with Zone 1 compacted fill to restore grade to the original ground surface. The extent of over-excavation and replacement of the waste fill was anticipated to be approximately 250 feet in the upstream direction, and "as directed by Engineer" in the downstream direction. However, the extent of removal of the loose fill laterally (northward) into the abutment was constrained by the presence of an existing building at the top

of the slope and it is not known for certain that "firm residual soils" were reached all along the left abutment contact.

1.2 Embankment Materials

The SCS designers concluded that selective borrowing to construct a zoned embankment would be very difficult because of the stratified nature of the alluvial deposits in the borrow areas. Based on that assessment, the embankment was designed as a homogeneous section, with a vertical chimney drain zone. An upstream berm was incorporated in the design to improve stability during rapid drawdown. A typical cross section of the embankment is shown as Figure 1. The embankment Zone I, Zone II drain, and Zone III berm materials generally have the characteristics summarized on Table 1, based on the project specifications.

Table 1. Sunset FRS - Embankment Material Zones

Zone	Description	USCS	Properties	
I	Embankment earth fill –gravelly, clayey sands and gravelly silty sands	SM, SW-SM, SP-SC	10% fines min., 6" max. rock size	
II	Chimney drain – clean, coarse sandy gravel from imported source	GP	Sieve	% Passing
			3-inch	100
			1½-inch	75-100
			¾ - inch	55-85
			⅜ - inch	25-60
			No. 4	0-40
			No.10	0-15
			No. 20	0-5
III	Upstream berm– gravelly sands or sandy gravels from designated borrow	variable	Fines limited to 10% max., 9" max. rock size	

Laboratory testing of representative borrow soils was reported in the geologic report (SCS, 1974). The data were compiled for this Phase I assessment, and are summarized on Table 2. The primary source for borrow materials was the recent alluvial deposits located within the sediment pool and dam foundation. The geologic report described these deposits as inter-layered, inter-fingering, lenticular layers of predominantly silty sands (SM) and slightly silty, well-graded sands (SW-SM), with minor quantities of slightly clayey poorly graded sands (SP-SC). Three borrow areas were identified as follows: Borrow Area I located in the north arm of the sediment pool immediately upstream from the dam, Borrow Area IA located in the north arm of the sediment pool upstream from Area I, and Borrow Area II located in the south arm of the sediment pool.

Table 2. Summary of Representative Laboratory Test Results - Borrow Area Samples

Borrow Area (Samples)	USCS	Gradation (%)			PI (%)	G_s	γ_d (pcf)	w_{opt} (%)	Shear strength	
		Gravel (>#4)	Sand	Fines (-#200)					φ	c
Dam Footprint (2002.1, 2002.2, 2005.1, 2005.2)	SM	17	46	29	NP	2.68				
	SW-SM	3	88	9	NP	2.64				
	SC	23	60	17	8	2.71				
	SP-SC/ SP-SM	8	81	11	5	2.70				
I (2106.1, 2106.2)	SM	16	38	22	NP	2.68	116.8	11.7	36°	0
	SP-SC	33	55	12	15	2.72				
IA (2107.1)	SW-SM	18	74	8	NP	2.70				
II (2102.2)	SW-SM	14	74	12	NP	2.64	105	17.9	34°	0
II-west (2109.1, 2109.2)	GW	61	35	4	8	2.66				
	SW-SM	19	74	7	NP	2.70				
Unknown (ATL samples)*	SM	13	66	21			124	9.7		
	SM	17	62	21			116.5	12.2		
	SM	25	65	10			118	12		

* These samples were tested at Arizona Testing Laboratories (ATL), and the locations (source) of the samples were not indicated on the laboratory data sheets. All other samples were tested at the SCS laboratory in Portland, Oregon.

Borrow Area I was indicated as the primary source for the cutoff trench and central portion of the Zone I fill. Once the SC materials were depleted, the remainder of Zone I was to be derived from Borrow Areas IA and II. A small zone of clean gravel materials was encountered during the site investigation in a specified area in the west portion of Borrow Area II. The draft design memorandum recommended these clean gravel materials be used in the Zone III berm. It was also recommended that other clean sands and gravels, if encountered, be routed to Zone III.

1.3 Original Slope Stability Analyses

Based in part on the laboratory tests as summarized on Table 2 (developed from data sheets attached with the geologic report, SCS, 1974), the designers assumed the parameters shown on Table 3 for the slope stability analyses. Slope stability analysis results were reported for the loading conditions shown on Table 4.

