AECOM

Cholla Power Plant
Bottom 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: Bottom Ash Pond Dam Monitored Instrumentation and Seep Location Map

Attachment A: APS, 2025b. *Bottom Ash Pond Seepage Intercept System Extraction Well Pump Installation for the Cholla Generating Station*. As-Built. Engineer: WSP Environment & Infrastructure Inc. Relevant sheets.

Attachment B: AECOM, 2021. *Cholla Power Plant Bottom 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 Bottom 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 Bottom 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, 2023. *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, 2025a. *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 Bottom 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 consistent, remaining at 12 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, flowmeters and pumps to be replaced, 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 settlement monuments installed along the downstream shoulder of the crest indicate little to no net movement over the last five years. Settlement Monuments M-12 and M-13 indicate settlement of 0.78 inches and 1.12 inches, respectively, since 2021. These two settlement monuments are located above the thickest section of soil and the tallest section of the Bottom Ash Dam embankment. The horizontal movement at M-12 and M-13 was recorded as 0.32 inches and 0.43 inches, respectively, since 2021. The measured settlement and horizontal movement at the other settlement monuments has either been negligible since 2021 or within historic fluctuations.

The water levels recorded in the piezometers monitored at the Bottom Ash Dam generally followed the trends exhibited by the Bottom Ash Pond level throughout the 2021-2025 review timeframe.

Quarterly seepage volumes and rates recorded at the P-226 seep (a.k.a., TWX), the Petroglyph Seep, the Tanner Wash Seep, the West Abutment Weir, and the Toe Drain Seep were generally consistent between 2021 and 2024 when the seepage upgrades were undertaken (Section 4.3). New totalizers were installed at the P-226 seep (a.k.a., TWX), the Petroglyph Seep, the Tanner Wash Seep, and the Toe Drain Seep; and the West Abutment Weir monitoring point was removed in 2024.

The seepage reading schedules for the new totalizers (i.e., those installed in 2024) changed from quarterly to monthly in November 2025. In the limited dataset since the seepage piping was replaced, there has not been a significant increase in the seepage volume recorded at the totalizers. Only the volume recorded at the Toe Drain Seep has increased, which appears to be the result of rerouting the seepage volume from the West Abutment Seep to the Toe Drain and the increased collection capacity at the West Abutment (Section 4.3).

The 2021-2025 Annual Inspection Reports also provide information on the water levels in the Bottom Ash Pond, the approximate impounded volume in the Bottom 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 (APS 2022-2026).

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 adequately represent current conditions 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. The majority of the Bottom Ash Pond's description remains relevant; however, upgrades to the seepage collection and pumpback system in 2024, as well as the cessation of coal-fired power generation with removal of Units 1 and 3 from service in March 2025, require revisions to the associated portions of the Facility Description. 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 Bottom Ash Pond by June 30, 2025. 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 major surface impoundments located off-site; 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 Bottom Ash Pond Dam.

The Bottom Ash Pond consists of a reservoir located in the southern portion of the pond, directly upstream of the Bottom Ash Dam and two coal combustion waste storage cells, the West Cell and the East Cell, located in the northern portion of the reservoir. The Bottom Ash Pond received waste water from the Bottom Ash Transfer Sump that contained water and solids from the following sources: the bottom ash overflow sump; bottom ash slurry from Units 1 through 4; the Area 1, 2, and 3 Area Drainage Sumps; the Units 1, 2, 3, and 4 Bottom Ash Hoppers; General Water Sump Liquids and Solids; Sedimentation Pond effluent (prior to the Sedimentation Pond closing in 2021); the Units 1, 2, 3, and 4 Oil Water Separators; boiler cleaning waste; and water siphoned back from the Bottom Ash Pond seepage collection system. In addition, the following were discharged to the Bottom Ash Pond: scrubber sludge, Bottom Ash Pond stormwater, Units 3 and 4 Cooling Tower Basin solids, seepage and stormwater from the Bottom Ash Monofill retention basins, General Water Sump Solids, Sedimentation Pond solids (prior to the Sedimentation Pond closing in 2021), WARP solids, flue gas desulfurization wastes, and oil/water separator solids. The CCR wastes and other discharges were pumped to one of the two upstream waste storage cells, where the bottom ash was allowed to settle and the water was decanted to the reservoir for reuse at Cholla. At any given time, one of the waste storage

cells was receiving bottom ash while the other was being drained, excavated, and transported to the Bottom Ash Monofill north of the pond where the bottom ash was historically dry stacked. Excess water from the upstream cells was drained to the downstream reservoir via 12-inch pipes that outlet directly to the reservoir. The water was decanted from the reservoir for reuse at Cholla through a siphon system. As of 2026, the infrastructure used to fill the Bottom Ash Pond remained intact, though it has been inactive since the cessation of coal-fired power generation in March 2025.

The Bottom Ash Pond has a total surface area of about 80 acres and a total storage capacity including solids and water of about 2,300 acre-feet when at its ADWR permitted maximum storage pool water level of EL 5,117.8 feet (NGVD29; ADWR 1998). The impoundment is surrounded on its west and north sides by natural topography consisting of rock outcrops of mudstones, siltstones, and sandstones. On the south and east sides, the impoundment is enclosed by the Bottom Ash Pond Dam, Arizona Department of Water Resources (ADWR) Dam No. 09.27, which spans the width of natural drainage features. The Bottom Ash Dam is classified under ADWR regulations as a high hazard impoundment due to 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 Bottom 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 the Bottom Ash Pond started receiving CCR materials in 1978. In 1993, the dam crest was raised 3.3 feet when it became apparent that the storage volume of the pond was inadequate and the pond was filling faster than anticipated. In 1999, the impoundment was altered to its current configuration consisting of two upstream waste drainage cells and a downstream reservoir. The two waste cells were alternately dried and dredged (via removal to the Bottom Ash Monofill) to facilitate the removal of bottom ash from the sluice water which drains to the reservoir. By this procedure, the total storage volume in the pond remained relatively constant. The dam is approximately 4,200 feet in length with a maximum toe to crest height of 73 feet and crest width of 12 feet. The crest elevation is EL 5125.7 feet NAVD88 (EL 5123.3 feet NGVD29) after APS completed the 1993 crest raise providing 5.5 feet of 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) slope, except for the upper portion of the slopes constructed during the crest raise, where the slopes are inclined at a 1.5H:1V vertical slope. Both upstream and downstream slopes are lined with riprap facing to prevent erosion. The current surface of the reservoir (water below EL 5107 feet NGVD29 (EL 5109.4 feet NAVD88) and the typical elevation of the solids at approximately EL 5115 feet NGVD29 (EL 5117.4 feet NAVD88)) is at least 2.8 feet below the maximum operating water surface elevation and is expected to remain so since APS no longer discharges to the Bottom Ash Pond. The volume associated with the IDF would be fully contained below the maximum operating water surface elevation, primarily in the southern end of the reservoir.

To limit seepage beneath the foundation, the central clay core of the Bottom 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 to the bedrock or stiff clay. The Bottom 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 storm water and groundwater and return it to the pond. These include seepage collection systems for the West Abutment Seep, the Toe Drain Seep, the Petroglyph Seep, the P-226 Seep (a.k.a. the TWX Seepage Area), and the Tanner Wash Seep.

The Bottom Ash Pond has no fixed intake or outlet water work structures. Sluiced bottom ash was pumped from Cholla to the pond through a discharge line that runs up the right dam abutment, adjacent to the embankment and to a screening plant that removed a portion of the solids for commercial use by the Salt River Materials Group. The processed slurry was pumped to one of the two waste cells where the bottom ash settled, and the excess water was decanted to the reservoir. Water levels within the pond were controlled by varying the pumping rate from Cholla and seepage control system to balance the seepage, evaporation, and siphon system in the reservoir. The siphon system consists of three 12-inch diameter high density polyethylene (HDPE) pipes that float near the surface of the reservoir. The pipes were originally 8-inch in diameter but were replaced with the current 12-inch diameter pipes in the late 2000's. The siphon system pipes extend through the upper portion of the dam at a pipe invert elevation of about EL 5120.5 feet and continue down the downstream face of the dam to a common valve chamber. 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 contain all of 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 5,119.3 feet (NGVD29) based on an expected water level rise of 1.5 feet during the PMF (Dames & Moore, 1991).

Piezometers, settlement monuments, flow measurement devices, and water level gauges are installed at the Bottom Ash Pond Dam to monitor the performance of the embankment. Measurements from the monitoring instruments are reviewed by APS and documented annually in a data report. Starting on October 19, 2015, the piezometer, survey monuments, V-notch weirs, and sumps 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 of the Bottom Ash Pond are performed by a qualified person at intervals not exceeding seven days. The inspections examine the Bottom Ash Pond Dam for actual or potential conditions that could disrupt the operation or safety of the impoundment, and the results of the inspection are placed 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; observations, maintenance, and operation activities conducted by Plant personnel; a field inspection of the dam, slopes, abutments, and downstream toe; and documentation of findings and recommendations in a dam safety inspection report.

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 Bottom 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 incorporated from the 2016 Certification Report adequately represents the conditions up to October 2021 and is incorporated into this 2026 Certification Report. Descriptions of significant additional construction since 2021 are provided in the following paragraphs:

Throughout most of 2024 (prior to the cessation of coal-fired power generation in 2025), APS initiated construction of several environmental upgrades, improvements, and additions to the existing system of wells and intercept trenches as part of interim Corrective Measures to divert seepage water from the Bottom Ash Pond to the recently constructed Evaporation Ponds for disposal in preparation for closing the Bottom Ash Pond. APS also removed the West Abutment Weir from service, installed a perforated collection pipe, and routed the seepage to the Bottom Ash Toe Drain sump. 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 2025b).

Water from the seepage collection locations is now directed into a new 8,400-gallon Equalization Tank, which drains to the new Evaporation Ponds via gravity, instead of returning seepage water back to the Bottom Ash Pond.

APS has historically excavated bottom ash from the Bottom Ash Pond at various times throughout the year, placing it in the Bottom Ash Monofill. Due to a variety of factors associated with closure planning, APS did not transfer bottom ash out of the Bottom Ash Pond during the 2021-2025 timeframe.

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 satisfies the requirements of the CCR Rule for existing CCR units.

AECOM assesses that the design, construction, operation, and maintenance of the Bottom Ash Pond Dam 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 safety factor assessment completed for the original 2016 Certification Report, incorporated into the 2021 Certification Report, featured two critical cross-sections – one near the right abutment of the dam that intersects a rock outcrop and one near the center of the South Embankment. 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 (ICC 2004). In the 2016 Safety Factor assessment, the Bottom 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 Bottom Ash Dam, the original design drawings (APS Drawing G-44556) indicate as much as 91 feet of alluvium (silty clay, sandy clay, silty sand, clayey sand, and gravelly sand) near the maximum section of the dam (along the South Embankment) overlying rock. The alluvial channel is narrow and the foundation along the East Embankment is almost entirely rock in the upper 100 feet. Conservatively assuming that the average shear wave velocity in the upper 100 feet of the Bottom Ash Dam footprint can be classified as "DE – Loose sand or medium stiff clay" under the ASCE 7-22 Site Classification, the PGA_M for the Bottom Ash Dam is 0.13g. This value is the same as the 0.13g value used in the 2016 Certification Report for PGA_M. In the 2016 slope stability analysis, the seismic coefficient was modified to account for variations between the peak transverse base acceleration and the peak transverse crest acceleration for a tall embankment; however, the result was that for a deep-seated failure surface, the horizontal seismic coefficient remained 0.13g. A similar process would not change the horizontal seismic coefficient from the ASCE 7-22 classification.

The calculated safety factors for the Bottom 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

None identified and none recommended.

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 – Bottom Ash Pond, Cholla Power Plant – Joseph City, Arizona*. Prepared for: Arizona Public Service. AECOM Job No. 60445840. August 2016.

AECOM, 2021. *Cholla Power Plant Bottom 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, 2023. *Cholla Power Plant – Fly Ash Dam, Bottom Ash Dam, and Bottom Ash Monofill – Annual CCR Impoundment and Landfill Inspection Report – 2022*. January 2023.

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

APS, 2025b. *Bottom Ash Pond Seepage Intercept System Extraction Well Pump Installation for the Cholla Generating Station*. As-Built. Engineer: WSP Environment & Infrastructure Inc. Relevant sheets.

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.

American Society of Civil Engineers (ASCE), 2022. *ASCE/SEI 7-22: Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Errata dated September 20, 2024.

American Society of Civil Engineers (ASCE), 2022. ASCE 7 Hazard Tool. Accessed September 16, 2026. https://ascehazardtool.org/.

Ebasco Services Inc. (Ebasco), 1975, *Arizona Public Service Company Cholla Generating Station Ash Disposal Sites Seepage and Foundation Studies: Volume II of II Field and Laboratory Tests*. August.

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

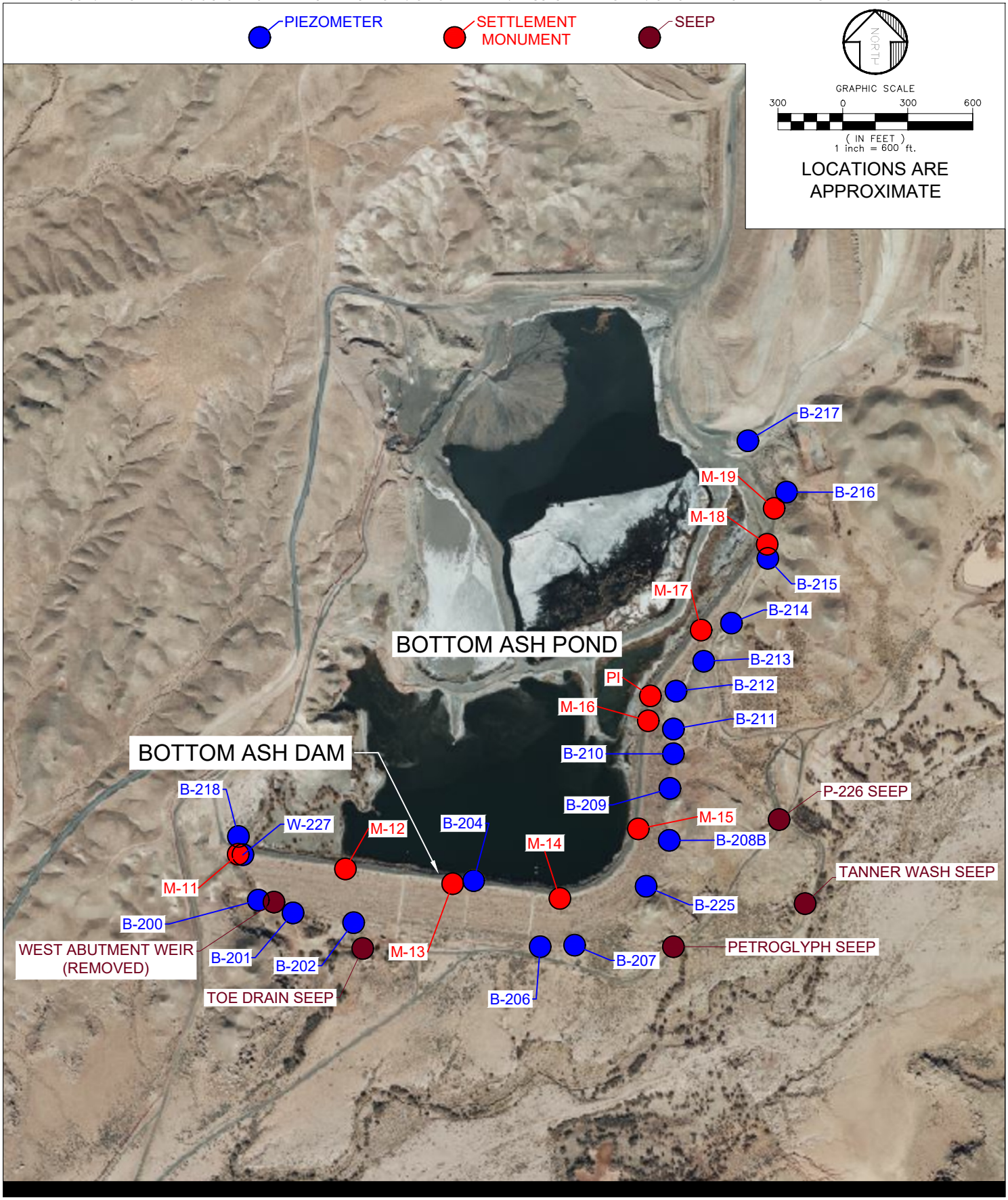
Figure 1-2:
Bottom Ash Pond Dam Monitored Instrumentation and Seep Location Map

Attachment A:
APS, 2025b. *Bottom Ash Pond Seepage Intercept System Extraction Well Pump Installation for the Cholla Generating Station*. As-Built. Engineer: WSP Environment & Infrastructure Inc. Relevant sheets.

Attachment B:
AECOM, 2021. *Cholla Power Plant Bottom 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
Bottom Ash Pond Dam Monitored Instrumentation and Seep Location Map

Last saved by: LEE.WRIGHT(2026-10-09) Last Plotted: 2026-10-09
Filename: AECOM\APS - GENERAL\2026 CHOLLA 5-YEAR RECERTIFICATIONS\BOTTOM ASH DAM\STRUCTURAL INTEGRITY\BOTTOM ASH POND DAM INSTRUMENTS.DWG
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CHOLLA POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE

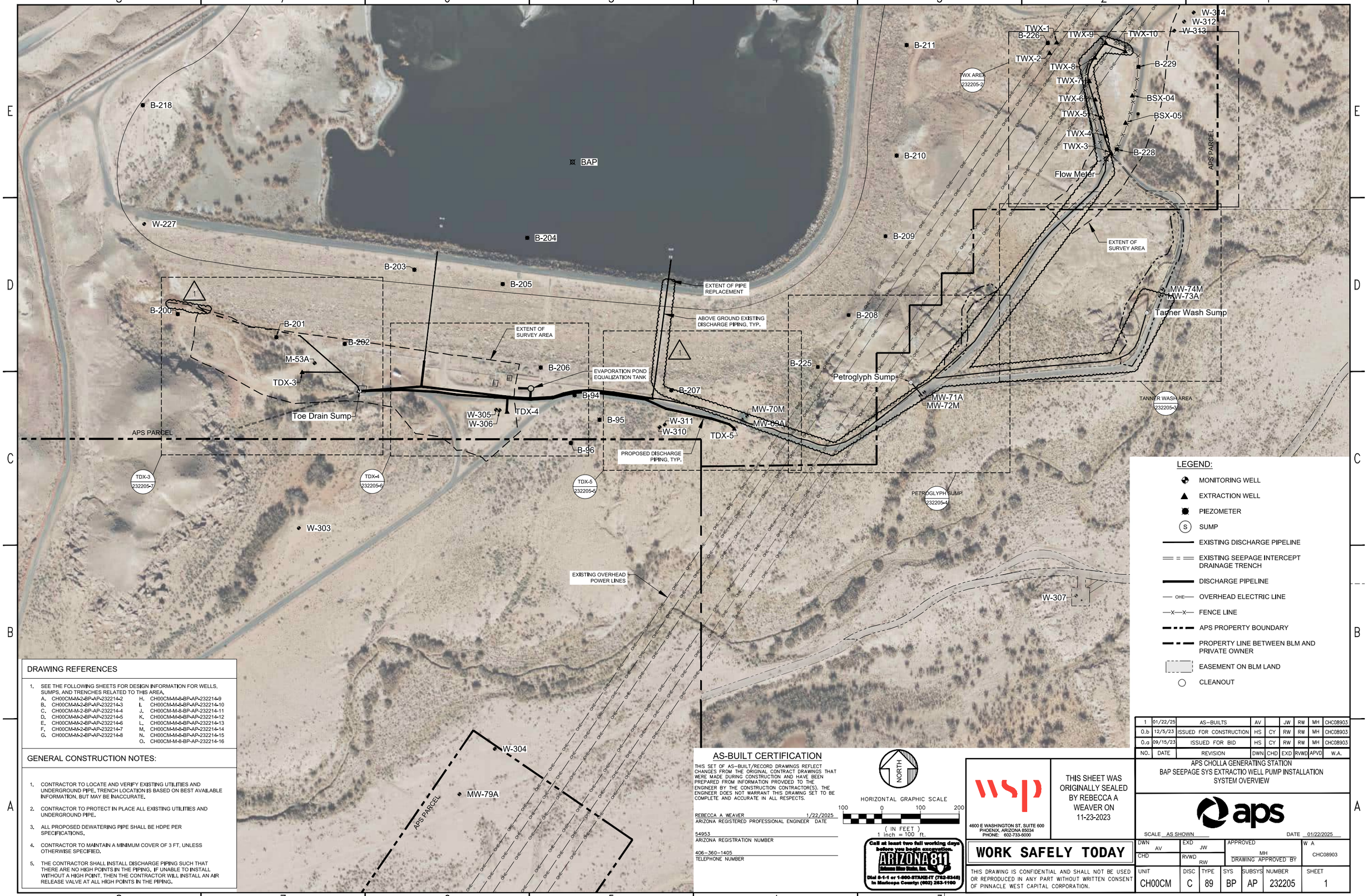
BOTTOM ASH POND DAM
MONITORED INSTRUMENTATION
AND SEEP LOCATION MAP

AECOM

FIGURE 1-2

ATTACHMENT A

APS, 2025b. *Bottom Ash Pond Seepage Intercept System Extraction Well Pump Installation for the Cholla Generating Station*. As-Built. Engineer: WSP Environment & Infrastructure Inc. Relevant sheets.



DRAWING REFERENCES

1. SEE THE FOLLOWING SHEETS FOR DESIGN INFORMATION FOR WELLS, SUMPS, AND TRENCHES RELATED TO THIS AREA.
   - A. CH00CM-M-2-BP-AP-232214-2
   - B. CH00CM-M-2-BP-AP-232214-3
   - C. CH00CM-M-2-BP-AP-232214-4
   - D. CH00CM-M-2-BP-AP-232214-5
   - E. CH00CM-M-2-BP-AP-232214-6
   - F. CH00CM-M-2-BP-AP-232214-7
   - G. CH00CM-M-2-BP-AP-232214-8
   - H. CH00CM-M-8-BP-AP-232214-9
   - I. CH00CM-M-8-BP-AP-232214-10
   - J. CH00CM-M-8-BP-AP-232214-11
   - K. CH00CM-M-8-BP-AP-232214-12
   - L. CH00CM-M-8-BP-AP-232214-13
   - M. CH00CM-M-8-BP-AP-232214-14
   - N. CH00CM-M-8-BP-AP-232214-15
   - O. CH00CM-M-8-BP-AP-232214-16

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 DEWATERING PIPE SHALL BE HDPE PER SPECIFICATIONS.
4. CONTRACTOR TO MAINTAIN A MINIMUM COVER OF 3 FT, UNLESS OTHERWISE SPECIFIED.
5. THE CONTRACTOR SHALL INSTALL DISCHARGE PIPING SUCH THAT THERE ARE NO HIGH POINTS IN THE PIPING. IF UNABLE TO INSTALL WITHOUT A HIGH POINT, THEN THE CONTRACTOR WILL INSTALL AN AIR RELEASE VALVE AT ALL HIGH POINTS IN THE PIPING.

LEGEND:

- MONITORING WELL
- EXTRACTION WELL
- PIEZOMETER
- (S) SUMP
- EXISTING DISCHARGE PIPELINE
- EXISTING SEEPAGE INTERCEPT DRAINAGE TRENCH
- DISCHARGE PIPELINE
- OHE OVERHEAD ELECTRIC LINE
- FENCE LINE
- APS PROPERTY BOUNDARY
- PROPERTY LINE BETWEEN BLM AND PRIVATE OWNER
- EASEMENT ON BLM LAND
- CLEANOUT

AS-BUILT CERTIFICATION

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REBECCA A WEAVER 1/22/2025
ARIZONA REGISTERED PROFESSIONAL ENGINEER DATE

54953
ARIZONA REGISTRATION NUMBER

406-360-1405
TELEPHONE NUMBER

NORTH

HORIZONTAL GRAPHIC SCALE
100 0 100 200
(IN FEET)
1 inch = 100 ft.

Call at least two full working days before you begin excavation.
ARIZONA 811
Arizona Blue Stake, Inc.
Dial 8-1-1 or 1-800-STAKE-IT (782-5348)
In Maricopa County (602) 263-1100

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
1	01/22/25	AS-BUILTS	AV		JW	RW	MH	CHC08903
0.b	12/5/23	ISSUED FOR CONSTRUCTION	HS	CY	RW	RW	MH	CHC08903
0.a	09/15/23	ISSUED FOR BID	HS	CY	RW	RW	MH	CHC08903

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

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WORK SAFELY TODAY

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APS CHOLLA GENERATING STATION
BAP SEEPAGE SYS EXTRACTIO WELL PUMP INSTALLATION
SYSTEM OVERVIEW

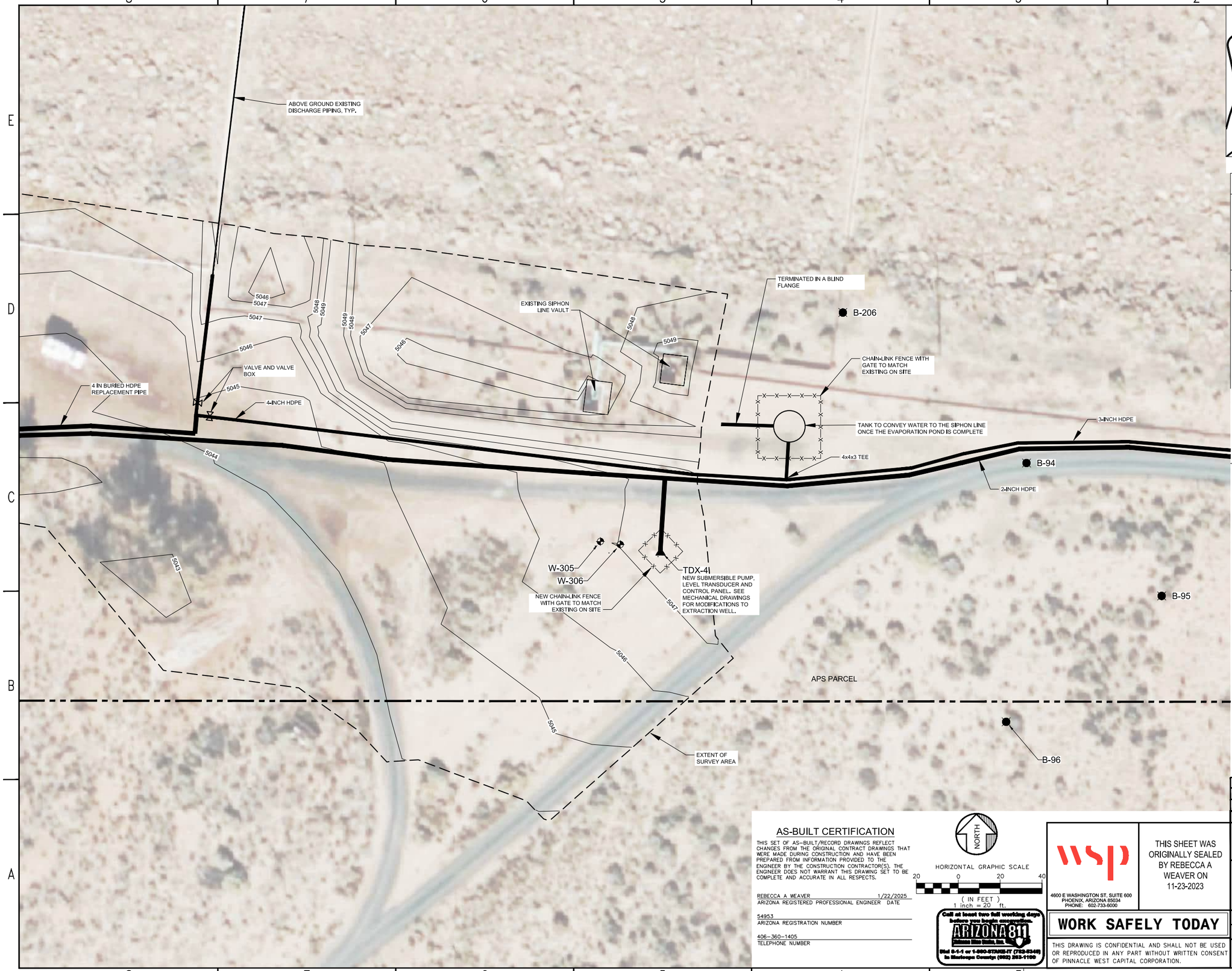
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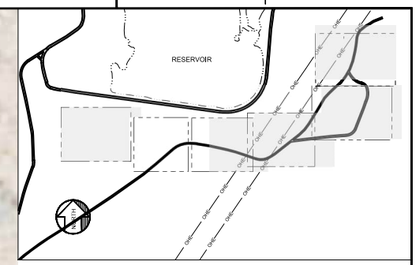
SCALE AS SHOWN DATE 01/22/2025

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AV	JW	MH	CHC08903
CHD	RWD	DRAWING APPROVED BY	
	RW		

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	C	89	BP	AP	232205	1

T:\APS Cholla temp\CIVIL\7.0 Civil\4 - system overview.dwg - Overview 01/22/2025 1:32pm USAV721062





KEY MAP

DRAWING REFERENCES

1. SEE THE FOLLOWING SHEETS FOR DESIGN INFORMATION FOR WELLS, SUMPS, AND TRENCHES RELATED TO THIS AREA.
   - A. CH00CM-M-2-BP-AP-232214-2
   - B. CH00CM-M-2-BP-AP-232214-3
   - C. CH00CM-M-2-BP-AP-232214-4
   - D. CH00CM-M-2-BP-AP-232214-5
   - E. CH00CM-M-2-BP-AP-232214-6
   - F. CH00CM-M-2-BP-AP-232214-7
   - G. CH00CM-M-2-BP-AP-232214-8
   - H. CH00CM-M-8-BP-AP-232214-9
   - I. CH00CM-M-8-BP-AP-232214-10
   - J. CH00CM-M-8-BP-AP-232214-11
   - K. CH00CM-M-8-BP-AP-232214-12
   - L. CH00CM-M-8-BP-AP-232214-13
   - M. CH00CM-M-8-BP-AP-232214-14
   - N. CH00CM-M-8-BP-AP-232214-15
   - O. CH00CM-M-8-BP-AP-232214-16

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 DEWATERING PIPE SHALL BE HDPE PER SPECIFICATIONS.
4. CONTRACTOR TO MAINTAIN A MINIMUM COVER OF 3 FT., UNLESS OTHERWISE SPECIFIED.
5. SEE MECHANICAL DRAWINGS FOR DETAILS.
6. THE CONTRACTOR SHALL INSTALL DISCHARGE PIPING SUCH THAT THERE ARE NO HIGH POINTS IN THE PIPING. IF UNABLE TO INSTALL WITHOUT A HIGH POINT, THEN THE CONTRACTOR WILL INSTALL AN AIR RELEASE VALVE AT ALL HIGH POINTS IN THE PIPING.
7. CONTRACTOR TO SPREAD 1 INCH MINUS CRUSHED ROCK INSIDE FENCED AREAS.

