Cholla Power Plant
Fly Ash Dam
Periodic Structural Integrity Assessment

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

October 2026

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Figure 1-2: Fly Ash Pond Dam Monitored Instrumentation and Seepage Location Map

Attachment A: APS, 2023b. *Fly Ash Pond Seepage Intercept System Extraction Well Pumps Installation for the Cholla Generating Station*. As-Built. Engineer: WSP Environment & Infrastructure Inc. Relevant sheets.

Attachment B: AECOM, 2021. *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.

1. Introduction

This periodic update to the Structural Integrity Assessment for the Fly Ash Dam at the Cholla Power Plant (Cholla), 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”) and the specific requirements within 40 CFR 257.73 for a periodic (every 5 years) assessment regarding structural integrity.

2. Methodology

The methodology used to prepare this 2026 Periodic Assessment of Hazard Potential Classification, Structural Stability Assessment, and Periodic Safety Factor Assessment for the Fly Ash Dam 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 the inspection report for 2021 was published:
   i. APS, 2022. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2021*. January 2022.
   ii. APS, 2023a. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2022*. January 2023.
   iii. APS, 2024. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2023*. January 2024.
   iv. APS, 2025. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2024*. January 2025.
   v. APS, 2026. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2025*. January 2026.

b) Perform a documented review of each major component of the contributing technical information from the 2021 *Periodic Structural Integrity Assessment* (AECOM 2021; hereafter referred to as the "2021 Certification Report" and incorporated and referenced directly as Attachment B to this document); and

c) Consider and document whether the 2021 Certification Report and its conclusions:
   i. Meet the current reporting requirements of the CCR 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 performed in order to complete this 2026 Periodic Structural Integrity Assessment.

This report documents the results of these considerations, incorporates the 2021 Certification Report as an attachment, identifies any additional technical investigation or evaluations (if needed), and presents an updated certification by the QPE.

3. 2021–2025 Annual Inspection Reports

AECOM reviewed the previous inspection reports and available documents in the operating record that are relevant to the general site conditions and current adequacy and performance of the Fly Ash Dam embankment. No issues were identified during the review that would adversely affect the performance of the system and its compliance, as described in the 2021 Certification 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 in the annual list of “Observed Conditions” in the annual inspection reports since the 2021 Certification Report has been relatively consistent, ranging between 7 and 11 action items during the 5-year timeframe. Action items repeated each year describe the need for ongoing, continued monitoring and maintenance for such observations as scattered animal burrows, ant hills, erosion, vegetation, and seepage. Other action items, such as erosion at specific locations, updates to the Emergency Action Plan, and changes in actively monitored conditions have either been addressed and removed from the list of action items or the proposed solution has evolved from year to year.

The most significant action item mentioned in the annual inspection reports pertains to the turbidity measured at the Geronimo Seep. After Cholla began using a new turbidimeter in 2021, the measured NTU remained elevated compared to the preceding seven years. During the September 2022-December 2022 timeframe, Cholla was engaged in an upgrade project at the Geronimo Seep that consisted of installing new extraction wells and new piping for the system. Turbidity measurements recorded after construction was complete have risen overall since construction was completed. The flow rate measured at the Geronimo Seep has generally been directly correlated to the elevation of the Fly Ash Pond such that relatively higher seepage rates have been recorded during times when the Fly Ash Pond was relatively higher. The flow rates measured at the Hunt totalizer further downstream of the Fly Ash Dam have been less variable. As it is not associated with an increased seepage rate and began after the seepage collection upgrades, the increased turbidity is not currently considered to be a sign of structural weakness.

The record of settlement monument movements shows no changes of significance over the five-year reporting period. While most of the settlement monuments are below the design crest elevation and they continued to indicate continued settlement between 2021 and 2025, half of the total settlement took place within the first 10-15 years after the Fly Ash Dam was in service. The lowest elevation recorded between 2021 and 2025 was at settlement monument M-5B when it was surveyed at EL 5117.429 feet (NGVD29) in June 2025. In general, the greatest settlement has been recorded at the settlement monuments farthest from the abutments and located over the thickest portions of alluvium. Although settlement is continuing, APS has reduced the Fly Ash Pond water level by 8.5 feet from EL 5085.609 feet (NGVD29) in January 2021 to EL 5077.058 feet (NGVD29) in December 2025. The current water surface of the reservoir in the Fly Ash Pond is more than 40 feet below settlement monument M-5B, resulting in a sufficient volume of storage available to contain the design run-on.

The settlement monuments along the western portion of the embankment indicate the most downstream movement, while the settlement monuments along the southern portion of the embankment have indicated upstream movement. The historic transition between upstream and

downstream movement appears to be between settlement monuments M-5 and M-6, near the Geronimo Knob.

In May 2021, APS abandoned and replaced piezometers F-91 (believed to have a faulty seal), F-92 (blocked at a depth of 23 feet), F-111 (unresponsive during recent aquifer tests), and well W-123 (was likely screened across the Moqui member of the Moenkopi formation and the overlying alluvium). Subsequent water level measurements in the replacement wells appeared to be reliable.

At various times during the 2021-2025 timeframe, piezometers F-105 (screened in the alluvium on the upstream side of the cutoff wall), F-110 (screened in the alluvium underlying the dam), F-117 (screened in the Moqui member of the Moenkopi Formation), and all four piezometers screened in the core of the dam (F-123, F-124, F-128, and F-132) were higher than the water level measured in the Fly Ash Pond. The water level in piezometer F-129 (screened in the shell) was above the water level in the Fly Ash Pond for most of 2021; however, this was likely due to a shift in the deposition path along the beach and not related to slope instability.

As APS continued to reduce the water level in the Fly Ash Pond, the water levels recorded in the embankment, shell, and alluvium piezometers have also decreased. In early 2024 through 2025, the water levels recorded in several piezometers (F-89, F-100, F-104, F-105, F-108, F-110, F-126, F-127, F-129, F-130, F-131, and F-134) began to decrease rapidly. The steep declines were attributed to the removal of drainable pore water by various phases of wells installed in the beach beginning in 2023 (Section 4.3), and cessation of discharge to the Fly Ash Pond in March 2025.

The 2021-2025 Annual Inspection Reports also provide information on the water levels in the Fly Ash Pond, the approximate impounded volume in the Fly Ash Pond at the time of each inspection, and other observations pertaining to the operation of the CCR unit throughout the previous year. No conditions associated with structural weakness were identified during any of the 2021-2025 annual inspections.

4. 2021 Certification – Review by Section

Other than as described in the remainder of this section, the details presented in this section of the 2021 Certification Report have been assessed to represent current conditions adequately and satisfy the requirements of the CCR Rule.

4.1 "1.4 Facility Description"

The 2021 Certification Report stated that the details presented in this section of the 2016 Certification Report adequately represented the then-current conditions. Since 2021, there have been several changes to the operation of the power plant and, in turn, the Fly Ash Dam. The following paragraphs present these changes, reiterate applicable statements, and replace the Facility Description presented in the 2016 Certification Report.

The Cholla Power Plant is an electric generating station located in the town of Joseph City, Navajo County, Arizona. The station currently consists of four inactive coal-fired units. Units 1, 2, and 3 are owned by APS and Unit 4 is owned by PacifiCorp. Cholla began operation of Unit 1 in 1961 and Units 2, 3, and 4 were constructed between 1976 and 1981. Unit 2 was removed from service on October 1, 2015. Unit 4 was removed from service on December 28, 2020. Units 1 and 3 were removed from service in March 2025 and APS ceased discharging to the Fly Ash Pond by June 30, 2025. Pre-closure activities for the Fly Ash Pond closure construction continued throughout the 2021-2026 time period, with closure construction beginning in 2026. In July 2026, APS announced plans to convert Units 1 and 3 to burn natural gas, with construction of the gas conversion expected to begin in 2028.

CCR generated at Cholla was disposed in two primary surface impoundments: the Fly Ash Pond located approximately 1.5 miles east of Cholla, and the Bottom Ash Pond located approximately 2 miles north of Cholla. This assessment evaluates the structural integrity of the Fly Ash Pond Dam.

The Fly Ash Pond received discharges from the following sources: Slurry Disposal; the General Water Sump; the Fly Ash Pond Seepage Collection System; Sedimentation Pond Solids; the 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 were pumped as slurry through three 6-inch diameter pipes into the impoundment where the solids settled out and the remaining water evaporated. There was no means to return the excess water to Cholla for reuse.

The Fly Ash Pond has a total surface area of about 420 acres and storage capacity of approximately 16,500 acre-feet when at its permitted maximum storage pool water level of EL 5114 feet (NGVD29; ADWR 1986). The impoundment is surrounded on its west, northwest, and east sides by rock outcrops of mudstones, siltstones, and sandstones. On the south and southwest sides, 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 Dam is 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), the Cholla Power Plant, a downstream 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 4,583 feet in length and is composed of two linear segments. The westernmost segment starts at the Right Abutment and extends approximately 3,100 feet to a rock outcropping referred to as Geronimo Knob. At Geronimo Knob the dam centerline pivots approximately 40 degrees toward the north to form an east-west linear segment which extends to the Left Abutment. The location of the dam's maximum height is between the Right Abutment and Geronimo Knob, where the maximum toe to crest is 80 feet. Both the upstream and downstream slopes are inclined at a 3 horizontal to 1 vertical (3H:1V) angle with riprap facing to prevent erosion. The design crest width is 24 feet. The design crest is EL 5120 feet (NGVD29), producing 6 feet of total freeboard above the maximum permitted storage pool water level; however, APS monitors the crest elevation by surveying 16 settlement monuments on a monthly basis. As of July 2026, the lowest settlement monument along the crest was measured to be at EL 5117.424 feet (NGVD29). The most recent pond level was recorded to be EL 5077.729 feet (NGVD29) on January 2, 2026. Since January, the water level in the Fly Ash Pond has been too low to access safely to measure.

The Fly Ash 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 contain fully the expected 2.0 feet of storm runoff on top of the maximum permitted storage pool water level (EL 5114 feet NGVD29). The water level resulting from storing the design storm runoff on top of the maximum permitted storage pool is defined as the maximum surcharge pool water level, and is estimated to be at EL 5116 feet (NGVD29) (Ebasco 1976). The current surface of the reservoir (water below approximate EL 5078 feet NGVD29 and solids below EL 5106 feet NGVD29) in the Fly Ash Pond is more than 8 feet below the maximum permitted storage pool water level and is expected to remain so since APS no longer discharges to the Fly Ash Pond. The volume associated with the IDF would be fully contained below the maximum permitted storage pool water level, primarily in the northern end of the reservoir.

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 feet. In the center portion of the dam where the depth to bedrock is greater than 20 feet, a slurry cutoff wall extends from the clay core 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 Fly Ash Pond (historically) or the Evaporation Pond (starting in 2025).

The Fly Ash Dam has no intake or outlet structures. The Evaporation Pond construction was completed in 2025, at which time APS permanently redirected seepage flows from the Hunt and Geronimo Seeps to the Evaporation Pond via a connection to the Charlie Line in the Fly Ash Dam piping.

Piezometers, settlement monuments, and flow measurement devices (totalizers and sumps) 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. The

piezometer, survey monuments, and totalizers are read at intervals not exceeding 30 days per the requirements of 40 CFR 257.83(a)(1)(iii). The locations of the piezometers, survey monuments, and flow totalizers are shown on Figure 1-2 (attached).

Inspections required under 40 CFR 257.83(a)(1)(i) are conducted 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 document 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 pertaining to 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. These reports are posted to the APS CCR website.

4.2 "2 Hazard Potential Classification"

The assessments presented in this section of the 2016 and 2021 Certification Reports adequately represent current conditions and satisfy the requirements of the CCR Rule for existing CCR units.

Based on a review of the information presented in the 2016 and 2021 Certification Reports and annual dam safety inspection reports (APS 2022-2026), the Fly Ash Dam currently satisfies the criteria for a "high hazard potential CCR surface impoundment" classification.

4.3 "3 History of Construction"

The information presented in this section of the 2021 Certification Report and incorporated from the 2016 Certification Report adequately represents the conditions up to October 2021. Descriptions of significant additional construction since 2021 are provided in the following paragraphs.

Maintenance and Repair of Existing Instrumentation

In May 2021, APS replaced extraction wells GSX-1 and GSX-2; monitoring wells W-123, W-126, and W-307; and piezometers F-91, F-92, and F-111 as part of a broader wells and piezometer abandonment and replacement program. APS also installed four new extraction wells (EW-01, EW-02, EW-03, and EW-04) in the Geronimo Seep area (Wood 2022). APS also installed new piping between the wells and the Geronimo Seep discharge line as needed. Relevant sheets from the as-built drawing set are included as Attachment A to this Certification Report to augment the as-built drawings included in the 2016 Certification Report (APS 2023b).

New Construction Activities

Between June 2020 and December 2021, APS relocated CCR from Ash Pond 1 (south of I-40) to the eastern portion of the Fly Ash Pond reservoir. The CCR was placed between EL 5090 feet (NAVD88) and EL 5114 feet (NAVD88). The CCR was stockpiled above the Fly Ash Pond elevation at the time.

Between August 2023 and May 2025, APS relocated CCR from the Bottom Ash Monofill to the eastern portion of the Fly Ash Pond reservoir. The CCR was placed on top of the CCR relocated from Ash Pond 1, between EL 5114 feet (NAVD88) and EL 5184 feet (NAVD88).

APS began preparing to close the Fly Ash Pond in 2023 when it installed the first phase of extraction and observation wells in the impounded CCR to facilitate water removal. By the 2025 inspection, APS had installed 89 vertical extraction wells screened in CCR, 28 observation wells (both single and nested wells), and a 1,350-foot long horizontal well along the bottom of the CCR. The wells initially discharged back into the Fly Ash Pond until January 2024 when the EcoVap system was installed within the reservoir near the Left Abutment, but have discharged to the East Temporary Basin since it was completed in June 2025. The East Temporary Basin is a 17.3-acre pond lined with a 60-mil scrim reinforced polyethylene geomembrane divided into a north cell and a south cell. The East Temporary Basin was constructed in the northeast corner of the Fly Ash Pond to facilitate water evaporation and the overall closure process. The nominal crest of the East Temporary Basin is EL 5100 feet (NAVD88). The bottoms of the north and south cells are flat at a nominal elevation of 5087.0 feet (NAVD88).

4.4 “4 Structural Stability Assessment”

The assessment presented in this section of the 2021 Certification Report, incorporated from the 2016 Certification Report, adequately represents current or potential conditions and satisfy the requirements of the CCR Rule for existing CCR units.

AECOM assesses that the design, construction, operation, and maintenance of the Fly Ash Pond Dam are consistent with recognized and generally accepted good engineering practice.

4.5 “5 Safety Factor Assessment”

The safety factor assessment completed for the original 2016 Certification Report, incorporated into the 2021 Certification Report, featured three critical cross-sections – one in the western portion of the main embankment, one near the center of the main embankment, and one at the Geronimo Knob. The critical cross-section configurations, as well as the material properties assigned in the model, remain the same as used in the 2016 analyses and no revisions are required for these aspects of the assessment.

The original 2016 Safety Factor Assessment used a horizontal seismic coefficient that was developed using the procedure described in the National Earthquake Hazards Reduction Program (NEHRP) (USGS 2008) and adjusts the peak horizontal bedrock acceleration (PGA) for site conditions to produce the site-adjusted ground surface acceleration (PGA_M) using the site classification definitions in the 2003 International Building Code (IBC) (ICC 2004). In the 2016 Safety Factor assessment, the Fly Ash Dam site was assigned a Site Classification of “D – Stiff soil profile” in the analysis, and the resulting PGA was 0.0807g. The site coefficient for Site Class D and PGA ≤ 0.1 was 1.6. Multiplying the site coefficient (1.6) and the PGA for rock (0.0807g) produced a PGA_M = 0.129g, which was then amplified to 0.13g using Holzer’s (1998) procedure to estimate the peak transverse acceleration at the crest and Makdisi and Seed’s (1977) relationship between the maximum acceleration ratio and the depth of the sliding mass.

In 2021, ASCE released ASCE 7-22 (ASCE 2022), which refers the user to the ASCE Hazard Tool. The ASCE Hazard Tool (https://ascehazardtool.org/) for ASCE 7-22 explicitly provides the PGA_M given the Site Classification. ASCE 7-22 divides the Site Classification further than the IBC (ICC 2004) by adding the intermediate "BC," "CD," and "DE" classifications based on a finer delineation of the average shear wave velocity profile in the upper 100 feet of the site. While no shear wave velocity data are available for the Fly Ash Dam, the original design drawings (APS Drawings F-145586, F-145587, and G-145783) indicate approximately 50-60 feet of alluvium (silty clay, sand, and gravel) overlying rock. Assuming that the average shear wave velocity in the upper 100 feet of the Fly Ash Dam footprint can be classified as "D – Medium Dense Sand or Stiff Clay" under the ASCE 7-22 Site Classification, the PGA_M for the Fly Ash Dam is 0.12g. This value is less than the 0.13g value used in the 2016 Certification Report for PGA_M. Consequently, the amplified horizontal acceleration applied in a slope stability model would be proportionately lower as well, resulting in higher safety factors for the "Seismic Loading" case than the results documented in the 2016 and 2021 Certification Reports.

The calculated safety factors for the Fly Ash Dam cross section in the 2016 and 2021 Certification Report exceed the required minimum values for the long-term, maximum storage pool; the maximum surcharge pool; and the seismic (pseudo-static) loading conditions. The conclusion in the 2016 Certification Report that the Clay Core and Alluvium Overburden are not susceptible to liquefaction remains applicable. Therefore, no revisions to the stability or liquefaction calculations are necessary.

The information presented in this section of the 2021 Certification Report, referencing the 2016 Certification Report, adequately represents current conditions and satisfies the requirements of the CCR Rule for existing CCR units.

4.6 "6 Conclusions"

The conclusions presented in this section of the 2021 Certification Report adequately represent current conditions and satisfy the requirements of the CCR Rule for existing CCR units.

5. Recommended Additional Technical Investigations or Evaluations

AECOM recommends that APS continue to monitor the turbidity at the Geronimo Seep. APS should investigate the recent increase in turbidity, including the particles causing the turbidity and their origin.

AECOM also recommends continuing to monitor the settlement monuments on the crest as the water level in the reservoir recedes and closure construction begins.

6. Conclusion

This Certification Report and its conclusions meet the current reporting requirements of the CCR 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 basis for the periodic structural integrity assessment presented in this report is the information presented in the 2016 and 2021 Certification Reports and relies on and incorporates any Limitations expressed therein.

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. References

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 2016.

AECOM, 2021. *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.

APS, 2022. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2021*. January 2022.

APS, 2023a. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2022*. January 2023.

APS, 2023b. *Fly Ash Pond Seepage Intercept System Extraction Well Pumps Installation for the Cholla Generating Station*. As-Built. Engineer: WSP Environment & Infrastructure Inc. Relevant sheets.

APS, 2024. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2023*. January 2024.

APS, 2025. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2024*. January 2025.

APS, 2026. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2025*. January 2026.

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American Society of Civil Engineers (ASCE), 2022. ASCE 7 Hazard Tool. Accessed August 8, 2026. https://ascehazardtool.org/.

Holzer, Thomas L, 1998. *The Loma Prieta Earthquake of October 17, 1989, Earth Structures and Engineering Characterization of Ground Motion*. USGS Professional Paper 1552-D, U.S. Government Printing Office.

International Code Council (ICC), Inc., 2004. *2003 International Building Code*. Fifth printing. May.

Makdisi, F.I. and H. Bolton Seed, 1977. *A Simplified Procedure for Estimating Earthquake-Induced Deformations in Dams and Embankments*. Report No. UCB/EERC-77/19. University of California, Berkeley. August.

9. 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, Jeffery B. Heyman, 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 and 2021 Certification Reports, were conducted in accordance with the requirements of 40 CFR 257.73.

Jeffery B. Heyman, P.E.
Printed Name



October 16, 2026
Date

Figure 1-2:
Fly Ash Pond Dam Monitored Instrumentation and Seepage Location Map

Attachment A:

APS, 2023b. *Fly Ash Pond Seepage Intercept System Extraction Well Pumps Installation for the Cholla Generating Station*. As-Built. Engineer: WSP Environment & Infrastructure Inc. Relevant sheets.

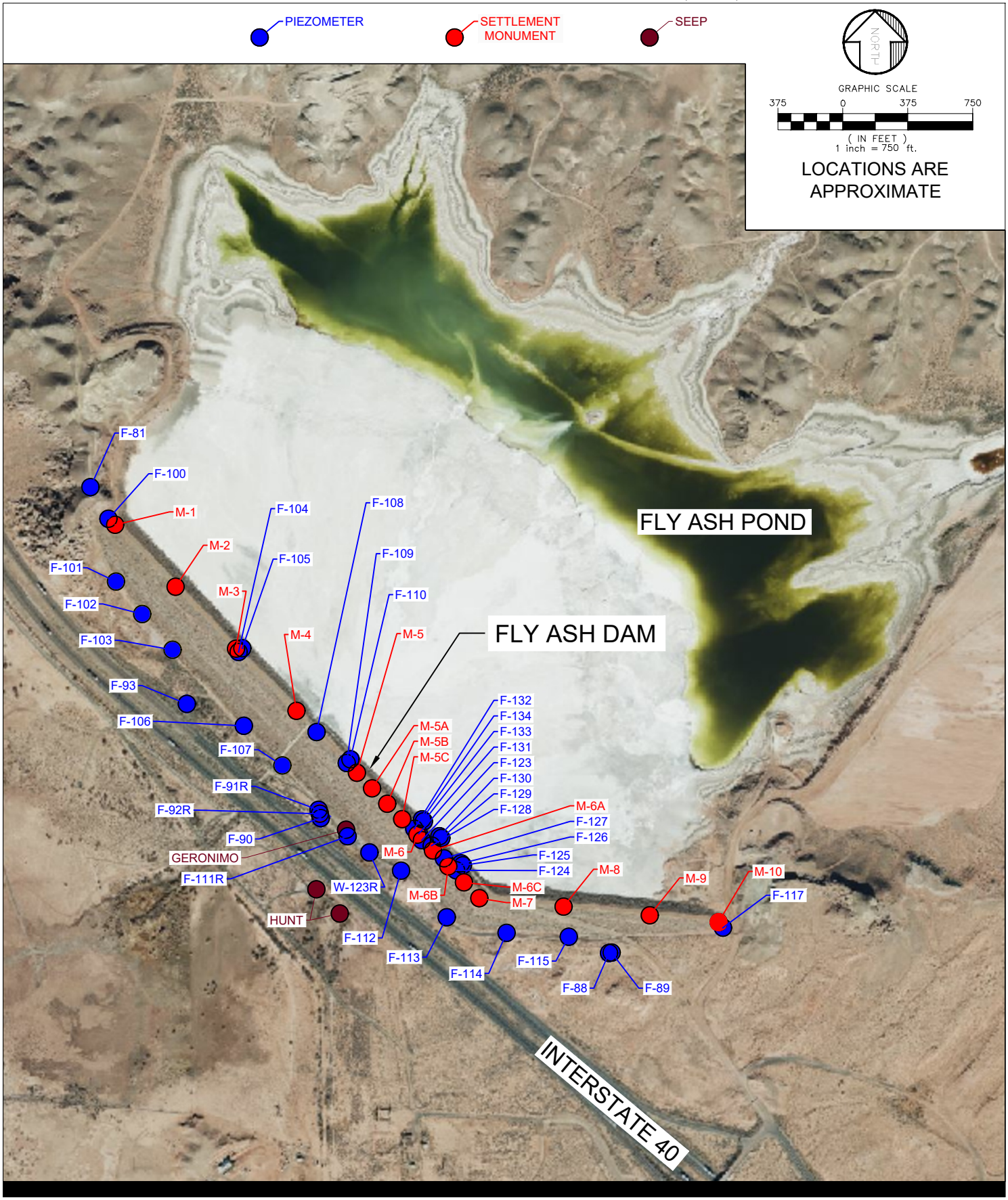
Attachment B:

AECOM, 2021. *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.

Figure 1-2
Fly Ash Pond Dam Monitored Instrumentation and Seepage Location Map

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Filename: AECOM\APS - GENERAL\2026 CHOLLA 5-YEAR RECERTIFICATIONS\FLY ASH DAM\STRUCTURAL INTEGRITY\FIGURE 1-2 (20261009).DWG

ANSI A 8.5" x 11"



CHOLLA POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE

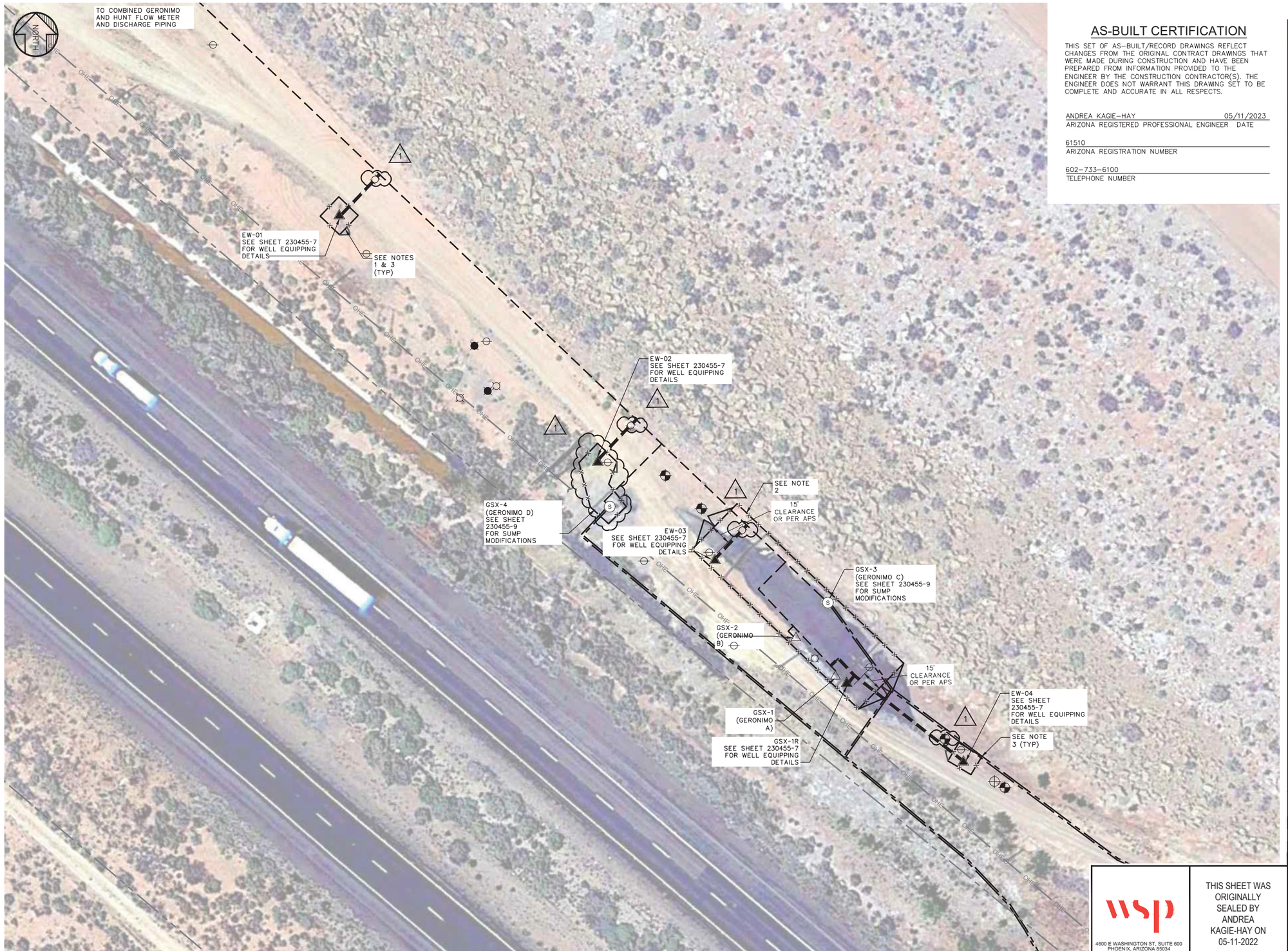
FLY ASH POND DAM
MONITORED INSTRUMENTATION
AND SEEPAGE LOCATION MAP

AECOM

FIGURE 1-2

ATTACHMENT A

APS, 2023b. *Fly Ash Pond Seepage Intercept System Extraction Well Pumps Installation for the Cholla Generating Station*. As-Built. Engineer: WSP Environment & Infrastructure Inc. Relevant sheets.



PLAN VIEW
SCALE: 1"=25' (FULL SIZE)

AS-BUILT CERTIFICATION

THIS SET OF AS-BUILT/RECORD DRAWINGS REFLECT CHANGES FROM THE ORIGINAL CONTRACT DRAWINGS THAT WERE MADE DURING CONSTRUCTION AND HAVE BEEN PREPARED FROM INFORMATION PROVIDED TO THE ENGINEER BY THE CONSTRUCTION CONTRACTOR(S). THE ENGINEER DOES NOT WARRANT THIS DRAWING SET TO BE COMPLETE AND ACCURATE IN ALL RESPECTS.

