AECOM

CHOLLA POWER PLANT FLY ASH POND – Periodic Structural Integrity Assessment

Periodic Hazard Potential Classification
Periodic Structural Stability Assessment
Periodic Safety Factor Assessment

October 2021
AECOM Project 60664605

Delivering a better world

Prepared for:

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Attachment

1. Introduction

This periodic update to the Structural Integrity Assessment for the Fly Ash Pond (FAP) at Cholla Power Plant operated by Arizona Public Service (APS) has been prepared in accordance with the requirements of Title 40 of the Code of Federal Regulations Part 257 (40 CFR 257) ("the Coal Combustion Residuals [CCR] Rule" or "the Rule") and the specific requirements within 40 CFR § 257.73 for periodic (every 5 years) assessment regarding structural integrity.

2. Methodology

The methodology used to prepare this 2021 Periodic Assessment of Hazard Potential Classification, Structural Stability Assessment, and Periodic Safety Factor Assessment for the FAP at the Cholla Power Plant is for the certifying Qualified Professional Engineer (QPE) to:

a. Perform a documented review of the 5 years of annual inspection reports since 2016, the most recent of which is:
    i. APS, 2020. Annual CCR Impoundment and Landfill Inspection Report: Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, Sedimentation Pond, and Bottom Ash Monofill. Generation Engineering, Phoenix, AZ.
b. Perform a documented review of each major component of the contributing technical information from:
    i. AECOM, 2016. *Final Summary Report, Structural Integrity Assessment: Fly Ash Pond, Cholla Power Plant, Joseph City, Arizona*. Prepared for: Arizona Public Service, AECOM Job No. 60445840, August 26, 2016 (hereafter referred to as the "2016 Report" and incorporated and referenced directly as Attachment A to this document); and
c. Consider and document whether the 2016 Report and its conclusions:
    i. Meet the current reporting requirements of the Rule;
    ii. Reflect the current condition of the structure, as known to the QPE and documented in the annual inspections;
    iii. Are compromised by any identified issues of concern; and
    iv. Are consistent with the standard of care of professionals performing similar evaluations in this region of the country; and
d. Identify any additional analyses, investigations, inspections, and/or repairs that should be completed in order to complete this 2021 Periodic Assessment.

This report documents the results of these considerations, incorporates the 2016 Report as an Appendix, identifies any additional technical investigation or evaluations (if needed), and presents an updated certification by the QPE.

3. 2017–2021 Annual Inspection Reports

Information relevant to the general site conditions and current adequacy and performance of the FAP embankment and outlet works have been considered. No issues were identified during the review that would affect the performance of the system and its compliance, as described in the 2016 Report, with the various requirements of the CCR Rule relative to (1) hazard potential classification, (2) structural stability, or (3) safety factor assessment.

The number of entries to the annual list of "Observed Conditions," over the last 5 years of reports, has remained roughly consistent. The most consistently observed conditions involve: (1) animal burrows minor erosion holes in the crest; (2) excess vegetation on upstream and downstream slope faces; (3) riprap deterioration; and (4) minor slope erosion issues, somewhat consistently at the groin of the right (west or north) abutment near Geronimo sump. The action item recommendation for the majority of these conditions has been for regular Plant operations and maintenance remedial actions which, generally, have been completed.

The 2017-2021 Annual Inspection Reports also provide information on minimum and maximum values for various types of geotechnical instrumentation installed within the embankments and foundations. Periodically, deviations or technical issues may be identified that limit or alter readings and these instances are reported in the Annual Inspection Reports. For the FAP, the instruments consist of standpipe piezometers, surface settlement monuments, and seepage flow totalizers. The following trends were noted in review of the five years of reports:

a. The record of standpipe piezometer levels have shown no changes of significance over the five-year reporting period, with the following exception:

    i. Several standpipe piezometer levels have not fallen as quickly as the FAP impounded reservoir level has fallen:

        1. Since 2017, the impounded reservoir level is *lower* than piezometric levels in F-110, screened in the alluvium underlying the dam (ADWR Basic Data Report Figure 3), and F-128, screened in the dam core (ADWR Basic Data Report Figure 9).

        2. Other instruments screened in the same zones as F-110 and F-128, such as F-124 and F-132, both screened in the core, are not yet higher than the impounded reservoir level but are not falling as quickly as the reservoir level.

    ii. The issue being monitored is whether piezometric levels within the shell, core, and foundation of the embankment will equilibrate fast enough to avoid a "rapid-drawdown"-type upstream slope instability. The mitigating factor against upstream slope instability is the buttressing effect of the impounded CCR solids. This condition will be monitored closely during the final year of pond operation as enhanced water level reduction measures are introduced and the reservoir level reduction accelerates.

b. The record of settlement monument movements have shown no changes of significance over the five-year reporting period.

c. The record of seepage monitoring location turbidity readings have shown no long-term changes over the five-year reporting period.

d. The record of seepage monitoring location flow totalizer readings are interpreted to have shown no changes of significance over the five-year reporting period. One deviation attributed to equipment issues is:

    i. The Geronimo totalizer occasionally records negative flowrates. Plant staff report this is because smaller volumes of water pumped upwards to the pond don't always make it up the embankment and then flow back, causing the totalizer to run backwards.

The annual inspection reports, including instrumentation records, were reviewed and no significant, adverse trends were identified that would cause structural instability or change in safety factor.

4. 2016 Certification – Review by Section

Other than as described in the remainder of this section, the details presented in this section of the 2016 Report adequately represent current conditions and satisfy the requirements of the Rule.

4.1 "1.4 Facility Description"

The details presented in this section of the 2016 Report adequately represent current conditions and satisfy the requirements of the Rule.

4.2 "2 Hazard Potential Classification"

The details presented in this section of the 2016 Report adequately represent current conditions and satisfy the requirements of the Rule.

Based on a review of the information presented in the 2016 Report, the LAI impoundment currently satisfies the criteria for High Hazard Potential classification.

4.3 "3 History of Construction"

The details presented in this section of the 2016 Report adequately represent current conditions and satisfy the requirements of the Rule.

4.4 "4 Structural Stability Assessment"

The details presented in this section of the 2016 Report adequately represent current conditions and satisfy the requirements of the Rule.

AECOM assesses that the design, construction, operation, and maintenance of the FAP are consistent with recognized and generally accepted good engineering practice for the maximum volume of CCR and CCR wastewater that can be impounded therein.

4.5 "5 Safety Factor Assessment"

The details presented in this section of the 2016 Report adequately represent current conditions and satisfy the requirements of the Rule.

AECOM is not aware of any new information that would warrant re-evaluation of any material properties or cross-section configurations of the perimeter embankment. Relative to piezometric conditions, the potential for excess pore water pressure during reservoir drawdown is being tracked and may trigger a future evaluation of piezometric conditions within the perimeter embankment.

The calculated factors of safety for the three critical cross sections along the FAP perimeter embankment exceeded the required minimum values for the long-term, maximum storage pool; the maximum surcharge pool; and the seismic (pseudo-static) loading conditions.

4.6 "6 Conclusions"

The details presented in this section of the 2016 Report adequately represent current conditions and satisfy the requirements of the Rule.

5. Recommended Additional Technical Investigations or Evaluations

AECOM recommends that APS continue to track pore water pressure conditions within the core and shell of the embankment as the reservoir level is drawn down, recognizing the buttressing effect of the impounded CCR solids.

No other measures are identified nor recommended.

6. Conclusion

The 2016 Report and its conclusions meet the current reporting requirements of the Rule, reflect the current condition of the structure as known to the QPE and documented in the annual inspections, are not compromised by any identified issues of concern, and are consistent with the standard of care of professionals performing similar evaluations in this region of the country.

7. Limitations

This report is for the sole use of APS on this project only and is not to be used for other projects. In the event that conclusions based upon the data presented in this report are made by others, such conclusions are the responsibility of others.

The Periodic Structural Integrity Assessment presented in this report is based on the 2016 Report and relies and incorporates any Limitations expressed in that report.

The Certification of Professional Opinion in this report is limited to the information available to AECOM at the time this Assessment was performed in accordance with current practice and the standard of care. Standard of care is defined as the ordinary diligence exercised by fellow practitioners in this area performing the same services under similar circumstances during the same period. Professional judgments presented herein are primarily based on information from previous reports that have been assumed to be accurate, knowledge of the site, and partly on our general experience with dam safety evaluations performed on other dams.

No warranty or guarantee, either written or implied, is applicable to this work. The use of the word “certification” and/or “certify” in this document shall be interpreted and construed as a Statement of Professional Opinion and is not and shall not be interpreted or construed as a guarantee, warranty, or legal opinion.

8. Certification Statement

Certification Statement for:

- 40 CFR § 257.73(a)(2)(ii) – Periodic Hazard Potential Classification for an Existing CCR Surface Impoundment
- 40 CFR § 257.73(d)(3) – Periodic Structural Stability Assessment for an Existing CCR Surface Impoundment
- 40 CFR § 257.73(e)(2) – Periodic Safety Factor Assessment for an Existing CCR Surface Impoundment

I, Alexander W. Gourlay, being a Registered Professional Engineer in good standing in the State of Arizona, do hereby certify, to the best of my knowledge, information, and belief, that the information contained in this certification has been prepared in accordance with the accepted practice of engineering. I certify, for the above-referenced CCR Unit, that the periodic hazard potential classification, periodic structural stability assessment, and periodic safety factor assessment provided in this Periodic Structural Integrity Assessment Report, and referencing the 2016 Report, were conducted in accordance with the requirements of 40 CFR § 257.73.

Alexander W. Gourlay, P.E.
Printed Name

October 11, 2021
Date

Registered Professional Engineer (Civil)
CERTIFICATE NO.
22867
ALEXANDER W.
GOURLAY
Date Signed 10/11/2021
ARIZONA, USA

Attachment A:

AECOM, 2016. *Final Summary Report, Structural Integrity Assessment: Fly Ash Pond, Cholla Power Plant, Joseph City, Arizona*. Prepared for: Arizona Public Service, AECOM Job No. 60445840, August 26, 2016

ATTACHMENT A

AECOM, 2016. *Final Summary Report, Structural Integrity Assessment: Fly Ash Pond, Cholla Power Plant, Joseph City, Arizona*. Prepared for: Arizona Public Service, AECOM Job No. 60445840, August 26, 2016

AECOM

Submitted to
Arizona Public Service
Generation Engineering
P.O. Box 53999
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Submitted by
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August 26, 2016

Final Summary Report
Structural Integrity Assessment

Fly Ash Pond
Cholla Power Plant
Joseph City, Arizona

Prepared for:
Arizona Public Service

AECOM Job No. 60445840
August 2016

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List of Acronyms

ADWR	Arizona Department of Water Resources
APS	Arizona Public Service
CCR	Coal Combustion Residual
CFR	Code of Federal Regulations
EAP	Emergency Action Plan
EPA	Environmental Protection Agency
ft	feet
HPC	Hazard Potential Classification
pcf	pounds per cubic foot
PMF	Probable Maximum Flood
USCS	Unified Soil Classification System
USGS	United States Geological Survey

Certification Statement

Certification Statement for:

- **40 CFR § 257.73(a)(2)(ii) – Initial Hazard Potential Classification for an Existing CCR Surface Impoundment**
- **40 CFR § 257.73(d)(3) – Initial Structural Stability Assessment for an Existing CCR Surface Impoundment**
- **40 CFR § 257.73(e)(2) – Initial Safety Factor Assessment for an Existing CCR Surface Impoundment**

CCR Unit: Arizona Public Service Company; Cholla Power Plant; Fly Ash Pond

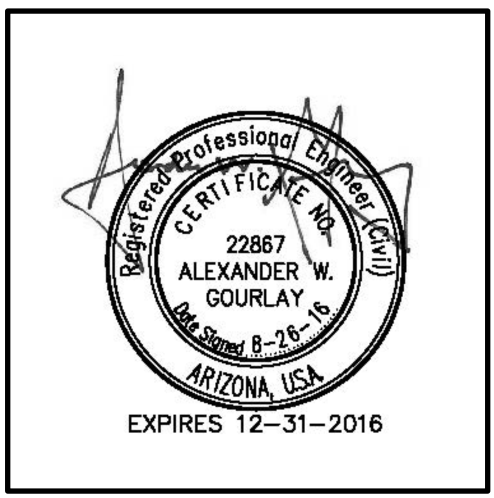
I, Alexander Gourlay, being a Registered Professional Engineer in good standing in the State of New Mexico, do hereby certify, to the best of my knowledge, information, and belief, that the information contained in this certification has been prepared in accordance with the accepted practice of engineering. I certify, for the above-referenced CCR Unit, that the initial hazard potential classification, initial structural stability assessment, and initial safety factor assessment as included in the Structural Integrity Assessment Report dated August 26, 2016 was conducted in accordance with the requirements of 40 CFR § 257.73.

Alexander W. Gourlay, P.E.

Printed Name

August 26, 2016

Date



1 Introduction

Arizona Public Service Company (APS) contracted URS Corporation, a wholly owned subsidiary of AECOM, to assist in the initial structural integrity assessment of the existing coal combustion residual (CCR) surface impoundments at the Cholla Power Plant in Joseph City, Arizona. Figure 1-1 shows the location of the CCR Impoundments at the Cholla Power Plant. This Summary Report documents the AECOM structural integrity assessment for the Fly Ash Pond, Arizona Department of Water Resources (ADWR) Dam No. 09.28. Assessments of other CCR Impoundments at the Cholla Power Plant are presented in separate reports.

1.1 Report Purpose and Description

The purpose of this report is to document the initial structural integrity assessment for the Fly Ash Pond located at the Cholla Power Plant. The Fly Ash Pond is an existing CCR surface impoundment owned and operated by APS that is regulated by the Arizona Department of Water Resources (ADWR). In 2015, the United States Environmental Protection Agency (EPA) finalized Federal Rule (Rule) 40 Code of Federal Regulations (CFR) *§* 257.73 (EPA, 2015) regulating CCRs under Subtitle D of the Resource Conservation and Recovery Act. As part of this Rule, owners and operators of existing CCR surface impoundments must complete initial and periodic structural integrity assessments to document whether the CCR unit poses a reasonable probability of adverse effects on health and the environment.

1.2 EPA Regulatory Requirements

Pursuant to Rule 40 CFR *§* 257.73 (EPA, 2015), each existing CCR surface impoundment must have initial and periodic structural integrity assessments to evaluate whether the CCR unit poses a reasonable probability of adverse effects on health and the environment. The assessment must address the following elements:

- *Periodic Hazard Potential Classification Assessment (40 CFR § 257.73(a)(2))* - Document the hazard potential classification of each CCR unit as either a high hazard, significant hazard, or low hazard potential CCR unit.

- *Emergency Action Plan (EAP) (40 CFR § 257.73(a)(3))* - Prepare and maintain a written EAP for high and significant hazard CCR units. The EAP must be evaluated at least every five years and, if necessary, updated and revised to maintain accurate information of current CCR unit conditions. The evaluation and certification of the EAP is provided in a separate report.

In addition, the following elements must be addressed for CCR units, such as the Fly Ash Pond, that have a height of five feet (ft) or more and a storage volume of 20 acre-ft or more, or have a height of 20 ft or more:

- *History of Construction (40 CFR § 257.73(c)(1))* - Compile a history of construction of the CCR unit including elements of operation, location, design, monitoring instrumentation, maintenance and repair, and historic structural instabilities.

- *Periodic Structural Stability Assessment (40 CFR § 257.73(d))* - Document whether the design, construction, operation and maintenance of the CCR unit is consistent with recognized and generally accepted good engineering practice for the maximum volume of CCR and CCR wastewater which can be impounded therein.

- *Periodic Safety Factor Assessment (40 CFR § 257.73(e))* - Document whether the calculated factors of safety for each CCR unit achieve minimum safety factors for the critical cross section of the embankment under long-term, maximum storage pool loading conditions, maximum surcharge loading conditions, seismic loading conditions, and post-earthquake loading conditions for dikes constructed of soils susceptible to liquefaction.

Existing CCR surface impoundments, such as the Fly Ash Pond, are required to have an initial structural integrity assessment within 18 months of publication of the EPA Rule on April 17, 2015 and subsequent periodic assessments performed every five years thereafter.

1.3 Report Organization

This Summary Report has been organized into the following sections:

Report Section	Applicable CFR 40 Part 257 Citation
• Section 1 – Introduction	
• Section 2 – Hazard Potential Classification	§ 257.73(a)(2) Periodic hazard classification assessments
• Section 3 – History of Construction	§ 257.73(c)(1) History of construction
• Section 4 – Structural Stability Assessment	§ 257.73(d) Periodic structural stability assessment
• Section 5 – Safety Factor Assessment	§ 257.73(e) Periodic safety factor assessment
• Section 6 – Conclusions	
• Section 7 – Limitations	
• Section 8 – References	
• Figures	
• Appendix A – Historic Drawings	
• Appendix B – Safety Factor Calculation	

1.4 Facility Description

The Cholla Power Plant is an electric generating station located in the town of Joseph City, Navajo County, Arizona. The station consists of four coal-fired units. Units 1, 2 (decommissioned), and 3 are owned by APS and Unit 4 is owned by PacifiCorp. CCR generated at the power plant are disposed of at two major surface impoundments located off-site; the Fly Ash Pond located about one-and-a-half miles east of the plant and the Bottom Ash Pond located about two miles north of the plant. Figure 1-1 shows the location of the Fly Ash Pond and Bottom Ash Pond in relation to the power plant. This assessment evaluates the structural integrity of the Fly Ash Pond.

The Fly Ash Pond receives discharges from the following sources: Slurry Disposal; General Water Sump; Fly Ash Pond Seepage Collection System; Sedimentation Pond Solids; Unit 3 and Unit 4 Cooling Tower(s) Basin Solids; General Water Sump Solids; Unit 1, 2, 3, and 4 Oil Water Separator Solids; WARP Solids; CCR Wastes; Flue Gas Desulfurization Wastes; and Fly Ash Pond Area Stormwater. The CCR and other wastes are pumped as slurry through three 6-inch diameter pipes into the impoundment where the solids settle out and the remaining water evaporates. There is no means to return the excess water to the plant for reuse.

The Fly Ash Pond has a total surface area of about 420 acres and storage capacity of about 16,500 acre-feet when at its permitted maximum storage pool water level of EL 5,114 ft (ADWR, 1986). The impoundment is surrounded on its west, north, and east sides by natural topography consisting of rock outcrops of mudstones, siltstones, and sandstones. On the south side, the impoundment is enclosed by the Fly Ash Pond Dam, ADWR Dam No. 09.28, which spans the width of a natural wash. The Fly Ash Pond has been classified under ADWR regulations as a high hazard impoundment due to the probable loss of human life at the nearby U.S. Interstate 40 (I-40), Cholla Power Plant, freight railroad line, and downstream residences, in the event of a dam breach.

The Fly Ash Pond Dam is an earthen, zoned embankment dam consisting of a central clay core surrounded by an outer sand and gravel shell (random material zone). Construction began on the dam in 1976 and it started receiving CCR materials in 1978. The dam is approximately 4,580 ft in length and is composed of two linear segments. The western most segment starts at the right abutment and extends approximately 3,100 ft to a rock outcropping referred to as Geronimo Knob. At Geronimo Knob the dam centerline pivots approximately 40 degrees to the north forming the second linear segment which extends to the left abutment. The maximum height of the dam occurs between the right abutment and Geronimo Know with a maximum toe to crest height of 80 ft and crest width of 24 ft. The top of crest elevation is 5,120 ft producing 6 ft of total freeboard above the maximum permitted storage pool water level. Both the upstream and downstream slopes are inclined at a three horizontal to one vertical (3H:1V) angle with riprap facing to prevent erosion.

To limit seepage beneath the foundation, the central clay core of the Fly Ash Pond Dam extends to bedrock at relatively shallow depths, less than 20 ft. In the center portion of the dam where the depth to bedrock is greater than 20 ft, a slurry cutoff wall extends from the clay core to into the bedrock. The Fly Ash Pond Dam has no internal drain system; however, where seepage has been observed downstream of the dam, sumps have been installed to collect surface and groundwater and return it to the pond. These include systems for the Geronimo and Hunt Seeps that collect and return the water back to the Fly Ash Pond and the I-40 Seep that collects the water for evaporation.

The Fly Ash Pond has no intake or outlet water work structures. Water levels within the pond are controlled by varying the pumping rate from the plant and seepage collection system to balance with seepage and evaporation from the pond. Sluiced fly ash is pumped from the plant to the pond through three 6-inch diameter pressured discharge lines. The lines pass underneath of I-40, proceed up the downstream face of the embankment, pass over the dam crest, and empty into the pond basin. The dam was constructed without an overflow spillway channel. To prevent overtopping during the design storm event, defined as the probable maximum flood (PMF), the pond was constructed to fully contain the storm runoff on top of the maximum permitted storage pool water level. This water level, defined as the maximum surcharge pool water level, is estimated at EL 5,116 ft based on an expected water level rise of 2.0 ft during the PMF (Ebasco, 1976).

Piezometers, settlement monuments, flow measurement devices, and water level gauges are installed at the Fly Ash Pond to monitor the performance of the dam. Measurements from the monitoring instruments are reviewed and documented annually in a data report. Starting on October 19, 2015, the piezometer, survey monuments, and flow totalizers are read at intervals not exceeding 30 days per the requirements of 40 CFR § 257.83(a)(1)(iii). The locations of the monitored piezometers, survey monuments, and flow totalizers are shown on Figure 1-2.

Inspections of the Fly Ash Pond are performed by a qualified person at intervals not exceeding seven days. The inspections examine the Fly Ash Pond for actual or potential conditions that could disrupt the operation or safety of the impoundment and documents the results of the inspection in the facility's operating record. In addition, a more detailed annual inspection is performed by a qualified professional engineer. The annual inspection includes a review of available information on the dam, including the past year of monitoring data, a field inspection of the dam, abutment, and downstream toe and documentation of findings and recommendations in a dam safety inspection report. The most recent annual inspection of the Fly Ash Pond was performed on October 16, 2015 (AECOM & APS, 2016).

2 Hazard Potential Classification

This section summarizes the initial Hazard Potential Classification (HPC) for the Fly Ash Pond. This initial HPC is intended to meet the requirement for periodic hazard potential classification assessment of existing CCR surface impoundments per Rule 40 CFR § 257.73(a)(2).

2.1 Methodology and Design Criteria

Per the Rule, the hazard potential classification provides an indication of the possible adverse incremental consequences that result from the release of water or stored contents due to failure or mis-operation of the CCR surface impoundment. The classification is based solely on the consequences of failure. As such, it is not dependent of the condition of the embankment or the likelihood of failure. Classifications per the Rule are separate from relevant and/or applicable federal, state or local dam safety regulatory standards, which may also include hazard classification definitions, and are not intended to substitute for other regulatory hazard potential classifications.

Rule 40 CFR § 257.53 defines three hazard potential classifications as follows:

High hazard potential CCR surface impoundment – A diked surface impoundment where failure or mis-operation will probably cause loss of human life.

Significant hazard potential CCR surface impoundment – A diked surface impoundment where failure or mis-operation results in no probable loss of human life, but can cause economic loss, environmental damage, disruption of lifeline facilities, or impact other concerns.

Low hazard potential CCR surface impoundment – A diked surface impoundment where failure or mis-operation results in no probable loss of life and low economic and/or environmental losses. Losses are principally limited to the surface impoundment's owner's property.

The hazard potential of the Fly Ash Pond was assessed qualitatively, per the above definitions. The qualitative assessment process is generally performed in a step-wise manner by first determining whether the pond could be classified as low hazard potential, based on immediately obvious factors such as proximity to property lines and/or surface water bodies. After determining that a structure does not meet the criteria for a Low Hazard Potential classification, the structure is assessed to determine whether it meets the criteria for High Hazard Potential. The potential for loss of life differentiates between high and significant hazard potential in the Final CCR Rule; therefore, if the Dam does not meet the criteria for high hazard potential, it would be classified as a Significant Hazard Potential structure.

The potential for downstream loss of life is assessed by reviewing land use in areas downstream (to the south) from the Dam, where inundation is likely in the event of a release. No quantitative dam break or inundation studies were performed. The United States Geological Survey (USGS) 7.5-Minute Quadrangle topographic map of Joseph City, Arizona and associated digital orthoimage data (USGS, 2013) were used to review downstream areas for existing permanent and temporary land use. Permanent land uses include permanently inhabited dwellings and worksite areas that would likely contain workers on a daily basis (public utilities, power plants, water and sewage treatment plants, private industrial plants, sand and gravel plants, farm operations, fish hatcheries). Temporary land uses include primary roads, established campgrounds, or other recreational areas.

2.2 Hazard Potential Classification Results

Inspection of the Fly Ash Pond Dam and its immediate surrounding based on review of the USGS 7.5-Minute Quadrangle topographic map of Joseph City, AZ (USGS, 2013) identifies that the downstream toe of the Fly Ash Pond Dam is located within 100 ft of Interstate 40 (I-40), a major east-west route of the Interstate Highway System. A catastrophic and unexpected failure of the Fly Ash Pond Dam would likely inundate the travel lanes of I-40 and could result in loss of life. The Fly Ash Pond is therefore classified as a High Hazard Potential CCR surface impoundment.

3 History of Construction

This section summarizes the history of construction for the Fly Ash Pond. This information is intended to meet the requirement for compilation of the history of construction for each CCR surface impoundment per Rule 40 CFR § 257.73(c)(1).

3.1 Methodology

AECOM reviewed available documents obtained from APS, the ADWR Document Repository, or in-house resources for information regarding the history of construction for the Fly Ash Pond. Per the Rule, the compiled history of construction should include, to the extent feasible, the following information:

- Information identifying the CCR Unit, its purpose and the name and address of the owner/operator;
- The location of the CCR unit on the most recent USGS or other topographic map;
- Name and size of the watershed within which the CCR unit is located;
- A description of the physical and engineering properties of the foundation and abutment materials on which the CCR unit was constructed;
- A description of the type, size, and physical and engineering properties of each embankment zone;
- Provide detailed engineering drawings;
- A description of the type, purpose and location of existing instruments;
- Area-capacity curves for the CCR unit;
- A description of spillway and diversion design features;
- Construction specifications and provisions for surveillance, maintenance, and repair of the CCR unit; and
- Any record of knowledge of structural instability.

3.2 Fly Ash Pond Construction Summary

The history of construction dating back to the original construction that began in 1976 is summarized in Table 3-1 below.

Table 3-1. History of Construction for Cholla Fly Ash Pond

Item	As-Constructed/ Current	Comments	Reference Document
Name and Address of Owner	Arizona Public Service Company (APS): P.O. Box 53999, Phoenix, Arizona 85072	---	---
State ID No.	09.28	---	ADWR License of Approval dated October 8, 1986
Size Classification	Intermediate	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Hazard Classification	High	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Construction Date	Original: 1976 to 1977 Seepage Collection System: 1993	---	• Ash Pond Construction Memorandum (Temchin, 1977) • As-built Drawings APS No. G-44557 and G-44558 (Ebasco, 1977) • Seepage Intercept System Drawings No. D-114438, Sheets 1, 3 and 4 of 4 (APS, 1993)
Location on USGS Quadrangle Map	Joseph City Quadrangle: Section 24/19 and 25/30, Township 18 North, Range 20 East	See Figure 3-1	Joseph City Quadrangle (USGS, 2013)
Statement of Purpose	Fly ash containment		Seepage and Foundation Studies: Volume I of II Engineering Report (Ebasco, 1975).
Name of Watershed	---	---	---
Size of Watershed (ac)	1,230	---	• Seepage and Foundation Studies: Volume I of II Engineering Report (Ebasco, 1975) • Flood Routing Report (Ebasco, 1976)
Area Capacity Curve	See Figure 3-2	---	Seepage and Foundation Studies: Volume I of II Engineering Report (Ebasco, 1975)
Embankment Type	Zoned earth fill dam consisting of a clay core and shell	---	As-built Drawing APS No. G-44558 (Ebasco, 1977)
Embankment Maximum Height (ft)	80	---	As-built Drawing APS No. G-44558 (Ebasco, 1977)
Design Total Freeboard (ft)	6	Minimum residual freeboard following PMP event is 4 ft	Summary of Review of Plans and Specifications (AWC, 1976)

Item	As-Constructed/ Current	Comments	Reference Document
Embankment Length (ft)	4,580	---	Drawing No. G-558, Rev. No. 7 (Ebasco, 1977)
Embankment Crest Elevation (ft)	5,120		As-built Drawing APS No. G-44558 (Ebasco, 1977)
Embankment Crest Width (ft)	24	---	As-built Drawing APS No. G-44558 (Ebasco, 1977)
Embankment Slopes	3H:1V (downstream & upstream)	---	As-built Drawing APS No. G-44558 (Ebasco, 1977)
Slope Protection	Riprap and random rock		As-built Drawing APS No. G-44558 (Ebasco, 1977)
Maximum Operating Storage Level (ft)	5,114	Previous maximum storage levels were: 5,116 ft (1981)	• Summary of Review of Plans and Specifications (AWC, 1976) • ADWR License dated October 8, 1986
Storage Capacity (ac-ft)	Original design: 16,500	Storage at EL 5,116 ft	Seepage and Foundation Studies: Volume I of II Engineering Report (Ebasco, 1975)
Surface Area (ac)	440	Area at EL 5,116 ft	• Seepage and Foundation Studies: Volume I of II Engineering Report (Ebasco, 1975) • Flood Routing Report (Ebasco, 1976)
Clay Core Properties			
Physical Properties	The clay core consists of compacted sandy lean clay and sandy fat clay.	---	• Seepage and Foundation Studies: Volume II of II Field and Laboratory Tests (Ebasco, 1975) • Safety Inspection Report (Harza, 1987) • Evaluation of Dam Embankment Crack (Dames & Moore, 1999)
Engineering Properties	• Moist Unit Weight = 120 pounds per cubic foot (pcf) • Saturated Unit Weight = 125 pcf • Effective Cohesion = 0 pounds per square foot (psf) • Effective Friction Angle = 28^{o} • Undrained strength ratio = 0.38	---	
Shell (Random Zone) Properties			
Physical Properties	The shell consists of compacted silty or clayey sand and sandy lean clay.	---	• Seepage and Foundation Studies: Volume II of II Field and Laboratory Tests (Ebasco, 1975) • Safety Inspection Report (Harza, 1987) • Evaluation of Dam Embankment Crack (Dames & Moore, 1999)
Engineering Properties	• Moist Unit Weight = 125 pcf • Saturated Unit Weight = 130 pcf • Effective Cohesion = 0 psf • Effective Friction Angle = 33^{o}	---	

<table>
<tr><th>Item</th><th>As-Constructed/ Current</th><th>Comments</th><th>Reference Document</th></tr>
<tr><td colspan="4">Foundation Conditions</td></tr>
<tr><td>Physical Properties</td><td>The embankment is founded on an engineered keyway consisting of the compacted clay core extending to competent bedrock. The exposed bedrock was cleaned and treated with grout or concrete prior to placement of fill material. Where bedrock is deeper than 20 ft, a soil-bentonite cutoff wall extends through the alluvium to bedrock or stiff clay. The alluvium is a Quaternary age wash deposit consisting of unconsolidated clays, silts, and sands. The underlying bedrock consists of mudstone, siltstone, and sandstone associated with the Chinle and Moenkopi Formations.</td><td>---</td><td rowspan="2">Seepage and Foundation Studies: Volume II of II Field and Laboratory Tests (Ebasco, 1975)
Various Construction Reports (Ebasco, 1977)
Safety Inspection Report (Harza, 1987)
Evaluation of Dam Embankment Crack (Dames & Moore, 1999)</td></tr>
<tr><td>Engineering Properties</td><td><u>Alluvium:</u>
Moist Unit Weight = 120 pcf
Saturated Unit Weight = 120 pcf
Effective Cohesion = 0 psf
Effective Friction Angle = 26^o
<u>Bedrock:</u>
Moist Unit Weight = 150 pcf
Saturated Unit Weight = 150 pcf
Effective Cohesion = 1,000 psf
Effective Friction Angle = 65^o
<u>Cutoff Wall:</u>
Moist Unit Weight = 106 pcf
Saturated Unit Weight = 106 pcf
Effective Cohesion = 0 psf
Effective Friction Angle = 28^o
Undrained Strength = 10 psf</td><td>---</td></tr>
</table>

Item	As-Constructed/ Current	Comments	Reference Document
Abutment Conditions			
Physical Properties	The abutments consist of bedrock comprising mudstone, siltstone, and sandstone associated with the Chinle and Moenkopi Formations. A clay blanket was placed along a 250-foot section of the right abutment.	---	• Seepage and Foundation Studies: Volume II of II Field and Laboratory Tests (Ebasco, 1975) • As-built Drawings No. G-557 and G-558 • Safety Inspection Report (Harza, 1987) • Evaluation of Dam Embankment Crack (Dames & Moore, 1999)
Engineering Properties	• Moist Unit Weight = 150 pcf • Saturated Unit Weight = 150 pcf • Effective Cohesion = 1,000 psf • Effective Friction Angle = 65^{o}	---	
Spillway	None	The impoundment has sufficient storage volume above the maximum storage pool water level to store the IDF (PMF) and maintain at least four ft of freeboard.	Summary of Review of Plans and Specifications (AWC, 1976)
Construction Specifications	Clay Core: • Fines content ranging from 50% to 100% • No particle sizes greater than 3 inches • Initial plasticity index range from 15 to 50; changed to 10 to 50 in July 1977 • Fill lift thickness = 8 inches • Initial minimum degree of compaction = 90% (modified Proctor); changed to 95% (standard Proctor) in June 1977. • Test frequency = 60,000 ft^2/test Shell (Random Zone): • Maximum rock fraction greater than 3 inches = 10% • Fill lift thickness = 12 inches • Minimum degree of compaction = 100% (standard Proctor) • Test frequency = 60,000 ft^2/test	---	Ash Pond Construction Memorandum (Temchin, 1977)

Item	As-Constructed/ Current	Comments	Reference Document

Construction Specifications (continued)	Cutoff Wall:
	• Preparation:
	  ○ Minimum unit weight = 1.02 grams/cubic centimeter (g/cm^3)
	  ○ Minimum viscosity = 35 sec-marsh
	  ○ Maximum filtration loss = 30 cm^3
	  ○ Minimum pH = 8
	• In Trench:
	  ○ Unit weight range between 1.05 and 1.4 g/cm^3
	• Backfill Mix at Trench:
	  ○ Slump ranging between 3 and 6 inches
	  ○ Percent passing 3/8-inch between 70 and 100%
	  ○ Percent passing No. 20 sieve between 40 and 80%
	Fines content between 10 and 35%
		---	Ash Pond Construction Memorandum (Temchin, 1977)

Detailed Drawings	See Appendix A for drawings	---	• Original As-built (Ebasco, 1977) • Seepage Interception System (APS, 1993)
Existing Instrumentation			
Type and Purpose of Instrumentation	• Open standpipe piezometers and wells installed for monitoring the phreatic levels in the embankment, foundation, and surrounding area. • Settlement monuments for monitoring movement of the embankment. • Water level gauge for monitoring water level in reservoir. • Flowmeters measuring seepage rates.	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)

Item	As-Constructed/ Current	Comments	Reference Document
Location of Instrumentation	• Open standpipe piezometers and wells located in and around the embankment. • Movement monuments located along the embankment crest. • Water level gauge located in the reservoir. • Seepage monitoring systems located along the downstream toe.	See Figure 1-2	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Provisions for Surveillance, Maintenance and Repair	• Visual inspections of the dam by a qualified person on a frequency not exceeding seven days. • Visual inspections of the dam conducted annually by a qualified professional engineer. • Phreatic level behavior from piezometric measurements and reservoir water level from gauge collected on a frequency not exceeding 30 days. • Embankment settlement using movement monuments survey data collected on a frequency not exceeding 30 days. • Seepage monitoring at the downstream toe on a frequency not exceeding 30 days.	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Record of Structural Instability (See Section 4 for more details)	• Historic seepage at downstream toe and right abutment. Seepage areas near the downstream toe are identified as Hunt Seep and Geronimo Seep, and I-40 Seep. • Crack within clay core near Geronimo Knob, generally between survey monuments M6 and M7.	See Figure 1-2 for the Hunt and Geronimo Seeps. The seepage areas are captured and monitored by a seepage interceptor system near the downstream toe.	• Transverse Crack Evaluation (URS, 2001) • Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)

Notes: 1) Site elevations use National Geodetic Vertical Datum (NGVD) 1929

4 Structural Stability Assessment

This section summarizes the structural stability assessment for the Fly Ash Pond. This information is intended to satisfy the requirement of Rule 40 CFR *§* 257.73(d).

