FOUR CORNERS POWER PLANT COMBINED WASTE TREATMENT POND – Periodic Structural Integrity Assessment

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

October 2021
AECOM Project 60664563

Delivering a better world

Prepared for:

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Attachment

1. Introduction

This periodic update to the Structural Integrity Assessment for the Combined Waste Treatment Pond (CWTP) at Four Corners 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 CWTP at the Four Corners 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: Four Corners Power Plant Lined Ash Impoundment, Lined Decant Water Pond, Combined Waste Treatment Pond, and Dry Fly Ash Disposal Area. 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: Combined Waste Treatment Pond, Four Corners Power Plant, Fruitland, New Mexico. Prepared for: Arizona Public Service, AECOM Job No. 60445844, August 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 CWTP 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 gradually increased each year, reflecting the continued need for crest maintenance, grass and bush cutting/removal on the downstream slope, and possible sloughing-type erosion, above the riprap zone, of the over-steepened downstream slope. At the scale reported, none of these conditions threaten the structural stability of the embankment.

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. There are no instruments associated with the CWTP.

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"

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.

The CWTP is no longer an operating CCR surface impoundment. The CWTP no longer discharges to Morgan Lake through a National Pollutant Discharge Elimination System (NPDES)-permitted internal outfall. APS provided notification, dated November 23, 2020, of its intent to close the CWTP and APS ceased discharge of CCR to the CWTP on or before November 23, 2020. APS is currently considering construction bids for a closure-by-removal project that will be accomplished largely by dredging. A temporary pump system delivers water from the Plant's "hot canal" into the CWTP to maintain a minimum pond level for stability of the dike and to feed to Navajo Mine's water intake pump station. APS intends to breach the dike with a culvert to provide free flow from the hot canal after successful completion and certification of closure-by-removal.

The downstream face of the CWTP embankment was reinforced with approximately 1,160 cubic yards of additional riprap in 2017. Riprap was placed from 5 feet below to 2 feet above the canal water surface elevation and between Stations 1+00 and 3+50 and Stations 8+00 and 12+90.

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 CWTP impoundment currently satisfies the criteria for Low 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.

This review notes that the identified "interim conditions" listed in Section "4.9 Structural Stability" have been resolved as follows:

a. The "(l)ack of adequate erosion protection along the downstream slope of the CWTP embankment" was remedied by a riprap placement project in 2017; and
b. The "(r)eduction of the crest elevation about a one foot below the design elevations" was not remedied, although a small amount of fill was placed to restore the crest elevation of the south abutment to the same El. 5337 feet (North American Vertical Datum of 1988 [NAVD88]) as the rest of the crest.

The current 1-foot difference between the design and actual crest elevations is no longer considered to be a concern because, following cessation of discharge in 2020, the normal level of the pond has been lowered to El. 5330.0 feet (NAVD88) from the previous active operating level (El. 5332.6 feet, NAVD88) reported in Table 3-1 (AECOM, 2016).

AECOM assesses that the design, construction, operation, and maintenance of the CWTP 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.

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

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.

This review notes that the activities recommended in the 2016 Report to address identified “interim conditions” have been completed as described:

a. The “(l)ack of adequate erosion protection along the downstream slope of the CWTP embankment” was remedied by a riprap placement project in 2017; and

b. The “(r)eduction of the crest elevation about a one foot below the design elevations” was not remedied, although a small amount of fill was placed to restore the crest elevation of the south abutment to the same El. 5337 feet (NAVD88) as the rest of the crest.

The current 1-foot difference between the design and actual crest elevations is no longer considered to be a concern because, following cessation of discharge in 2020, the normal operating level of the pond has been lowered to El. 5330.0 feet (NAVD88) from the active operating level (El. 5332.6 feet, NAVD88) reported in Table 3-1 (AECOM, 2016).

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

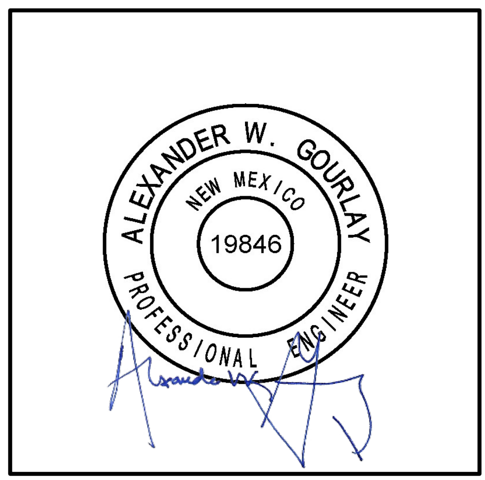
Certification Statement for:

- 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 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 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: Combined Waste Treatment Pond, Four Corners Power Plant, Fruitland, New Mexico*. Prepared for: Arizona Public Service, AECOM Job No. 60445844, August 2016.

ATTACHMENT A

AECOM, 2016. *Final Summary Report, Structural Integrity Assessment: Combined Waste Treatment Pond, Four Corners Power Plant, Fruitland, New Mexico*. Prepared for: Arizona Public Service, AECOM Job No. 60445844, August 2016.

AECOM

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August 26, 2016

Final Summary Report
Structural Integrity Assessment

Combined Waste Treatment Pond
Four Corners Power Plant
Fruitland, New Mexico

Prepared for:
Arizona Public Service

AECOM Job No. 60445844
August 2016

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List of Acronyms

APS	Arizona Public Service
CCR	Coal Combustion Residual
bgs	below ground surface
bpf	blows per foot
CFR	Code of Federal Regulations
CWTP	Combined Waste Treatment Pond
EAP	Emergency Action Plan
EPA	Environmental Protection Agency
EL	Elevation
EPA	Environmental Protection Agency
FCPP	Four Corners Power Plant
ft	feet
HPC	Hazard Potential Classification
LAI	Lined Ash Impoundment
LDWP	Lined Decant Water Pond
MCS	Modified California Split-spoon
NAVD	National American Veritcal Datum
NGVD	National Geodatic Vertical Datum
NMOSE	New Mexico Office of the State Engineer
NPDES	National Pollutant Discharge Elimination System
pcf	pounds per cubic foot
psf	pounds per square foot
RCP	reinforced concrete pipe
RCRA	Resource Conservation and Recovery Act
SPT	Standard Penetration Test
URS	Upper Retention Sump
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; Four Corners Power Plant; Combined Waste Treatment Pond

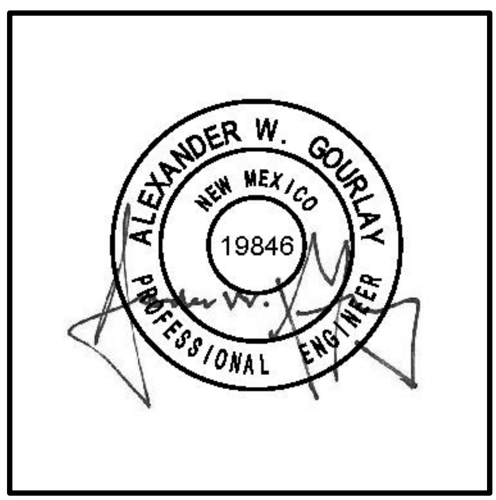
I, Alexander Gourlay, being a Registered Professional Engineer in good standing in the State of New Mexico, do hereby certify, to the best of my knowledge, information, and belief, that the information contained in this certification has been prepared in accordance with the accepted practice of engineering.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

Alexander W. Gourlay, P.E.