ANDREA KAGIE-HAY 05/11/2023
ARIZONA REGISTERED PROFESSIONAL ENGINEER DATE

61510
ARIZONA REGISTRATION NUMBER

602-733-6100
TELEPHONE NUMBER

DRAWING REFERENCES

1. SEE THE FOLLOWING SHEETS FOR DESIGN INFORMATION FOR WELLS, SUMPS, AND TRENCHES RELATED TO THIS AREA.
   A. D-114479-1
   B. G-114437-1
   C. G-114438-1
   D. G-114438-3
   E. G-114438-4

NOTES:

1. INSTALL CHAIN LINK FENCE WITH ACCESS GATE AROUND NEW WELLS WHERE SHOWN. MATCH EXISTING FENCING FOR GENERAL DESIGN, SIZING, GATE SIZE AND LOCATION AROUND INDIVIDUAL WELLS. SEE ELECTRICAL DRAWINGS FOR FENCE GROUNDING/CATHODIC PROTECTION REQUIREMENTS.
2. REINSTALL AND EXTEND EXISTING LARGE ENCLOSURE BY DEMOLISHING AND SALVAGING EXISTING FENCING, IF IN GOOD CONDITION, AND INSTALL NEW AS NEEDED TO ENCLOSE EW-03 AND GSX-1R WITHIN EXISTING. PROVIDE TWO DOUBLE-SWING VEHICLE GATES ON EITHER END FOR TRUCK ACCESS. SEE ELECTRICAL DRAWINGS FOR FENCE GROUNDING/CATHODIC PROTECTION REQUIREMENTS.
3. INSTALL FENCE OUTSIDE OF ROADWAY.

GENERAL CONSTRUCTION NOTES:

1. CONTRACTOR TO LOCATE AND VERIFY EXISTING UTILITIES AND UNDERGROUND PIPE. TRENCH LOCATION IS BASED ON BEST AVAILABLE INFORMATION, BUT MAY BE INACCURATE.
2. CONTRACTOR TO PROTECT IN PLACE ALL EXISTING UTILITIES AND UNDERGROUND PIPE.
3. ALL PROPOSED BELOW-GRADE DEWATERING PIPE SHALL BE HDPE PER SPECIFICATIONS.
4. CONTRACTOR TO MAINTAIN A MINIMUM COVER OF 3'.

LEGEND:

- POWERPOLE
- INSTALLED EXTRACTION WELL - TO BE EQUIPPED
- EXISTING ONLINE EXTRACTION WELL - UPGRADES PROPOSED
- EXISTING EXTRACTION WELL - ABANDONED
- EXISTING MONITORING WELL
- MONITORING WELL - ABANDONED
- EXISTING PIEZOMETER
- PIEZOMETER - ABANDONED
- VWP PIEZOMETER
- EXISTING SUMP LOCATION - UPGRADES PROPOSED
- NEW CORP STOP VALVE WITH COVER
- ABOVE-GROUND EXISTING DEWATERING LINE
- UNDER-GROUND EXISTING DEWATERING LINE
- ABOVE-GROUND PROPOSED DEWATERING LINE
- UNDER-GROUND PROPOSED DEWATERING LINE
- EXISTING SEEPAGE COLLECTION TRENCH
- OVERHEAD ELECTRIC LINE
- EXISTING FENCE
- PROPOSED FENCE
- LEASE BOUNDARY

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
1	5/23	AS-BUILT	DDM		RAW	ALK	MH	CHC08903
0.b	5/22	ISSUED FOR CONST	ACK	ALK	TSL	RAW	MH	CHC08903
0.a	3/22	ISSUED FOR BID	CRJ	ALK	TSL	RAW	MH	CHC08903

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PHOENIX, ARIZONA 85034
PHONE: 602-733-6000

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CHOLLA GENERATING STATION
FAP SEEPAGE SYS EXTRACTION WELLS PUMP INSTALLATION
GERONIMO SYSTEM PLAN

aps

SCALE 1"=25' DATE 05/11/2022

DWN	EXD	APPROVED	W A
CRJ	TSL	MAREN HENLEY	CHC08903
CHD	RVWD	DRAWING APPROVED BY	
ALK	RAW		

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	C	47	BP	AP	230454	2

Figure 15



PLAN VIEW
SCALE: 1"=20' (FULL SIZE)

AS-BUILT CERTIFICATION

THIS SET OF AS-BUILT/RECORD DRAWINGS REFLECT CHANGES FROM THE ORIGINAL CONTRACT DRAWINGS THAT WERE MADE DURING CONSTRUCTION AND HAVE BEEN PREPARED FROM INFORMATION PROVIDED TO THE ENGINEER BY THE CONSTRUCTION CONTRACTOR(S). THE ENGINEER DOES NOT WARRANT THIS DRAWING SET TO BE COMPLETE AND ACCURATE IN ALL RESPECTS.

ANDREA KAGIE-HAY 05/11/2023
ARIZONA REGISTERED PROFESSIONAL ENGINEER DATE

61510
ARIZONA REGISTRATION NUMBER

602-733-6100
TELEPHONE NUMBER

DRAWING REFERENCES

1. SEE THE FOLLOWING SHEETS FOR DESIGN INFORMATION FOR WELLS, SUMPS, AND TRENCHES RELATED TO THIS AREA.
   A. D-114438-1
   B. G-114438-3
   C. G-114438-4
   D. G-143718-1
   E. G-143718-2

GENERAL CONSTRUCTION NOTES:

1. CONTRACTOR TO LOCATE AND VERIFY EXISTING UTILITIES AND UNDERGROUND PIPE. TRENCH LOCATION IS BASED ON BEST AVAILABLE INFORMATION, BUT MAY BE INACCURATE.
2. CONTRACTOR TO PROTECT IN PLACE ALL EXISTING UTILITIES AND UNDERGROUND PIPE.
3. ALL PROPOSED BELOW-GRADE DEWATERING PIPE SHALL BE HDPE PER SPECIFICATIONS.
4. CONTRACTOR TO MAINTAIN A MINIMUM COVER OF 3'.

LEGEND:

- POWERPOLE
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- EXISTING ONLINE EXTRACTION WELL - UPGRADES PROPOSE
- EXISTING EXTRACTION WELL - ABANDONED
- EXISTING MONITORING WELL
- MONITORING WELL - ABANDONED
- EXISTING PIEZOMETER
- PIEZOMETER - ABANDONED
- VWP PIEZOMETER
- (S) EXISTING SUMP LOCATION - UPGRADES PROPOSED
- ABOVE-GROUND EXISTING DEWATERING LINE
- UNDER-GROUND EXISTING DEWATERING LINE
- ABOVE-GROUND PROPOSED DEWATERING LINE
- UNDER-GROUND PROPOSED DEWATERING LINE
- EXISTING SEEPAGE COLLECTION TRENCH
- OHE OVERHEAD ELECTRIC LINE
- EXISTING FENCE
- PROPOSED FENCE
- LEASE BOUNDARY

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
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0.a	3/22	ISSUED FOR BID	CRJ	ALK	TSL	RAW	MH	CHC08903

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CHOLLA GENERATING STATION
FAP SEEPAGE SYS EXTRACTION WELLS PUMP INSTALLATION
HUNT SYSTEM PLAN

aps

SCALE 1"=20' DATE 05/11/2022

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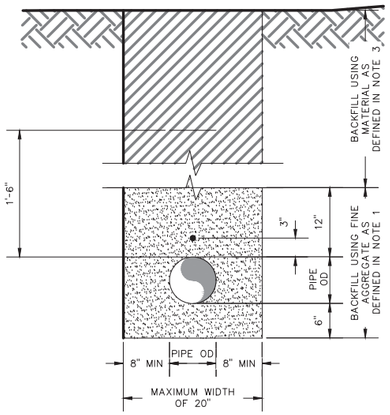
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DWN	EXD	APPROVED	W A
CRJ	TSL	MAREN HENLEY	CHC08903
CHD ALK	RWD RAW	DRAWING APPROVED BY	

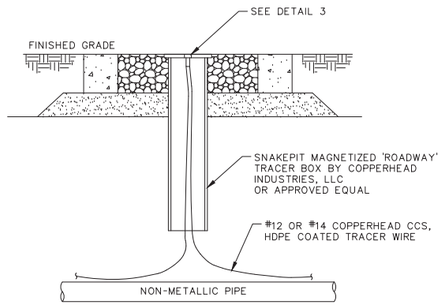
UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	C	47	BP	AP	230454	3

Figure 16



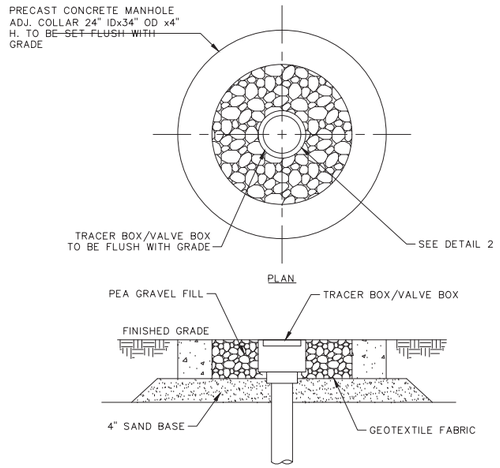
NOTES:
1. CONTRACTOR TO USE FINE AGGREGATE AS INITIAL BACKFILL AS DEFINED AS BEING ABLE TO PASS A NO. 4 US STANDARD SIEVE BUT NOT A NO. 200 US STANDARD SIEVE AND MUST NOT CONTAIN ANY DELETERIOUS MATERIALS.
2. CONTRACTOR TO BACKFILL AND COMPACT TO 95% IN 8 INCH LIFTS. SEE TECHNICAL NOTES.
3. CONTRACTOR TO USE CLSM, ABC, AND/OR GRANULAR OR SCREENED NATIVE BACKFILL.
4. INSTALL WARNING TAPE ENTIRE LENGTH OF BURIED PIPING. SEE TECHNICAL SPECIFICATIONS.

DETAIL 1 - NTS — TRENCH DETAIL

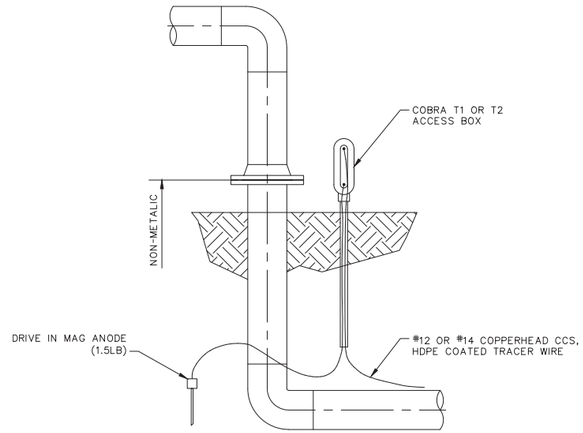


NOTES:
1. TRACER BOXES SHALL BE LOCATED AT EACH ELBOW.
2. MAXIMUM SPACING OF TRACER BOXES SHALL BE 1000'.

DETAIL 2 - NTS — TYPICAL TRACER BOX — PIPES THAT ARE BELOW GROUND

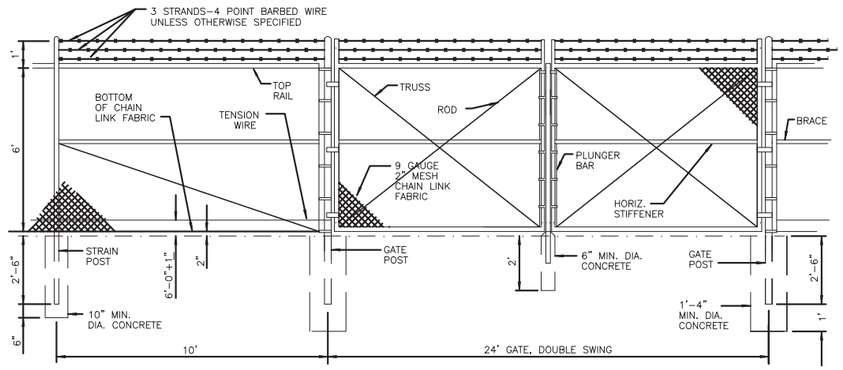


DETAIL 3 - NTS — TYPICAL TRACER BOX — PROTECTION COLLAR



NOTES:
1. TRACER BOXES SHALL BE LOCATED AT EACH ELBOW.
2. MAXIMUM SPACING OF TRACER BOXES SHALL BE 1000'.
3. COBRA T1 USED WITH NO GROUNDING ANODE.
4. COBRA T2 USED WITH GROUNDING ANODE.

DETAIL 4 - NTS — TYPICAL TRACER BOX — PIPES THAT COME ABOVE GROUND



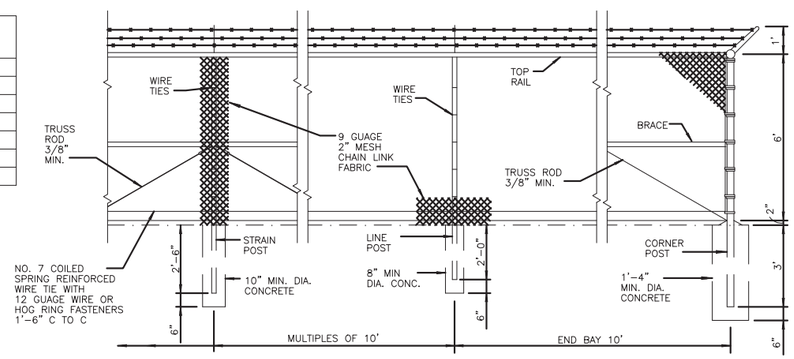
NOTES

1. ALL CONCRETE SHALL BE CLASS 'C' PER SECT. MAG 725.
2. FITTINGS NOT SPECIFICALLY DETAILED SHALL BE HEAVY DUTY DESIGN.
3. STRAIN POSTS SHALL BE SPACED AT 500' MAXIMUM SPACING.
4. BOTH CORNER AND STRAIN POSTS SHALL HAVE STRAIN PANELS.
5. ALL POSTS SHALL BE CAPPED.
6. MEMBER SIZES SHALL BE THE FOLLOWING:

MEMBER	AISC SIZE	OUTSIDE DIA.
CORNER POST	2-1/2"	2.875"
LINE POST	1-1/2"	1.900"
STRAIN POST	1-1/2"	1.900"
BRACE	1-1/4"	1.666"
STRETCH BAR	3/16"x3/4" FLAT	3/16"x3/4" FLAT
GATE POST	3-1/2"	4.000"
TOP RAIL	1-1/4"	1.666"

7. CONSTRUCTION AND MATERIALS SHALL CONFORM TO MAG SECT. 420 AND 772, RESPECTIVELY. SEE TABLE 772-1 FOR WEIGHTS OF MEMBERS.

DETAIL 160 FROM MARICOPA ASSOCIATION OF GOVERNMENTS (MAG).



DETAIL 5 - NTS — 6' CHAIN LINK FENCE AND GATE

AS-BUILT CERTIFICATION

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1	5/23	AS-BUILT	DDM		TSL	ALK	MH	CHC08903
0.b	5/22	ISSUED FOR CONST	ACK	ALK	TSL	RAW	MH	CHC08903
0.a	3/22	ISSUED FOR BID	CRJ	ALK	TSL	RAW	MH	CHC08903

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FLOW METER DETAILS

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SCALE NTS DATE 03/18/2022

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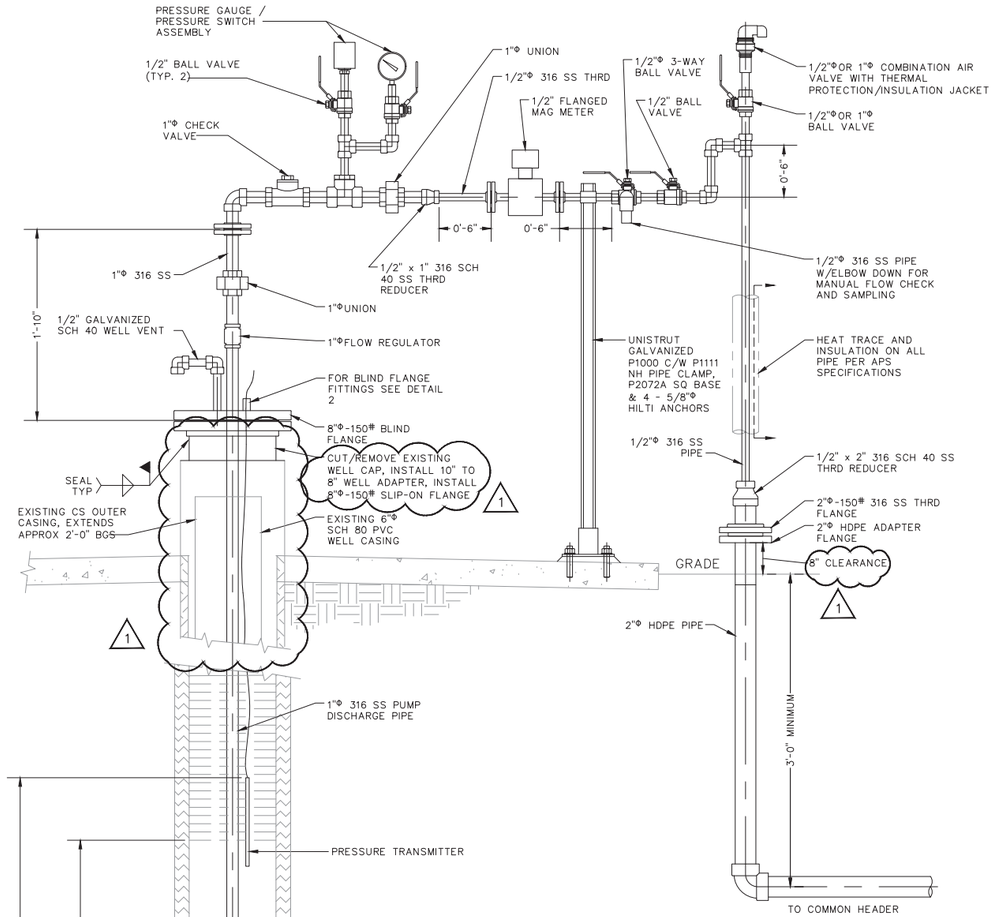
DWN	CRJ	EXD	TSL	APPROVED	W A
CHD	ALK	RVWD	RAW	MAREN HENLEY DRAWING APPROVED BY	CHC08903

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	C	64	BP	AP	230454	4

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Figure 17

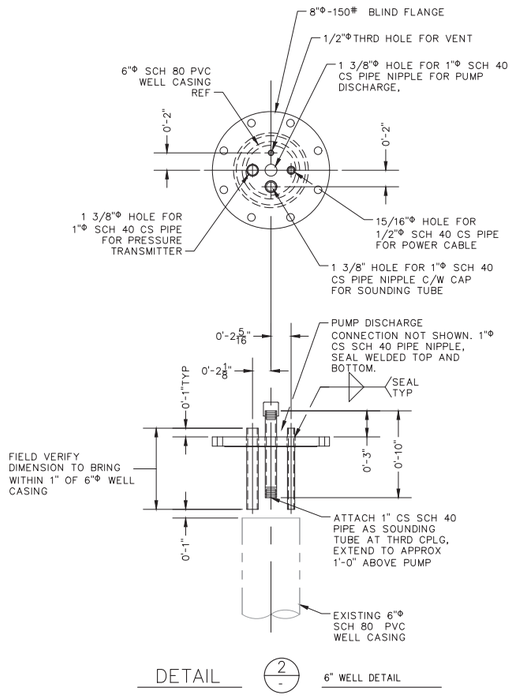


WELL DIMENSION TABLE						
WELLS NO	WELL CASING SIZE	OUTER CASING / MONUMENT SIZE	BORE HOLE DIAMETER	APPROX. WELL DEPTH	SCREENED SECTION	PRESSURE TRANSMITTER DEPTH (4mA)
EW-01	6" SCH 80	10"	10"	55'-0"	20-50 FT BGS	49.8 FT BGS
EW-02	6" SCH 80	10"	10"	53'-0"	18-48 FT BGS	49.0 FT BGS
EW-03	6" SCH 80	10"	10"	52'-0"	17-47 FT BGS	46.0 FT BGS
EW-04	6" SCH 80	10"	10"	24'-0"	9-19 FT BGS	19.2 FT BGS
GSX-1R	6" SCH 80	10"	10"	54'-0"	29-49 FT BGS	47.8 FT BGS

CONTROL SET POINTS:				
WELLS	ANTICIPATED PUMP RATE	REGULATOR PUMP RATE	PUMP ON DEPTH/ TRANSMITTER SET POINT	PUMP OFF DEPTH/ TRANSMITTER SET POINT
EW-01	1.5	1.0 GPM	25.0' BGS / 10.0 mA	49.3' BGS / 4.2 mA
EW-02	5.0	4.0 GPM	25.0' BGS / 10.0 mA	48.5' BGS / 4.2 mA
EW-03	5.0	4.0 GPM	20.0' BGS / 9.0 mA	45.5' BGS / 4.2 mA
EW-04	2.5	1.5 GPM	15.0' BGS / 7.5 mA	18.7' BGS / 4.2 mA
GSX-1R	0.75	0.75 GPM	20.0' BGS / 9.0 mA	47.3' BGS / 4.2 mA

NOTES:
1. TRANSMITTER ON/OFF LEVEL SETPOINTS BASED ON
   A. UTILIZING PRESSURE TRANSMITTER WITH PRESSURE RANGE OF 0-30 PSI, AND
   B. TRANSMITTER INSTALLATION DEPTH
2. LEVEL INDICATOR IS INTENDED TO BE SET TO READ FT BGS. TO DO SO, THE 4 mA SIGNAL READOUT WILL BE LARGER THAN 20 mA SIGNAL READOUT. DESIGN CONDITIONS: SET 4 mA TO TRANSMITTER SET DEPTH (FT BGS) AND 20 mA TO TRANSMITTER SET DEPTH MINUS 69.3 (30 PSI). SEE SET POINTS IN TABLE. ADJUST TO ACTUAL CONDITIONS IN THE FIELD.
3. ALL WELL DEPTH MEASUREMENTS ARE FROM THE GROUND SURFACE (GRADE). MEASURED DEPTHS NEED TO BE ADJUSTED IF ALTERNATIVE MEASURING POINTS ARE USED, SUCH AS THE TOP OF CASING OR FLANGE.
4. PRESSURE SWITCH - N.C. OPENS AT 100 PSI.

DETAIL 1/- 6" EXTRACTION WELL EQUIPMENT
SCALE: 1 1/2"=1'-0"



DETAIL 2/- 6" WELL DETAIL

AS-BUILT CERTIFICATION

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0.a	3/22	ISSUED FOR BID	CRJ	ALK	TSL	RAW	MH	CHC08903

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PHOENIX, ARIZONA 85034
PHONE: 602-733-6000

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CHOLLA GENERATING STATION
FAP SEEPAGE SYS EXTRACTION WELLS PUMP INSTALLATION
WELL AND PIPING DETAILS - SHT 1

aps
SCALE 1 1/2"=1'-0" DATE 03/18/2022

WORK SAFELY TODAY

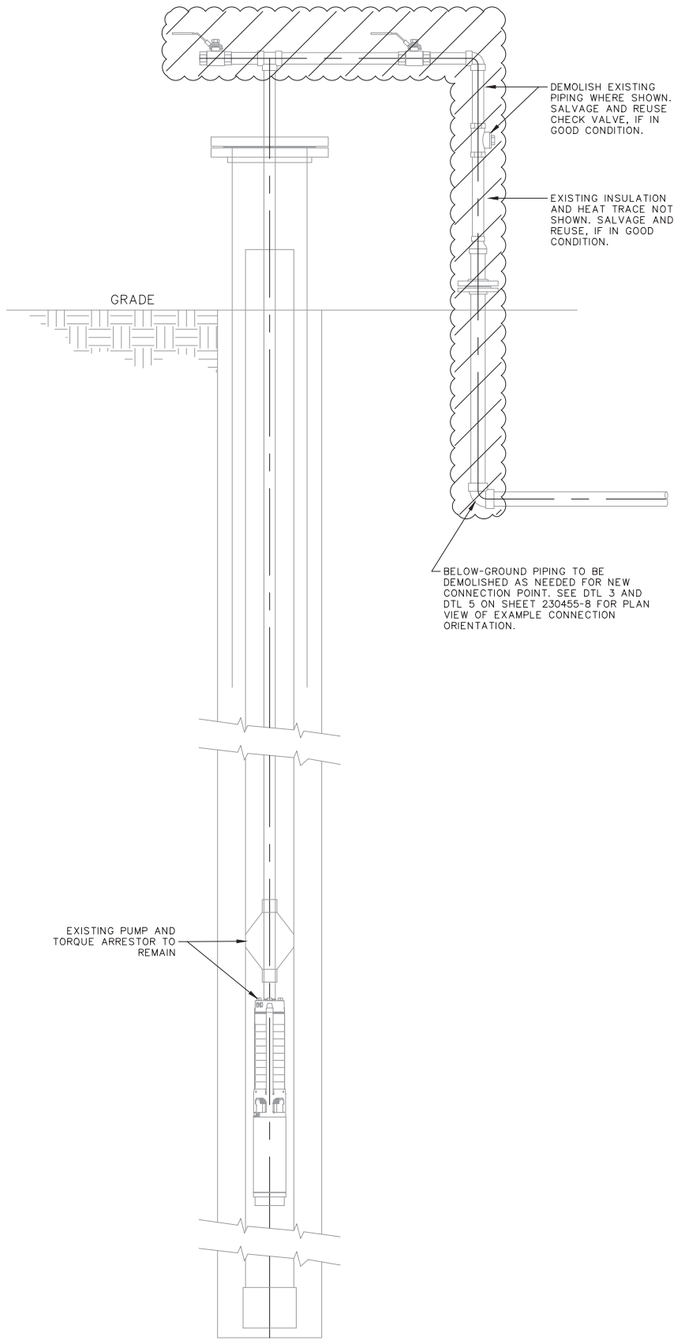
DWN	EXD	APPROVED	W A
CRJ	TSL	MAREN HENLEY	CHC08903
CHD ALK	RVWD RAW	DRAWING APPROVED BY	

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UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	M	65	BP	AP	230455	7

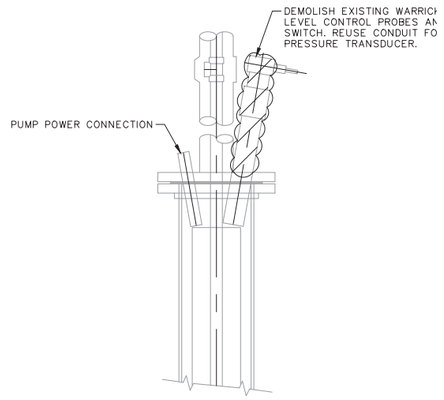
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Arizona Blue Stake, Inc.
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Figure 19



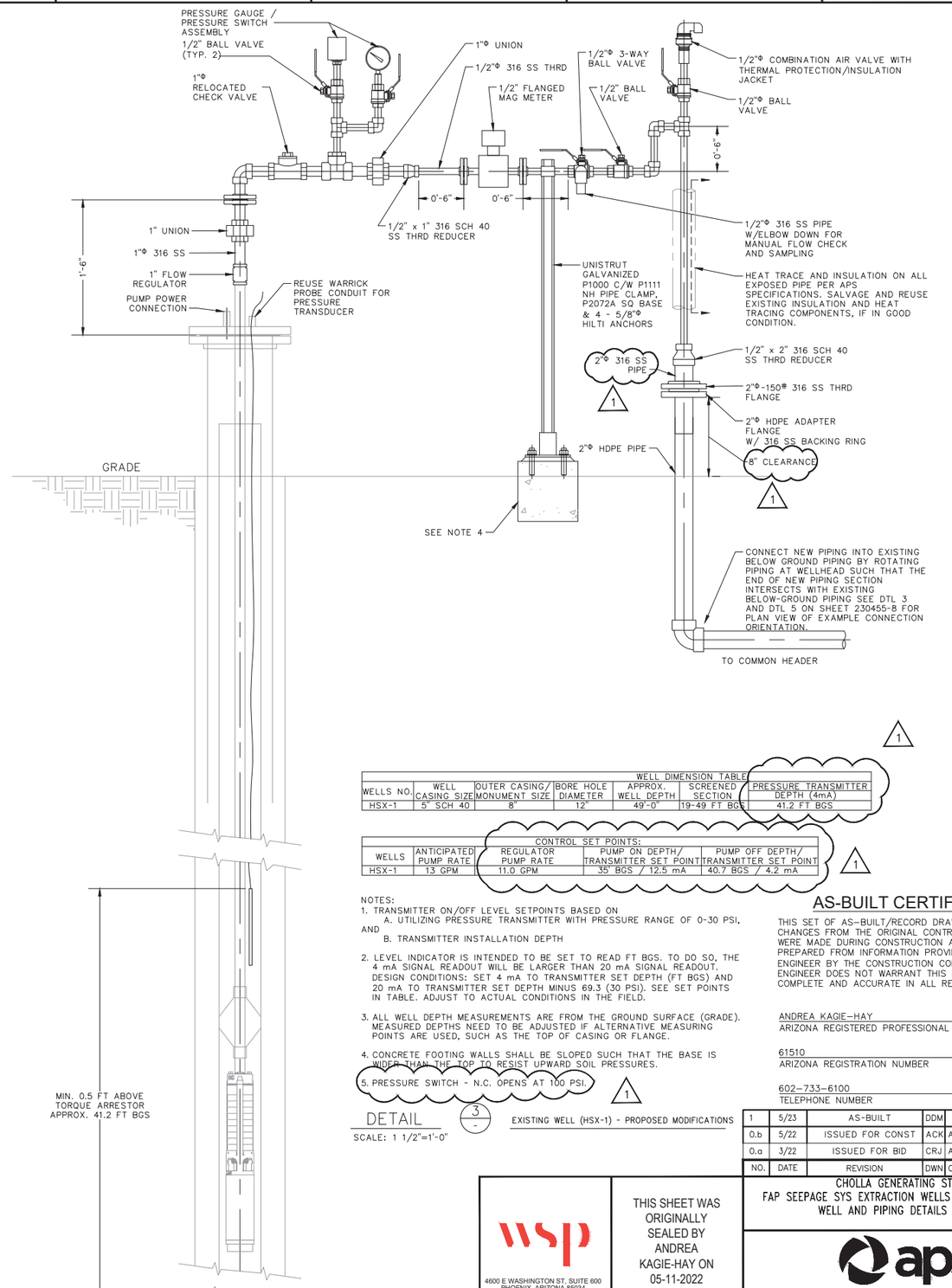
DETAIL 1/- EXISTING WELL (HSX-1) - PROPOSED DEMOLITION

SCALE: 1 1/2"=1'-0"



DETAIL 2/- TYP. WELL HEAD

SCALE: 1 1/2"=1'-0"



DETAIL 3/- EXISTING WELL (HSX-1) - PROPOSED MODIFICATIONS

SCALE: 1 1/2"=1'-0"

1

WELL DIMENSION TABLE						
WELLS NO.	WELL CASING SIZE	OUTER CASING/ MONUMENT SIZE	BORE HOLE DIAMETER	APPROX. WELL DEPTH	SCREENED SECTION	PRESSURE TRANSMITTER DEPTH (4mA)
HSX-1	5" SCH 40	8"	12"	49'-0"	19-49 FT BGS	41.2 FT BGS

1

CONTROL SET POINTS:				
WELLS	ANTICIPATED PUMP RATE	REGULATOR PUMP RATE	PUMP ON DEPTH/ TRANSMITTER SET POINT	PUMP OFF DEPTH/ TRANSMITTER SET POINT
HSX-1	13 GPM	11.0 GPM	35' BGS / 12.5 mA	40.7 BGS / 4.2 mA

1

NOTES:
1. TRANSMITTER ON/OFF LEVEL SETPOINTS BASED ON
   A. UTILIZING PRESSURE TRANSMITTER WITH PRESSURE RANGE OF 0-30 PSI, AND
   B. TRANSMITTER INSTALLATION DEPTH
2. LEVEL INDICATOR IS INTENDED TO BE SET TO READ FT BGS. TO DO SO, THE 4 mA SIGNAL READOUT WILL BE LARGER THAN 20 mA SIGNAL READOUT. DESIGN CONDITIONS: SET 4 mA TO TRANSMITTER SET DEPTH (FT BGS) AND 20 mA TO TRANSMITTER SET DEPTH MINUS 69.3 (30 PSI). SEE SET POINTS IN TABLE. ADJUST TO ACTUAL CONDITIONS IN THE FIELD.
3. ALL WELL DEPTH MEASUREMENTS ARE FROM THE GROUND SURFACE (GRADE). MEASURED DEPTHS NEED TO BE ADJUSTED IF ALTERNATIVE MEASURING POINTS ARE USED, SUCH AS THE TOP OF CASING OR FLANGE.
4. CONCRETE FOOTING WALLS SHALL BE SLOPED SUCH THAT THE BASE IS WIDER THAN THE TOP TO RESIST UPWARD SOIL PRESSURES.
5. PRESSURE SWITCH - N.C. OPENS AT 100 PSI.