LEGEND:

- MONITORING WELL
- EXTRACTION WELL
- PIEZOMETER
- (S) SUMP
- EXISTING DISCHARGE PIPELINE
- EXISTING SEEPAGE INTERCEPT DRAINAGE TRENCH
- DISCHARGE PIPELINE
- 5050 EXISTING GROUND SURFACE ELEVATION CONTOUR (1-FT INVERVAL) (NAVD-88)
- OHE OVERHEAD ELECTRIC LINE
- X—X— FENCE LINE
- APS PROPERTY BOUNDARY
- EASEMENT ON BLM LAND

AS-BUILT CERTIFICATION

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REBECCA A WEAVER 1/22/2025
ARIZONA REGISTERED PROFESSIONAL ENGINEER DATE

54953
ARIZONA REGISTRATION NUMBER

406-360-1405
TELEPHONE NUMBER

NORTH

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(IN FEET)
1 inch = 20 ft.

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0.a	09/15/23	ISSUED FOR BID	HS	CY	RW	RW	MH	CHC08903

APS CHOLLA GENERATING STATION
BAP SEEPAGE SYS EXTRACTION WELL PUMP INSTALLATION
TDX-4 SITE PLAN

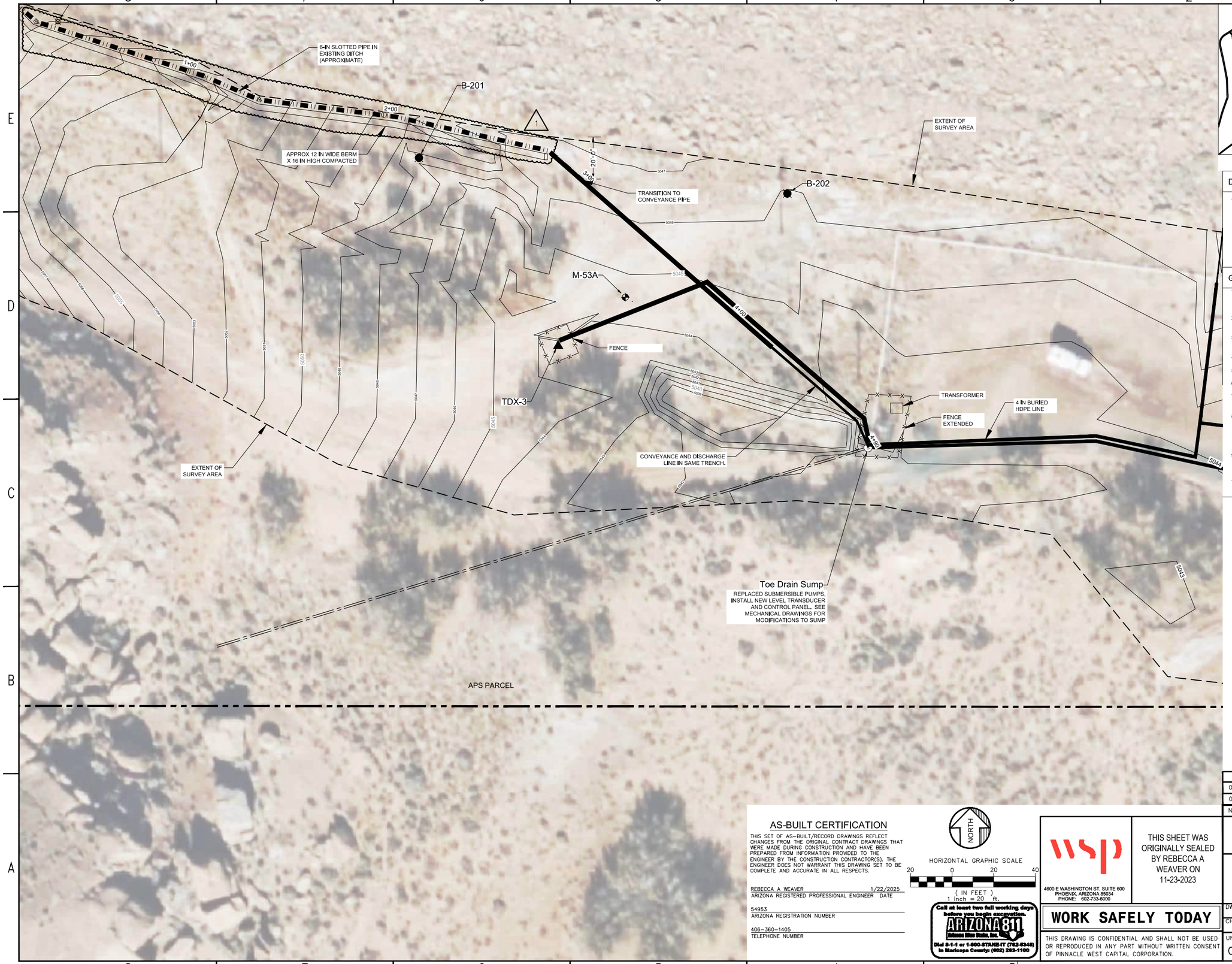
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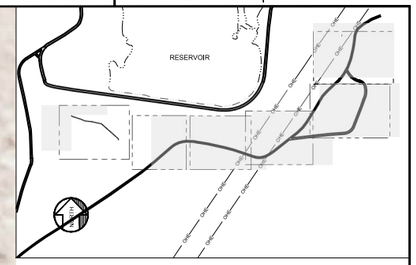
SCALE AS SHOWN DATE 01/22/2025

DWN	EXD	APPROVED	W A
AV	JW	MH	CHC08903
CHD	RVWD	DRAWING APPROVED BY	
	RW		

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	C	16	BP	AP	232205	6

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DRAWING REFERENCES

1. SEE THE FOLLOWING SHEETS FOR DESIGN INFORMATION FOR WELLS, SUMPS, AND TRENCHES RELATED TO THIS AREA.
   - A. CH00CM-M-2-BP-AP-232214-2
   - B. CH00CM-M-2-BP-AP-232214-3
   - C. CH00CM-M-2-BP-AP-232214-4
   - D. CH00CM-M-2-BP-AP-232214-5
   - E. CH00CM-M-2-BP-AP-232214-6
   - F. CH00CM-M-2-BP-AP-232214-7
   - G. CH00CM-M-2-BP-AP-232214-8
   - H. CH00CM-M-8-BP-AP-232214-9
   - I. CH00CM-M-8-BP-AP-232214-10
   - J. CH00CM-M-8-BP-AP-232214-11
   - K. CH00CM-M-8-BP-AP-232214-12
   - L. CH00CM-M-8-BP-AP-232214-13
   - M. CH00CM-M-8-BP-AP-232214-14
   - N. CH00CM-M-8-BP-AP-232214-15
   - O. CH00CM-M-8-BP-AP-232214-16

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2. CONTRACTOR TO PROTECT IN PLACE ALL EXISTING UTILITIES AND UNDERGROUND PIPE.
3. ALL PROPOSED DEWATERING PIPE SHALL BE HDPE PER SPECIFICATIONS.
4. CONTRACTOR TO MAINTAIN A MINIMUM COVER OF 3'.
5. DISCHARGE LINES ARE ABOVE-GRADE.
6. SEE MECHANICAL DRAWINGS FOR DETAILS.
7. CONTRACTOR TO SPREAD 1 INCH MINUS CRUSHED ROCK INSIDE FENCED AREAS.
8. MATERIAL PIPING AND CONNECTIONS SHALL BE THOSE USED IN SEPTIC SYSTEMS; STANDARD BEDDING SAND; 1" MINUS CRUSHED ROCK WITH LESS THAN 10% FINES; STANDARD TRACE WIRE; AND 8 OZ/SY NONWOVEN GEOTEXTILE. PROVIDE SUBMITTALS FOR ALL MATERIAL IN ADVANCE.
9. USE CLAYEY PORTIONS OF SITE SOIL TO INSTALL A 5-FEET LONG PLUG IN THE TRENCH AT STA. 3+00 WHERE THE PIPING TRANSITIONS FROM SLOTTED TO SOLID.
10. USE A MINIMUM OF 6 INCHES OF CRUSHED ROCK ABOVE THE SEPARATION GEOTEXTILE AT ROAD CROSSING.

LEGEND:

- MONITORING WELL
- EXTRACTION WELL
- PIEZOMETER
- (S) SUMP
- EXISTING DISCHARGE PIPELINE
- EXISTING SEEPAGE INTERCEPT DRAINAGE TRENCH
- DISCHARGE PIPELINE
- 5050 EXISTING GROUND SURFACE ELEVATION CONTOUR (1-FT INVERVAL) (NAVD-88)
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- FENCE LINE
- APS PROPERTY BOUNDARY
- EASEMENT ON BLM LAND

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1	01/22/25	AS-BUILTS	AV		JW	RW	MH	CHC08903
0.b	12/5/23	ISSUED FOR CONSTRUCTION	HS	CY	RW	RW	MH	CHC08903
0.a	09/15/23	ISSUED FOR BID	HS	CY	RW	RW	MH	CHC08903

AS-BUILT CERTIFICATION

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54953
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406-360-1405
TELEPHONE NUMBER

NORTH

HORIZONTAL GRAPHIC SCALE
20 0 20 40
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wsp
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PHOENIX, ARIZONA 85034
PHONE: 602-733-6000

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APS CHOLLA GENERATING STATION
BAP SEEPAGE SYS EXTRACTION WELL PUMP INSTALLATION
TOE DRAIN / TDX-03 SITE PLAN

aps

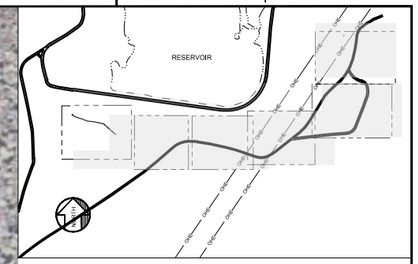
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AV	JW	MH	CHC08903
CHD	RVWD	DRAWING APPROVED BY	
	RW		

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	C	16	BP	AP	232205	7

T:\APS Cholla temp\CIVIL\7.0 Civil\10 - toedrain_BSX-1.dwg - ToeDrain_BSX-1 01/22/2025 1:43pm USAV721062





KEY MAP

DRAWING REFERENCES

1. SEE THE FOLLOWING SHEETS FOR DESIGN INFORMATION FOR WELLS, SUMPS, AND TRENCHES RELATED TO THIS AREA.
   - A. CH00CM-M-2-BP-AP-232218-1
   - B. CH00CM-M-2-BP-AP-232219-1
   - C. CH00CM-M-2-BP-AP-232220-1
   - D. CH00CM-M-2-BP-AP-232221-1
   - E. CH00CM-M-2-BP-AP-232222-1
   - F. CH00CM-M-2-BP-AP-232223-1
   - G. CH00CM-M-2-BP-AP-232224-1
   - H. CH00CM-M-8-BP-AP-232225-1
   - I. CH00CM-M-8-BP-AP-232226-1
   - J. CH00CM-M-8-BP-AP-232227-1
   - K. CH00CM-M-8-BP-AP-232228-1
   - L. CH00CM-M-8-BP-AP-232229-1
   - M. CH00CM-M-8-BP-AP-232230-1
   - N. CH00CM-M-8-BP-AP-232231-1
   - O. CH00CM-M-8-BP-AP-232232-1

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2. CONTRACTOR TO PROTECT IN PLACE ALL EXISTING UTILITIES AND UNDERGROUND PIPE.
3. ALL PROPOSED DEWATERING PIPE SHALL BE HDPE PER SPECIFICATIONS.
4. CONTRACTOR TO MAINTAIN A MINIMUM COVER OF 3'.
5. DISCHARGE LINES ARE ABOVE-GRADE.
6. SEE MECHANICAL DRAWINGS FOR DETAILS.
7. CONTRACTOR TO SPREAD 1 INCH MINUS CRUSHED ROCK INSIDE FENCED AREAS.

LEGEND:

- MONITORING WELL
- EXTRACTION WELL
- PIEZOMETER
- (S) SUMP
- TEST PIT
- EXISTING DISCHARGE PIPELINE
- EXISTING SEEPAGE INTERCEPT DRAINAGE TRENCH
- EXISTING EASEMENT UNDER AZA-018679
- DISCHARGE PIPELINE
- SEEPAGE INTERCEPT DRAINAGE TRENCH
- SLOTTED SEEPAGE INTERCEPT PIPELINE
- 5050 EXISTING GROUND SURFACE ELEVATION CONTOUR (1-FT INVERVAL) (NAVD-88)
- OHE OVERHEAD ELECTRIC LINE
- X—X FENCE LINE
- LEASE BOUNDARY
- CLEANOUT

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
1	01/22/25	AS-BUILTS	AV		JW	RW	MH	CHC08903
0.b	12/5/23	ISSUED FOR CONSTRUCTION	HS	CY	RW	RW	MH	CHC08903
0.a	09/15/23	ISSUED FOR BID	HS	CY	RW	RW	MH	CHC08903

AS-BUILT CERTIFICATION

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REBECCA A WEAVER 1/22/2025
ARIZONA REGISTERED PROFESSIONAL ENGINEER DATE

54953
ARIZONA REGISTRATION NUMBER

406-360-1405
TELEPHONE NUMBER

NORTH

HORIZONTAL GRAPHIC SCALE
20 0 20 40
(IN FEET)
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CHOLLA BAP DESIGN

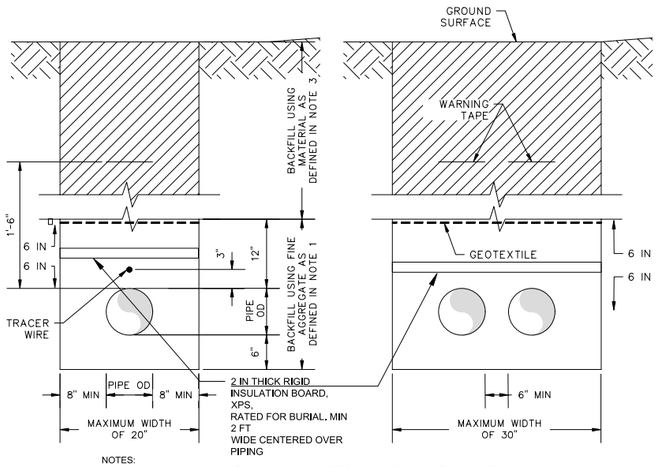
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aps

SCALE AS SHOWN DATE 01/22/2025

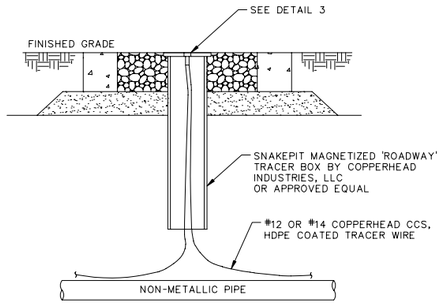
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AV	JW	MH	CHC08903
CHD	RVWD	DRAWING APPROVED BY	
	RW		

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	C	16	BP	AP	232205	8



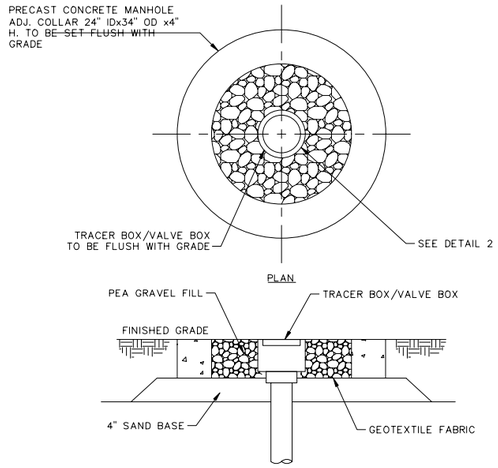
NOTES:
1. CONTRACTOR TO USE FINE AGGREGATE AS INITIAL BACKFILL AS DEFINED AS BEING ABLE TO PASS A NO. 4 US STANDARD SIEVE BUT NOT A NO. 200 US STANDARD SIEVE AND MUST NOT CONTAIN ANY DELETERIOUS MATERIALS.
2. CONTRACTOR TO BACKFILL AND COMPACT TO 95% IN 8 INCH LIFTS. SEE TECHNICAL NOTES.
3. CONTRACTOR TO USE CLSM, ABC, AND/OR GRANULAR OR SCREENED NATIVE BACKFILL.
4. INSTALL WARNING TAPE ENTIRE LENGTH OF BURIED PIPING. SEE TECHNICAL SPECIFICATIONS.
5. REPAIR ROAD SURFACE AT ALL ROAD CROSSINGS WITH LIKE MATERIAL, AT A MINIMUM OF 24INCHES THICKNESS.

DETAIL 1/- NTS — TRENCH DETAIL

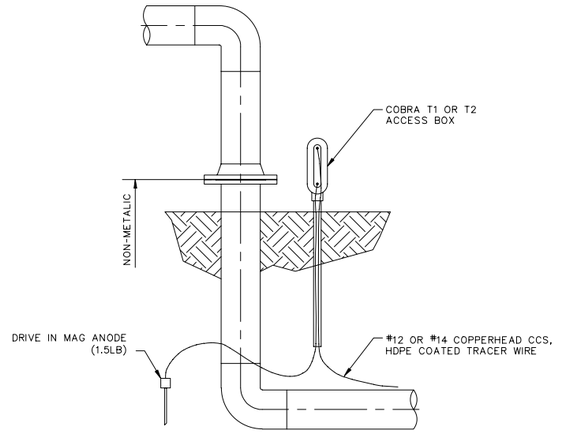


NOTES:
1. TRACER BOXES SHALL BE LOCATED AT EACH ELBOW.
2. MAXIMUM SPACING OF TRACER BOXES SHALL BE 1000'.

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

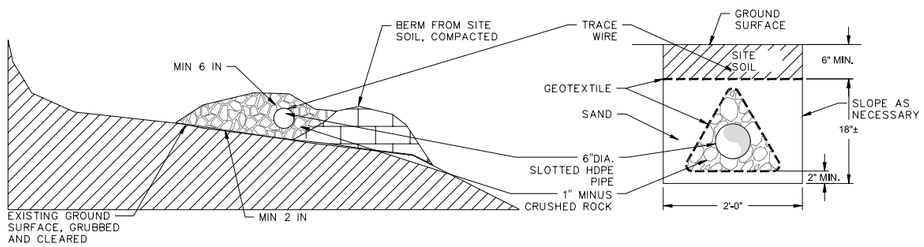


DETAIL 3/- NTS — TYPICAL TRACER BOX, PROTECTION COLLAR

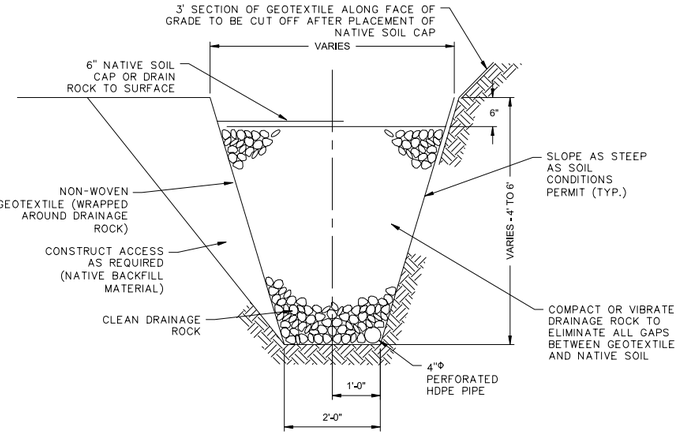


NOTES:
1. TRACER BOXES SHALL BE LOCATED AT EACH ELBOW.
2. MAXIMUM SPACING OF TRACER BOXES SHALL BE 1000'.
3. COBRA T1 USED WITH NO GROUNDING ANODE.
4. COBRA T2 USED WITH GROUNDING ANODE.

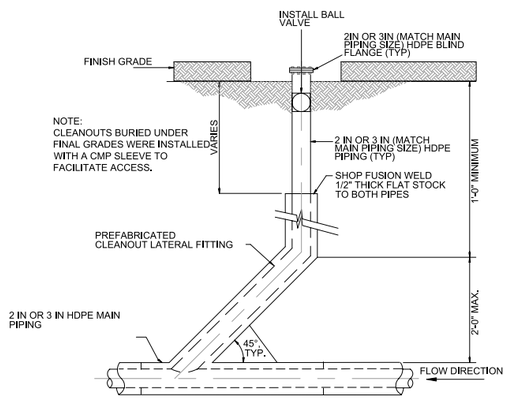
DETAIL 4/- NTS — TYPICAL TRACER BOX, PIPES THAT COME ABOVE GROUND



DETAIL 5/- NTS — COLLECTION TRENCH SECTION

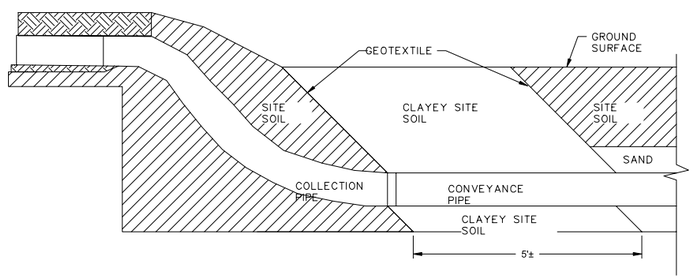


DETAIL 7/- NTS — TYPICAL INTERCEPT TRENCH



FORCE MAIN PIPE CLEANOUT (TYP.)

DETAIL 8/- NTS — M-101



DETAIL 6/- NTS — COLLECTION TO CONVEYANCE PIPE TRANSITION DETAIL

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0.a	09/15/23	ISSUED FOR BID	HS	CY	RW	RW	MH	CHC08903

wsp
4600 E WASHINGTON ST, SUITE 600
PHOENIX, ARIZONA 85034
PHONE: 602-733-8000

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APS CHOLLA GENERATING STATION
BAP SEEPAFE SYS EXTRACTION WELL PUMP INSTALLATION
CIVIL DETAILS

aps

SCALE AS SHOWN DATE 01/22/2025

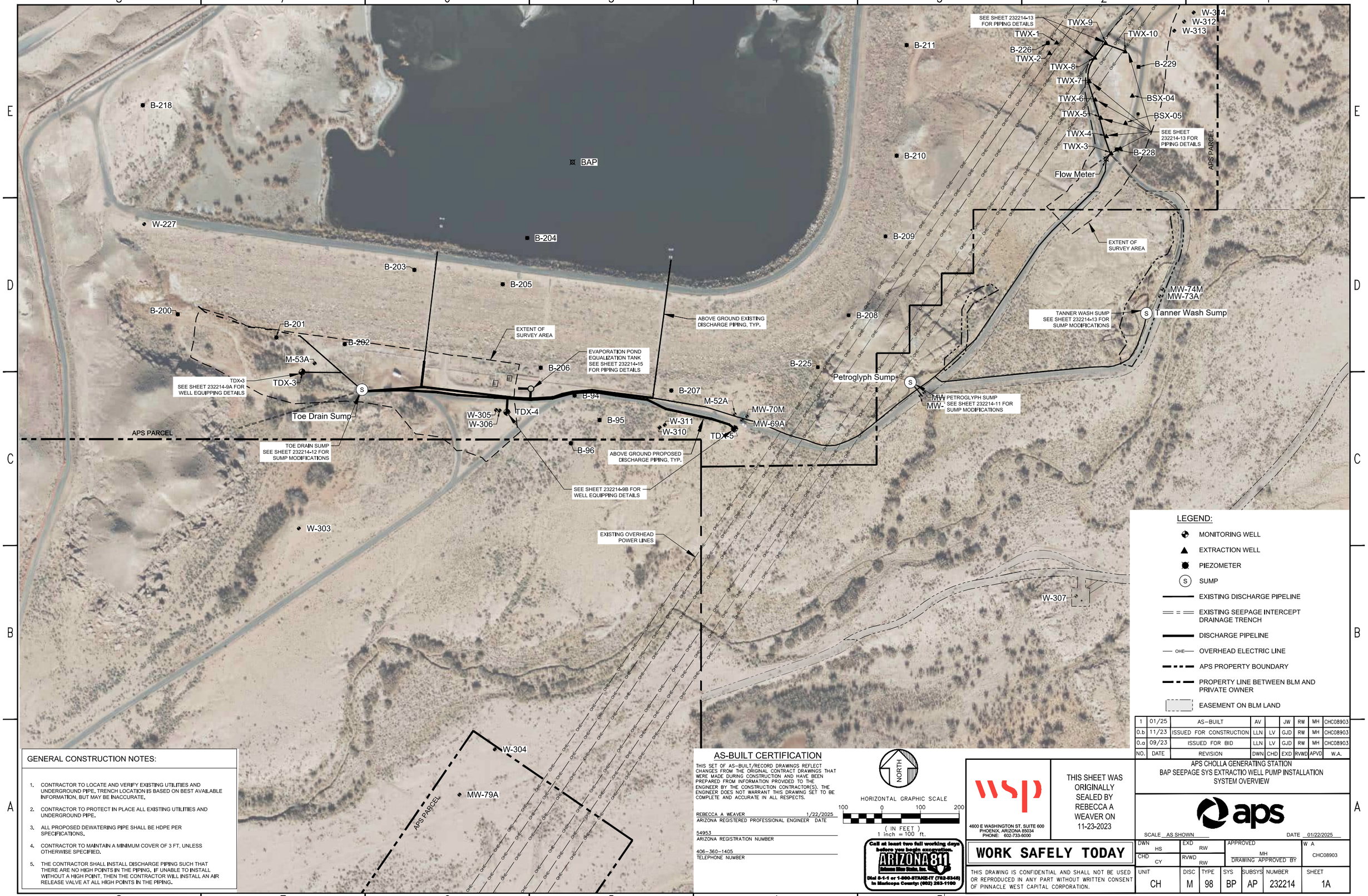
WORK SAFELY TODAY

DWN	EXD	APPROVED	W A
AV	JW	MH	CHC08903
CHD	RVWD	DRAWING APPROVED BY	
	RW		

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UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH00CM	C	39	BP	AP	232205	10

T:\APS Cholla temp\CIVIL\7.0 CIVIL\13 - Civil Details.dwg - Dtls 01/22/2025 1:44pm USAV721062



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 DEWATERING PIPE SHALL BE HDPE PER SPECIFICATIONS.
4. CONTRACTOR TO MAINTAIN A MINIMUM COVER OF 3 FT, UNLESS OTHERWISE SPECIFIED.
5. THE CONTRACTOR SHALL INSTALL DISCHARGE PIPING SUCH THAT THERE ARE NO HIGH POINTS IN THE PIPING. IF UNABLE TO INSTALL WITHOUT A HIGH POINT, THEN THE CONTRACTOR WILL INSTALL AN AIR RELEASE VALVE AT ALL HIGH POINTS IN THE PIPING.