Printed Name

August 26,2016

Date



1 Introduction

Arizona Public Service Company (APS) contracted URS Corporation, a wholly owned subsidiary of AECOM, to assist in the initial structural integrity assessment of the existing coal combustion residual (CCR) surface impoundments at the Four Corners Power Plant (FCPP) on the Navajo Nation in Fruitland, New Mexico. Figure 1-1 shows the location of the CCR Impoundments at the FCPP. This Summary Report documents the AECOM structural integrity assessment for the Combined Waste Treatment Pond (CWTP). Assessments of other CCR Impoundments at the FCPP 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 CWTP located at the FCPP. The CWTP is an existing CCR surface impoundment owned and operated by APS. The CCR impoundment is not regulated by the New Mexico Office of the State Engineer (NMOSE). 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 assessment must address the following elements:

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

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

In addition, the following elements must be addressed for CCR units, such as the CWTP, 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 CWTP, 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	
• Appendix C – Liquefaction Triggering Calculation	

1.4 Facility Description

The FCPP is an electric generating station located on the Navajo Nation in Fruitland, San Juan County, New Mexico. The station is operated by APS and owned by a consortium of five utility companies with APS possessing a majority stake. The FCPP consists of two coal-fired electrical generating units, Units 4 and 5. Units 1, 2, and 3 were decommissioned in 2013. The two generating units are cooled by water from Morgan Lake, a man-made reservoir located immediately north of the plant. Four existing CCR surface impoundments are located at the FCPP: the Combined Waste Treatment Pond (CWTP) located immediately east of the plant, the Lined Ash Impoundment (LAI) located about one mile west of the plant, the LDWP located about one and a half miles west of the plant and adjacent to the LAI, and the Upper Retention Sump (URS) located immediately southeast of the plant. CCR generated at the power plant are disposed of at a landfill, the Dry Fly Ash Disposal Area, and the LAI, while the CWTP and LDWP are used as water decant ponds. The URS is an incised surface impoundment receiving storm water from the flue gas desulfurization thickener system. Figure 1-1 shows the location of the CWTP, LAI, and LDWP in relation to the power plant. This assessment evaluates the structural integrity of the CWTP.

The CWTP primarily receives bottom ash transport water from the bottom ash recovery and transport processes at Units 4 and 5 (U45), but also receives smaller amounts of low-volume waste water from the plant. The bottom ash transport water is conveyed to the CWTP after it is separated from the bottom ash solids at the U45 hydrobins. Ash and other sediment within the transport water that have bypassed the hydrobins is allowed to settle out in one of seven parallel earthen decant basins located along the western side of the CWTP footprint. The water in the decant basins overflows into the CWTP. The decant basins are periodically dredged to remove the settled solids. Two 36-inch gated reinforced concrete pipes (RCPs) at the southeast corner of the CWTP allow discharge into the cooling water return canal which empties into Morgan Lake. The discharge into Morgan Lake is monitored through a National Pollutant Discharge Elimination System (NPDES) permitted discharge point (NPDES Permit No. NM0000019, Internal Outfall No. 01E).

The CWTP has a total surface area of about seven acres; however, the storage capacity is unknown. The impoundment is surrounded on the west and south sides by native ground and fill. On the north and east side, the impoundment is enclosed by an embankment that separates the CWTP from the cooling water return canal. The CWTP embankment is not regulated by NMOSE and has not previously been classified for hazard potential.

The CWTP embankment is an earthen, impoundment dike that separates the CWTP from the plant cooling water return canal. The embankment was constructed in 1978 in order to form the northern and eastern perimeter of the CWTP and the impoundment basin. It is not a zoned embankment; rather it consists mostly of a relatively uniform sandy lean clay fill. The embankment is approximately 1,800 ft in length with a 45 degree curve near the center from Station 9+38 to 11+96. The maximum structural height of the embankment is about 32 ft occurring near the eastern abutment and the average height over the length of the embankment is about 23 ft. According to historic drawings, the top of crest elevation is nominally elevation (EL) 5338 ft-NAVD88 (EL 5335 ft-NGVD29) with upstream and downstream slopes inclined at a nominal two horizontal to one vertical angle (2H:1V); however, a recent survey of the embankment indicates the crest elevation is approximately EL 5337 ft-NAVD88 (EL 5334 ft-NGVD29) with upstream and downstream slopes inclined as steep as 1.4H:1V. A crest elevation of EL 5337 ft-NAVD88 results in about 4.4 ft freeboard above the maximum storage pool water level (EL 5332.6 ft-NAVD88). The crest width is about 24 ft over the length of the embankment, which is less than the design width of 30 ft. The slopes are clad with sandstone facing slabs approximately three to four f long and one-and-a-half ft wide starting at about one foot above the maximum water pool level in the CWTP and extending below the water surface.

The CWTP embankment is founded on bedrock near the abutments and a layer of bottom ash near the center. The bottom ash appears to be about 20 ft thick at its greatest extent with little indication of mechanical compaction. No seepage control measures are installed to limit flow beneath the embankment. The CWTP embankment has no internal drain system.

Water levels within the pond are controlled by varying the pumping rate from the plant to balance with seepage, evaporation, and flow out of the pond through the discharge spillway. The discharge spillway consists of a weir with stop logs that spills into twin-barrel 36-inch diameter RCP pipes that penetrate the embankment near the eastern corner of the CWTP. The spillway intake is at EL 5331.8 ft-NAVD (EL 5328.8 ft-NGVD); however, the maximum storage pool water level is defined at EL 5332.6 ft-NAVD88 (EL 5329.6 ft-NGVD29) based on the fixed weir position and water surface elevation recorded in December 2015. The spillway outlet is at EL 5330.9 ft-NAVD88 (EL 5333.9 ft-NGVD29). During the design storm event, defined as the 100-year, 24-hour storm, the water level within the CWTP is expected to rise to EL5335.1 ft-NAVD88 (5332.1 ft-NGVD29). This water level, defined as the maximum surcharge pool water level, would leave 1.9 ft of freeboard below the embankment crest.

Monitoring wells and flow measurement devices are installed at the CWTP embankment. The monitoring wells were installed in September 2015 and are used to monitor CCR groundwater, while the totalizer measures flows through the discharge spillway. The totalizer has been monitored on a daily basis. Water levels within the pond are maintained by the stop log weir at the discharge spillway. Measurements from the monitoring instruments are reviewed and documented annually as part of the annual inspection. The CCR monitoring wells and flow measurement devices are not dam instrumentation, therefore, they are excluded from the requirement to read at intervals not exceeding 30 days per Rule 40 CFR § 257.83(a)(1)(iii). The locations of the four CCR monitoring wells and one spillway discharge are shown on Figure 1-2.

Inspections of the CWTP are performed by a qualified person at intervals not exceeding seven days. The inspections examine the CWTP for actual or potential conditions that could disrupt the operation or safety of the impoundment and documents the results of the inspection in the unit 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 CWTP was performed on October 14, 2015 (AECOM & APS, 2016).

2 Hazard Potential Classification

This section summarizes the initial Hazard Potential Classification (HPC) for the CWTP. 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 CWTP 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 Low Hazard Potential classifications, 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 Significant Hazard Potential structure.

The potential for downstream loss of life is assessed by reviewing land use in area downstream (to the north and east) from the embankment, where inundation was likely in the event of a release. No quantitative dam break or inundation studies were performed. The United States Geological Survey (USGS) 7.5-Minute Quadrangle topographic map of Fruitland, New Mexico 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 CWTP and its immediate surrounding identified the following considerations for assessing the initial HPC of the impoundment:

- APS leased lands encompass the CWTP and the FCPP cooling water return canal but not the remainder of Morgan Lake.

- The water surface elevation differential between the CWTP and the cooling water return canal is about three ft, effectively limiting the volume and rate of discharge from the CWTP to the canal in the event of some catastrophic failure of the CWTP embankment.

- Given a maximum three-foot head differential and 7-acre area of the CWTP, the maximum volume that could drain from the CWTP to Morgan Lake is 26 acre-ft, which is 0.07 percent of the 39,000 acre-ft maximum storage capacity of Morgan Lake.

- Under normal operations, all water contained in the CWTP eventually discharges to the cooling water return canal and then to Morgan Lake.