1

AS-BUILT CERTIFICATION

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0.a	3/22	ISSUED FOR BID	CRJ	ALK	TSL	RAW	MH	CHC08903

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CHOLLA GENERATING STATION
FAP SEEPAGE SYS EXTRACTION WELLS PUMP INSTALLATION
WELL AND PIPING DETAILS - SHT 2

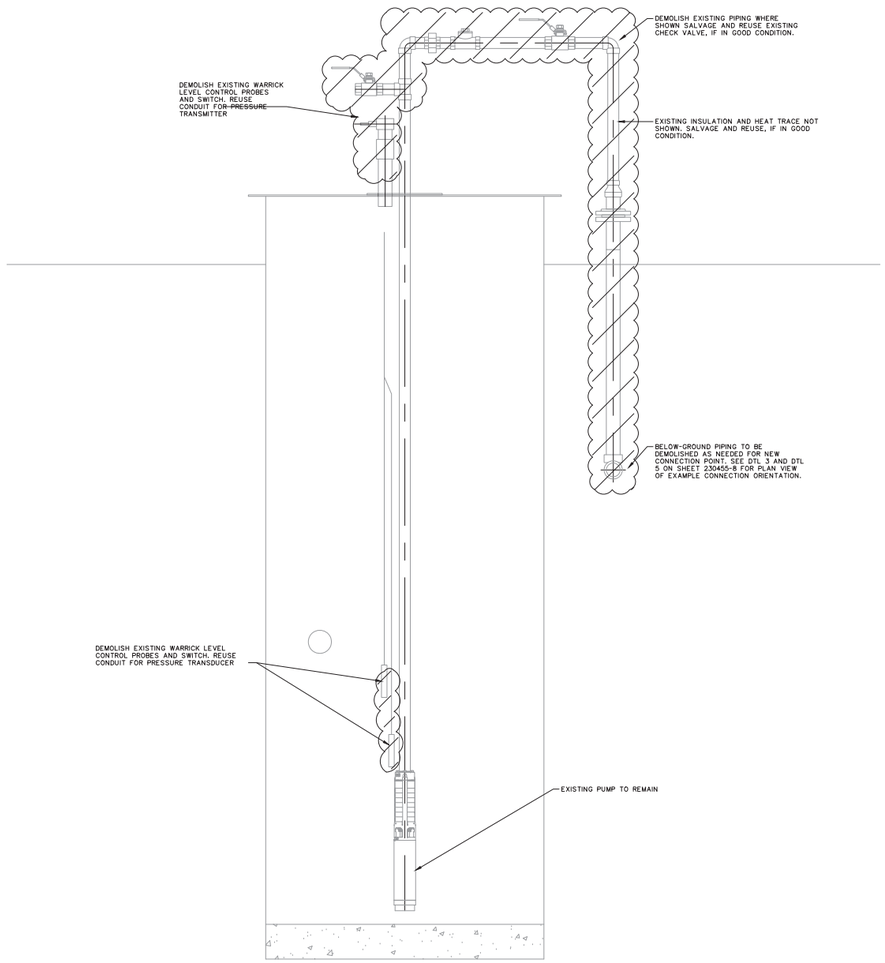
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SCALE 1 1/2"=1'-0" DATE 03/18/2022

DWN	EXD	APPROVED	W A
CRJ	TSL	MAREN HENLEY	CHC08903
CHD ALK	RVWD RAW	DRAWING APPROVED BY	

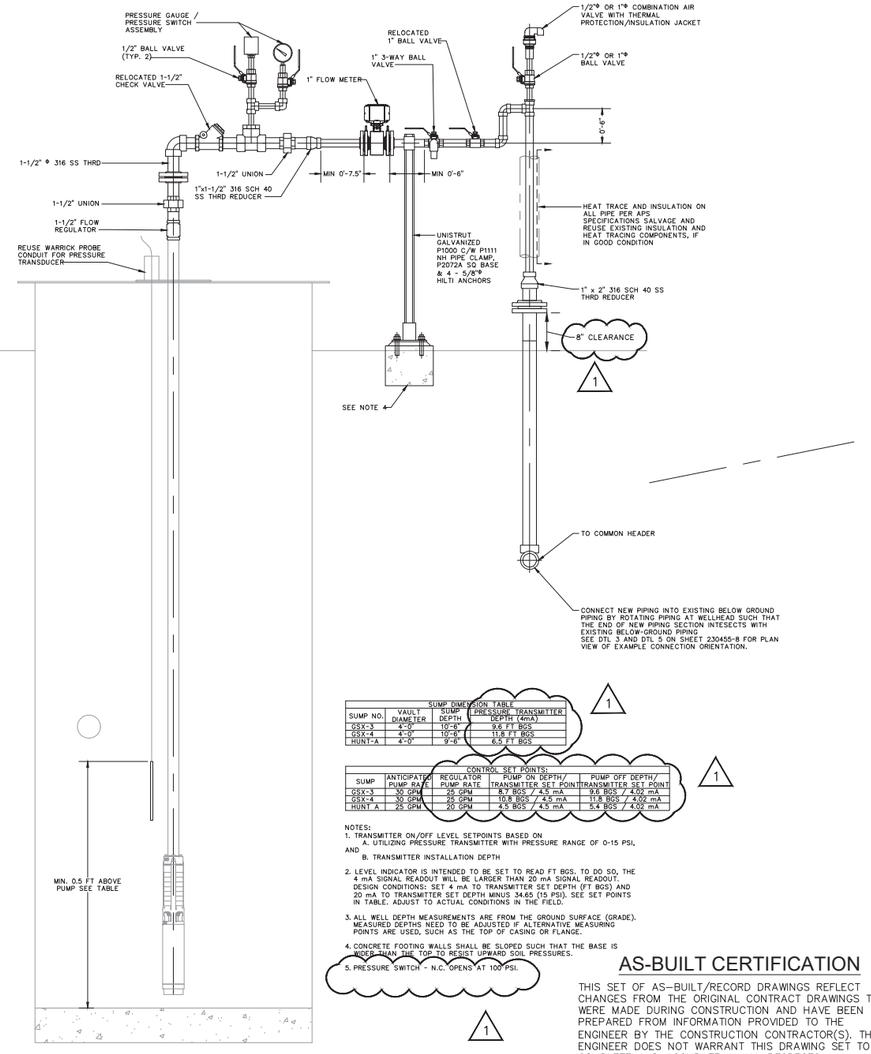
UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	M	65	BP	AP	230455	8

Figure 20



DETAIL 4 - EXISTING SUMPS - PROPOSED DEMOLITION

SCALE: 1 1/2"=1'-0"



DETAIL 5 - EXISTING SUMPS - PROPOSED MODIFICATIONS

SCALE: 1 1/2"=1'-0"

SUMP DIMENSION TABLE			
SUMP NO.	VAULT DIAMETER	SUMP DEPTH	PRESSURE TRANSMITTER DEPTH (4mA)
GSX-3	4'-0"	10'-6"	9.6 FT BGS
GSX-4	4'-0"	10'-6"	11.8 FT BGS
HUNT-A	4'-0"	9'-6"	6.5 FT BGS

CONTROL SET POINTS:				
SUMP	ANTICIPATED PUMP RATE	REGULATOR PUMP RATE	PUMP ON DEPTH/ TRANSMITTER SET POINT	PUMP OFF DEPTH/ TRANSMITTER SET POINT
GSX-3	30 GPM	25 GPM	8.7 BGS / 4.5 mA	9.8 BGS / 4.02 mA
GSX-4	30 GPM	25 GPM	10.8 BGS / 4.5 mA	11.8 BGS / 4.02 mA
HUNT A	25 GPM	20 GPM	4.5 BGS / 4.5 mA	5.4 BGS / 4.02 mA

NOTES:
1. TRANSMITTER ON/OFF LEVEL SETPOINTS BASED ON
   A. UTILIZING PRESSURE TRANSMITTER WITH PRESSURE RANGE OF 0-15 PSI, AND
   B. TRANSMITTER INSTALLATION DEPTH
2. LEVEL INDICATOR IS INTENDED TO BE SET TO READ FT BGS. TO DO SO, THE 4 mA SIGNAL READOUT WILL BE LARGER THAN 20 mA SIGNAL READOUT. DESIGN CONDITIONS: SET 4 mA TO TRANSMITTER SET DEPTH (FT BGS) AND 20 mA TO TRANSMITTER SET DEPTH MINUS 34.65 (15 PSI). SEE SET POINTS IN TABLE. ADJUST TO ACTUAL CONDITIONS IN THE FIELD.
3. ALL WELL DEPTH MEASUREMENTS ARE FROM THE GROUND SURFACE (GRADE). MEASURED DEPTHS NEED TO BE ADJUSTED IF ALTERNATIVE MEASURING POINTS ARE USED, SUCH AS THE TOP OF CASING OR FLANGE.
4. CONCRETE FOOTING WALLS SHALL BE SLOPED SUCH THAT THE BASE IS WIDER THAN THE TOP TO RESIST UPWARD SOIL PRESSURES.
5. PRESSURE SWITCH - N.C. OPENS AT 100 PSI.

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ANDREA KAGIE-HAY 05/11/2023
ARIZONA REGISTERED PROFESSIONAL ENGINEER DATE

61510
ARIZONA REGISTRATION NUMBER

602-733-6100
TELEPHONE NUMBER

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
1	5/23	AS-BUILT	DDM		RAW	ALK	MH	CHC08903
0.b	5/22	ISSUED FOR CONST	ACK	ALK	TSL	RAW	MH	CHC08903
0.a	3/22	ISSUED FOR BID	CRJ	ALK	TSL	RAW	MH	CHC08903

wsp
4600 E WASHINGTON ST, SUITE 600
PHOENIX, ARIZONA 85034
PHONE: 602-733-6000

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CHOLLA GENERATING STATION
FAP SEEPAGE SYS EXTRACTION WELLS PUMP INSTALLATION
WELL AND PIPING DETAILS - SHT 3

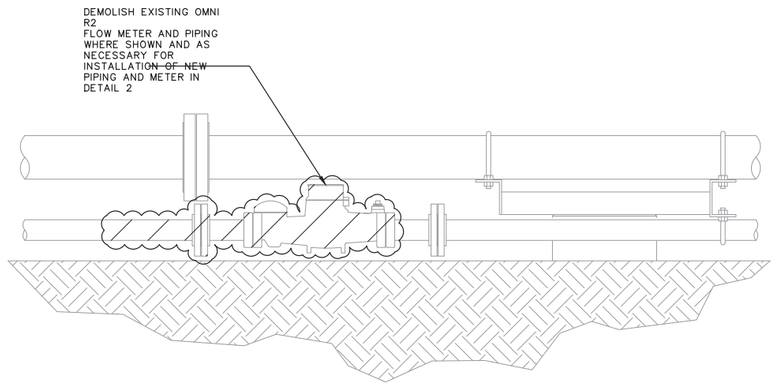
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SCALE 1 1/2"=1'-0" DATE 03/18/2022

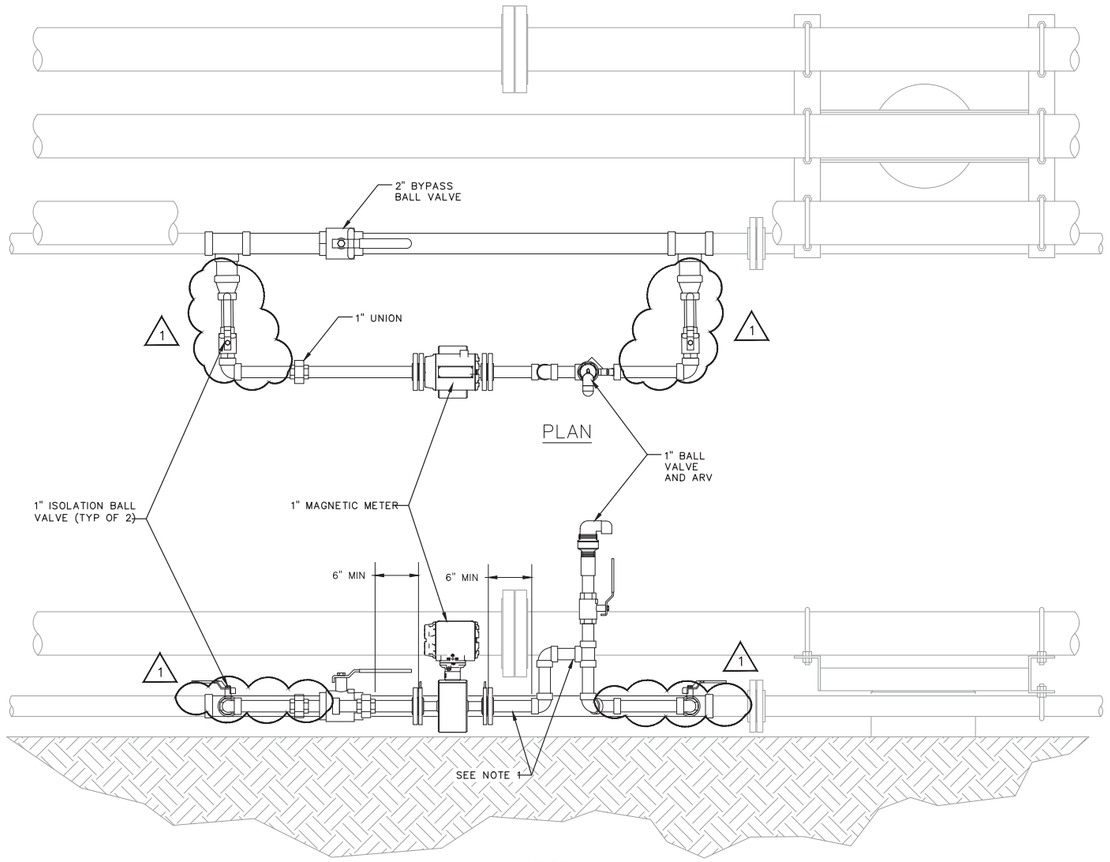
DWN	EXD	APPROVED	W A
CRJ	TSL	MAREN HENLEY	CHC08903
CHD ALK	RVWD RAW	DRAWING APPROVED BY	

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	M	65	BP	AP	230455	9

Figure 21



DETAIL 1/- NTS — HUNT SYSTEM FLOW METER DEMOLITION



SECTION

DETAIL 2/- NTS — HUNT SYSTEM FLOW METER INSTALLATION

NOTES:
1. PROVIDE PIPE SUPPORTS FOR NEW PIPING AT METER AND AIR RELEASE VALVE AT A MINIMUM UTILIZING UNISTRUT AND PIPE STRAPS. SUPPORT OFF OF EXISTING W-BEAMS WHERE POSSIBLE. POUR CONCRETE PADS WHERE NO SUPPORT STRUCTURE EXISTS.
2. CONCRETE FOOTING WALLS SHALL BE SLOPED SUCH THAT THE BASE IS WIDER THAN THE TOP TO RESIST UPWARD SOIL PRESSURES.
3. REINSTALL, REPLACE, AND PROVIDE NEW INSULATION AS NEEDED ON THE FLOW METER, VALVES AND PIPING INCLUDING THE BYPASS LINE AND AN INSULATION JACKET ON THE ARV. EXTEND INSULATION AT LEAST 10 FEET ON EITHER SIDE OF THE FLOW METER ASSEMBLY.

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0.b	5/22	ISSUED FOR CONST	ACK	ALK	TSL	RAW	MH	CHC08903
0.a	3/22	ISSUED FOR BID	CRJ	ALK	TSL	RAW	MH	CHC08903

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PHONE: 602-733-6000

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FAP SEEPAGE SYS EXTRACTION WELLS PUMP INSTALLATION
HUNT SYSTEM FLOW METER DETAILS

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SCALE NTS DATE 03/18/2022

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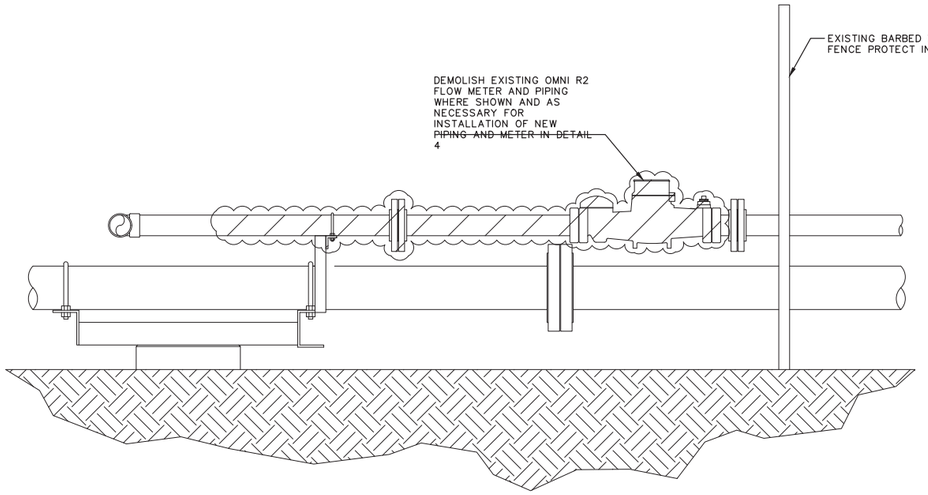
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CRJ	TSL	MAREN HENLEY	CHC08903
CHD	RVWD	DRAWING APPROVED BY	
ALK	RAW		

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	M	65	BP	AP	230455	10

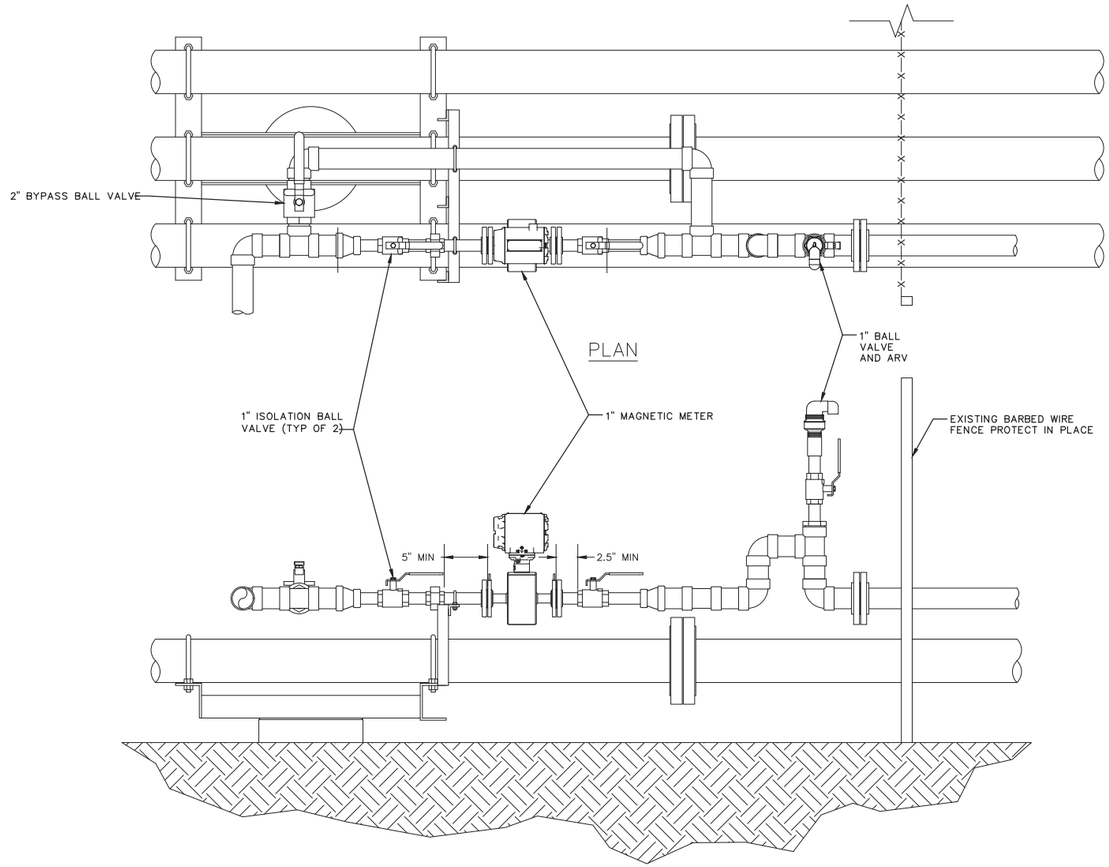
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Figure 22



DETAIL 3 - NTS — COMBINED GERONIMO/HUNT SYSTEM FLOW METER DEMOLITION



DETAIL 4 - NTS — COMBINED GERONIMO/HUNT SYSTEM FLOW METER INSTALLATION

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ANDREA KAGIE-HAY 05/11/2023
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61510
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602-733-6100
TELEPHONE NUMBER

NOTES:
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NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
1	5/23	AS-BUILT	DDM		RAW	ALK	MH	CHC08903
0.b	5/22	ISSUED FOR CONST	ACK	ALK	TSL	RAW	MH	CHC08903
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FAP SEEPAGE SYS EXTRACTION WELLS PUMP INSTALLATION
HUNT/GERONIMO SYSTEM FLOW METER DETAILS

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SCALE NTS DATE 03/18/2022

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DWN	EXD	APPROVED	W A
CRJ	TSL	MAREN HENLEY	CHC08903
CHD	RVWD	DRAWING APPROVED BY	
ALK	RAW		

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UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	M	65	BP	AP	230455	11

Figure 23

ATTACHMENT B

AECOM, 2021. *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.

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:

Arizona Public Service
400 North 5th Street
Phoenix, AZ 85004

Prepared by:

AECOM
7720 North 16th Street, Suite 100
Phoenix, AZ 85020
aecom.com

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Attachment

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

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.

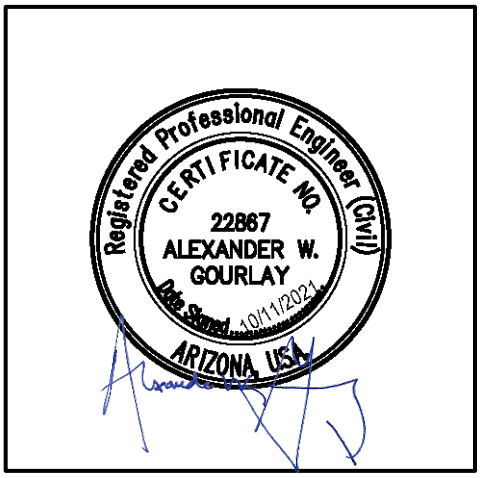
8. Certification Statement

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

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
Phoenix, AZ 85072

Submitted by
AECOM
7720 North 16th Street
Suite 100
Phoenix, AZ 85020
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}	---	

Item	As-Constructed/ Current	Comments	Reference Document
Foundation Conditions			
Physical Properties	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.	---	• 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)
Engineering Properties	<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	---	

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	<u>Clay Core:</u> • 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 (<u>Random Zone):</u> • 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)	<u>Cutoff Wall:</u>
	• 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



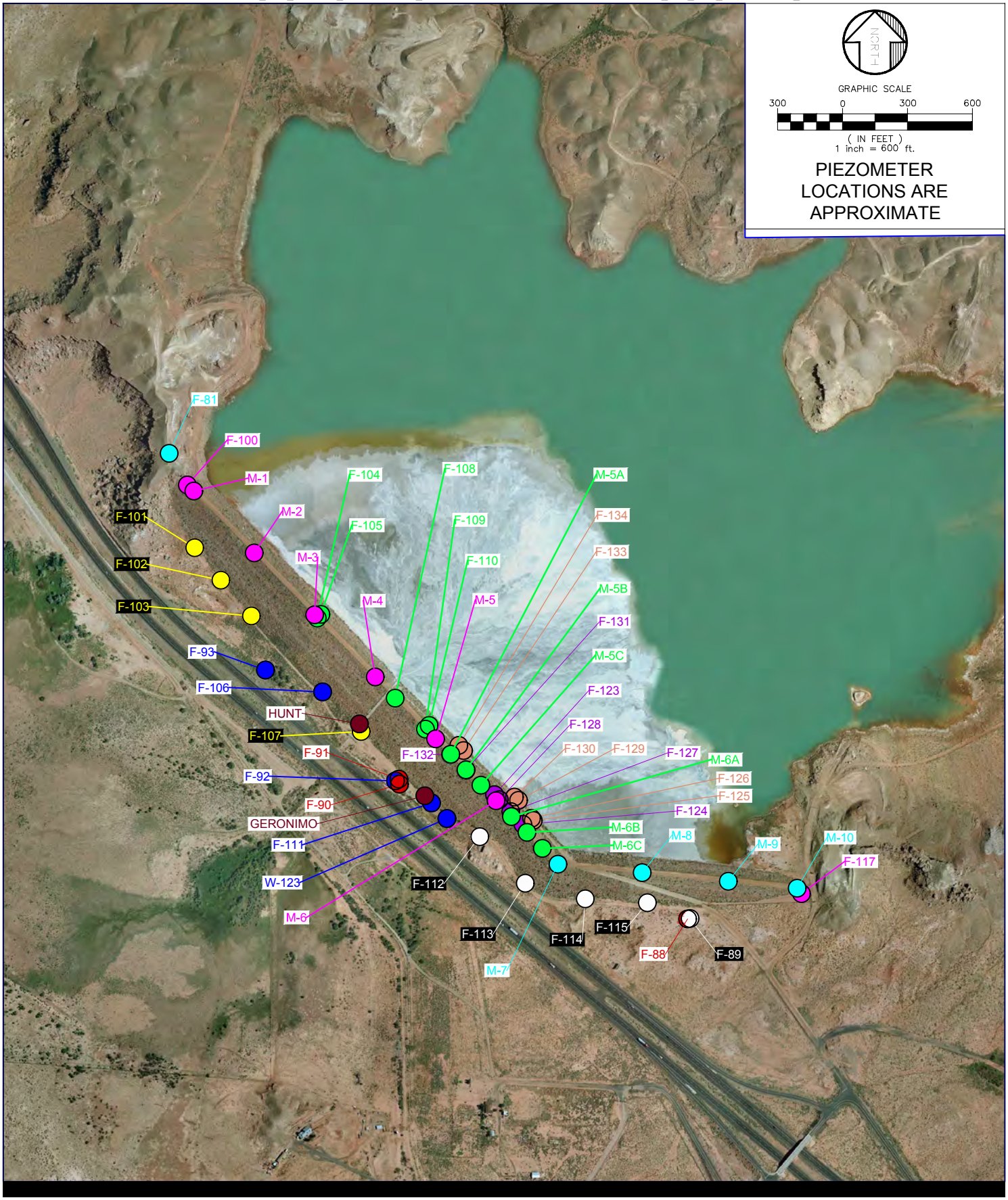
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SITE VICINITY MAP