4.1 Foundation and Abutments

Per the requirements of 40 CFR *§* 257.73(d)(1)(i), an existing CCR impoundments must be assessed for "*Stable foundations and abutments*."

The Fly Ash Pond Dam is founded on alluvium overburden associated with a local wash with both abutments resting on bedrock consisting of mudstone, siltstone, and sandstone associated with the Chinle and Moenkopi Formations. Review of the as-built design drawings of the dam (Ebasco, 1977) and construction inspection reports prepared by ADWR (formerly the Arizona Water Commission) indicate a cut off trench was excavated at the abutments to extend the clay core to bedrock. When the depth to bedrock was greater than 20 ft, a soil-bentonite slurry cut-off wall was installed to the bedrock which extended to a maximum depth of about 40 ft below the original ground surface. In addition, an approximately 250-ft long clay blanket was installed on the upstream slope of the right abutment directly adjacent to the embankment to help control seepage through the surrounding Moenkopi bedrock formation. Review of construction records indicates that where the cutoff trench was excavated to bedrock, loose rock was scaled from the foundation, dental concrete was applied to irregularities to create a relatively level surface, and a thin lift of wet cement tack coat was applied to the bedrock surface before placement of the clay core. For the shell of the dam, which is founded on alluvium overburden soils, the alluvium foundation was proof-compacted using a heavy dynamic compactor and surface stringers of sandy soils that crossed the dam foundation were removed.

Several seepage locations have been observed downstream of the dam since the Fly Ash Pond went into operation. These seeps are thought to occur due to a combination of normal flow through the embankment, discontinuities in the foundation near the groin of the abutment at Geronimo Knob, and flow through gypsum seams in the Moenkopi Formation. Drain systems have been installed at most of the seepage locations, typically consisting of underground French drains connected to a collection sump. Two sumps have been installed at the following seeps: the Geronimo Seep and the Hunt Seep. The locations of the seeps are shown in Figure 1-2. Flow from the sumps and weir installed at the seeps are monitored and presented in the annual dam inspection reports. Flow rates ranging from 6 to 40 gallons per minute over the last ten years were measured at the sumps (AECOM & APS, 2016), indicating low to moderate flow. The turbidity of the seep water observed at the sumps was low. The long-term steady and low to moderate flow rate, combined with the lack of turbidity, indicate a low potential of internal erosion of the dam embankment or foundation.

Review of the measured displacements of the survey monuments at the crest of the Fly Ash Pond Dam, as presented in the 2015 annual dam inspection report (AECOM & APS, 2016), indicates total settlements along the crest of the dam of four to seven inches and horizontal movements of four inches or less in the last ten years. Settlement rates appear relatively consistent over the last ten years at about one half of an inch per year, except in 2010 when recalibration of the survey base point appears to have increased the reported settlement by one additional inch. The relatively small settlement and horizontal movements measured at the Fly Ash Pond Dam are an indication of stability in the dam foundation and abutments.

4.2 Slope Protection

Per the requirements of 40 CFR *§* 257.73(d)(1)(ii), an existing CCR impoundments must be assessed for "*Adequate slope protection to protect against surface erosion, wave action, and adverse effects of sudden drawdown*."

A review on the as-built drawing of the Fly Ash Pond Dam (Ebasco, 1977), indicates the dam was constructed with a two foot thick layer of random rock fill (riprap) to protect the upstream and downstream slopes against erosion. No specifications for riprap size were shown on the drawings; however, visual observations performed during dam inspection suggest they are cobble to boulder sized. The 2015 annual dam inspection report (AECOM & APS, 2016) reported no significant erosion of the dam slopes indicating the riprap slope protection is performing adequately. Based on the inspection report and experience with similar riprap slope protection designs, the Fly Ash Pond has adequate slope protection to protect against surface erosion, wave action, and adverse effects of sudden drawdown.

4.3 Dike Compaction

Per the requirements 40 CFR *§* 257.73(d)(1)(iii), an existing CCR impoundments must be assessed for "*Dikes mechanically compacted to a density sufficient to withstand the range of loading conditions in the CCR unit*."

Based on review of a memorandum summarizing construction of the Fly Ash Pond Dam (Temchin, 1977), the dam (or dike) was constructed by placement of soils in mechanically compacted thin lifts of a foot or less. Construction control of the compaction process was maintained using a method procedure where the soil preparation, placement, watering, blading, final watering, rolling, and lift thickness are specified based on the results of test fill pads conducted prior to start of earthwork (Ebasco, 1977).

In addition to the method controls discussed above, quality control testing consisting of comparison of in-situ measurements of soil density to Standard Proctor maximum dry density, American Society for Testing and MaterialsD 698, was performed at intervals of once every 60,000 square ft of material placed. Results of quality control testing are summarized in Ebasco Drawing APS-2742-SK-CH-J13 (Temchin 1977). The drawing indicates 622 tests were conducted on Clay Core materials with 609 of the tests measuring densities greater than 95 percent of the Standard Proctor maximum density and a mean percent compaction of all tests of 98.9 percent of the standard proctor maximum density. The drawing indicates 811 tests were conducted on the outer shell materials with 748 of the tests measuring densities greater than 100 percent of the Standard Proctor maximum density and a mean percent compaction of all tests of 101.7 percent of the Standard Proctor maximum density.

Based on the compaction method described in the construction summary memorandum and the quality control test results presented in Drawing APS-2742-SK-CH-J13, the Fly Ash Pond Dam has been mechanically compacted to a density sufficient to withstand the range of loading conditions expected at the Fly Ash Pond site.

4.4 Slope Vegetation

Per the requirements 40 CFR *§* 257.73(d)(1)(iv), an existing CCR impoundments must be assessed for "*Vegetated slopes of dikes and surrounding areas, except for slopes which have an alternate form or forms of slope protection*." Note that the United States Court of Appeals for the District of Columbia Circuit remanded with vacatur the phrase "*not to exceed a height of six inches above the slope of the dike*" from this subsection of the Rule.

As noted in Section 4.2, the dam was constructed with a two foot thick layer of random rock fill (riprap) slope protection; therefore, the dam is excluded from the vegetated slope requirements since it uses an alternate form of slope protection.

4.5 Impoundment Capacity

Per the requirements 40 CFR *§* 257.73(d)(1)(v), an existing CCR impoundment must be assessed for "*A single spillway or a combination of spillways configured as specified in paragraph (d)(1)(v)(A) of this sections. The combined capacity of all spillways must be designed, constructed, operated, and maintained to adequately manage flow during and following the peak discharge from the event specified in paragraph (d)(1)(v)(B) of this section*."

The Fly Ash Pond Dam was constructed without a spillway or other water release structure. To manage flow during the design storm event, the Fly Ash Pond has been designed, constructed, operated, and maintained with sufficient storage volume over and above the maximum permitted storage pool water level at EL 5,114 ft to store the PMF storm water inflow at EL 5,116 ft and to maintain an additional four ft of freeboard; therefore, the Fly Ash Pond impoundment is capable of adequately managing (containing) the flow during and following the peak discharge from the PMF event as required for high hazard potential CCR impoundments.

4.6 Hydraulic Structures

Per the requirements 40 CFR § 257.73(d)(1)(vi), an existing CCR impoundments must be assessed for "*Hydraulic structures underlying the base of the CCR unit or passing through the dike of the CCR unit that maintain structural integrity and are free of significant distortion, bedding deficiencies, sedimentation, and debris which may negatively affect the operation of the hydraulic structures*."

No hydraulic structures are present that underlie the base of the Fly Ash Pond or pass through the Fly Ash Pond Dam.

4.7 Downstream Water Body

Per the requirements 40 CFR § 257.73(d)(1)(vii), an existing CCR impoundments must be assessed as follows "*For CCR units with downstream slope which can be inundated by the pool of an adjacent water body, such as a river, stream or lake, downstream slopes that maintain structural stability during low pool of the adjacent water body or sudden drawdown of the adjacent water body*."

No structural stability deficiencies are presently associated with inundation of the downstream slope of the Fly Ash Pond Dam by an adjacent body of water since no pool of water, such as a river, stream or lake, is present downstream of the dam which could inundate the downstream slope.

4.8 Other Issues

In July 1998, transverse and longitudinal cracking was observed along the Fly Ash Pond Dam crest in the vicinity of the Geronimo Knob, the rock outcropping near the center of the dam. A subsequent study of the cracks, consisting of exploration trenches and borings along the crest of the dam, exposed thirty-one (31) visible cracks with six (6) cracks considered "significant" (defined as cracks with widths equal to or greater than ½-inch.) Crack depths ranged from 0.5 to 12 ft below the top of crest (Dames & Moore, 1999). The study postulated the cracking was due to differential settlement of the dam embankment on the sloping bedrock foundation created by the Geronimo Knob (URS, 2001). The dam crest was repaired by re-compaction of the clay core spoils excavated during the trenching. As an additional precaution, the discharge to the impoundment was changed so that deposited fly ash would create a beach that would prevent free water from ponding within 300 ft of the crack area. Since 2002, continued monitoring of the dam crest has noted only minor cracking, most likely associated with surface desiccation typical for embankments in the arid US Southwest. While monitoring of the dam crest for cracking is still performed during the annual dam inspections, the Geronimo Knob crack is considered to have been mitigated by the changed deposition plan, has not reappeared, and it is not considered a continuing dam safety concern or structural integrity deficiency.

No deficiencies were identified for the Fly Ash Pond that could affect the structural stability of the impoundment. However, during the most recent dam inspection (AECOM & APS, 2016), observations of excessive vegetation consisting of small to medium sized desert brush and small animal burrows were noted along the slopes and crest of the Fly Ash Pond Dam. APS work crews subsequently removed part of the vegetation in the identified areas with the remainder scheduled for removal in the upcoming year. Although both the vegetation and the animal burrows were not of sufficient size to cause concern for the stability or erosion of the embankment, failure to promptly identify and correct these issues could lead to eventual deterioration of the embankment slope. It is recommended, therefore, to continue inspection and maintenance activities of the impoundment to identify and correct minor issues in order to prevent progressive deterioration of the embankment.

4.9 Structural Stability Assessment Results

AECOM did not identify any structural stability deficiencies that would affect the structural condition of the Fly Ash Pond CCR Impoundment based on the documents provided and reviewed as part of this assessment. AECOM assesses that the design, construction, operation and maintenance of the Fly Ash Pond are consistent with recognized and generally accepted good engineering practice for the maximum volume of CCR and CCR wastewater which can be impounded therein.

5 Safety Factor Assessment

This section summarizes the safety factor assessment for the Fly Ash Pond. This assessment is intended to satisfy the requirement of Rule 40 CFR § 257.73(e).

5.1 Methodology and Design Criteria

Slope stability analyses were performed to document minimum factors of safety for loading conditions identified by 40 CFR § 257.73(e) using the software program SLOPE/W (GEO-SLOPE International, 2012). The analyses were performed using Spencer's Method; a limit equilibrium method of slices that satisfies both force and moment equilibrium and incorporates the effects of interslice forces. The analyses incorporate strength and density properties and pore pressure distributions described in Sections 5.4 and 5.5. The slope stability models are presented in Appendix B.

5.2 Critical Cross Section

Safety factors were calculated for three cross sections of the Fly Ash Pond Dam selected to represent different embankment geometries, heights, and stratigraphic conditions to provide confidence that the critical cross section was identified. The critical cross section is the cross section that is anticipated to be most susceptible to structural failure for a given loading condition. The critical cross section thus represents a "most-severe" case. Section locations were selected based on variation in the embankment height, presence of cutoff trench/cutoff wall, and stratigraphic conditions. Subsurface soil profiles were developed using as-built drawings and historical borings reported by Ebasco (1975) and Harza (1987). The locations of the cross sections along the Fly Ash Pond Dam are shown in Figure 5.1. The three cross sections analyzed are:

Fly Ash Pond Cross Section 1: This cross-section corresponds approximately to Section B as shown in Figure 5-1 and on the as-built section (Ebasco, 1977). This section represents the highest dam section where bedrock is shallow and, thus, includes an extension of the embankment clay core forming a cutoff trench that is keyed into bedrock. The embankment is approximately 80 ft high and the upstream and downstream slopes are at 3H:1V. The zoned embankment at this section consists of a sandy lean clay core with an outer clayey sand shell and the foundation consists of about 20 ft of alluvial clays, silts, and sands overlying bedrock consisting of mudstones, siltstones, and sandstones. The clay core extends to form a cutoff trench that is keyed into the top of bedrock.

Approximately 60 ft of hydraulically-placed fly ash is impounded behind the embankment at the Cross Section 1 location, based on comparison between pre-construction topographic survey data (Ebasco, 1975) and topographic survey data collected in 2014 (APS, 2016).

Fly Ash Pond Cross Section 2: This cross-section corresponds approximately to Section D as shown on Figure 5-1 and the as-built section (Ebasco, 1977). This section represents the section at the greatest depth to bedrock. The cross-section is located approximately 50 ft west of a long-standing downstream seep, the Geronimo Seep, which lies near Geronimo Knob. The section includes a cutoff slurry wall beneath the embankment clay core. The embankment is approximately 80 ft high and the upstream and downstream slopes are at 3H:1V. The zoned embankment at this section consists of a sandy lean clay core with an outer clayey sand shell and the foundation consists of approximately 52 ft of alluvial overburden (clays, silts, and sands) overlying interbedded layers of mudstone, siltstone, and sandstone bedrock.

Approximately 60 ft of hydraulically-placed fly ash is impounded behind the embankment at the Cross Section 2 location, based on comparison between pre-construction topographic survey data (Ebasco 1975) and topographic survey data collected in 2014 (APS, 2016). Calculated factors of safety for Section 2 were lower than those calculated for Sections 1 and 3. Section 2 is, therefore, designated the critical cross section.

Fly Ash Pond Cross Section 3: This cross section corresponds approximately to Section E as shown on Figure 5-1 and the as-built section (Ebasco, 1977). At this cross section location, the Fly Ash Pond intersects Geronimo Knob along its downstream slope. This section includes an extension of the embankment clay core forming a cutoff trench that is keyed into bedrock. The embankment is approximately 68 ft high and the upstream and downstream slopes are at 3H:1V. The zoned embankment at this section consists of a sandy lean clay core with an outer clayey sand shell and the foundation consists of approximately four to nine ft of alluvial overburden (clays, silts, and sands) overlying interbedded layers of mudstone, siltstone, and sandstone bedrock.

Approximately 50 ft of hydraulically-placed fly ash is impounded behind the embankment at the Cross Section 3 location, based on comparison between pre-construction topographic survey data (Ebasco, 1975) and topographic survey data collected in 2014 (APS, 2016).

5.3 Subsurface Stratigraphy

Idealized models of subsurface stratigraphic conditions for each cross section were developed based on design drawings (Ebasco, 1977) and previous geotechnical site investigations (Ebasco, 1975, Harza, 1987, and Dames & Moore, 1999). The following stratigraphic units were used to develop SLOPE/W models for each cross section:

Embankment Core: The zoned embankment includes a central impervious clay core with 1H:1V side slopes and a clay cap at the embankment crest. Fine-grained material was obtained from upstream borrow pits along the dam alignment and mechanically compacted in lifts to construct the clay core. The clay core soils consist predominately of Sandy Lean Clay (CL) with isolated zones of Sandy Fat Clay (CH) based on the Unified Soil Classification System (USCS).

Embankment Shell (Random Zone): The zoned embankment includes a more pervious zone of random material, or shell that flanks the clay core to support and protect the impervious core. The shell provides stability against rapid drawdown (upstream shell) and drainage (downstream shell). Shell material was obtained from upstream borrow pits along the dam alignment and mechanically compacted in lifts. Shell soils consist predominately of Silty Sand (SM), Clayey Sand (SC), and Sandy Lean Clay (CL) based on the USCS.

Alluvium: Alluvial deposits overlie the bedrock beneath the embankment and are the foundation bearing layer over most of the embankment alignment. The alluvium consists of a Quaternary Age, heterogeneous mixture of unconsolidated clays, silts, and sands deposited by flows in an unnamed tributary to the Little Colorado River prior to the construction of the Fly Ash Pond.

Bedrock: Bedrock beneath the embankment consists of mudstones, siltstones, and sandstones of the Triassic-age Chinle and Moenkopi Formations.

Slurry Cutoff Wall: A slurry cutoff wall was constructed using soil-bentonite slurry where the depth to bedrock is greater than 20 ft and extended into either the bedrock or dense clay soils.

Fly Ash: Fly ash waste product from the power generating process is pumped from the plant to the Fly Ash Pond and allowed to settle hydraulically.

5.4 Material Properties

Material properties for soil, rock and embankment construction materials were developed based on an analysis and interpretation of historical geologic and geotechnical data presented in:

- Ebasco Services Inc., “Arizona Public Services Cholla Generating Station Ash Disposal Sites Seepage and Foundation Studies: Volume I of II Engineering Report” (Ebasco, 1975),
- Harza Engineering Company, “Safety Inspection Report on Fly Ash Dam, Bottom Ash Dam, and Cooling Dike” (Harza, 1987), and
- Dames & Moore, ” Interim Report, Geotechnical Investigation for Evaluation of Dam Embankment Crack, Fly Ash Pond Dam, Cholla Power Plant, Joseph City, Arizona” (Dames & Moore, 1999).

The material properties developed by the dam designers and subsequent investigators were assessed for reliability and applicability to this safety factor assessment. The design report (Ebasco, 1975) indicated that soil strength parameters were obtained from laboratory testing. Specific details of the soil strength property derivations used for the original design stability analyses were not provided in the design report. The Harza investigation (1987) included more detailed documentation of the laboratory testing, soil strength derivations, and stability analyses performed in 1987. The parameters developed by Harza were used in subsequent stability analyses performed by Dames & Moore (1991). AECOM assessed the historical soil strength data and parameters used by previous investigators and found the Harza (1987) data to be the most reliable and applicable to this safety factor assessment.

The material properties selected for use in the slope stability analyses of the Fly Ash Pond Dam are presented in Table 5-1. The drained strength values presented in Table 5-1 were taken from Harza (1987). The undrained strength value presented in Table 5-1 for the Embankment Core was derived by AECOM based on interpretation of the Harza Triaxial Compression Test data. Undrained strength properties were not needed for other material types for the safety factor calculations. Moist unit weight values used in this safety factor assessment were taken from Dames & Moore (1991); saturated unit weights were interpreted by AECOM based on the moist unit weights and material types reported by previous investigators. The Fly Ash unit weight was selected by AECOM to be 90 pounds per cubic foot (pcf) based on engineering experience with similar materials.

Table 5-1. Selected Material Parameters – Fly Ash Pond Safety Factor Assessment

Material	Saturated Unit Weight, γ_{sat} (pcf)	Moist Unit Weight, γ_m (pcf)	Effective Strengths		Total Strengths	
			Cohesion, c' (psf)	Friction Angle, ϕ' (degrees)	Undrained Strength, S_u (psf)	Undrained Strength Ratio
Embankment Core	125	120	0	28	-	0.38
Embankment Shell	130	125	0	33	-	-
Alluvium	120	120	0	26	-	-
Bedrock	150	150	1,000	65	-	-
Slurry Cutoff Wall	106	106	0	28	10	-
Fly Ash	90	90	0	0	-	-

5.5 Embankment Pore Pressure Distribution

Water levels have been historically monitored weekly to quarterly and are now monitored on an interval not exceeding 30 days in piezometers installed along or near the Fly Ash Pond and reported annually in an inspection report (AECOM & APS, 2016). These data were considered to be the most reliable indicators of pore pressure distribution within the Fly Ash Pond Dam embankment. The pore pressure distributions were estimated for each section using water level measurements obtained from:

- Cross Section 1: Piezometers F-93, F104, and F-105;
- Cross Section 2: Piezometers F-90, F-91, F-92, F-109, F-110, F-132, and F-134;
- Cross Section 3: Piezometers F-112, F-127, F-128, F-129, and F-130.

Piezometer locations are shown on Figure 1-2. Piezometer data were used, along with pond water level under steady-state, maximum permitted storage pool conditions (ADWR, 1986), and pond water levels under maximum surcharge pool conditions (Ebasco, 1975) to estimate pore pressure distributions within the embankment sections.

The pore water levels measured in the piezometers near Cross Section 2 reflect the influence of the Geronimo Seep collection system. The collection system consists of an underground French drain system and wellpoints and has been in continuous operation since the early 1990s. The seep collection system presumably lowers the phreatic water level at the downstream toe of the dam in the vicinity of the wellpoints. Since the radial influence of the collection system is not documented, a conservative assumption of a non-operational Geronimo Seep seepage collection system was used in the stability analysis of

Cross Section 2. This assumption corresponds to the condition of raising the water level downstream of the dam to near the ground surface.

5.6 Embankment Loading Conditions

Per 40 CFR *§* 257.73(e)(1)(i) through (iv), the following loading conditions were analyzed for each developed stability cross section:

- Long-term, maximum storage pool
- Maximum surcharge pool
- Seismic loading, and
- Liquefaction

These loading conditions are described in the following sub-sections.

Long-Term, Maximum Storage Pool: The maximum storage pool loading is the maximum water level that will be maintained for a sufficient length of time for steady-state seepage or hydrostatic conditions to develop within the embankment. This loading condition is evaluated to document whether the CCR surface impoundment can withstand a maximum expected pool elevation with full development of saturation in the embankment under long-term loading.

The long-term, maximum storage pool loading condition was evaluated using the permitted water level of the pond, as stated in the ADWR operating license for the dam. Since the dam has no outlet structure and relies on pumping rate from plant, seepage, and evaporation to control water levels, the maximum storage pool was set at the maximum ADWR-permitted water levels. For the Fly Ash Pond, the safety factor was calculated for the long-term maximum storage pool at EL 5,114 ft (ADWR, 1985).

Maximum Surcharge Pool: The maximum surcharge pool loading is the temporary rise in pool elevation above the maximum storage pool elevation to which the CCR surface impoundment could be subject under inflow design flood state. This loading condition is evaluated to document whether the downstream slope of the CCR surface impoundment embankment can withstand the short-term impact of a raised pool level.

The maximum surcharge pool considers a temporary pool elevation that is higher than the maximum storage pool that persists for a length of time sufficient for steady-state seepage or hydrostatic conditions to fully develop within the embankment. The maximum surcharge pool loading condition was evaluated using the expected water level raise during the design PMF of 2.0 ft (Ebasco, 1976). For the Fly Ash Pond, the safety factor was calculated for the maximum surcharge pool at EL 5,116 ft.

Seismic Loading: Seismic loading is evaluated to document whether the embankment is capable of withstanding a design earthquake without damage to the foundation or embankment that would cause a discharge of contents. The seismic loading condition is assessed for a seismic loading event with a two percent probability of exceedance in 50 years, equivalent to a return period of approximately 2,500 years. A pseudo-static analysis was used to represent the seismic loading condition.

The seismic response of soil embankments is incorporated into the analysis method by adding a horizontal force to simulate the seismic force acting on the embankment during an earthquake. The horizontal force is applied in the pseudo-static analyses through the addition of a seismic coefficient into the limit equilibrium calculations. The seismic coefficient was selected using the following procedure:

1. Determine the peak horizontal ground acceleration (PGA) generated in bedrock at the site by an earthquake having the 2% probability of exceedance in 50 years;

2. Select a Site Class, per International Building Code definitions, which incorporates the effects of seismic wave propagation through the top 100 ft of the soil profile above bedrock, and calculate the adjusted for Site Class effects, PGA_M;

3. Calculate the maximum transverse acceleration at the crest of the embankment, PGA_{crest}, using the PGA_M from step two; and

4. Adjust the PGA_{crest} using the method developed by Makdisi and Seed (1977) to account for the variation of induced average acceleration with embankment depth to calculate the seismic coefficient.

Each of these steps is discussed in more detail in the calculation presented in Appendix B. The maximum average acceleration for the potential sliding mass was incorporated into the pseudo-static safety factor analyses as the horizontal seismic coefficient equal to 0.13, corresponding to the calculated, adjusted PGA_{crest} value.

The water level in the Fly Ash Pond for the seismic loading analysis was set to EL 5,114 ft to match the long-term, maximum storage pool. The Clay Core and Cutoff Wall materials were assigned total strengths because it is anticipated that they would behave in an undrained manner due to the relatively rapid loading induced during the seismic event and the relatively low hydraulic conductivity of these materials. All, other materials used effective strength parameters.

Liquefaction: The liquefaction factor of safety is evaluated for CCR embankments and foundation soils that are believed to be susceptible to liquefaction based on representative soil sampling and construction documentation or anecdotal evidence from personnel with knowledge of the CCR unit's construction., The liquefaction factor of safety is calculated to document whether the CCR unit would remain stable if the soils in the embankment and/or foundation experienced liquefaction.

Post-construction geotechnical exploration of the Fly Ash Pond Dam (Harza, 1987 and Dames & Moore, 1999) indicated the Clay Core (embankment) and Alluvium Overburden (foundation) materials have plasticity indexes and fine contents as shown in Table 5-2. Data are not presented in Table 5-2 for the Embankment Shell material because of limited available geotechnical data because the Embankment Shell material was sourced from the Alluvium Overburden and is anticipated to have similar properties. Generally, the behavior of soils that have fines contents greater than 35 percent are dominated by the plasticity of the fines (Idriss and Boulanger, 2008). Fines with Plasticity Indices (PI) less the seven tend to behave more sand-like and are susceptible to soil liquefaction, while those with PI greater than seven tend to behave more clay-like and are not susceptible to liquefaction. The lowest measured value of PI for both the Clay Core and Alluvium Overburden is 12, indicating these soils would tend to behave in a clay-like manner during a seismic event and not be susceptible to soil liquefaction. Therefore, a liquefaction factor of safety analysis was not assessed as being necessary and was not performed for this impoundment.

Table 5-2. Range of Plasticity Index and Fines Content Values for Site Materials

Material	Plasticity Index, %		Fines Contents, %	
	Minimum Value	Maximum Value	Minimum Value	Maximum Value
Clay Core	12	39	48	88
Alluvium Overburden	12	17	30	54

5.7 Safety Factor Assessment Results

Table 5-3 summarizes the results of the safety factor analysis for the Fly Ash Pond Dam, for a more detailed discussion of the results see the safety factor calculation presented in Appendix B.

Table 5-3. Summary of Calculated Safety Factors

Loading Condition	Required Safety Factor[1]	Calculated Safety Factor		
		Section 1	Section 2	Section 3
Long-term, maximum storage pool	1.50	1.63	1.53	1.73
Maximum surcharge pool	1.40	1.61	1.52	1.70
Seismic	1.00	1.08	1.02	1.15

Notes: [1] From 40 CFR § 257.73(e)(1)(i) through (iii) (EPA, 2015)

The calculated factors of safety for the three critical cross sections along the Fly Ash Pond Dam exceeded the required minimum values for the long-term, maximum storage pool; the maximum surcharge pool; and the seismic (pseudo-static) loading conditions.

6 Conclusions

Based on the findings and results of the structural integrity assessment, AECOM provides the following conclusions regarding the structural integrity of the Fly Ash Pond at the Cholla Power Plant.

- The Fly Ash Pond is classified as a High Hazard Potential CCR surface impoundment.
- The embankment is founded on stable foundations and abutments. Seepage is limited by a clay core that extends to the bedrock in shallow locations or a cutoff slurry wall where the depth to bedrock is greater than 20 ft. Downstream seeps exist and are captured and monitored by drainage systems typically consisting of French drains connected to sumps.
- The embankment has adequate slope protection consisting of riprap on both the upstream and downstream slopes.
- Based on the available quality control test results, the Fly Ash Pond Dam embankment was mechanically compacted to a density sufficient to withstand the range of loading conditions anticipated at the site.
- The Fly Ash Pond impoundment is capable of adequately managing the flow during and following the peak discharge from the PMF event without a spillway or other water release structures because the pond has been designed, constructed, operated, and maintained with sufficient storage volume above the maximum storage pool water level to store the PMF inflow and maintain at least four ft of freeboard.
- Factors of safety greater than the minimum values required by the CCR Rule were calculated for three cross sections along the Fly Ash Pond Dam for loading conditions associated with the maximum storage pool water level, maximum surcharge pool water level, and design level seismic event. The liquefaction factor of safety of the impoundment was not analyzed due to the low potential for soil liquefaction of the embankment and foundation soils as determined from index test results.
- Based on review of available records concerning the Fly Ash Pond and the results of the stability analyses, no deficiencies were noted that would affect the structural condition of the dam.