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.

REBECCA A WEAVER — 1/22/2025
ARIZONA REGISTERED PROFESSIONAL ENGINEER — DATE

54953
ARIZONA REGISTRATION NUMBER

406-360-1405
TELEPHONE NUMBER

NORTH

HORIZONTAL GRAPHIC SCALE
100 0 100 200
(IN FEET)
1 inch = 100 ft.

Call at least two full working days before you begin excavation.
ARIZONA 811
Arizona Blue Stake, Inc.
Dial 8-1-1 or 1-800-STAKE-IT (782-5348)
In Maricopa County (602) 263-1100

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

THIS SHEET WAS ORIGINALLY SEALED BY REBECCA A WEAVER ON 11-23-2023

WORK SAFELY TODAY

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LEGEND:

- MONITORING WELL
- EXTRACTION WELL
- PIEZOMETER
- (S) SUMP
- EXISTING DISCHARGE PIPELINE
- EXISTING SEEPAGE INTERCEPT DRAINAGE TRENCH
- DISCHARGE PIPELINE
- OHE OVERHEAD ELECTRIC LINE
- APS PROPERTY BOUNDARY
- PROPERTY LINE BETWEEN BLM AND PRIVATE OWNER
- EASEMENT ON BLM LAND

NO.	DATE	REVISION	DWN	CHD	EXD	RVWD	APVD	W.A.
1	01/25	AS-BUILT	AV		JW	RW	MH	CHC08903
0.b	11/23	ISSUED FOR CONSTRUCTION	LLN	LV	GJD	RW	MH	CHC08903
0.a	09/23	ISSUED FOR BID	LLN	LV	GJD	RW	MH	CHC08903

APS CHOLLA GENERATING STATION
BAP SEEPAGE SYS EXTRACTIO WELL PUMP INSTALLATION
SYSTEM OVERVIEW

aps

SCALE AS SHOWN — DATE 01/22/2025

DWN	EXD	APPROVED	W A
HS	RW	MH	CHC08903
CHD CY	RVWD RW	DRAWING APPROVED BY	

UNIT	DISC	TYPE	SYS	SUBSYS	NUMBER	SHEET
CH	M	98	BP	AP	232214	1A

ATTACHMENT B

AECOM, 2021. *Cholla Power Plant Bottom 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 BOTTOM 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: Bottom 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 Bottom Ash Pond (BAP) 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 BAP 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: Bottom 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 BAP 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; (4) issues, including clogging by vegetation, at seepage areas and seepage monitoring locations; (5) erosion around the siphon line encasements on the South Embankment crest. 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 BAP, 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.

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:

    i. On limited occasions, readings may have indicated short-term turbidity increases that were attributed to surface water runoff.

    ii. On some inspections, vegetation growing at the seep monitoring/measurement locations have challenged the observation of seepage turbidity but the vegetation has been removed by the subsequent annual inspection.

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, although there have been deviations that have been attributed, with hindsight, to equipment reading issues and to pond-level changes:

    i. The flowrate recorded at the P-226 seep rose rapidly to 147.82 gpm in late 2018 without a then-apparent cause (ADWR Basic Data Report Figure 36).

ii. An immediate confirmation reading indicated the measured flows were back down to normal and stayed relatively low thereafter.

iii. By 2020, the P-226 seepage flowrates had increased but the increase was correlated to an elevated reservoir level earlier in the year.

iv. The final conclusion was that the totalizer may have malfunctioned on that day in late 2018 but that seepage rates do rise and fall in relation to the BAP water level.

e. The flowrate recorded at the Tanner Wash seep rose rapidly to 17.04 gpm at the end of 2020 (ADWR Basic Data Report Figure 38).

i. The delayed increase was attributed to the relatively distant location of the seep, 700 feet downstream of the embankment, and a high reservoir level earlier in the same year.

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 BAP 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 reevaluation of any material properties, cross-section configurations, or piezometric conditions of the perimeter embankment.

The calculated factors of safety for the two critical cross sections along the BAP 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

None identified and none 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.

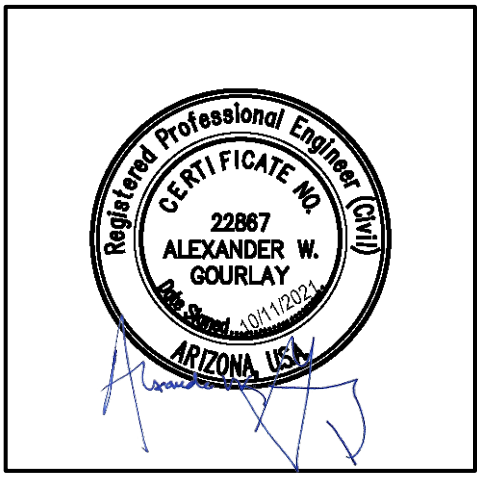
8. Certification Statement

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: Bottom 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: Bottom 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

Bottom 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
EL	Elevation
EPA	United States Environmental Protection Agency
ft	feet
HDPE	High Denisty Polyethylene
HPC	Hazard Potential Classification
I-40	Interstate 40
pcf	pounds per cubic foot
psf	pounds per square foot
PI	Plasticity Index
PMF	Probable Maximum Flood
RCRA	Resource Conservation and Recovery Act
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; Bottom 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 17, 2016 was conducted in accordance with the requirements of 40 CFR § 257.73.

Alexander W. Gourlay, P.E.

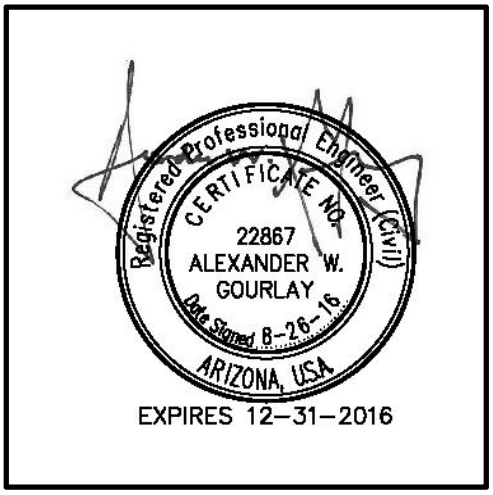
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Printed Name

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 Bottom Ash Pond, Arizona Department of Water Resources (ADWR) Dam No. 09.27. 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 Bottom Ash Pond located at the Cholla Power Plant. The Bottom 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 (RCRA). 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 assessments 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 Bottom 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 Bottom 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 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 Bottom Ash Pond.

The Bottom Ash Pond consists of a reservoir located in the southern portion of the pond, directly upstream of the dam and two coal combustion waste storage cells, the West Cell and the East Cell, located in the northern portion of the pond. The Bottom Ash Pond receives waste water from the Bottom Ash Transfer Sump that contains water and solids from the following sources: bottom ash overflow sump; bottom ash slurry from Units 1 through 4; Area 1, 2, and 3 Area Drainage Sumps; Units 1, 2, 3, and 4 Bottom Ash Hoppers; General Water Sump Liquids and Solids; Sedimentation Pond effluent; Units 1, 2, 3, and 4 Oil Water Separators; boiler cleaning waste; and water siphoned back from the Bottom Ash Pond. In addition, the following are discharged to the Bottom Ash Pond: scrubber sludge, Bottom Ash Pond stormwater, Units 3 and 4 Cooling Tower Basin Solids, seepage and stormwater from the Bottom Ash Monofill retention basins, General Water Sump Solids, Sedimentation Pond solids, WARP Solids, Flue Gas Desulfurization Wastes, and oil/water separator solids. The CCR wastes and other discharges are pumped to one of the two upstream waste storage cells, where the bottom ash is allowed to settle and the water is decanted to the reservoir for reuse at the power plant. At any given time, one of the waste storage cells is receiving bottom ash while the other is drained, excavated, and transported to a monofill north of the pond where the bottom ash is dry stacked. Excess water from the upstream cells are drained to the downstream reservoir via 12-inch pipes that outlet directly to the reservoir. The water is decanted from the reservoir for reuse at the power plant through a siphon system.

The Bottom Ash Pond has a total surface area of about 80 acres and a total storage capacity including solids and water of about 2,300 acre-ft when at its ADWR permitted maximum storage pool water level of EL 5,117.8 ft (ADWR, 1986). The impoundment is surrounded on its west and north sides by natural topography consisting of rock outcrops of mudstones, siltstones, and sandstones. On the south and east side, the impoundment is enclosed by the Bottom Ash Pond Dam, Arizona ADWR Dam No. 09.27, which spans the Tanner Wash. The Bottom Ash Pond has been classified under ADWR regulations as a high hazard impoundment due to 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 Bottom 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. In 1993 the dam crest was raised 3.3 ft when it became apparent that the storage volume of the pond was inadequate and the pond was filling faster than anticipated. In 1999 the impoundment was altered to its current configuration consisting of two upstream waste drainage cells and a downstream reservoir. The two waste cells are alternately dried and dredged to facilitate the removal of bottom ash from the sluice water which drains to the reservoir. By this procedure, the total storage volume in the pond remains relatively constant. The dam is approximately 4,200 ft in length with a maximum toe to crest height of 73 ft and crest width of 12 ft. The top of crest elevation is elevation (EL) 5,123.3 ft after the 1993 crest raise providing 5.5 ft of 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 except for the upper portion of the slopes constructed during the crest raise where the slopes are inclined at a 1.5H:1V vertical angle. Both upstream and downstream slopes are lined with riprap facing to prevent erosion.

To limit seepage beneath the foundation, the central clay core of the Bottom 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 the bedrock or stiff clay. The Bottom 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 storm water and groundwater and return it to the pond. These include seepage collection systems for the West Abutment Seep, the Petroglyph Seep, the P-226 Seep, and the Tanner Wash Seep.

The Bottom Ash Pond has no fixed intake or outlet water work structures. Sluiced bottom ash is pumped from the plant to the pond through a discharge line that runs up the right dam abutment, adjacent to the embankment and to a screening plant that scalps off some of the bottom ash solids for commercial use as light weight aggregate by the Salt River Materials Group. The processed slurry is then pumped to one of the two waste cells where the bottom ash settles and the excess water is decanted to the reservoir. Water levels within the pond are controlled by varying the pumping rate from the plant and seepage control system to balance with seepage, evaporation, and siphon system in the reservoir. The siphon system consists of three 12-inch diameter high density polyethylene (HDPE) pipes that float near the surface of the reservoir. The pipes were originally 8-inch in diameter but were replaced with the current 12-inch diameter pipes in the late 2000s. The siphon system pipes extend through the upper portion of the dam at a pipe invert elevation of about EL 5,120.5 ft and continue down the downstream face of the dam to a common valve chamber that combines the flow into a return pipe. 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 5,119.3 ft based on an expected water level rise of 1.5 ft during the PMF (Dames & Moore, 1991).

Piezometers, settlement monuments, flow measurement devices, and water level gauges are installed at the Bottom Ash Pond to monitor the performance of the dam. Measurements from the monitoring instruments are reviewed by AECOM and documented annually in a data report. Starting on October 19, 2015, the piezometer, survey monuments, V-notch weirs, and sumps 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, weir, and flow totalizers are shown on Figure 1-2.

Inspections of the Bottom Ash Pond are performed by a qualified person at intervals not exceeding seven days. The inspections examine the Bottom 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 Bottom 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 Bottom 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 Bottom 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. A dam break analysis and inundation mapping has been documented for the Bottom Ash Pond (Stantec, 2000). The inundation was reportedly mapped downstream in the Tanner Wash to the Joseph City Wash. Habitable structures reported in the inundation area included I-40, the Burlington Northern & Santa Fe Railroad, and the Cholla Power Plant (Stantec, 2000). 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 Bottom 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) and the dam break analysis report (Stantec, 2000) identifies that the Bottom Ash Pond is located approximately 2,000 ft upstream of I-40, a major east-west route of the Interstate Highway System. A catastrophic and unexpected failure of the Bottom Ash Pond Dam would likely inundate the travel lanes of I-40 and could result in loss of life. The Bottom 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 Bottom 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 Bottom 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 Bottom 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 Bottom 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.	09.27	---	ADWR License of Approval dated December 11, 1998
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 Toe Drain System: 1979 Embankment Raise: 1993 Seepage Collection System: 1993 Impoundment Reconfiguration: 1999	---	• Ash Pond Construction Memorandum (Ebasco, 1977) • As-built Drawing No. G-44556 (Ebasco, 1977) • Toe Drain System Drawing No. D-82671 (APS, 1991) • Siphon System & Floating Pipeline As-built APS Drawing No. G566-S02 (APS, 1993)
Location on USGS Quadrangle Map	Joseph City Quadrangle: Section 13, Township 18 North, Range 19 East	See Figure 3-1	Joseph City Quadrangle (USGS, 2013)
Statement of Purpose	Bottom ash containment		Seepage and Foundation Studies: Volume I of II Engineering Report (Ebasco, 1975).
Name of Watershed	---	---	---
Size of Watershed (ac)	128	---	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 No. G-44556 (Ebasco, 1977)
Embankment Maximum Height (ft)	73	---	• As-built Drawing No. G-556 (Ebasco, 1977) • Siphon System & Floating Pipeline As-built APS Drawing No. G566-S02 (APS, 1993)
Design Total Freeboard (ft)	5.5	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)

Item	As-Constructed/ Current	Comments	Reference Document
Embankment Length (ft)	4,040	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Embankment Crest Elevation (ft)	Original: 5,120 Modified: 5123.3	Embankment raised in 1993	• As-built Drawing No. G-556 (Ebasco, 1977) • Siphon System & Floating Pipeline As-built APS Drawing No. G566-S02 (APS, 1993)
Embankment Crest Width (ft)	Original: 16 Modified: 12	---	• As-built Drawing No. G-44556 (Ebasco, 1977) • Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM, 2016)
Embankment Slopes	Original 3H:1V (downstream & upstream) Raised Upper 3 ft: 1.5H:1V (downstream & upstream)	---	As-built Drawing No. G-44556 (Ebasco, 1977)
Slope Protection	Riprap		Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Maximum Operating Storage Level (ft)	5117.8	Previous maximum storage levels were: 5,116 ft (1981); 5,114 ft (1984, 1986); 5,115 ft (1990, 1992); 5,118.6 ft (1993)	ADWR License of Approval dated December 11, 1998
Storage Capacity (ac-ft)	2,300	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Surface Area (ac)	80	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
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			

Item	As-Constructed/ Current	Comments	Reference Document
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}	---	
Foundation Conditions			
Physical Properties	The embankment is generally founded on an engineered keyway consisting of the compacted clay core extending to competent bedrock. The exposed bedrock was cleaned and received grout treatment 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) • Safety Inspection Report (Harza, 1987)
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}	---	
Engineering Properties (continued)	<u>Cutoff Wall:</u> • Moist Unit Weight = 106 pcf	---	• Seepage and Foundation Studies: Volume II of II Field and Laboratory Tests (Ebasco, 1975)

Item	As-Constructed/ Current	Comments	Reference Document
	• Saturated Unit Weight = 106 pcf • Effective Cohesion = 0 psf • Effective Friction Angle = 28^{o} Undrained Strength = 10 psf		• Safety Inspection Report (Harza, 1987)
Abutment Conditions			
Physical Properties	The abutments consist of bedrock comprising mudstone, siltstone, and sandstone associated with the Chinle and Moenkopi Formation. The cut off wall that was part of the engineered foundation of the embankment was extended 350 ft beyond the end of the dam into the right abutment.	---	• 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 = 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 feet of freeboard.	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Construction Specifications	<u>Clay Core:</u> • Fines content ranging from 50% to 100% • No particle sizes greater than 3 inches • Plasticity index ranging from 15 to 50 • Fill lift thickness = 8 inches • Minimum degree of compaction = 95% (standard Proctor) • Test frequency = 60,000 ft^2/test	---	Ash Pond Construction Memorandum (Ebasco, 1977)
Construction Specifications (continued)	Shell (<u>Random Zone):</u> • Maximum rock fraction greater than	---	Ash Pond Construction Memorandum (Ebasco, 1977)

Item	As-Constructed/ Current	Comments	Reference Document
	3 inches = 10% • Fill lift thickness = 12 inches • Minimum degree of compaction = 100% (standard Proctor) • Test frequency = 60,000 ft^2/test Cutoff Wall: • Preparation: o Minimum unit weight = 1.02 grams/cubic centimeter (g/cm^3) o Minimum viscosity = 30 sec-marsh o Maximum filtration loss = 30 cm^3 o Minimum pH = 8 • In Trench: o Unit weight range between 1.05 and 1.3 g/ cm^3 • Backfill Mix at Trench: o Slump ranging between 3 and 6 inches o Percent passing 3/8-inch between 60 and 90% o Percent passing No. 20 sieve between 30 and 70% Fines content between 15 and 30%		
Detailed Drawings	See Appendix A for drawings	---	• Original As-built (Ebasco, 1977) • Ash Disposal Line Reroute (Ebasco, 1980) • Crest restoration (APS, 1990) • Siphon System & Floating Pipeline (APS, 1993) • Seepage Interception System (APS, 1993)
Existing Instrumentation			

Item	As-Constructed/ Current	Comments	Reference Document
Type and Purpose of Instrumentation	• Open standpipe piezometers and wells for monitoring the phreatic levels in the embankment and foundation. • Settlement monuments for monitoring movement of the embankment. • Water level gauge for monitoring water level in reservoir. • V-notch weir and seepage monitoring systems for measuring seepage rates.	---.	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
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. • V-notch weir and 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 professional engineer. • Phreatic level behavior from piezometric measurements and reservoir water level from gauge collected on an interval not exceeding 30 days. • Embankment settlement using movement monuments survey data collected on an interval not exceeding 30 days. • Seepage monitoring at the downstream toe collected on an interval not exceeding 30 days.	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)

Item	As-Constructed/ Current	Comments	Reference Document
Record of Structural Instability (See Section 4 for more details)	• Historic seepage at downstream toe • Active seepage at the right abutment. • Seepage areas near the downstream toe are identified as Toe Drain Seep, Petroglyph Seep, Tanner Wash Seep, and P-226 Seep.	See Figure 1-2 for seepage areas. The seepage areas are captured and monitored by a seepage interceptor system near the downstream toe.	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 Bottom 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), existing CCR impoundments must be assessed for "*Stable foundations and abutments*."

The Bottom Ash Pond Dam is founded on alluvium overburden associated with the Tanner 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,1990) and construction inspection reports prepared by ADWR (formerly 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 or to a stiff clay layer found about 60 to 70 ft below the original ground surface. In addition, an approximately 350 ft long slurry wall was installed beyond the right abutment to help control seepage through the Moenkopi bedrock formation. Review of the 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 Bottom 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, 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. Four sumps and one weir have been installed at the following seeps: the P-226 Seep, the Petroglyph Seep, the Tanner Wash Seep, and the West Abutment 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 0 to 25 gallons per minute over the last ten years were measured at the sumps and weirs (AECOM & APS, 2016), indicating very low to moderate flow. The turbidity of the seep water observed at the sumps was also low. Both the low flow rate and 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 Bottom Ash Pond Dam, as presented in the 2015 annual dam inspection report (AECOM & APS, 2016), indicates settlements along the crest of the dam of three to eight inches and horizontal movements of three inches or less in the last ten years. Settlement rates appear relatively constant over the last ten years at about one quarter of an inch per year with little horizontal movement upstream or downstream. The relatively small settlement and horizontal movements measured at the Bottom Ash Pond Dam are an indication of stability in the dam foundation and abutments.

4.2 Slope Protection

Per the requirements 40 CFR *§* 257.73(d)(1)(ii), 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 Bottom Ash Pond Dam (Ebasco, 1990), 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 Bottom 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), 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 Bottom 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 fill pad testing 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 Materials D 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-J12 (Temchin, 1977). The drawing indicates 368 tests were conducted on Clay Core materials with 357 of the tests measuring densities greater than 95 percent of the Standard Proctor maximum density and a mean percent compaction of all tests of 98.4 percent of the Standard Proctor maximum density. The drawing indicates 104 tests were conducted on the outer shell materials with 91 of the tests measuring densities greater than 100 percent of the Standard Proctor maximum density and a mean percent compaction of all tests of 101.2 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 Bottom Ash Pond Dam has been mechanically compacted to a density sufficient to withstand the range of loading condition expected at the Bottom Ash Pond site.

4.4 Slope Vegetation

Per the requirements 40 CFR § 257.73(d)(1)(iv), 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), existing CCR impoundments 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 Bottom Ash Pond Dam was constructed without a spillway or other water release structure. To manage flow during the design storm event, the Bottom Ash Pond has been designed, constructed, operated, and maintained with sufficient storage volume over and above the maximum permitted storage pool water level (EL 5,117.8 ft) to store the PMF storm water inflow at EL 5,119.3 ft and maintain four ft of freeboard; therefore, the Bottom 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), 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*."

Three 12-inch diameter HDPE pipes associated with the siphoning system which returns water back to the plant are the only hydraulic structures that penetrate the Bottom Ash Pond Dam embankment. Review of the as-built drawings of the siphon system (Ebasco, 1993) indicates the pipes are installed near the crest of the dam with penetration invert elevations of EL 5120.5 ft. The maximum surcharge pool water level, the highest water level anticipated within the pond, is at EL 5,119.3 ft a little over a foot below the pipe penetrations. Since it is not anticipated that the water level will rise to the elevation of the pipe penetration, the pipes are not expected to negatively impact the operation of the dam. Furthermore, since the pipes are buried at a relatively shallow depth beneath the crest, significant distortion or bending of the pipes would be readily apparent and can be easily repaired.

4.7 Downstream Water Body

Per the requirements 40 CFR § 257.73(d)(1)(vii), 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 present associated with inundation of the downstream slope of the Bottom 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

No deficiencies were identified for the Bottom 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 Bottom Ash Pond Dam. APS work crews subsequently removed vegetation in the identified areas. 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 Bottom 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 Bottom 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 two cross sections of the Bottom 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 Bottom Ash Pond Dam are shown in Figure 5-1. The cross sections analyzed are:

Bottom Ash Pond Cross Section 1: This cross section corresponds approximately to Section A as shown on Figure 5-1 and the as-built plan (Ebasco, 1990). This section represents the maximum section in areas 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 73 ft high and the upstream and downstream slopes are at 3H:1V except for the top 3.3 ft of the dam where they are 1.5H: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 10 ft of alluvial overburden (clays, silts, and sands) overlying interbedded layers of mudstone, siltstone, and sandstone bedrock. The clay core extends to form a cutoff trench that is keyed into the top of bedrock.

Approximately 28 ft of hydraulically-placed bottom 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 (Cooper Aerial Surveys, 2014).

Bottom Ash Pond Cross Section 2: This cross section corresponds approximately to Section H as shown on Figure 5-1 and the as-built plan (Ebasco, 1990). This section represents the maximum section in area of the deepest bedrock (85 ft below the ground surface). The section includes a cutoff slurry wall beneath the embankment clay core. The embankment is approximately 73 ft high and the upstream and downstream slopes are at 3H:1V except for the top 3.3 ft of the dam where they are 1.5H: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 85 ft of alluvial overburden (clays, silts, and sands) overlying interbedded layers of mudstone, siltstone, and sandstone bedrock. The slurry cutoff wall consists of a minimum two ft thick soil-bentonite wall that extends from the clay core of the dam to a layer of dense clay at about 15 ft above the bedrock.

5.3 Subsurface Stratigraphy

Idealized models of subsurface stratigraphic conditions for each cross section were developed based on design drawings (Ebasco, 1990) and previous geotechnical site investigations (Ebasco, 1975, Harza, 1987, and Dames & Moore, 1999). The stratigraphic units described as follows 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, 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 Tanner Wash prior to the construction of the Bottom 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.

Bottom Ash: Bottom ash waste product from the power generating process is pumped from the plant to the Bottom Ash Pond and allowed to settle hydraulically in two coal combustion waste storage cells upstream of the dam. Excess water from the storage cells decants to a reservoir directly behind the dam.

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, FAP 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 Bottom 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 an 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 bottom ash unit weight was selected by AECOM to be 75 pounds pcf based on engineering experience with similar materials.

Table 5-1. Selected Material Parameters – Bottom 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	-
Bottom Ash	75	75	0	25	-	-

5.5 Embankment Pore Pressure Distribution

Water levels have historically been measured quarterly, but beginning in October of 2015, they are now monitored on an interval not exceeding 30 days in the piezometers installed along or near the Bottom 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 Bottom Ash Pond Dam embankment. The pore pressure distributions were estimated for each section using water level measurements obtained from:

- Section 1: piezometers W-227, B-200, B-201, and B-218;
- Section 2: piezometers W-305, B-203, B-204, and B-205

Piezometer locations are shown on Figure 1-2. Piezometer data were used, along with pond water level under steady-state, maximum storage pool conditions (ADWR, 1986 and ADWR, 1998), and pond water levels under maximum surcharge pool conditions (Ebasco, 1975 and Dames & Moore, 1991) to estimate pore pressure distributions with the embankment sections. Piezometer data are presented in the calculation in Appendix B.

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 the plant, seepage, evaporation, and the siphon return system to control water levels, the maximum storage pool was set at the maximum ADWR-permitted water levels. For the Bottom Ash Pond, the safety factor was calculated for the long-term maximum storage pool at EL 5,117.8 ft (ADWR, 1998).

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 1.5 ft (Dames & Moore, 1991). For the Bottom Ash Pond, the safety factor was calculated for the maximum surcharge pool at EL 5,119.3 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 percent 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 2; 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 Bottom Ash Pond for the seismic loading analysis was set to EL 5,117.8 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 will 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 Bottom 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 included in Table 5-2 for the Embankment Shell material due to the very limited amount of available geotechnical data. 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 Index (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. Consequently, a liquefaction factor of safety analysis 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 Bottom 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
Long-term, maximum storage pool	1.50	1.58	1.55
Maximum surcharge pool	1.40	1.56	1.53
Seismic	1.00	1.05	1.07

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

The calculated factors of safety for the two critical cross sections along the Bottom 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 for the Bottom Ash Pond at the Cholla Power Plant.