AECOM

FIGURE 1-1

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Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 1-2_FLY_ASH_INSTRUMENT_LOCATIONS.DWG



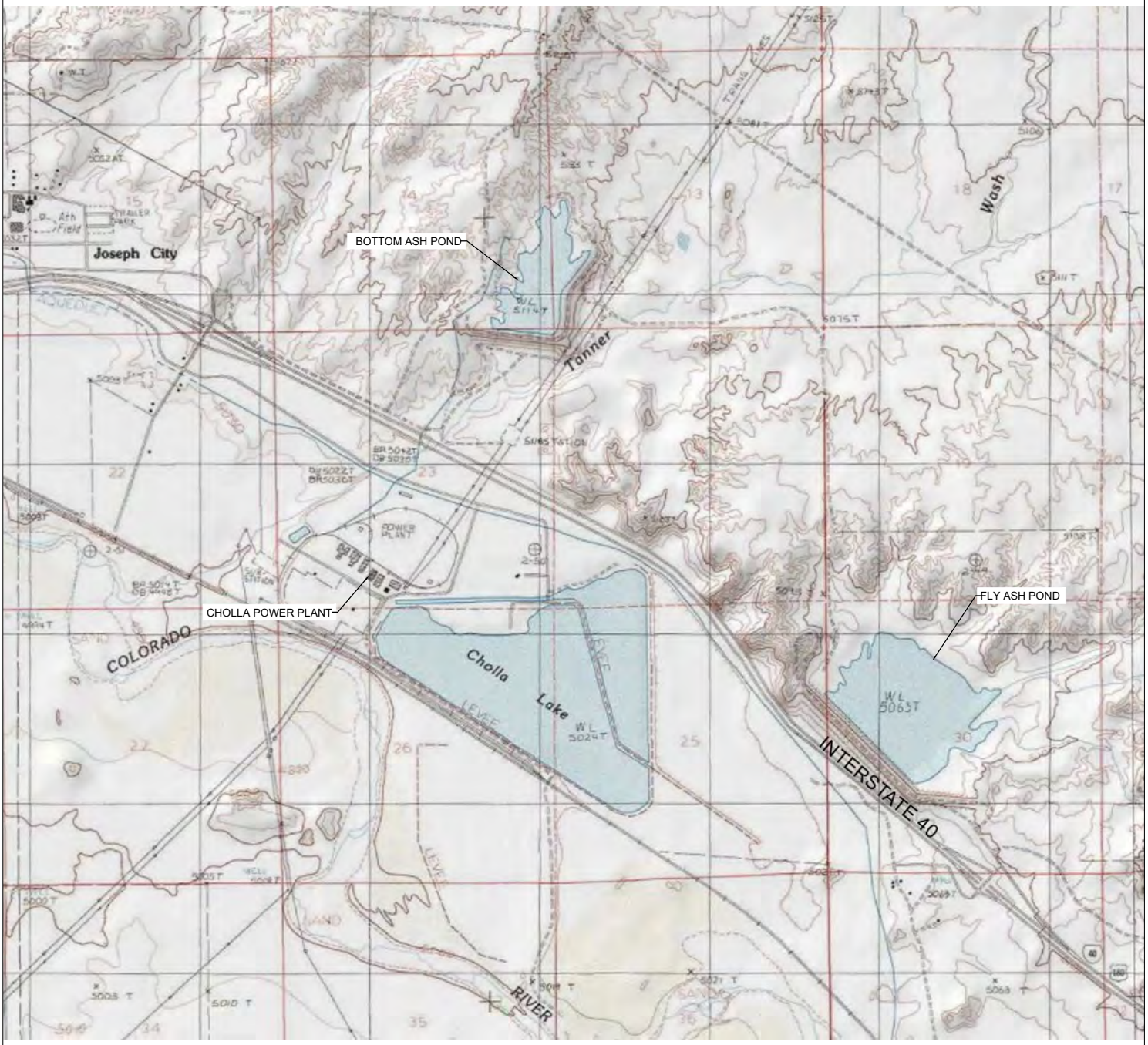
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FLY ASH POND
MONITORED INSTRUMENTATION
AND SEEPAGE LOCATION MAP

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FIGURE 1-2

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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

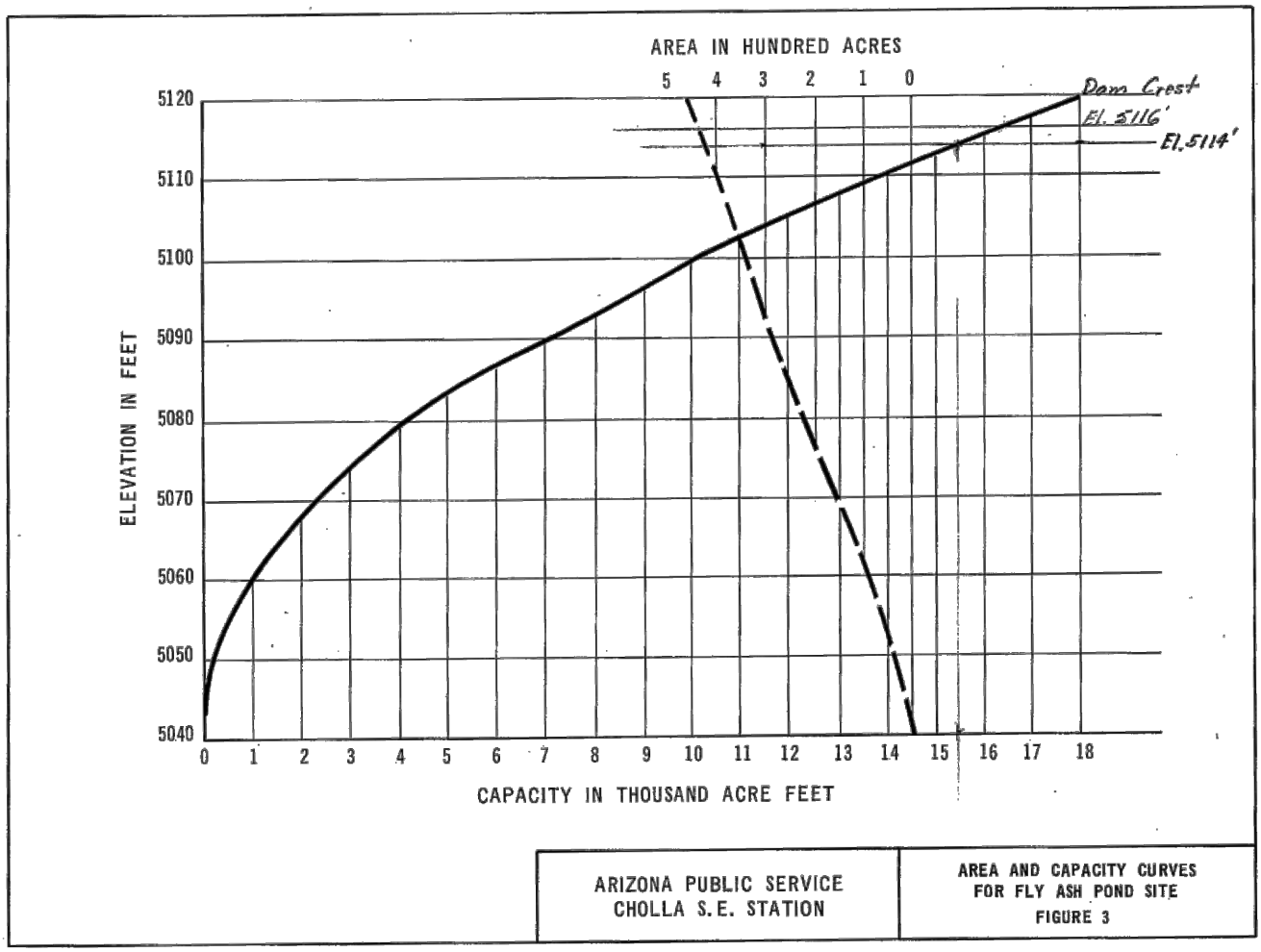
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SITE TOPOGRAPHIC MAP

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FIGURE 3-1

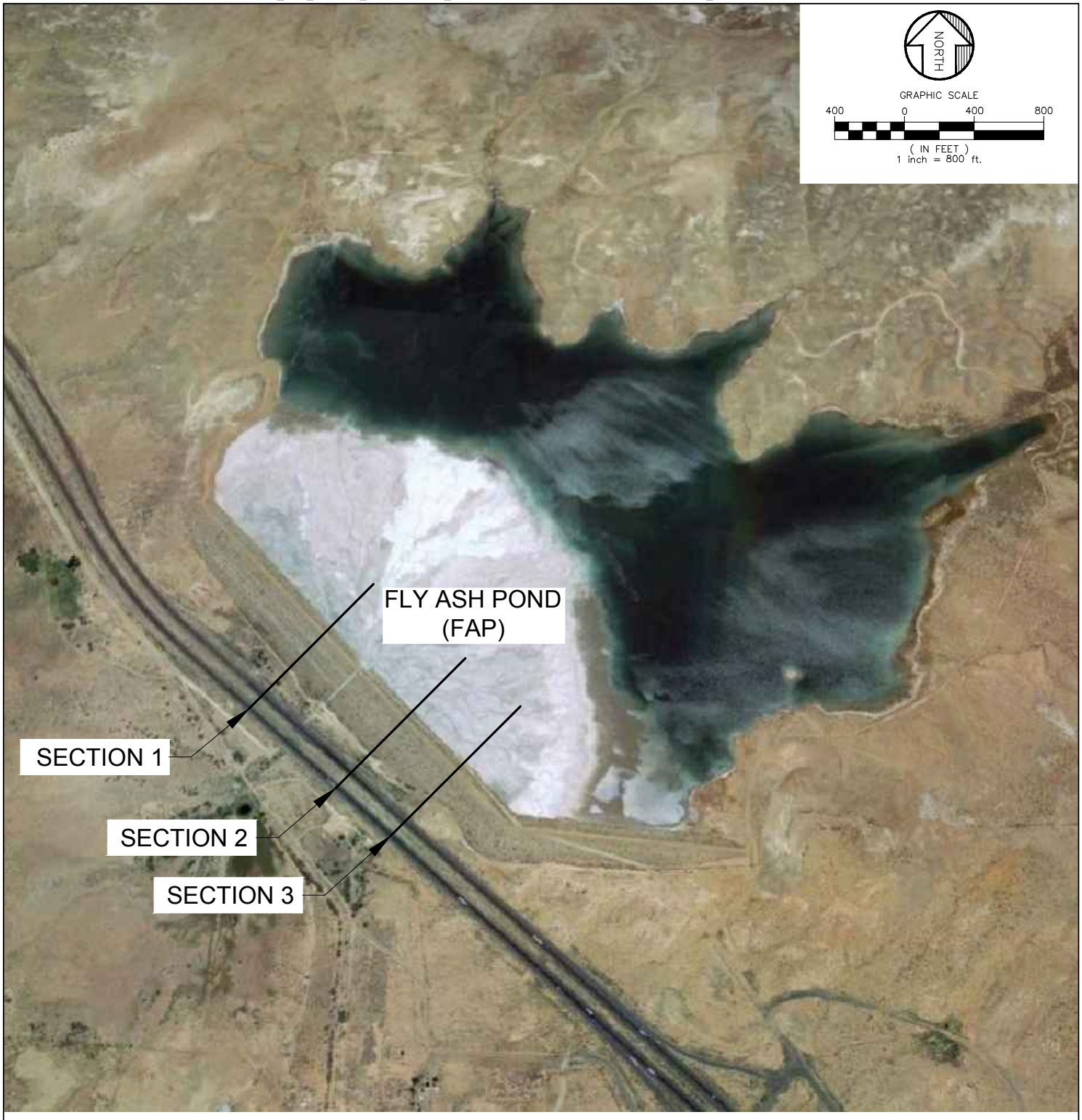
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Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 4-2_AREA CAPACITY CURVE_JSK.DWG



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 ANSI A 8.5" x 11"
Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 5-1_CROSS SECTION LOCATIONS SFA - FAP.DWG



FOUR CORNERS POWER PLANT
STRUCTURAL INTEGRITY REPORT
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CROSS SECTION LOCATIONS
SAFETY FACTOR ASSESSMENT

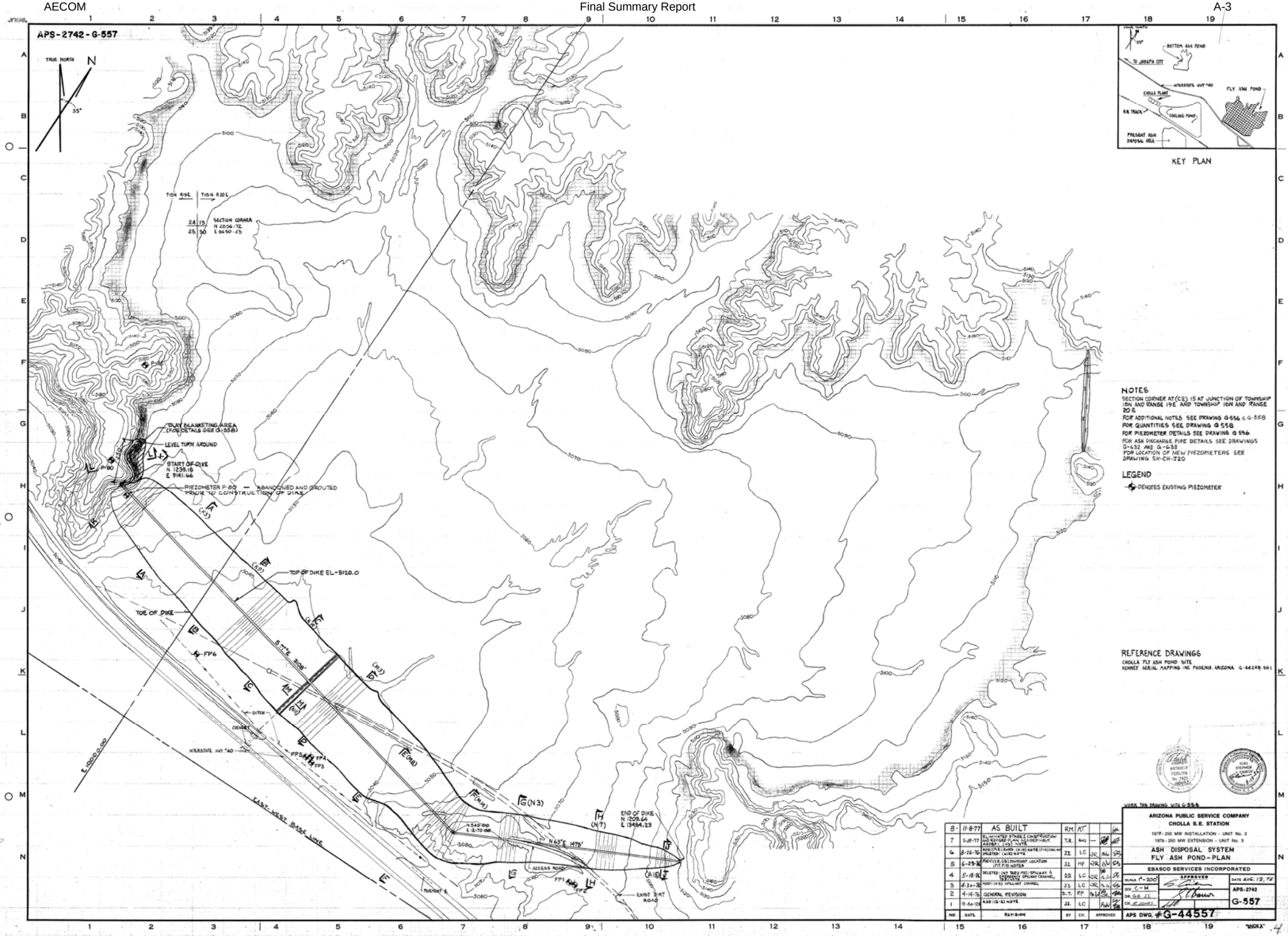
AECOM

FIGURE 5-1

Appendix A.
Historic Drawings

ORIGINAL AS-BUILT DRAWINGS

(Ebasco, 1977)



NOTES

SECTION CORNER AT (C2) IS AT JUNCTION OF TOWNSHIP 10N AND RANGE 19E AND TOWNSHIP 10N 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-633

FOR LOCATION OF NEW PIEZOMETERS SEE DRAWING SK-CH-720

LEGEND

DENOTES EXISTING PIEZOMETER

REFERENCE DRAWINGS

CHOLLA FLY ASH POND SITE
KENNEY AERIAL MAPPING INC PHOENIX ARIZONA G-44248 SH1

NO.	DATE	REVISION	BY	CH.	APPROVED
8	11-8-77	AS BUILT	RM	MT	
7	7-28-77	ELIMINATED STAGES CONSTRUCTION AND REVISED PLAN AS ASSOCIATED (+3) NOTE	TR	RMS	—
6	8-26-76	ADD (7) & (8) NOTE (7) NOTE DELETED: (4)(8) NOTE	JJ	LC	
5	6-29-76	REVISED (C2) TOWNSHIP LOCATION (7) (7) NOTES	JJ	MF	
4	5-18-76	DELETED: (10) TREE PROTECTION & EMERGENCY SPILLWAY CHANNEL REVISED	DD	LC	
3	4-30-76	ADD (12) SPILLWAY CHANNEL	JJ	LC	
2	4-14-76	GENERAL REVISION	D.T.	RP	
1	0-30-75	ADD (13) (15) NOTE	JJ	LC	

WORK THIS DRAWING WITH G-558

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

ASH DISPOSAL SYSTEM
FLY ASH POND - PLAN

EBASCO SERVICES INCORPORATED

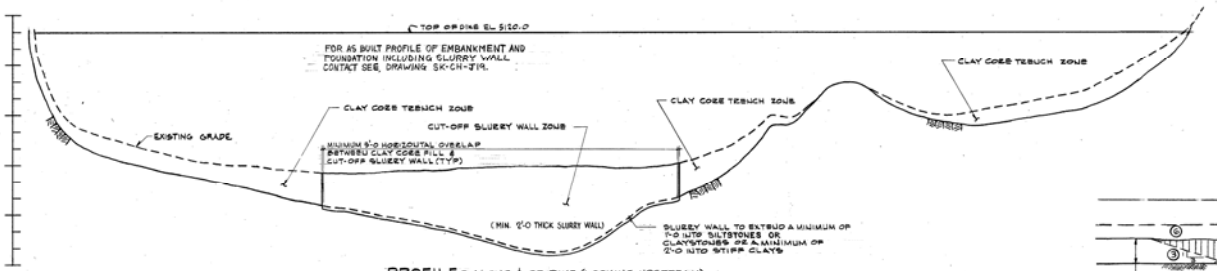
SCALE 1" = 200'
DES. C-H
DR. G.H. J.J.
CH. E.Cook

APPROVED
DATE AUG. 12, 75

APS-2742
G-557

APS DWG. #G-44557

APS-2742 G-558



PROFILE - ALONG ℄ OF DIKE (LOOKING UPSTREAM)
VERT 1" = 80'
HOR 1" = 200'

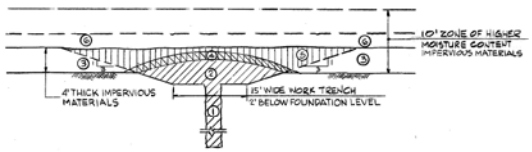


SECT J (N16)

NOT RELEASED

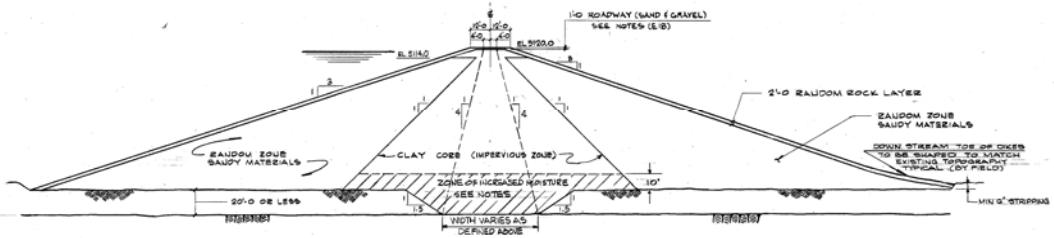
QUANTITIES (LIST, BY FIELD UNLESS NOTED)

STRIPPING (CUT-OFF TRENCH EXCAVATION	109,100 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	84,100 SQ FT

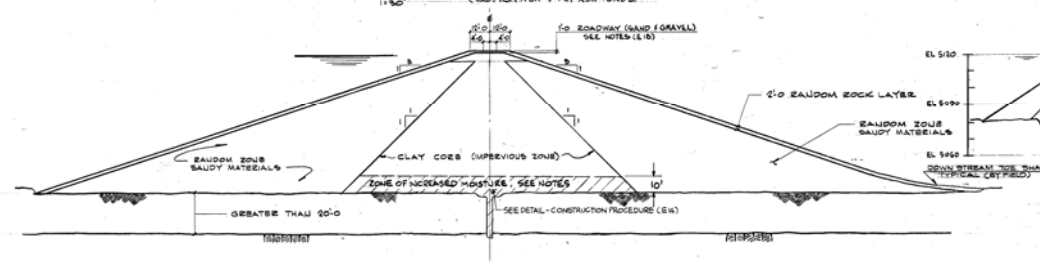


CONSTRUCTION PROCEDURE
SLURRY WALL - IMPERVIOUS ZONE CONTACT (T5)

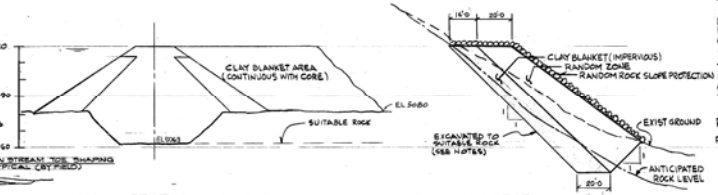
① SLURRY WALL TO BE CONSTRUCTED WITHIN MINIMUM 12' WIDE BY 2' DEEP WORK AREA.
② COMPLETED WALL THEN BACKFILLED WITH SOIL BENTONITE FROM WALL EXCAVATION OR PROCESSED MATERIAL TO BE CROWNED TO 2' ABOVE ORIGINAL BASE OF IMPERVIOUS ZONE.
③ IMPERVIOUS ZONE MATERIAL AT ABOVE OPTIMUM MOISTURE IS PLACED AND 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.



TYPICAL SECTION (IF DISTANCE FROM GRADE TO ROCK IS LESS THAN OR EQUAL TO 20'-0)
(BOTTOM ASH ℄ FLY ASH PONDS)
1" = 30'



TYPICAL SECTION (IF DISTANCE FROM GRADE TO ROCK IS GREATER THAN 20'-0)
(BOTTOM ASH ℄ FLY ASH PONDS)
1" = 30'



SECT K (H1)
1" = 30'

SECT L (H2)
1" = 30'

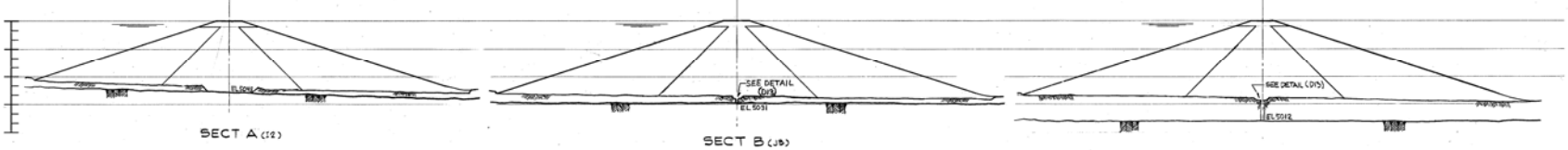
NOTES

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 18'-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 (T15) TYPICAL FOR BOTH FLY ASH AND BOTTOM ASH DAMS.

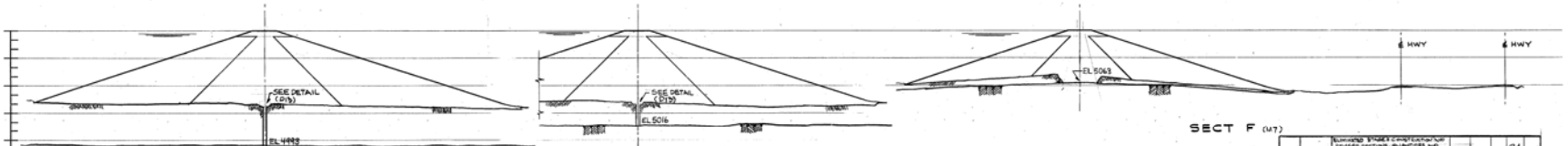
LOWER TEN FEET OF IMPERVIOUS ZONES FOR BOTH FLY ASH AND BOTTOM ASH DAMS TO BE PLACED AND COMPACTED AT OPTIMUM MOISTURE CONDITIONS AS PER SPECIFICATION APS - 2742-34-09.

THE FOUNDATION FOR THE CLAY BLANKET (H15) SHALL BE EXCAVATED TO SUITABLE ROCK AND CLEANED, MAPPED AND SLUSH GROUTED AS PER SECTION 7 OF SPEC. APS-2742-84-09.

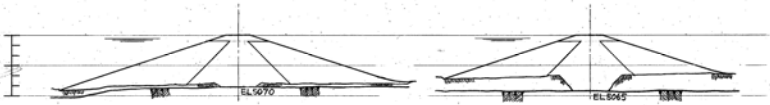
REFERENCE DRAWINGS
FOR REF. DWG SEE G-556



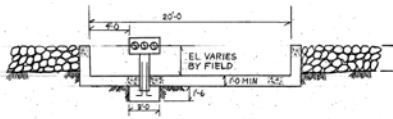
SECT A (I2)
SECT B (J3)
SECT C (K4)



SECT D (L3)
SECT E (M4)
SECT F (M7)



SECT G (N3)
SECT H (N5)



SECT M (L4)
(N.T.S.)