Based on the identified considerations, physical losses associated with failure of the CWTP embankment would be to the embankment itself and, at worst case, in some manner, to the FCPP discharge canal, both of which are on APS leased lands. The impact to Morgan Lake would only be an uncontrolled release of no more than 26 acre-ft of water to the 1,200 acre lake.

Based on review of the USGS 7.5-Minute Quadrangle topographic map of Fruitland, NM (USGS, 2013), the potential for downstream loss of life is negligible since the area downstream of the embankment where inundation is likely in the event of a release does not contain any areas of permanent or temporary land use. Therefore, since failure or mis-operation of the CWTP impoundment results in no probable loss of life and low economic and/or environmental losses, and the losses are principally limited to the APS leased lands, the CWTP impoundment satisfies the criteria for Low Hazard Potential classification.

3 History of Construction

This section summarizes the history of construction for the CWTP. 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 or in-house resources for information regarding the history of construction for the CWTP. 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 CWTP Construction Summary

The history of construction dating back to the original construction that began in 1978 is summarized in Table 3-1 below.

Table 3-1. History of Construction for the CWTP

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.	N/A	---	---
Size Classification	Small	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Hazard Classification	Low	---	Section 2.2
Construction Date	Initial Configuration: 1978 Decant Cells Addition: 2012	---	APS Drawing No. G-67227 Sheet 1 of 5 (APS, 1978)
Location on USGS Quadrangle Map	Fruitland Quadrangle: Section 25, Township 29 North, Range 16 West	See Figure 3-1	Fruitland Quadrangle (USGS, 2013)
Statement of Purpose	Settling pond for CCR-impacted process and surface water		---
Name of Watershed	Not named	---	---
Size of Watershed (ac)	166	Includes flows pumped to CWTP from other areas	2016 Four Corners CWTP Inflow Flood Control Plan
Area Capacity Curve	None	---	---
Embankment Type	Unzoned earth fill	---	APS Drawing No. G-67227 Sheet 1 of 5 (APS, 1978)
Embankment Maximum Height (ft)	32	Average height is 22.8 ft	APS Drawing No. G-67227 Sheet 3 of 5 (APS, 1978)
Total Operating Freeboard (ft)	4.4	Freeboard relative to the normal operating pool level.	
Embankment Length (ft)	1,800	---	APS Drawing No. G-67227 Sheet 2 of 5 (APS, 1978)
Embankment Crest Elevation (ft)	Design: ±5,335 NGVD29 (±5,338 NAVD88) Current: 5,337 (NAVD88)	Ranged from EL 5,334.7 ft (NGVD29) at the left abutment to EL 5,335.9 ft (NGVD29) at the right abutment	• APS Drawing No. G-67227 Sheet 1 of 5 (APS, 1978) • APS Drawing No. G-67227 Sheet 3 of 5 (APS, 1978) • 2016 Survey (J. Marbles Land Surveying, 2016)

Item	As-Constructed/ Current	Comments	Reference Document
Embankment Crest Width (ft)	Design: 30 2016 Survey: 24	---	• APS Drawing No. G-67227 Sheet 1 of 5 (APS, 1978) • 2016 Survey (J. Marbles Land Surveying, 2016)
Embankment Slopes	Design: 2H:1V (downstream & upstream) Current: 1.4H:1V to 1.5H:1V (downstream); 1.8H:1V to 1.9H:1V (upstream)	---	• APS Drawing No. G-67227 Sheet 1 of 5 (APS, 1978) • 2016 Survey (J. Marbles Land Surveying, 2016)
Slope Protection	Face stone	---	APS Drawing No. G-67227 Sheet 1 of 5 (APS, 1978)
Normal Operating Storage Level (ft)	Design: 5,328 (NGVD29) Current: 5,332.6 (NAVD88)	---	• APS Drawing No. G-67227 Sheet 1 of 5 (APS, 1978) • Groundwater Evaluation (AECOM, 2015)
Storage Capacity (ac-ft)	Overall: 137 Stormwater: 27	Stormwater storage capacity above normal operating storage EL 5,332.6 ft (NAVD88)	• 2015 Annual Inspection Report (AECOM& APS, 2016) • 2016 Four Corners CWTP Inflow Flood Control Plan
Surface Area (ac)	7.23	---	Aerial Survey (APS, 2014)
Material Properties:			
	Embankment Properties		
Physical Properties	The embankment consists of compacted sandy lean clay and clayey sand with varying amounts of gravel.	---	2016 AECOM Geotechnical Investigation
Engineering Properties	• Moist Unit Weight = 115 pounds per cubic foot (pcf) • Saturated Unit Weight = 130 pcf • Effective Cohesion = 175 pounds per square foot (psf) • Effective Friction Angle = 36^{o} • Total Cohesion = 250 psf • Total Friction Angle = 19^{o}	---	

<table>
<tr><th>Item</th><th>As-Constructed/ Current</th><th>Comments</th><th>Reference Document</th></tr>
<tr><td colspan="4">Foundation Conditions</td></tr>
<tr><td>Physical Properties</td><td>The embankment is generally founded on bottom ash or weathered bedrock. Bottom ash is generally classified as silty sand with the upper 5 ft exhibiting lower blow counts indicating loose material. The weathered bedrock consists of sandstone and shale.</td><td>Bottom ash beneath the embankment ranges from approximately 3 to 20 ft thick.</td><td rowspan="2">• 2016 AECOM Geotechnical Investigation
• Raising Ash Dams 3 and 6 (Dames & Moore, 1990)
• LAI Geotechnical Analysis Report (URS, 2008)
• LAI Engineering Design Report (URS, 2012)</td></tr>
<tr><td>Engineering Properties</td><td><u>Bottom Ash:</u>
• Moist Unit Weight = 80 pcf
• Saturated Unit Weight = 91 pcf
• Effective Cohesion = 0 psf
• Effective Friction Angle = 31^{o} (upper layer); 39^{o} (bottom layer)
<u>Bedrock (Weathered Shale):</u>
• Moist Unit Weight = 120 pcf
• Saturated Unit Weight = 125 pcf
• Total Cohesion = 600 psf
• Total Friction Angle = 20^{o}</td><td>---</td></tr>
<tr><td colspan="4">Abutment Conditions</td></tr>
<tr><td>Physical Properties</td><td>The abutments consist of bedrock comprising weathered sandstone and shale.</td><td>---</td><td rowspan="2">• Engineering Geology Report (Ebasco, 1959)
• APS Drawing No. G-67227 Sheet 3 of 5 (APS, 1978)
• LAI Geotechnical Analysis Report (URS, 2008)
• LAI Engineering Design Report (URS, 2012)</td></tr>
<tr><td>Engineering Properties</td><td>• Moist Unit Weight = 120 pcf
• Saturated Unit Weight = 125 pcf
• Total Cohesion = 600 psf
• Total Friction Angle = 20^{o}</td><td>---</td></tr>
<tr><td>Spillway Type</td><td>Gated, twin-barrel RCPs</td><td>---</td><td>APS Drawing No. G-67227 Sheet 4 of 5 (APS, 1979)</td></tr>
<tr><td>Spillway Pipe Diameter (in)</td><td>36 (each pipe)</td><td>---</td><td>APS Drawing No. G-67227 Sheet 4 of 5 (APS, 1979)</td></tr>
<tr><td>Spillway Invert Elevation (ft)</td><td>5,328.77 (NGVD29)</td><td>---</td><td>APS Drawing No. G-67227 Sheet 4 of 5 (APS, 1979)</td></tr>
<tr><td>Spillway Discharge Capacity (cfs)</td><td>133.35</td><td>---</td><td>2016 Four Corners CWTP Inflow Flood Control Plan</td></tr>
<tr><td>Construction Specifications</td><td>None available</td><td>---</td><td>---</td></tr>
</table>