Table 3. Embankment Soil Properties Used in Stability Analysis

Property	Zones I and III
Dry unit weight (γ_d) (pcf)	110
Moist unit weight (γ) (pcf)	118
Saturated unit weight (γ_{sat}) (pcf)	124.5
Angle of internal friction (φ)	34°
Cohesion (c)	0

Table 4. Original Slope Stability Analyses Results

Slope	Conditions	Minimum F.S.
2H:1V downstream	Dry slope – infinite slope analysis	1.35
3H:1V upstream with Zone III berm	Rapid drawdown	1.8
	Steady seepage, full reservoir to emergency spillway crest	1.99

The designers evaluated the upstream slope for a rapid drawdown condition under the assumptions that a full phreatic line could develop up to the emergency spillway elevation, and that no dissipation of pore pressures would occur following drawdown. These assumptions resulted in a factor of safety < 1 for a 3H:1V upstream slope during drawdown. A 12 ft wide, "free-draining" berm was included in the design to achieve adequate factors of safety under the assumed drawdown loading condition.

Downstream slope stability initially was evaluated for an assumed steady seepage condition without the internal drain (phreatic line emerging on the downstream slope). Computed factors of safety were unacceptably low, even for slopes as flat as 3H:1V with this assumed steady seepage condition. Slope stability analysis for the downstream slope under the dry condition (with the drain) was not documented except to evaluate the infinite slope factor of safety = 1.35 for the assumed shear strength of $\varphi = 34°$, $c = 0$.

The design intent was for the upstream berm to be highly pervious and serve as a free-draining, stabilizing buttress to improve stability during drawdown. The Specifications limited fines content in Zone III to 10% to accommodate materials from the available local borrow sources. This fines content may 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 the dam, and used this as primary justification for incorporating the upstream berm and an internal drain zone (Zone II). A more critical purpose for Zone II is actually as a filter to protect against internal erosion and piping. Supplemental geotechnical analyses were performed as part of this Phase I Structures Assessment to 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 conducted preliminary supplemental seepage and slope stability analysis for Sunset FRS to document the expected stability of the structure under anticipated loading conditions. The assumptions used in the original stability analysis by SCS 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 the dam because (1) the embankment was likely to be highly stratified and have high horizontal permeability, (2) maximum release rate through the orifice-controlled low-stage inlet to the outlet works would be slow, and the inlet is vulnerable to plugging, and (3) the sediment pool drain is gate controlled and there could be an uncertain time delay in releasing impounded water from the sediment pool level below the orifice inlet.
  - **Revised Assumption No. 1: Development of a high-level steady state phreatic line is not likely because** (1) the stratified embankment will have high horizontal permeability and will therefore drain quickly, and (2) the maximum detention time for a 100-year event will be less than 10 days[1], assuming the outlet does not clog. In our estimation, this is insufficient time for a high-level steady state seepage line to develop. (3) Failure to release the sediment pool through manual operation of the gated outlet is a possibility. Gannett Fleming conducted supplemental slope stability analyses assuming that water remains in the sediment pool for extended time periods because the low level gate remains closed.

- **Assumption No. 2: Zone III is free-draining** - Rapid drawdown stability was achieved for the steady-state phreatic line pore pressure assumption by incorporating a "free-draining" Zone III berm.
  - **Revised Assumption No. 2: Zone III is not free-draining.** The Zone III materials, contain up to 10% 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, and pore pressure dissipation will occur during normal drawdown rates (see following seepage analysis), rapid drawdown is not anticipated to cause slope instability, even if the berm is not essentially free draining.

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.

[1] The release rate for the orifice inlet is estimated to be between 5 and 7 cfs. At an average discharge rate of 6 cfs, and an estimated volume of 47 ac-ft between the emergency spillway elevation (2131 ft) to the low-stage outlet elevation (2120 ft), Gannett Fleming estimates the total time to drain to the low-stage orifice is about 4 days.

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 temporary impoundment event. A standard "Silty Sand" material type was selected from the model's database to represent the Sunset FRS embankment materials. The database provides the necessary unsaturated hydraulic parameters for use in the simulation. The vertical saturated hydraulic conductivity was assumed to be 0.3 ft/day (1 X 10^{-4} cm/s). This was the value used by the SCS designers for the filter/drain design, based on laboratory tests on representative sample 2102.2, as reported in the draft design documents (SCS, 1975a). The embankment was modeled as a homogeneous section, 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). It was assumed that the low level outlet would remain closed, therefore the drawdown between events and following the two floods was limited to elevation 2120, at the top of the berm. This impoundment scenario was modeled to estimate a conservative phreatic line for use in evaluating slope stability during drawdown. It is evident that even following multiple storm events, the wetting front will advance to a very limited extent into the dam. Also, the model results indicate rapid dissipation of the upstream pore pressures as the pool level drops.

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 above the berm at two times: (1) during drawdown after the 2nd flood impoundment (factor of safety = 2.4), and (2) after drawdown to the sediment pool level immediately following two consecutive impoundment events (factor of safety = 2.3). The factor of safety is slightly higher at the intermediate impoundment stage (during drawdown) because the pool provides additional buttressing against the slope. Note that the slope is predicted to be completely drained (low phreatic line) immediately following the two events, based on the assumptions used in the model for impoundment times, drawdown times, and hydraulic conductivity of the materials.