NO	DATE	REVISION	BY	CH.	APPROVED
10	11-9-77	AS BUILT	RM	NT	
9	5-10-77	REVISED SHAPE OF DIKE TOE (F10), (J10) & (M10)	AC	JE	—
8	2-28-77	ELIMINATED SUNDRY CONFIGURATION REVISED SECTION, QUANTITIES AND NOTES ACCORDINGLY. ADDED (E/B)(E/B) CONSTRUCTION PROCEDURE DETAILS E4, CHG'D SHT. 1	T.R.	BAD	—
7	9-29-76	REV: DELETED (D13,D5) SECT & NOTES	RM	JTB	—
6	8-24-76	REV'D (B3) REF DWGS (D7,G2) NOTES 6 ADD'D (E/B) NOTES	JJ	LC	—
5	7-12-76	REVS: 1 PIEZOMETER ADD'D (D15) SECT, (B3) NOTE	JJ	DC	—
4	5-18-76	REVISED BASE TO READ EL 5120 (SECT D) DELETED (B10 THRU B17 PROFILE OF MAIN WALL & TRENCH & SECT K, (A1, C3) WORDED	DD	LC	—
3	4-14-76	GENERAL REVISION	RP	DT	—
2	12-15-75	REV (N, AA, AT) ADD NOTE ADD'D (E/B) NOTES	JJ	LC	—
1	9-24-75	ADD'D (A1, AT) NOTE REV'D (E 5) NOTE, (C8) ROCKLINE	JJ	LC	—

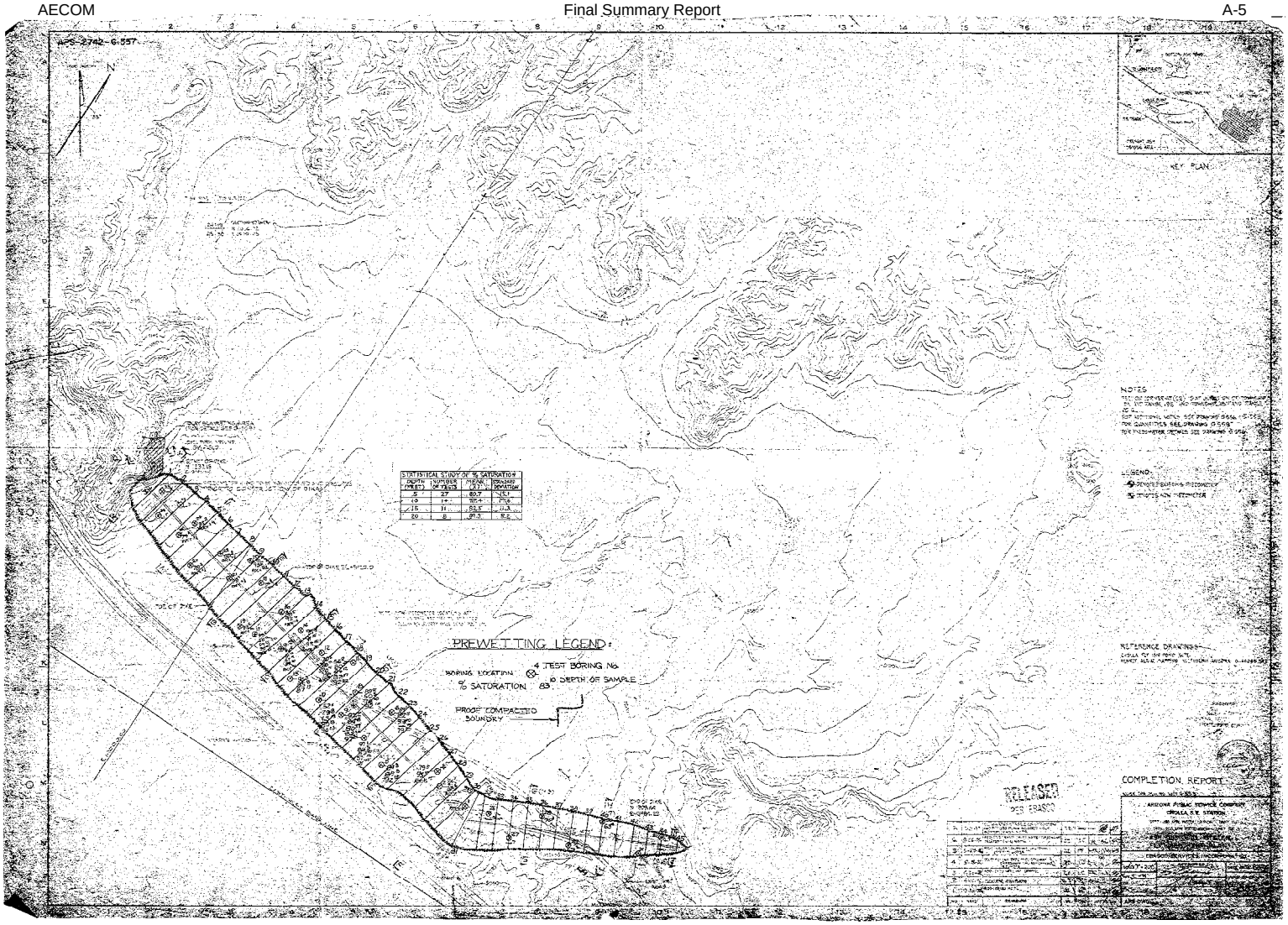
WORK THIS DWG WITH G-556 ℄ G-557

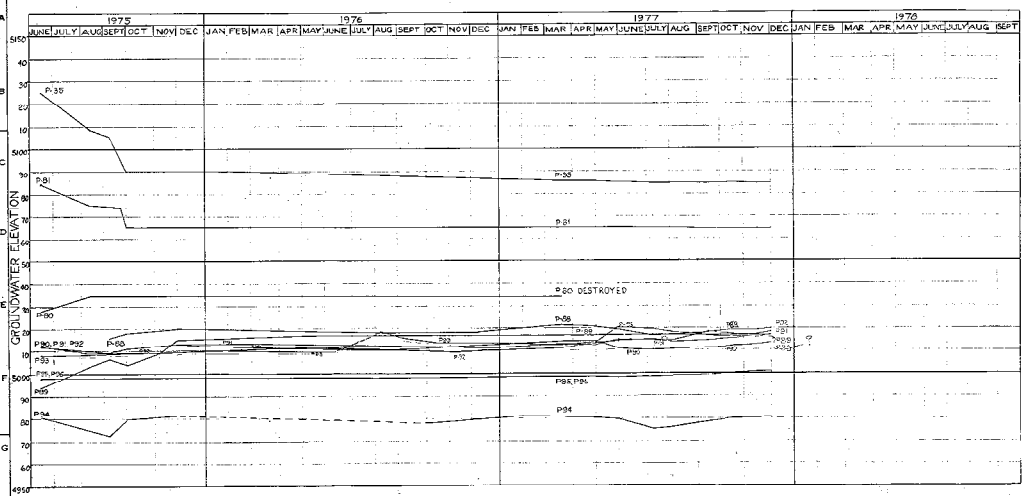
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1977-250 MW INSTALLATION - UNIT No. 2
1978-250 MW EXTENSION - UNIT No. 3
ASH DISPOSAL SYSTEM
FLY ASH POND DIKE-SECTIONS & DETAILS

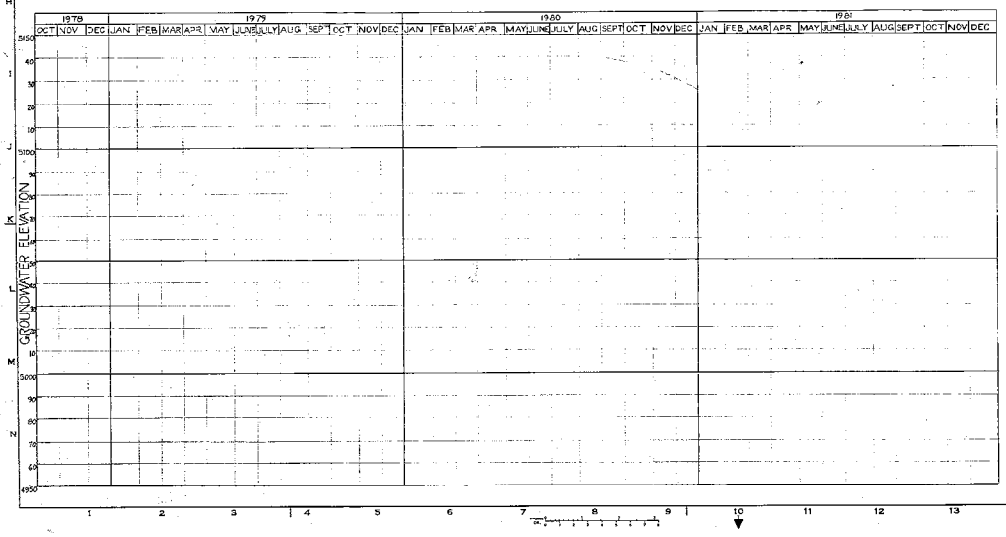
EBASCO SERVICES INCORPORATED

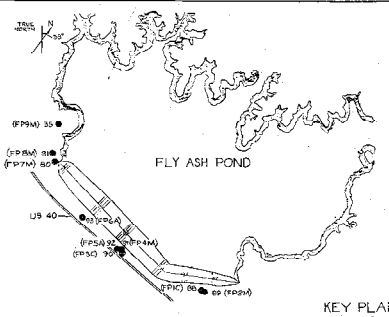
SCALE 1"=50'	APPROVED
DR CH	DATE AUG. 19, 25
DES J.SEPRATE	APS-2742
CH R.JONES	G-558

APS DWG #G-44558

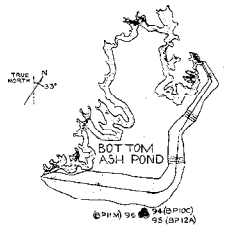

W.O. 2742-G.557
KEY PLAN
NOTES
LEGEND:
STATISTICAL STUDY OF % SATURATION
DEPTH (FEET)
NUMBER OF TESTS
MEAN (%)
STANDARD DEVIATION
PREWETTING LEGEND:
BORING LOCATION
TEST BORING NO.
% SATURATION
@ DEPTH OF SAMPLE
PROOF COMPACTED BOUNDRY
REFERENCE DRAWINGS
RELEASED
PER ERASCO
COMPLETION REPORT







KEY PLANS
N.T.S



PIEZOMETER LEGEND:
F - FLY ASH
B - BOTTOM ASH
P - PIEZOMETER
C - COCONINO
M - MOENKOPI
A - ALLUVIUM

INSTALLATION DATA 6-75

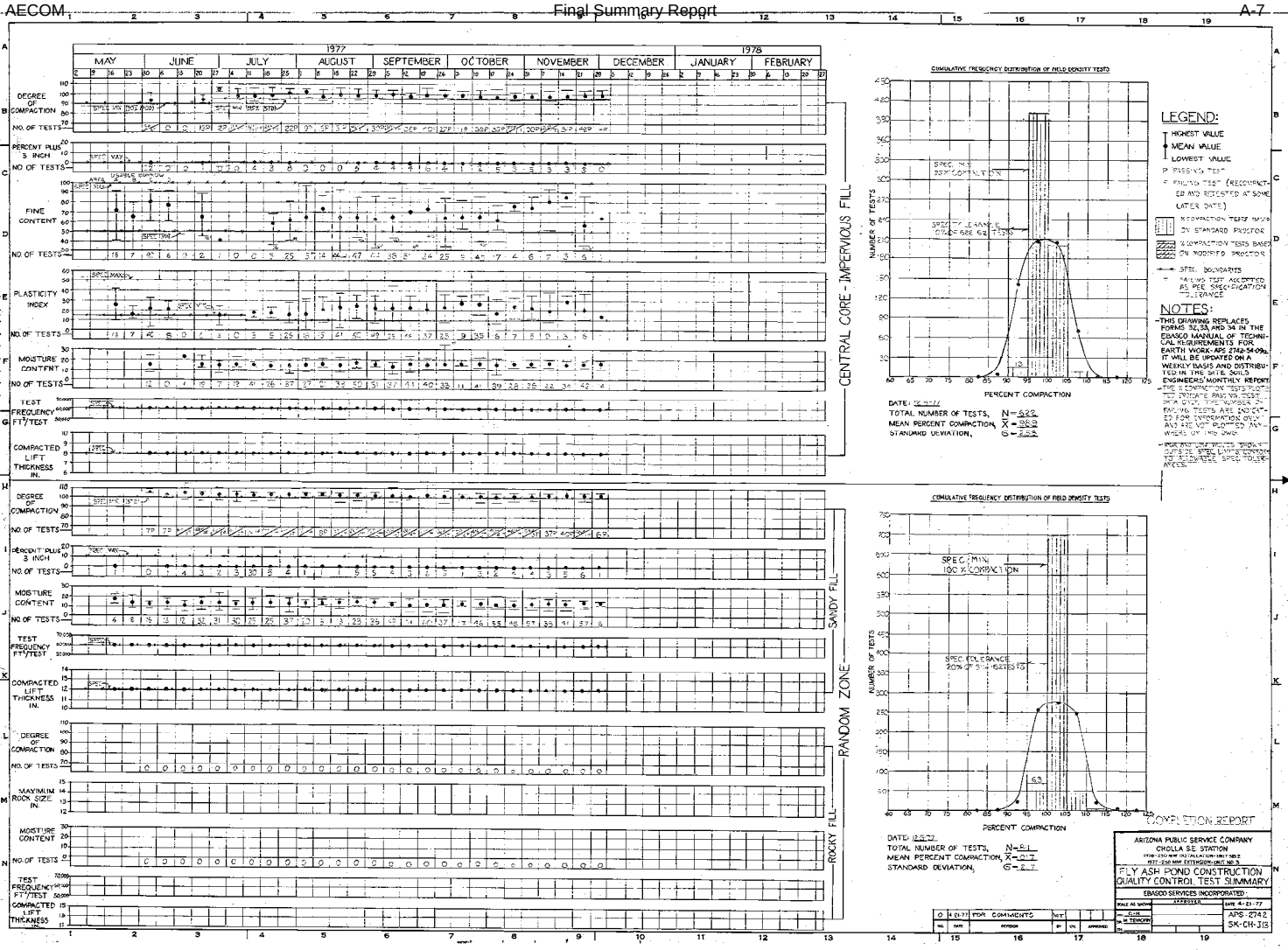
PIEZOMETER NUMBER	DRILLED DEPTH	DEPTH TO SCREEN	TOP OF RISER PIPE ELEVATION	RISER HEIGHT
35 (FP9M)	132	32	5191.3	3.2
80 (FP7M)	58	43	5083.0	1.7
81 (FP8M)	112	92	5164.0	1.5
88 (FP1C)	191	175	5086.5	2.0
89 (FP2M)	92	52	5085.9	1.8
90 (FP3C)	146	133	5036.7	3.0
91 (FP4M)	77	57	5025.1	1.3
92 (FP5A)	56	41	5034.3	1.2
93 (FP6A)	26	16	5034.2	2.0
94 (BP10C)	292	277	5030.7	3.0
95 (BP12A)	102	87	5049.3	2.4
96 (BP11M)	134	114	5049.9	3.1

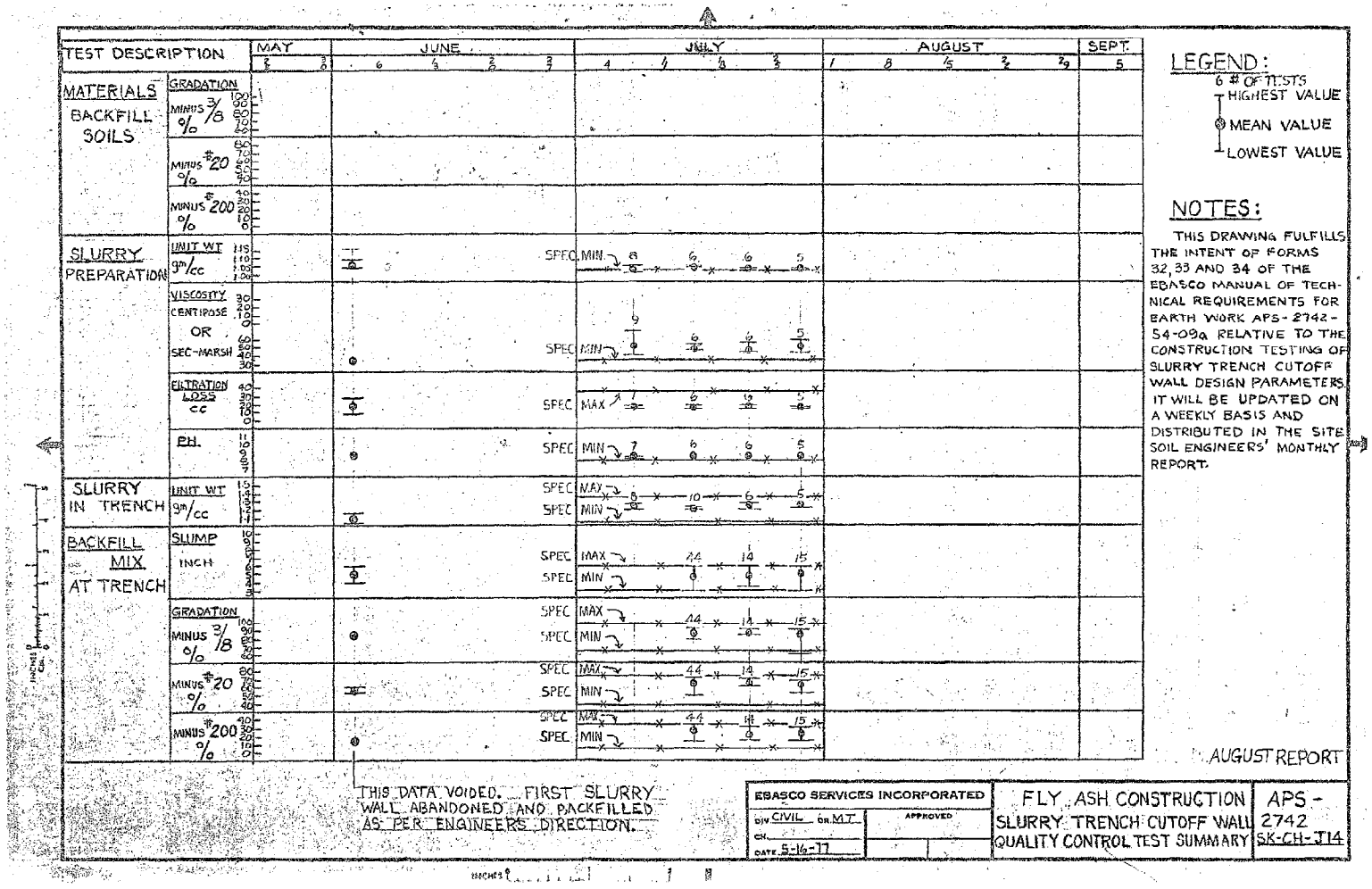
NOTES:

COMPLETION REPORT

NO.	DATE	REVISION	BY	CH.	APPROVED
0	11-29-77	FOR COMMENTS	MT		

ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
ASH POND PIEZOMETERS
GROUNDWATER ELEVATIONS
EBASCO SERVICES INCORPORATED
DRAWING NO. 12789-5-10
DATE 5-26-77
DR. E.C.H.
DES. D. TEMENS
CH.
APS-2742
SK-CH-7H


1977
MAY
JUNE
JULY
AUGUST
SEPTEMBER
OCTOBER
NOVEMBER
DECEMBER
1978
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
SPEC. MIN 95% COMPACTION
SPEC. TOLERANCE 20% OF 682 TESTS
DATE: 12-7-77
TOTAL NUMBER OF TESTS, N=682
MEAN PERCENT COMPACTION, X̄=98.2
STANDARD DEVIATION, σ=2.55
CUMULATIVE FREQUENCY DISTRIBUTION OF FIELD DENSITY TESTS
SPEC. MIN 100 % COMPACTION
SPEC. TOLERANCE 20% OF 511 TESTS
DATE: 12-5-77
TOTAL NUMBER OF TESTS, N=511
MEAN PERCENT COMPACTION, X̄=101.7
STANDARD DEVIATION, σ=2.7
LEGEND:
HIGHEST VALUE
MEAN VALUE
LOWEST VALUE
P PASSING TEST
F FAILING TEST (RECONSTRUCTED AND RETESTED AT SOME LATER DATE)
% COMPACTION TESTS BASED ON STANDARD PROCTOR
% COMPACTION TESTS BASED ON MODIFIED PROCTOR
SPEC. BOUNDARYS
** FAILING TEST ACCEPTED AS PER SPECIFICATION TOLERANCE
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.
-THE % COMPACTION TESTS PLOTTED ARE PRIVATE PROCTOR TEST DATA ONLY. THE NUMBER OF FAILING TESTS ARE INDICATED FOR INFORMATION ONLY AND ARE NOT PLOTTED ANYWHERE ON THIS DRAWING.
-FOR ENTOMBMENT VALUES SHOWN OUTSIDE SPEC LIMITS REFER TO ASSOCIATED SPEC TOLERANCES.
COMPLETION REPORT
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA SE STATION
FLY ASH POND CONSTRUCTION
QUALITY CONTROL TEST SUMMARY
EBASCO SERVICES INCORPORATED
APS-2742
SK-CH-313
0 4-21-77 FOR COMMENTS


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 CENTIPOISE 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
THIS DATA VOIDED. FIRST SLURRY WALL ABANDONED AND BACKFILLED AS PER ENGINEERS DIRECTION.
LEGEND:
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
EBASCO SERVICES INCORPORATED
DIV. CIVIL DR. MT
CH.
DATE 5-16-77
APPROVED
FLY ASH CONSTRUCTION
SLURRY TRENCH CUTOFF WALL
QUALITY CONTROL TEST SUMMARY
APS-2742
SK-CH-J14

SANDY MATERIAL RIPPED WEATHERED ROCK BORROW AREA (A)

CLAYEY MATERIAL BORROW AREA (B2)

CLAYEY MATERIAL BORROW AREA (B1)

HIEROGLYPHIC AREA FEDERAL LAND TO BE EXCLUDED

BORROW AREA (EXISTING)

WL 5118

WE 5030.5

N-S BASE LINE

E-W BASE LINE

ASH PIPES

DITCH

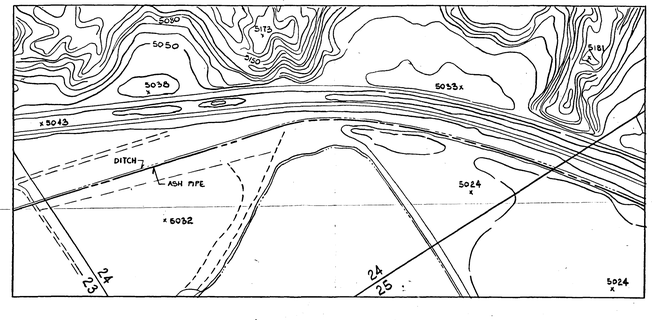
COAL PILE

PLANT SITE

TRANSFORMER SITE

SLUICE

SEE AREA BELOW



CLAYEY MATERIAL BORROW AREA (A)

SANDY MATERIAL BORROW AREA (E)

CLAYEY MATERIAL BORROW AREA (C)

CLAYEY MATERIAL BORROW AREA (B)

CLAYEY MATERIAL BORROW AREA (D)

SANDY MATERIAL BORROW AREA (F)

WL 5120

500' DISTANCE

PRELIMINARY

LEGEND:
- PROPERTY LINE
- ASH PIPE LINE
- WATER RETURN LINE

CREST OF DIKES EL 5120·0
IMPOUNDING LEVEL EL 5116·0
CONTOUR INTERVALS 10'

REVISED NOV 25, 1975
REVISED SEP 10, 1975

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

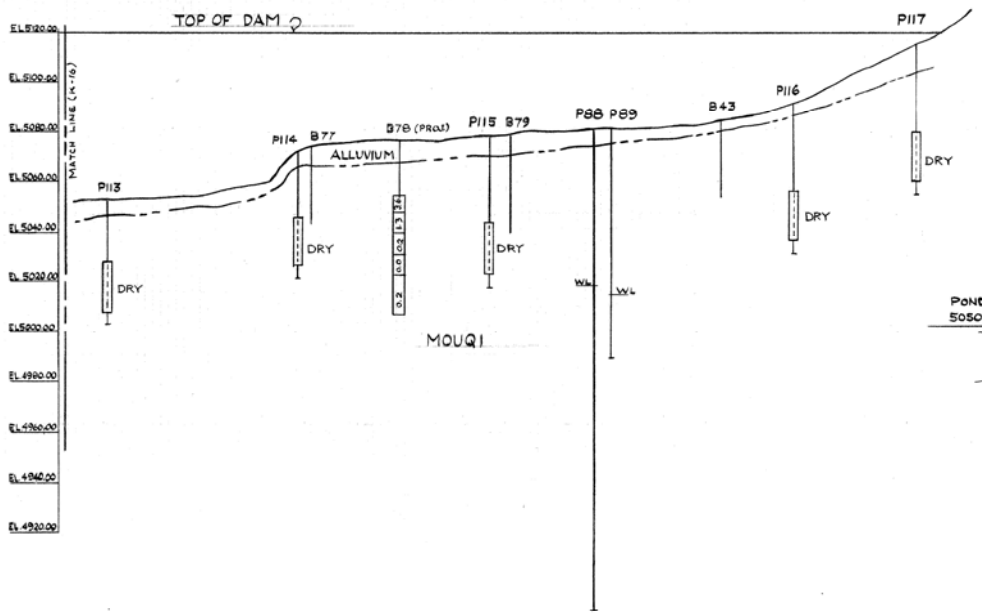
SCALE 1"=400'-0	APPROVED	DATE
DIV. C-H		APS-2742
DR D.KOVACHEV		SK-CH-160
CH.		

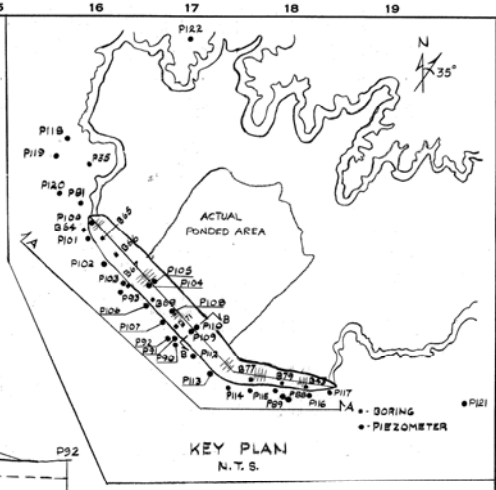
APS DWG. G44254 P2

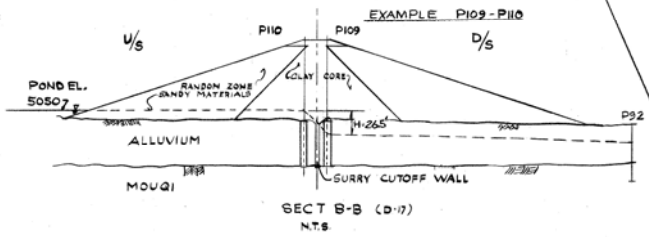
NO.	DATE	REVISION	BY	CH.	APPROVED

ENGINEERING DEPT. AUG 6 1990

09.27
09.28







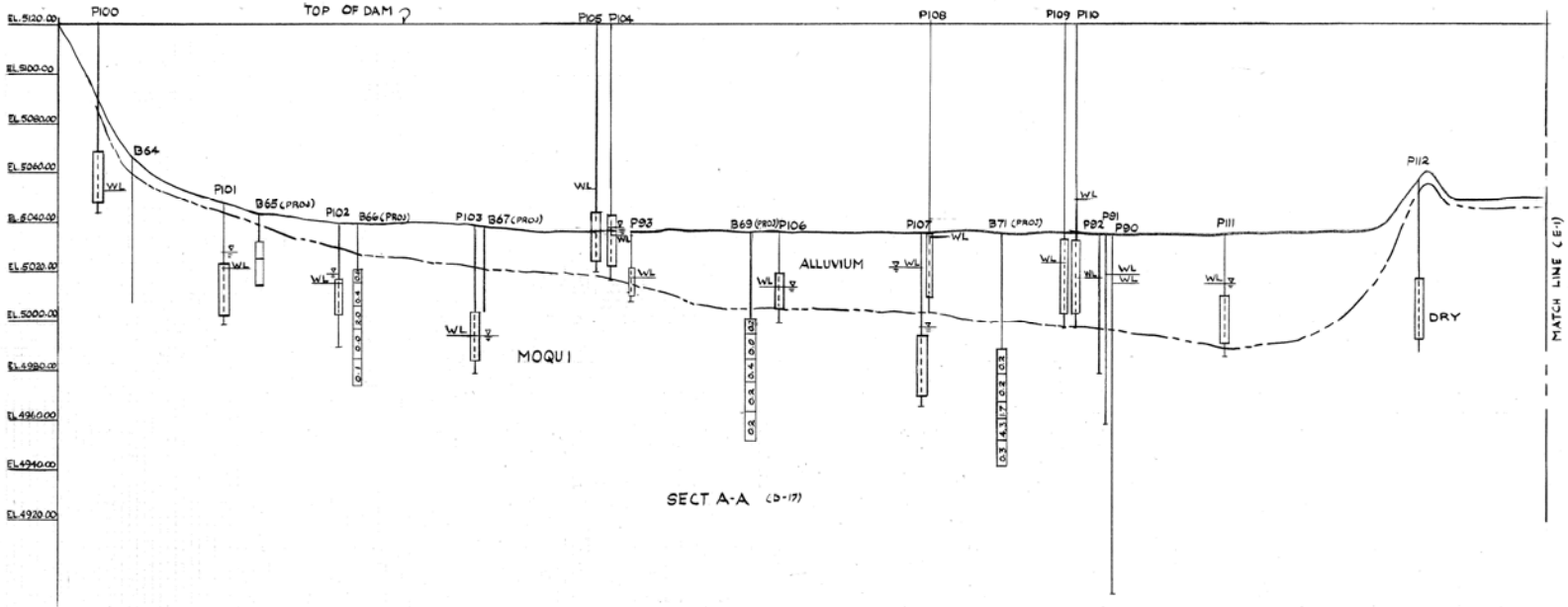
LEGEND

WL - PRESENT PIEZOMETER LEVEL

▽ - WATER LEVEL AT TIME OF INSTALLATION

- WATER PRESSURE TEST RESULT IN G.P.M.