Item	As-Constructed/ Current	Comments	Reference Document
Detailed Drawings	See Appendix A for drawings	---	• Original Drawings (APS, 1978 and 1979) • 2016 Survey (J. Marbles Land Surveying, 2016)
Existing Instrumentation			
Type and Purpose of Instrumentation	• None for dam integrity assessment • CCR wells for monitoring the groundwater. • Totalizer measuring flow through spillway.	---	CCR Monitoring Well Network Installation Report and Certification (URS, 2016)
Location of Instrumentation	• CCR monitor wells located in the embankment, abutment, and upstream of the impoundment. • Totalizer at the spillway inlet.	See Figure 1-2	CCR Monitoring Well Network Installation Report and Certification (URS, 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.	---	Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016)
Record of Structural Instability	None noted	---	---

4 Structural Stability Assessment

This section summarizes the structural stability assessment for the CWTP. 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 CWTP embankment is founded on bottom ash, the deposition of which predated the construction of the impoundment, resting on bedrock of weathered shale and sandstone of the Pictured Cliffs Formation. The bottom ash does not appear to be present near the abutment where the embankment bears directly on bedrock. Near the center of the embankment the bottom ash reaches a maximum observed thickness of about 20 ft. The upper two to five ft of the bottom ash, referred to as the upper bottom ash, appears to be of lower density and strength then the lower portions of the deposit which is medium dense. The underlying weathered shale and sandstone appear competent within the embankment footprint based on exploratory borings drilled to bedrock during the 2016 AECOM Geotechnical Investigation

No construction records for the CWTP embankment could be found in the project records; therefore, the method of foundation preparation prior to construction of the embankment is unknown. There is no indication in design drawings or site exploration records that measures were taken to limit seepage beneath the embankment through the relatively porous bottom ash material; however, the head differential across the embankment is expected to be low due to the adjacent cooling water return canal which is typically about three ft below the water level within the CWTP pond.

No displacement survey monuments are present along the crest of the CWTP embankment to measure vertical or horizontal displacements with time. Comparison of the nominal design crest elevation to the crest elevation measured during a site survey of the embankment conducted as part of the 2016 AECOM Geotechnical Investigation indicates the crest is about one foot lower than anticipated. This could indicate a moderate settlement of the embankment, but also could be due to removal of material during maintenance (grading) of the crest, deviations from the design during construction, or erosion of the crest. Also, no cracking of the embankment has been observed during inspections and maintenance of the embankment an indication that differential settlement, if present within the embankment, is small. The presence of competent bedrock foundation and observations of small to moderate settlement of the embankment are an indication of stability of the 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*."

Design drawings of the CWTP embankment indicate the upstream and downstream slopes are protected from erosion by a 2 to 3.5-foot thick layer of face stone on top of one foot of filter backing. Examination of the recent survey of the embankment conducted as part of the 2016 AECOM Geotechnical Investigation of the CWTP shows that the embankment slopes are steeper than the 2H:1V vertical angle indicated in the design drawings. The difference is especially pronounced on the downstream slope where they are as steep as 1.4H:1V. This difference could be due to deviations from the design during construction or also could indicate erosion of the slopes on the downstream side of the embankment where the flow velocity of the adjacent cooling water return canal is high,

Vegetation near the crest of the embankment appears to provide erosion protection for the upper portions of the slope above the waterline where the facing stone is not present. On the upstream slopes, the vegetation is generally less than six inch in length. On the downstream slope, the vegetation was noted during the most recent inspection (AECOM & APS, 2016), to be excessive, consisting of desert shrubs and small trees. The excess vegetation was subsequently removed in late 2015; however, the removal has now left the upper portion of the slope without erosion protection. This portion of the upstream slope requires additional slope protection to prevent erosion; the work is expected to be performed in 2017.

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*."

The CWTP embankment is composed primarily of a sandy lean clay fill. No construction records or specifications for construction of the embankment could be found to indicate the method by which it was constructed. Borings drilled through the embankment crest during the 2016 AECOM Geotechnical Investigation, recorded Standard Penetration Test (SPT) blow counts ranging from 12 to 27 blows per foot (bpf) indicating a stiff to very stiff consistency. Below the embankment fill lies a layer of bottom ash from prior operations. Some low SPT blow count material was observed in this layer.

Based on review of the field exploratory borings performed along the crest of the dike which indicates the fill is stiff to very stiff, the CWTP embankment has been mechanically compacted to a density sufficient to withstand the range of loading condition expected at the CWTP site.

4.4 Slope Vegetation

Per the requirements 40 CFR § 257.73(d)(1)(iv), existing CCR impoundment 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 sandstone facing slabs for slope protection; therefore, the dam is excluded from the vegetated slope requirements since it uses an alternate form of slope protection.

4.5 Spillways

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 spillway for the CWTP pond consists of a weir and two twin 36-inch diameter RCPs that penetrate the embankment near the eastern corner of the CWTP embankment. The weir structure at the inlet of the spillway includes stop logs to control the elevation of the water level in the pond. The spillway empties into the cooling water return canal. Analysis conducted as part of the 2016 Four Corners CWTP Inflow Flood Control Plan Certification, under separate cover, shows the spillway has capacity to adequately manage flow during and following the peak discharge from the 100-year flood event, as required for a low hazard potential impoundment, and still maintain about two feet of freeboard. Recent inspections of the spillway (AECOM & APS, 2016) found it to be in good working order with no visible damage.

Based on review of the spillway design drawings, the 2016 Four Corners CWTP Inflow Flood Control Plan Certification for the CWTP, and the most recent inspection report, the spillway at the CWTP has been designed, constructed, and maintained to adequately manage flow during and following the peak discharge of the 100-year 24-hour storm event, as required for low hazard potential impoundments. Further, the spillway is of non-erodible construction and is designed to carry sustained flows.

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*."

Two twin 36-inch diameter RCPs associated with the pond spillway penetrate the CWTP embankment. Design drawings indicate the pipes are installed at a flow line intake elevation of EL 5,331.5 ft-NAVD (EL 5,328.5 ft-NGVD) and flow line outlet elevation of EL 5,331.15 ft-NAVD (EL 5,328.15 ft-NGVD). The pipes penetrate the embankment at an oblique angle for a distance of about 48 ft. No construction or as-built records could be found to indicate embedment of the pipe in anything other than compacted earth fill. Recent inspections of the spillway (AECOM and APS, 2016), found the spillway pipes appeared to be working effectively with no evidence of subsidence or other indication of potential structural deterioration.

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*."

The cooling water return canal inundates the downstream slope of the CWTP embankment to an elevation of EL 5329.5 ft-NAVD (EL 5326.5 ft-NGVD). The canal discharges directly into Morgan Lake about 1,100 ft downstream from the CWTP. Due to the close proximity of the lake, water levels in the canal at the CWTP are approximately equivalent to those within Morgan Lake. No rapid drawdown of Morgan Lake is possible other than a catastrophic failure of the Morgan Lake Dam, which is unlikely to occur due to regular inspection and maintenance of the dam. Since the Lake acts as the primary cooling water source for the plant, operations do not include rapid fluctuations of the water level.

Based on the inability of Morgan Lake and the hydraulically connected cooling water return canal to change water levels rapidly, a rapid drawdown scenario for the downstream slope of the CWTP embankment was considered unlikely and no rapid drawdown analysis was performed.

4.8 Other Issues

No deficiencies were identified for the CWTP that could affect the structural stability of the impoundment. However, during a survey of the embankment conducted as part of the 2016 AECOM Geotechnical Investigation, it was noted that the crest of the containment embankment was at an approximate elevation of EL 5,337 ft-NAVD (EL 5,334 ft-NGVD), which is about one foot lower than the design elevation of the crest. An assessment and recommendation regarding the current normal operating level, the recommended freeboard above the maximum flood pool elevation, and the corresponding recommended minimum crest elevation is provided in the separate Inflow Design Flood Control System plan.