- The Bottom 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 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 information and quality control test results, the Bottom Ash Pond Dam embankment was mechanically compacted to a density sufficient to withstand the range of loading conditions anticipated at the site.
- The Bottom 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 feet of freeboard.
- Factors of safety greater than the minimum values required by the CCR Rule were calculated for two critical cross sections along the Bottom 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 Bottom 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

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Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 1-1_SITE VICINITY MAP.DWG



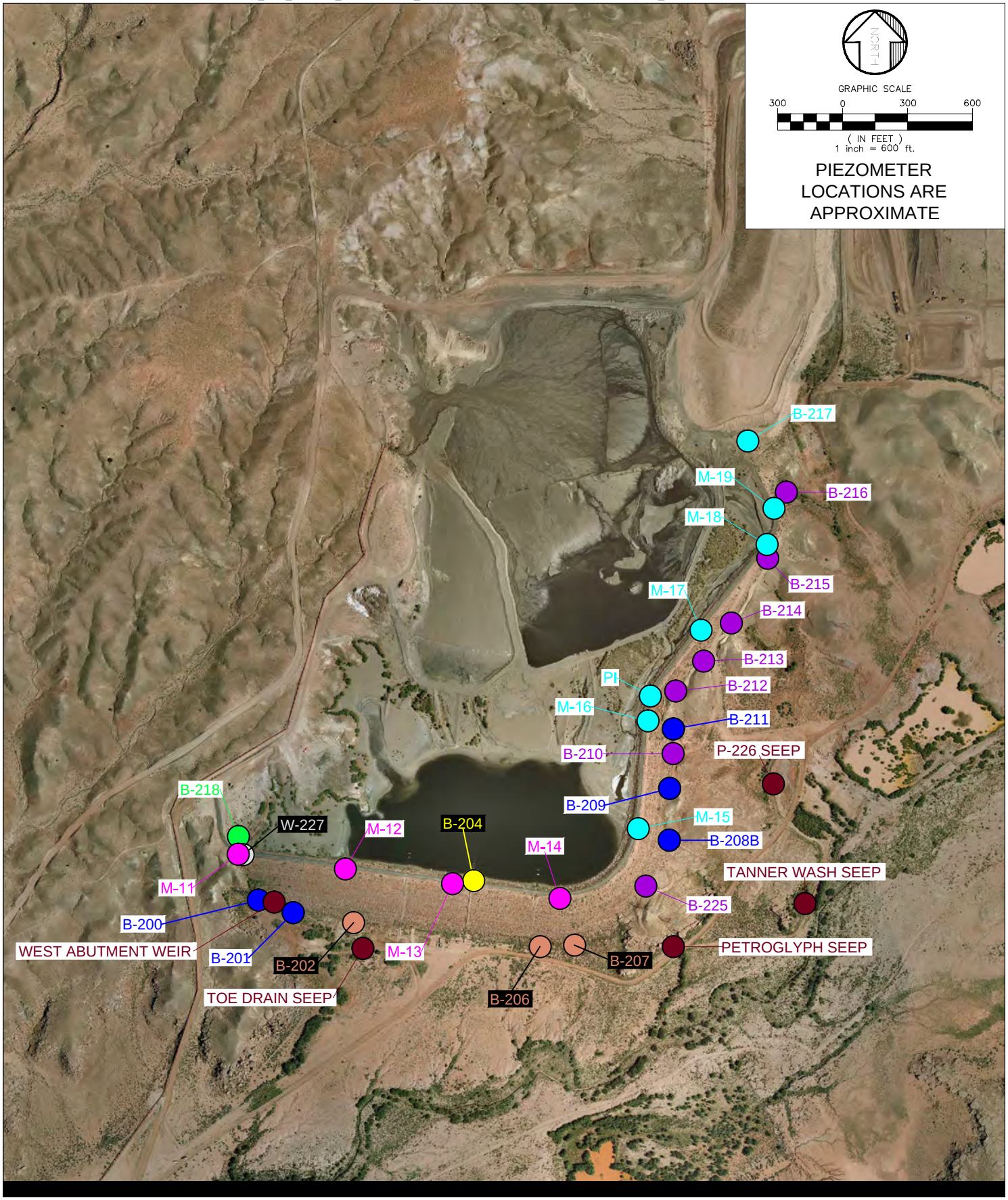
CHOLLA POWER PLANT
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SITE VICINITY MAP

AECOM

FIGURE 1-1

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



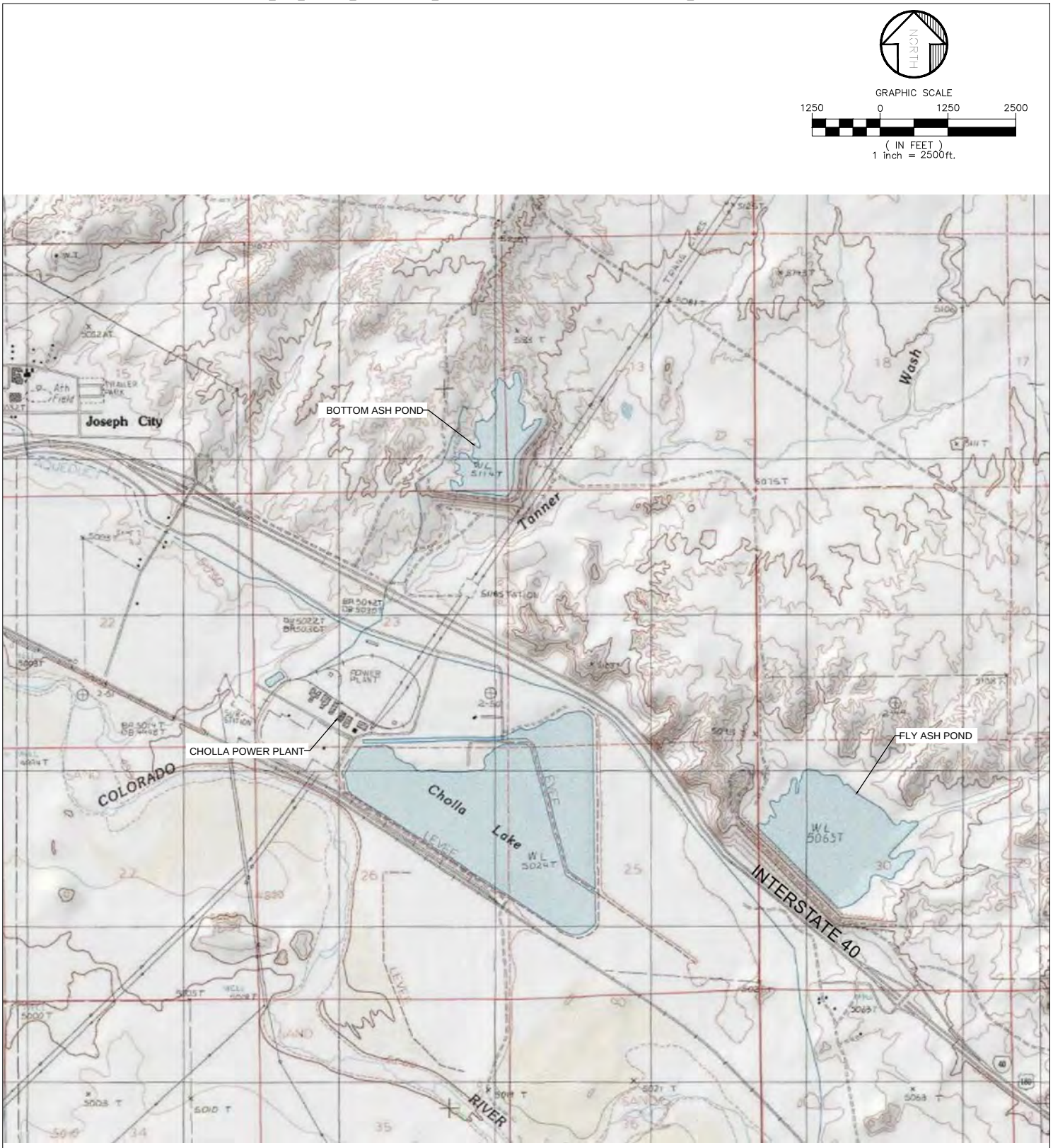
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BOTTOM ASH POND
MONITORED INSTRUMENTATION
AND SEEP LOCATION MAP

AECOM

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

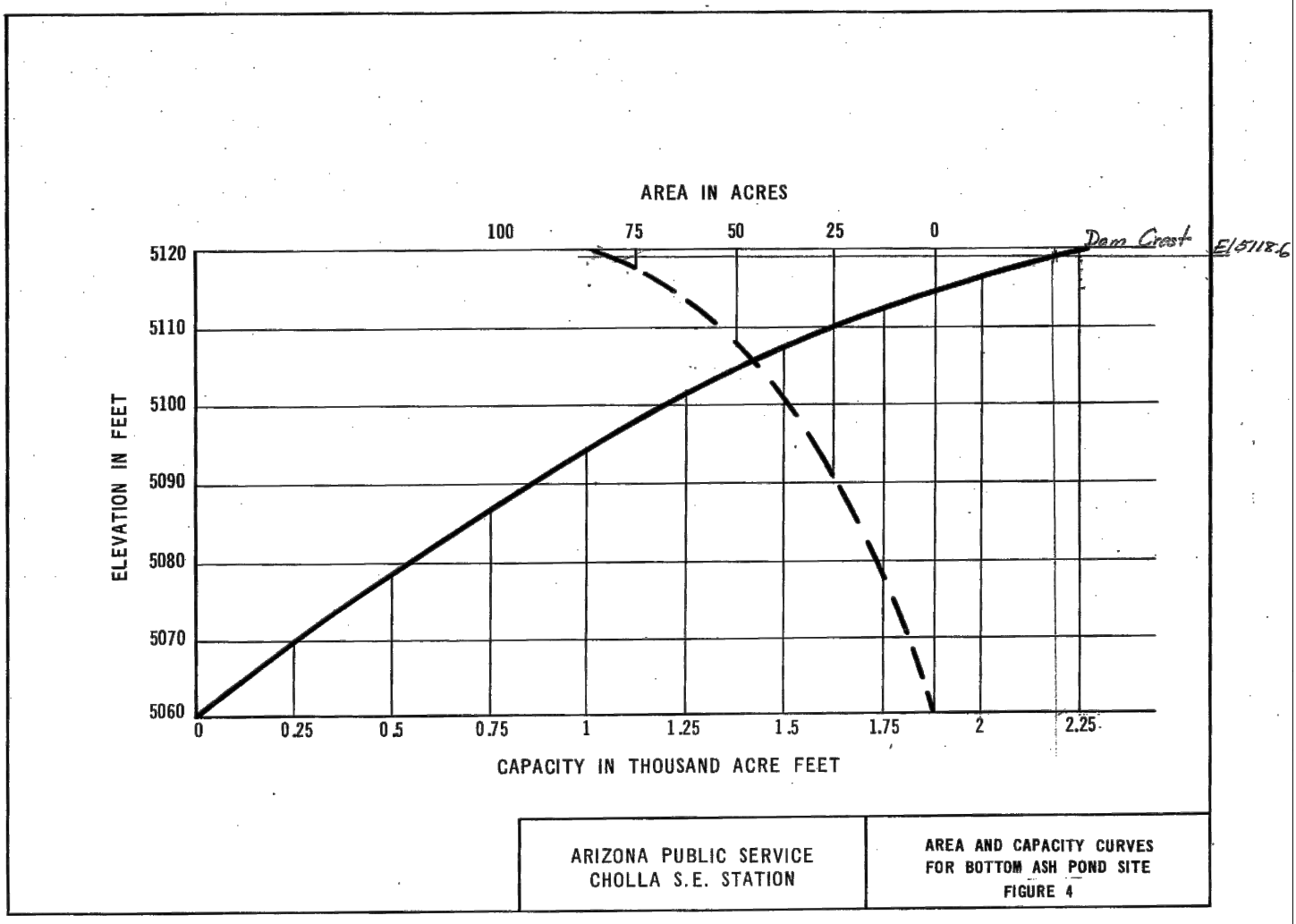
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SITE TOPOGRAPHIC MAP

AECOM

FIGURE 3-1

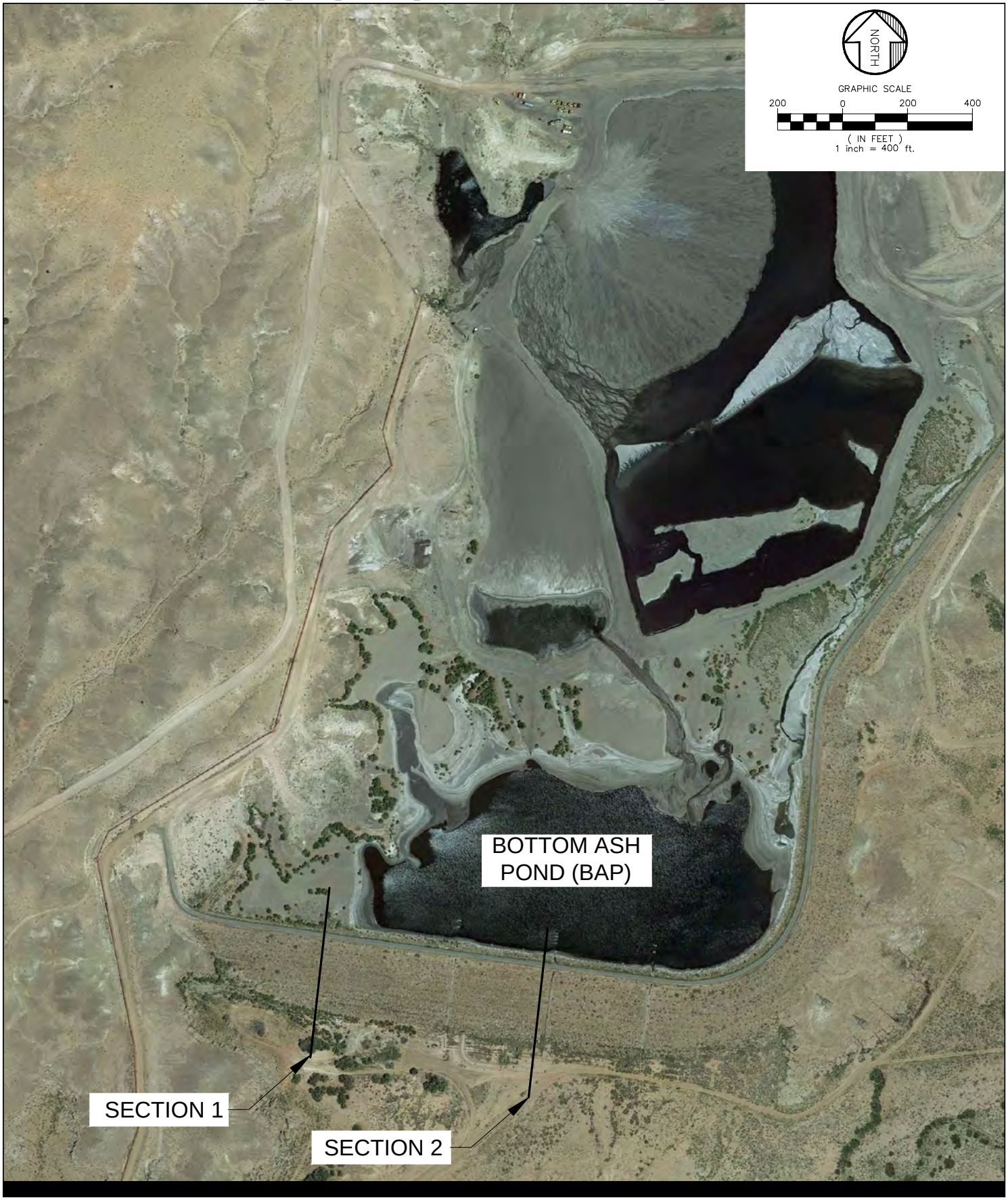
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Filename: P:\ARIZONA PUBLIC SERVICE\60445840_APS_CHOLLA_STRUCTURAL_INTEGRITY\CADD FILES\FIGURES\FIGURE 4-2_AREA CAPACITY CURVE.DWG



SOURCE:

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

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FOUR CORNERS POWER PLANT
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CROSS SECTION LOCATIONS
SAFETY FACTOR ASSESSMENT

AECOM

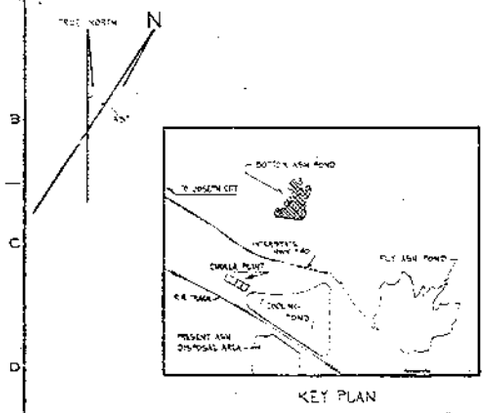
FIGURE 5-1

Appendix A.
Historic Drawings

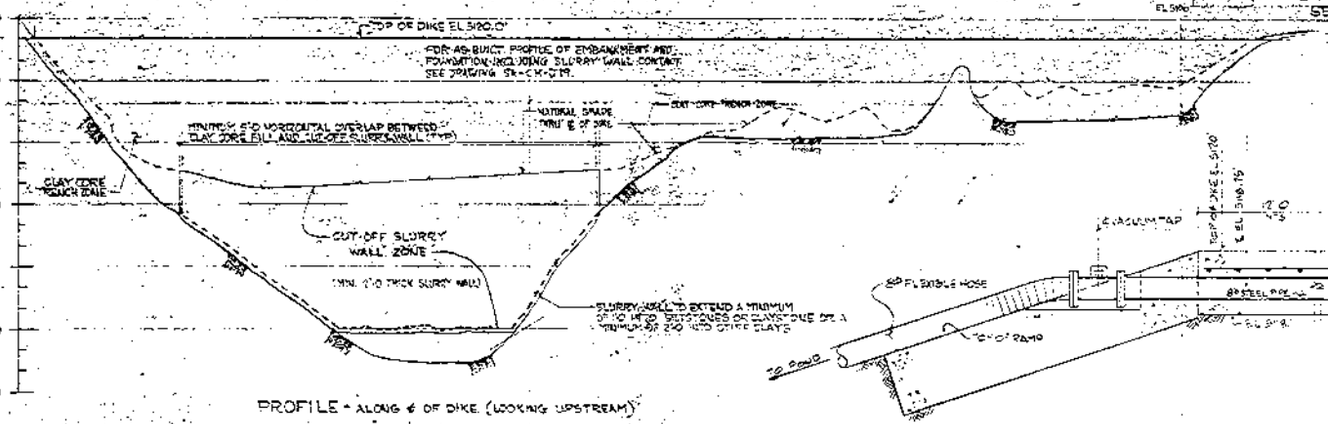
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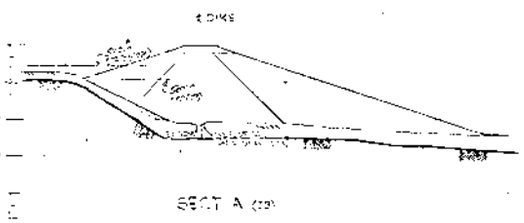
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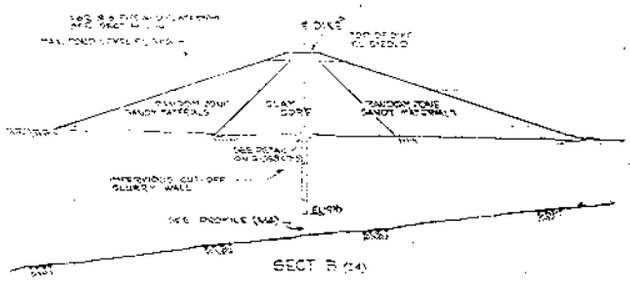
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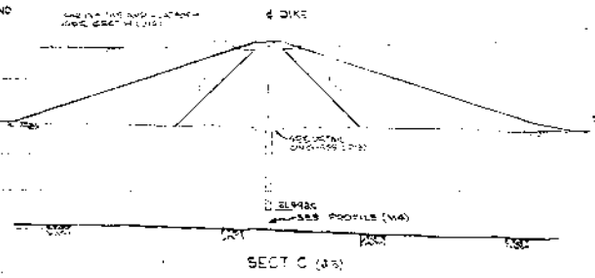


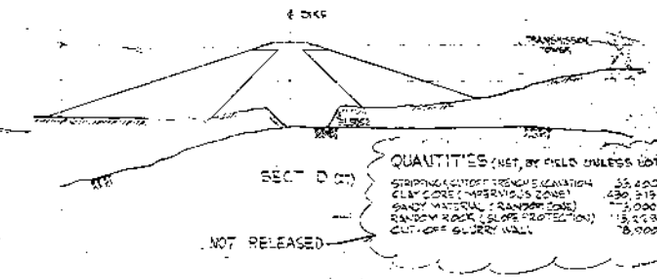
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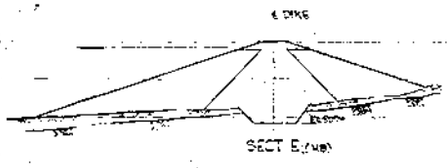


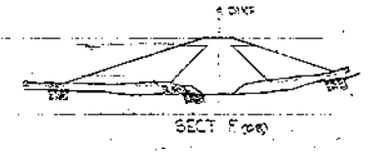


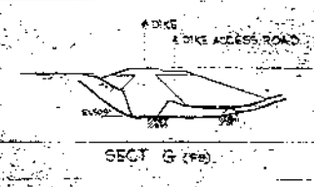
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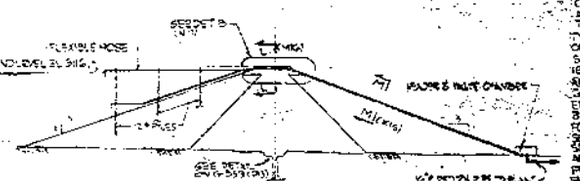
QUANTITIES (NET, BY FIELD UNLESS NOTED)

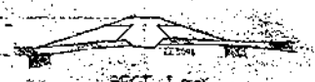
STRIPPING & CUTOFF TRENCH EXCAVATION	55,600 CU YD
CLAY CORE (IMPERVIOUS ZONE)	430,319 CU YD
SANDY MATERIAL (RANDOM ZONE)	1,000 CU YD
RANDOM ROCK (SLOPE PROTECTION)	13,200 CU YD
CUT-OFF SLURRY WALL	8,700 SQ FT

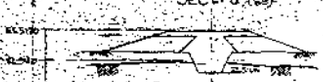


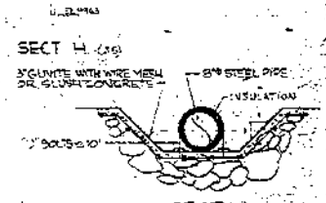


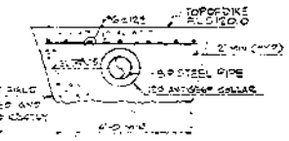












NOTES

DIKE TO BE CONSTRUCTED IN ACCORDANCE WITH REQUIREMENTS OF SPECIFICATION APS-2742-54-09 "EMBANKMENT, DIKES-ASH POND SITES" LATEST REVISION.

CLEARED OF ALL BRUSH, GRASS, DEBRIS, OBSTRUCTIONS AND OTHER ENCUMBRANCES AND THEN STRIPPED TO A MINIMUM DEPTH OF SIX INCHES.

TYPICAL EMBANKMENT DIKE SECTIONS ARE SHOWN ON DRAWING 2742-6-S-S555.

SECTION CORNERS SHOWN AT NE 15-19 TOWNSHIP 18N AND RANGE 7E.

ALL COORDINATES REFER TO THE PLANT AREA GRID SYSTEM.

EMBANKMENT DIKE GEOMETRY TO BE VERIFIED BY FIELD SURVEYS.

FOR LOCATIONS OF BORINGS, BORING LOGS AND SURFACE & SUBSURFACE SOILS AND BEDROCK AREA CONTOUR MAPS, BASED ON BORING LOGS AND GEOLOGICAL AND FOUNDATION STUDIES, INCLUDES SLOPE STABILITY

PIEZOMETERS SHALL BE INSTALLED AT SITE LOCATIONS SHOWN ON THE PLAN FOLLOWING COMPLETION OF CONSTRUCTION. ANY PIEZOMETER DAMAGED BY CONSTRUCTION ACTIVITY SHALL BE REPLACED AT A LOCATION SPECIFIED BY OWNER.

MINIMUM RADIUS OF DIKE ℄ 100' UNLESS NOTED.

FOR LOCATION OF NEW PIEZOMETERS SEE DRAWING SK-C-H-T21.

THE MAXIMUM AVERAGE SOLIDS STORAGE LEVEL INDICATED ON THE TYPICAL DIKE SECTIONS REPRESENTS THE LEVEL THAT IS COMPLETELY DRY AT THE PROBABLE MAXIMUM THUNDERSTORM RUNOFF WITH 4 FEET OF MINIMUM FREEBOARD.

PROBABLE MAXIMUM THUNDERSTORM RUNOFF VOLUMES ARE:

FLY ASH POND 308.5 ACRE-FT

BOTTOM ASH POND 1017 ACRE-FT

LEGEND

NI INDICATES NEW PIEZOMETER

EI INDICATES EXISTING PIEZOMETER

REFERENCE DRAWINGS

FLY ASH POND DIKE - SECT & DET. 2742-6-S-S556

KEYNOTE AERIAL MAPPING INC. PHOENIX, ARIZ. GUARANTEED SECTION 6SW POND SYPHON SYSTEM PLOTTING PLOTTED 6-5-75

WORK THIS DRAWING WITH 2742-6-S-S555

AS BUILT

NO	DATE	REVISION	BY	CK	APPROVED
10	11-8-77	AS BUILT	BM	SF	
9	2-28-77	SUBMITTED AS BUILT. CONSTRUCTION AND AS BUILT DRAWINGS SUBMITTED AND APPROVED. ALL ELEVATIONS REVISED TO MEET CONTRACT REQUIREMENTS. (2) TYPICAL SECTIONS REVISED.	SB	BM	
NO	DATE	REVISION	BY	CK	APPROVED

NO	DATE	REVISION	BY	CK	APPROVED
8	11-22-76		BM	JE	
7	6-22-76		BM	JE	
6	7-12-76		SE	DE	
5	6-24-76		SE	DE	
4	6-14-76		SE	DE	
3	5-14-76	GENERAL REVISIONS	DT	RP	
2	4-16-76		JE	LC	
1	3-26-76		EB	LC	
NO	DATE	REVISION	BY	CK	APPROVED

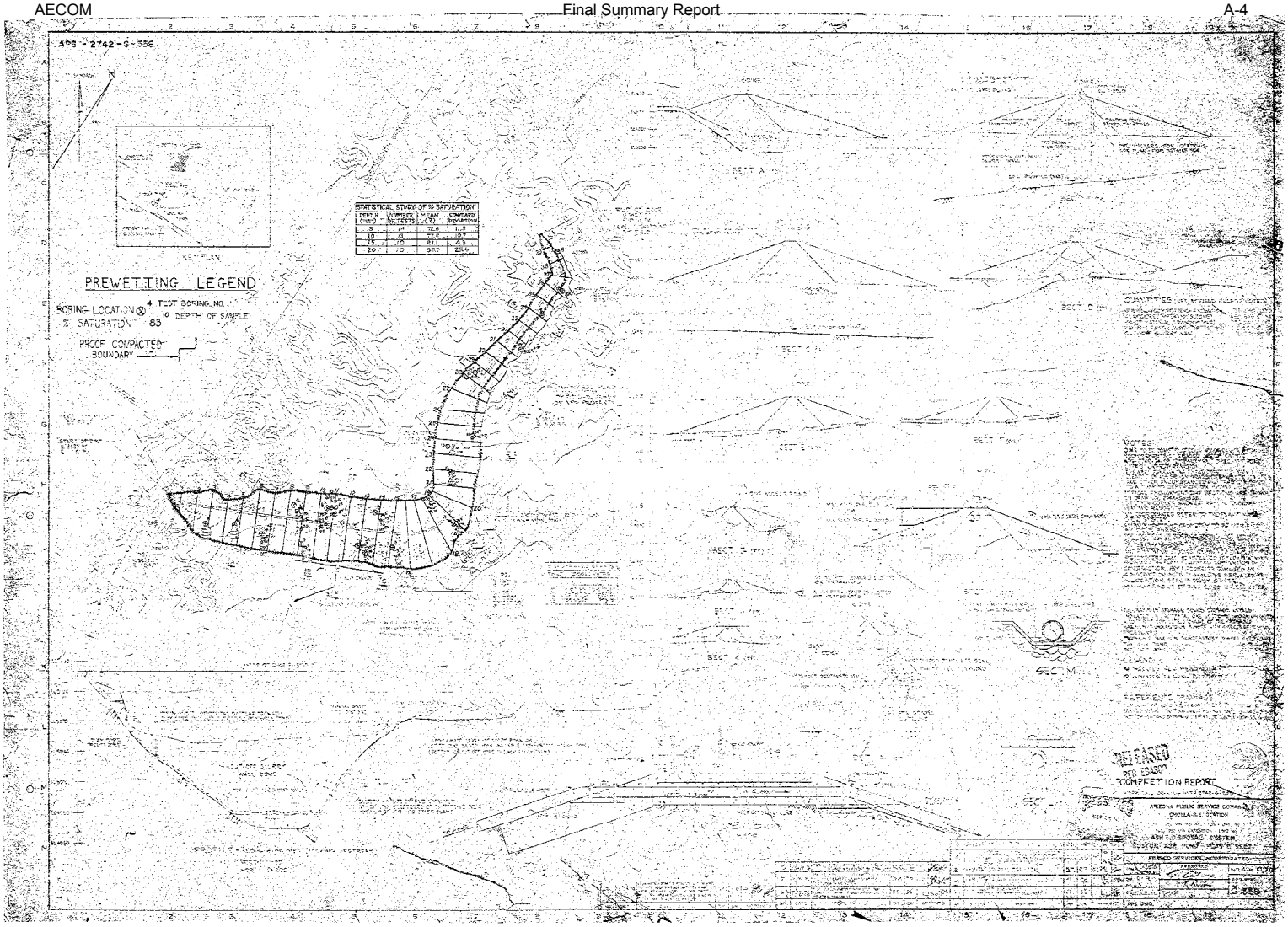
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1977 - 350 MW INSTALLATION - UNIT No. 2
1978 - 350 MW EXTENSION - UNIT No. 3
ASH DISPOSAL SYSTEM
SECTION - DIKE - PLAN & PROFILE