- PIEZOMETER SCREEN LOCATION



UPDATED TO 6-1-79

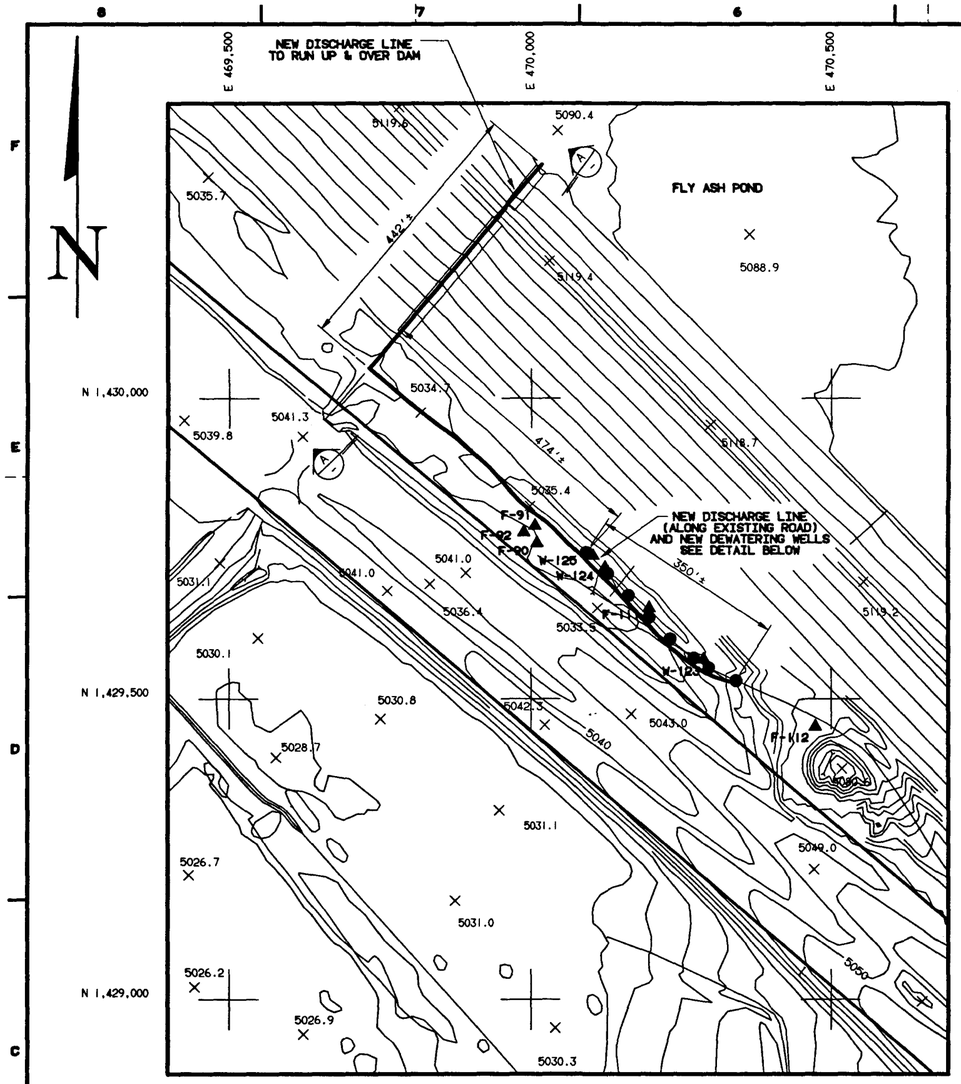
ARIZONA PUBLIC SERVICE CO. CHOLLA G.E. STATION	
FLY ASH DAM PIEZOMETER RESPONSE - PROFILES	
EBASCO SERVICES INCORPORATED	
SCALE AS NOTED	APPROVED DATE 5-10-79
DW. CIVIL CK. R. HOWARD CH. W. TOMNEY	

NO. | DATE | REVISION | BY | CH. | APPROVED

SHEET #G-77376

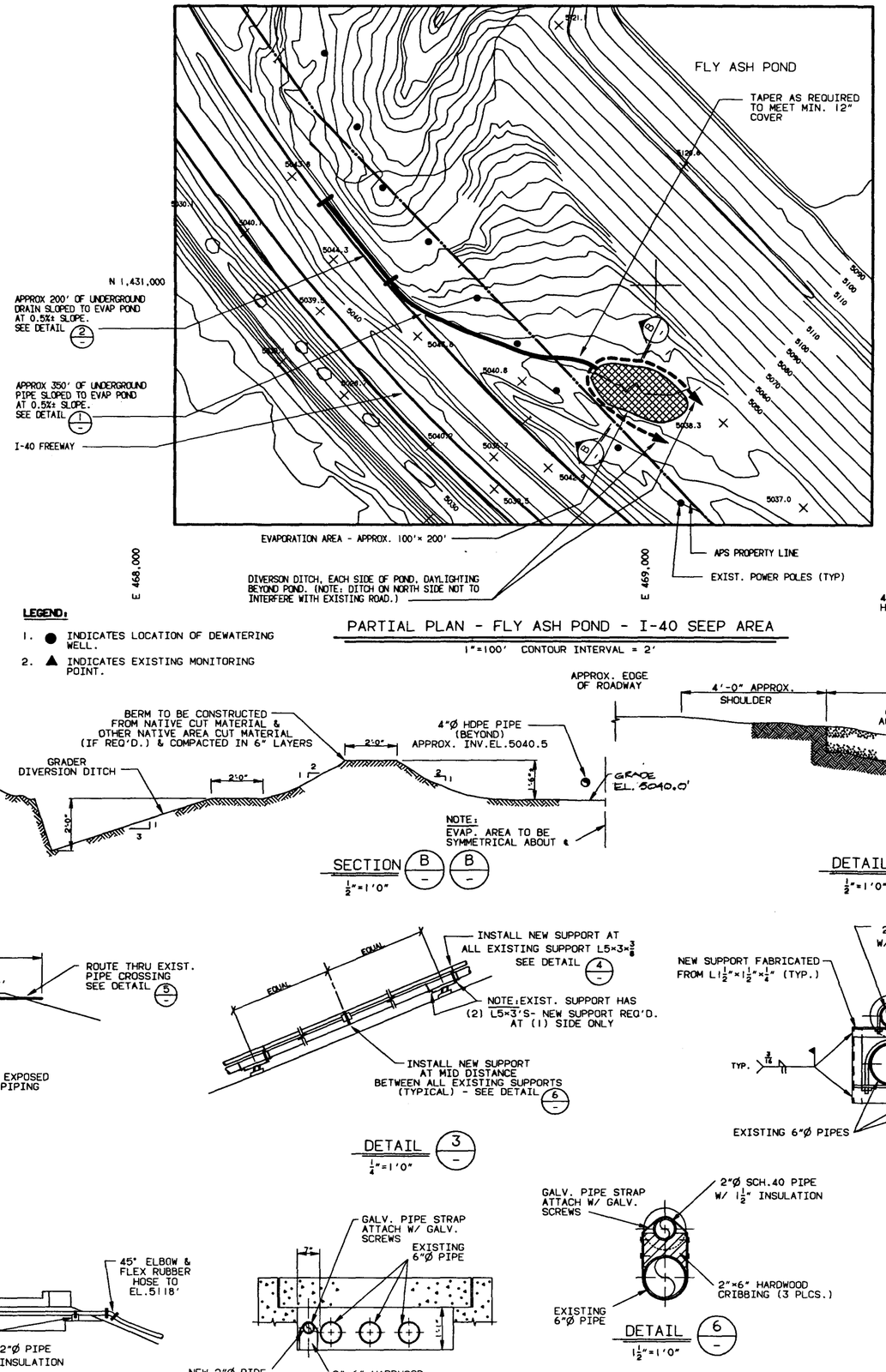
SEEPAGE INTERCEPT SYSTEM DRAWINGS

(APS, 1993)



PARTIAL PLAN - FLY ASH POND AREA - GERONIMO SEEP

1" = 100' CONTOUR INTERVAL=2"



PARTIAL PLAN - FLY ASH POND - I-40 SEEP AREA

1"=100' CONTOUR INTERVAL = 2'

LEGEND:

1. ● INDICATES LOCATION OF DEWATERING WELL.
2. ▲ INDICATES EXISTING MONITORING POINT.

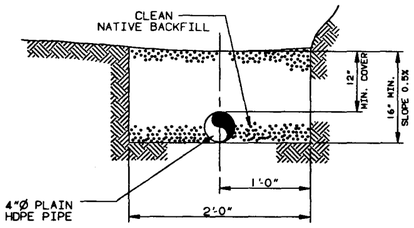
GENERAL NOTES

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 (Nº 100 TO Nº 200 U.S. STD. SIEVE SIZES) WITH A MIN. THICKNESS OF 1mm.
2. DRAINAGE ROCK SHALL BE CLEAN CRUSHED ROCK WITH THE FOLLOWING GRADATION:

100%	PASSING	1½" SIEVE
30% - 60%	"	1" "
0% - 15%	"	¾" "
0% - 2%	"	⅜" "

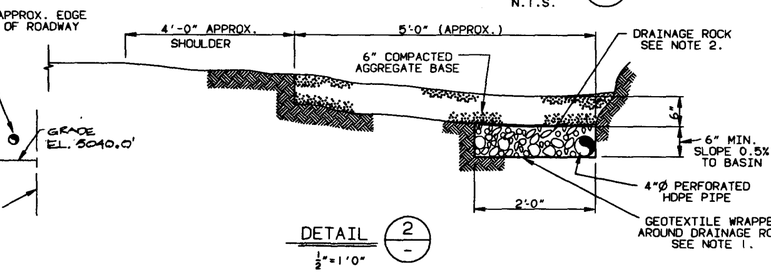
AN ACCEPTABLE SOURCE FOR DRAINAGE ROCK IS THE BLACK BASALT SUPPLIED BY BRIMHALL SAND & ROCK, SNOWFLAKE, AZ. (602)536-4226.

3. HDPE DRAINAGE PIPE SHALL BE CORRUGATED POLYETHYLENE PIPE MANUF. BY HANCOR INC. OR EQUAL. THE PIPE SHALL CONFORM TO AASHTO M294-831. 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.) WITH 2⅔"×½" 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'.

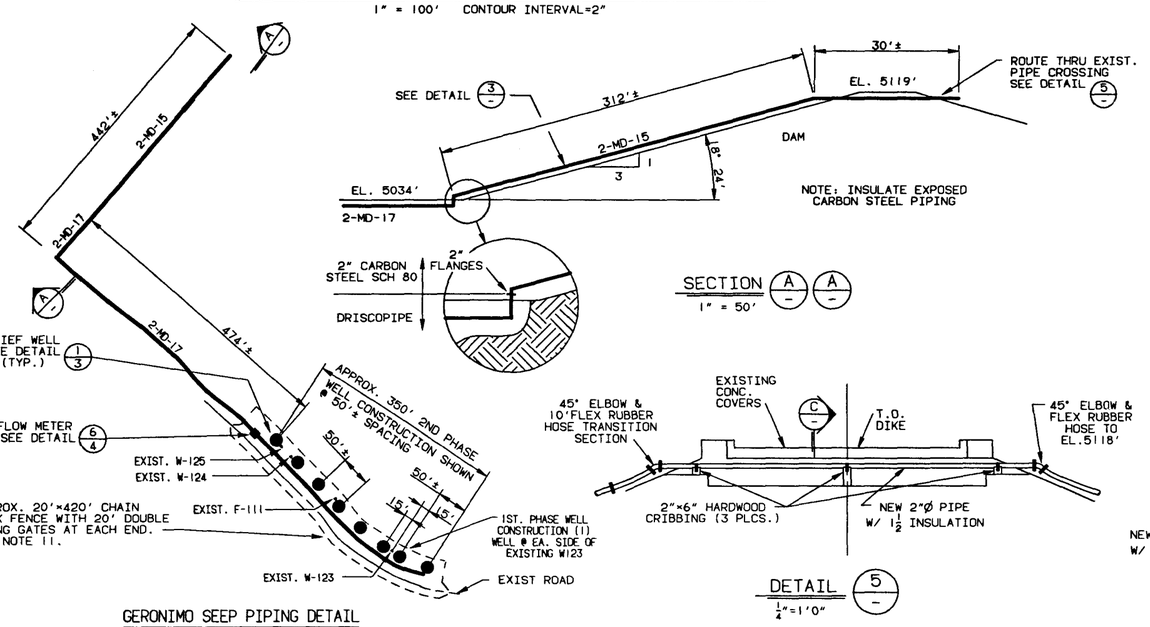


DETAIL 1 — N.T.S.

SECTION B — B — ½"=1'0"



DETAIL 2 — ½"=1'0"

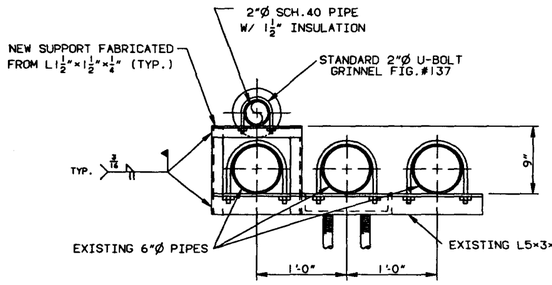


GERONIMO SEEP PIPING DETAIL

SECTION A — A — 1" = 50'

DETAIL 5 — ¼"=1'0"

DETAIL 3 — ¼"=1'0"



DETAIL 4 — 1½"=1'0"

SECTION C — ¾"=1'0"

DETAIL 6 — 1½"=1'0"

NO	DATE	REVISION	DRN	CHK	END	RVWD	APVD	U A

CHOLLA S.E.S. COMMON
BA/FA PONDS SEEPAGE INTERCEPT SYSTEM
FLY ASH POND AREA PLANS & DETAILS

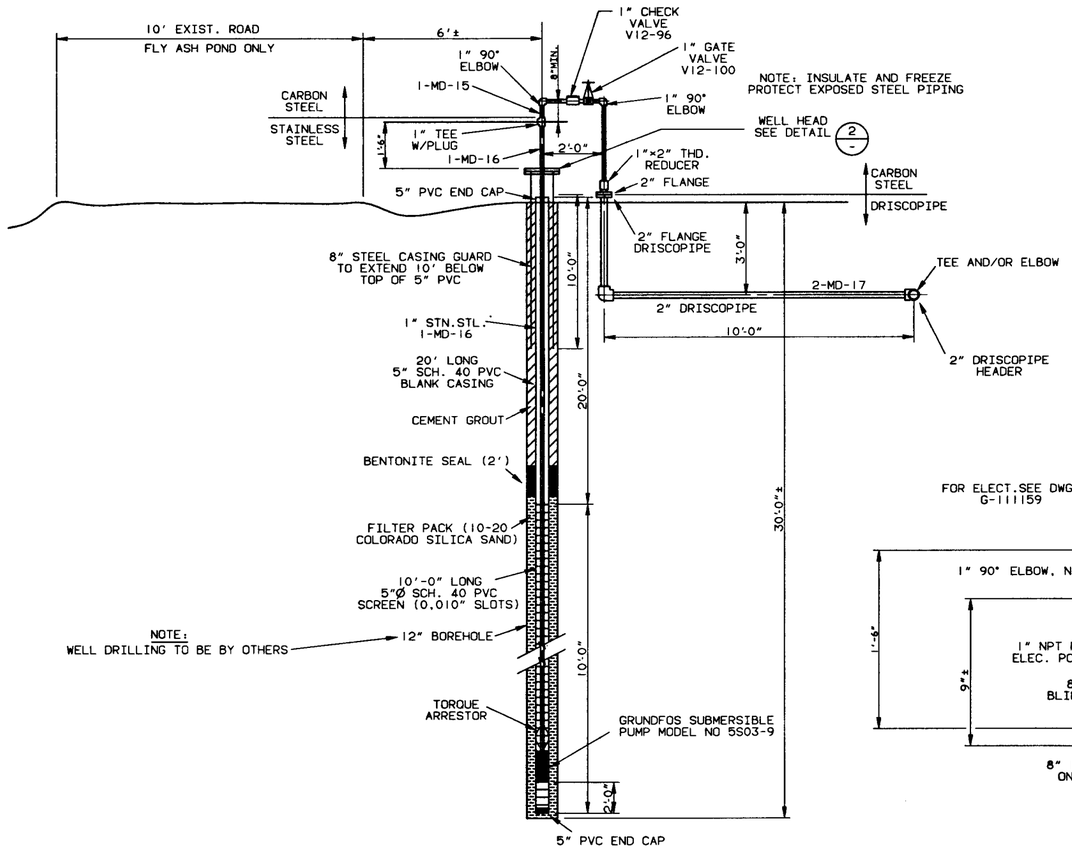
ARIZONA PUBLIC SERVICE COMPANY

SCALE: NOTED		DATE: 03-03-93	
DESIGNED: AGS/RP	APPROVED:		DRAWING NO. G-114438
DRAWN: TRH	REVIEWED:	ext. 62-9072	SHEET 1 OF 4
CHECKED:			

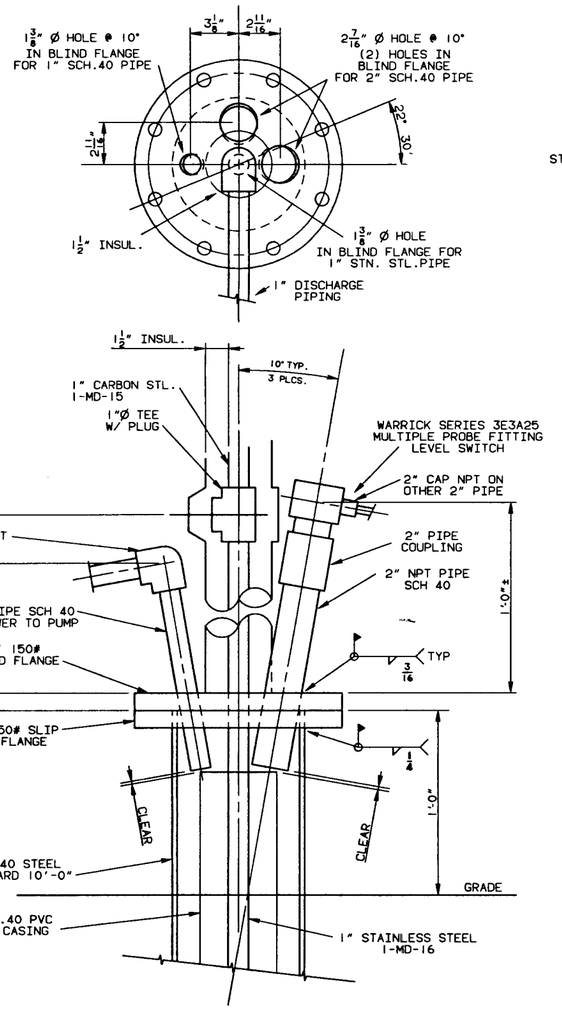
U A NO	
ENGLISH ENGRG	
CONSTR REVIEW	
FOR BID ONLY	
APVD FOR CONSTR	
AS BUILT	
REF ONLY	

THIS IS A CADAM DRAWING, DO NOT REVISE MANUALLY

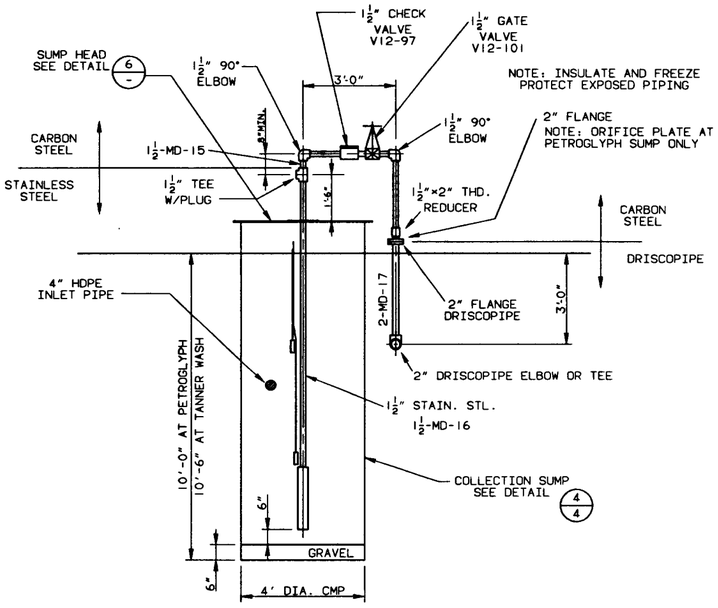
-- PLOT DATE: 04/09/93 AM
GROUP: GENE -- SUBGROUP: PRATT -- DWG ID: JF JG 114438 0001



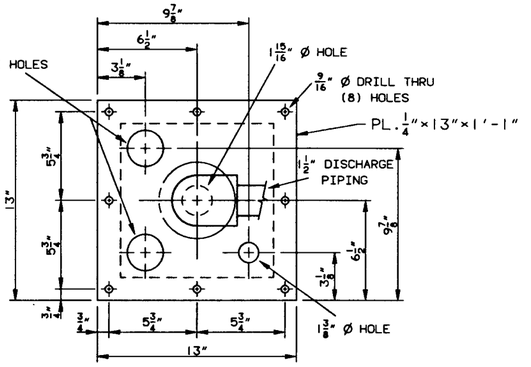
DETAIL (1) TYPICAL WELL INSTALLATION
1/2"=1'-0"



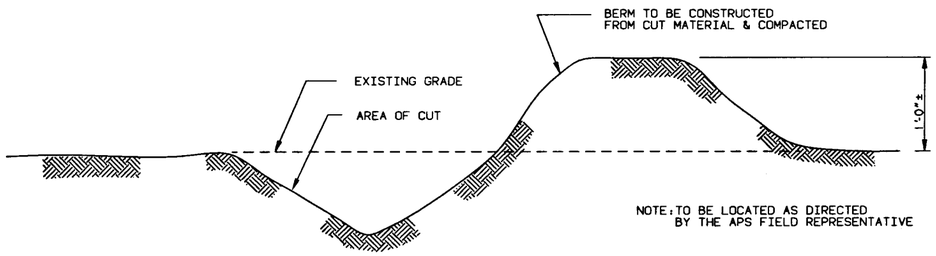
DETAIL (2) TYP. WELL HEAD
3"=1'-0"



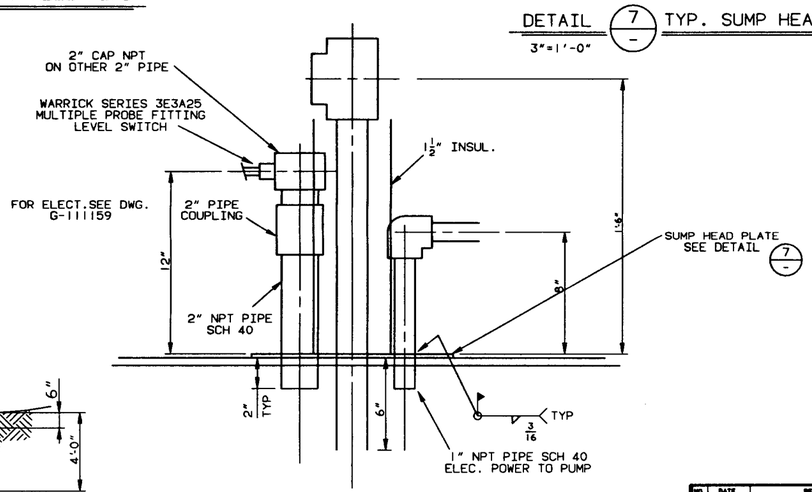
DETAIL (3) TYPICAL SUMP INSTALLATION
1/2"=1'-0"



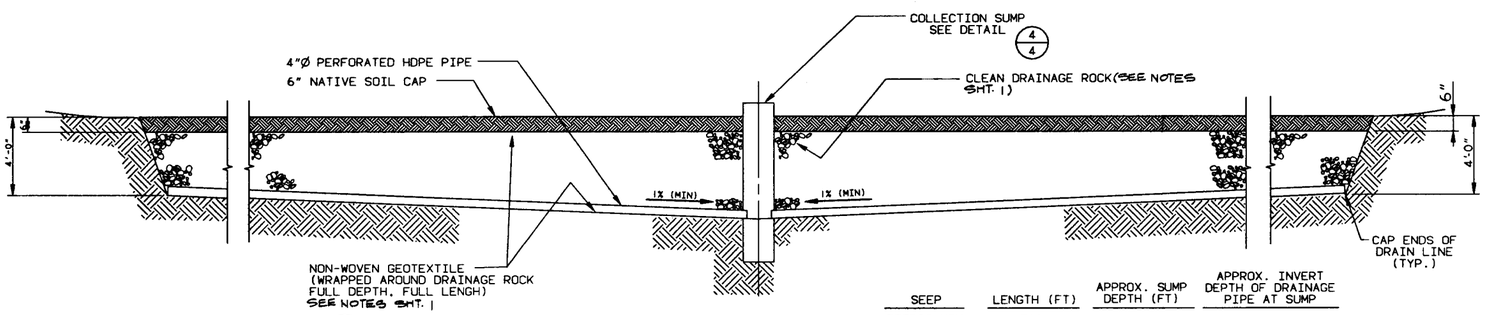
DETAIL (7) TYP. SUMP HEAD PLATE
3"=1'-0"



DETAIL (4) BERM TO CONTAIN RUNOFF WITHIN SALT SPOT
NO SCALE



DETAIL (6) TYP. SUMP HEAD
3"=1'-0"



DETAIL (5) TYPICAL DRAINAGE TRENCH LONGITUDINAL VIEW
NO SCALE

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"

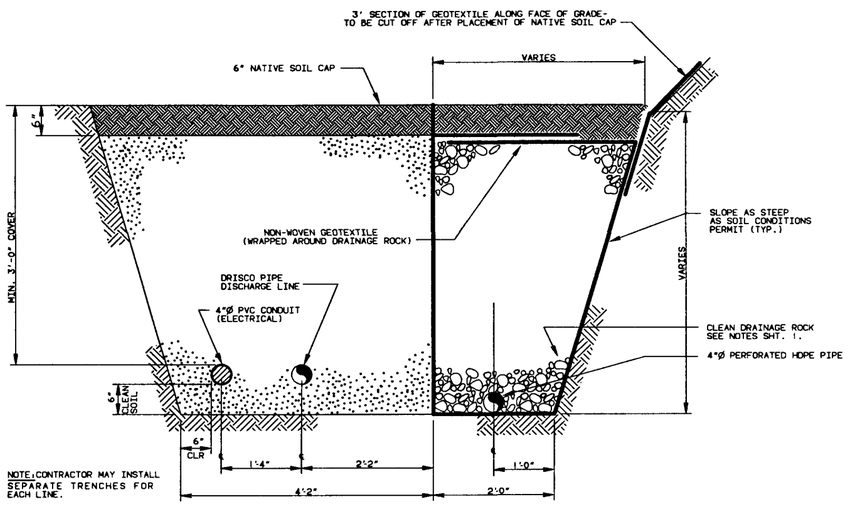
NO	DATE	REVISION	DWN	CHK	ENG	RVWD	APPD	U A

CHOLLA S.E.S. COMMON
BA/FA PONDS SEEPAGE INTERCEPT SYSTEM
SECTIONS & DETAILS

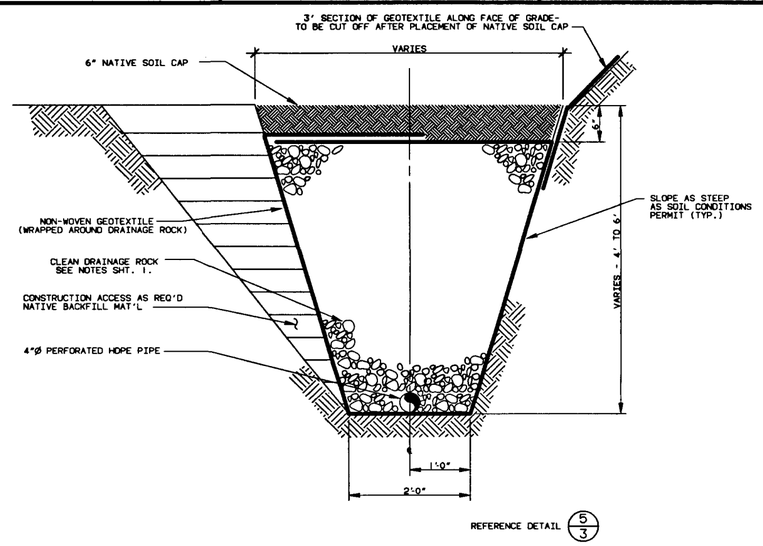
ARIZONA *PUBLIC SERVICE* COMPANY

SCALE NOTED DATE 03-30-93
DESIGN TRH
DRAWN PRATT
CHECKED
APPROVED
ENGINEERING MANAGER
REVIEWED
DRAWING NO. G-114438
SHEET 3 OF 4
M. Santos
U.A. 30-9072

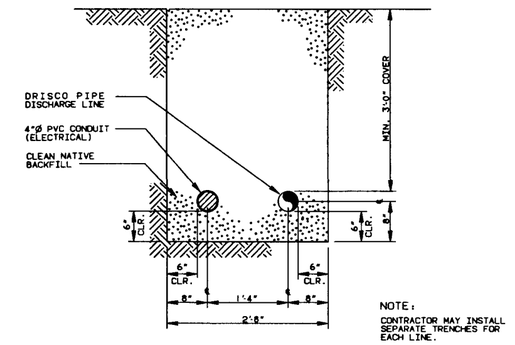
-- PLOT DATE: 04/06/93 mm
GROUP: GENE -- SUBGROUP: PRATT -- DWG ID: JF JG 114438 0003
THIS IS A CADAM DRAWING, DO NOT REVISE MANUALLY



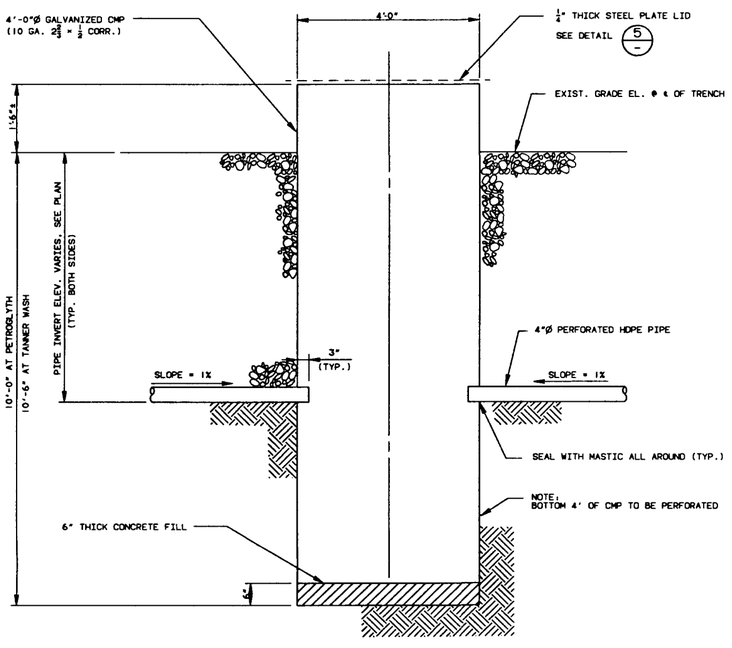
DETAIL 1 / 1"=1'-0" — TYP. JOINT TRENCH



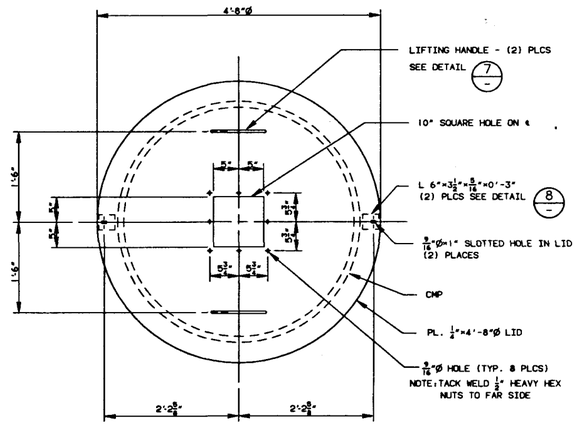
DETAIL 2 / 1"=1'-0" — TYPICAL INTERCEPT TRENCH