The most recent dam inspection (AECOM & APS, 2016) reported observations of excessive vegetation, consisting of small desert brush and small trees, along the downstream slopes of the CWTP embankment. APS work crews subsequently removed vegetation in the identified areas. Although the vegetation 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 CWTP CCR Impoundment based on the documents provided and reviewed as part of this assessment; however, AECOM did identify several interim conditions that should be addressed in the upcoming project works for 2017. The noted interim conditions consist of:

- Lack of adequate erosion protection along the downstream slope of the CWTP embankment, and
- Reduction of the crest elevation about a one foot below the design elevations.

AECOM assesses that the design, construction, operation and maintenance of the CWTP 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 CWTP. 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 CWTP embankment 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 information from the 2016 AECOM Investigation. The locations of the cross sections along the CWTP embankment are shown in Figure 5-1. The cross sections analyzed are:

CWTP Cross Section 1: This cross section is located along the northern boundary of the CWTP as shown in Figure 5-1. This section represents the maximum section in the western portion of the CWTP embankment where a layer of low blow-count bottom ash was encountered. The embankment is approximately 22 ft high, with a 1.4H:1V downstream slope and a 1.8H:1V upstream slope. The embankment fill consists of sandy lean clay over approximately 14 ft of bottom ash with bedrock at approximately 36 ft below ground surface (bgs). Modified California Split-spoon penetration testing in the upper zone of the bottom ash at the boring CWTP-4 located to the east of Section 1 resulted in an uncorrected N-value of two (indicating very loose soil). This condition was represented in the model using a five foot thick layer of "Upper Bottom Ash" having strength lower than the underlying bottom ash).

CWTP Cross Section 2: This cross section is located along the northeastern boundary of the CWTP as shown in Figure 5-1. This section represents the maximum section in the eastern portion of the CWTP embankment having the steepest downstream slope. The embankment is approximately 20 ft high, with a 1.5H:1V downstream slope and a 1.9H:1V upstream slope. The embankment fill consists of sandy lean clay over approximately three ft of bottom ash with bedrock at approximately 24 ft bgs. No SPT blow counts were recorded for the limited thickness of the bottom ash in boring CWTP-2 located near Section 2. Therefore, the bottom ash material in CWTP Section 2 was represented in the model as "Upper Bottom Ash" and assigned a lower strength than the underlying Bottom Ash encountered in Section 1.

The continuity of the facing slabs observed below the water surface and on the design drawings of the embankment slopes in March 2016 is unknown. Therefore, this layer was, conservatively, not included in the stability models.

5.3 Subsurface Stratigraphy

Idealized models of subsurface stratigraphic conditions for each cross section were developed based on the 2016 AECOM Geotechnical Investigation. The stratigraphic units described as follows were used to develop SLOPE/W models for each cross section.

Embankment Fill: Embankment fill was encountered in each of the borings drilled on the crest of CWTP embankment from the ground surface to depths ranging from 12 to 22 bgs. The embankment materials were generally classified as Sandy Lean Clay with Gravel (CL) or Clayey Sand with Gravel (SM) based on the Unified Soil Classification System (USCS). Gravel content ranged from 3 to 21 percent, by weight, in the embankment fill. Blow count values (uncorrected) ranged from 12 to 27, indicating stiff to very stiff soil.

Bottom Ash: Bottom ash was encountered below the embankment fill in three of the four borings drilled on the crest of the CWTP embankment. The thickness of the bottom ash was observed to range between approximately 3 ft and 20 ft. The bottom ash was generally classified as Silty Sand (SM) based on the USCS. The density of the bottom ash varied considerably between the upper three to five feet, where the ash is very loose as indicated by SPT blow count of two. Below the upper two to five ft, the bottom ash is moderately dense to dense with blow counts ranging from 11 to 32. Due to the considerable difference in density and therefore strength of the bottom ash with depth, the bottom ash was divided into an Upper Bottom Ash and Bottom Ash layer in the cross sections.

Bedrock: Weathered shale and weathered sandstone foundation materials were encountered in two of the four borings (CWTP-2 and CWTP-4) located between Sections 1 and 2. Auger refusal at CWTP-1 and CWTP-3 was interpreted to be bedrock at these locations; however, no foundation material was recovered. Blow count values ranged from 67 to auger refusal, indicating very dense or hard soil. The weathered shale stratum was observed to contain partings and thin (less than 0.125 inches thick) discontinuous seams filled with gypsum.

5.4 Material Properties

Material properties for embankment fill were developed based on the field and laboratory data from the 2016 AECOM investigation. Material properties for the bottom ash material and bedrock were based on previous investigations and testing for Dams 3 and 6 (Dames & Moore, 1990), and the Lined Ash Impoundment (URS, 2008 and 2012).

The material properties selected for use in the slope stability analyses of the CWTP are presented in Table 5-1.

Table 5-1. Selected Material Parameters – CWTP 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)	Cohesion, c (psf)	Friction Angle, ϕ (degrees)
Embankment Fill	130	115	175	36	250	19
Upper Bottom Ash	91	80	0	31	--	--
Bottom Ash	91	80	0	39	--	--
Bedrock	125	120	--	--	600	20

5.5 Embankment Pore Pressure Distribution

Water levels were measured in the CWTP and the cooling water return canal in December, 2015 (URS, 2016). These measurements were considered to be the most reliable indicators of pore pressure distribution within the CWTP embankment. The pore pressure distribution in the embankment was estimated using water level measurements and assuming a straight line phreatic surface between the water level in the pond and in the canal. The water level in the CWTP is controlled through the positioning of stop logs on the weir at the discharge pipes (spillway) and was measured at 5,332.6 ft in December 1, 2015; the water level is reportedly relatively constant over time. The canal is hydraulically connected to Morgan Lake via open-channel flow; the downstream water level was assumed to be relatively constant at 5329.5 ft, measured on December 1, 2015 (AECOM, 2016).

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 considers a pool elevation in the CCR unit that is equivalent to the lowest elevation of the invert of the spillway (i.e., the lowest overflow point of the perimeter of the embankment) using shear strengths expressed as effective stress and with pore water pressures that correspond to the long-term condition.

For the CWTP embankment, the safety factor was calculated for the long-term, maximum storage pool at 5332.6 ft (NAVD88), based on the fixed weir position and water surface elevation recorded in December 2015.

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.

For the CWTP embankment, the safety factor was calculated for the maximum surcharge pool conservatively assumed to be at the crest elevation of the embankment of 5336.5 ft (NAVD88), which represents a complete loss of freeboard.

Seismic Loading: Seismic loading was 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 its 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 pseudostatic analysis was used to represent the seismic loading condition.

The seismic response of soil embankments is incorporated into the limit equilibrium 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 in the soil profile above bedrock, and calculate the adjusted for Site Class effects, PGA_M;
3. Calculate the maximum transverse acceleration at the crest of the embankment, PGA_{crest}, using the PGA_M from step two; and
4. Adjust the PGA_{crest} using the method developed by Makdisi and Seed (1977) to account for the variation of induced average acceleration with embankment depth to calculate the seismic coefficient.

Each of these steps is discussed in more detail in Appendix B. The pseudostatic analyses incorporated a horizontal seismic coefficient of 0.083g, corresponding to the calculated, adjusted PGA_{crest} value.

The water level in the CWTP for the seismic loading analysis was set to EL 5332.6 ft-NAVD88 to match the long-term, maximum storage pool. For the seismic loading condition, effective shear strength parameters summarized in Table 5-1 were used for free-draining soils (bottom ash) and total shear strength parameters summarized in Table 5-1 were used for low-permeability soils (embankment fill and weathered shale) because it is anticipated that they would behave in an undrained manner due to the relatively rapid loading induced during the seismic event.

Liquefaction: The liquefaction factor of safety is evaluated for CCR embankments that show, through representative soil sampling, construction documentation, or anecdotal evidence from personnel with knowledge of construction of the CCR units, that soils of the embankment or foundation are susceptible to liquefaction.