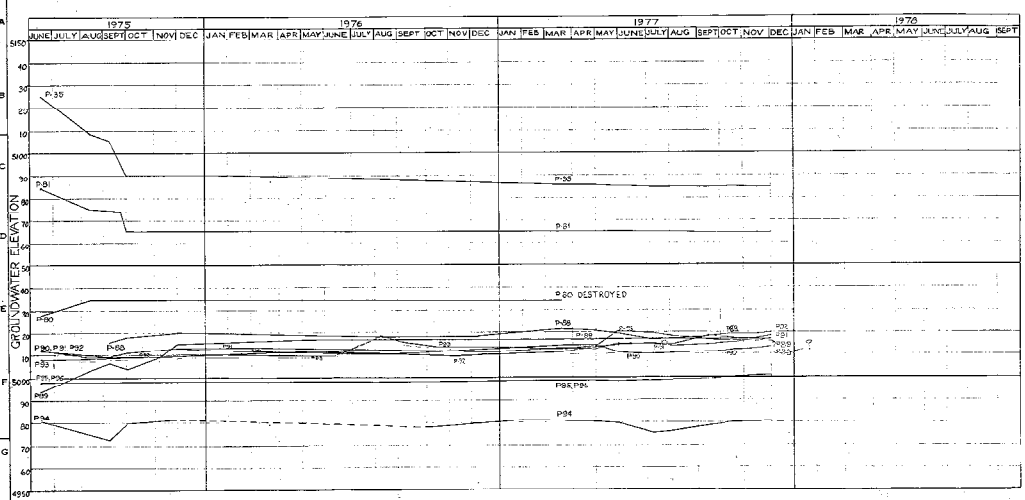
BECHTEL SERVICES INCORPORATED

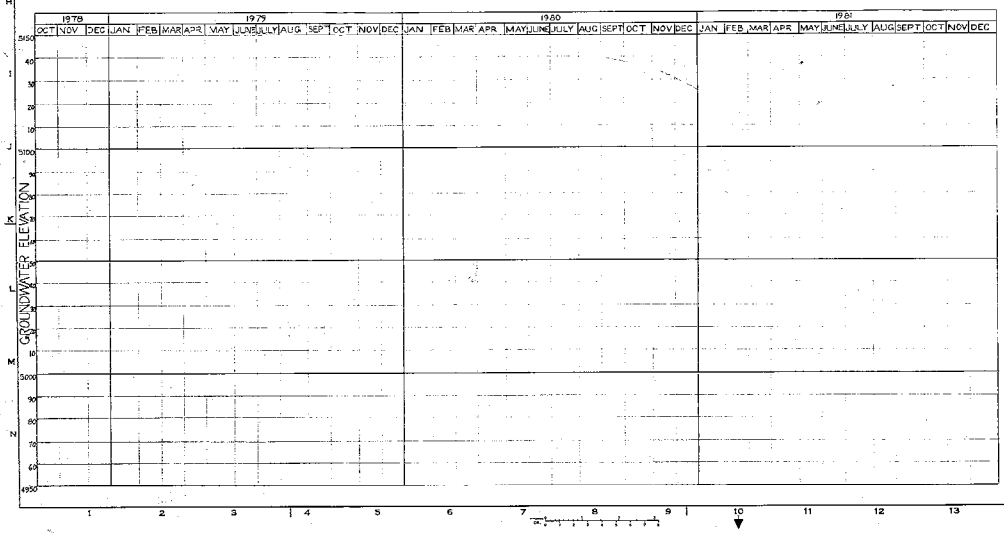
	APPROVED	DATE
DESIGN ENGINEER		
PROJECT ENGINEER		
PROJECT MANAGER		

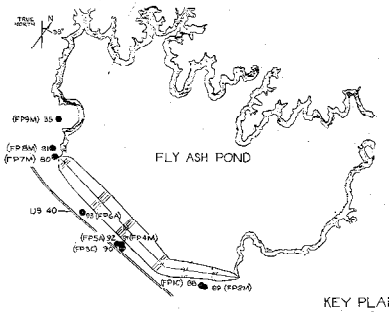
SCALE AS NOTED

N 18 19 09-27

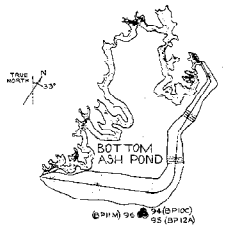

APS - 2742-6-356
KEY PLAN
PREWETTING LEGEND
SORING LOCATION ⊗ 4 TEST BORING NO.
% SATURATION 83 10 DEPTH OF SAMPLE
PROOF COMPACTED BOUNDARY
STATISTICAL STUDY OF % SATURATION
SECT A
SECT B
SECT C
SECT D
SECT E
SECT F
SECT G
SECT H
SECT J
SECT K
SECT L
SECT M
RELEASED FOR ISSUE
COMPLETION REPORT
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA U-1 STATION
ASH DISPOSAL SYSTEM
BOTTOM ASH POND
BECHTEL POWER CORPORATION







KEY PLANS
NTS



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

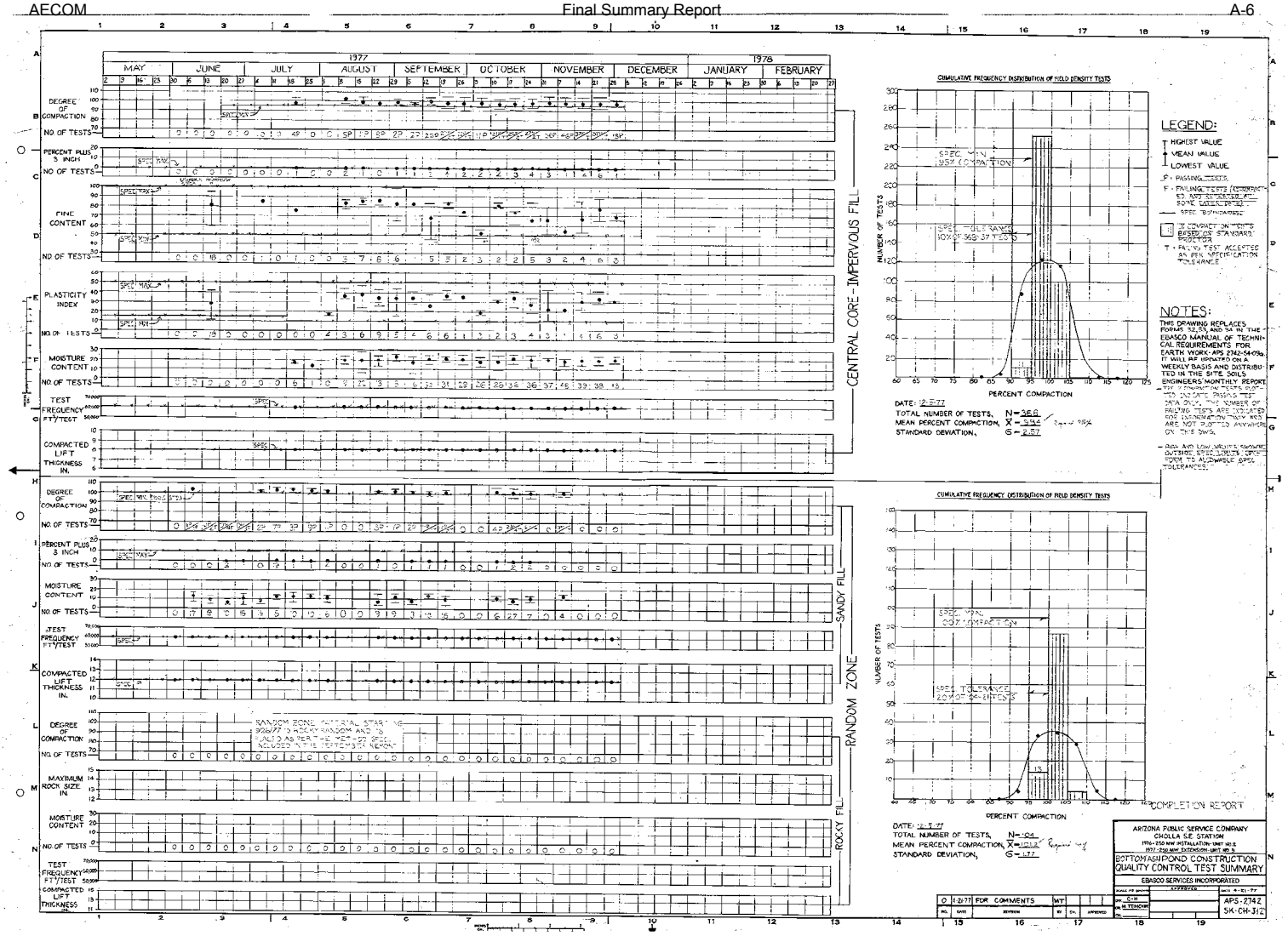
NO.	DATE	REVISION	BY	CH	APPROVED
0	11-29-77	FOR COMMENTS	MT		

ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
195MW COAL INSTALLATION - UNIT No. 2
(OPTIONAL 195 MW EXTENSION-UNIT No. 3

ASH POND PIEZOMETERS
GROUNDWATER ELEVATIONS

EBASCO SERVICES INCORPORATED

DESIGNED	APPROVED
DR. E.C.H.	DATE 5-26-77
CH. D. TEMENS	APS-2742
CH.	SK-CH-7H



LEGEND:

- T HIGHEST VALUE
- MEAN VALUE
- L LOWEST VALUE
- P - PASSING TESTS
- F - FAILING TESTS (DISCONTINUED AFTER 10% OF TESTS FAILED ABOVE SPECIFIED)
- SPEC "BOUNDARIES"
- [ ] % COMPACTION VALUES BASED ON STANDARD PROCTOR
- T - 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 2742-54-09a. IT WILL BE UPDATED ON A WEEKLY BASIS AND DISTRIBUTED IN THE SITE SOILS ENGINEERS MONTHLY REPORT.

- TOP VALUE IN TEST COLUMNS INDICATE PASSING TEST DATA ONLY. THE NUMBER OF FAILING TESTS ARE INDICATED FOR INFORMATION ONLY AND ARE NOT PLOTTED ANYWHERE ON THIS SHEET.
- HIGH AND LOW VALUES SHOWN OUTSIDE SPECIFICATION LIMITS REFER TO ALLOWABLE SPEC. TOLERANCES.

COMPLETION REPORT

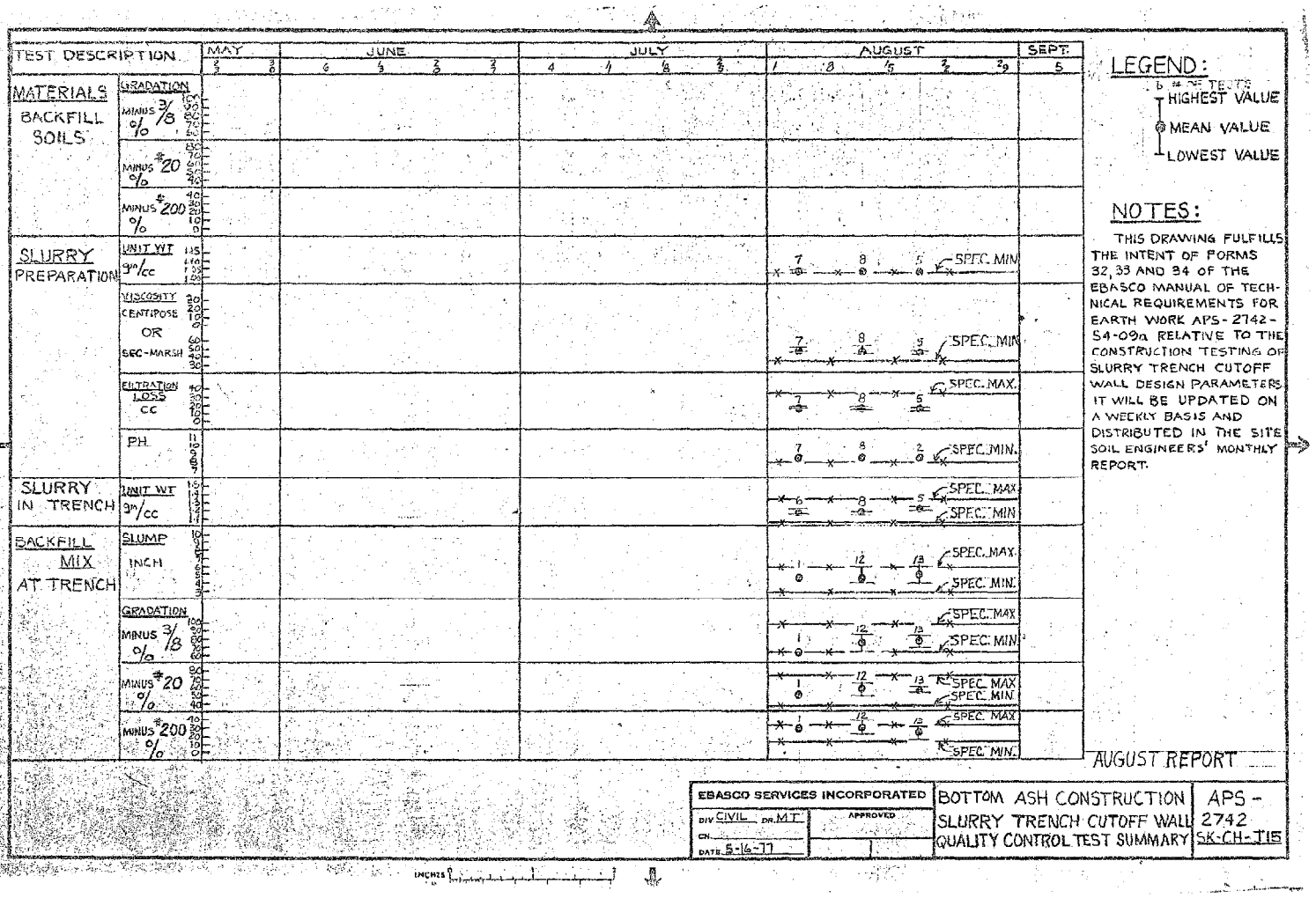
NO.	DATE	REVISION	BY	CH.	APPROVED
0	4-21-77	FOR COMMENTS	MT		

ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1976-750 MW INSTALLATION-UNIT NO.4
1977-250 MW EXTENSION-UNIT NO. 5

BOTTOM ASH POND CONSTRUCTION QUALITY CONTROL TEST SUMMARY

EBASCO SERVICES INCORPORATED

DRAWN BY	APPROVED	DATE 4-21-77
DR. G.H.		APS-2742
DES. M. TENNANT		SK-CH-312
CHK.		


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.
SPEC. MIN.
SPEC. MAX
SPEC. MIN
LEGEND:
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
BOTTOM ASH CONSTRUCTION
SLURRY TRENCH CUTOFF WALL
QUALITY CONTROL TEST SUMMARY
APS - 2742
SK-CH-715
INCHES

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

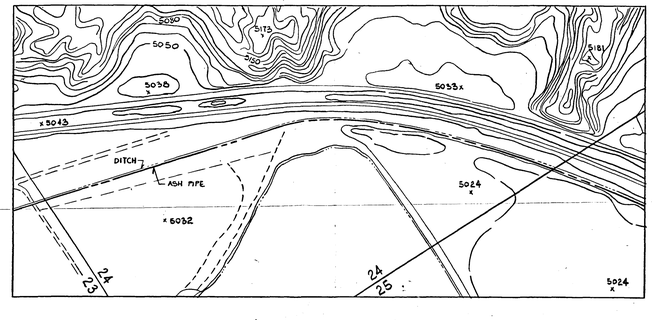
COAL PILE

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
DR D.KOVACHEV
CH.
APS-2742
SK-CH-160

APS DWG. G44254 P2

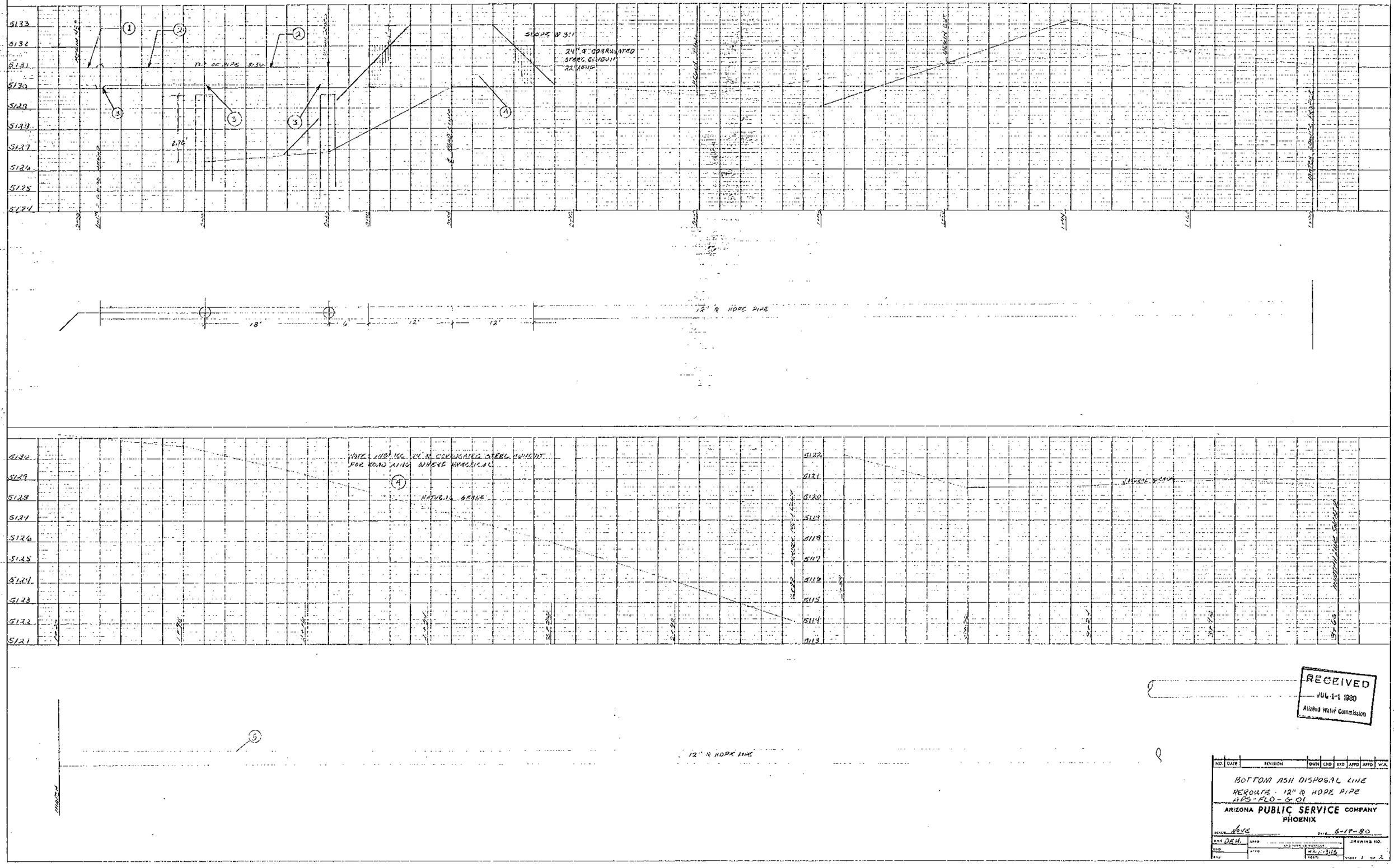
NO.	DATE	REVISION	BY	CH.	APPROVED

ENGINEERING DEPT. AUG 6 1990

09.27
09.28

REROUTE 12” DIAMETER HDPE PIPE DRAWINGS

(APS, 1980)



NO	DATE	REVISION	DWN	CHD	EXD	APPD	APPD	W.A.

BOTTOM ASH DISPOSAL LINE
REROUTE - 12" Ø HDPE PIPE
APS-FLO-G.01

ARIZONA PUBLIC SERVICE COMPANY
PHOENIX

SCALE NONE DATE 6-19-80
DWN DRH APPD
CHD APPD
EXD
APPD
DRAWING NO.
W.A. 3115
SHEET 1 OF 2

9.27

EXISTING ACCESS

NOTE: SLOPE MAY CHANGE DEPENDING UPON CONDITIONS OF GROUND UPON EXCAVATION

NOTE: ALL MATERIAL FOR SPILLWAY SHALL BE PLACED ON NATURAL GROUND

Bill of Materials

Item	Amount	Description
1	2	6" Short radius "Durite" 90°
2	1	Pipe joiner, "Durite" 18" x 12" ø
3	1	Mission Coupling
4	1	Durite 12" ø Reducer
5	100'	Pipe, DURITE, 12" ø Low Pres. SWC
6	8	U-bolts 5/8" ø Min thread 11" ø
7	8	2 hole nuts 5/8" ø x 2½" thread
8	17	5/8" ø x 11" Round Steel
9	2	5/8" ø Washer
10	8	6" x 6" x ¼" Steel Plate
11	15	Channel 2'-1" x 10" x ½"
12	8	Plates, 1' x 1' x ½" Thick Steel Plate
13	16'	Durite pipe (Scrap)
14	12	½" ø Rebar

KEY PLAN

U.S. BASELINE

MIR

P203

CAISSON PIPE SUPPORT DETAIL

SECTION D-D

5/8" ø U-BOLTS

SECTION C-C

CONCRETE PAD 2'-3"x3'-1"

SPILLWAY

SECTION A-A

RIPRAP W/ MINIMUM 6" ACCEPTABLE SLURRY

STA 3+33 APPROX. FINISH GRADE

SECTION B-B

CAISSON SUPPORT DETAIL

TOP CAISSON @ STA 0+00 = 5127.15

GROUND LEVEL 5120.15'

DURITE PIPE MUST BE USED FOR CAISSON SLEEVING

TOP OF CAISSON STA 0+36 = 5129.15

GROUND LEVEL 5126.20

DETAIL 1 SPILLWAY PLAN

STA 3+33

LEVEL BOTTOM OF DITCH

SECTION B-B

DISCHARGE PAD DETAIL

5/8" ø U-BOLTS

FIG 15" ø U-BOLTS TO CONCRETE PRIOR TO POUR

SADDLE PIPE, 15" IN CONCRETE

RIPRAP W/ SLURRY

RECEIVED
JUL 11 1980
Arizona Water Commission

NO.	DATE	REVISION	DWN	CHD	ENG	AE	PD	APPD	W.A.
		BOTTOM ASH DISPOSAL LINE REROUTE 12" ø HDPE PIPE							

APS-PLD-G-01

ARIZONA PUBLIC SERVICE COMPANY
PHOENIX

SCALE: None DATE: 6-19-80

DRAWING NO.

SHEET 2 OF 2

Rerouting of Slurry Disch. Line Bottom Ash Dam (9-27)

PROFILE — NOTE BOOK — No.

PLAN — NOTE BOOK — No.

9-27

CREST RESTORATION AS-BUILT

(Ebasco, 1990)

STATE OF ARIZONA
DEPARTMENT OF WATER RESOURCES
SAFETY OF DAMS SECTION
THIS DRAWING IS APPROVED FOR CONSTRUCTION
DEPUTY DIRECTOR, ENGINEERING
APPLICATION NO. 09-27
DATE OCT 16 1981

TRUE NORTH N

KEY PLAN

DETAIL 1

PROFILE ℄ OF DAM EXTENSION

DIKE ACCESS ROAD

PLAN
1" = 200'

PROFILE - ALONG ℄ OF DIKE (LOOKING UPSTREAM)
VERT: 1" = 20'
HOR: 1" = 200'

SECT A

SECT B

SECT C

SECT D

SECT E

SECT F

SECT G

SECT H

SECT J

SECT K

SECT M

SECT

SECT B

NOT RELEASED

QUANTITIES (NET, BY FIELD UNLESS NOTED)

NOTES

LEGEND

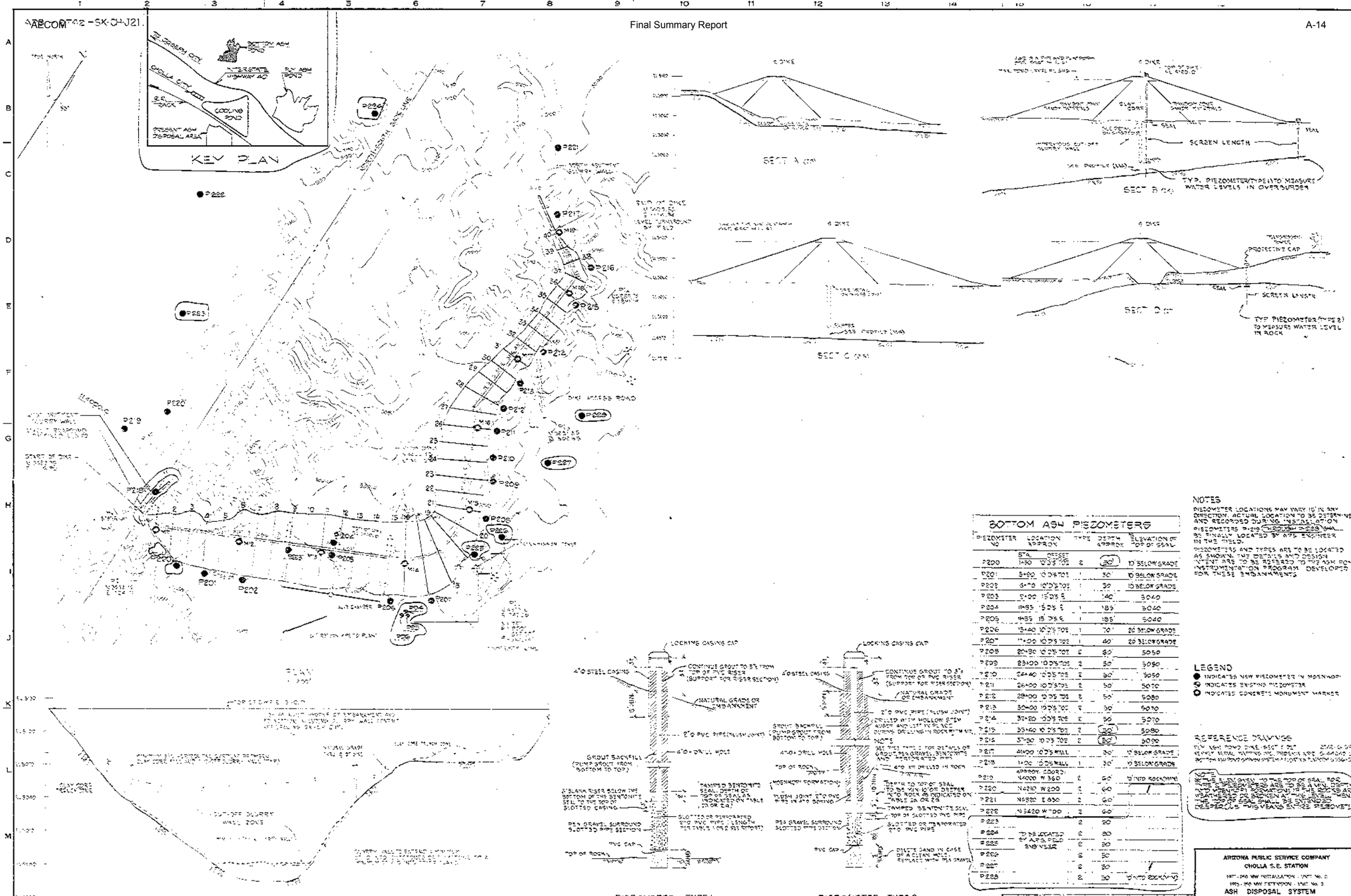
REFERENCE DRAWINGS

WORK THIS DRAWING WITH 2742-G-559

ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.F. STATION
ASH DISPOSAL SYSTEM
BOTTOM ASH POND - PLAN & SECT
EBASCO SERVICES INCORPORATED

APPROVED
DATE AUG. 15
APS-2742
G-556

APS DWG. #G-44555 09-27
"FILE COPY"



BOTTOM ASH PIEZOMETERS

PIEZOMETER NO	LOCATION APPROX STA.	OFFSET	TYPE	DEPTH APPROX	ELEVATION OF TOP OF SEAL
P200	1+50	10'D 5'S	2	30'	10' BELOW GRADE
P201	5+00	10'D 5'S	1	30'	10' BELOW GRADE
P202	6+70	10'D 5'S	1	30'	10' BELOW GRADE
P203	9+00	15'D 5' S	1	140'	5040
P204	11+85	5'D 5'S	1	185'	5040
P205	11+85	15' D 5' S	1	185'	5040
P206	15+40	10'D 5'S	1	70'	20 BELOW GRADE
P207	17+00	10'D 5'S	1	40'	20 BELOW GRADE
P208	20+30	10'D 5'S	2	80'	5050
P209	23+00	10'D 5'S	2	50'	5050
P210	24+40	10'D 5'S	2	90'	5050
P211	26+00	10'D 5'S	2	50'	5070
P212	28+00	10'D 5'S	2	54'	5080
P213	30+00	10'D 5'S	2	30'	5070
P214	32+20	10'D 5'S	2	50	5070
P215	35+60	10'D 5'S	2	80'	5080
P216	37+50	10'D 5'S	2	80'	5080
P217	ALIGN	10'D S WALL	1	30'	10' BELOW GRADE
P218	1+00	10'D S WALL	1	30'	10' BELOW GRADE
P219	APPROX. LOCATED: N4000 W 360		2	90'	10' INTO ROCK(TYP)
P220	N4210 W 200		2	60	
P221	N5820 E 350		2	60'	
P222	N 5420 W 700		2	60	
P223	TO BE LOCATED BY A.P.S. FIELD ENGINEER		2	80	
P224			2	80	
P225			2	80	
P226			2	50	
P227			2	50	
P228			2	50	10' INTO ROCK(TYP)

NOTES

PIEZOMETER LOCATIONS MAY VARY 15' IN ANY DIRECTION. ACTUAL LOCATION TO BE DETERMINED AND RECORDED DURING INSTALLATION.