7 Limitations

This report is for the sole use of APS on this project only, and is not to be used for other projects. In the event that conclusions based upon the data obtained in this report are made by others, such conclusions are the responsibility of others. The Initial Structural Stability Assessment presented in this report was based on available information identified in Reference Section of the report that AECOM has relied on but not independently verified. Therefore, the Certification of Professional Opinion is limited to the information available to AECOM at the time the Assessment was performed in accordance with current practice and the standard of care. Standard of care is defined as the ordinary diligence exercised by fellow practitioners in this area performing the same services under similar circumstances during the same period. Professional judgments presented herein are primarily based on information from previous reports that were assumed to be accurate, knowledge of the site, and partly on our general experience with dam safety evaluations performed on other dams. No warranty or guarantee, either written or implied, is applicable to this work.

The use of the words "certification" and/or "certify" in this document shall be interpreted and construed as a Statement of Professional Opinion and is not and shall not be interpreted or construed as a guarantee, warranty, or legal opinion.

8 References

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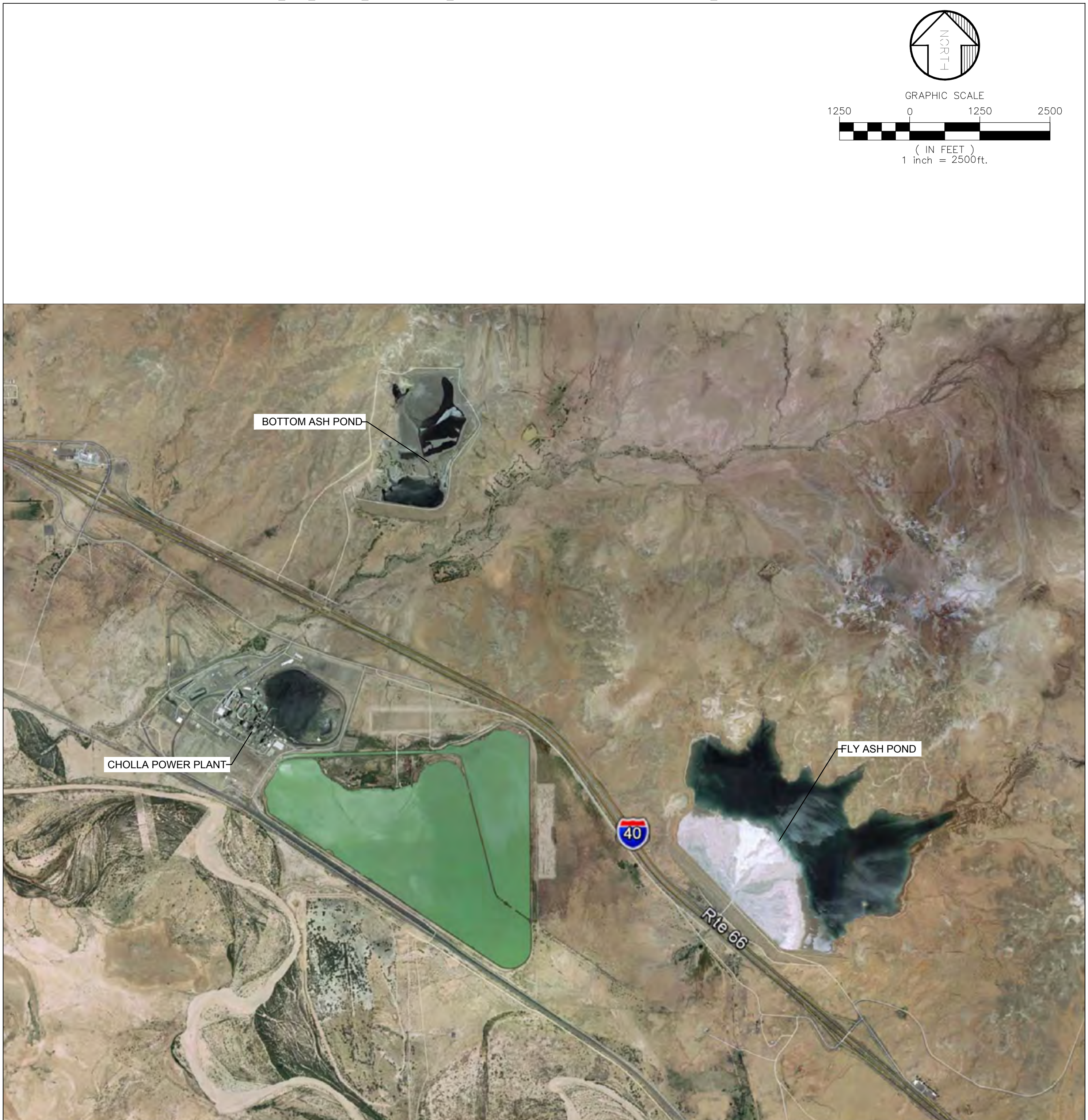
Figures

Last saved by: ALEISA_KRUG(2016-06-16) Last Plotted: 2016-06-16
Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 1-1_SITE VICINITY MAP.DWG



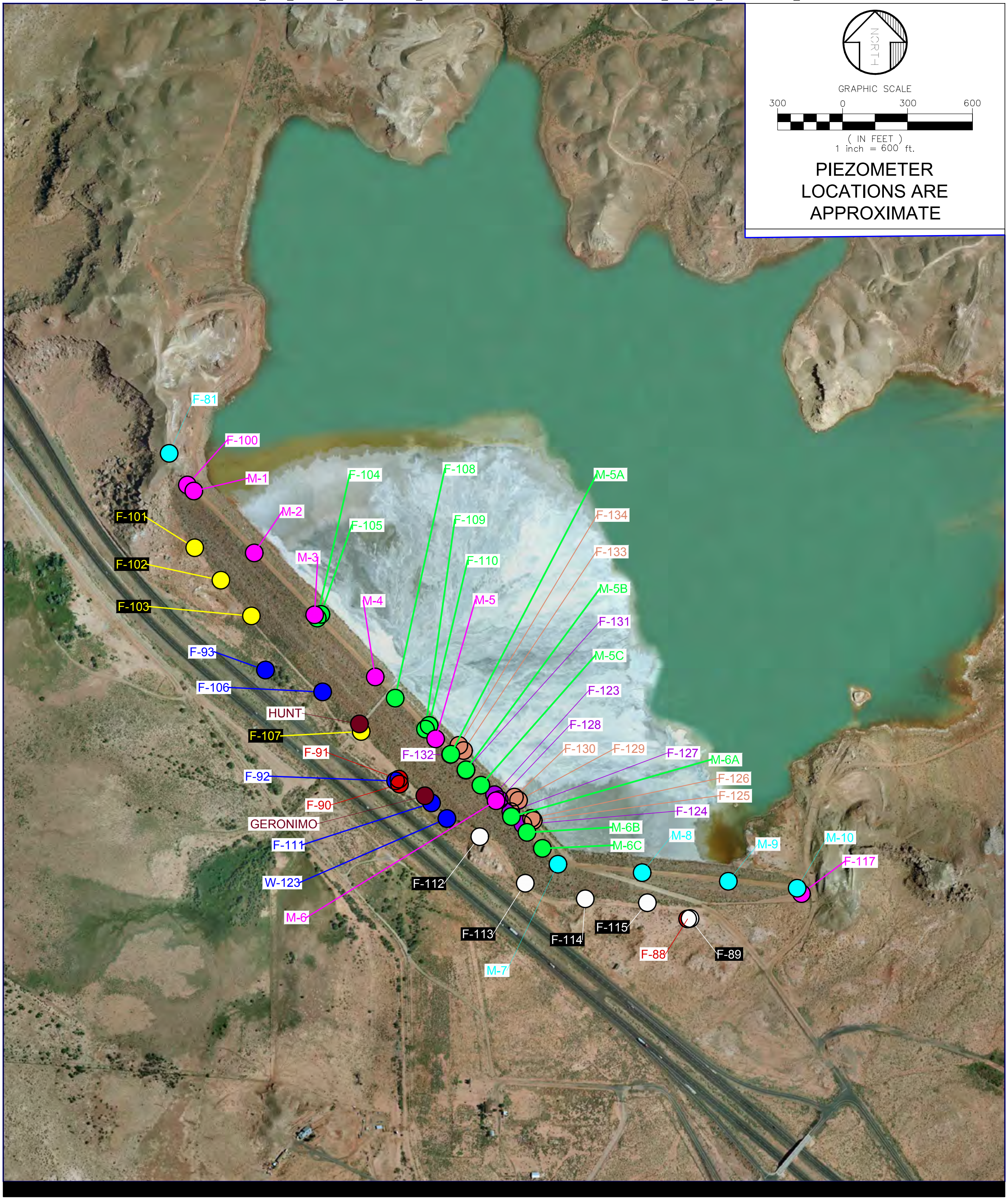
CHOLLA POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445840

SITE VICINITY MAP

AECOM

FIGURE 1-1

Last saved by: KING, JEREMY(2016-06-20) Last Plotted: 2016-06-20
Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 1-2_FLY_ASH_INSTRUMENT_LOCATIONS.DWG



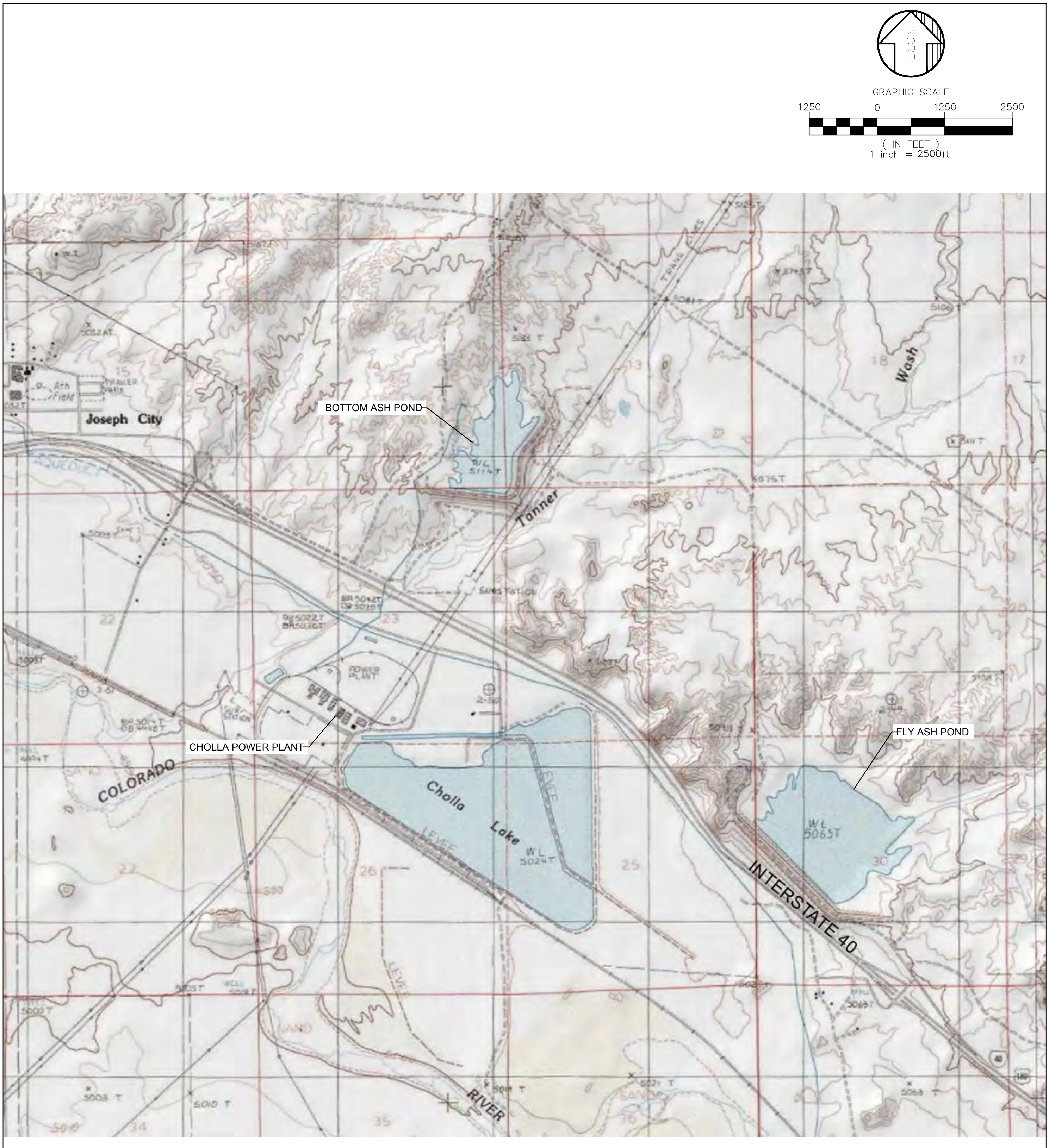
CHOLLA POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445840

FLY ASH POND
MONITORED INSTRUMENTATION
AND SEEPAGE LOCATION MAP

AECOM

FIGURE 1-2

Last saved by: ALEISA_KRUG(2016-06-16) Last Plotted: 2016-06-16
Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 4-1_SITE TOPO MAP.DWG



SOURCE: ARCGIS NATIONAL GEOGRAPHIC WORLD BASEMAP, 2013

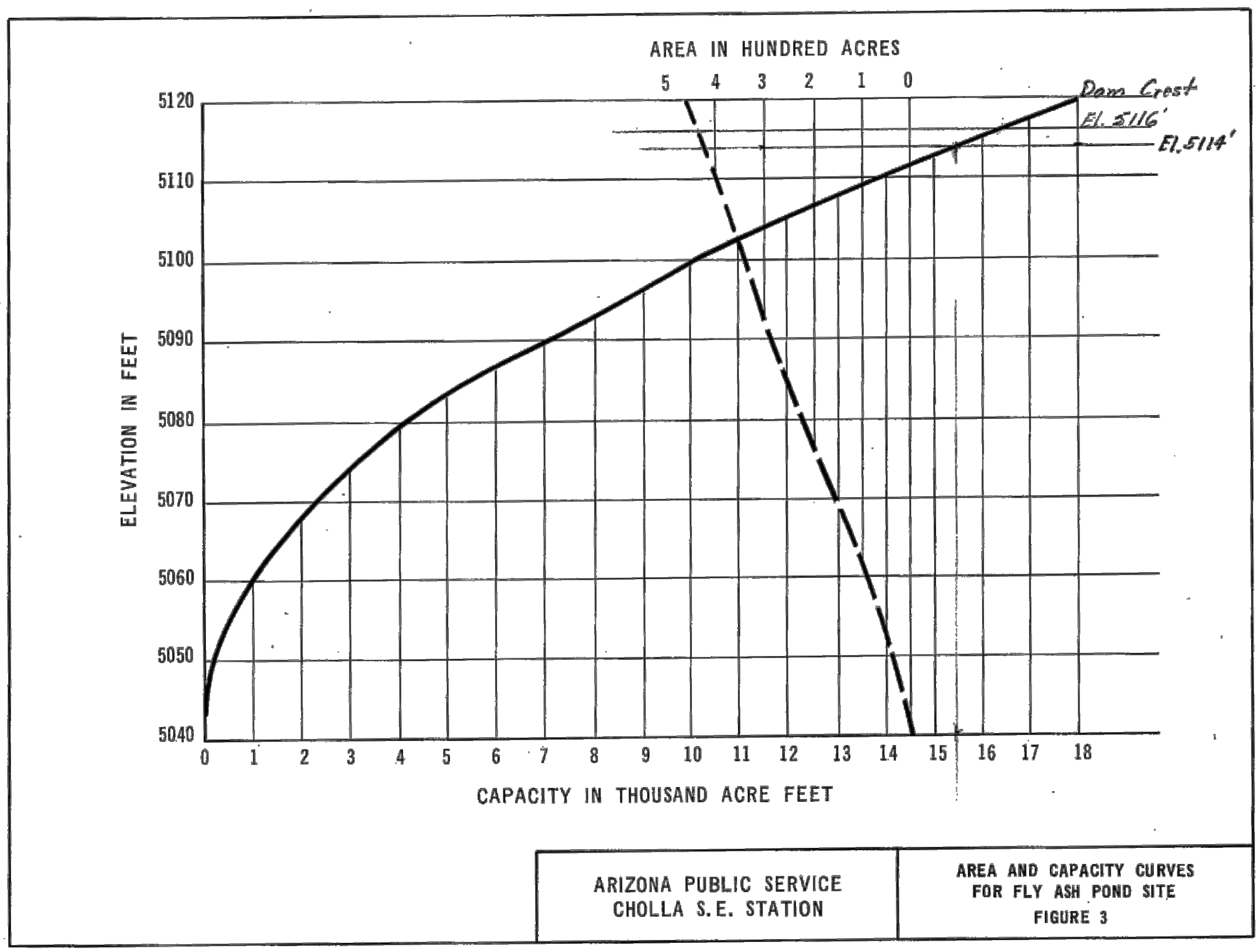
CHOLLA POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445840

SITE TOPOGRAPHIC MAP

AECOM

FIGURE 3-1

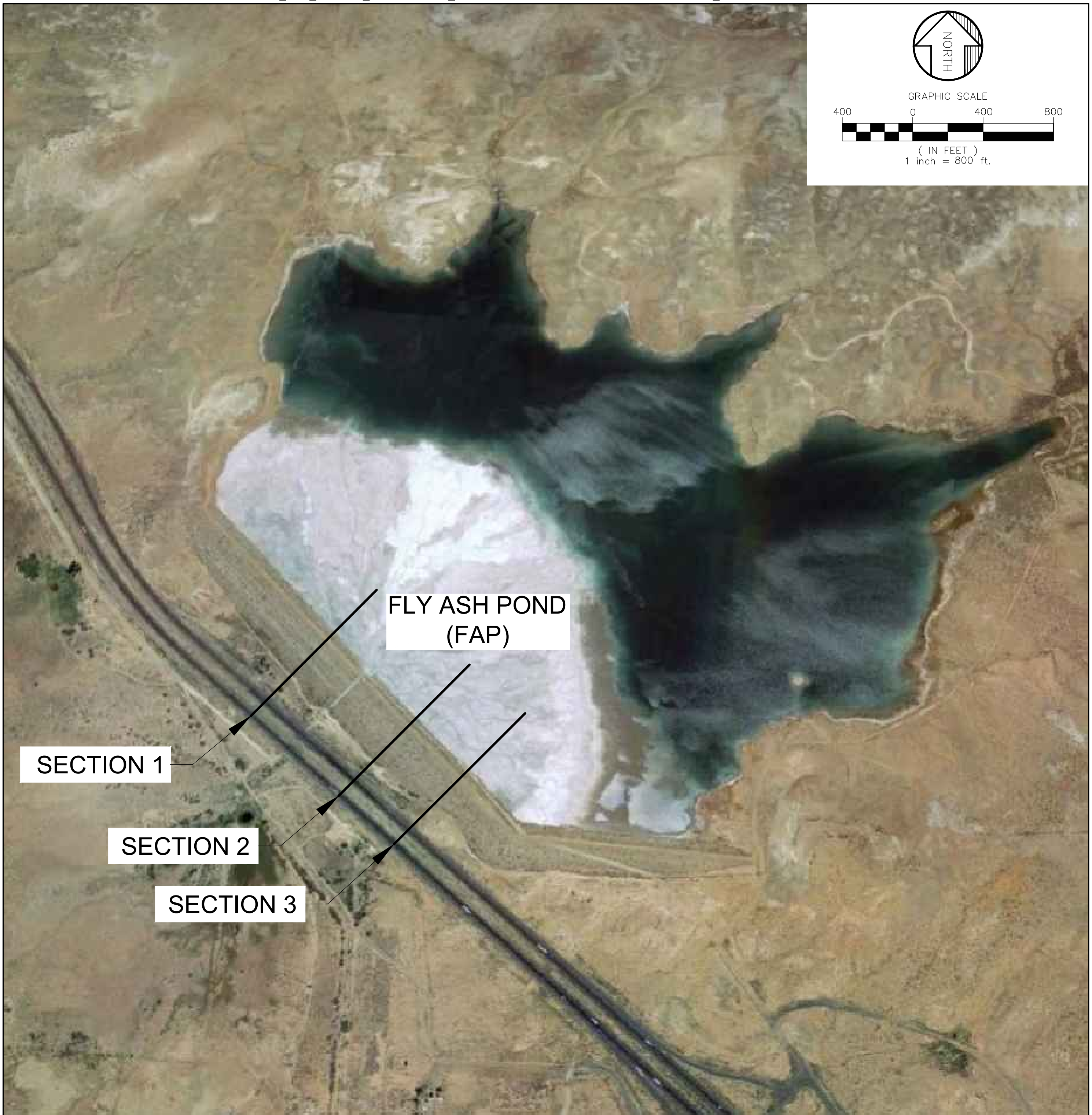
Last saved by: KING, JEREMY(2016-06-20) Last Plotted: 2016-06-20
Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 4-2_AREA CAPACITY CURVE_JSK.DWG



SOURCE:

SEEPAGE AND FOUNDATION STUDIES: VOLUME I OF II ENGINEERING REPORT (EBASCO,1975)

Last saved by: ALEISA_KRUG(2016-08-11) Last Plotted: 2016-08-11
Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 5-1_CROSS SECTION LOCATIONS SFA - FAP.DWG
ANSI A 8.5" x 11"



FOUR CORNERS POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445840

CROSS SECTION LOCATIONS
SAFETY FACTOR ASSESSMENT

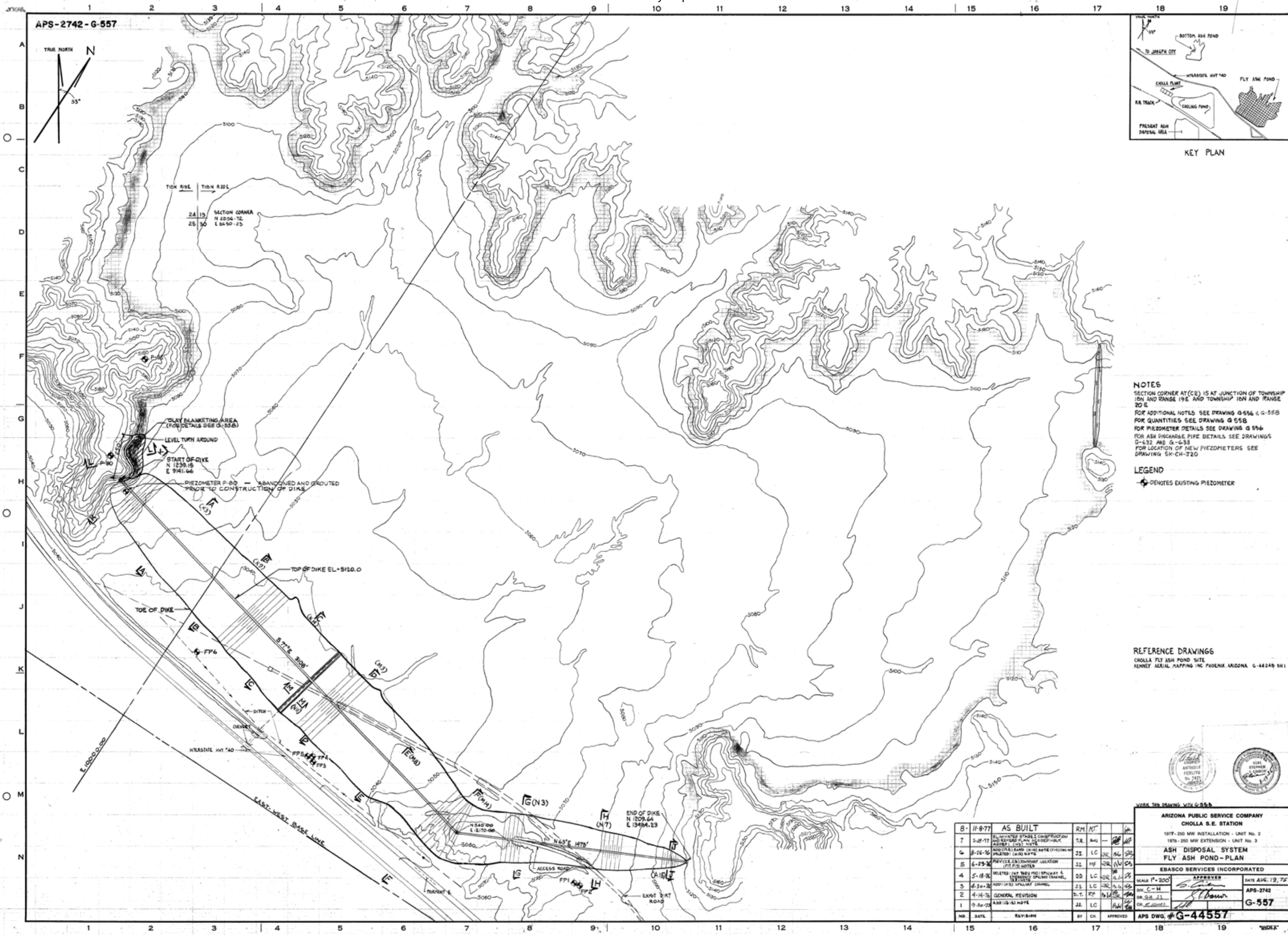
AECOM

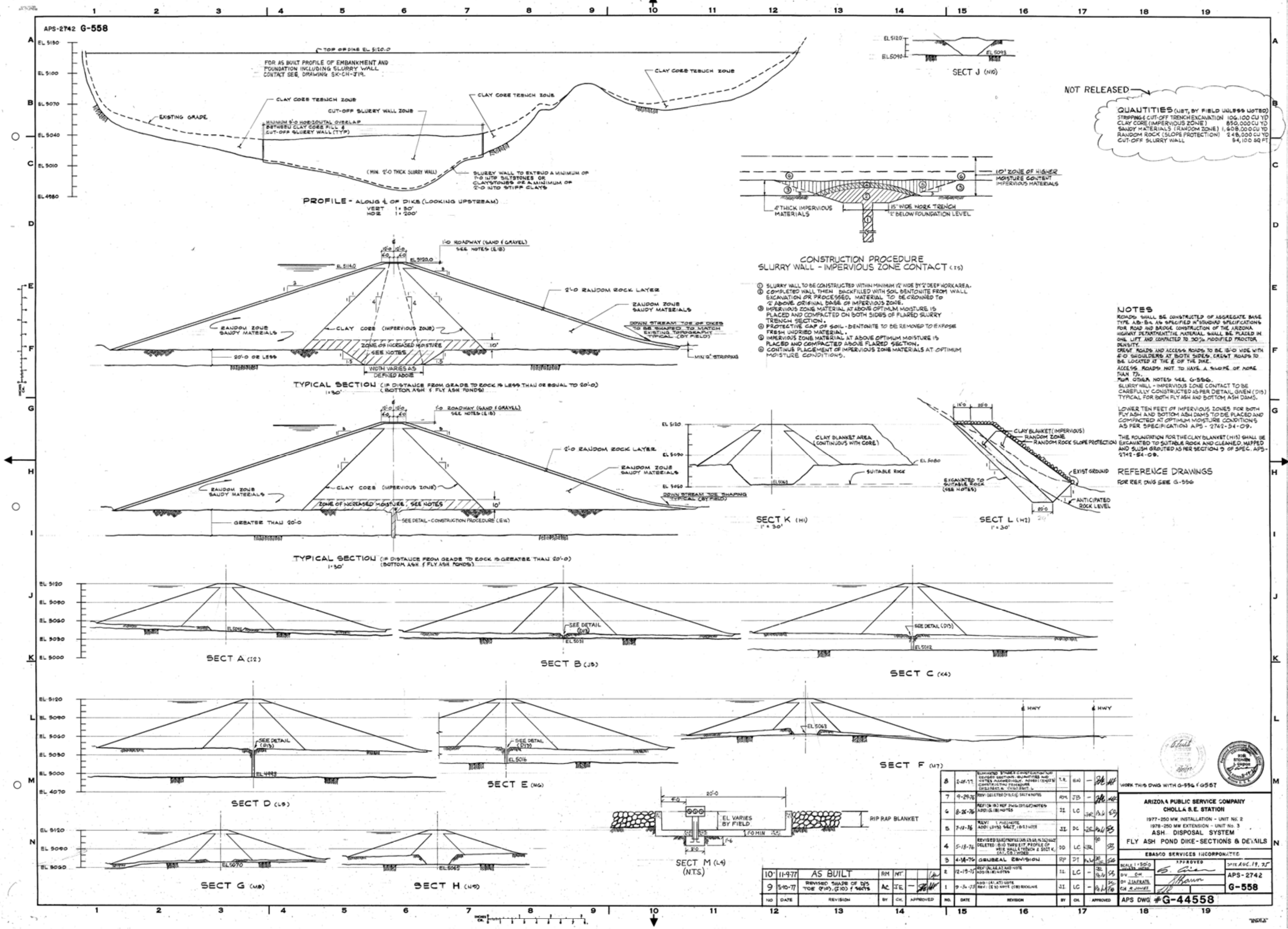
FIGURE 5-1

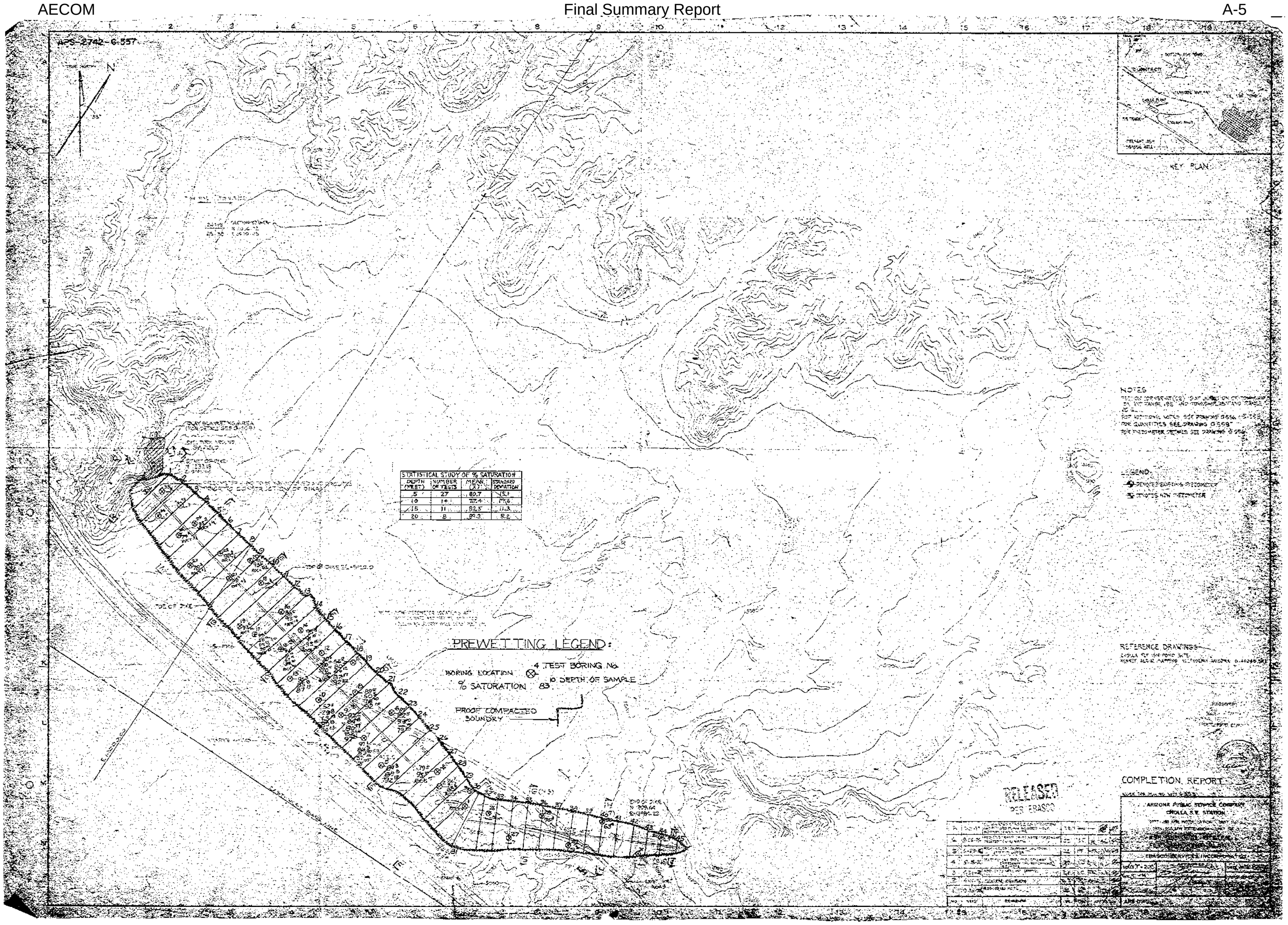
Appendix A.
Historic Drawings

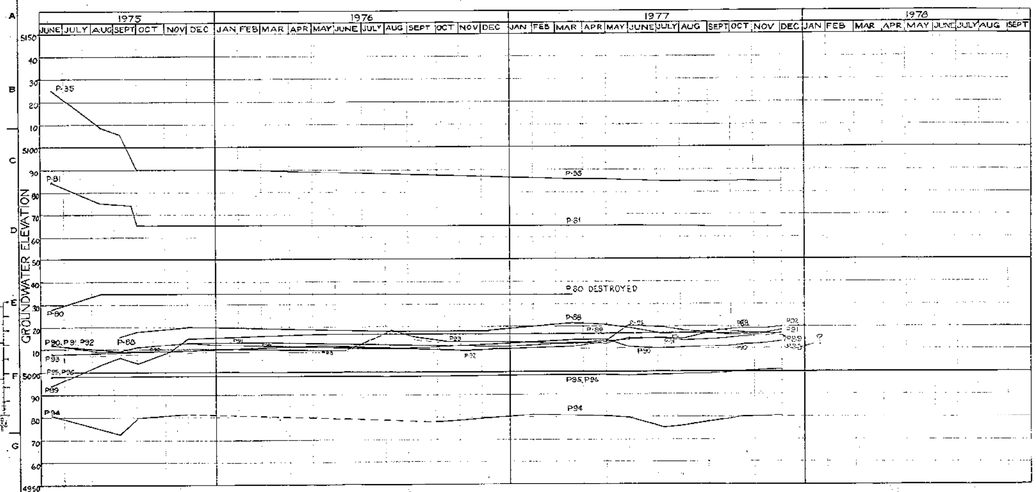
ORIGINAL AS-BUILT DRAWINGS

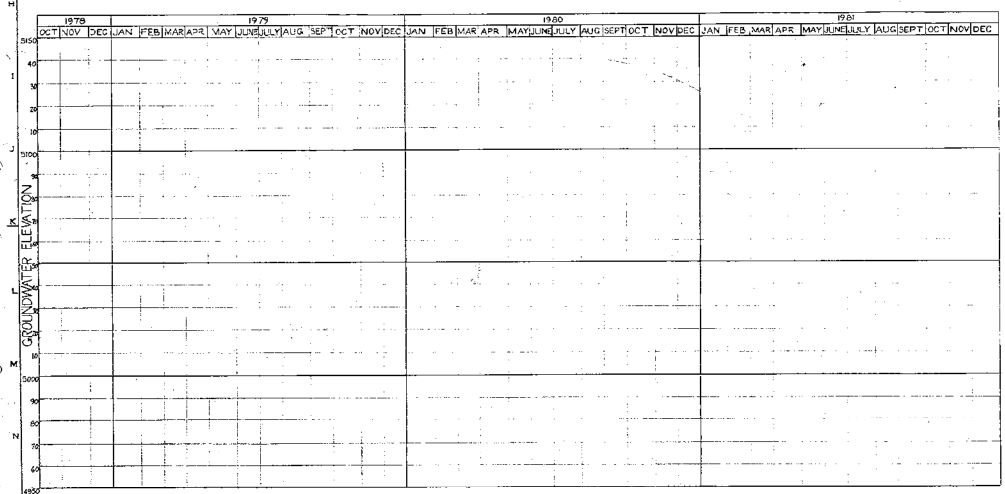
(Ebasco, 1977)