Slope stability for the downstream slope was also re-evaluated in order to document a factor of safety for a more meaningful failure mode than the shallow, infinite slope analysis that was completed during original design. The factor of safety shown (1.7) 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 in construction.

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	2.4
	4	Immediately following two, 100-yr impoundment events	2.3
2H:1V downstream	5	Dry slope – critical failure surface	1.7

* 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 positioned downstream from the dam crest. 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 in the event of transverse crack development.

Because of its critical function as a filter, the Zone II gradation was checked against current filter criteria in accordance with the NRCS, National Engineering Handbook, Chapter 26 "Gradation Design of Sand and Gravel Filters" (NRCS, 1994). Figure 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 was developed for Field Sample 2106.1 data from the geologic report (SCS, 1974). The sample was taken from Borrow Area #1, and is described as a "light brown, calcareous, gravelly silty sand, that classifies as SM according to the Unified Soil Classification system (USCS). A second base soil gradation curve is also shown (graphed with solid blue circles) to represent a more average gradation of the Zone 1. The "average" base soil gradation (blue circles) was developed from sample 2102.2, derived from Borrow Area #2, which is described as "reddish brown, calcareous, well graded silty, gravelly sand".

The base soil gradation curves (solid symbols) were adjusted for gravel content as shown by the curves graphed with open red triangular symbols (for sample 2106.1) and open blue circles (sample 2102.2). The filtering and permeability (k) criteria for the adjusted curves are shown by the solid circles and trianges on the 15% passing line. The coarse side of the Zone II specification band is too coarse to achieve the recommended filtering limit for the finest base soils. However, Zone II does meet both filtration and permeability criteria for the "average" base soil gradation. Thus it is possible that some fines from Zone I could penetrate into Zone II under a concentrated leak through a transverse crack, if the Zone II materials were graded on the coarse band in accordance with the specified gradation limits. Considering the variability in gradation of the Zone I materials, and the fact that Zone II is meets the criteria except for the finest base soil and coarsest filter possibilities, it is likely that Zone II is providing adequate filter protection. Additional analyses may be done to further evaluate the efficacy of the Zone II filter, as outlined under Recommendations.

3.0 RECOMMENDATIONS FOR FUTURE ACTIONS

3.1 Monitoring

In recognition of the presence of potentially weak, loose fill, and uncemented native materials in the upper left abutment, we recommend monitoring for seepage through the left abutment during impoundment events, and regular visual inspection of the clay blanket that extends approximately 400 feet upstream from the left abutment to ensure it is being properly maintained.

3.2 Supplemental Geotechnical Analyses

3.2.1 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 Sunset 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 Comparison

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 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)	1.8	2.1	(1)
Steady seepage w/o seismic forces, phreatic surface fully developed w/reservoir at principal spillway elevation (downstream slope)	1.35	1.7	(2)
Steady seepage w/ phreatic surface developed from critical partial pool elevation (upstream slope)	1.99 (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 temporary impoundment events, and to assess the upstream slope stability under normal drawdown rates (with the lower intake functioning at capacity). These analyses show that it is very unlikely that a steady state phreatic line would develop in the Sunset 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. In the original analysis, rapid drawdown was evaluated assuming a fully developed phreatic line from the normal high reservoir pool elevation due to a clogged outlet, followed by instantaneous drawdown (magical instant removal of the reservoir). In the original design analysis, the upstream Zone III berm was assumed to be "free-draining" providing a high strength, fully-drained buttress under the rapid drawdown loading. It is likely that the upstream berm is not free-draining, and a more realistic, but still conservative rapid drawdown analysis would involve the following steps:

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. Realistic hydraulic conductivities can be used for Zones I and III, rather than assuming Zone I is impervious and Zone III completely drained as was done in the original drawdown analysis. 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 transient 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 original minimum factor of safety that was computed for the dry downstream slope (1.35) does not achieve the minimum criteria of 1.5 (see Table 6). However, 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 1.7. No additional analyses for the downstream slope are considered necessary.

(3) **Upstream slope stability under steady seepage, partial pool**: The original analysis evaluated upstream slope stability under steady seepage for the maximum pool elevation, resulting in a minimum factor of safety of 1.99. 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 that result in a lower factor of safety 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 Cassandro 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.

3.2.2 Phase II Slope Stability and Seepage Analysis of Left Abutment

Additional seepage and slope stability analysis could be performed to evaluate the candidate failure mode associated with potential slope instability in the left abutment. The analysis is outlined as follows:

1) Develop geologic cross sections through the abutment, beyond the extent of the Zone I/Zone II fill. Sections should be drawn for both the upstream and downstream abutment slopes. Use boring log and laboratory test data from the geologic report (SCS, 1974), and the as-built construction plans to estimate the subsurface slope stratigraphy and geometry as accurately as possible. Assign material parameters (hydraulic conductivity and shear strength) for the various layers in the slope using available information and judgement.
2) Conduct seepage analysis to estimate the range or extent of saturation and seepage through the abutment during impoundment events.
   a) CASE 1 – Establish a "worst case" seepage line through the abutment. A conservative analysis could assume that the outlet intake is clogged, allowing a steady-state seepage line to develop around the left end of the dam. Also, it could be assumed that the clay blanketing layer is very thin, or has been damaged, allowing reservoir seepage to quickly penetrate into coarse, loose layers of debris fill and highly stratified alluvial deposits in the abutment. Further, it could be assumed that the slope has been "pre-saturated" by vertical infiltration from the building and parking area at the top of the slope. In this worst case scenario, it is likely that seepage would emerge on the downstream left groin of the dam. The critical slope analysis for CASE 1 would be the downstream abutment slope.
   b) CASE 2 – Estimate a more likely extent of saturation and pore pressure development in the abutment slope by running a seepage analysis with the clay blanketing intact, and assuming limited detention time in the reservoir following an impoundment event or events. In this case the seepage line would likely only partially penetrate the upstream slope and may not emerge on the downstream slope. The critical slope stability analysis for this case would be the upstream abutment slope under drawdown conditions.
3) Conduct slope stability analysis for CASE 1 and CASE 2 pore pressure conditions and report the factors of safety for the worst case and expected case conditions.
4) Evaluate whether the factors of safety against this failure mode are acceptable, and, if not, recommend remedial actions that could be taken to improve the conditions in the left abutment of the dam.

3.2.3 Phase II Additional Evaluation of Zone II Filter/Drain

The Phase I evaluation of the Zone II as a protective filter is described in Section 2.2 of this memo. These preliminary analyses indicate that the Zone II filter/drain does not strictly meet filtering requirements for the finest materials that may have been used in the Zone I fill. Additional analyses could be done to evaluate the effectiveness of the Zone II using a

methodology developed from recent research at the University of New South Wales, Australia (Foster and Fell, 2001). The procedure outlined by Foster and Fell (2001) is a method for assessing the gradation of filters in dams that may not meet the criteria in the strict sense, but could provide a degree of protection. The methodology can be used to determine whether filters that are too coarse according to modern criteria are sufficiently fine to eventually seal, or are anticipated to allow continuous erosion that could result in piping failure.