PIEZOMETERS P-219 THROUGH P-228 SHALL BE FINALLY LOCATED BY APS ENGINEER IN THE FIELD.

PIEZOMETERS AND TYPES ARE TO BE LOCATED AS SHOWN. THE DETAILS AND DESIGN INTENT ARE TO BE REFERRED TO THE ASH POND INSTRUMENTATION PROGRAM DEVELOPED FOR THESE EMBANKMENTS.

LEGEND

- INDICATES NEW PIEZOMETER IN MONUMENT
- INDICATES EXISTING PIEZOMETER
- INDICATES CONCRETE MONUMENT MARKER

REFERENCE DRAWINGS

NOTE

ALL ELEVATIONS TO THE TOP OF SEAL FOR WEATHERED BOULDERS ARE TO BE A MIN. DEPTH OF 10' BELOW ELEV. WHERE THE TOP OF SEAL SHALL BE EXTENDED DEEPER AS THIS MEANS ENTIRE PIEZOMETER

NOTES

GROUT BACKFILL - (1) CEMENT-WATER WITH POWDERED BENTONITE ADDED TO STABILIZE THE MIX, MAX 6% OF CEMENT. IMPORTANT: BENTONITE WITH WATER PRIOR TO MIXING. (2) USE MIN. PUMP GROUT INTO HOLE.

PEA GRAVEL: MAX. 3/4" SIZE, MIN. NO. 10 SIEVE SIZE.

BENTONITE SEAL: USE POWDERED BENTONITE OR MANUFACTURED BENTONITE BALLS.

SLOTTED PVC PIPE: 2"Ø SLOTTED, 48 SLOTS/INCHES OF.

ALTERNATE: IF PEA GRAVEL BRIDGES IN HOLE AND WILL NOT SEAT TO DESIRED LEVEL, THEN REPLACE WITH CLEAN SAND.

ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
1977 - 260 MW INSTALLATION - UNIT No. 2
1985 - 250 MW EXTENSION - UNIT No. 3
ASH DISPOSAL SYSTEM
BOTTOM ASH POND - INSTRUMENTATION

EBASCO SERVICES INCORPORATED

NO	DATE	REVISION	BY	CK.	APPROVED
4	2-7-79	ADDED P-219 THRU P-228			
3	11-29-78	ADDED (APS) NOTE			
2	11-20-78	REV. P-2 DETAIL & DETAILS			
1	9-13-78	REVISED DWG. FORMAT ADDED PIEZOMETER DETAILS			

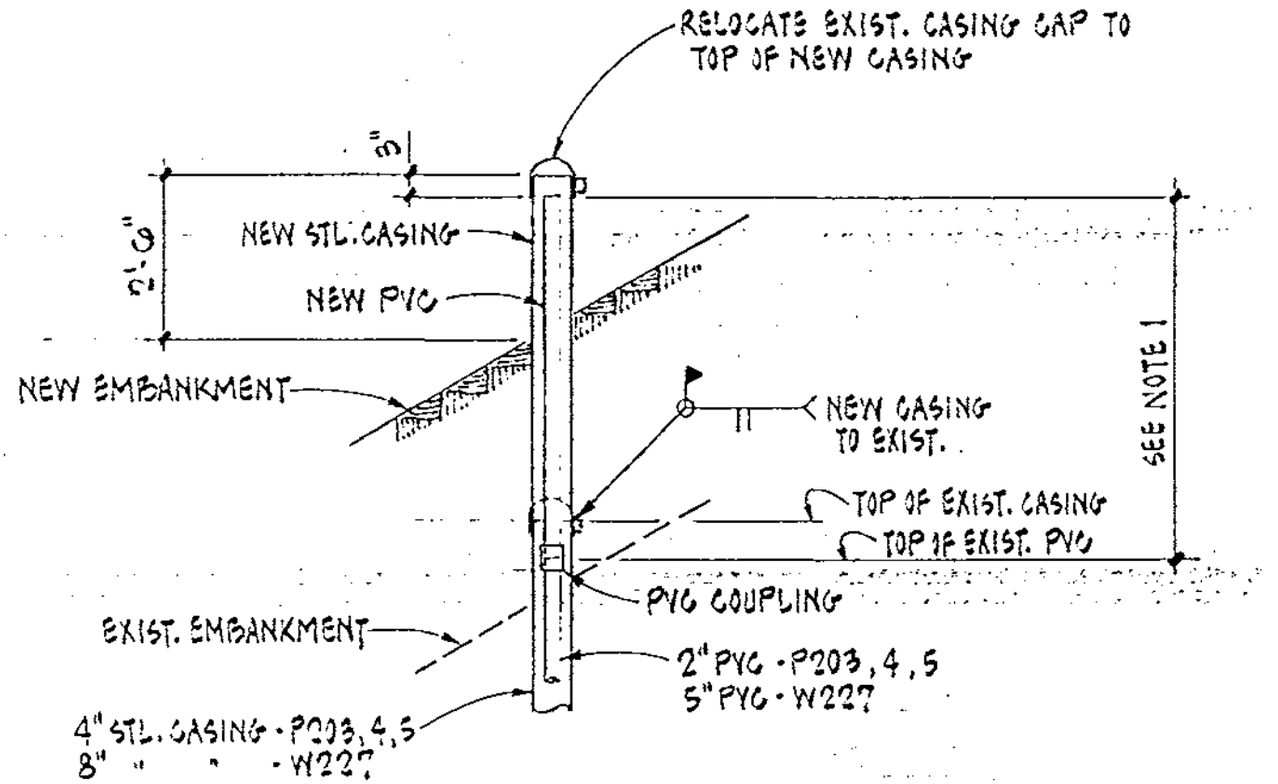
APPROVED | DATE
APS-2742
APS DWG. SKCHJ21

09-27

NOTES:

1. DOCUMENT DISTANCE FROM TOP OF EXIST. PVC TO TOP OF NEW PVC AT EACH LOCATION.
2. NEW CASING & PVC SIZE & MATERIAL TO MATCH EXIST.

"FILE COPY"



STATE OF ARIZONA
DEPARTMENT OF WATER RESOURCES
SAFETY OF DAMS SECTION
THIS DRAWING IS APPROVED FOR CONSTRUCTION

DEPUTY DIRECTOR, ENGINEERING

APPLICATION NO. 09-27

DATE OCT 11 2001

Registered Professional Engineer — Certificate No. 15294 — Marta M. [illegible] — 10/15/01 — Arizona, U.S.A.

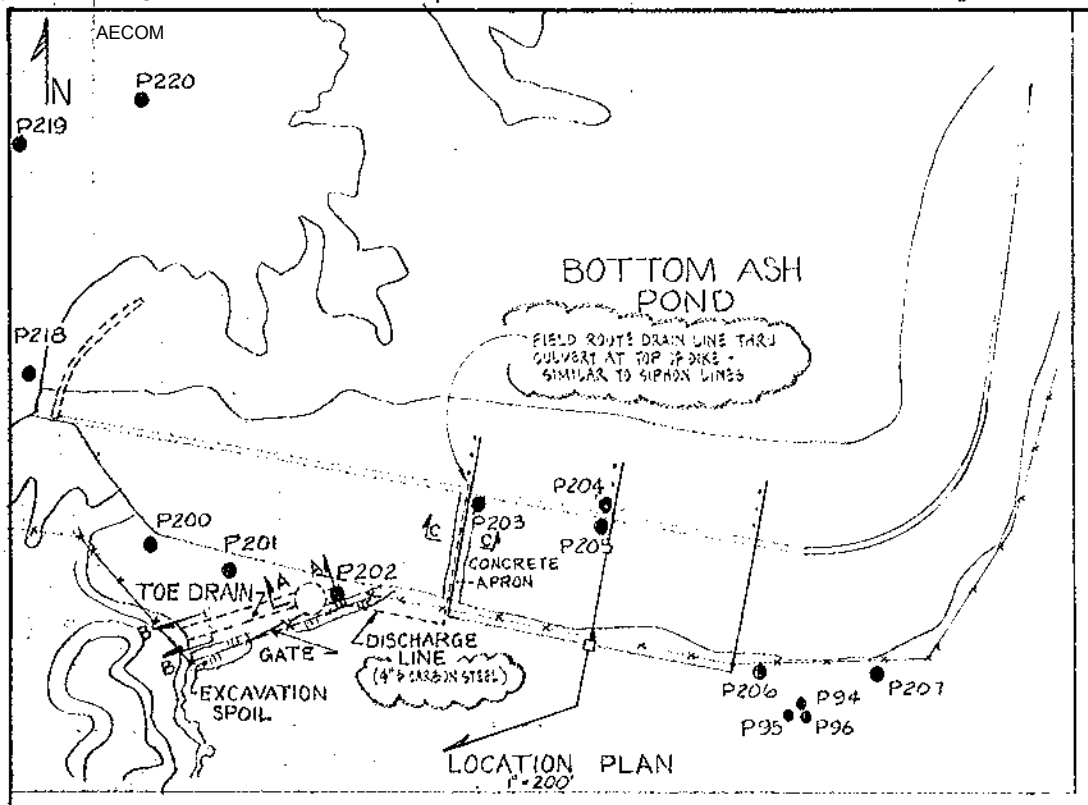
CHOLLA S.E.S. B.A. POND
PIEZOMETER & WELL
EXTENSION DETAILS

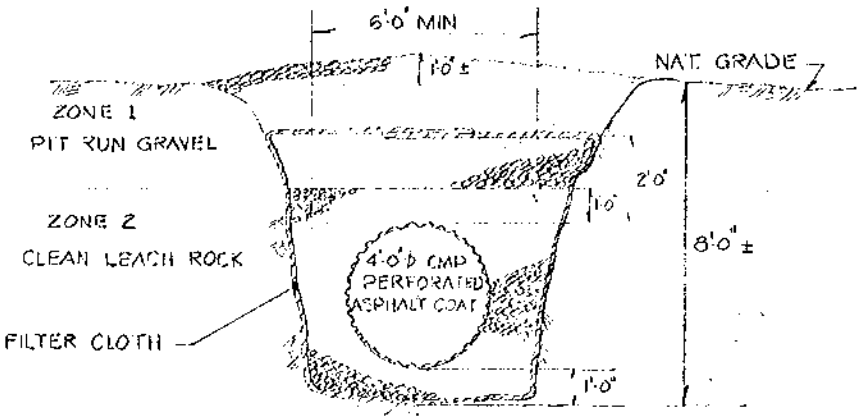
DWN: TRH CHD: DDM APPD: MMS

AKA-T009 TRH

TOE DRAIN SYSTEM

(APS, Rev. 1991)





SECTION B-B
3/8" = 1'-0"

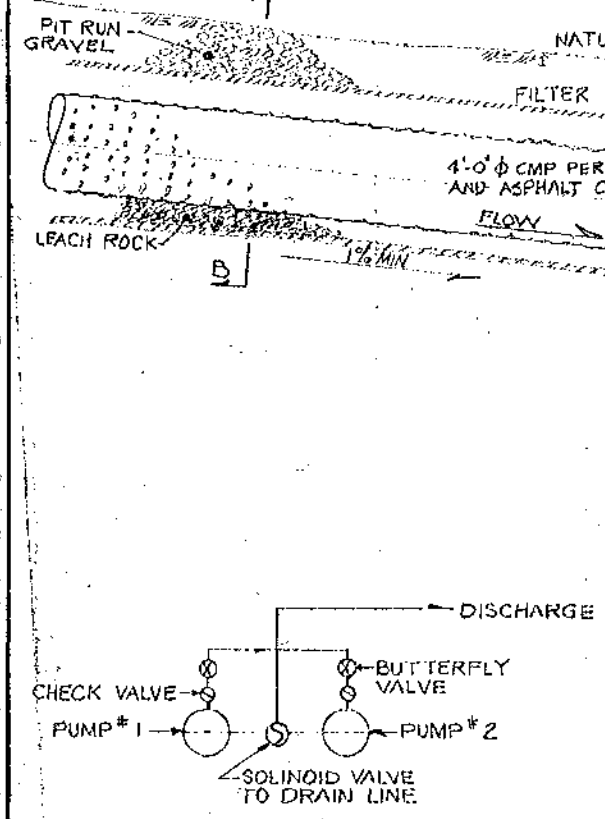
NOTES:

TOE DRAIN SYSTEM TO BE CONSTRUCTED IN ACCORDANCE WITH THE DETAILS OF THIS DWG AND APS SPECIFICATION-CHOLLA PROJECT ENGINEERING - CPE NO. 1, TOE DRAIN INSTALLATION

EXCAVATION DIMENSIONS MAY VARY WITH THE ENGINEER'S APPROVAL.

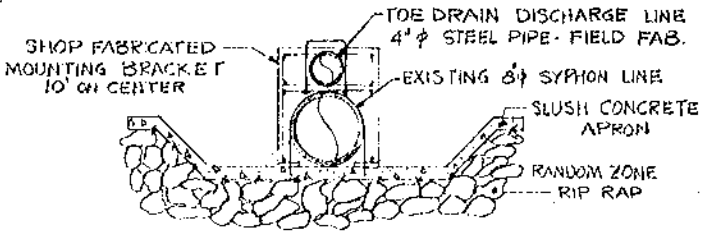
EXACT DRAIN LOCATION BY FIELD

NO EXCAVATION WITHIN 20 FOOT OF THE EMBANKMENT TOE.



SECTION A-A
1/4" = 1'-0"

PUMP HEADER LOGIC



SECTION C-C
1/2" = 1'-0"

REFERENCE DWGS:
B.A. POND PLANS & SIPHON SYSTEM ---- 6-44556

NO	DATE	REVISION	DWN	CHD	ENG	APPD	APPD	M.A.
2	2/26/71	ADD NOTE & REF DWG.	TRM	WPP	M.T.	2/26	2/26	
1	6/22/70	CHANGED DWG. # FROM D-CPE-1 TO D-62671	DJG	KMH	MT			

ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
BOTTOM ASH DAM
TOE DRAIN SYSTEM

ARIZONA PUBLIC SERVICE COMPANY
PHOENIX

SCALE AS SHOWN DATE 10-10-79

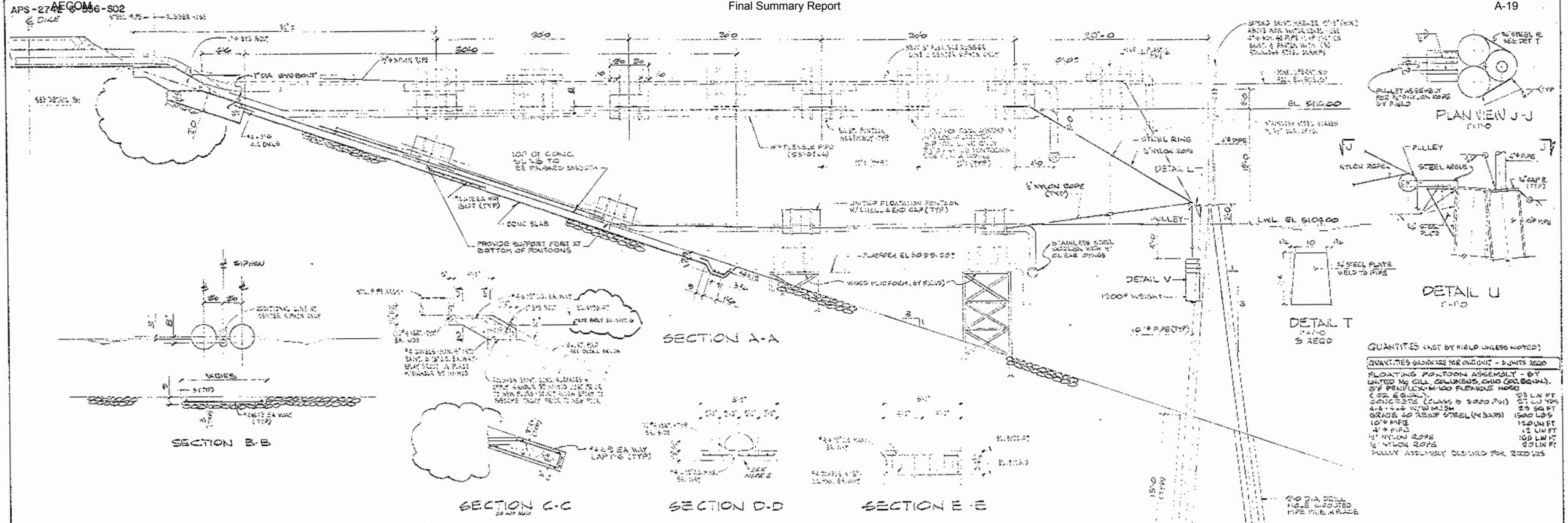
DWN M.T. CHD B.F. ENG APPD ENGINEER OR MANAGER APPD M.A.

DRAWING NO. D-62671 SHEET 1 OF 1

0 9. 27

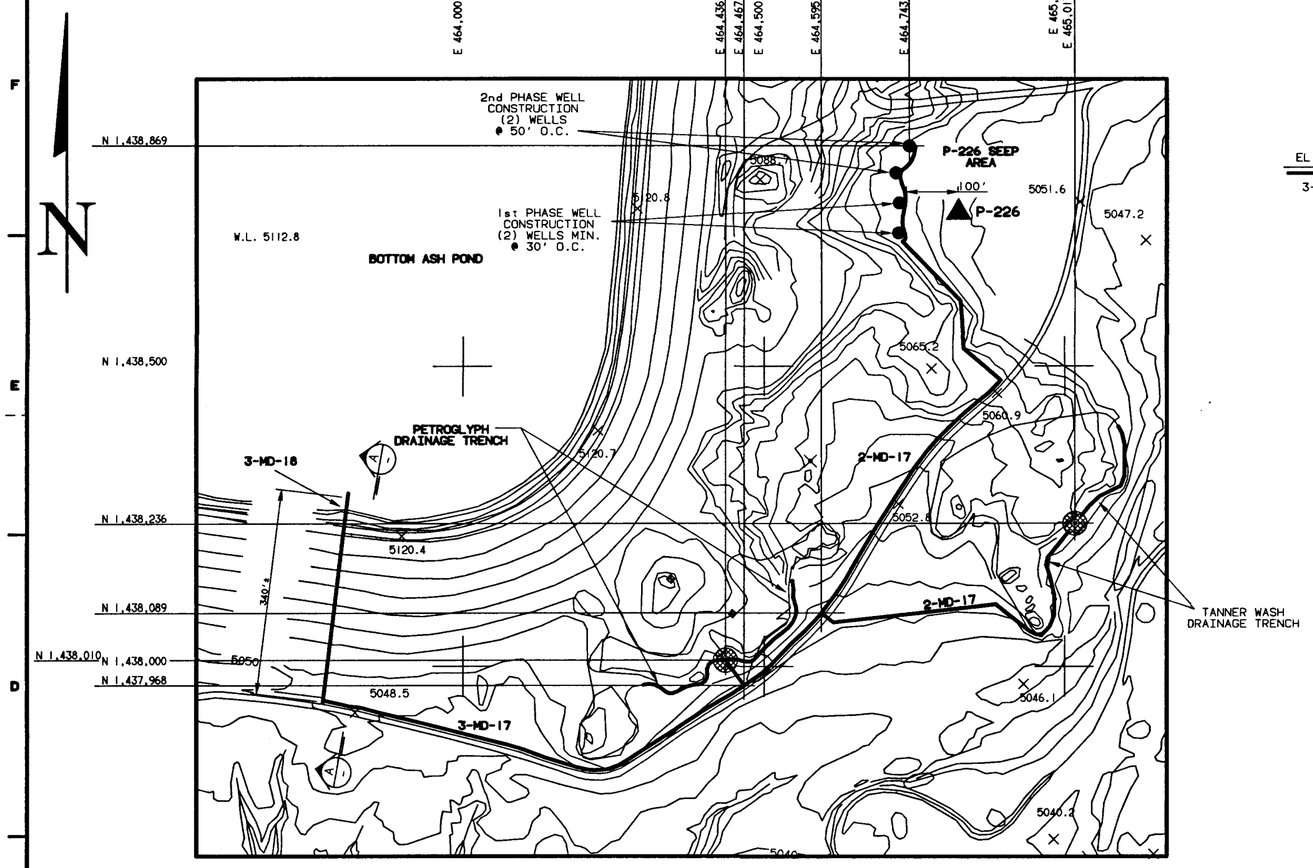
AS-BUILT SIPHON SYSTEM & FLOATING PIPE

(Ebasco, 1993)


SECTION A-A
SECTION B-B
SECTION C-C
SECTION D-D
SECTION E-E
PLAN VIEW J-J
DETAIL U
DETAIL T
DETAIL V
DETAIL L
QUANTITIES (NET BY FIELD UNLESS NOTED)
QUANTITIES SHOWN ARE FOR ONE UNIT - 2 UNITS REQD
FLOATING PONTOON ASSEMBLY - BY UNITED MC GILL, COLUMBUS, OHIO (SCALE 1/4"=1'-0")
BY PENFLEX-M-100 FLEXIBLE HOSE (SR. ENGR.)
CONCRETE (CLASS B 3000 PSI) 27 CU YDS
4"x4"x#4 W/W MESH 25 SQ FT
GRADE 60 REINF STEEL (#4 BARS) 1500 LB'S
10"ø PIPE 150 LIN FT
4"ø PIPE 12 LIN FT
1/4" NYLON ROPE 100 LIN FT
1/2" NYLON ROPE 40 LIN FT
PULLEY ASSEMBLY DESIGNED FOR 2500 LBS
NOTES
1. ALL EXPOSED METAL TO BE COATED WITH ENGARD 465 COAL TAR EPOXY
2. ALL PILES TO BE AUGERED & CONCRETED IN PLACE TO ELEV AS NOTED & TO DEVELOPE 17ton PER PILE LATERAL LOAD CAPACITY
3. FOR LOCATION OF P.I. SEE G-556
4. MINIMUM SIPHON OPERATING LEVEL EL 510.0
5. ALL STRUCTURAL CONCRETE SHALL CONFORM TO APS STD. SPECIFICATION NO. C-SCH-000010 AND SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 3000 PSI AT 28 DAYS
6. ALL REINFORCING STEEL SHALL CONFORM TO ASTM A 615, GRADE 60
7. LEAN CONCRETE FOR CULVERT BEDDING SHALL BE HIGH SLUMP (6") MASS CONCRETE WITH A CEMENT CONTENT OF 282 LB (3 SACK) PER CUBIC YARD AND 3" MAXIMUM AGGREGATE
REFERENCE DRAWINGS
BOT ASH POND - PLAN & SECT APS-2742-G556
YARD PIPING - 16" WATER LINE - SH 6 APS-2742-G495
TILE DRAIN SYSTEM D-8007
ADDITIONAL SUPPORTING BUILDING (IF NEEDED) A-1103
SECTION F-F
SECTION G-G
SECTION H-H
DETAIL W
DETAIL V
DETAIL S
SECTION G
PLAN
END VIEW
PONTOON DETAILS
NO SCALE
AS BUILT
ARIZONA PUBLIC SERVICE COMPANY
CHOLLA S.E. STATION
#27-750 MW INSTALLATION - UNIT No. 2
1976-250 MW EXTENSION - UNIT No. 3
ASH DISPOSAL SYSTEM
BOTTOM ASH POND
SIPHON SYSTEM & FLOATING PIPELINE
EBASCO SERVICES INCORPORATED
APPROVED
DATE 11-12-77
APS-2742
G-556-S02
#G-44556 sht2
09-27

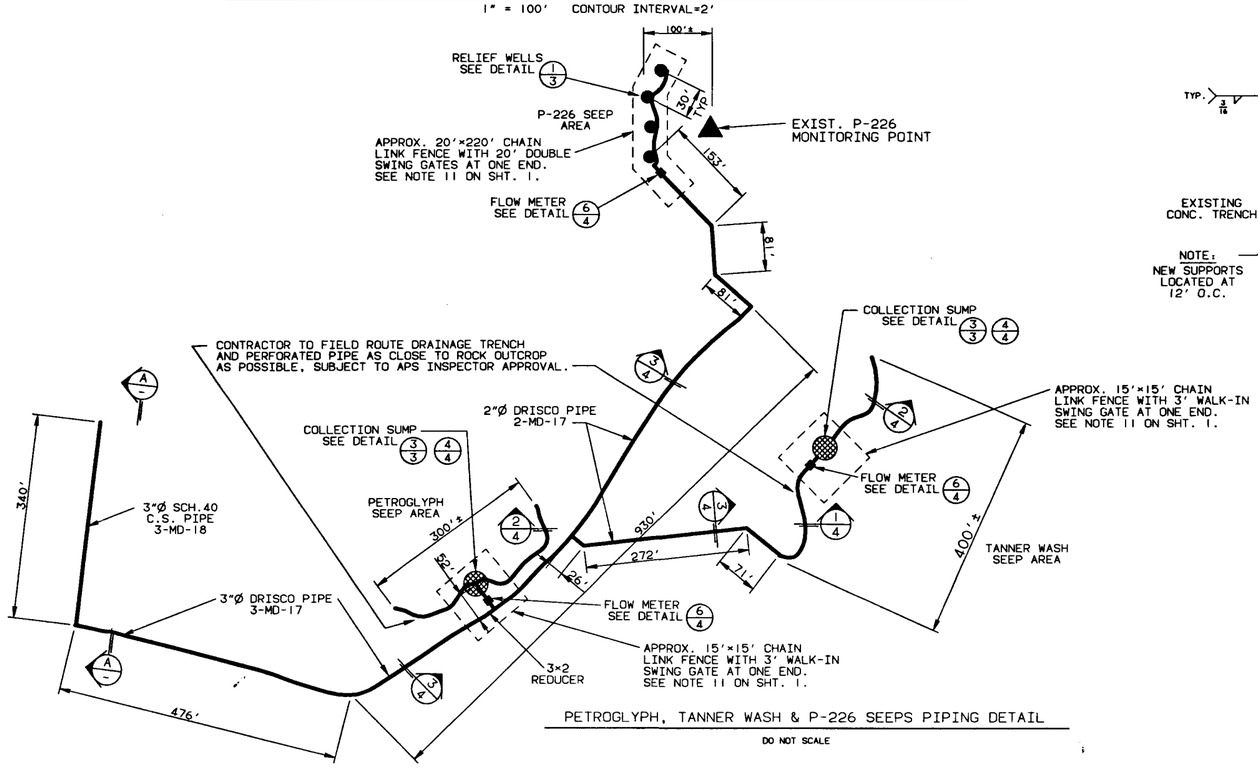
SEEPAGE INTERCEPT SYSTEM DRAWINGS

(APS, 1993)



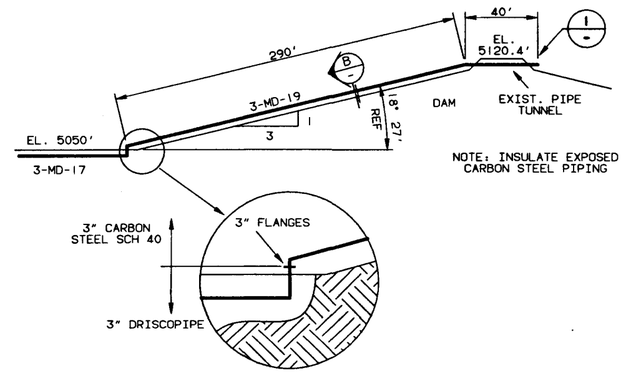
PARTIAL PLAN - BOTTOM ASH POND AREA - PETROGLYPH, TANNER WASH & P-226 SEEPS

1" = 100' CONTOUR INTERVAL=2'



PETROGLYPH, TANNER WASH & P-226 SEEPS PIPING DETAIL

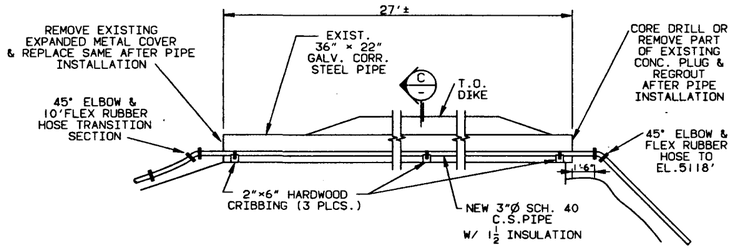
DO NOT SCALE



SECTION A/- A/-

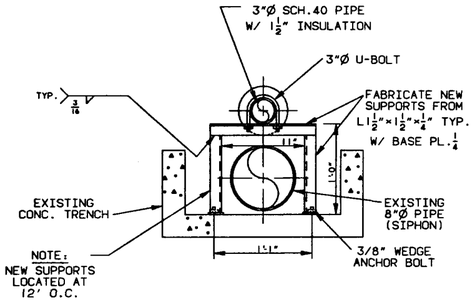
1" = 50'

NOTES:
1. ● INDICATES LOCATION OF DEWATERING WELL.
2. ▨ INDICATES LOCATION OF DEWATERING SUMP.