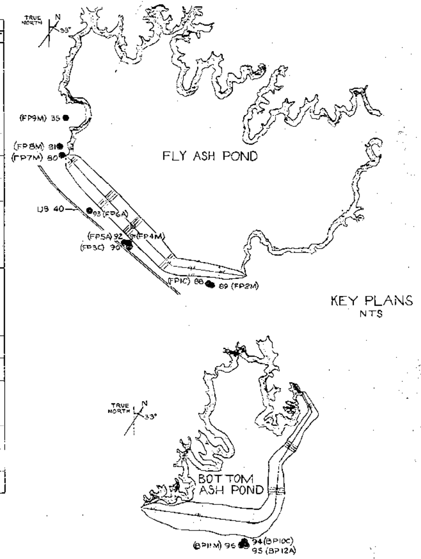

APS-2742-G-557
TRUE NORTH
N
KEY PLAN
BOTTOM ASH POND
TO JOSEPH CITY
INTERSTATE HWY 40
FLY ASH POND
CHOLLA PLANT
COOLING POND
R.R. TRACK
PRESENT ASH DISPOSAL AREA
T19N R19E | T19N R20E
SECTION CORNER N 2056.72 E 8450.25
CLAY BLANKETING AREA (FOR DETAILS SEE G-558)
LEVEL TURN AROUND
START OF DIKE N 1239.16 E 9141.66
PIEZOMETER P-80 — ABANDONED AND GROUTED PRIOR TO CONSTRUCTION OF DIKE
TOP OF DIKE EL=5120.0
TOE OF DIKE
S 77°E 3108'
DITCH
CULVERT
INTERSTATE HWY 40
EAST-WEST BASE LINE
N 63°E 1475'
END OF DIKE N 1209.64 E 13484.23
ACCESS ROAD
EXIST DIRT ROAD
NOTES
SECTION CORNER AT (C2) IS AT JUNCTION OF TOWNSHIP 18N AND RANGE 19E AND TOWNSHIP 18N AND RANGE 20E
FOR ADDITIONAL NOTES SEE DRAWING G-556 & G-558
FOR QUANTITIES SEE DRAWING G 558
FOR PIEZOMETER DETAILS SEE DRAWING G 556
FOR ASH DISCHARGE PIPE DETAILS SEE DRAWINGS G-632 AND G-638
FOR LOCATION OF NEW PIEZOMETERS SEE DRAWING SK-CH-J20
LEGEND
DENOTES EXISTING PIEZOMETER
REFERENCE DRAWINGS
CHOLLA FLY ASH POND SITE
KENNEY AERIAL MAPPING INC PHOENIX ARIZONA G-44248 SH1
8 11-8-77 AS BUILT
WORK THIS DRAWING WITH G-558
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1977 - 250 MW INSTALLATION - UNIT No. 2
1978 - 250 MW EXTENSION - UNIT No. 3
ASH DISPOSAL SYSTEM
FLY ASH POND - PLAN
EBASCO SERVICES INCORPORATED
SCALE 1" = 200'
APPROVED
DATE AUG. 19, 75
DIV. C-H
APS-2742
G-557
APS DWG. #G-44557
"INDEX"


APS-2742 G-558
TOP OF DIKE EL 5120.0
FOR AS BUILT PROFILE OF EMBANKMENT AND FOUNDATION INCLUDING SLURRY WALL CONTACT SEE DRAWING SK-CH-319.
CLAY CORE TRENCH ZONE
CUT-OFF SLURRY WALL ZONE
EXISTING GRADE
(MIN. 2'-0 THICK SLURRY WALL)
PROFILE - ALONG ℄ OF DIKE (LOOKING UPSTREAM)
SECT J
NOT RELEASED
QUANTITIES (UST, BY FIELD UNLESS NOTED)
STRIPPING & CUT-OFF TRENCH EXCAVATION 106,100 CU YD
CLAY CORE (IMPERVIOUS ZONE) 850,000 CU YD
SANDY MATERIALS (RANDOM ZONE) 1,608,000 CU YD
RANDOM ROCK (SLOPE PROTECTION) 248,000 CU YD
CUT-OFF SLURRY WALL 54,100 SQ FT
10' ZONE OF HIGHER MOISTURE CONTENT IMPERVIOUS MATERIALS
4' THICK IMPERVIOUS MATERIALS
15' WIDE WORK TRENCH 2' BELOW FOUNDATION LEVEL
CONSTRUCTION PROCEDURE
SLURRY WALL - IMPERVIOUS ZONE CONTACT (I5)
① SLURRY WALL TO BE CONSTRUCTED WITHIN MINIMUM 12' WIDE BY 2' DEEP WORK AREA.
② COMPLETED WALL THEN BACKFILLED WITH SOIL BENTONITE FROM WALL EXCAVATION OR PROCESSED. MATERIAL TO BE CROWNED TO 2' ABOVE ORIGINAL BASE OF IMPERVIOUS ZONE.
③ IMPERVIOUS ZONE MATERIAL AT ABOVE OPTIMUM MOISTURE IS PLACED AND COMPACTED ON BOTH SIDES OF FLARED SLURRY TRENCH SECTION.
④ PROTECTIVE CAP OF SOIL-BENTONITE TO BE REMOVED TO EXPOSE FRESH UNDRIED MATERIAL.
⑤ IMPERVIOUS ZONE MATERIAL AT ABOVE OPTIMUM MOISTURE IS PLACED AND COMPACTED ABOVE FLARED SECTION.
⑥ CONTINUE PLACEMENT OF IMPERVIOUS ZONE MATERIALS AT OPTIMUM MOISTURE CONDITIONS.
1'-0 ROADWAY (SAND & GRAVEL) SEE NOTES (E18)
2'-0 RANDOM ROCK LAYER
RANDOM ZONE SANDY MATERIALS
CLAY CORE (IMPERVIOUS ZONE)
ZONE OF INCREASED MOISTURE SEE NOTES
DOWN STREAM TOE OF DIKES TO BE SHAPED TO MATCH EXISTING TOPOGRAPHY TYPICAL (BY FIELD)
MIN 2' STRIPPING
20'-0 OR LESS
WIDTH VARIES AS DEFINED ABOVE
TYPICAL SECTION (IF DISTANCE FROM GRADE TO ROCK IS LESS THAN OR EQUAL TO 20'-0) (BOTTOM ASH & FLY ASH PONDS)
NOTES
ROADS SHALL BE CONSTRUCTED OF AGGREGATE BASE TYPE AB-BA AS SPECIFIED IN "STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION OF THE ARIZONA HIGHWAY DEPARTMENT" THE MATERIAL SHALL BE PLACED IN ONE LIFT AND COMPACTED TO 90% MODIFIED PROCTOR DENSITY.
CREST ROADS AND ACCESS ROADS TO BE 15'-0 WIDE WITH 4'-0 SHOULDERS AT BOTH SIDES. CREST ROADS TO BE LOCATED AT THE ℄ OF THE DIKE.
ACCESS ROADS NOT TO HAVE A SLOPE OF MORE THAN 7%.
FOR OTHER NOTES SEE G-556.
SLURRY WALL - IMPERVIOUS ZONE CONTACT TO BE CAREFULLY CONSTRUCTED AS PER DETAIL GIVEN (D15) TYPICAL FOR BOTH FLY ASH AND BOTTOM ASH DAMS.
LOWER TEN FEET OF IMPERVIOUS ZONES FOR BOTH FLY ASH AND BOTTOM ASH DAMS TO BE PLACED AND COMPACTED AT OPTIMUM MOISTURE CONDITIONS AS PER SPECIFICATION APS-2742-54-09.
THE FOUNDATION FOR THE CLAY BLANKET (H15) SHALL BE EXCAVATED TO SUITABLE ROCK AND CLEANED, MAPPED AND SLUSH GROUTED AS PER SECTION 9 OF SPEC. APS-2742-54-09.
REFERENCE DRAWINGS
FOR REF. DWG SEE G-556
CLAY BLANKET AREA (CONTINUOUS WITH CORE)
SUITABLE ROCK
SECT K (H1)
CLAY BLANKET (IMPERVIOUS)
RANDOM ZONE
RANDOM ROCK SLOPE PROTECTION
EXIST GROUND
EXCAVATED TO SUITABLE ROCK (SEE NOTES)
ANTICIPATED ROCK LEVEL
SECT L (H2)
GREATER THAN 20'-0
SEE DETAIL - CONSTRUCTION PROCEDURE (E14)
TYPICAL SECTION (IF DISTANCE FROM GRADE TO ROCK IS GREATER THAN 20'-0) (BOTTOM ASH & FLY ASH PONDS)
SECT A (I2)
SECT B (J8)
SECT C (K4)
SECT D (L5)
SECT E (M6)
SECT F (M7)
SECT G (M8)
SECT H (N9)
EL VARIES BY FIELD
RIP RAP BLANKET
SECT M (L4) (N.T.S.)
10 11-9-77 AS BUILT
9 5-10-77 REVISED SHAPE OF D/S TOE (F10), (I10) & SECTS
3 4-14-76 GENERAL REVISION
WORK THIS DWG WITH G-556 & G557
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1977-250 MW INSTALLATION - UNIT NO. 2
1978-250 MW EXTENSION - UNIT No. 3
ASH DISPOSAL SYSTEM
FLY ASH POND DIKE - SECTIONS & DETAILS
EBASCO SERVICES INCORPORATED
APPROVED
DATE AUG. 19, 75
APS-2742
G-558
APS DWG #G-44558


APS-2742-G-557
N
KEY PLAN
NOTES:
LEGEND:
STATISTICAL STUDY OF % SATURATION
DEPTH (FEET) | NUMBER OF TESTS | MEAN (X) % | STANDARD DEVIATION
5 | 27 | 80.7 | 5.1
10 | 14 | 77.4 | 17.6
15 | 11 | 82.5 | 11.3
20 | 8 | 89.3 | 8.2
PREWETTING LEGEND:
BORING LOCATION
4 TEST BORING No.
% SATURATION 83
10 DEPTH OF SAMPLE
PROOF COMPACTED BOUNDRY
REFERENCE DRAWINGS
RELEASED
PER EBASCO
COMPLETION REPORT
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
EBASCO SERVICES INCORPORATED







PIEZOMETER LEGEND:

F – FLY ASH
B – BOTTOM ASH
P – PIEZOMETER
C – COCONINO
M – MOENKOPI
A – ALLUVIUM

INSTALLATION DATA 6-75

PIEZOMETER NUMBER	DRILLED DEPTH	DEPTH TO SCREEN	TOP OF RISER PIPE ELEVATION	RISER HEIGHT
35 (FP9M)	132	32	5191.3	3.2
80 (FP7M)	58	43	5083.0	1.7
81 (FP8M)	112	32	5164.0	1.5
88 (FP1C)	191	175	5086.5	2.0
89 (FP2M)	92	52	5085.9	1.8
90 (FP3C)	146	133	5036.7	3.0
91 (FP4M)	77	57	5035.1	1.5
92 (FP5A)	56	41	5034.9	1.5
93 (FP6A)	26	16	5034.2	2.0
94 (BP10C)	292	277	5050.7	3.0
95 (BP12A)	102	87	5049.3	2.4
96 (BP11M)	154	114	5049.9	3.1

NOTES:

COMPLETION REPORT

ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1976–250 MW INSTALLATION – UNIT No. 2
1977–250 MW EXTENSION–UNIT No 3

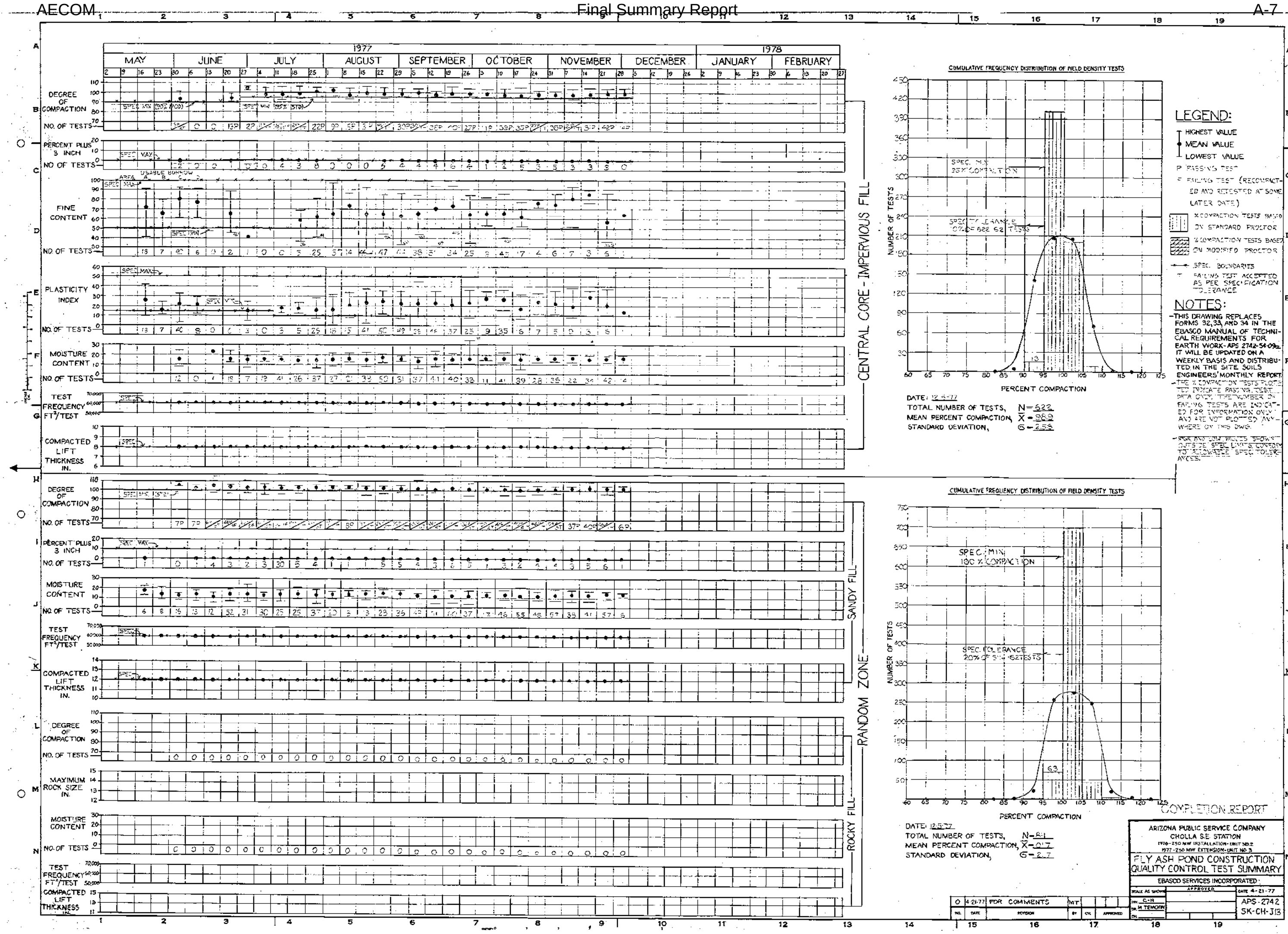
ASH POND PIEZOMETERS
GROUNDWATER ELEVATIONS

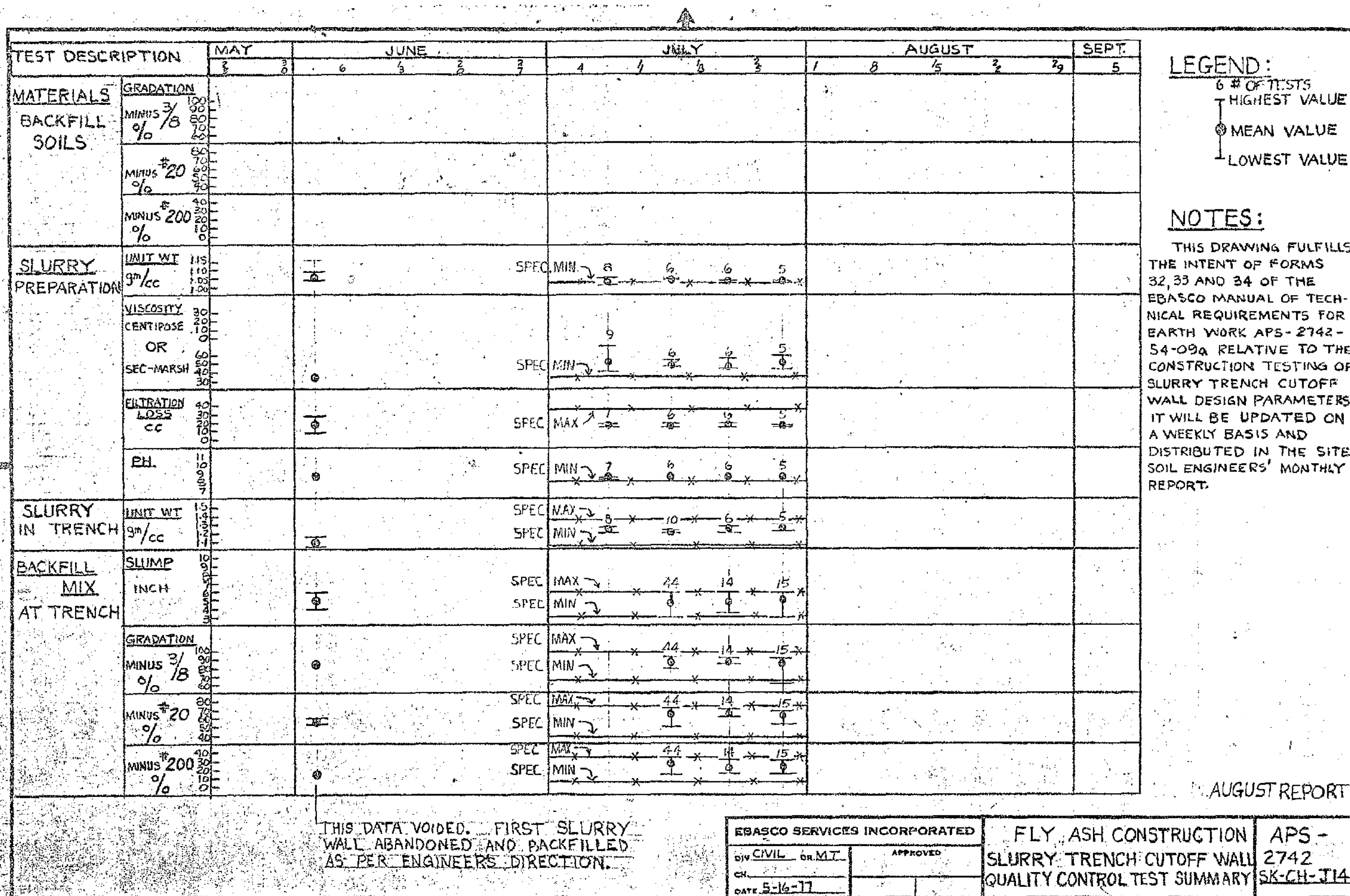
EBASCO SERVICES INCORPORATED

SCALE: AS SHOWN APPROVED DATE: 3-28-77
DR. C-H
CK. M. TEMCHIN

APS-2742
SK-CH-J11

0 3-28-77 FOR COMMENTS MT


1977
1978
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
JANUARY
FEBRUARY
DEGREE OF COMPACTION
NO. OF TESTS
PERCENT PLUS 3 INCH
FINE CONTENT
PLASTICITY INDEX
MOISTURE CONTENT
TEST FREQUENCY FT³/TEST
COMPACTED LIFT THICKNESS IN.
MAXIMUM ROCK SIZE IN.
CENTRAL CORE - IMPERVIOUS FILL
SANDY FILL
RANDOM ZONE
ROCKY FILL
CUMULATIVE FREQUENCY DISTRIBUTION OF FIELD DENSITY TESTS
NUMBER OF TESTS
PERCENT COMPACTION
DATE: 12-5-77
TOTAL NUMBER OF TESTS,
MEAN PERCENT COMPACTION,
STANDARD DEVIATION,
LEGEND:
HIGHEST VALUE
MEAN VALUE
LOWEST VALUE
NOTES:
THIS DRAWING REPLACES FORMS 32, 33, AND 34 IN THE EBASCO MANUAL OF TECHNICAL REQUIREMENTS FOR EARTH WORK-APS 2742-54-09. IT WILL BE UPDATED ON A WEEKLY BASIS AND DISTRIBUTED IN THE SITE SOILS ENGINEERS' MONTHLY REPORT.
COMPLETION REPORT
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1976-250 MW INSTALLATION-UNIT NO. 2
1977-250 MW EXTENSION-UNIT NO. 3
FLY ASH POND CONSTRUCTION
QUALITY CONTROL TEST SUMMARY
EBASCO SERVICES INCORPORATED
DATE 4-21-77
APS-2742
SK-CH-313
0 4-21-77 FOR COMMENTS MT
NO.
DATE
REVISION
APPROVED


TEST DESCRIPTION
MAY
JUNE
JULY
AUGUST
SEPT.
MATERIALS BACKFILL SOILS
GRADATION MINUS 3/8 %
MINUS #20 %
MINUS #200 %
SLURRY PREPARATION
UNIT WT gm/cc
VISCOSITY CENTIPOSE OR SEC-MARSH
FILTRATION LOSS cc
PH.
SLURRY IN TRENCH
UNIT WT gm/cc
BACKFILL MIX AT TRENCH
SLUMP INCH
GRADATION MINUS 3/8 %
MINUS #20 %
MINUS #200 %
SPEC MIN
SPEC MAX
LEGEND:
6 # OF TESTS
HIGHEST VALUE
MEAN VALUE
LOWEST VALUE
NOTES:
THIS DRAWING FULFILLS THE INTENT OF FORMS 32, 33 AND 34 OF THE EBASCO MANUAL OF TECHNICAL REQUIREMENTS FOR EARTH WORK APS-2742-54-09a RELATIVE TO THE CONSTRUCTION TESTING OF SLURRY TRENCH CUTOFF WALL DESIGN PARAMETERS. IT WILL BE UPDATED ON A WEEKLY BASIS AND DISTRIBUTED IN THE SITE SOIL ENGINEERS' MONTHLY REPORT.
AUGUST REPORT
THIS DATA VOIDED. FIRST SLURRY WALL ABANDONED AND BACKFILLED AS PER ENGINEERS DIRECTION.
EBASCO SERVICES INCORPORATED
DIV CIVIL DR. M.T.
APPROVED
DATE 5-16-77
FLY ASH CONSTRUCTION
SLURRY TRENCH CUTOFF WALL
QUALITY CONTROL TEST SUMMARY
APS-2742
SK-CH-J14

ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1976-250 MW INSTALLATION - UNIT No. 2
1977-250 MW EXTENSION - UNIT No. 3

ASH PONDS BORROW AREA MATERIAL PLAN

EBASCO SERVICES INCORPORATED

SCALE 1" = 400'-0 | APPROVED | DATE

DIV. C-H

DR. D. KOVACHEV

CH.