During the 2016 AECOM Geotechnical Investigation, relatively low-density bottom ash was encountered beneath the embankment in boring CWTP-4 that was considered potentially susceptible to liquefaction. Therefore, an SPT-based liquefaction triggering analysis was performed for each of the four CWTP embankment borings drilled during the investigation in accordance with the liquefaction triggering procedures specified by Idriss and Boulanger (2008). The liquefaction triggering calculation is included in Appendix C. The results indicate that the calculated factors of safety against the triggering of liquefaction are greater than 2.0 for all boring locations. Generally, factors of safety less than 1.0 indicate soils that are likely to liquefy during the design level seismic event. Since the calculated factors of safety are well above this minimum threshold, the soils of the CWTP embankment and foundation are considered not susceptible to liquefaction and therefore the liquefaction factor of safety loading condition was not considered for the CWTP.

5.7 Safety Factor Assessment Results

Table 5-2. summarizes the results of the safety factor analysis for the CWTP, for a more detailed discussion of the results see the safety factor calculation presented in Appendix B.

Table 5-2. Summary of Calculated Safety Factors

Loading Condition[1]	Required Safety Factor[2]	Calculated Safety Factor	
		Section 1	Section 2
Long-term, maximum storage pool	1.50	1.74	1.64
Maximum surcharge pool	1.40	1.65	1.54
Seismic	1.00	1.20	1.20

Notes: 1) Rapid Drawdown Loading Condition not considered per Section 4.7
2) CCR Final Rule (EPA, 2015), 40 CFR § 257.73

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

6 Conclusions

Based on the findings and results of the structural integrity assessment, AECOM provides the following conclusions for the CWTP at the FCPP.

- The CWTP is classified as a Low Hazard Potential CCR surface impoundment.
- The embankment is founded on stable foundations and abutments.
- The embankment has adequate slope protection consisting of face stone over a majority of the slope; however, the upper portion of the downstream slope requires installation of erosion protection where excess vegetation has been removed leaving the slope unprotected.
- Based on the available information from the 2016 AECOM Investigation, the CWTP embankment materials are sufficiently dense to withstand the range of loading conditions anticipated at the site.
- The spillway at the CWTP impoundment is capable of adequately managing the flow during and following the peak discharge from the 100-year, 24-hour storm event.
- The crest elevation of the CWTP embankment is currently about one foot below its design elevation of EL 5,338 ft-NAVD (EL 5,335 ft-NGVD).
- Factors of safety greater than the minimum values required by the CCR Rule were calculated for two cross sections along the CWTP embankment for loading conditions associated with the maximum storage pool water level, maximum surcharge pool water level, and design seismic event. Based on a liquefaction triggering analysis that showed the embankment and foundation materials are unlikely to liquefy during the design seismic event, liquefaction stability of the impoundment was not evaluated. Rapid drawdown stability of the CWTP embankment was also not considered, due to the inability of the downstream body of water, the cooling water return canal and hydraulically connected Morgan Lake, to drain rapidly.
- Based on review of limited available records concerning the CWTP and the results of the stability analyses, no deficiencies were noted that would affect the structural condition of the dam.

AECOM recommends the following activities to correct noted interim conditions (see Section 4.9) present at the CWTP.

- Installation of erosion protection along the downstream slope of the CWTP embankment. As AECOM understands, planning for a slope protection system is already being conducted by APS with anticipated installation in 2017.
- Restoration of the embankment crest to the design elevation of 5,335 ft NGVD29 (5,338 ft NAVD88) to reestablish the design freeboard of the CWTP pond.

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

AECOM and Arizona Public Services Company (APS), 2016, “Four Corners Power Plant Lined Ash Impoundment, Lined Decant Water Pond, Combined Waste Treatment Pond, Upper Retention Sump, Dry Fly Ash Disposal Area: Annual CCR Impoundment and Landfill Report 2015”, January.

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United States Geological Survey (USGS), 2013, 7.5-Minute Series Fruitland, New Mexico Quadrangle Map.

Figures

Last saved by: KINGJ3(2016-07-01) Last Plotted: 2016-07-11
Filename: P:\ARIZONA PUBLIC SERVICE\60445844_APS_FCPP_STRUCTURAL_INTEGRITY\STRUCTURAL INTEGRITY REPORTS\CADD\FIGURE 1-1_SITE VICINITY MAP.DWG

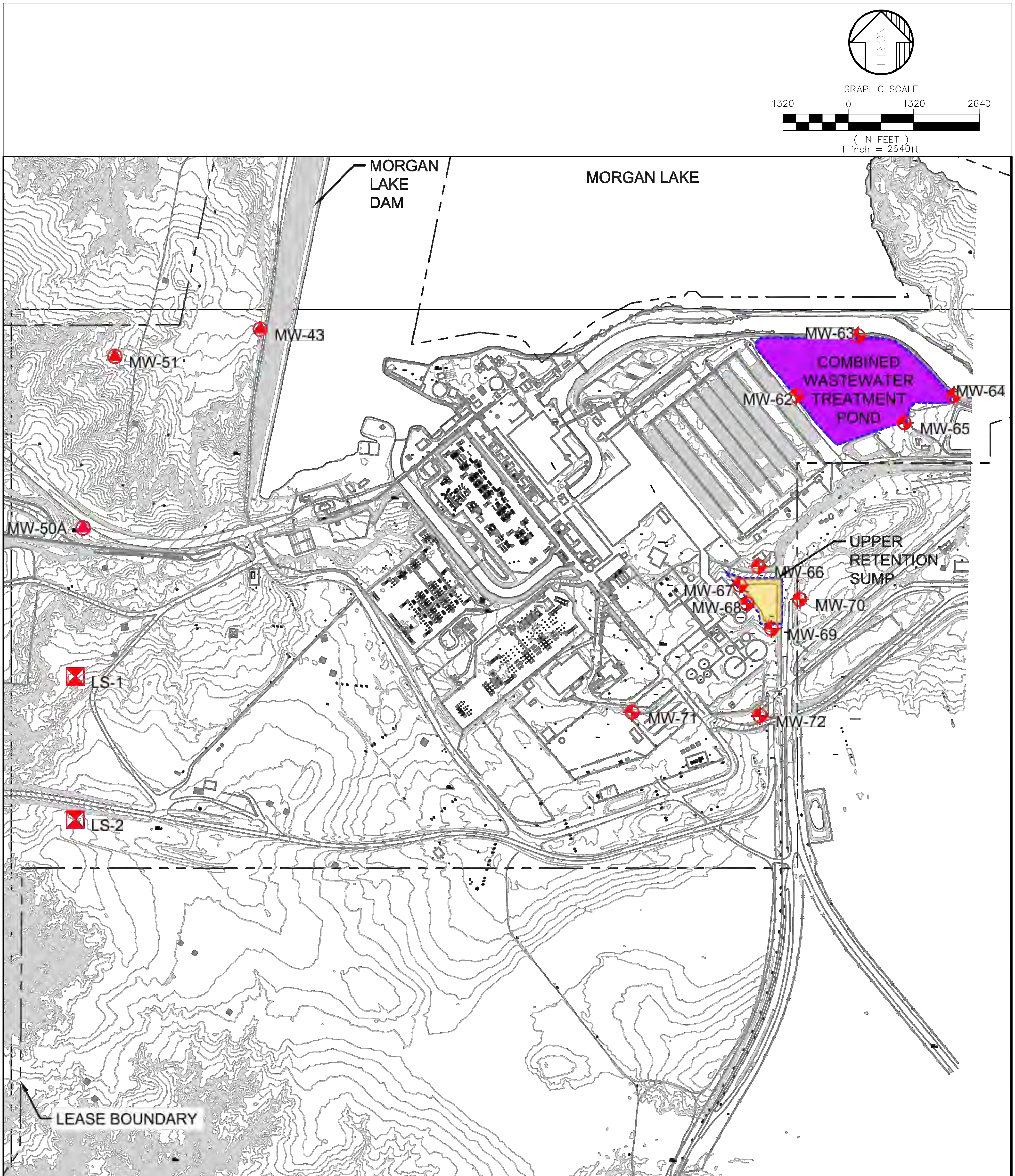
FOUR CORNERS POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445844