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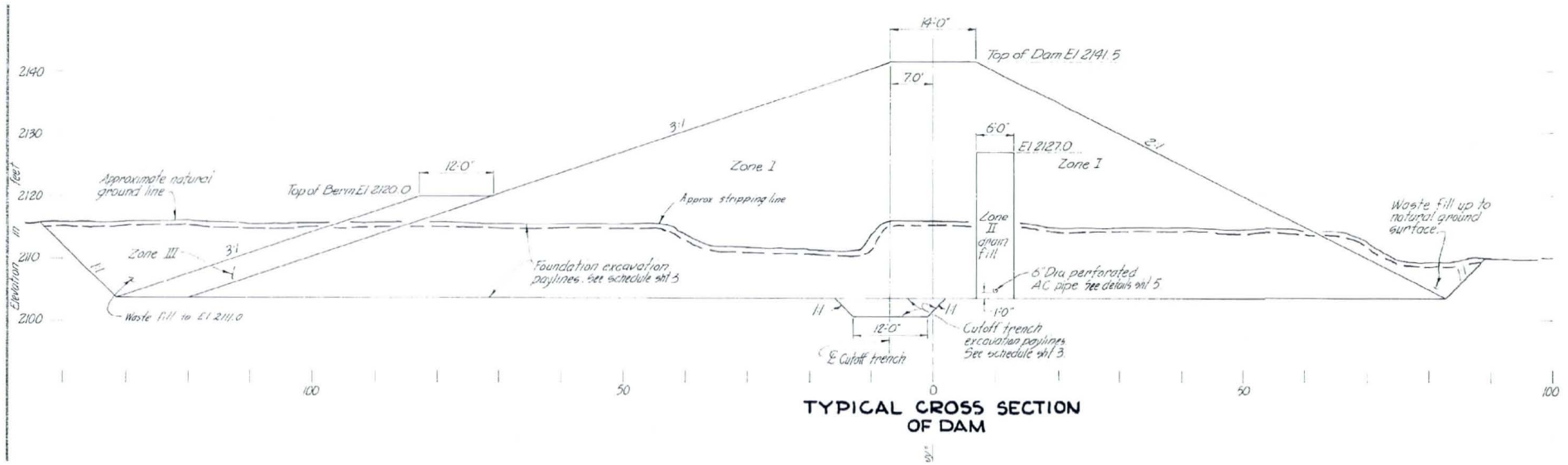
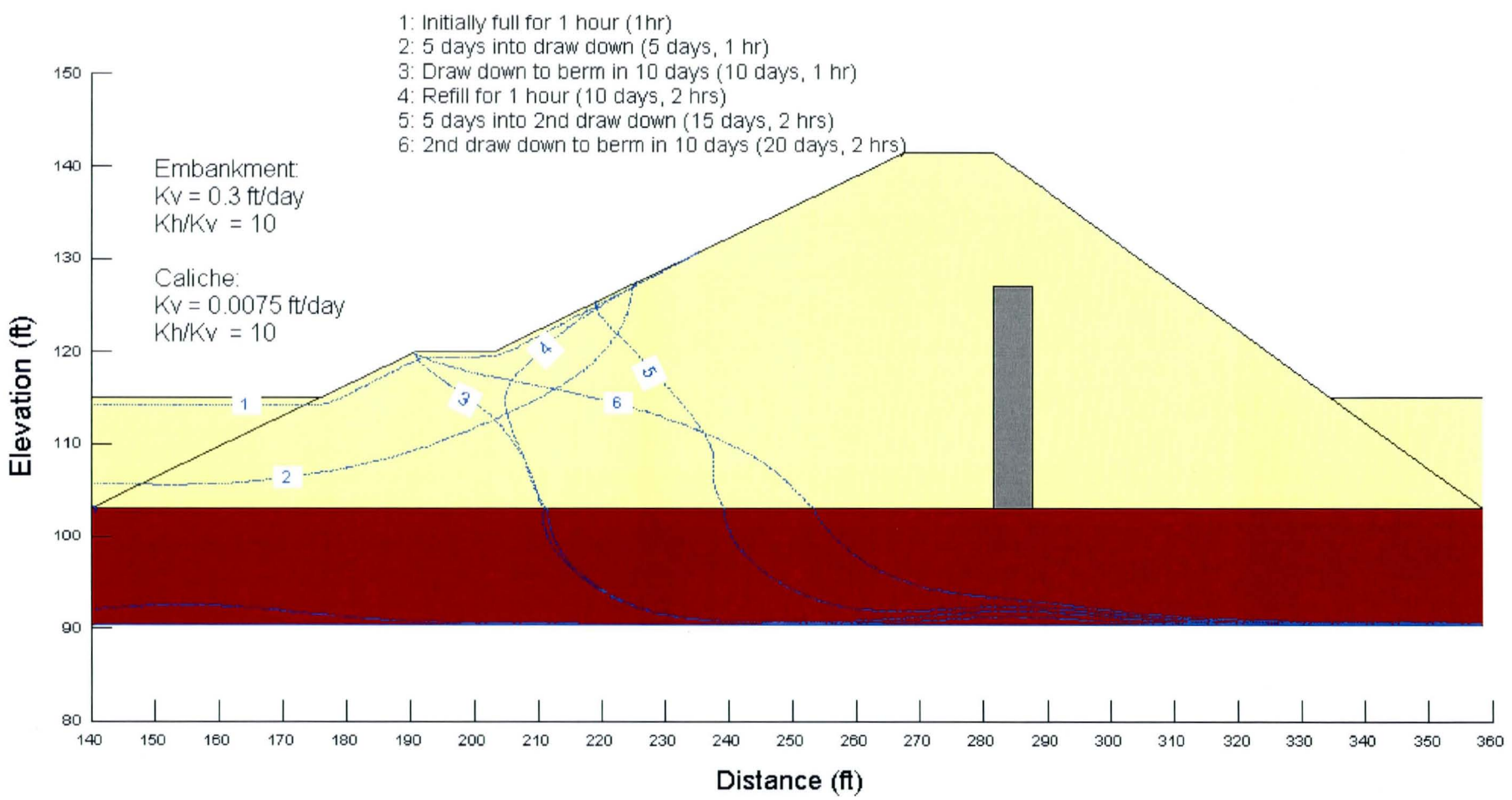