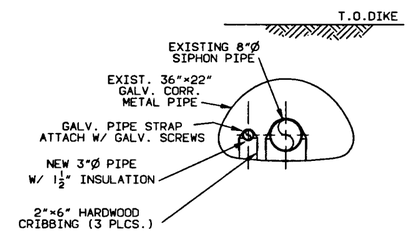
DETAIL 1/-

¼"=1'0"



SECTION B/-

1½"=1'0"



SECTION C/-

¾"=1'0"

NO	DATE	REVISION	DWN	CHK	ENG	SYSD	APVD	W A
W A NO								
PRELIM ENGRG								
CONSTR REVIEW								
FOR BID ONLY								
APVD FOR CONSTR								
AS BUILT								
REF ONLY								

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

ARIZONA PUBLIC SERVICE COMPANY

SCALE NOTED DATE 03-03-93

DRWN AGS/RP
SURVEYS TRH
REVISIONS M SAJS

APPROVED
ENGINEERING SUPERVISOR

DRAWING NO. G-114438

REVISIONS

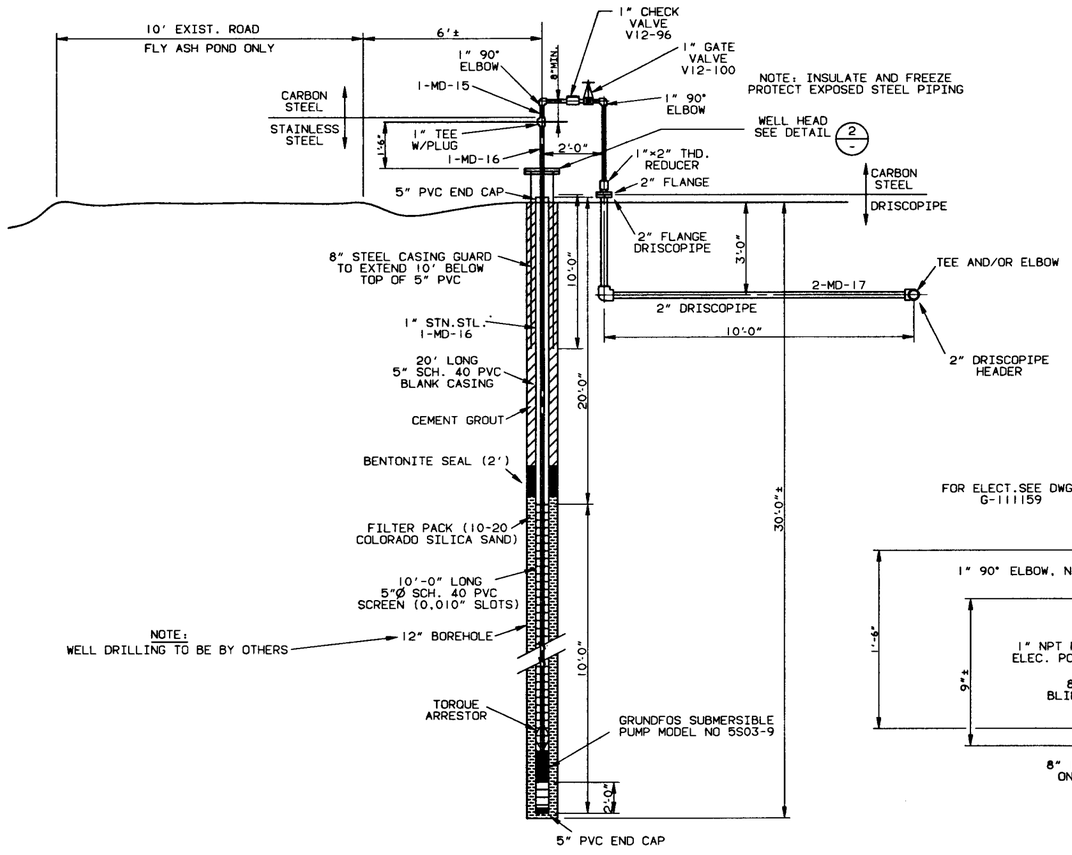
W.O. 62-9072

SHEET 2 OF 4

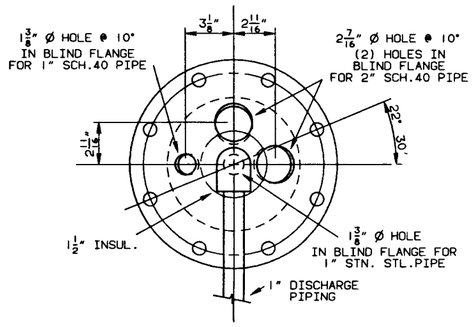
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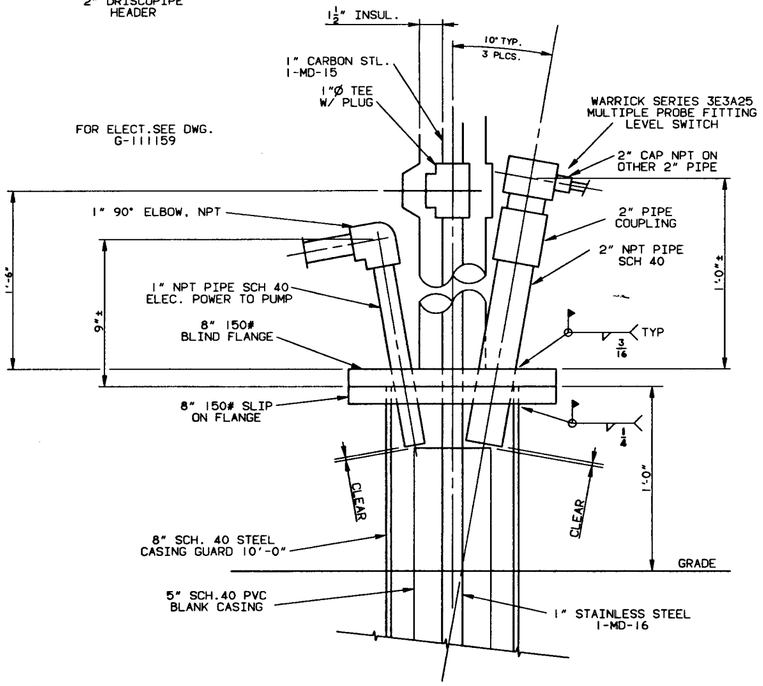
-- PLOT DATE: 04/09/93 -- GROUP: GENE -- SUBGROUP: PRATT -- DWG ID: 3F JG 114438 0002

09.27
09.28

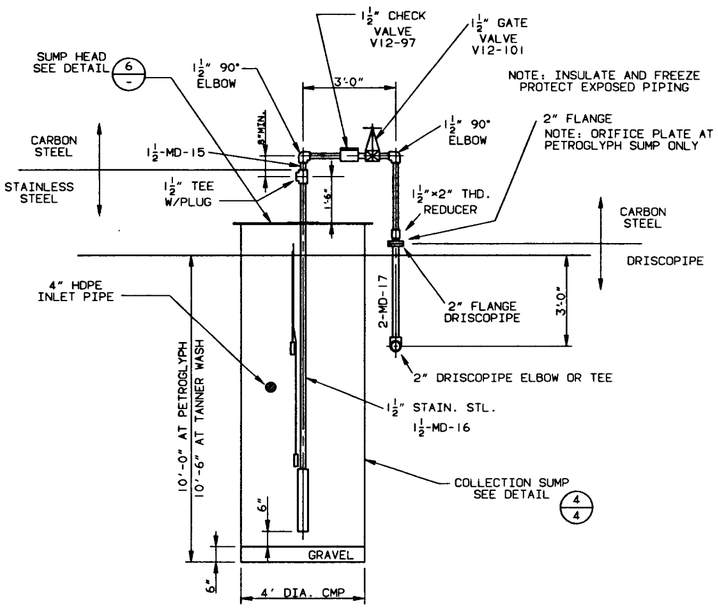


DETAIL 1 — TYPICAL WELL INSTALLATION
1/2"=1'-0"

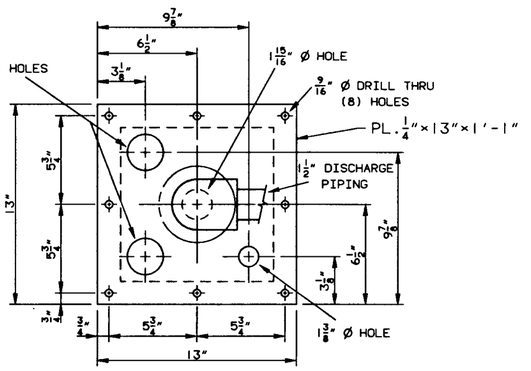




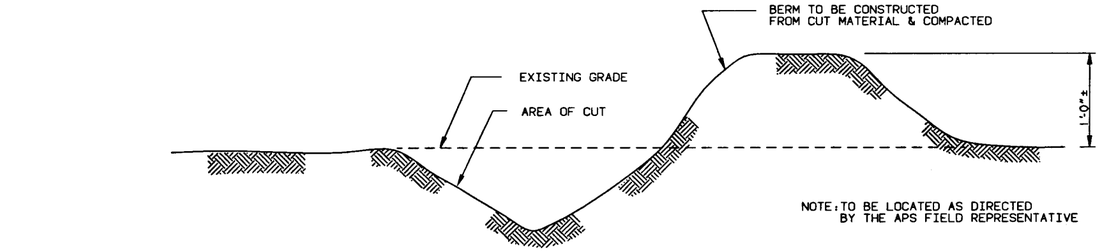
DETAIL 2 — TYP. WELL HEAD
3"=1'-0"



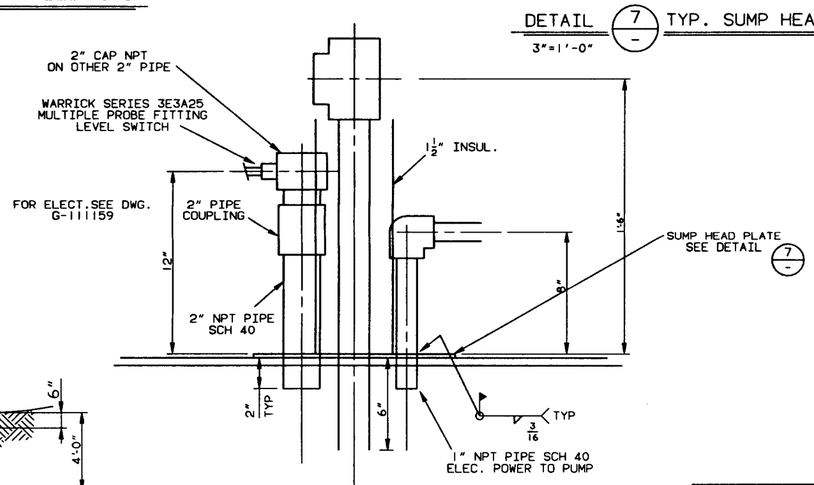
DETAIL 3 — TYPICAL SUMP INSTALLATION
1/2"=1'-0"



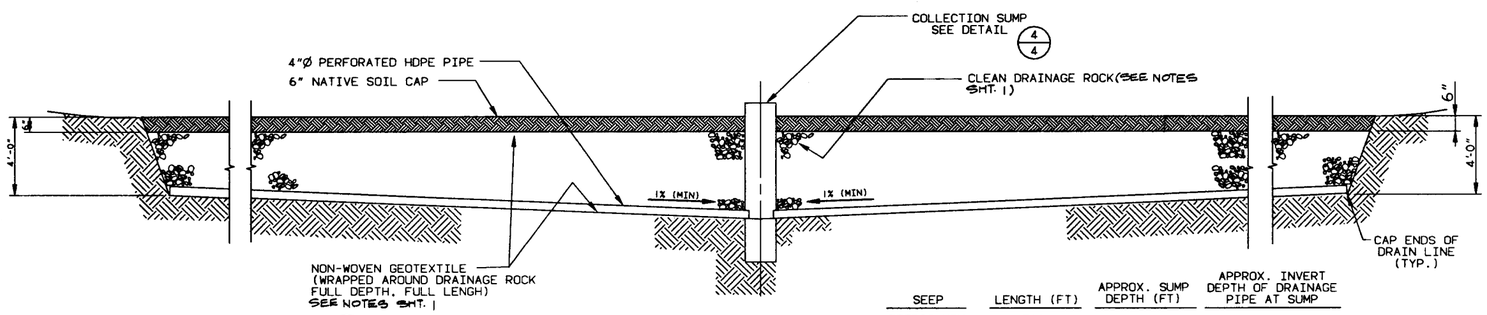
DETAIL 7 — TYP. SUMP HEAD PLATE
3"=1'-0"



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	B A

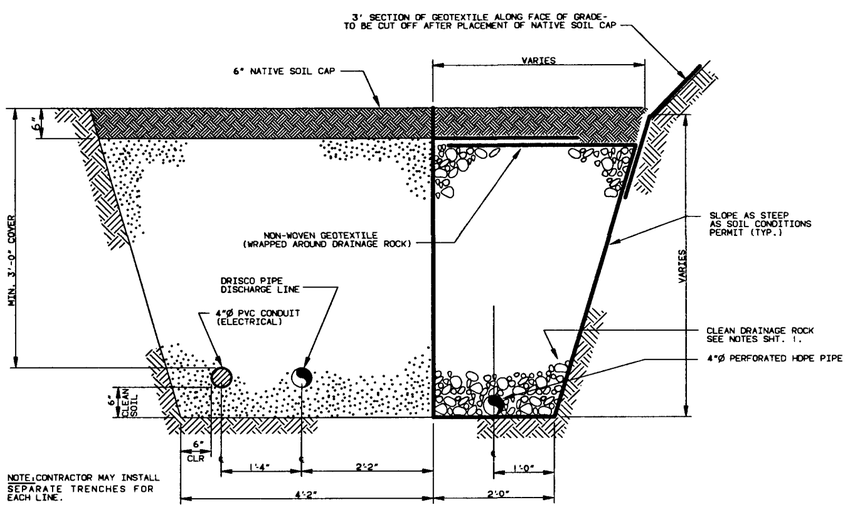
CHOLLA S.E.S. COMMON
BA/FA PONDS SEEPAGE INTERCEPT SYSTEM
SECTIONS & DETAILS

ARIZONA *PUBLIC SERVICE* COMPANY

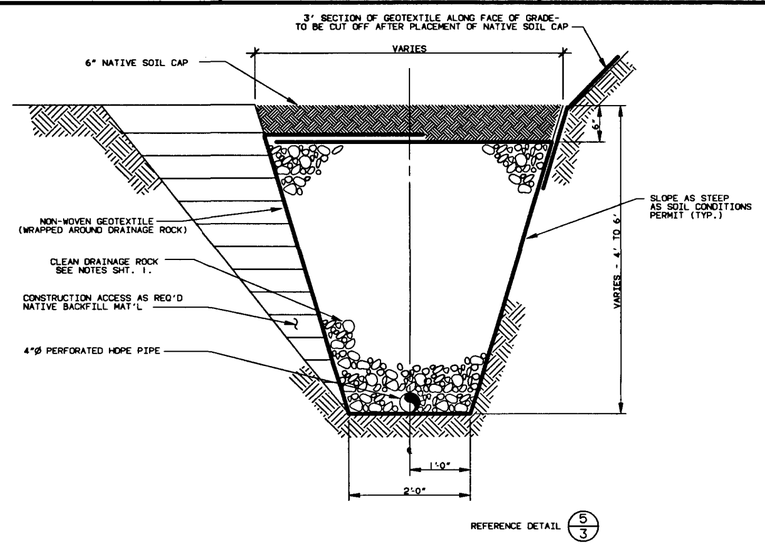
SCALE NOTED DATE 03-30-93
DESIGN TRH
DRAWN PRATT
CHECKED
APPROVED
ENGINEERING MANAGER
REVIEWED
DRAWING NO. G-114438
SHEET 3 OF 4

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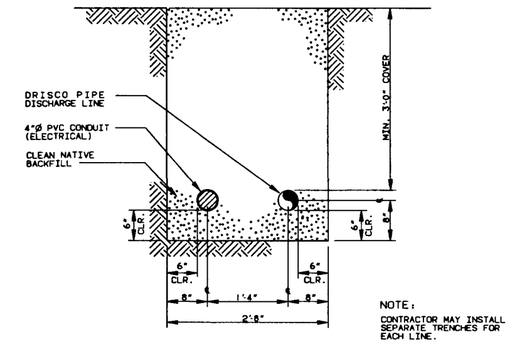
GROUP; GENE -- SUBGROUP; PRATT -- DWG ID; JF JG 114438 0003 -- PLOT DATE; 04/06/93



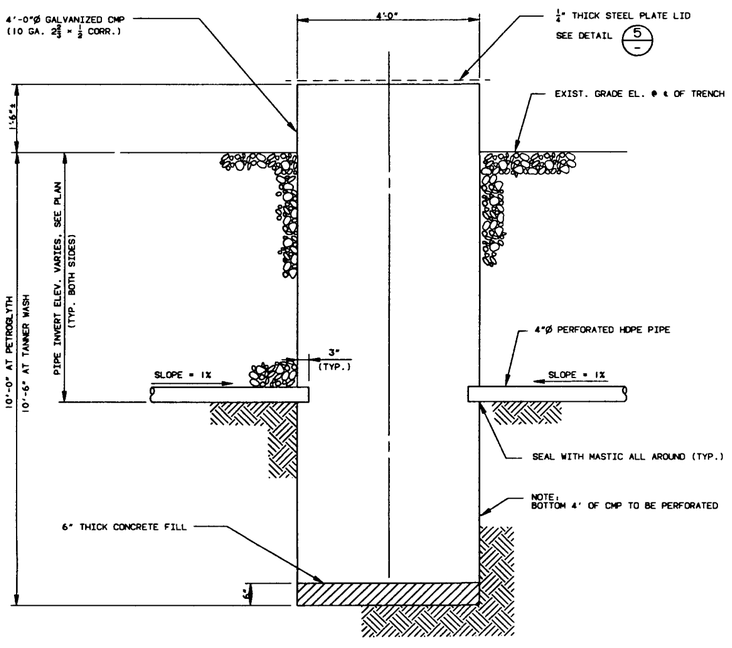
DETAIL 1/- 1"=1'-0" TYP. JOINT TRENCH



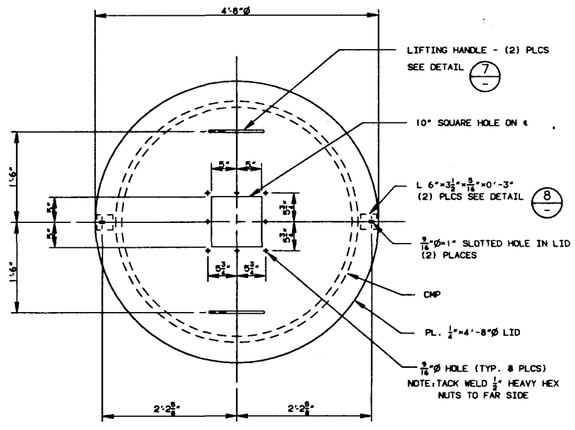
DETAIL 2/- 1"=1'-0" TYPICAL INTERCEPT TRENCH



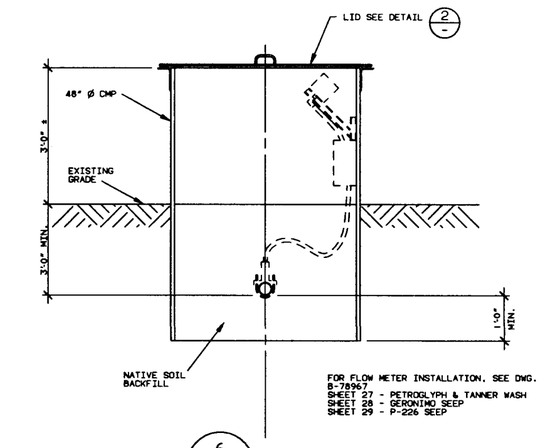
DETAIL 3/- 1"=1'-0" TYPICAL UTILITY TRENCH



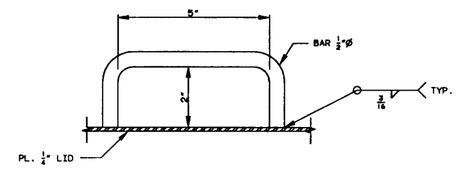
DETAIL 4/- ¾"=1'-0" TYPICAL COLLECTION SUMP



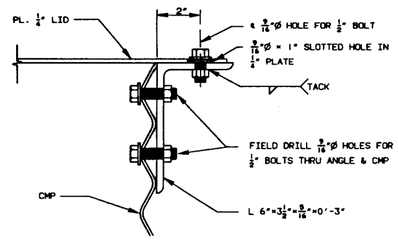
DETAIL 5/- 1"=1'-0" TYPICAL SUMP LID



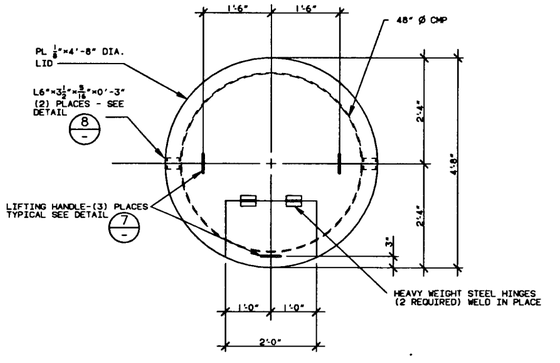
DETAIL 6/- ¾"=1'-0" TYP. FLOW METER ENCLOSURE



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 | DRAWING NO. G-114438

DRAWN TRH | CHECKED | ENGINEERING MANAGER

DRAWING M. Santos | REVIEWED Mike Ivon | ext. 62-9072 | SHEET 4 OF 4

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

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: Bottom 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

TABLE OF CONTENTS

1 Introduction 2

2 Analysis Criteria 2

3 Analysis Inputs 2

4 Assumptions 3

5 Stability Analysis 3

5.1 Critical Stability Cross Sections 4

5.2 Material Properties 5

5.3 Embankment Pore Pressure Distribution 6

5.4 Embankment Loading Conditions 6

6 Analysis Results and Conclusions 13

7 References 14

8 Attachments 15

Figures

Figure 1 Slope Stability Cross Section and Piezometer Locations Along the BAP Dam
Figure 2 Table 1615.1.1Site Class Definitions (IBC, 2003)
Figure 3 Table 11.8-1 (NEHRP, 2009)
Figure 4 Variations of Peak Transverse Crest Acceleration v. Peak Transverse Base Acceleration Based on Holzer (1998)
Figure 5 Variation of "Maximum Acceleration Ratio" with Depth of Sliding Mass after Makdisi and Seed (1977)

Tables

Table 1 Material Properties for the BAP Dam Safety Factor Analyses
Table 2 Range of Plasticity Index and Fines Content Values for Site Materials
Table 3 Safety Factor Results for the BAP Dam

DESIGN CALCULATION				
Calculation Title: Bottom Ash Pond Safety Factor Assessment	**Project Title:** APS Cholla Structural Integrity Assessment	**Project No:** 60445840	**Date:** 4/13/16	**Page No:** Page 2 of 15

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 Bottom Ash Pond (BAP) Dam, ADWR Dam #09.27,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 BAP 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 BAP Dam and surrounding terrain (Cooper Aerial Surveys Co. 2014).
- Subsurface stratigraphies were developed from as-built cross section drawings of the BAP Dam (Ebasco 1990).
- 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 inspection report (APS 2016).

DESIGN CALCULATION				
Calculation Title: Bottom 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 BAP (ADWR 1998).

- The surcharge pool water level of the CCR Pond was developed based on estimated water level for the design probable maximum flood (PMF) of the BAP (D&M 1991).

- The seismic loading for the BAP dam 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:

- No bathymetry data was available for the BAP; therefore, the upstream slope of the BAP is assumed to be 3H:1V (horizontal:vertical) below the reservoir water level with negligible accumulation of bottom ash deposits. This slope angle is based on the as-built BAP Dam cross section drawings (Ebasco 1990).

- 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 BAP Dam or the immediate surrounding areas since the survey was conducted, except for additional accumulation of bottom ash within the impoundment.

- The divider dikes associated with the waste coal combustion waste storage cells, located upstream of the BAP Dam, are internal dividers within the CCR surface impoundment and are not relied upon to maintain containment of the CCR. Consequently, the stability of the divider dikes is not analyzed in this calculation.

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

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 BAP 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 two cross-sections along the BAP Dam:

BAP Dam Cross Sections

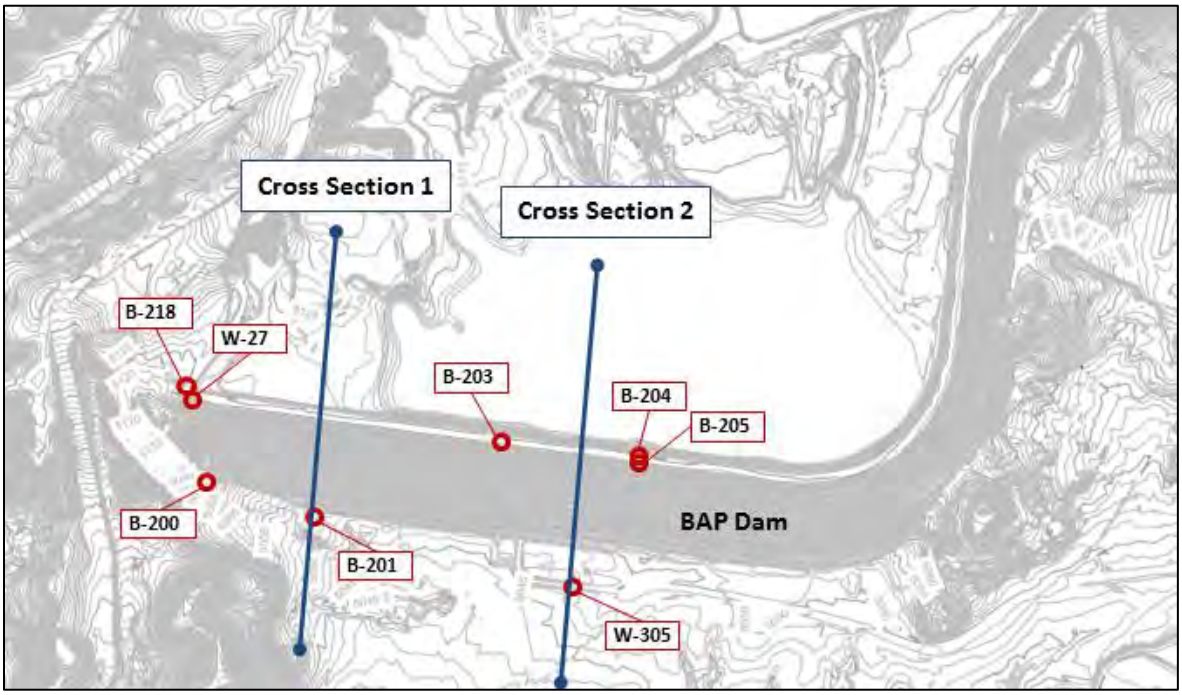


Figure 1. Slope Stability Cross Section and Piezometer Locations Along the BAP Dam

BAP Cross Section 1:

Cross Section 1 at the BAP was located near the left abutment of the dam, near piezometers W-227, B-200, and B-201. At this location, the dam intersects a rock outcropping forming the left abutment along its upstream slope. Consequently the upstream and downstream slope heights are considerably different at approximately 31 ft versus 73 ft, respectively although both slope angles are about 3H:1V. The dam at this cross section consists of a

sandy 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 approximately 10 ft 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. The upstream slope of the dam is confined by up to about 28 ft of hydraulically-placed bottom ash based on comparison between initial topographic surveys of the area (Ebasco 1975) and more recent surveys (Cooper Aerial Surveys 2014).