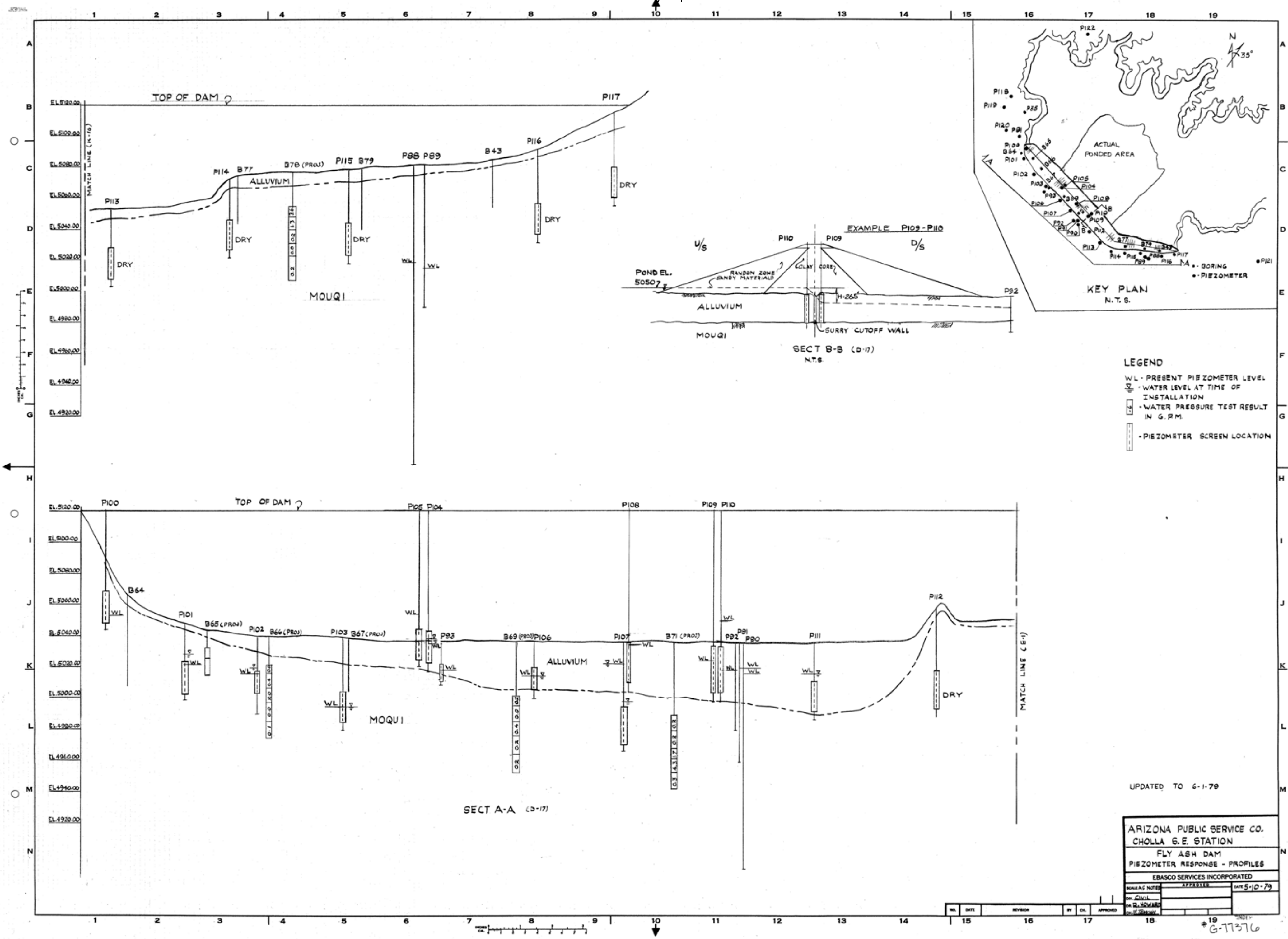
APS-2742
SK-CH-160

NO.	DATE	REVISION	BY	CH.	APPROVED

APS DWG. G44254 P2

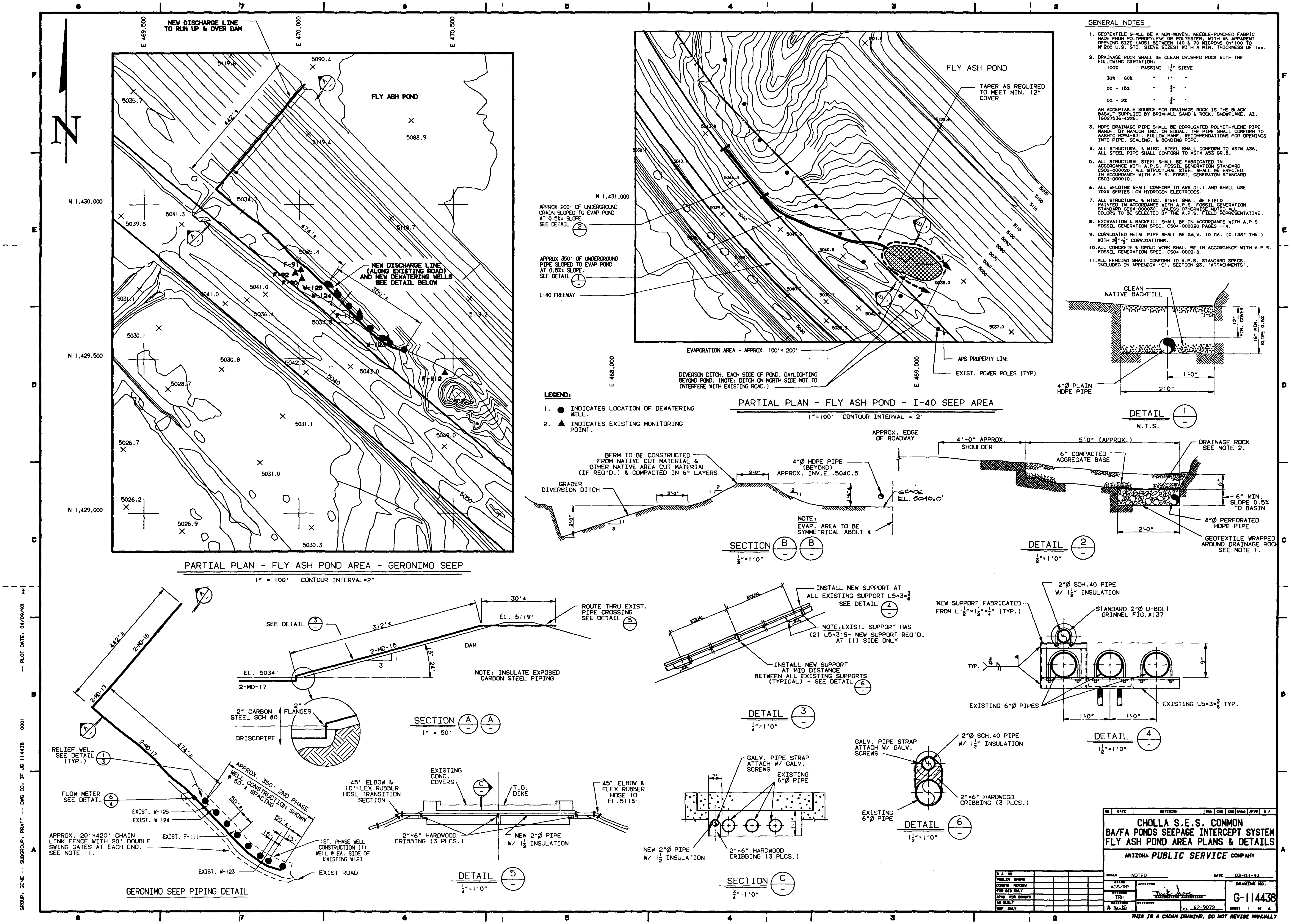
09.27
09.28

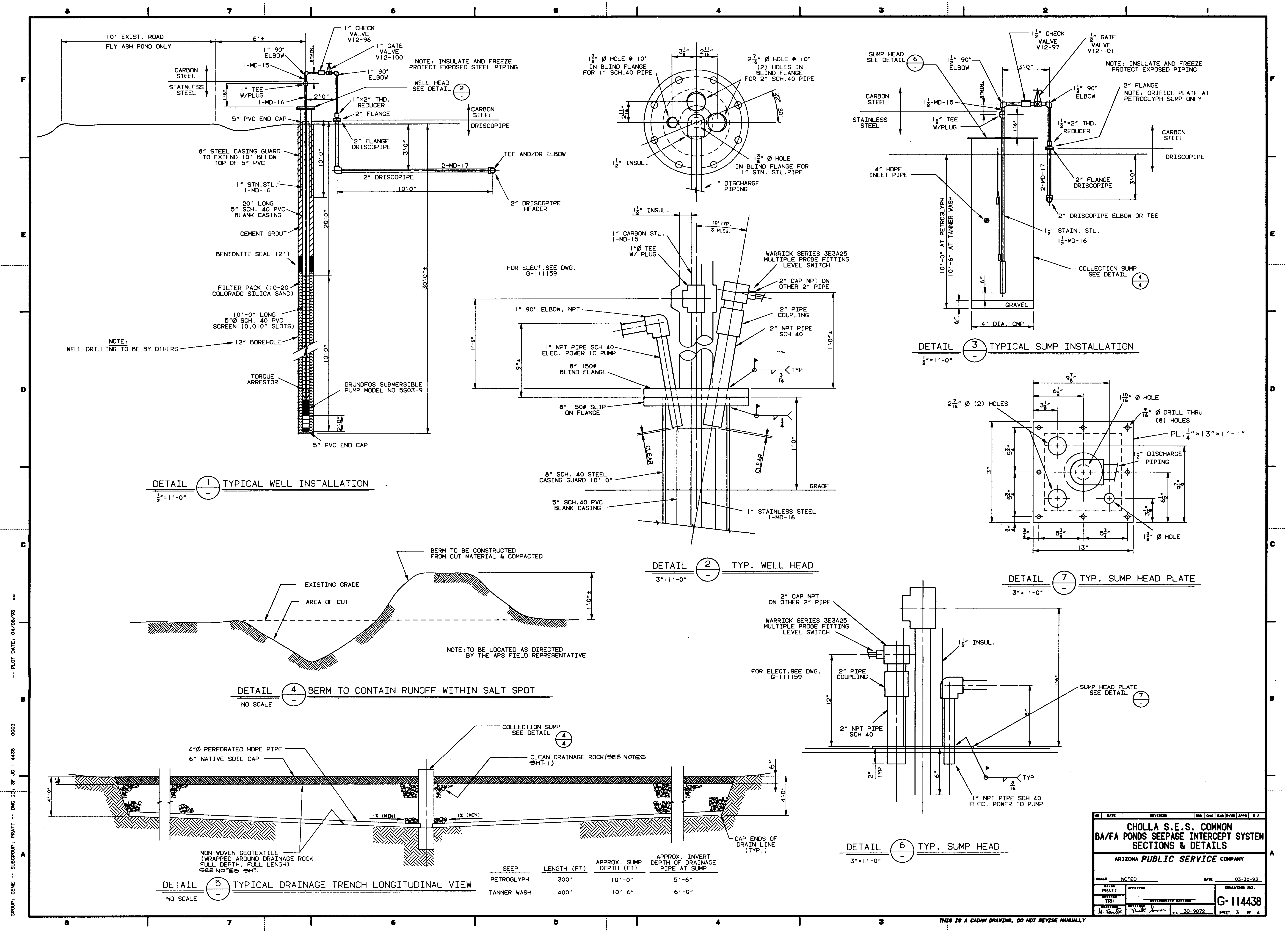
"INDEX"


TOP OF DAM
ALLUVIUM
MOQUI
DRY
MATCH LINE (K-16)
EXAMPLE P109-P110
U/S
D/S
POND EL. 5050
RANDOM ZONE SANDY MATERIAL
CLAY CORE
ALLUVIUM
MOQUI
SURRY CUTOFF WALL
SECT B-B (D-17)
N.T.S.
ACTUAL PONDED AREA
KEY PLAN
N.T.S.
• - BORING
• - PIEZOMETER
LEGEND
WL - PRESENT PIEZOMETER LEVEL
- WATER LEVEL AT TIME OF INSTALLATION
- WATER PRESSURE TEST RESULT IN G.P.M.
- PIEZOMETER SCREEN LOCATION
SECT A-A (D-17)
MATCH LINE (E-1)
UPDATED TO 6-1-79
ARIZONA PUBLIC SERVICE CO.
CHOLLA S.E. STATION
FLY ASH DAM
PIEZOMETER RESPONSE - PROFILES
EBASCO SERVICES INCORPORATED
5-10-79
#G-77376

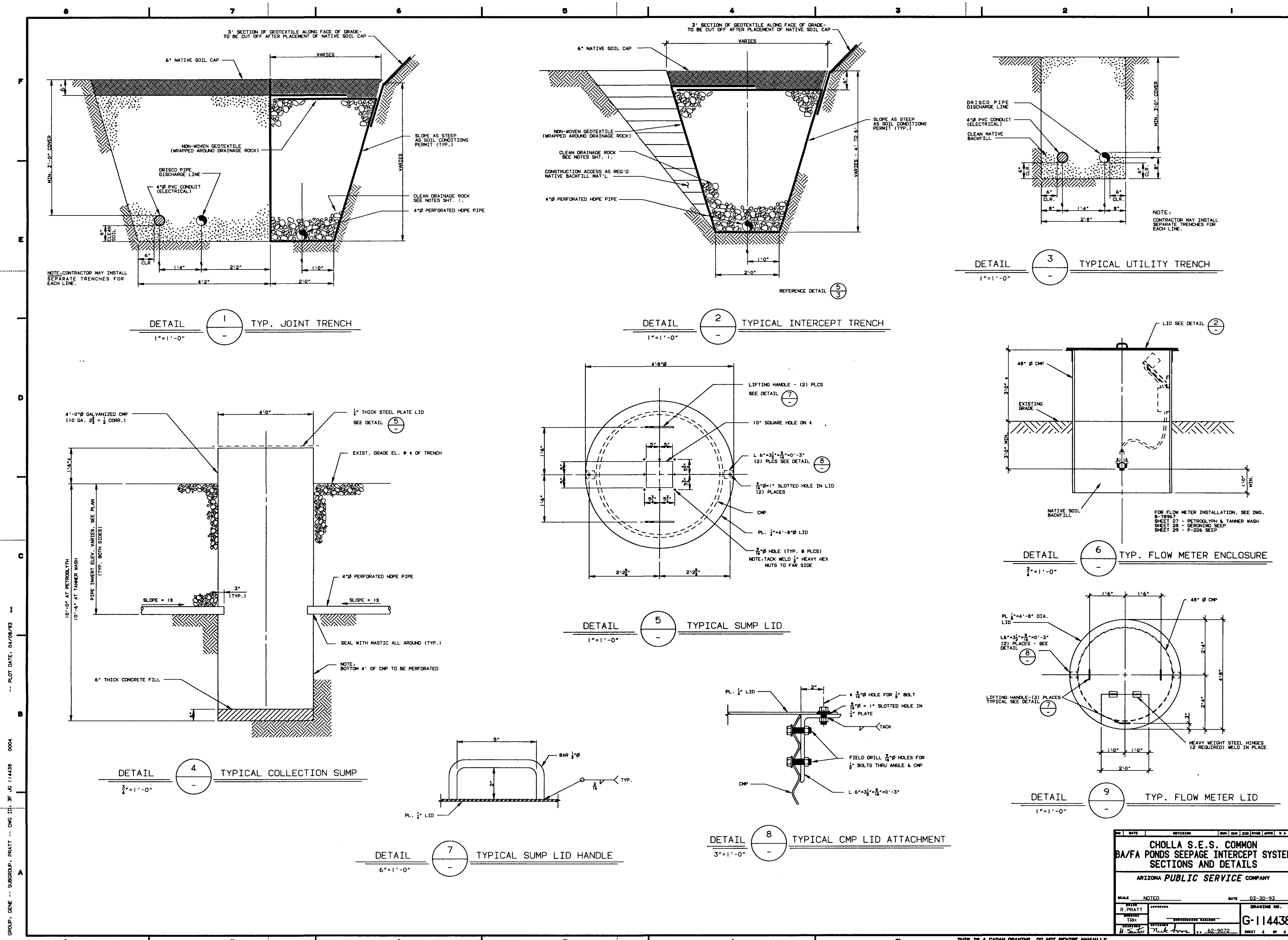
SEEPAGE INTERCEPT SYSTEM DRAWINGS

(APS, 1993)


PARTIAL PLAN - FLY ASH POND AREA - GERONIMO SEEP
1" = 100' CONTOUR INTERVAL=2"
NEW DISCHARGE LINE TO RUN UP & OVER DAM
FLY ASH POND
NEW DISCHARGE LINE (ALONG EXISTING ROAD) AND NEW DEWATERING WELLS SEE DETAIL BELOW
PARTIAL PLAN - FLY ASH POND - I-40 SEEP AREA
1"=100' CONTOUR INTERVAL = 2'
TAPER AS REQUIRED TO MEET MIN. 12" COVER
APPROX 200' OF UNDERGROUND DRAIN SLOPED TO EVAP POND AT 0.5%± SLOPE. SEE DETAIL 2
APPROX 350' OF UNDERGROUND PIPE SLOPED TO EVAP POND AT 0.5%± SLOPE. SEE DETAIL 1
I-40 FREEWAY
EVAPORATION AREA - APPROX. 100'× 200'
DIVERSON DITCH, EACH SIDE OF POND, DAYLIGHTING BEYOND POND. (NOTE: DITCH ON NORTH SIDE NOT TO INTERFERE WITH EXISTING ROAD.)
APS PROPERTY LINE
EXIST. POWER POLES (TYP)
LEGEND:
1. INDICATES LOCATION OF DEWATERING WELL.
2. INDICATES EXISTING MONITORING POINT.
GENERAL NOTES
1. GEOTEXTILE SHALL BE A NON-WOVEN, NEEDLE-PUNCHED FABRIC MADE FROM POLYPROPYLENE OR POLYESTER, WITH AN APPARENT OPENING SIZE (AOS) BETWEEN 140 & 70 MICRONS (N° 100 TO N° 200 U.S. STD. SIEVE SIZES) WITH A MIN. THICKNESS OF 1mm.
2. DRAINAGE ROCK SHALL BE CLEAN CRUSHED ROCK WITH THE FOLLOWING GRADATION.
AN ACCEPTABLE SOURCE FOR DRAINAGE ROCK IS THE BLACK BASALT SUPPLIED BY BRIMHALL SAND & ROCK, SNOWFLAKE, AZ. (602)536-4226.
3. HDPE DRAINAGE PIPE SHALL BE CORRUGATED POLYETHYLENE PIPE MANUF. BY HANCOR INC. OR EQUAL. THE PIPE SHALL CONFORM TO AASHTO M294-83I. FOLLOW MANF. RECOMMENDATIONS FOR OPENINGS INTO PIPE, SEALING, & BENDING PIPE.
4. ALL STRUCTURAL & MISC. STEEL SHALL CONFORM TO ASTM A36. ALL STEEL PIPE SHALL CONFORM TO ASTM A53 GR.B.
5. ALL STRUCTURAL STEEL SHALL BE FABRICATED IN ACCORDANCE WITH A.P.S. FOSSIL GENERATION STANDARD CS02-000020. ALL STRUCTURAL STEEL SHALL BE ERECTED IN ACCORDANCE WITH A.P.S. FOSSIL GENERATION STANDARD CS03-000010.
6. ALL WELDING SHALL CONFORM TO AWS D1.1 AND SHALL USE 70XX SERIES LOW HYDROGEN ELECTRODES.
7. ALL STRUCTURAL & MISC. STEEL SHALL BE FIELD PAINTED IN ACCORDANCE WITH A.P.S. FOSSIL GENERATION STANDARD GE04-000030, UNLESS OTHERWISE NOTED ALL COLORS TO BE SELECTED BY THE A.P.S. FIELD REPRESENTATIVE.
8. EXCAVATION & BACKFILL SHALL BE IN ACCORDANCE WITH A.P.S. FOSSIL GENERATION SPEC. CS04-000020 PAGES 1-4.
9. CORRUGATED METAL PIPE SHALL BE GALV. 10 GA. (0.138" THK.) CORRUGATIONS.
10. ALL CONCRETE & GROUT WORK SHALL BE IN ACCORDANCE WITH A.P.S. FOSSIL GENERATION SPEC. CS04-000010.
11. ALL FENCING SHALL CONFORM TO A.P.S. STANDARD SPECS. INCLUDED IN APPENDIX 'C', SECTION 23, 'ATTACHMENTS'.
CLEAN NATIVE BACKFILL
4"Ø PLAIN HDPE PIPE
DETAIL 1
N.T.S.
SECTION B
DETAIL 2
DRAINAGE ROCK SEE NOTE 2.
6" COMPACTED AGGREGATE BASE
4"Ø PERFORATED HDPE PIPE
GEOTEXTILE WRAPPED AROUND DRAINAGE ROCK SEE NOTE 1.
BERM TO BE CONSTRUCTED FROM NATIVE CUT MATERIAL & OTHER NATIVE AREA CUT MATERIAL (IF REQ'D.) & COMPACTED IN 6" LAYERS
GRADER DIVERSION DITCH
4"Ø HDPE PIPE (BEYOND) APPROX. INV.EL.5040.5
NOTE: EVAP. AREA TO BE SYMMETRICAL ABOUT ℄
GERONIMO SEEP PIPING DETAIL
SECTION A
NOTE: INSULATE EXPOSED CARBON STEEL PIPING
ROUTE THRU EXIST. PIPE CROSSING SEE DETAIL 5
DETAIL 3
DETAIL 4
DETAIL 5
DETAIL 6
SECTION C
CHOLLA S.E.S. COMMON BA/FA PONDS SEEPAGE INTERCEPT SYSTEM FLY ASH POND AREA PLANS & DETAILS
ARIZONA PUBLIC SERVICE COMPANY
DATE 03-03-93
G-114438
THIS IS A CADAN DRAWING. DO NOT REVISE MANUALLY

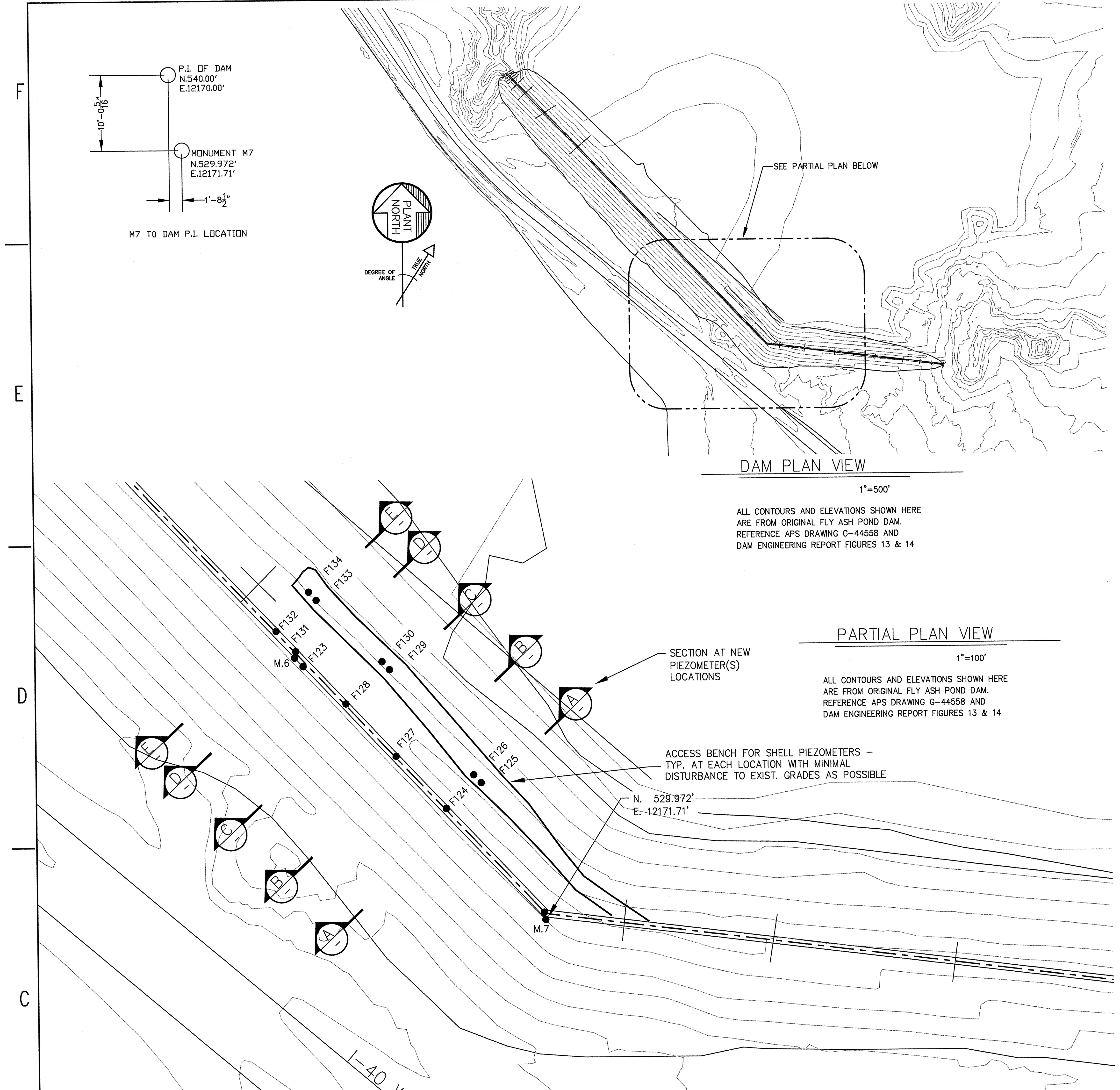


SEEP	LENGTH (FT)	APPROX. SUMP DEPTH (FT)	APPROX. INVERT DEPTH OF DRAINAGE PIPE AT SUMP
PETROGLYPH	300'	10'-0"	5'-6"
TANNER WASH	400'	10'-6"	6'-0"


3' SECTION OF GEOTEXTILE ALONG FACE OF GRADE-TO BE CUT OFF AFTER PLACEMENT OF NATIVE SOIL CAP
6" NATIVE SOIL CAP
VARIES
NON-WOVEN GEOTEXTILE (WRAPPED AROUND DRAINAGE ROCK)
SLOPE AS STEEP AS SOIL CONDITIONS PERMIT (TYP.)
DRISCO PIPE DISCHARGE LINE
4"Ø PVC CONDUIT (ELECTRICAL)
CLEAN DRAINAGE ROCK SEE NOTES SHT. 1.
4"Ø PERFORATED HDPE PIPE
MIN. 3'-0" COVER
6" CLEAN SOIL
NOTE: CONTRACTOR MAY INSTALL SEPARATE TRENCHES FOR EACH LINE.
DETAIL 1 TYP. JOINT TRENCH
1"=1'-0"
CLEAN DRAINAGE ROCK SEE NOTES SHT. 1.
CONSTRUCTION ACCESS AS REQ'D NATIVE BACKFILL MAT'L
VARIES - 4' TO 6'
REFERENCE DETAIL 5/3
DETAIL 2 TYPICAL INTERCEPT TRENCH
1"=1'-0"
CLEAN NATIVE BACKFILL
MIN. 3'-0" COVER
NOTE: CONTRACTOR MAY INSTALL SEPARATE TRENCHES FOR EACH LINE.
DETAIL 3 TYPICAL UTILITY TRENCH
1"=1'-0"
4'-0"Ø GALVANIZED CMP (10 GA. 2⅔ × ½ CORR.)
¼" THICK STEEL PLATE LID SEE DETAIL 5
EXIST. GRADE EL. @ ℄ OF TRENCH
PIPE INVERT ELEV. VARIES. SEE PLAN (TYP. BOTH SIDES)
10'-0" AT PETROGLYPH
10'-6" AT TANNER WASH
4"Ø PERFORATED HDPE PIPE
SLOPE = 1%
SEAL WITH MASTIC ALL AROUND (TYP.)
NOTE: BOTTOM 4' OF CMP TO BE PERFORATED
6" THICK CONCRETE FILL
DETAIL 4 TYPICAL COLLECTION SUMP
¾"=1'-0"
LIFTING HANDLE - (2) PLCS SEE DETAIL 7
10" SQUARE HOLE ON ℄
L 6"×3½"×5/16"×0'-3" (2) PLCS SEE DETAIL 8
9/16"Ø×1" SLOTTED HOLE IN LID (2) PLACES
CMP
PL. ¼"×4'-8"Ø LID
9/16"Ø HOLE (TYP. 8 PLCS)
NOTE: TACK WELD ½" HEAVY HEX NUTS TO FAR SIDE
DETAIL 5 TYPICAL SUMP LID
1"=1'-0"
BAR ½"Ø
PL. ¼" LID
DETAIL 7 TYPICAL SUMP LID HANDLE
6"=1'-0"
PL. ¼" LID
9/16"Ø HOLE FOR ½" BOLT
9/16"Ø × 1" SLOTTED HOLE IN ¼" PLATE
TACK
FIELD DRILL 9/16"Ø HOLES FOR ½" BOLTS THRU ANGLE & CMP
L 6"×3½"×5/16"×0'-3"
DETAIL 8 TYPICAL CMP LID ATTACHMENT
3"=1'-0"
LID SEE DETAIL 2
48" Ø CMP
EXISTING GRADE
NATIVE SOIL BACKFILL
FOR FLOW METER INSTALLATION, SEE DWG. 8-78967
SHEET 27 - PETROGLYPH & TANNER WASH
SHEET 28 - GERONIMO SEEP
SHEET 29 - P-226 SEEP
DETAIL 6 TYP. FLOW METER ENCLOSURE
¾"=1'-0"
PL ¼"×4'-8" DIA. LID
L6"×3½"×5/16"×0'-3" (2) PLACES - SEE DETAIL 8
LIFTING HANDLE-(3) PLACES TYPICAL SEE DETAIL 7
HEAVY WEIGHT STEEL HINGES (2 REQUIRED) WELD IN PLACE
DETAIL 9 TYP. FLOW METER LID
1"=1'-0"
CHOLLA S.E.S. COMMON
BA/FA PONDS SEEPAGE INTERCEPT SYSTEM
SECTIONS AND DETAILS
ARIZONA PUBLIC SERVICE COMPANY
SCALE NOTED
DATE 03-30-93
R.PRATT
TRH
DRAWING NO. G-114438
SHEET 4 OF 4
THIS IS A CADAN DRAWING, DO NOT REVISE MANUALLY
GROUP: GENE -- SUBGROUP: PRATT -- DWG ID: 3F JG 114438 0004 -- PLOT DATE, 04/08/93

EMBANKMENT MONITORING

(APS, 1999)



DAM PLAN VIEW

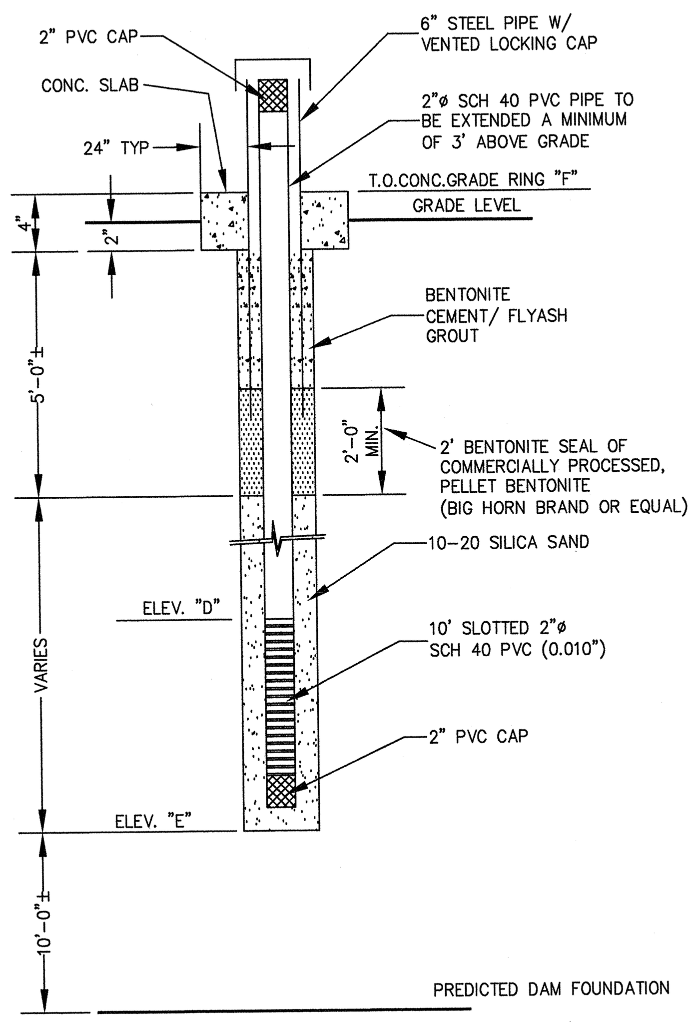
1"=500'

ALL CONTOURS AND ELEVATIONS SHOWN HERE ARE FROM ORIGINAL FLY ASH POND DAM. REFERENCE APS DRAWING G-44558 AND DAM ENGINEERING REPORT FIGURES 13 & 14

PARTIAL PLAN VIEW

1"=100'

ALL CONTOURS AND ELEVATIONS SHOWN HERE ARE FROM ORIGINAL FLY ASH POND DAM. REFERENCE APS DRAWING G-44558 AND DAM ENGINEERING REPORT FIGURES 13 & 14

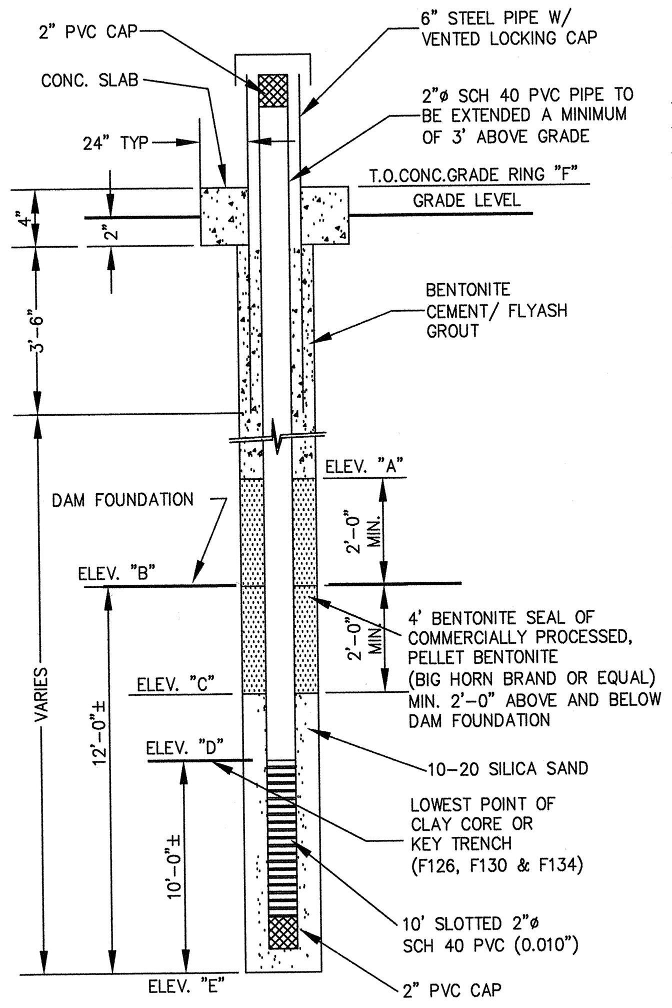


DETAIL 1

NO SCALE

PIEZOMETER DETAIL

REF. TABLE 1 FOR ALPHA LABELED ELEVATIONS NOTED

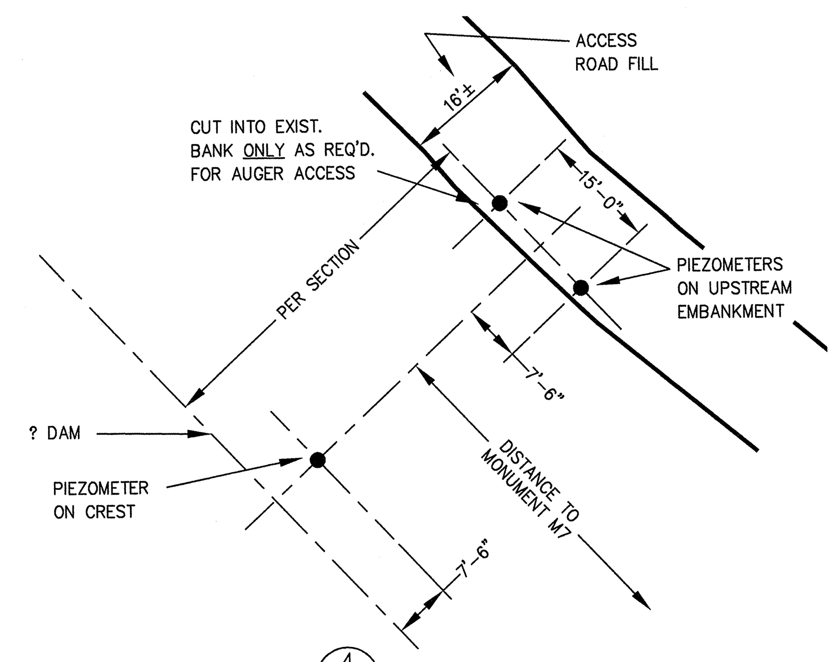


DETAIL 2

NO SCALE

PIEZOMETER DETAIL

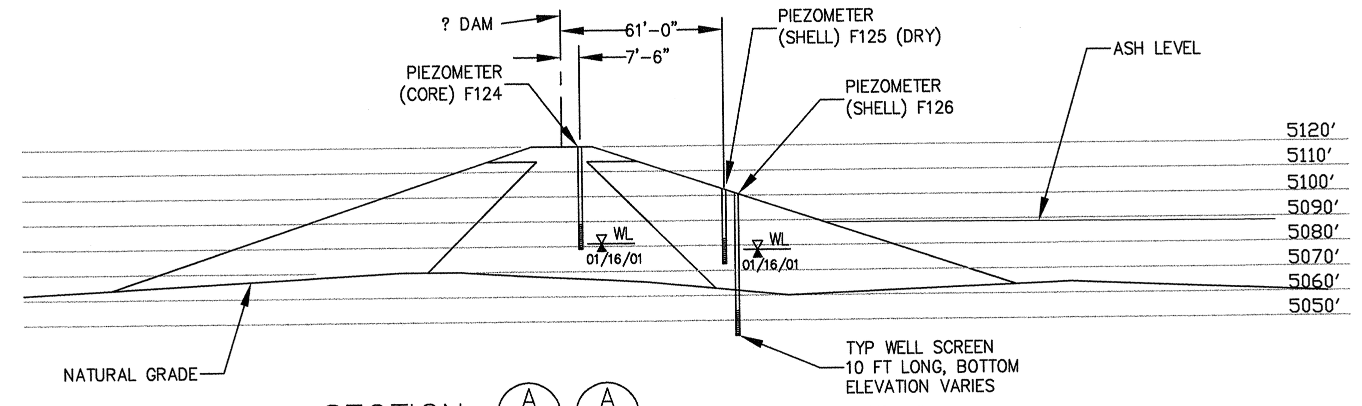
REF. TABLE 1 FOR ALPHA LABELED ELEVATIONS NOTED