SITE VICINITY MAP

AECOM

FIGURE 1-1

Last saved by: KINGJ3(2016-06-30) Last Plotted: 2016-07-01
Filename: P:\ARIZONA PUBLIC SERVICE\60445844_APS_FCPP_STRUCTURAL_INTEGRITY\STRUCTURAL INTEGRITY REPORTS\CADD\FIGURE 1-2_INSTRUMENTATION.DWG



FOUR CORNERS POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445844

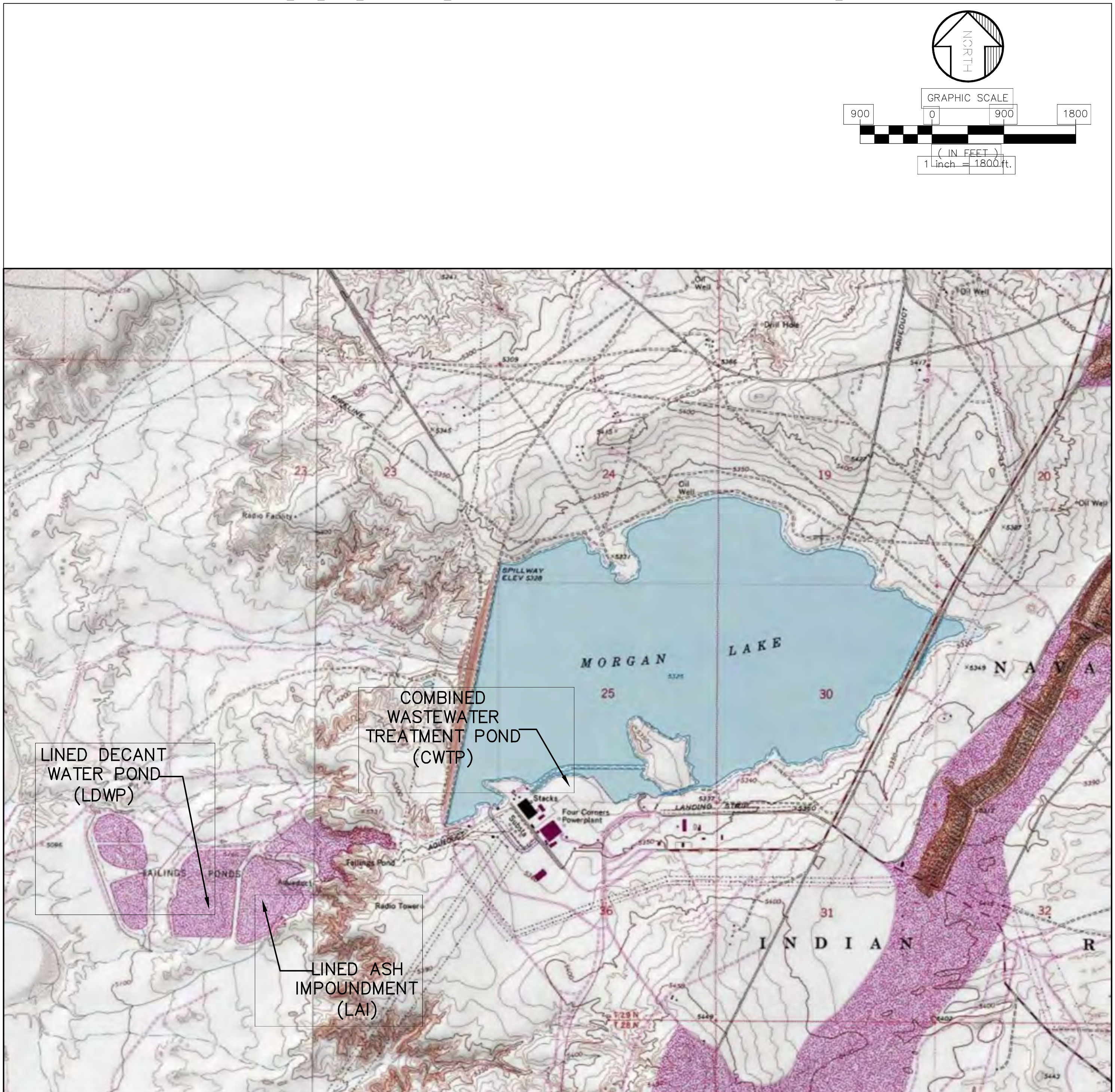
New CCR Monitor Well
Existing CCR Monitor Well
Existing Site Monitoring Well
Overhead Electric Line Tower
CWTP
Upper Retention Sump

CWTP MONITORED INSTRUMENTATION LOCATION MAP

AECOM

FIGURE 1-2

Last saved by: KINGJ3(2016-06-30) Last Plotted: 2016-07-01
Filename: P:\ARIZONA PUBLIC SERVICE\60445844_APS_FCPP_STRUCTURAL_INTEGRITY\STRUCTURAL INTEGRITY REPORTS\CADD\FIGURE 4-1_TOPOGRAPHY.DWG



FOUR CORNERS POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445844

SITE TOPOGRAPHIC MAP

FIGURE 3-1

Last saved by: ALEISA_KRUG(2016-08-09) Last Plotted: 2016-08-11 ANSI A 8.5" x 11"
Filename: P:\ARIZONA PUBLIC SERVICE\60445844_APS_FCPP_STRUCTURAL_INTEGRITY\STRUCTURAL INTEGRITY REPORTS\CADD\FIGURE 5-1 CROSS SECTIONS LOCATIONS SFA - CWTP.DWG