TYPICAL CROSS SECTION
OF DAM

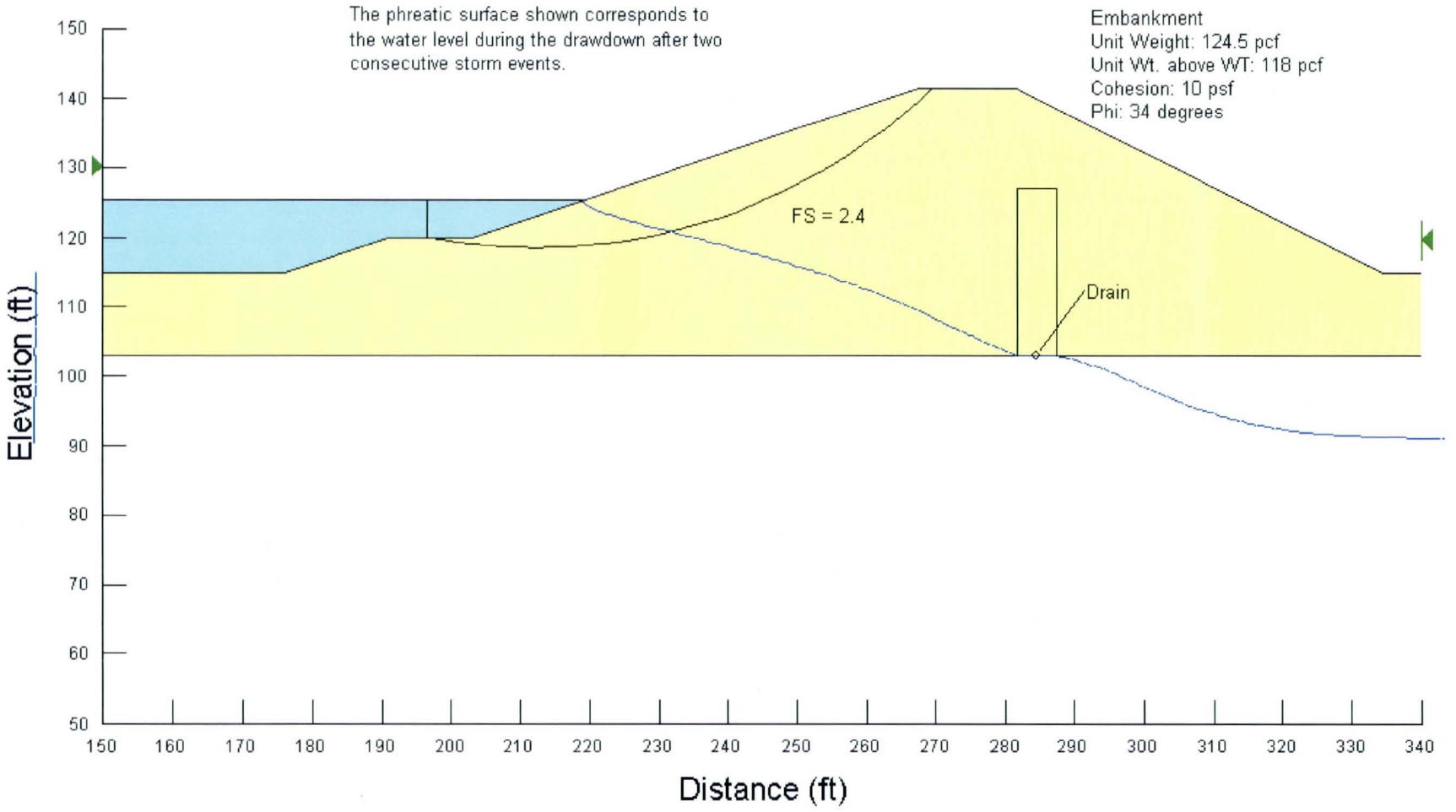
Gannett Fleming	TYPICAL CROSS SECTION SUNSET FLOOD RETARDING STRUCTURE FLOOD CONTROL DISTRICT OF MARICOPA COUNTY	FIGURE 1



Gannett Fleming

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