BAP Cross Section 2:

Cross Section 2 at the BAP was located near the center of the southern portion of the dam near piezometers W-305, B-203, B-204, and B-205. At this location, the dam is approximately 73 ft in height from EL 5,050 ft at the downstream toe to 5,123.3 ft at the crest; with upstream and downstream slope angles of about 3H:1V. Similar to Cross Section 1, the dam consists of a sandy lean clay core with an outer clayey sand shell. At this location the depth to bedrock is greatest along the dam at approximately 85 ft bgs. A cement-bentonite cutoff wall extends from the clay core of the dam to a layer of dense clay at about 15 ft above the bedrock and is used to control seepage beneath the dam. Water from the upstream coal combustion waste storage cells drains into a reservoir that lies upstream of the dam.

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 BAP 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 BAP 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	-
Bottom Ash	75	75	0	25	-	-

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 BAP Dam analysis, the pore pressure distribution was assigned using water level readings obtained from piezometers located near the stability cross sections (APS 2016). This distribution was adjusted based on engineering judgement to correspond with pond water level under steady-state, maximum storage pool conditions (ADWR 1998), and pond water level under maximum surcharge pool conditions (D&M 1991). The piezometers used to estimate the pore water pressure within the dam cross sections are shown in Figures 1.

The BAP (upstream) water level under maximum storage pool condition was based on the permitted water level of the ponds as stated in the ADWR operating license for the dam. Since the dam has no outlet work structures and rely on pumping rate from plant, seepage, evaporation, and a siphon system to control water levels, the maximum storage pool was set at the maximum permitted water levels. For the BAP this is EL 5,117.8 ft (ADWR 1998). The surcharge pool level is based on the expected water level raise during the design PMF and is EL 5,119.3 ft for the BAP (D&M 1991).

5.4 Embankment Loading Conditions

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

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

These loading conditions are described below.

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

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

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For this analysis, the long-term, maximum storage pool in the BAP was set at EL 5,117.8 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 BAP was set at EL 5,119.3 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

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.

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TABLE 1615.1.1
SITE CLASS DEFINITIONS

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

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

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

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)

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Table 11.8-1 Site Coefficient F_{PGA}

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

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

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

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

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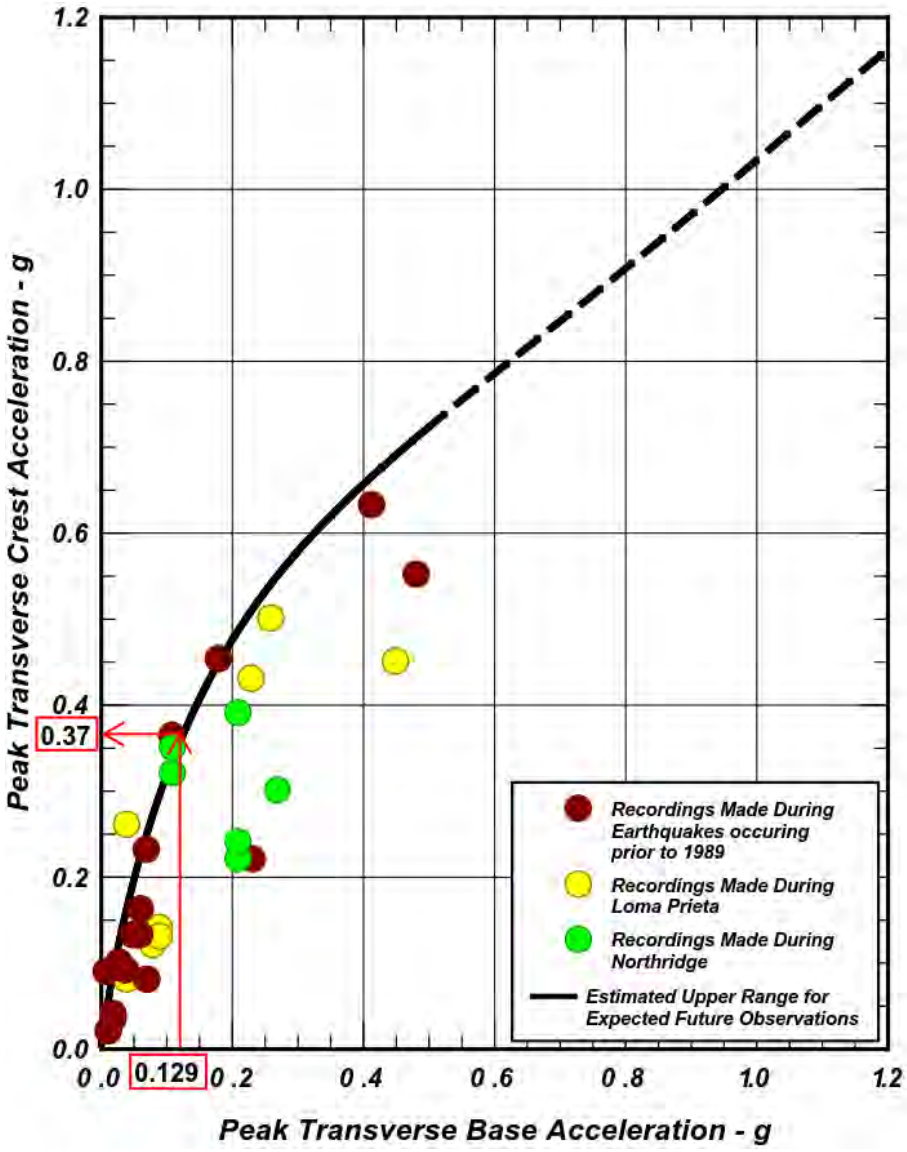


Figure 4. Variations of Peak Transverse Crest Acceleration vs. Peak Transverse Base Acceleration Based on Holzer (1998)

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

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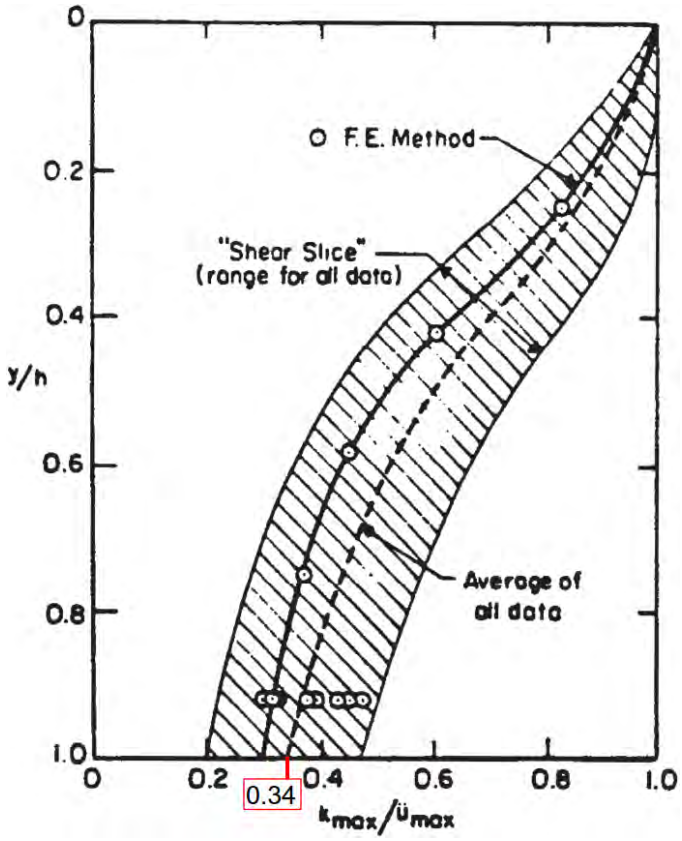


Figure 5. Variation of "Maximum Acceleration Ratio" with Depth of Sliding Mass after Makdisi and Seed (1977)

Therefore:

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

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.

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The water level in the BAP for the seismic loading analysis was set to EL 5,117.8 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 BAP and Fly 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 these structures.

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

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

6 ANALYSIS RESULTS AND CONCLUSIONS

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

Table 3. Safety Factor Results for the BAP Dam

Loading Condition	Required Safety Factor	Calculated Minimum Safety Factor	
		Cross Section 1	Cross Section 2
Long-term, maximum storage pool	1.50	1.58	1.55
Maximum surcharge pool	1.40	1.56	1.53
Seismic (Pseudo-Static)	1.00	1.05	1.07

The results of the safety factor analyses show that the BAP Dams 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

The following references were used in performing this calculation:

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United States Geological Survey (USGS), 2008. *2008 Interactive Deaggregations*. http://geohazards.usgs.gov/deaggint/2008/. Accessed March 11, 2016.

8 ATTACHMENTS

ATTACHMENT A USGS 2008 Seismic PSH Deaggregation

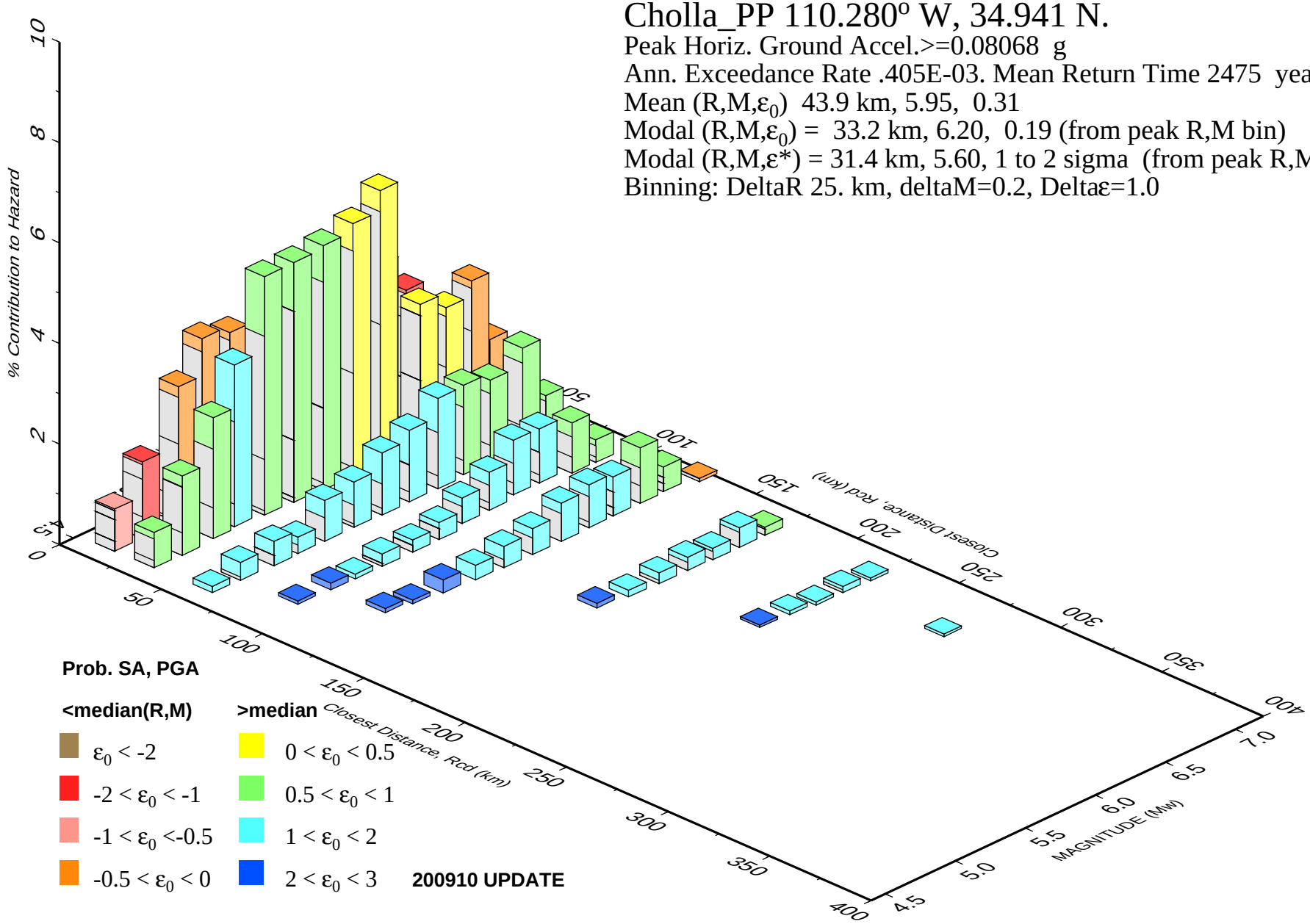
ATTACHMENT B Slope/W Output Figures

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
Bottom Ash Pond

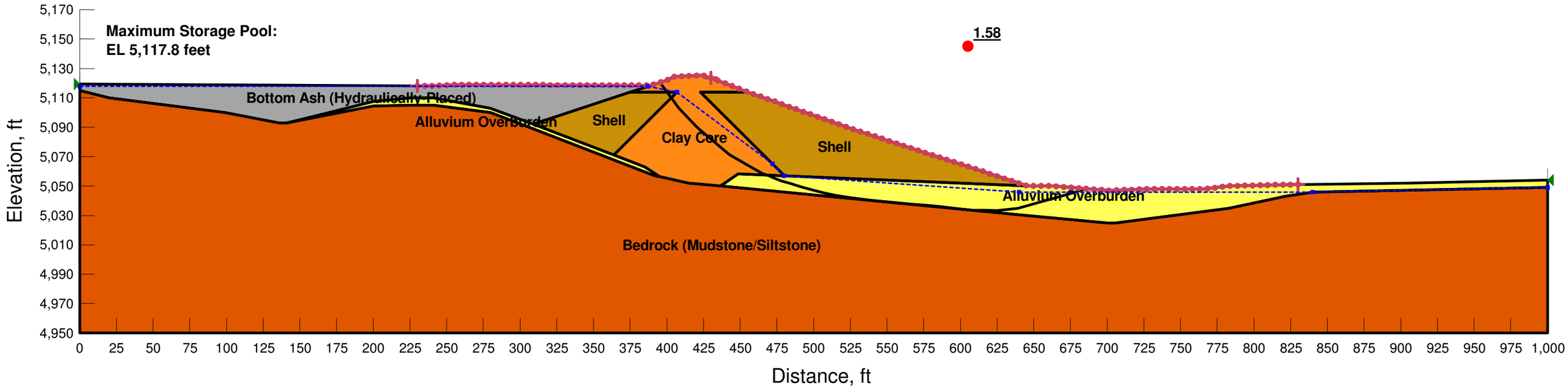
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

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 BAP Section 1 - Static.gsz
Date: 6/21/2016
Method: Spencer

Factor of Safety: 1.58

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 °
Bottom Ash (Hydraulically-Placed)	75 pcf	75 pcf	0 psf	25 °



Slope Stability Analysis
Cross Section 1
Bottom Ash Pond

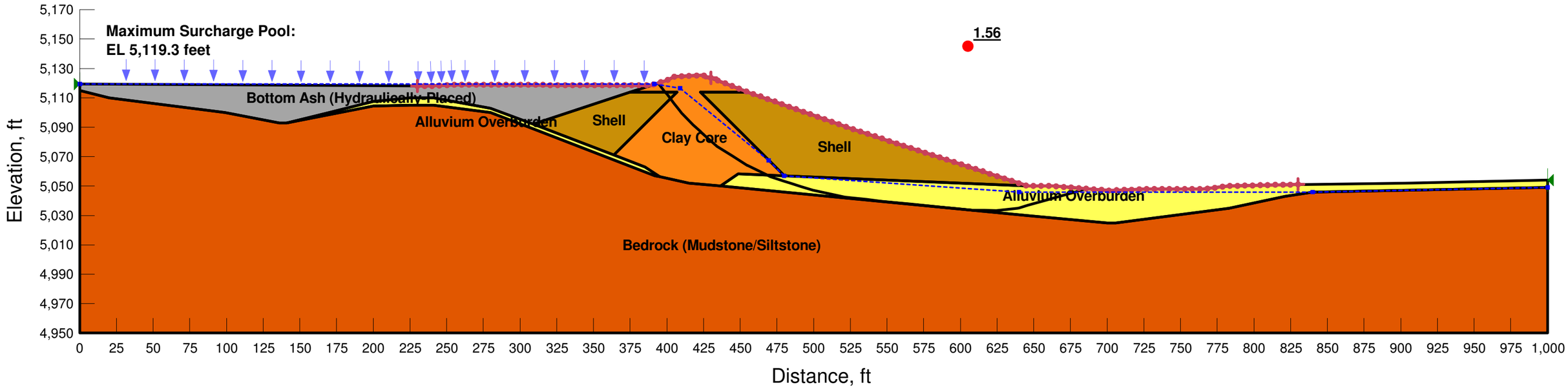
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

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

Figure B2) Static Maximum Surcharge Pool
File Name: APS Cholla BAP Section 1 - Static.gsz
Date: 6/21/2016
Method: Spencer

Factor of Safety: 1.56

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 °
Bottom Ash (Hydraulically-Placed)	75 pcf	75 pcf	0 psf	25 °



Slope Stability Analysis
Cross Section 1
Bottom Ash Pond

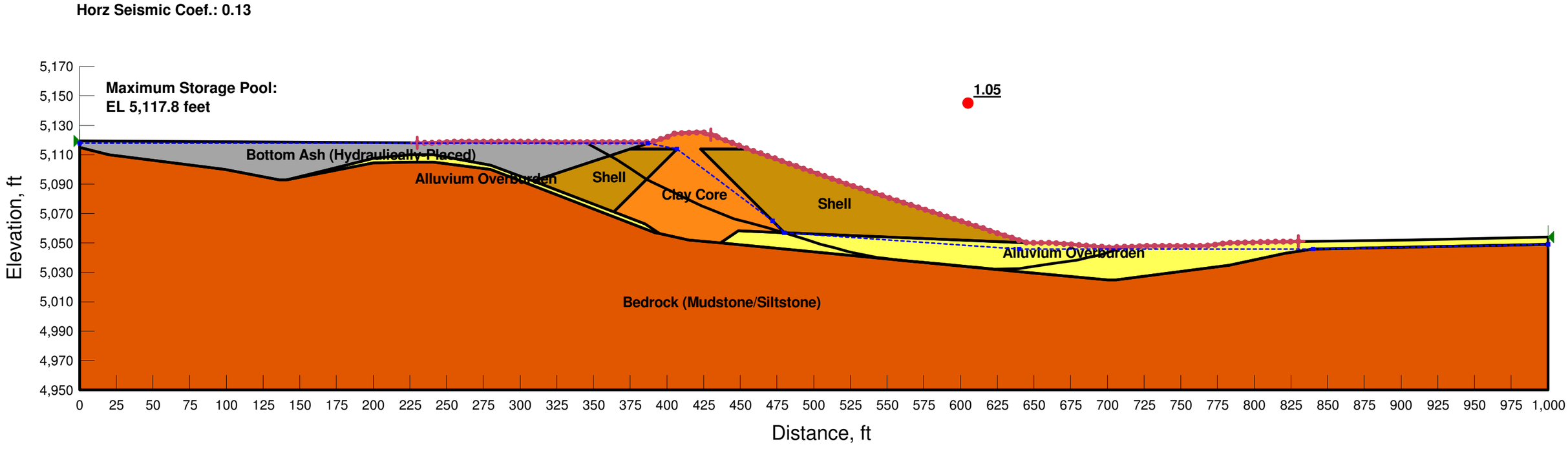
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

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

Figure B3) Seismic Maximum Storage Pool
File Name: APS Cholla BAP Section 1 - Seismic.gsz
Date: 6/21/2016
Method: Spencer

Factor of Safety: 1.05

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 °	--
Bottom Ash (Hydraulically-Placed)	75 pcf	75 pcf	0 psf	25 °	--



Slope Stability Analysis
Cross Section 2
Bottom Ash Pond

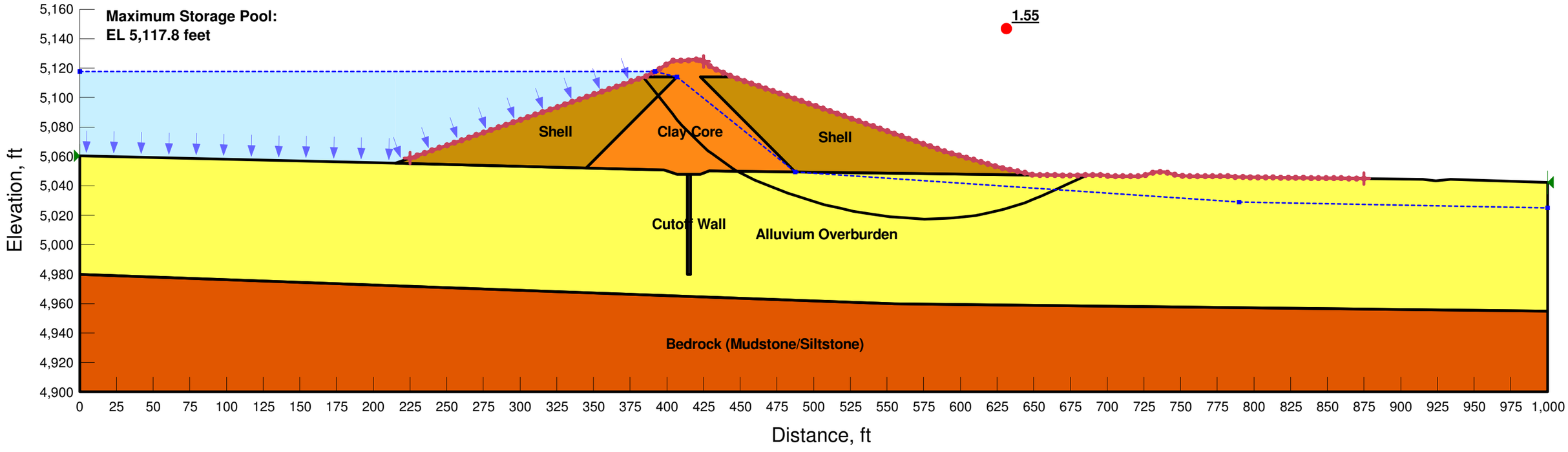
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

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

Figure B4) Static Maximum Storage Pool
File Name: APS Cholla BAP Section 2 - Static.gsz
Date: 6/21/2016
Method: Spencer

Factor of Safety: 1.55

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 °



Slope Stability Analysis
Cross Section 2
Bottom Ash Pond

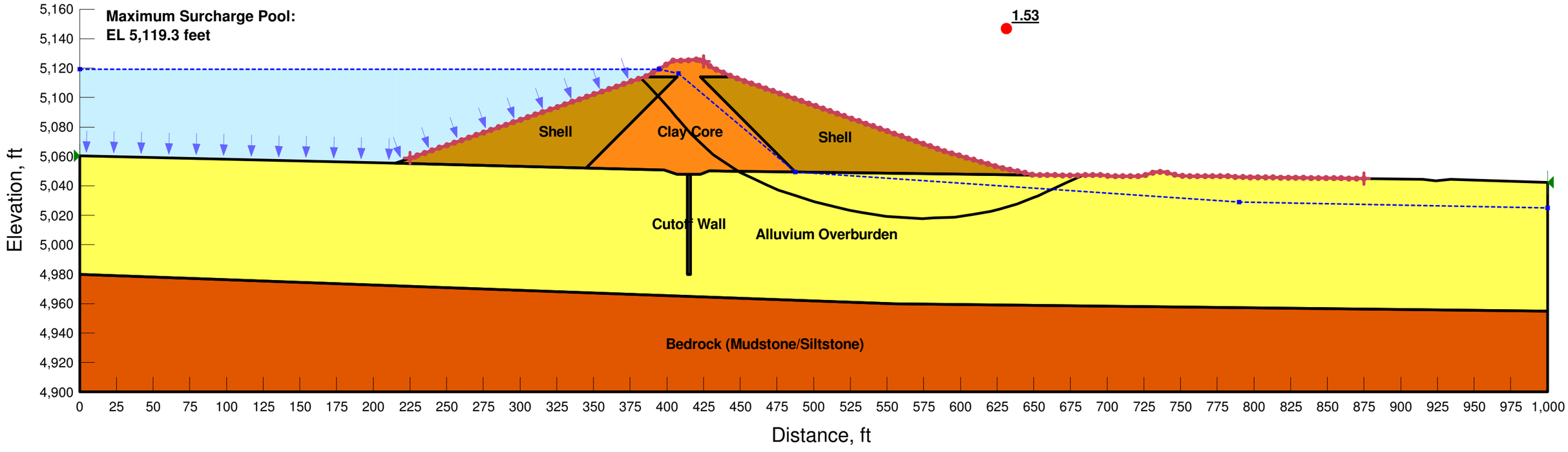
Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

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

Figure B5) Static Maximum Surcharge Pool
File Name: APS Cholla BAP Section 2 - Static.gsz
Date: 6/21/2016
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 °



Slope Stability Analysis
Cross Section 2
Bottom Ash Pond

Cholla Power Plant
Joseph City, Arizona
Arizona Public Service

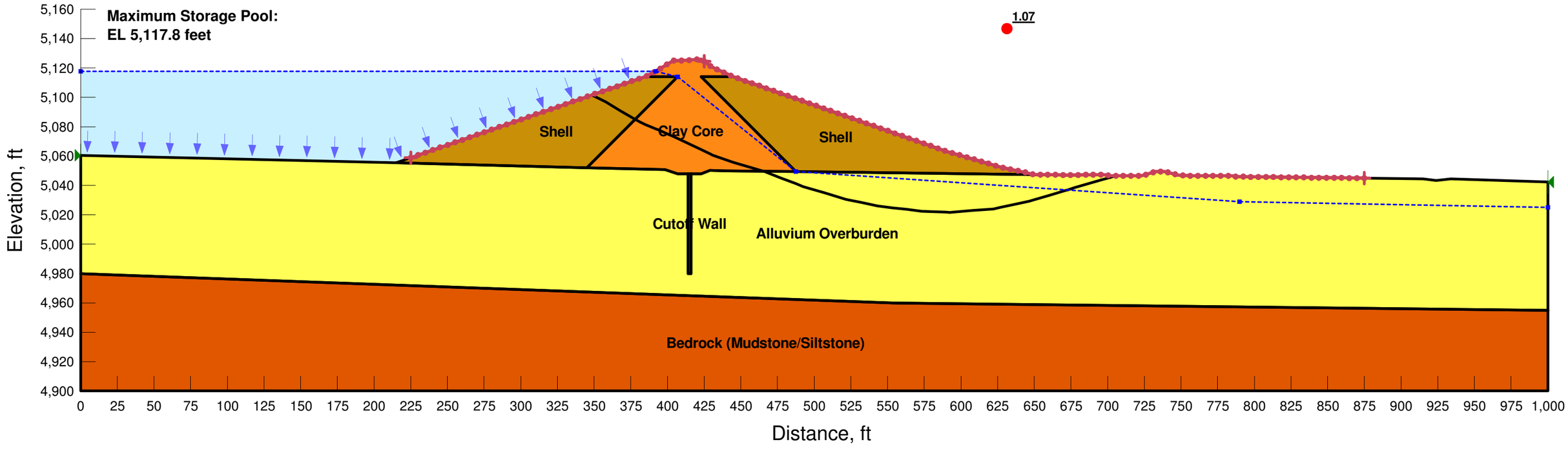
Note:
The results of analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Figure B6) Seismic Maximum Storage Pool
File Name: APS Cholla BAP Section 2 - Seismic.gsz
Date: 6/21/2016
Method: Spencer

Factor of Safety: 1.07

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 °	--

Horz Seismic Coef.: 0.13



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