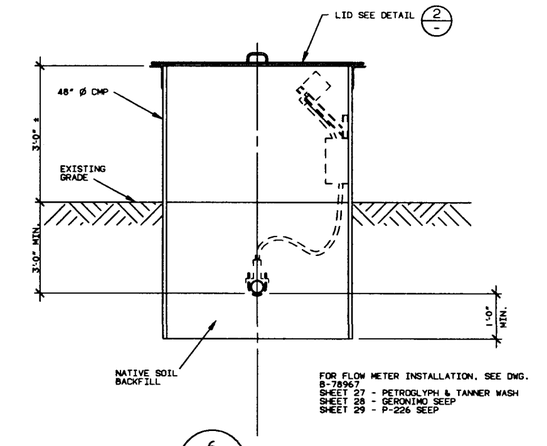
DETAIL 3 / 1"=1'-0" — TYPICAL UTILITY TRENCH



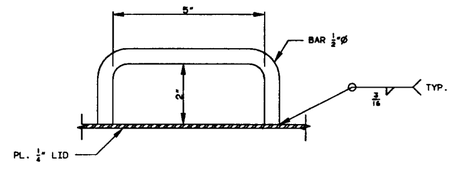
DETAIL 4 / ¾"=1'-0" — TYPICAL COLLECTION SUMP



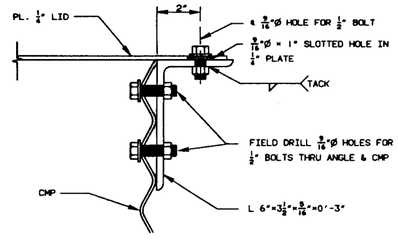
DETAIL 5 / 1"=1'-0" — TYPICAL SUMP LID



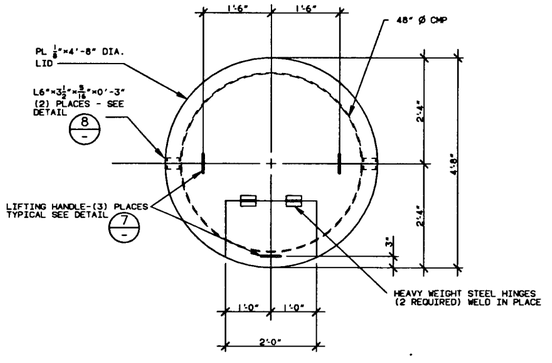
DETAIL 6 / ¾"=1'-0" — TYP. FLOW METER ENCLOSURE



DETAIL 7 / 6"=1'-0" — TYPICAL SUMP LID HANDLE



DETAIL 8 / 3"=1'-0" — TYPICAL CMP LID ATTACHMENT



DETAIL 9 / 1"=1'-0" — TYP. FLOW METER LID

NO	DATE	REVISION	DRN	CHK	END	RVWD	APPD	W A

CHOLLA S.E.S. COMMON BA/FA PONDS SEEPAGE INTERCEPT SYSTEM SECTIONS AND DETAILS

ARIZONA *PUBLIC SERVICE* COMPANY

SCALE NOTED — DATE 03-30-93

DESIGN R.PRATT — APPROVED — ENGINEERING MANAGER — DRAWING NO. G-114438

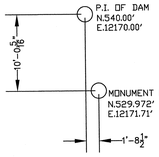
DRAWN M. Santos — CHECKED TRH — REVIEWED Mark Irwin — P.E. 62-9072 — SHEET 4 OF 4

THIS IS A CADAM DRAWING, DO NOT REVISE MANUALLY

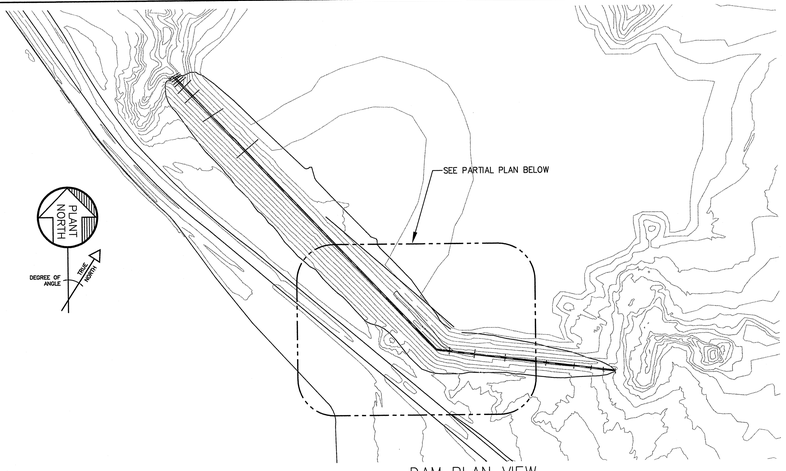
-- PLOT DATE: 04/08/93 -- GROUP: GENE -- SUBGROUP: PRATT -- DWG ID: 3F JG 114438 0004

EMBANKMENT MONITORING

(APS, 1999)



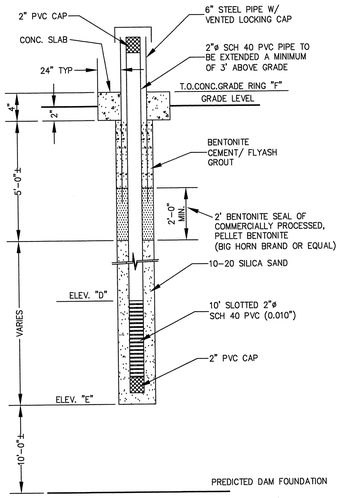
M7 TO DAM P.I. LOCATION



DAM PLAN VIEW

1"=500'

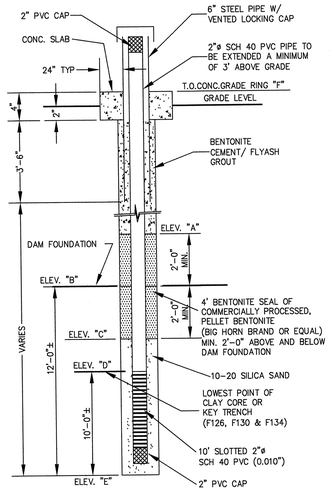
ALL CONTOURS AND ELEVATIONS SHOWN HERE ARE FROM ORIGINAL FLY ASH POND DAM. REFERENCE APS DRAWING G-44558 AND DAM ENGINEERING REPORT FIGURES 13 & 14



DETAIL (1) — PIEZOMETER DETAIL

NO SCALE

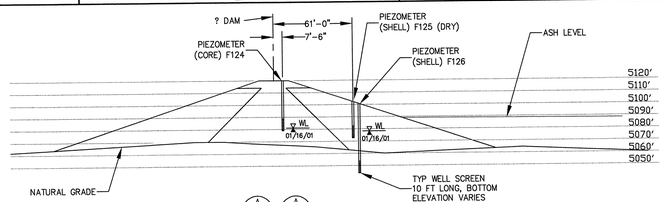
REF. TABLE 1 FOR ALPHA LABELED ELEVATIONS NOTED



DETAIL (2) — PIEZOMETER DETAIL

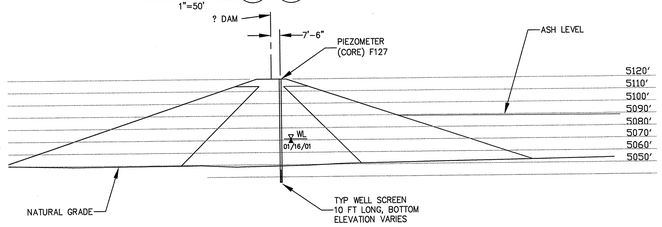
NO SCALE

REF. TABLE 1 FOR ALPHA LABELED ELEVATIONS NOTED



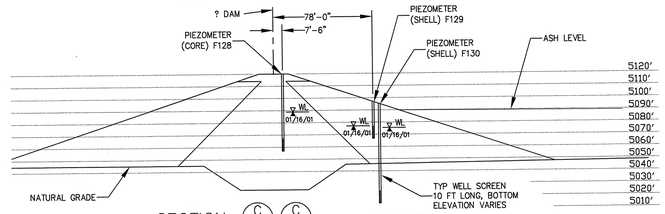
SECTION (A) (A) 203' WEST OF MONUMENT M7

1"=50'



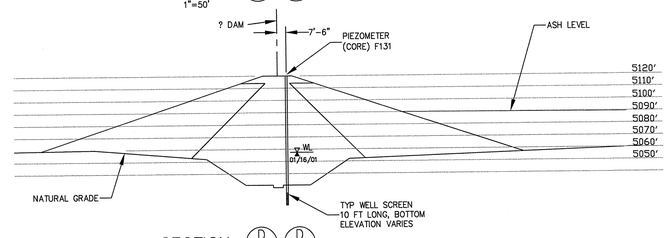
SECTION (B) (B) 303' WEST OF MONUMENT M7

1"=50'



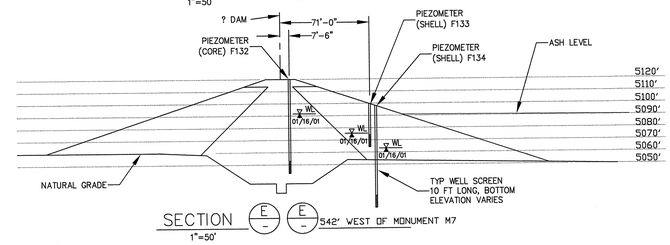
SECTION (C) (C) 403' WEST OF MONUMENT M7

1"=50'



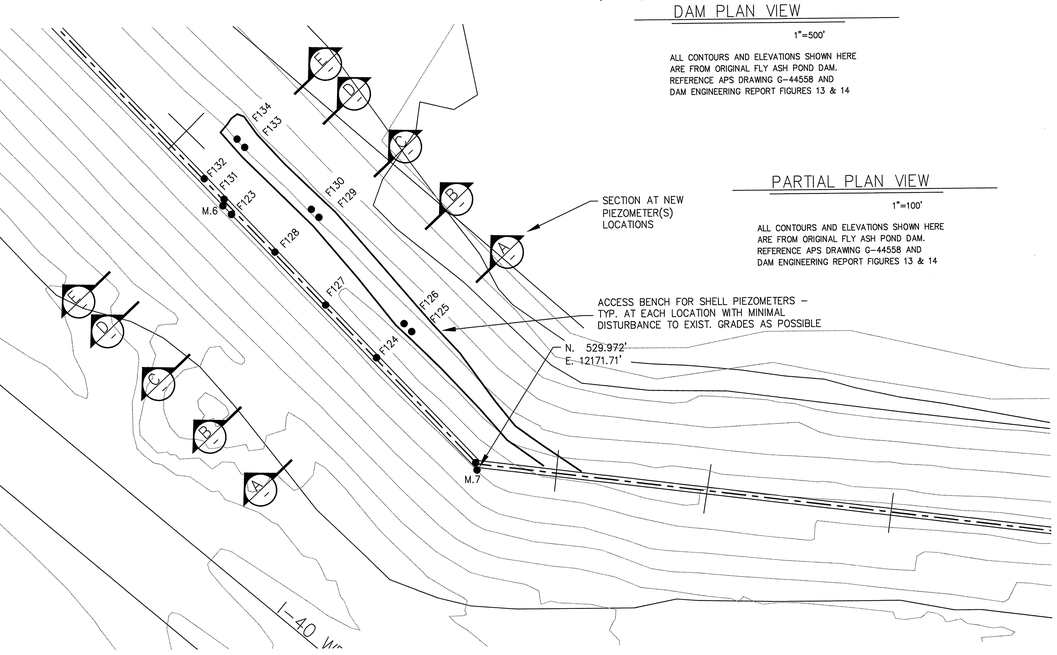
SECTION (D) (D) 503' WEST OF MONUMENT M7

1"=50'



SECTION (E) (E) 542' WEST OF MONUMENT M7

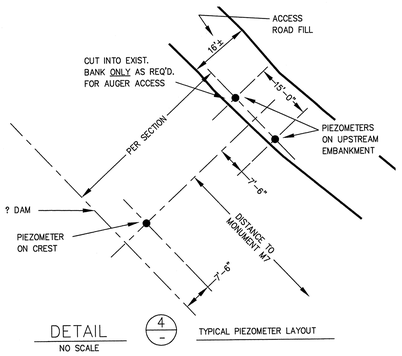
1"=50'



PARTIAL PLAN VIEW

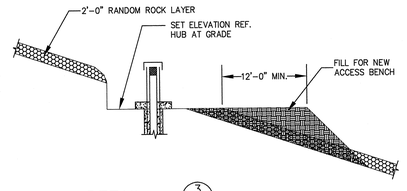
1"=100'

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



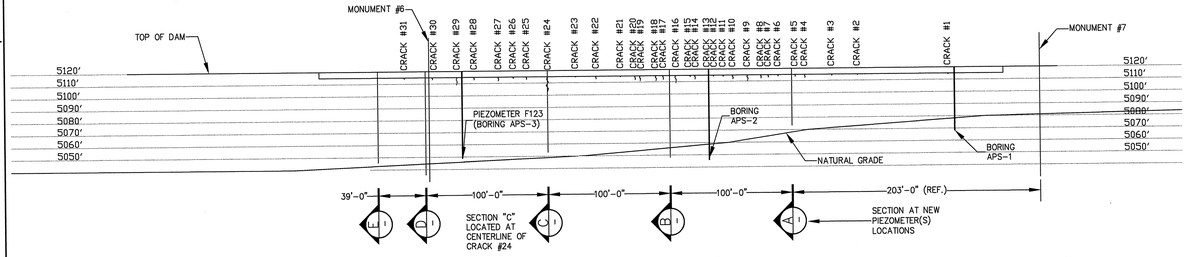
DETAIL (4) — TYPICAL PIEZOMETER LAYOUT

NO SCALE



DETAIL (3) — TYPICAL SECTION @ PIEZOMETER ON UPSTREAM FACE

NO SCALE



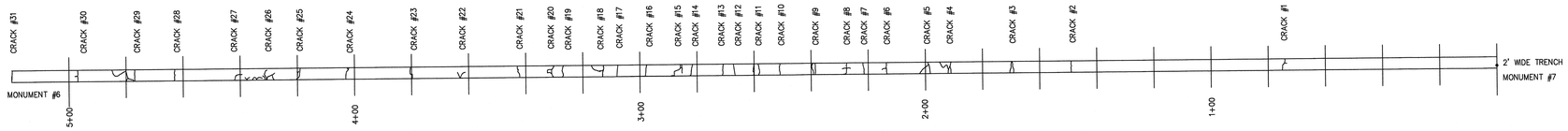
LONGITUDINAL VIEW AT CENTER OF DAM

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								



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

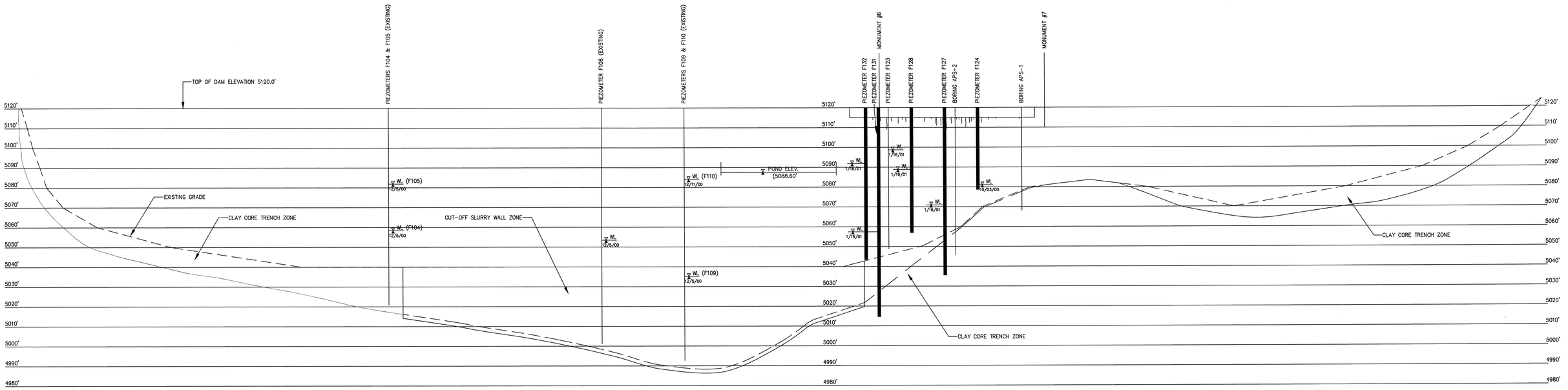
CHOLLA POWER PLANT
FLY ASH POND
EMBANKMENT MONITORING
PLAN AND SECTIONS

APS

SCALE NOTED — DATE 9-7-99

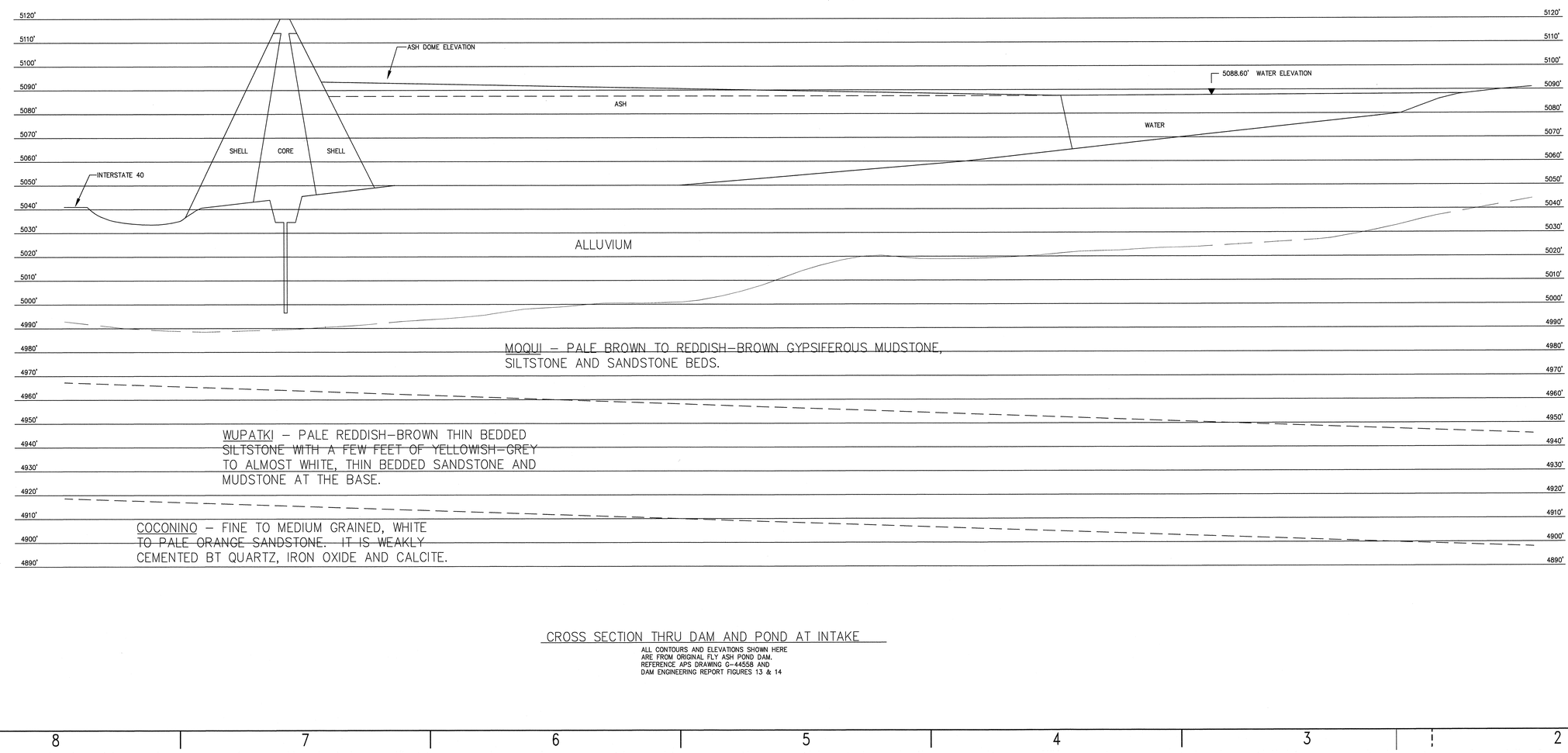
DWN	APVD	DRAWING NO.
KK/MLD	JOHN D. MITCHELL ENGINEERING SUPERVISOR	G-145746
CHD	RVWD	W.A. 30-6923
EXD RQF		SHEET 1 OF 2

AAAAMDL.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

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

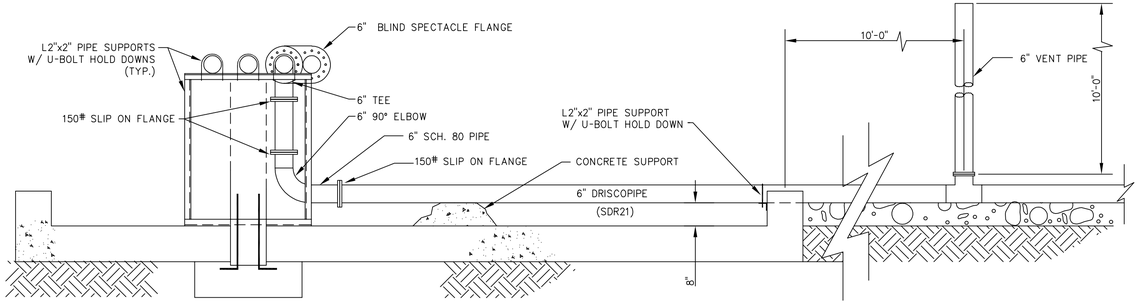
DWN	APVD	DRAWING NO.
KK/MLD	JOHN D. MITCHELL ENGINEERING SUPERVISOR	G-145746
CHD	RVWD	W A
EXD RQF		30-6923

SHEET 2 OF 2

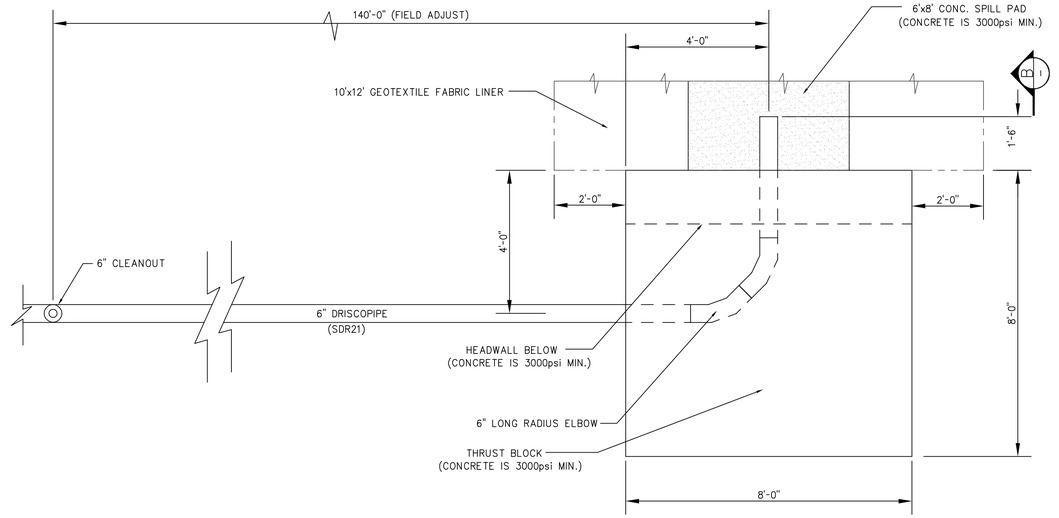
AAAAMDV.DWG

DISCHARGE PIPE DRAWINGS

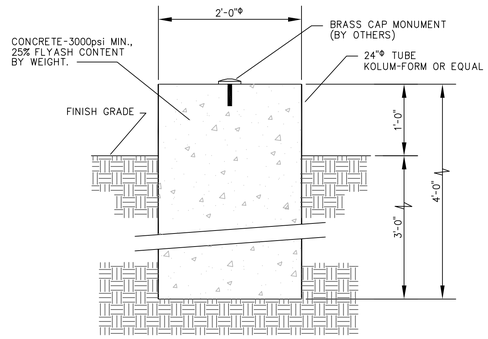
(APS, Rev. 2004)



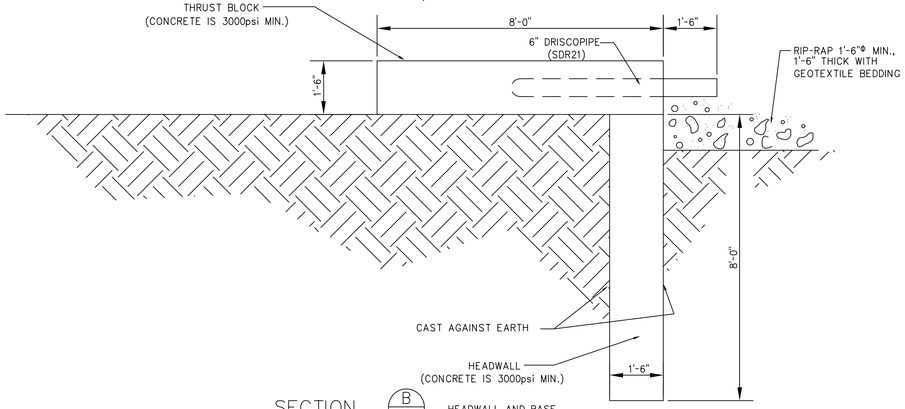
SECTION (A) NEW DISCHARGE CONNECTION
1/2"=1'-0"



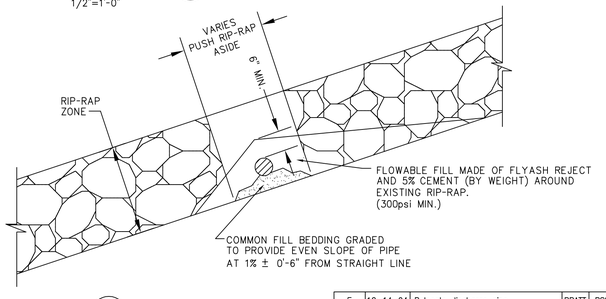
PLAN VIEW
(DISCHARGE POINT)
1/2"=1'-0"



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 — DATUM - WGS - 84 W/ TRUE NORTH ALIGNMENT
ELEVATION UNITS: US SURVEY FEET
PLANT COORDINATE UNITS: US SURVEY FEET — DATUM - PLANT SOURCE W/ PLANT NORTH ALIGNMENT

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

SCALE AS NOTED — DATE 01-19-01

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

REFERENCE DRAWINGS
G-145746
G-114438

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

AAAAOVR.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 <u>after</u> 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.

<u>and</u>

☒ 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				
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 1 of 15

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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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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 3 of 15

- 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

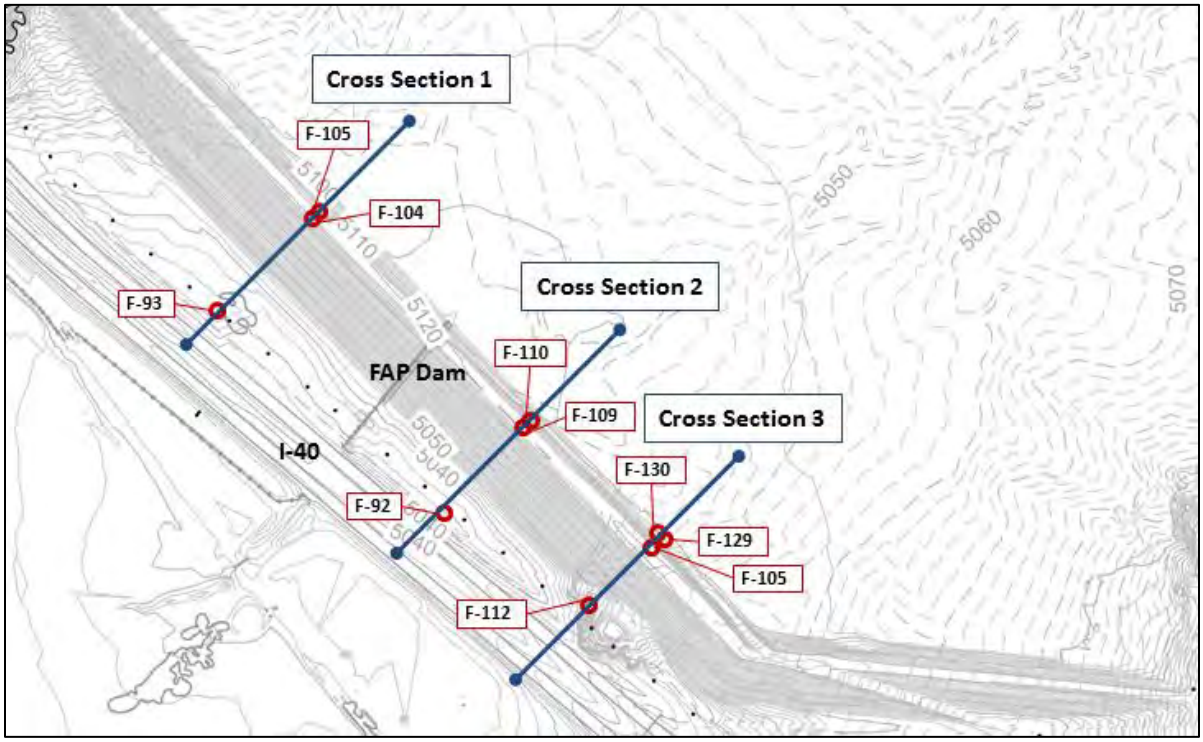


Figure 1. Slope Stability Cross Section and Piezometer Locations Along the FAP Dam

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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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 5 of 15

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).

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.

Long-Term, Maximum Storage Pool

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.

Maximum Surcharge Pool

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.