FIGURE
2



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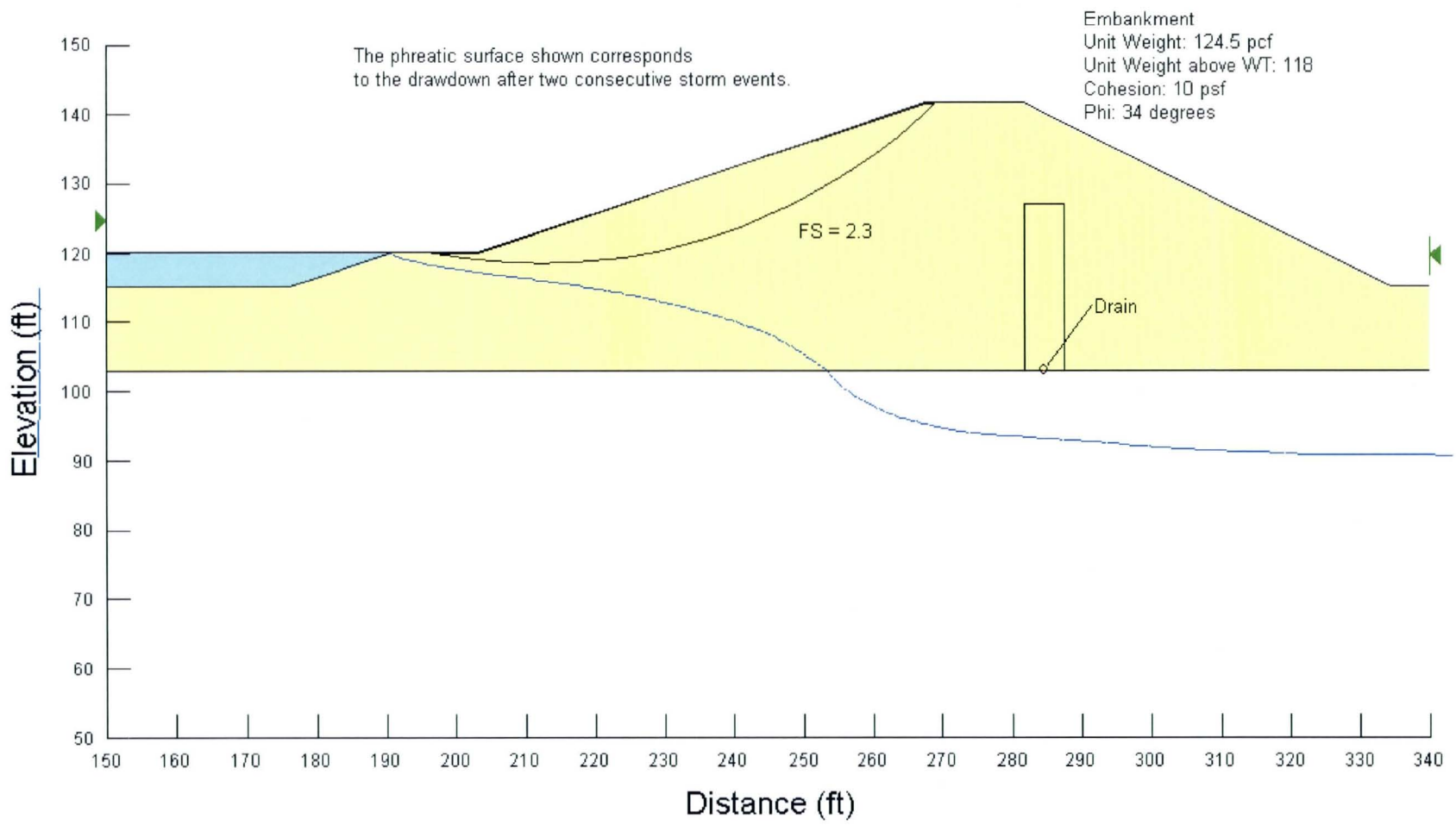
UPSTREAM STABILITY DURING DRAWDOWN AFTER TWO CONSECUTIVE EVENTS
SUNSET FLOOD RETARDING STRUCTURE
FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