DETAIL 4

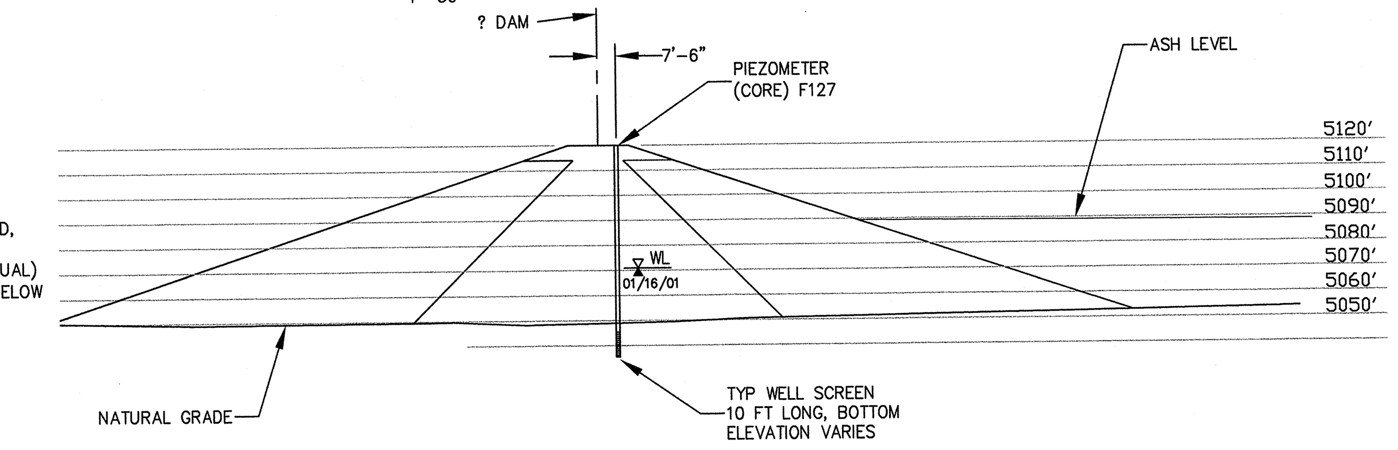
NO SCALE

TYPICAL PIEZOMETER LAYOUT



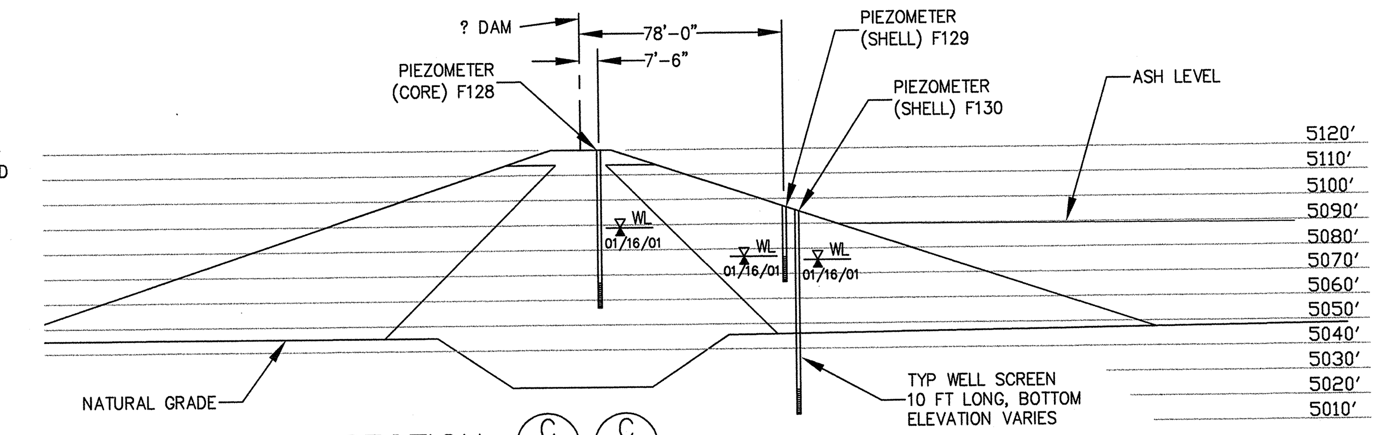
SECTION A A 203' WEST OF MONUMENT M7

1"=50'



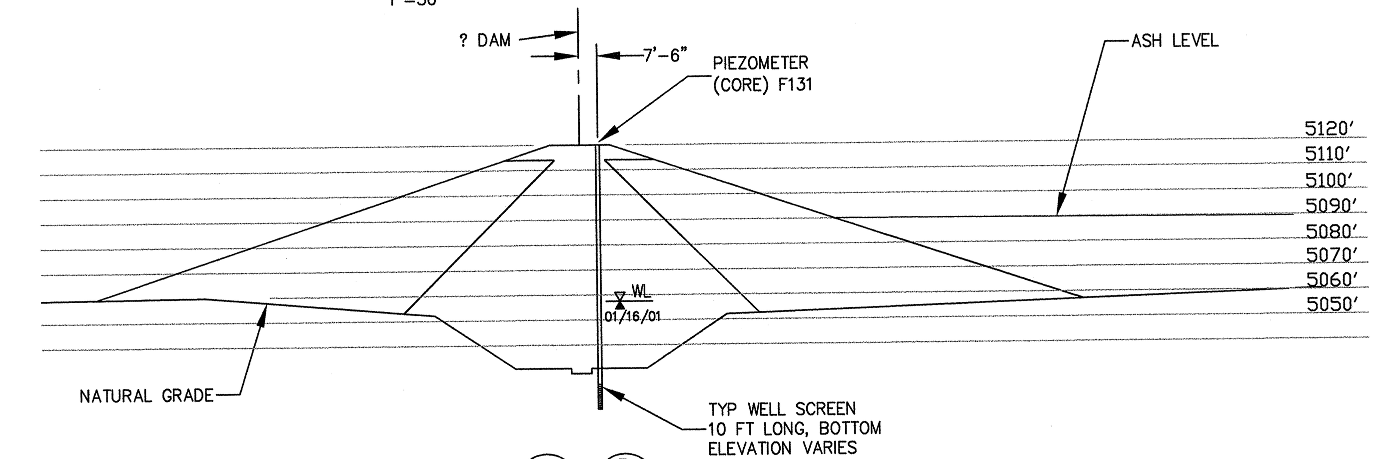
SECTION B B 303' WEST OF MONUMENT M7

1"=50'



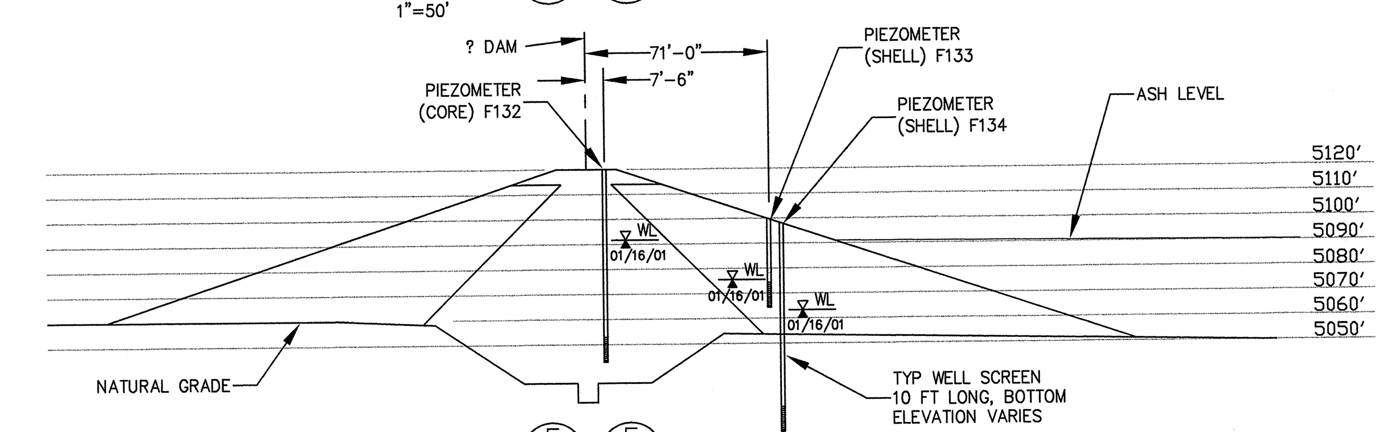
SECTION C C 403' WEST OF MONUMENT M7

1"=50'



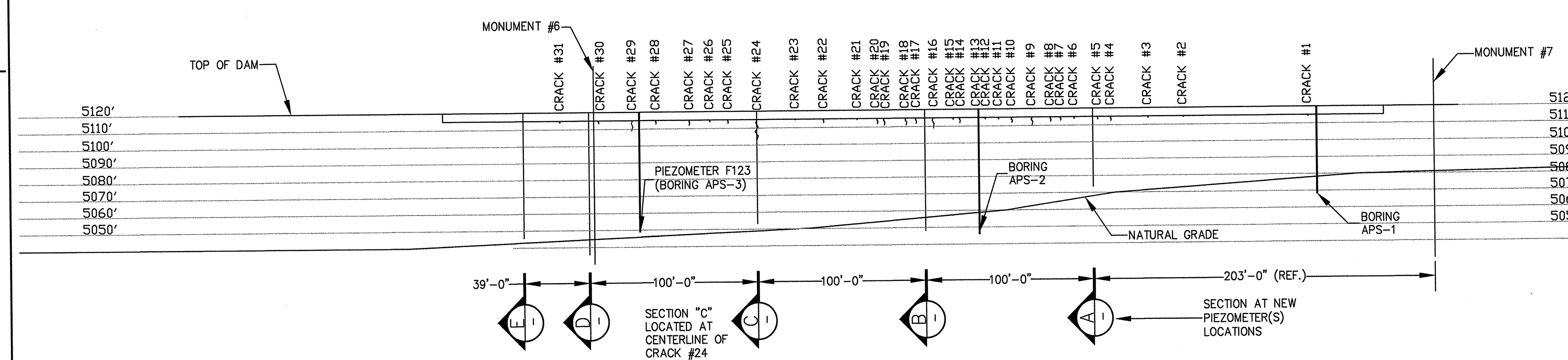
SECTION D D 503' WEST OF MONUMENT M7

1"=50'



SECTION E E 542' WEST OF MONUMENT M7

1"=50'



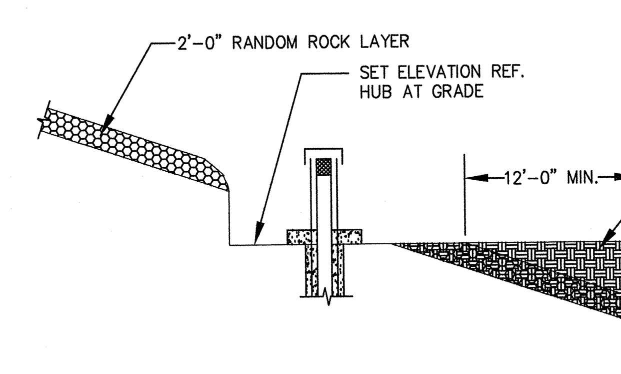
LONGITUDINAL VIEW AT CENTER OF DAM

1"=50'

ALL CONTOURS AND ELEVATIONS SHOWN HERE ARE FROM ORIGINAL FLY ASH POND DAM. REFERENCE APS DRAWING G-44558 AND DAM ENGINEERING REPORT FIGURES 13 & 14

TABLE 1

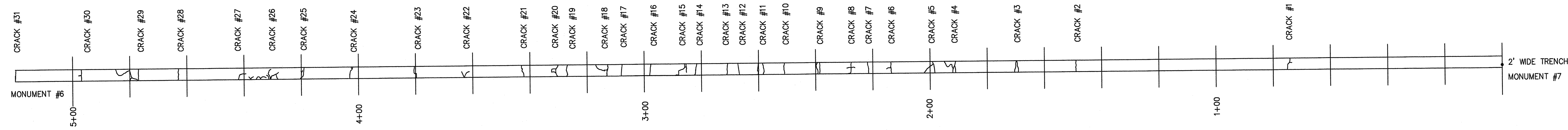
ITEM #	PIEZOMETER NAME	REF. DETAIL	PIEZOMETER ELEVATIONS REF. DETAILS 1 & 2						
			A	B	C	D	E	F	PERMIT NUMBER
1	F-124	1				5,088.99	5,078.99	5,118.99	579540
2	F-125	1				5,082.85	5,072.85	5,103.85	
3	F-126	2	5,069.94	5,066.94	5,063.94	5,053.94	5,043.94	5,103.94	579541
4	F-127	2	5,053.61	5,047.61	5,046.61	5,045.61	5,035.61	5,118.61	579546
5	F-128	1				5,066.91	5,056.91	5,118.91	579542
6	F-129	1				5,066.78	5,056.78	5,098.78	
7	F-130	2	5,054.70	5,052.70	5,048.70	5,023.70	5,013.70	5,098.70	579543
8	F-131	2	5,031.49	5,027.49	5,025.49	5,024.49	5,014.49	5,118.49	579545
9	F-132	1				5,053.10	5,043.10	5,118.10	
10	F-133	1				5,074.43	5,064.43	5,099.43	
11	F-134	2	5,056.83	5,048.83	5,046.83	5,024.83	5,014.83	5,099.83	579544
	TOTAL								



DETAIL 3

NO SCALE

TYPICAL SECTION @ PIEZOMETER ON UPSTREAM FACE



PLAN VIEW AT CENTER OF DAM

1"=20'

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
5	01/19/01	UPDATE WATER LEVELS	RHT					30-9238
4	10/13/00	UPDATE WATER LEVELS	RHT					30-9238
3	6/27/00	REV. PER AS-BUILT	RHT			RQF	JDM	30-9238
2	4-26-00	REV. PER AS-BUILT	KK					30-9238
1	11-15-99	CHGS. PER ADWR REVIEW	TRH		RQF		JDM	30-6923

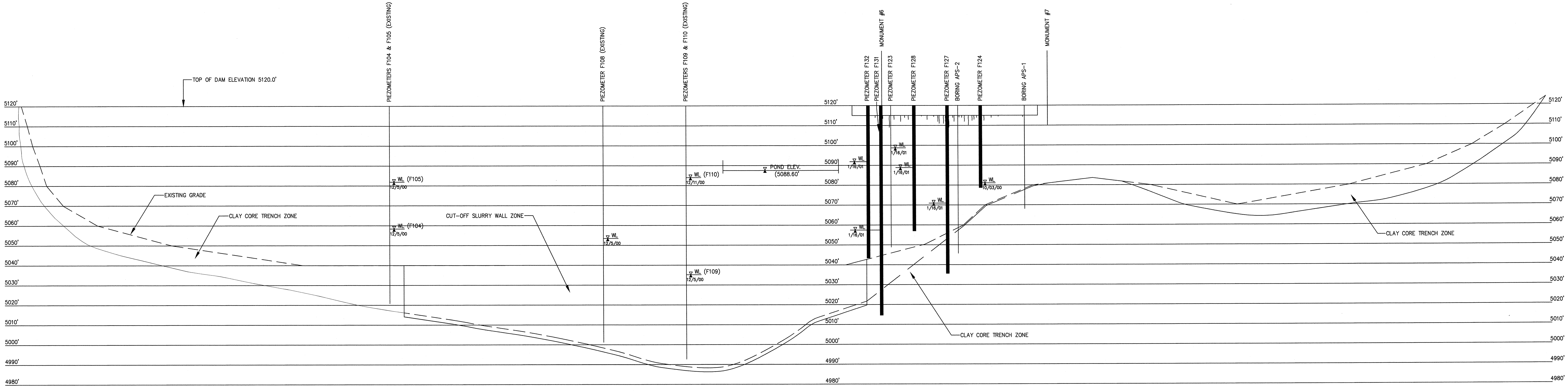
CHOLLA POWER PLANT
FLY ASH POND
EMBANKMENT MONITORING
PLAN AND SECTIONS

APS

SCALE NOTED
DATE 9-7-99

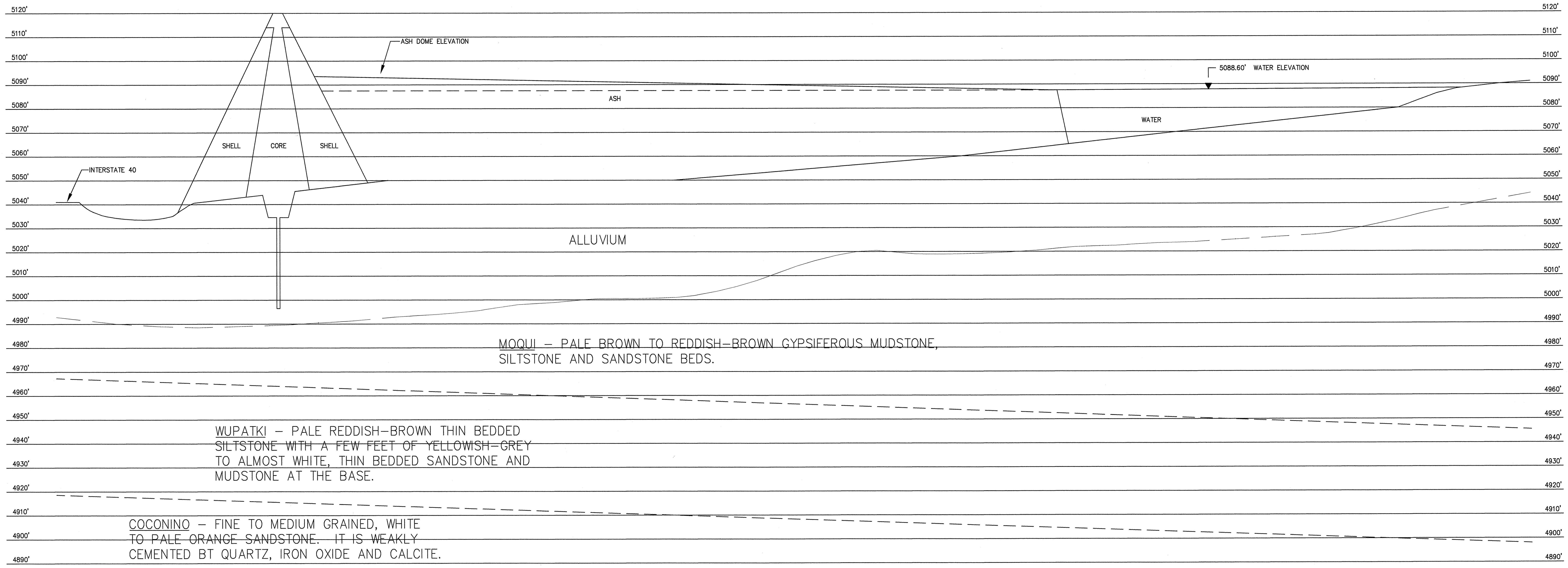
DWN KK/MLD
CHD
EXD RQF
APVD JOHN D. MITCHELL, ENGINEERING SUPERVISOR
RVWD
W A 30-6923
DRAWING NO. G-145746
SHEET 1 OF 2

AAAAAMDU.DWG



LONGITUDINAL VIEW AT CENTER OF DAM

ALL CONTOURS AND ELEVATIONS SHOWN HERE ARE FROM ORIGINAL FLY ASH POND DAM. REFERENCE APS DRAWING G-44558 AND DAM ENGINEERING REPORT FIGURES 13 & 14



CROSS SECTION THRU DAM AND POND AT INTAKE

ALL CONTOURS AND ELEVATIONS SHOWN HERE ARE FROM ORIGINAL FLY ASH POND DAM. REFERENCE APS DRAWING G-44558 AND DAM ENGINEERING REPORT FIGURES 13 & 14

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
5	01/22/01	UPDATE WATER LEVELS	RHT					30-9238
4	10/13/00	UPDATE WATER LEVELS	RHT					30-9238
3	8/22/00	REV. PER AS-BUILT	RHT			RQF	JDM	30-9238
2	6/27/00	REV. PER AS-BUILT	RHT			RQF	JDM	30-9238
1	4-26-00	REV. PER AS-BUILT	KK					30-9238

CHOLLA POWER PLANT
FLY ASH POND
EMBANKMENT MONITORING
PROFILES

APS

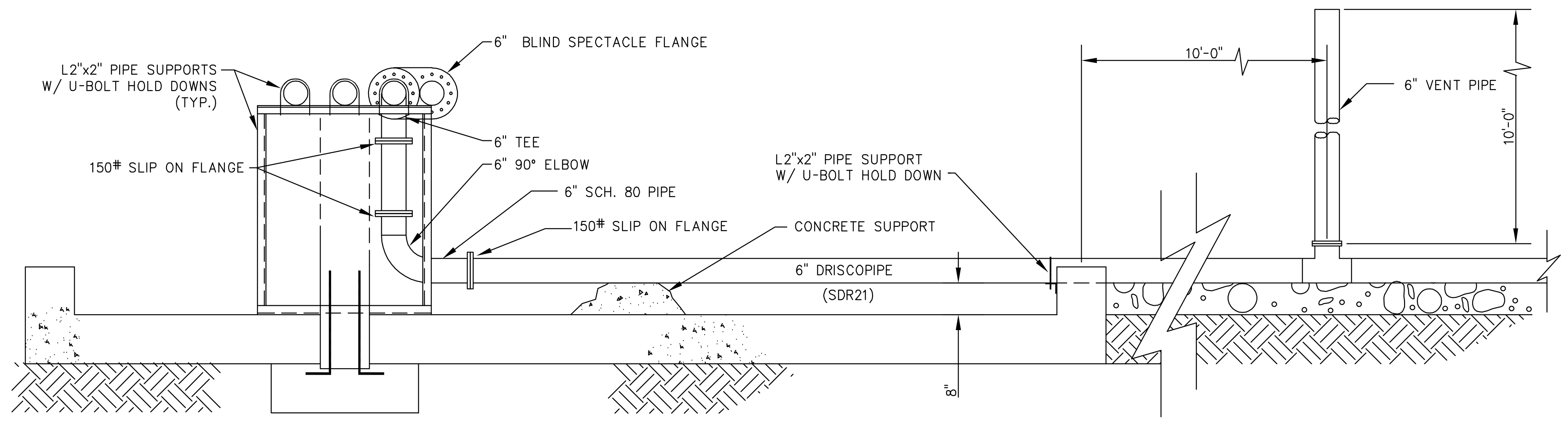
SCALE VERT. 1"=20' HORIZ. 1"=120'
DATE 9-7-99

DWN KK/MLD
CHD
EXD RQF
APVD JOHN D. MITCHELL, ENGINEERING SUPERVISOR
RVWD
W A 30-6923
DRAWING NO. G-145746
SHEET 2 OF 2

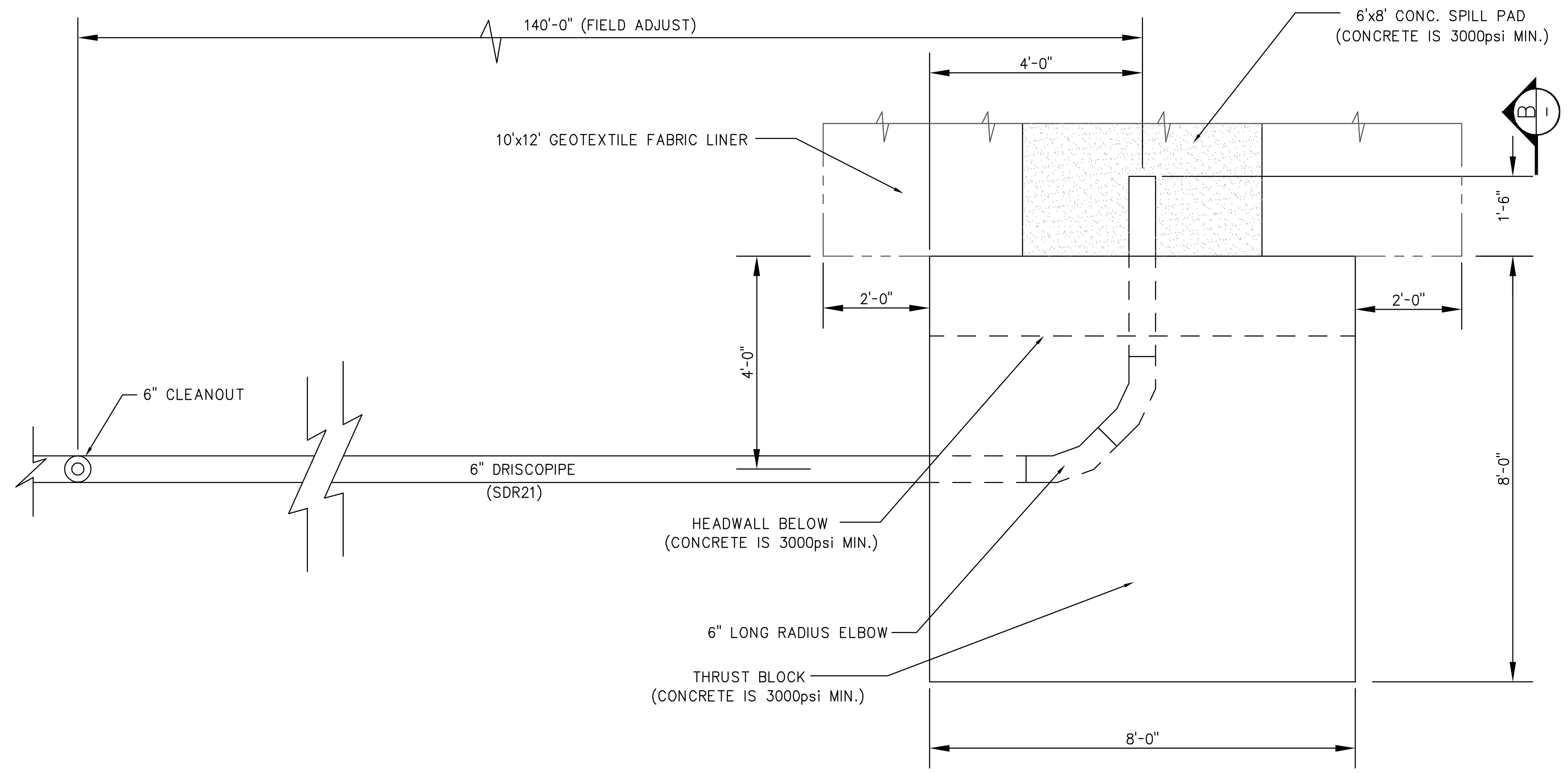
AAAAAMDV.DWG

DISCHARGE PIPE DRAWINGS

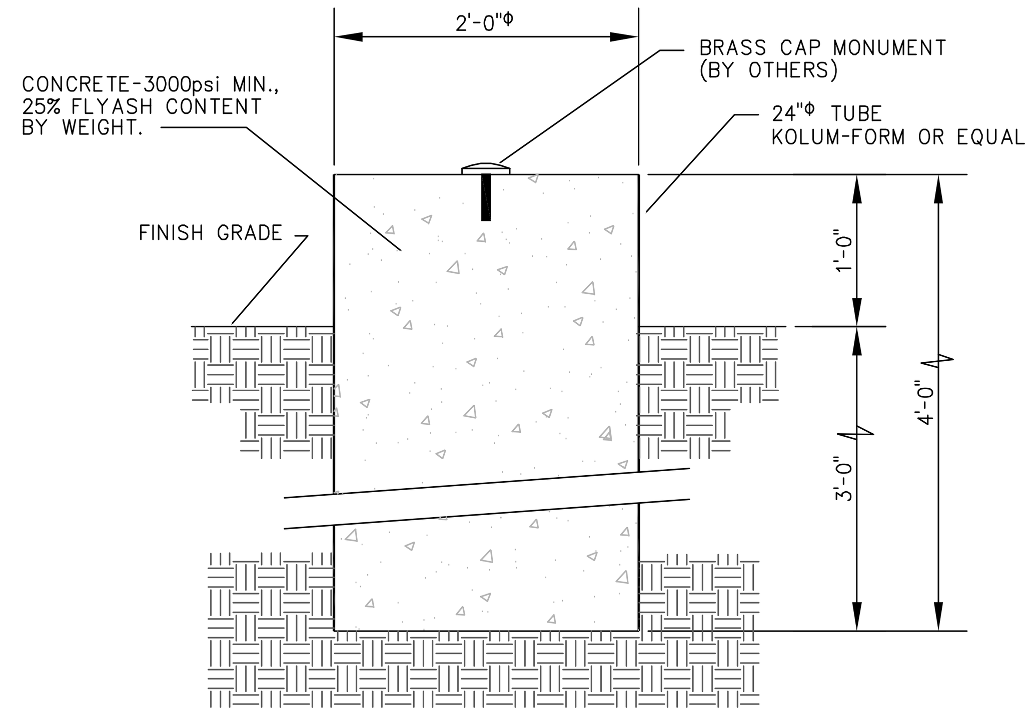
(APS, Rev. 2004)



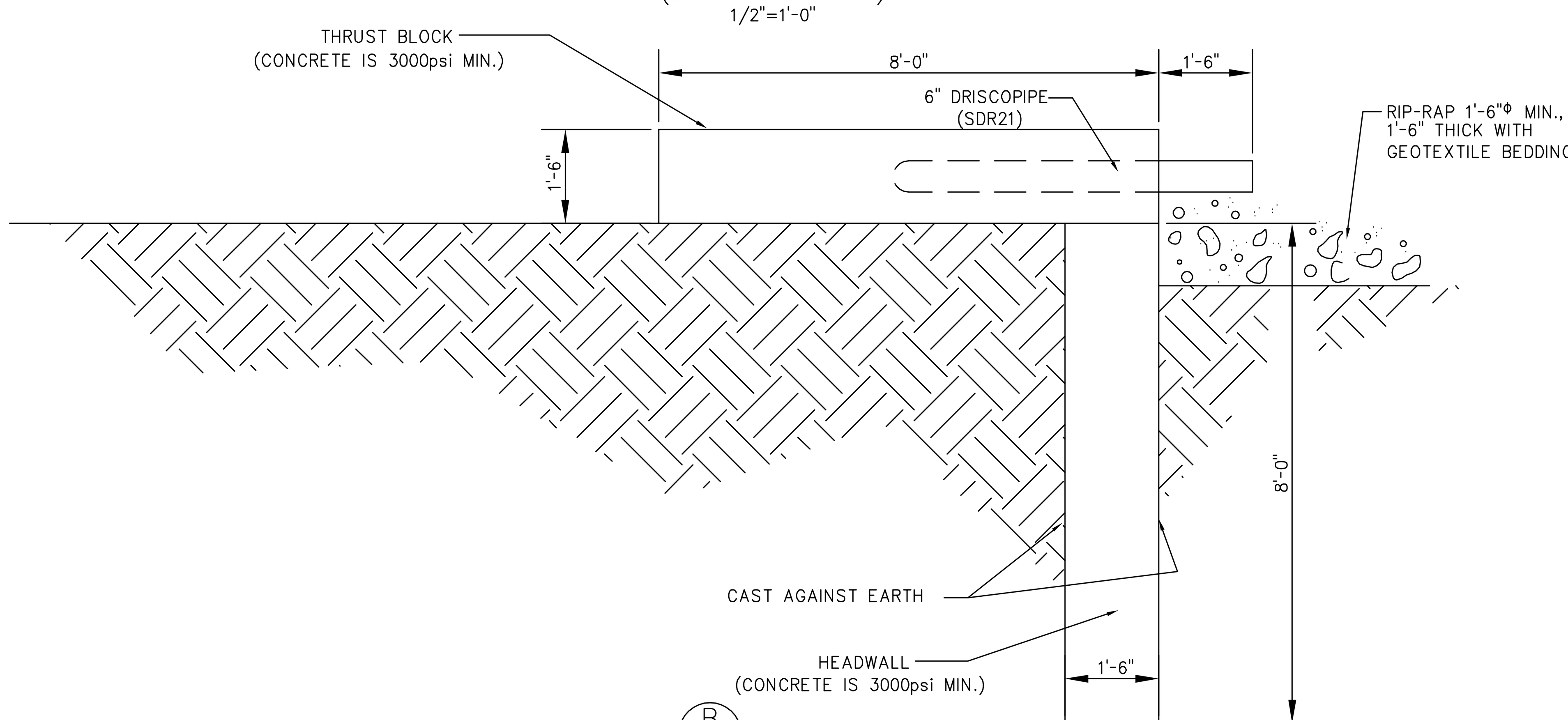
SECTION (A) NEW DISCHARGE CONNECTION
1/2"=1'-0"