Seismic Loading

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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 8 of 15

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, $\overline{v}_s$, (ft/s)	Standard penetration resistance, $\overline{N}$	Soil undrained shear strength, $\overline{s}_u$, (psf)
A	Hard rock	$\overline{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \overline{v}_s \le 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \overline{v}_s \le 2{,}500$	$\overline{N} > 50$	$\overline{s}_u \ge 2{,}000$
D	Stiff soil profile	$600 \le \overline{v}_s \le 1{,}200$	$15 \le \overline{N} \le 50$	$1{,}000 \le \overline{s}_u \le 2{,}000$
E	Soft soil profile	$\overline{v}_s < 600$	$\overline{N} < 15$	$\overline{s}_u < 1{,}000$
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 $\overline{s}_u < 500$ 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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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 9 of 15

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).

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 10 of 15

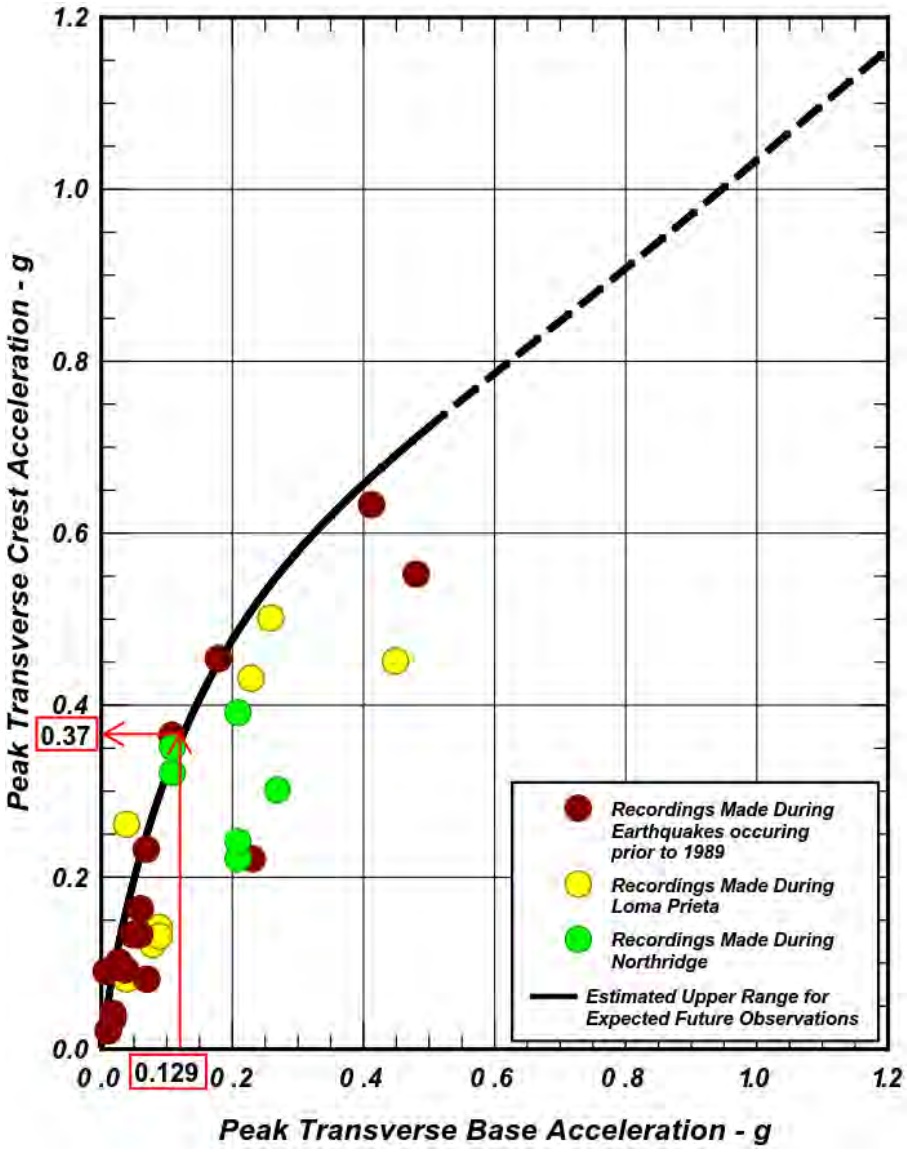


Figure 4. Variations of Peak Transverse Crest Acceleration vs. Peak Transverse Base Acceleration Based on Holzer (1998)

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:

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 11 of 15

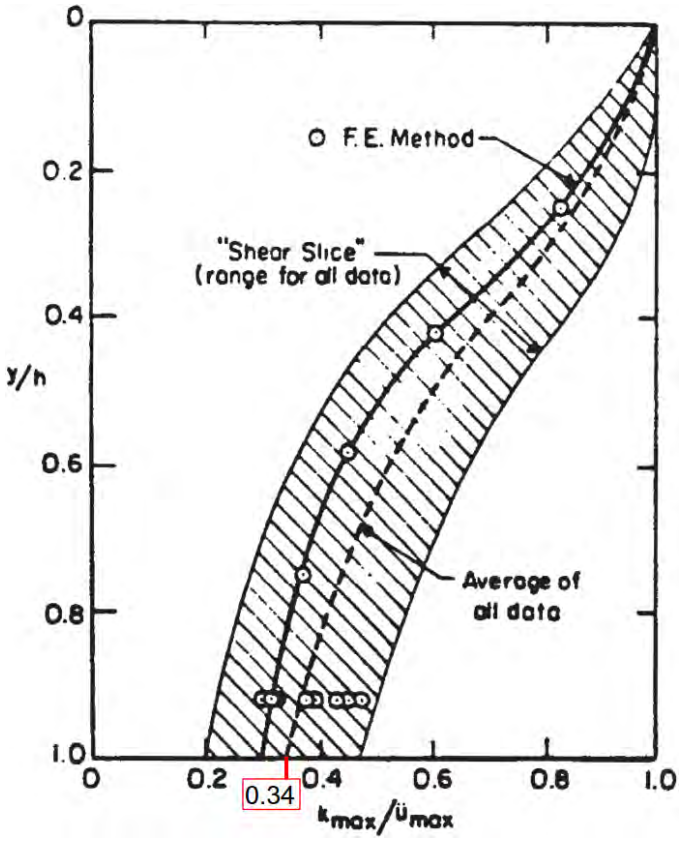


Figure 5. Variation of "Maximum Acceleration Ratio" with Depth of Sliding Mass after Makdisi and Seed (1977)

Therefore:

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

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

u_{max} = the maximum crest acceleration

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

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

$$k_{max} = 0.13g$$

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

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 12 of 15

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

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

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.

7 REFERENCES

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8 ATTACHMENTS

ATTACHMENT A USGS 2008 Seismic PSH Deaggregation

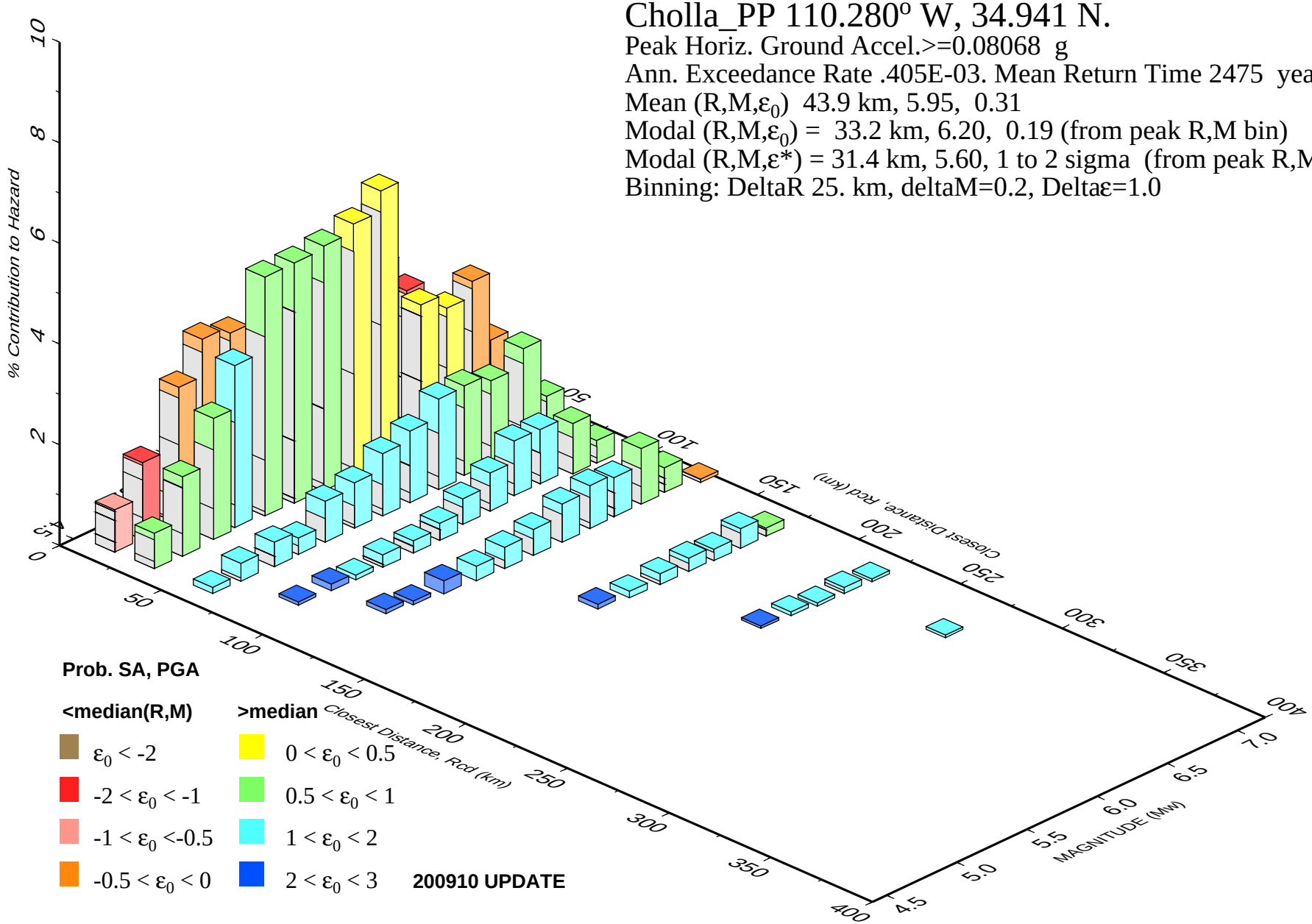
ATTACHMENT B Slope/W Output Figures

ATTACHMENT A

USGS 2008 Seismic PSH Deaggregation

PSH Deaggregation on NEHRP BC rock
Cholla_PP 110.280º W, 34.941 N.

Peak Horiz. Ground Accel.>=0.08068 g
Ann. Exceedance Rate .405E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 43.9 km, 5.95, 0.31
Modal (R,M,ε_0) = 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



ATTACHMENT B

Slope/W Output Figures

Slope Stability Analysis
Cross Section 1
Fly Ash Pond

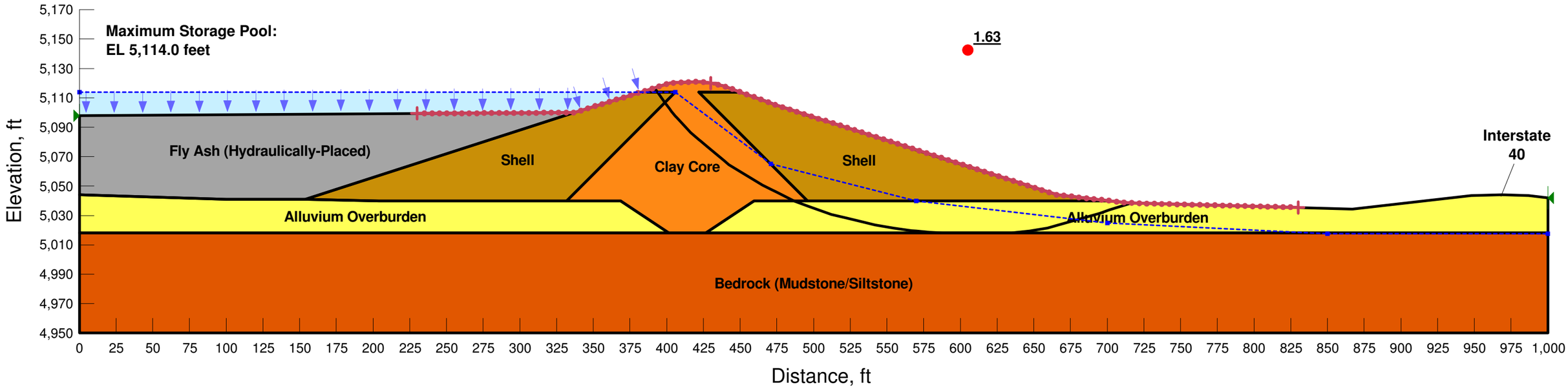
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

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

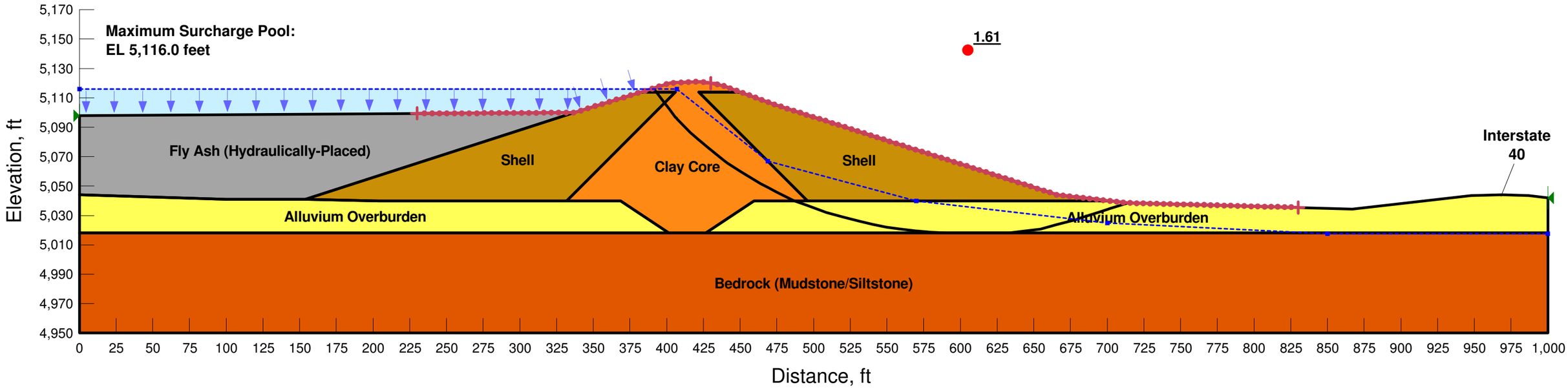
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
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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

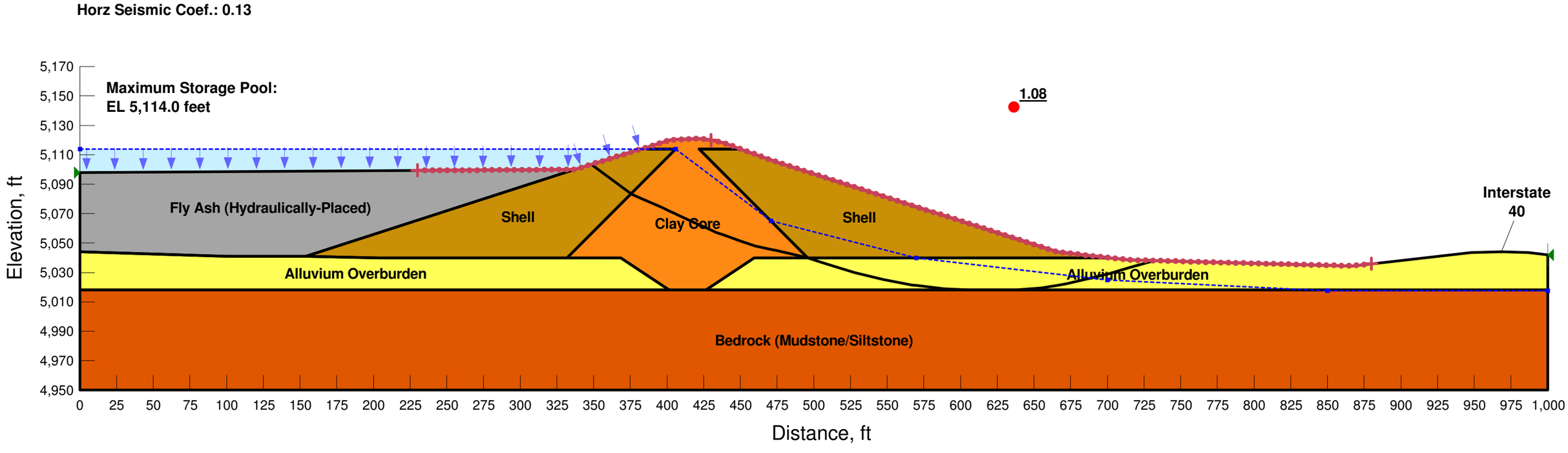
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
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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 °	--



Slope Stability Analysis
Cross Section 2
Fly Ash Pond

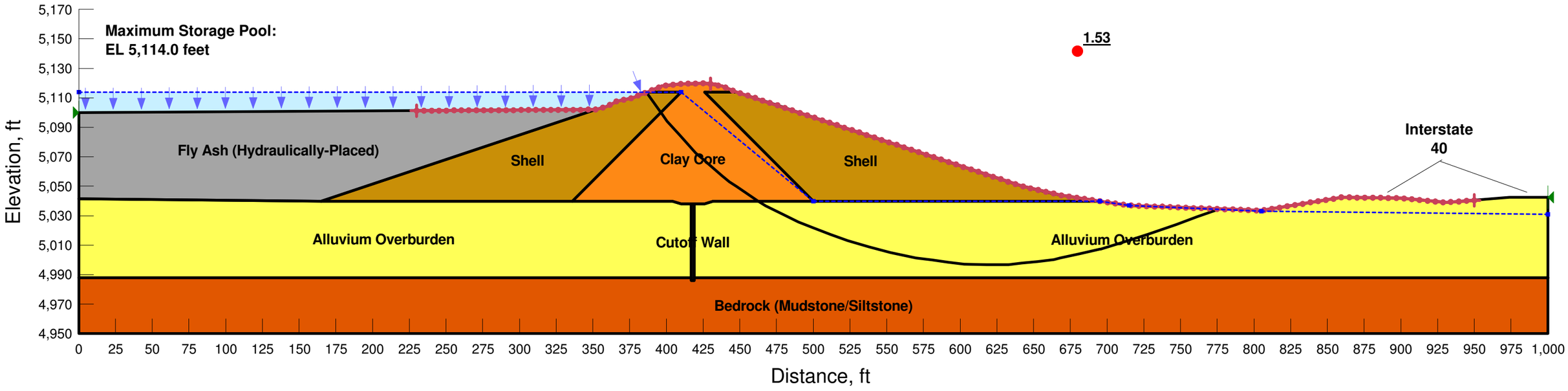
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Joseph City, Arizona
Arizona Public Service

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

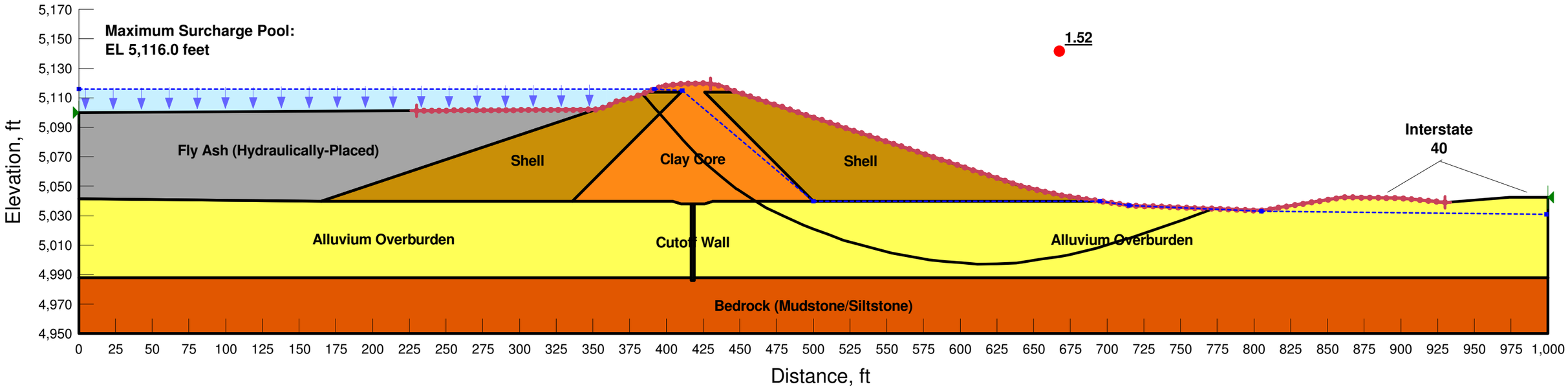
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Joseph City, Arizona
Arizona Public Service

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

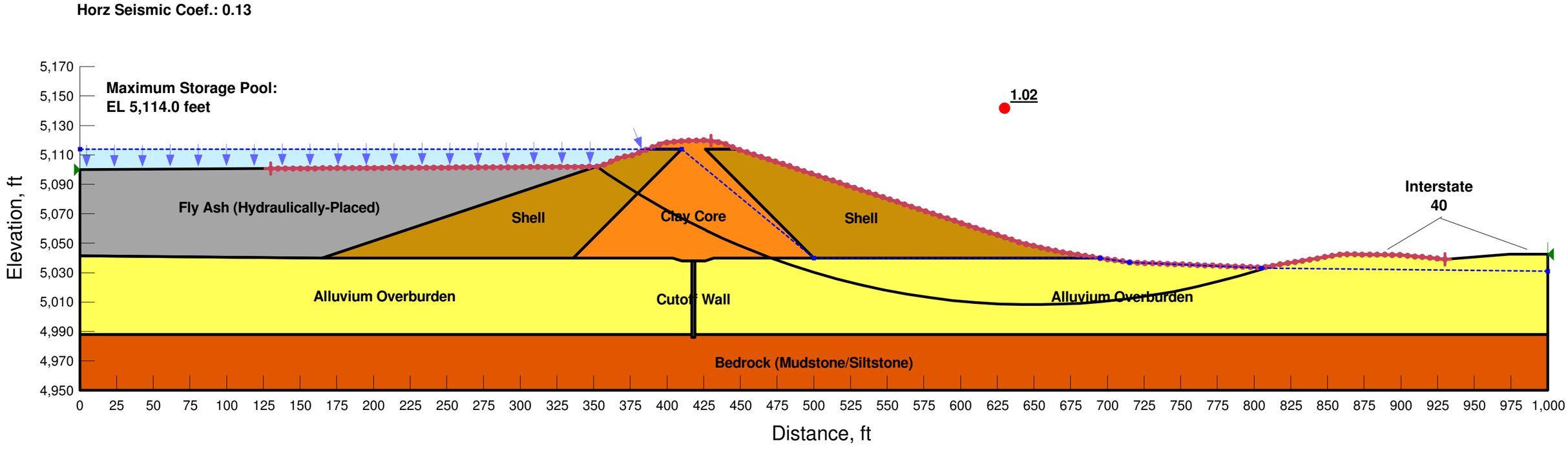
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

Note:
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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 °	--



Slope Stability Analysis
Cross Section 3
Fly Ash Pond

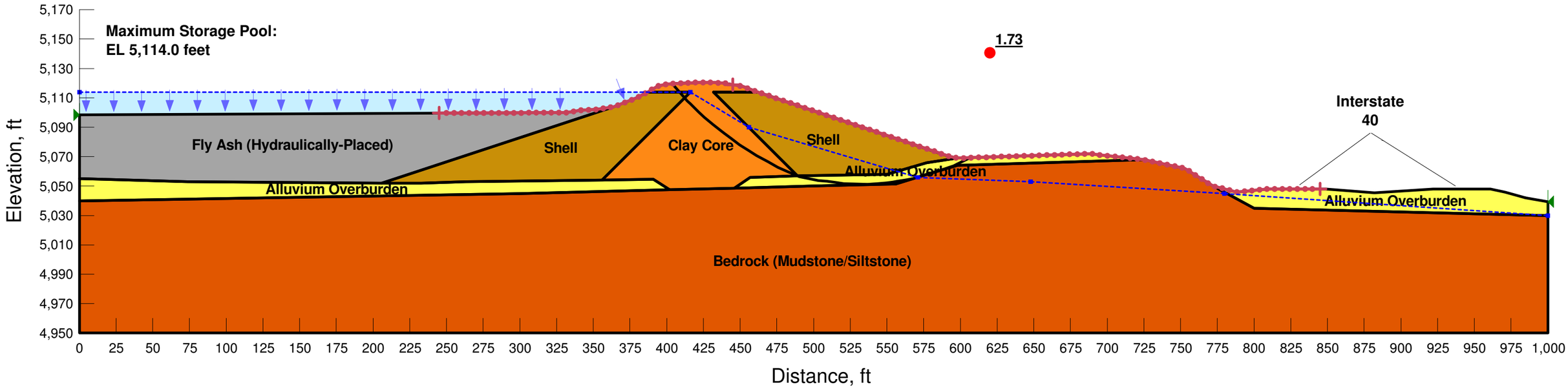
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Arizona Public Service

Note:
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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

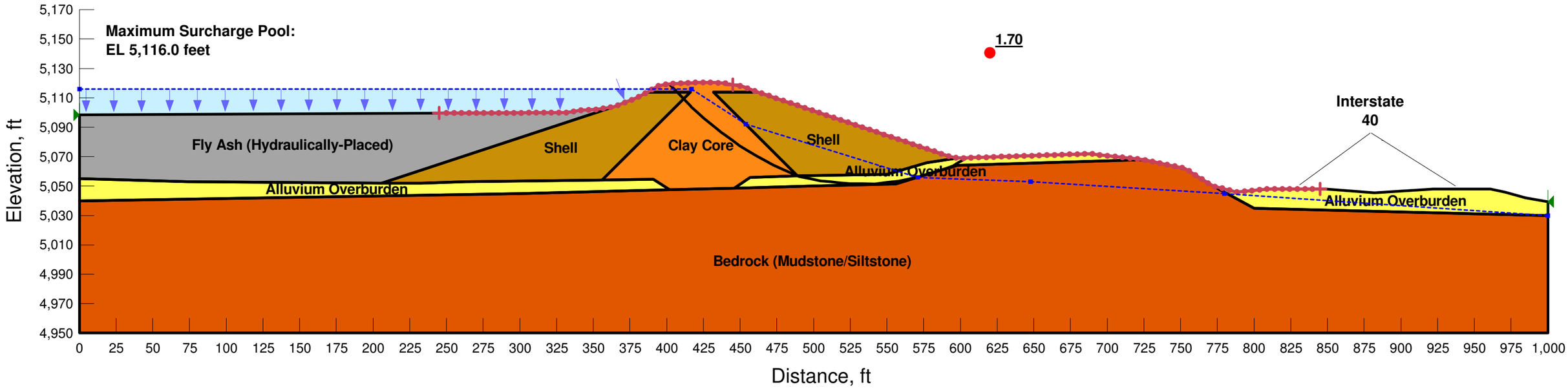
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Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

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

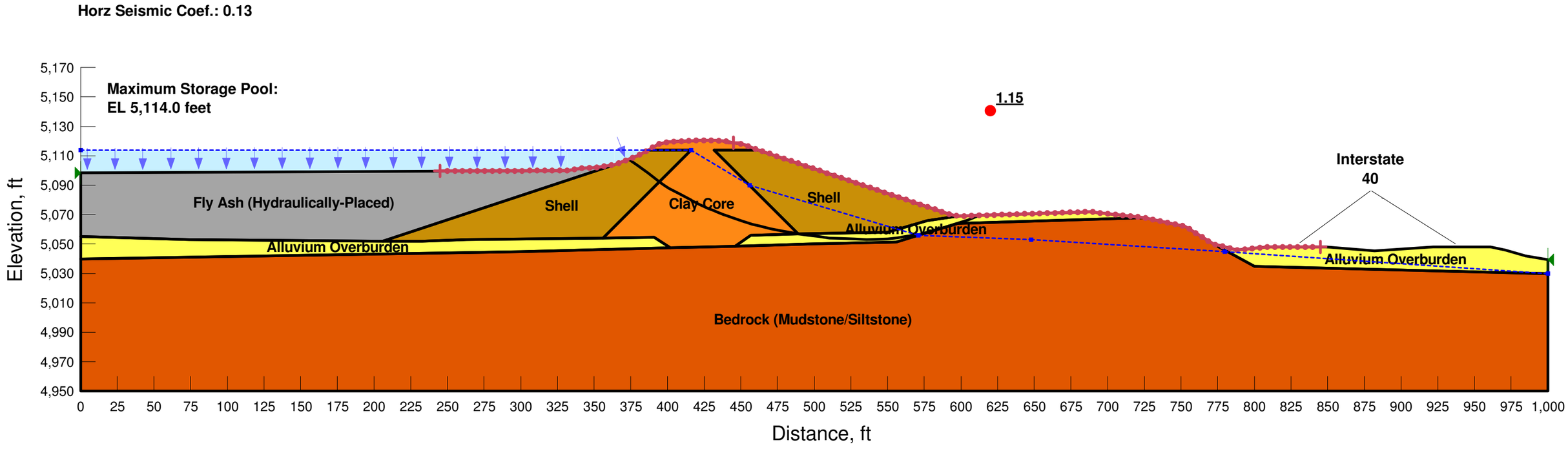
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Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

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 °	--



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