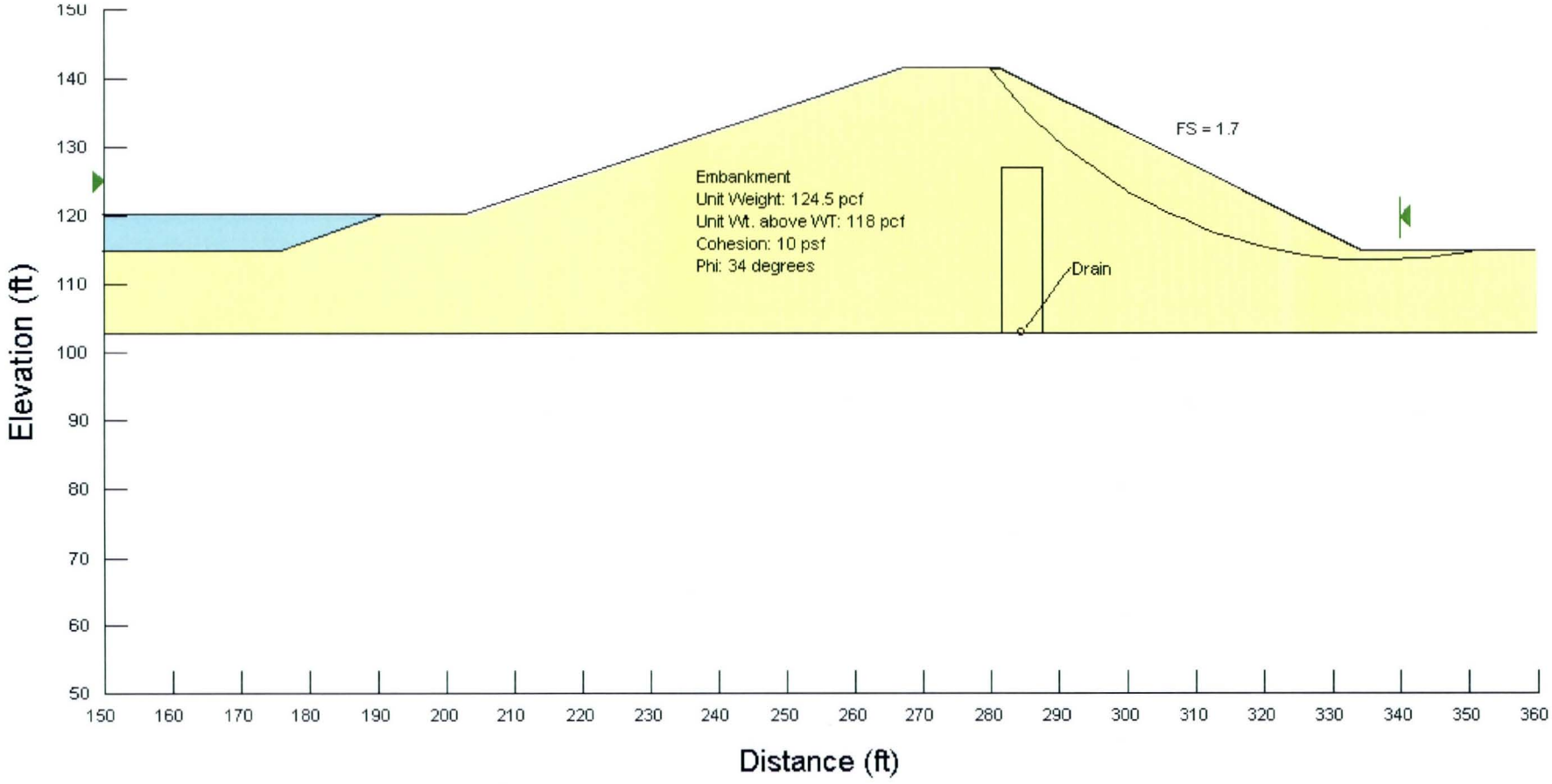
FIGURE 3



Gannett Fleming

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

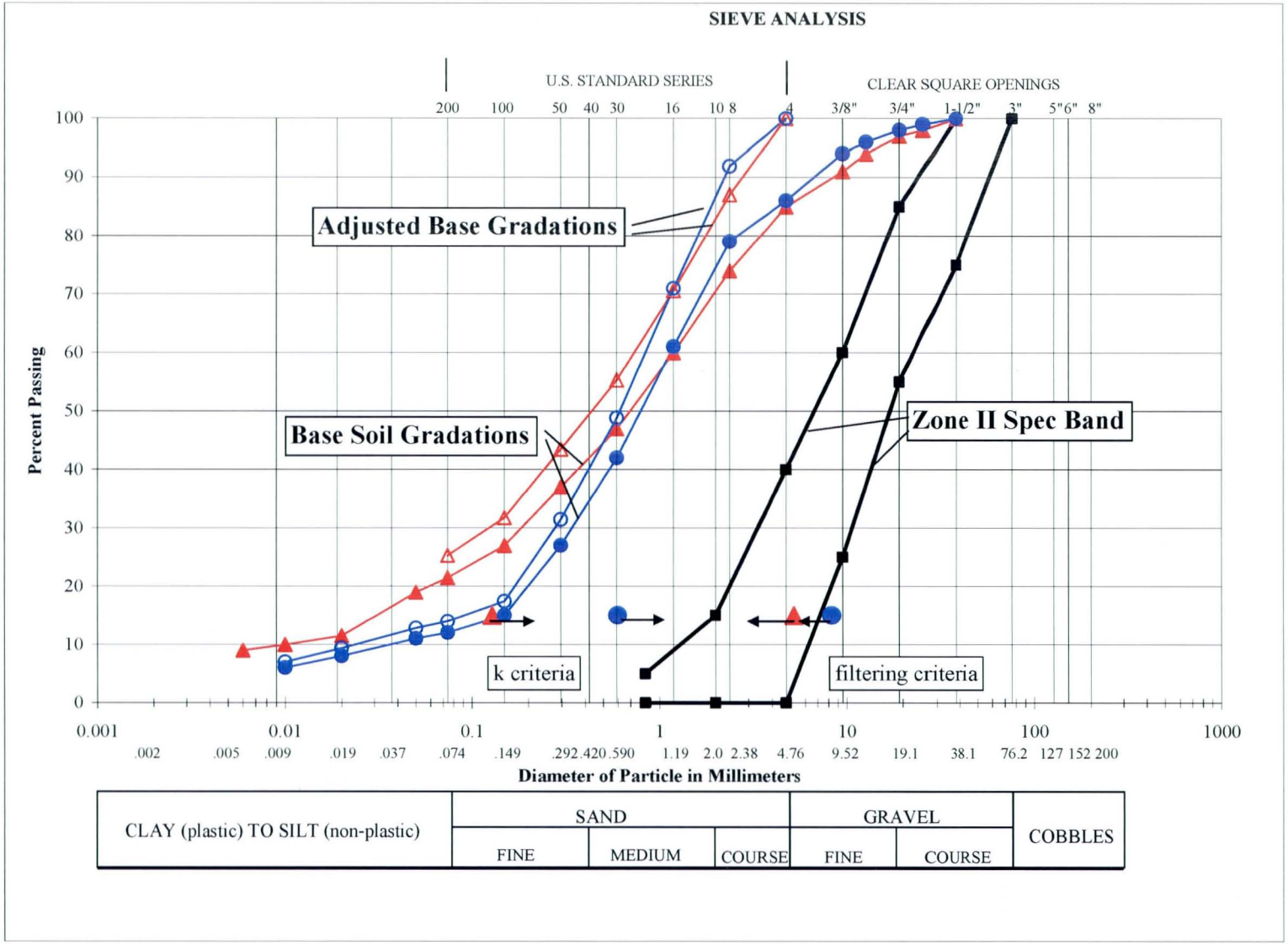
FIGURE
4



Gannett Fleming

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

FIGURE
5



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

FIGURE 6