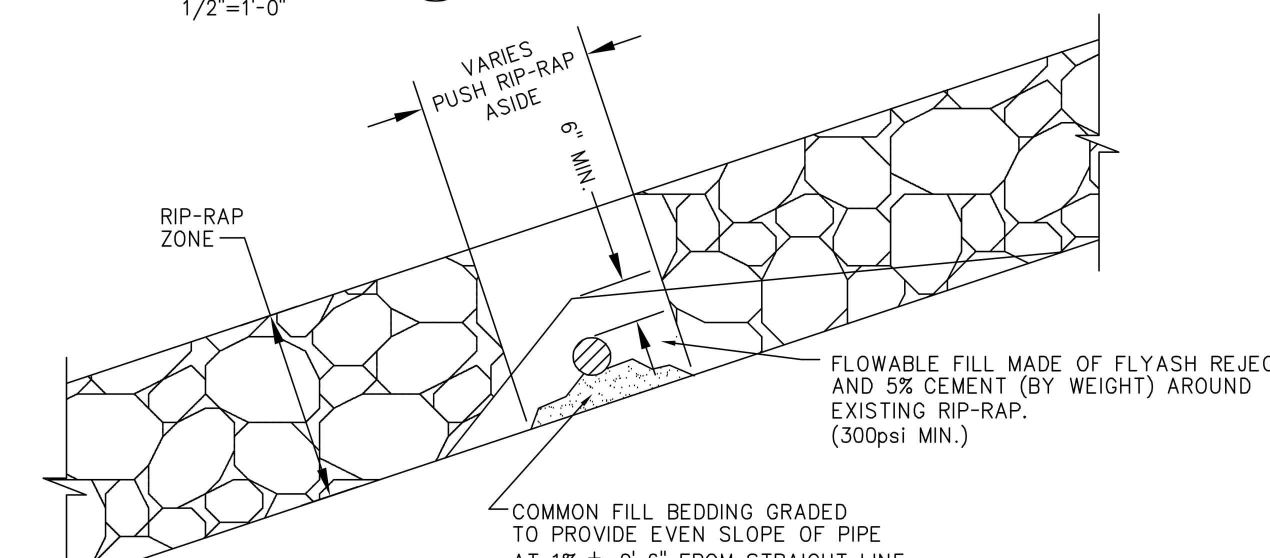
PLAN VIEW
(DISCHARGE POINT)
1/2"=1'-0"



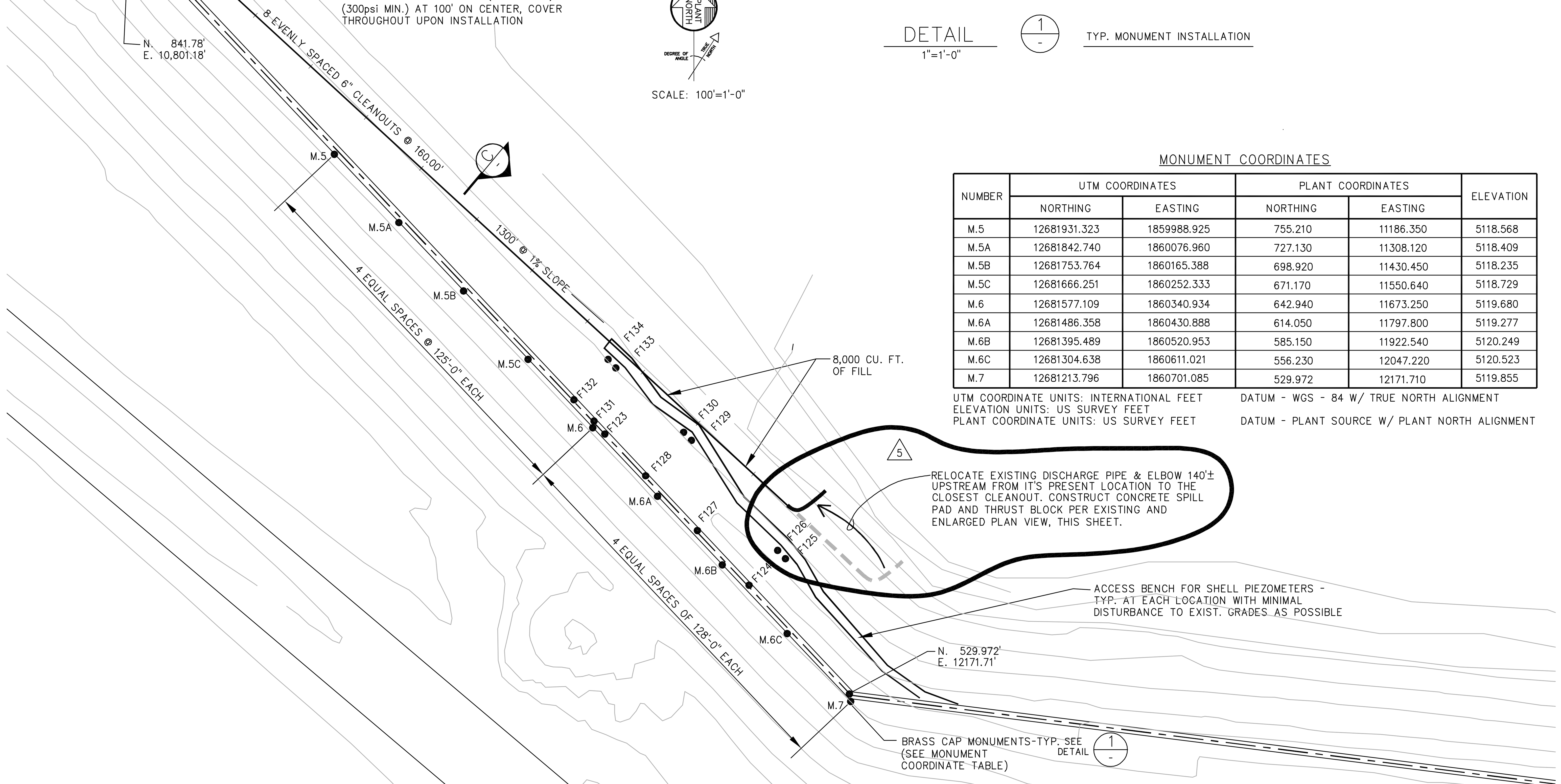
DETAIL (1) TYP. MONUMENT INSTALLATION
1"=1'-0"



SECTION (B) HEADWALL AND BASE
1/2"=1'-0"



SECTION (C) COVER AND BEDDING SECTION
1/2"=1'-0"



SCALE: 100'=1'-0"

MONUMENT COORDINATES

NUMBER	UTM COORDINATES		PLANT COORDINATES		ELEVATION
	NORTHING	EASTING	NORTHING	EASTING	
M.5	12681931.323	1859988.925	755.210	11186.350	5118.568
M.5A	12681842.740	1860076.960	727.130	11308.120	5118.409
M.5B	12681753.764	1860165.388	698.920	11430.450	5118.235
M.5C	12681666.251	1860252.333	671.170	11550.640	5118.729
M.6	12681577.109	1860340.934	642.940	11673.250	5119.680
M.6A	12681486.358	1860430.888	614.050	11797.800	5119.277
M.6B	12681395.489	1860520.953	585.150	11922.540	5120.249
M.6C	12681304.638	1860611.021	556.230	12047.220	5120.523
M.7	12681213.796	1860701.085	529.972	12171.710	5119.855

UTM COORDINATE UNITS: INTERNATIONAL FEET
ELEVATION UNITS: US SURVEY FEET
PLANT COORDINATE UNITS: US SURVEY FEET

DATUM - WGS - 84 W/ TRUE NORTH ALIGNMENT
DATUM - PLANT SOURCE W/ PLANT NORTH ALIGNMENT

REFERENCE DRAWINGS
G-145746
G-114438

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
5	12-14-04	Relocate discharge pipe	PRATT	PSS			DRR	1500-001
4	11-12-02	NEW DISCHARGE PIPE INSTALLED	DCM		RQF		JDM	30-6876
3	2-26-02	CORRECTION TO AS-BUILTS	PRATT		RQF		JDM	30-9238
2	6-21-01	REVISE PER AS-BUILTS	PRATT		RQF		JDM	30-9238
1	02-28-01	REVISED COVER AND BEDDING DETAIL	BTF		RQF		JDM	30-6923

CHOLLA POWER PLANT
FLY ASH POND
DISCHARGE CONNECTION
PLAN AND DETAILS

APS

SCALE AS NOTED
DATE 01-19-01

DWN RHT
CHD
EXD RQF
APVD JOHN D. MITCHELL, ENGINEERING SUPERVISOR
RVWD
W A 30-6923
DRAWING NO. E- 146981
SHEET 1 OF 1

AAAAOVR.DWG

DRAWING: L:\A_CAD_USERS\BOBJP\CC-C-39-WWD-146981-1.dwg

Appendix B.
Safety Factor Calculation

URS Quality - It's Good Business

QMS Form 3-4 (MM)

Date: September 2014

IE QMS

Check and Review Record

Project Name	CHPP SIA	**Client Name**	APS
Project Location	Cholla Power Plant	**PM Name**	Frances Ackerman, R.G., P.E.
Project Number / Office Code	60445840	**PIC Name**	Alexander Gourlay, P.E.

Type

☒ Detail Check	☐ Coordination Review	☐ Constructability Review	☐ Bidability Review
☐ Independent Technical Review (ITR)	☐ Calculation Check *(can also use QMS Form 3-3)*	☐ Other:	*For Subconsultant, Client, or Third-Party Information Review, use Form 3-11.*

Identifying Information

(This section is to be completed by the Project Manager or the PM's Designee.)

Individual Assigned:	Tafwachi Chamunda, P.E.	Comments Required by:	3/23/16
Work Product Originator:	Jed Stoken, P.E.	Title of Work Product:	Safety Factor Calculation Package

Review Scope

☒ Technical edit for elements such as grammar, punctuation and formatting.	☐ Completion of review of client and third-party information.	☒ Basis and validity of conclusion / recommendation.
☒ Detail Check of calculations and graphics.	☒ Soundness of approach/design.	☒ Organization, clarity and completeness.
☐ Completion of Detail Check	☒ Conformance with standards	☐ Application of Statements of Limitations.
☐ Other:		

Project Manager (or Designee) Signature — **Date** 8/12/16

Comments

Select:

☐ Checker / Reviewer has no comments.

or

☒ Comments have been provided on:

☒ Marked directly on work product (electronically or on hard copy).

☐ Comment and Disposition Record (QMS Form 3-5).

☐ Other:

Checker / Reviewer Signature — **Date** 3/23/16

Verification

(Note: Reviews and Checks are often iterative, requiring multiple rounds to verify accuracy and completeness of the work product. This section is to be completed by the Checker/Reviewer after verification of comment incorporation to include subsequent or new comments.)

Select:

☒ Checker / Reviewer has verified that comments have been adequately addressed. There are no outstanding issues.

or

☐ Checker / Reviewer has verified that comments have been adequately addressed. Any unresolved issues have been submitted to the Project Manager or Designee for final resolution.

and

☒ Checker / Reviewer confirms that the work product Check / Review is complete.

Checker / Reviewer Signature — **Date** 4/13/16

URS Quality - It's Good Business	QMS Form 3-4 (MM)
	Date: September 2014
IE QMS	Check and Review Record

Approval	*(This section is to be completed by the Project Manager or PM's designee.)* ☐ Project Manager or Designee confirms that the Check / Review process has been followed. **Project Manager (or Designee) Signature** — **Date** 8/12/16
DISTRIBUTION	Project Central File – Quality File Folder Other – Specify:

DESIGN CALCULATION				
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1 INTRODUCTION

The purpose of this calculation is to perform limit equilibrium slope stability analyses to assess the stability of the existing Coal Combustion Residual (CCR) surface impoundment dam Fly Ash Pond (FAP) Dam, ADWR Dam #09.28, at Arizona Public Service (APS)'s Cholla Power Plant near Joseph City, AZ.

2 ANALYSIS CRITERIA

The analyses were performed to meet the regulations set forth in the United States Environmental Protection Agency (EPA) 40 CFR Part 257.73(e) Structural Integrity Criteria for Existing CCR Surface Impoundments (EPA 2015). The code requires safety factor assessments for units containing CCRs. The safety factors for various embankment loading and tailwater conditions must meet the values outlined. For the FAP Dam, the following safety factors must be met:

- Long-term, maximum storage pool FS = 1.50;
- Maximum surcharge pool FS = 1.40;
- Seismic loading FS = 1.00; and
- Liquefaction loading FS = 1.20 (only for sites with liquefiable soils).

3 ANALYSIS INPUTS

The following inputs were used in the analysis:

- Surface profiles were developed from 2009 elevation contour drawings of the FAP Dam and surrounding terrain (Cooper Aerial Surveys Co. 2014).
- Subsurface stratigraphies were developed from as-built cross section drawings of the FAP Dam (Ebasco 1977).
- Material properties used in the model were developed in a separate calculation (AECOM 2016).
- Pore pressure distribution within the dam was developed from interpretation of water level readings for piezometers installed at the dam and surrounding area. Water level measurements are presented in the annual dam basic data report (APS 2016).

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- The maximum storage pool water level of the CCR Pond was based on the maximum permissible water level stated in the permitting license for the FAP (ADWR 1986).
- The surcharge pool water level of the CCR Pond was developed based on estimated water levels for the design probable maximum flood (PMF) of the FAP (Ebasco 1975).
- The seismic loading for the FAP was developed from the deaggregated seismic hazard at the site based on the 2008 United States Geological Survey (USGS) National Earthquake Hazards Reduction Program (NEHRP) Provisions (USGS 2008).

The slope stability analyses were performed using the software program SLOPE/W, commercially available through GEO-SLOPE International, Ltd. (GEO-SLOPE International 2012).

4 ASSUMPTIONS

The following assumptions were used in the analysis:

- The surface profile for the site was developed based on the most recent topographic survey available, from June of 2009. It is assumed that the surface topography shown in this survey is sufficiently representative of the current topography so as not to produce significant differences in the estimated safety factors. This seems reasonable since there have been no significant alterations to the FAP Dam or the immediate surrounding area since the survey was conducted, except for additional accumulation of fly ash within the impoundment.
- The water level measured in the piezometers near Cross Section 2, reflect the influence of the Geronimo Seep collection system. The collection system consists of an underground french drain system and wellpoints and has been in continuous operation since the early 1990s. The seep collection system presumably lowers the phreatic water level at the downstream toe of the dam in the vicinity of the wellpoints. Since it is difficult to assess the radial influence of the collection system, it is assumed the Geronimo Seep seepage collection system is non-operational for the stability analysis of Cross Section 2. This has the effect of raising the water level downstream of the dam to near the ground surface.

5 STABILITY ANALYSIS

Slope stability analyses were performed to document minimum factors of safety for loading conditions identified by 40 CFR Section 257.73(e) using the software program SLOPE/W (GEO-SLOPE International, Ltd. 2012). The analyses were performed using Spencer's Method, a limit

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equilibrium method of slices that satisfies both force and moment equilibrium in addition to incorporating the effects of interslice forces.

5.1 Critical Stability Cross Sections

Factors of safety were calculated for critical cross-sections of the FAP Dam. The critical cross section is the cross section that is anticipated to be most susceptible to structural failure for a given loading condition. The critical cross section thus represents a "most-severe" case. Section locations were selected based on variation in the embankment height and stratigraphic conditions to represent the most-severe case.

The safety factor assessments were performed for three cross-sections along the FAP Dam:

FAP Dam Cross Sections

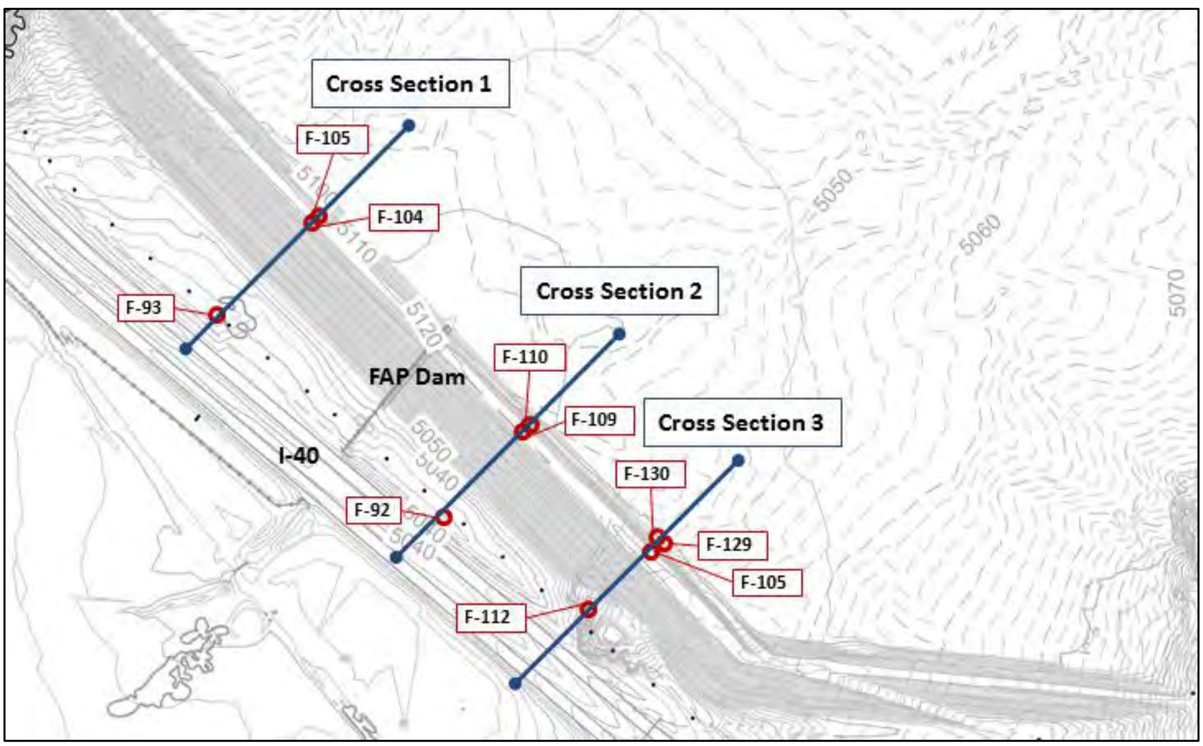

Figure 1. Slope Stability Cross Section and Piezometer Locations Along the FAP Dam

FAP Cross Section 1:

Cross Section 1 at the FAP was located along the western portion of the dam near piezometers F-93, F-104, and F-105. At this location, the dam is approximately 80 feet (ft) in height from EL 5,040 ft at the downstream toe to 5,120 ft at the crest; with upstream and downstream slope angles are about 3H:1V. The dam at this cross section consists of a sandy

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lean clay core with an outer clayey sand shell. The dam lies on a foundation of alluvial overburden consisting of clays, silts, and sands; overlying bedrock consisting of mudstones, siltstones, and sandstones. The depth to bedrock is about 20 ft below the ground surface (bgs). A cutoff trench filled with compacted clay extends from the clay core down to the bedrock and is used to control seepage beneath the dam, in lieu of a cutoff wall which is used for greater depths to bedrock. The upstream slope of the dam is confined by approximately 60 ft of hydraulically-placed fly ash based on comparison between initial topographic surveys of the area (Ebasco 1975) and 2009 surveys (Cooper Aerial Surveys 2014).

FAP Cross Section 2:

Cross Section 2 at the FAP was located near the center of the dam near piezometers F-92, F-109, and F-110. At this location, the dam is approximately 80 ft in height from EL 5,040 ft at the downstream toe to 5,120 ft at the crest; with upstream and downstream slope angles of about 3H:1V. Similar to Cross Section 1 described above, the dam consists of a sandy lean clay core with an outer clayey sand shell. At this location the depth to bedrock beneath the alluvial soils (same as those described for Section 1) is greatest along the dam at approximately 52 ft bgs. A cement-bentonite cutoff wall extends from the clay core of the dam to approximately 2 ft into the bedrock and is used to control seepage beneath the dam The upstream slope of the dam is confined by approximately 60 ft of hydraulically-placed fly ash based on comparison between initial topographic surveys of the area (Ebasco 1975) and 2009 surveys (Cooper Aerial Surveys 2014).

FAP Cross Section 3:

Cross Section 3 at the FAP was located along the eastern portion of the dam near piezometers F-112, F-127, F-129, and F-130. At this location, the dam intersects a rock outcropping commonly referred to as Geronimo Knob along its downstream slope. Consequently, the upstream and downstream slope heights are considerably different at approximately 68 ft versus 51 ft, respectively, although both slope angles are about 3H:1V. Similar to other cross sections described above, the dam consists of a sandy lean clay core with an outer clayey sand shell. The depth to bedrock beneath the alluvial soils (same as those described for Section 1) is shallow at this section, approximately 4 to 9 ft bgs. A cutoff trench filled with compacted clay extending to the bedrock is used to control seepage. The upstream slope of the dam is confined by approximately 50 ft of hydraulically-placed fly ash based on comparison between initial topographic surveys of the area (Ebasco 1975) and 2009 surveys (Cooper Aerial Surveys 2014).

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5.2 Material Properties

A material properties calculation package was prepared to present the methods and information supporting the parameter selection for the materials at the FAP Dam (AECOM 2016). The material properties identified in the calculation and used in the slope stability analyses are presented in Table 1 below.

Table 1. Material Properties for the FAP Dam Safety Factor Analyses

Material	Sat. Unit Weight, γ_{sat} (pcf)	Moist Unit Weight, γ_m (pcf)	Drained Strengths		Undrained Strengths	
			Cohesion, c' (psf)	Friction Angle, ϕ' (degrees)	Undrained Strength, S_u (psf)	Undrained Strength Ratio
Clay Core	125	120	0	28	-	0.38
Shell	130	125	0	33	-	-
Alluvium	120	120	0	26	-	-
Bedrock	150	150	1,000	65	-	-
Cutoff Wall	106	106	0	28	10	-
Fly Ash	90	90	0	0	-	-

5.3 Embankment Pore Pressure Distribution

Based on guidance from the EPA Regulations (EPA 2015), pore-water pressures are estimated from the most reliable of the following: *"1) Field measurements of pore pressures in existing slopes; 2) past experience and judgment of the Engineer; 3) hydrostatic pressures calculated for the no-flow condition; or 4) steady-state seepage analysis using flow nets or finite element analyses."* For the FAP analysis, the pore pressure distribution was assigned using water level readings obtained from piezometers located near the stability cross sections (APS 2014). This distribution was adjusted based on engineering judgement to correspond with pond water level under steady-state, maximum storage pool conditions (ADWR 1986), and pond water levels under maximum surcharge pool conditions (Ebasco 1975). The piezometers used to estimate the pore water pressure within the dam cross sections are shown in Figures 1.

The FAP (upstream) water level under maximum storage pool condition was based on the permitted water level of the pond as stated in the ADWR operating license for the dam. Since the dam has no outlet work structure and rely on pumping rate from plant, seepage, and evaporation to control water levels, the maximum storage pool was set at the maximum permitted water levels. For the FAP this is EL 5,114.0 ft (ADWR 1986). The surcharge pool level is based on the expected water level raise during the design PMF and is EL 5,116.0 ft for the FAP (Ebasco 1975).

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5.4 Embankment Loading Conditions

Per 40 CFR Section 257.73(e), the following loading conditions were considered for each selected stability cross section:

- Long-term, maximum storage pool;
- Maximum surcharge pool;
- Seismic loading; and
- Liquefaction.

These loading conditions are described below.

<u>Long-Term, Maximum Storage Pool</u>

The maximum storage pool loading is the maximum water level that can be maintained that will result in the full development of a steady-state seepage condition. This loading condition is evaluated to document whether the CCR surface impoundments can withstand the maximum expected pool elevation with full development of saturation in the embankment under long-term loading. The maximum storage pool considers a pool elevation in the CCR unit that is equivalent to the maximum permitted water levels using shear strengths expressed as effective stress with pore water pressures that correspond to the long-term condition.

For this analysis, the long-term, maximum storage pool in the FAP was set at EL 5,114.0 ft. Since the piezometric conditions within the dam are at steady-state flow, drained material strengths were used in the analysis.

<u>Maximum Surcharge Pool</u>

The maximum surcharge pool loading is the temporary rise in pool elevation above the maximum storage pool elevation for which the CCR surface impoundment is normally subject under the inflow design flood state. This loading condition is evaluated to document whether the CCR surface impoundments can withstand a short-term impact of a raised pool level on the stability of the downstream slope. The maximum surcharge pool considers a temporary pool elevation that is higher than the maximum storage pool assuming that it persists for a length of time sufficient for steady-state seepage or hydrostatic conditions to fully develop within the embankment.

For this analysis, the maximum surcharge pool in the FAP was set at EL 5,116.0 ft. Since the piezometric conditions within the dam are at steady-state flow for this loading condition, drained material strengths were used in the analysis.

<u>Seismic Loading</u>

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Seismic loading was evaluated to document whether the CCR surface impoundments are capable of withstanding a design earthquake without damage to the foundation or embankment that would cause a discharge of its contents. The seismic loading is assessed under seismic loading conditions for a seismic loading event with a 2% probability of exceedance in 50 years, equivalent to a return period of approximately 2,500 years. A pseudo-static analysis was used to represent the seismic loading.

The peak horizontal bedrock acceleration for a site classification of B “Rock” based on the USGS 2008 NEHRP seismic hazard map with a 2% probability of exceedance in 50 years is 0.0807g as presented in Attachment A (USGS 2008). Based on previous site explorations, a sit classification of D “Stiff Soil” was assigned to the site as illustrated in Table 1615.1.1 from the IBC (2003) shown in Figure 2.

TABLE 1615.1.1
SITE CLASS DEFINITIONS

SITE CLASS	SOIL PROFILE NAME	AVERAGE PROPERTIES IN TOP 100 feet, AS PER SECTION 1615.1.5		
		Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \le 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \le 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \ge 2{,}000$
D	Stiff soil profile	$600 \le \bar{v}_s \le 1{,}200$	$15 \le \bar{N} \le 50$	$1{,}000 \le \bar{s}_u \le 2{,}000$
E	Soft soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	$\bar{s}_u < 1{,}000$
E	—	Any profile with more than 10 feet of soil having the following characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \ge 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1 foot = 304.8 mm, 1 square foot = 0.0929 m^2, 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

Figure 2. Table 161.1.1 Site Class Definitions (IBC 2003)

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The PGA at the ground surface for Site Class D, or PGA_M, was determined by amplifying the PGA for rock (Site Class B) using the following equation presented in NEHRP, 2009:

$$PGA_M = F_{PGA}(PGA)$$

$$PGA_M = 1.6(0.0807g)$$

$$PGA_M = 0.129g$$

Where:

PGA_M = Maximum considered earthquake geometric mean peak ground acceleration adjusted for Site Class effects

PGA = Mapped maximum considered earthquake geometric mean peak ground acceleration

F_{PGA} = Site coefficient from Table 11.8-1 (Figure 3)

Table 11.8-1 Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.1	PGA = 0.2	PGA = 0.3	PGA = 0.4	PGA ≥ 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

Note: Use straight-line interpolation for intermediate values of PGA.

Figure 3. Table 11.8-1 Site Coefficient F_{PGA} (NEHRP 2009)

The PGA at the ground surface for Site Class D (PGA_M) was then used to estimate the peak transverse acceleration at the crest of the embankment, PGA_{crest} = 0.307g, as shown on Figure 4 and based on variations in recorded peak crest accelerations versus those recorded at the base of earth and rock fill dams by Idriss (2015) and on recorded values for Loma Prieta, and other earthquakes, by Holzer (USGS, 1998).

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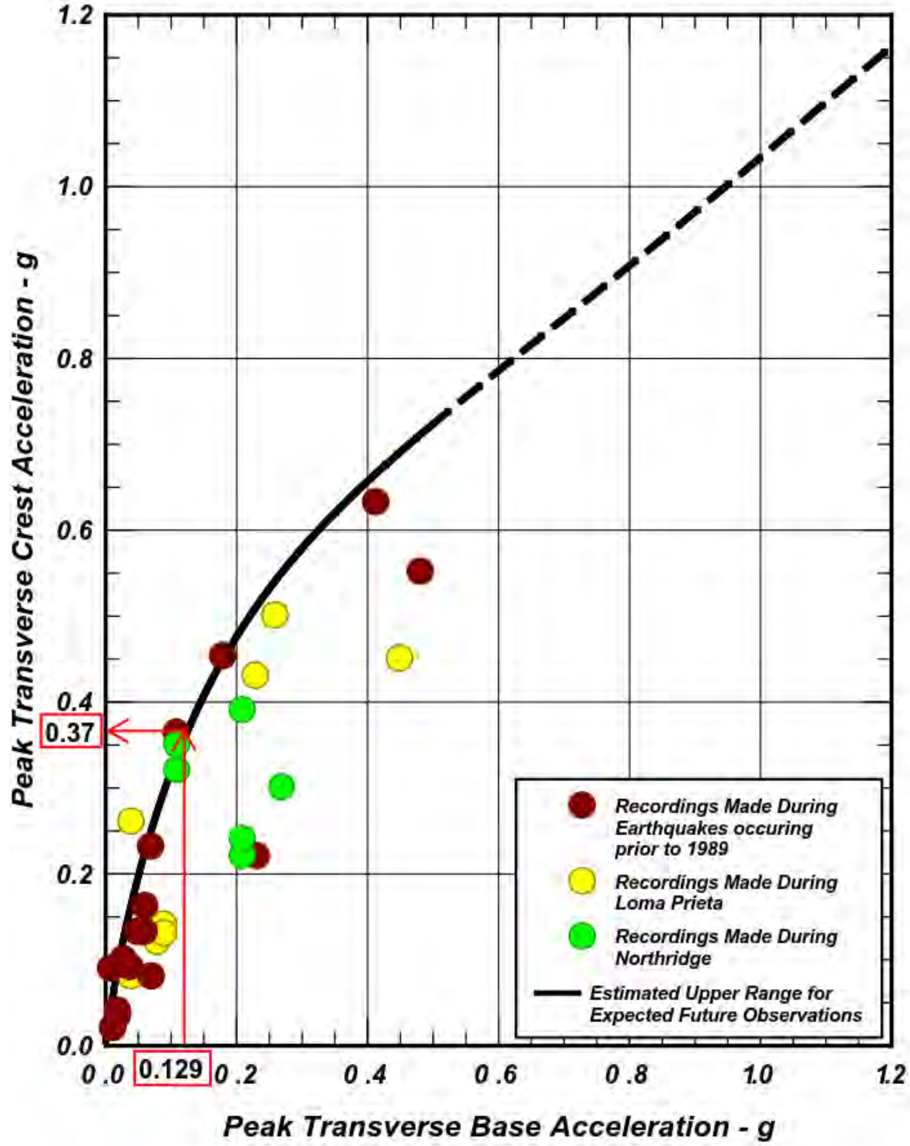

Figure 4. Variations of Peak Transverse Crest Acceleration vs. Peak Transverse Base Acceleration Based on Holzer (1998)

Makdisi and Seed (1977) notes that the "maximum acceleration ratio" varies with the depth of the sliding mass relative to the embankment height. Figure 5 (shown below) presents the relationship between maximum acceleration ratio (k_{max}/u_{max}) and depth of sliding mass (y/h). For deep-seated failure surfaces that involve the entire vertical profile of the dam slope and extend from the crest to the toe or below the toe of the embankment into the foundation soils, the acceleration at the crest can be as low as approximately 34 percent of the maximum value:

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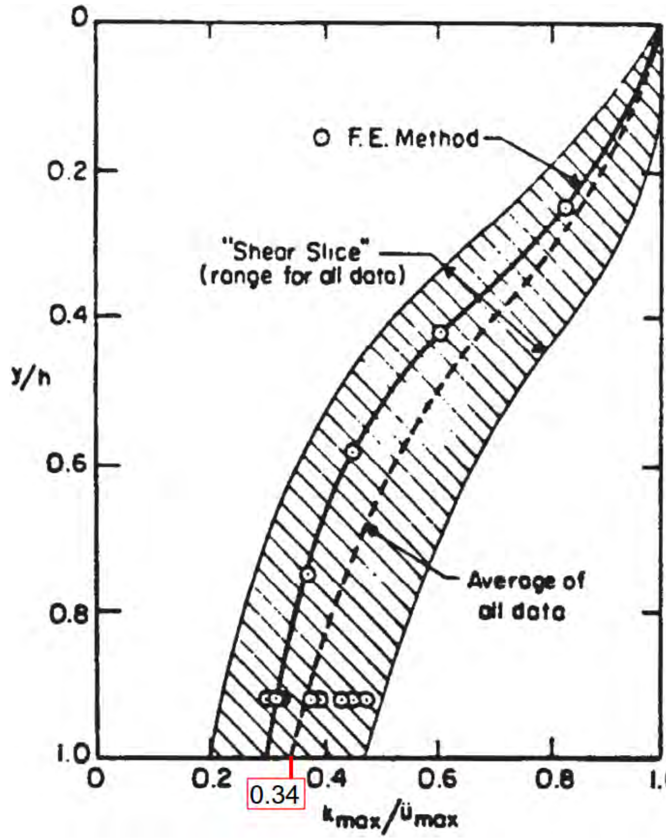

Figure 5. Variation of "Maximum Acceleration Ratio" with Depth of Sliding Mass after Makdisi and Seed (1977)

Therefore:

$$\frac{\mathrm{k_{max}}}{\mathrm{u_{max}}} = 0.34$$

Where: k_{max} = the maximum average acceleration for the potential sliding mass
u_{max} = the maximum crest acceleration

$$\mathrm{k_{max}} = 0.34(\mathrm{u_{max}})$$

$$\mathrm{k_{max}} = 0.34(0.37\mathrm{g})$$

$$\mathrm{k_{max}} = 0.13\mathrm{g}$$

The pseudo-static analyses incorporated a horizontal seismic coefficient of 0.13g.

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The water level in the FAP for the seismic loading analysis was set to EL 5,114.0 ft, to match the long-term, maximum storage pool. The Clay Core and Cutoff Wall materials were assigned undrained strength. Due to the relatively rapid loading induced during the seismic event and these materials' relatively low hydraulic conductivity, it is anticipated that the Clay Core and Cutoff Wall materials would behave in an undrained manner. All, other materials used drained strength parameters.

Liquefaction

The liquefaction factor of safety is evaluated for CCR units that show, through representative soil sampling and construction documentation that soils of the embankment and/or foundation are susceptible to liquefaction. The liquefaction factor of safety is calculated to document whether the CCR unit would remain stable if the soils in the embankment and/or foundation experienced liquefaction.

Post-construction geotechnical exploration of the FAP and Bottom Ash Pond Dams (Harza 1987 and D&M 1999) indicated the Clay Core (embankment) and Alluvium Overburden (foundation) materials have plasticity indexes and fine contents as shown in Table 2 below. Generally, the behavior of soils that have fines contents greater than 35 percent are dominated by the plasticity of their fines (Idriss and Boulanger 2008). Fines with Plasticity Index (PI) less the 7 tend to behave more sand-like and are susceptible to soil liquefaction, while those with PI greater than 7 tend to behave more clay-like and are not susceptible to liquefaction. The lowest measured value of PI for both the Clay Core and Alluvium Overburden is 12, indicating these soils would tend to behave in a clay-like manner during a seismic event and not be susceptible to soil liquefaction. Consequently, a liquefaction factor of safety analysis was not performed for the FAP.

Table 2. Range of Plasticity Index and Fines Content Values for Site Materials

	Plasticity Index		Fines Contents, %	
Material	**Minimum Value**	**Maximum Value**	**Minimum Value**	**Maximum Value**
Clay Core	12	39	48	88
Alluvium Overburden	12	17	30	54

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6 ANALYSIS RESULTS AND CONCLUSIONS

The results of the slope stability analysis are presented in Attachment B. Tables 3 below summarize the results of the safety factor analysis.

Table 3. Safety Factor Results for the FAP Dam

Loading Condition	Required Safety Factor	Calculated Minimum Safety Factor		
		Cross Section 1	Cross Section 2	Cross Section 3
Long-term, maximum storage pool	1.50	1.63	1.53	1.73
Maximum surcharge pool	1.40	1.61	1.52	1.70
Seismic (Pseudo-Static)	1.00	1.08	1.02	1.15

The results of the safety factor analyses show that the FAP Dam exceed the minimum required factors of safety for the long-term, maximum storage pool; the maximum surcharge pool; and the seismic (pseudo-static) loading conditions.

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7 REFERENCES

The following references were used in performing this calculation:

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Arizona Department of Water Resources (ADWR), 1986, *License of Approval*, Cholla Bottom Ash Pond Dam and Reservoir, State File No. 09.28, October 21.

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DESIGN CALCULATION				
Calculation Title: Fly Ash Pond Safety Factor Assessment	**Project Title:** APS Cholla Structural Integrity Assessment	**Project No:** 60445840	**Date:** 4/13/16	**Page No:** Page 15 of 15

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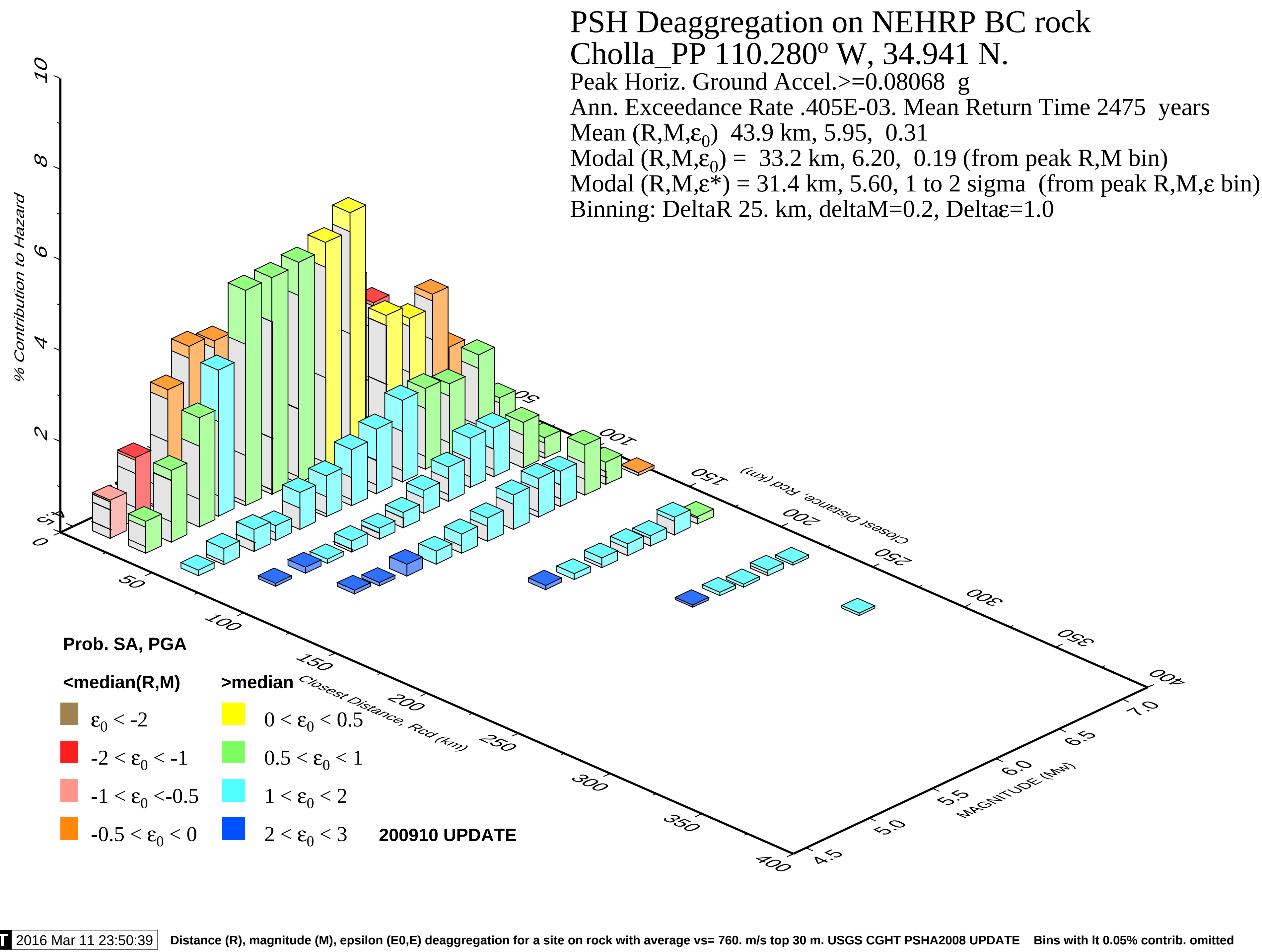
8 ATTACHMENTS

ATTACHMENT A USGS 2008 Seismic PSH Deaggregation

ATTACHMENT B Slope/W Output Figures

ATTACHMENT A

USGS 2008 Seismic PSH Deaggregation


PSH Deaggregation on NEHRP BC rock
Cholla_PP 110.280° W, 34.941 N.
Peak Horiz. Ground Accel.>=0.08068 g
Ann. Exceedance Rate .405E-03. Mean Return Time 2475 years
Mean (R,M,ε0) 43.9 km, 5.95, 0.31
Modal (R,M,ε0) = 33.2 km, 6.20, 0.19 (from peak R,M bin)
Modal (R,M,ε*) = 31.4 km, 5.60, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 25. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
0
2
4
6
8
10
50
100
150
200
250
300
350
400
Closest Distance, Rcd (km)
4.5
5.0
5.5
6.0
6.5
7.0
MAGNITUDE (Mw)
Prob. SA, PGA
<median(R,M)
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 < 0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 2
2 < ε0 < 3
200910 UPDATE
GMT 2016 Mar 11 23:50:39
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

ATTACHMENT B

Slope/W Output Figures

Slope Stability Analysis
Cross Section 1
Fly Ash Pond

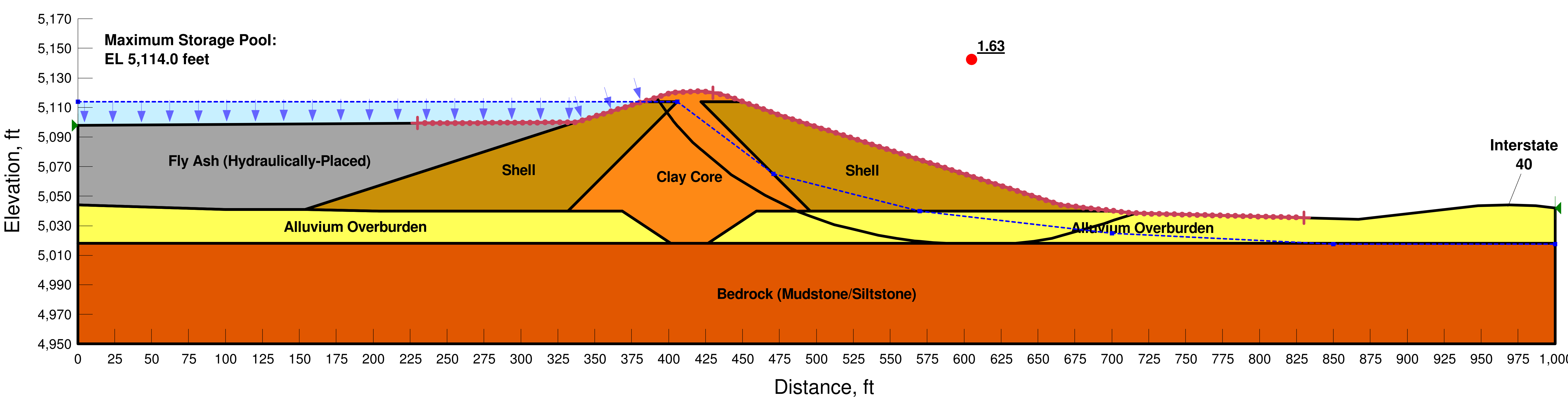
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B1) Static Maximum Storage Pool
File Name: APS Cholla FAP Section 1 - Static.gsz
Date: 4/13/2016
Method: Spencer

Factor of Safety: 1.63

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Clay Core	125 pcf	120 pcf	0 psf	28 °
Shell	130 pcf	125 pcf	0 psf	33 °
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °



Slope Stability Analysis
Cross Section 1
Fly Ash Pond

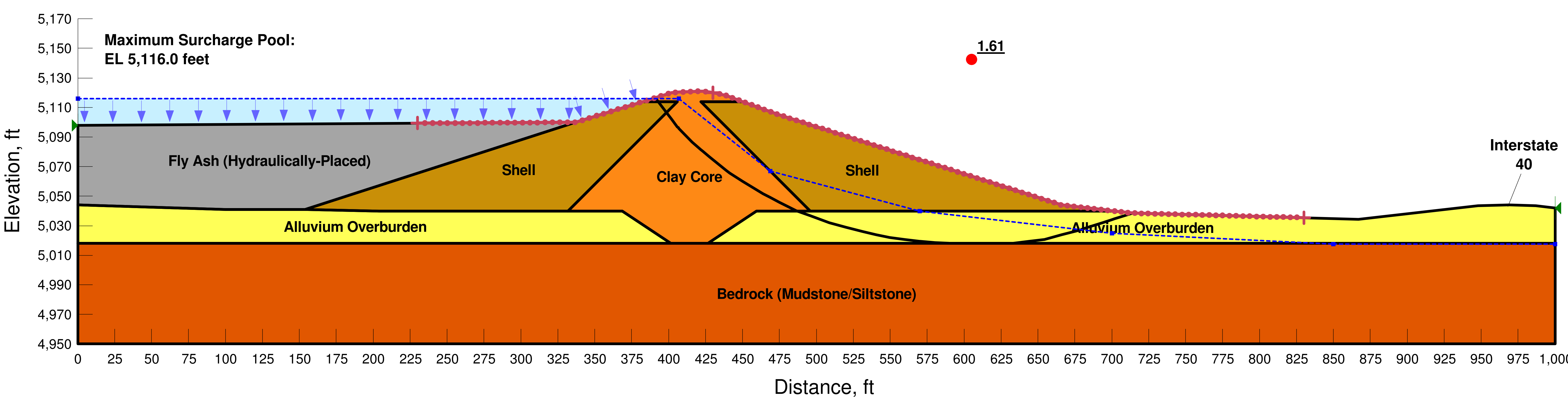
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B2) Static Maximum Surcharge Pool
File Name: APS Cholla FAP Section 1 - Static.gsz
Date: 4/13/2016
Method: Spencer

Factor of Safety: 1.61

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Clay Core	125 pcf	120 pcf	0 psf	28 °
Shell	130 pcf	125 pcf	0 psf	33 °
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °



Slope Stability Analysis
Cross Section 1
Fly Ash Pond

Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

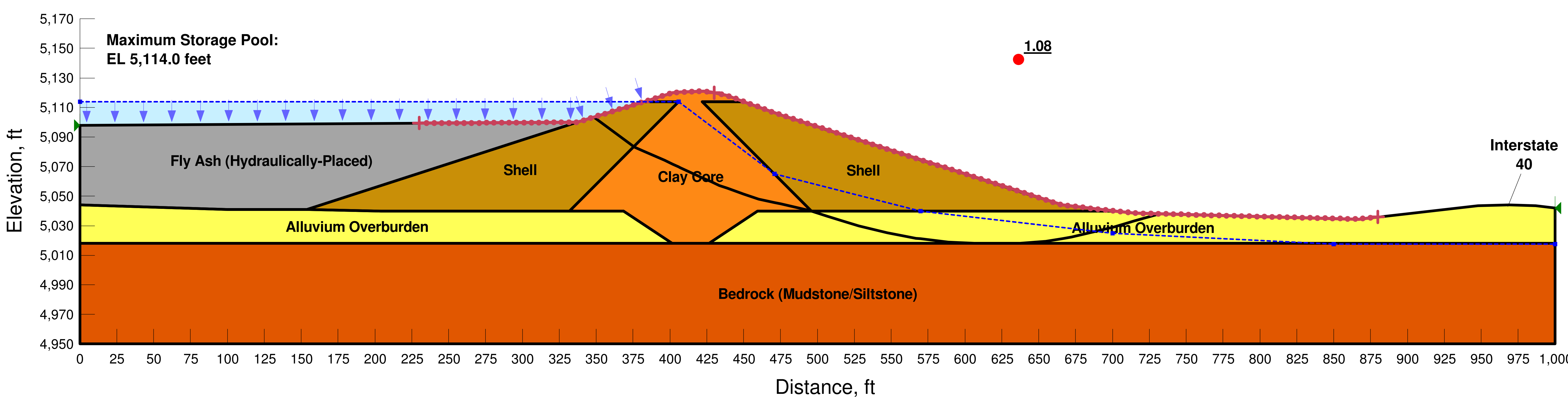
Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B3) Seismic Maximum Storage Pool
File Name: APS Cholla FAP Section 1 - Seismic.gsz
Date: 4/13/2016
Method: Spencer

Factor of Safety: 1.08

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:	Undrained Strength Ratio:
Clay Core	125 pcf	120 pcf	--	--	0.38
Shell	130 pcf	125 pcf	0 psf	33 °	--
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °	--
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °	--
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °	--

Horz Seismic Coef.: 0.13



Slope Stability Analysis
Cross Section 2
Fly Ash Pond

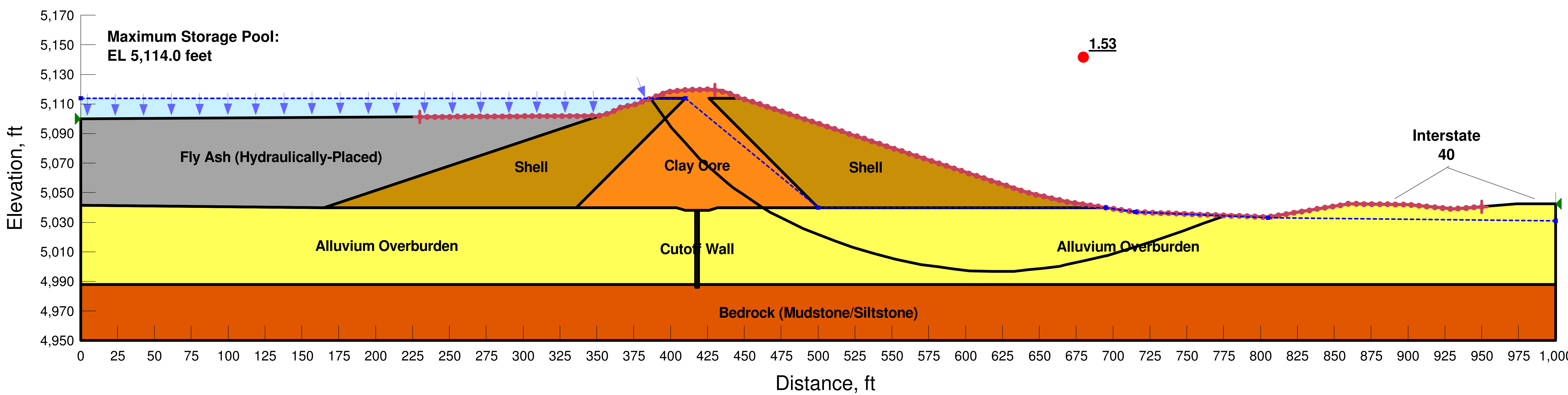
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B4) Static Maximum Storage Pool
File Name: APS Cholla FAP Section 2 - Static.gsz
Date: 6/20/2016
Method: Spencer

Factor of Safety: 1.53

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Clay Core	125 pcf	120 pcf	0 psf	28 °
Shell	130 pcf	125 pcf	0 psf	33 °
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °
Cutoff Wall	106 pcf	106 pcf	0 psf	28 °
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °



Slope Stability Analysis
Cross Section 2
Fly Ash Pond

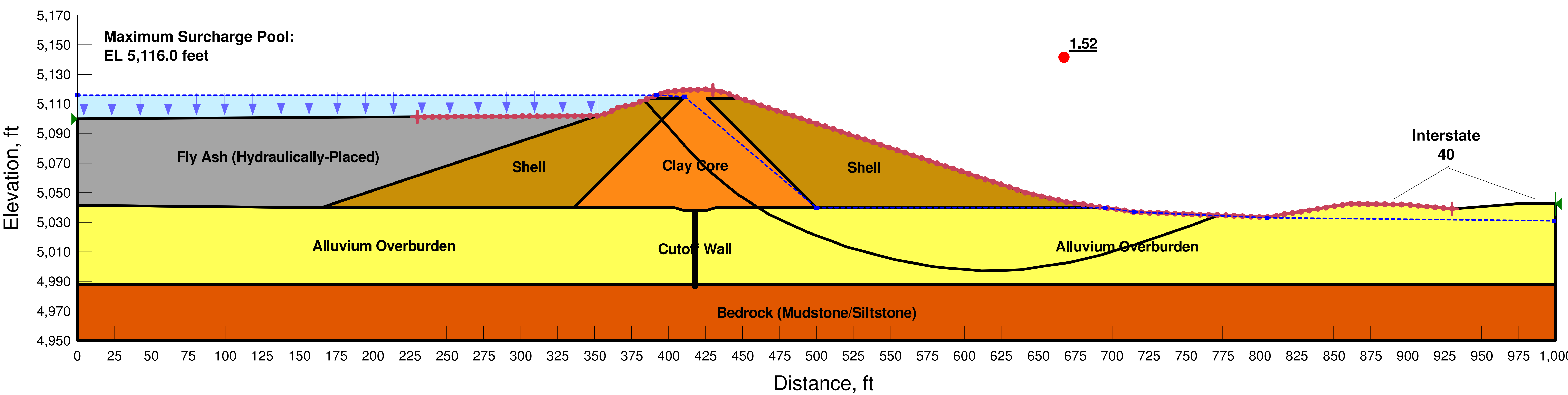
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B5) Static Maximum Surcharge Pool
File Name: APS Cholla FAP Section 2 - Static.gsz
Date: 6/20/2016
Method: Spencer

Factor of Safety: 1.52

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Clay Core	125 pcf	120 pcf	0 psf	28 °
Shell	130 pcf	125 pcf	0 psf	33 °
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °
Cutoff Wall	106 pcf	106 pcf	0 psf	28 °
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °



Slope Stability Analysis
Cross Section 2
Fly Ash Pond

Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

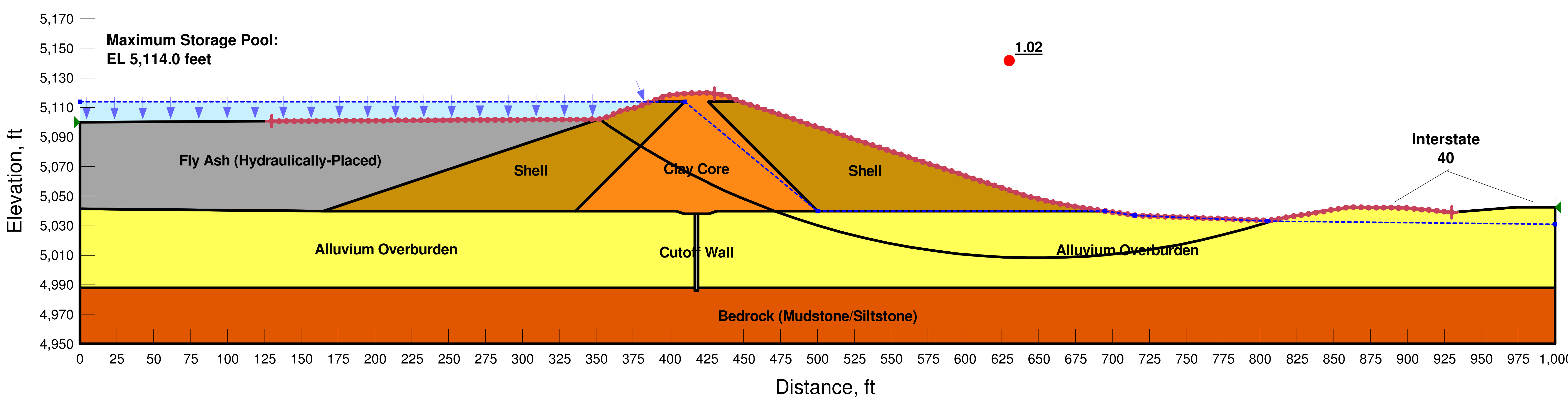
Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B6) Seismic Maximum Storage Pool
File Name: APS Cholla FAP Section 2 - Seismic.gsz
Date: 6/20/2016
Method: Spencer

Factor of Safety: 1.02

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:	Undrained Strength Ratio:
Clay Core	125 pcf	120 pcf	--	--	0.38
Shell	130 pcf	125 pcf	0 psf	33 °	--
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °	--
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °	--
Cutoff Wall	106 pcf	106 pcf	10 psf	0 °	--
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °	--

Horz Seismic Coef.: 0.13



Slope Stability Analysis
Cross Section 3
Fly Ash Pond

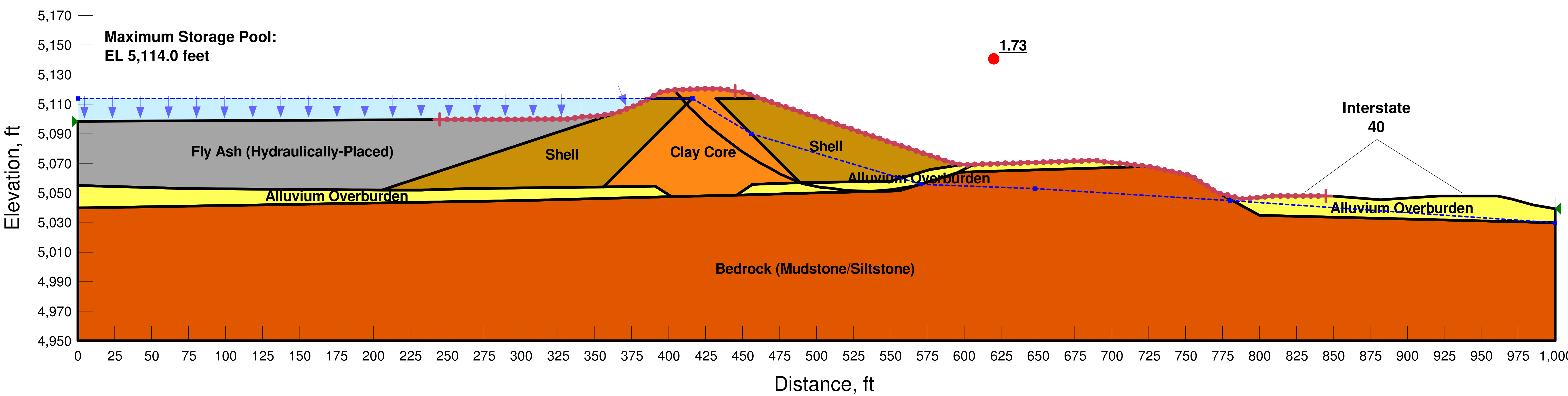
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B7) Static Maximum Storage Pool
File Name: APS Cholla FAP Section 3 - StaticA.gsz
Date: 4/13/2016
Method: Spencer

Factor of Safety: 1.73

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Clay Core	125 pcf	120 pcf	0 psf	28 °
Shell	130 pcf	125 pcf	0 psf	33 °
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °



Slope Stability Analysis
Cross Section 3
Fly Ash Pond

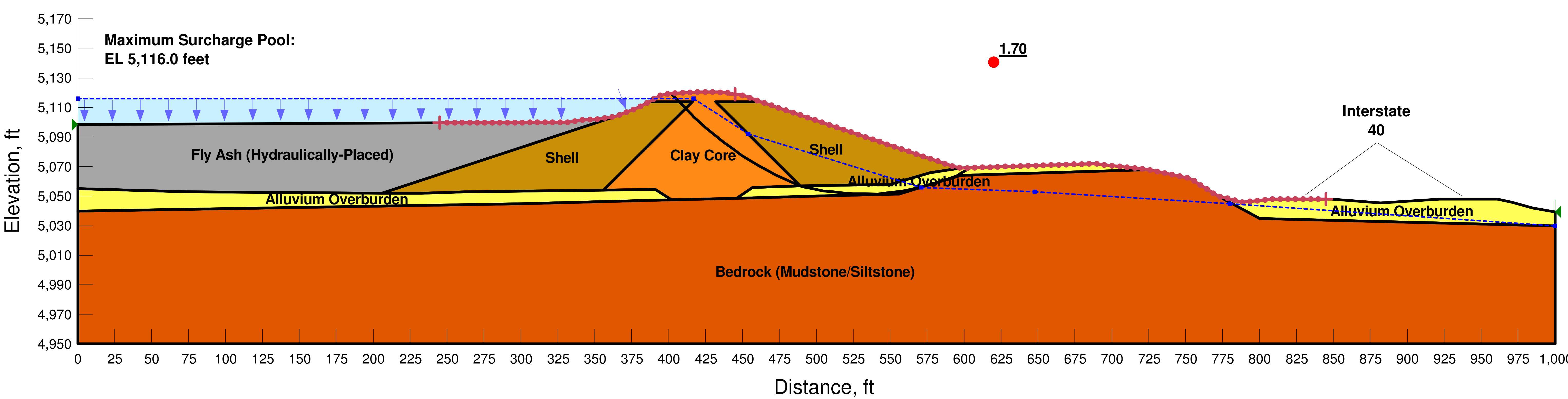
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B8) Static Maximum Surcharge Pool
File Name: APS Cholla FAP Section 3 - StaticA.gsz
Date: 4/13/2016
Method: Spencer

Factor of Safety: 1.70

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Clay Core	125 pcf	120 pcf	0 psf	28 °
Shell	130 pcf	125 pcf	0 psf	33 °
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °



Slope Stability Analysis
Cross Section 3
Fly Ash Pond

Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

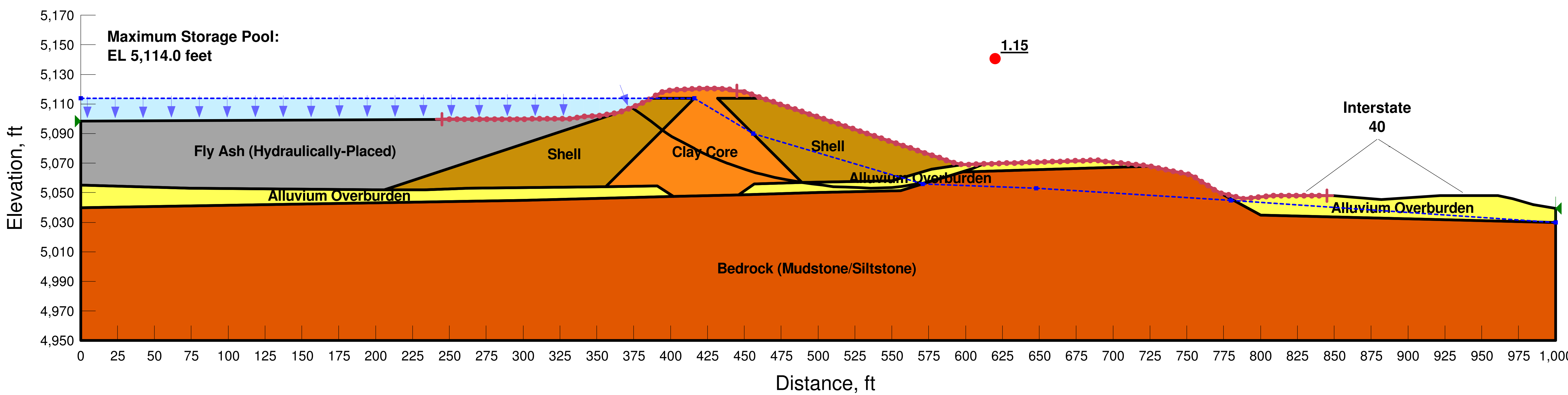
Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B9) Seismic Maximum Storage Pool
File Name: APS Cholla FAP Section 3 - Seismic.gsz
Date: 4/13/2016
Method: Spencer

Factor of Safety: 1.15

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:	Undrained Strength Ratio:
Clay Core	125 pcf	120 pcf	--	--	0.38
Shell	130 pcf	125 pcf	0 psf	33 °	--
Alluvium Overburden	120 pcf	120 pcf	0 psf	26 °	--
Bedrock (Mudstone/Siltstone)	150 pcf	150 pcf	1,000 psf	65 °	--
Fly Ash (Hydraulically-Placed)	90 pcf	90 pcf	0 psf	0 °	--

Horz Seismic Coef.: 0.13



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