FOUR CORNERS POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445844

CROSS SECTION LOCATIONS
SAFETY FACTOR ASSESSMENT

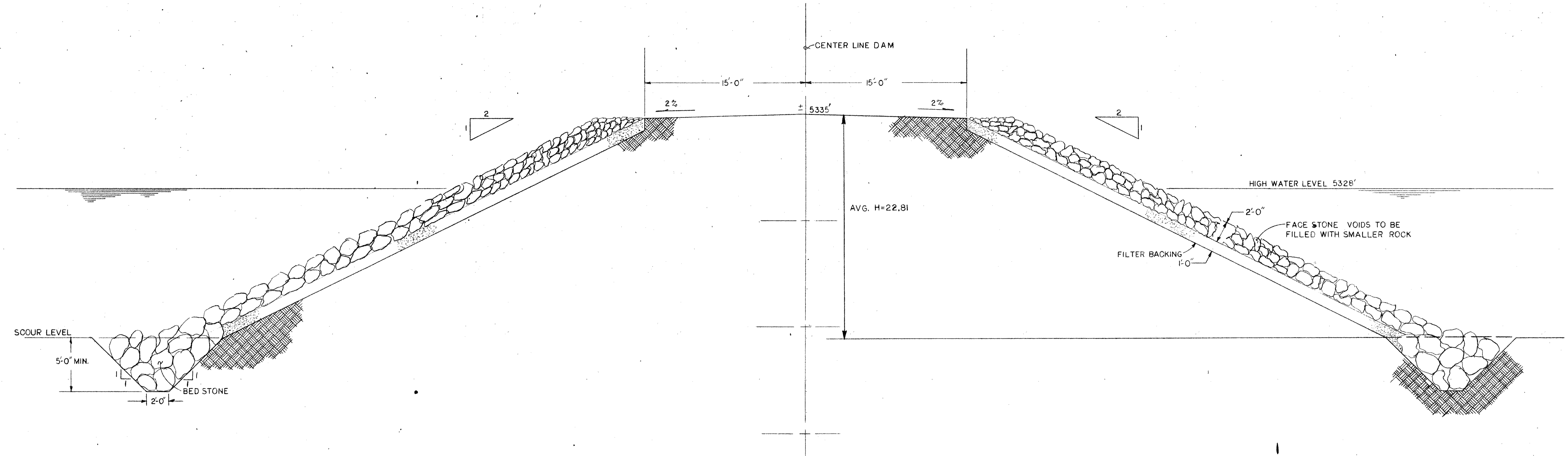
AECOM

FIGURE 5-1

Appendix A.
Historic Drawings

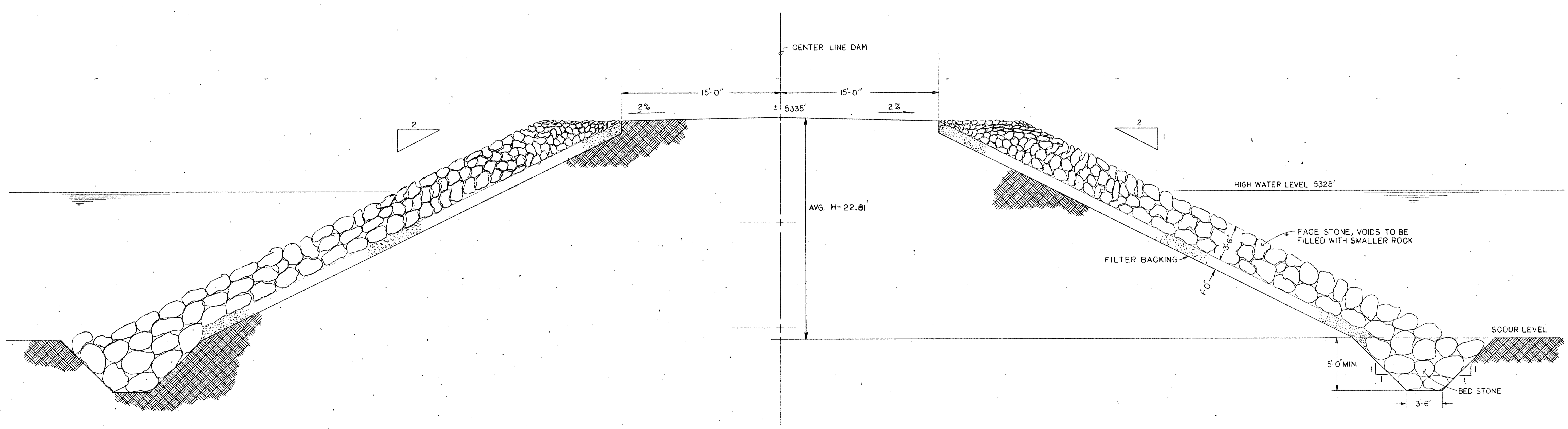
ORIGINAL DRAWINGS

(APS, 1978 and 1979)



TYPICAL TANGENT SECTION

APPROX. LIMITS: STA. 0+00 TO 9+38.244
℄ 11+95 TO 18+00



TYPICAL CURVED SECTION

APPROX. LIMITS: STA. 9+38.224 TO 11+95.773

MEAN STREAM VELOCITY	PARALLEL FLOW ALONG TANGENT BANK					IMPINGEMENT FLOW AGAINST CURVED BANK				
	CURRENT VELOCITY	MINIMUM STONE	PROTECTION CLASS	PLACEMENT METHOD	SECTION THICKNESS	CURRENT VELOCITY	MINIMUN STONE	PROTECTION CLASS	PLACEMENT METHOD	SECTION THICKNESS
10.5 FPS	7FPS	7 LB	FACING	B	2 FT	14 FPS	430 LB	1/2 TON	A	3.5 FT

NOTE:— ADD 20% SURCHARGE TO BORROW. (DAM EMBANKMENT ONLY)

REF. - G-67227- 5 OF 5

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

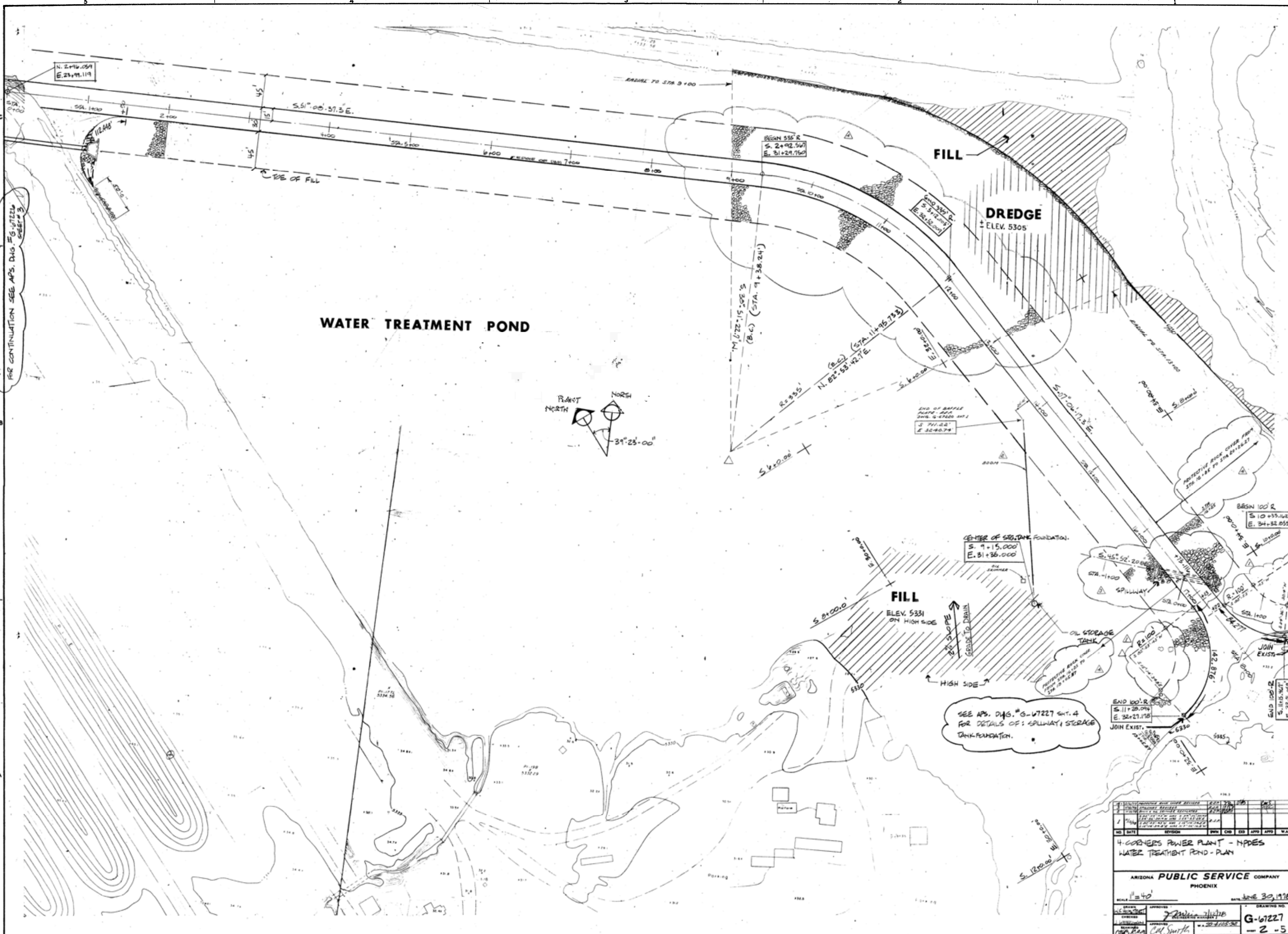
4 CORNERS POWER PLANT - NPDES
WATER TREATMENT POND DAM
TYPICAL SECTIONS

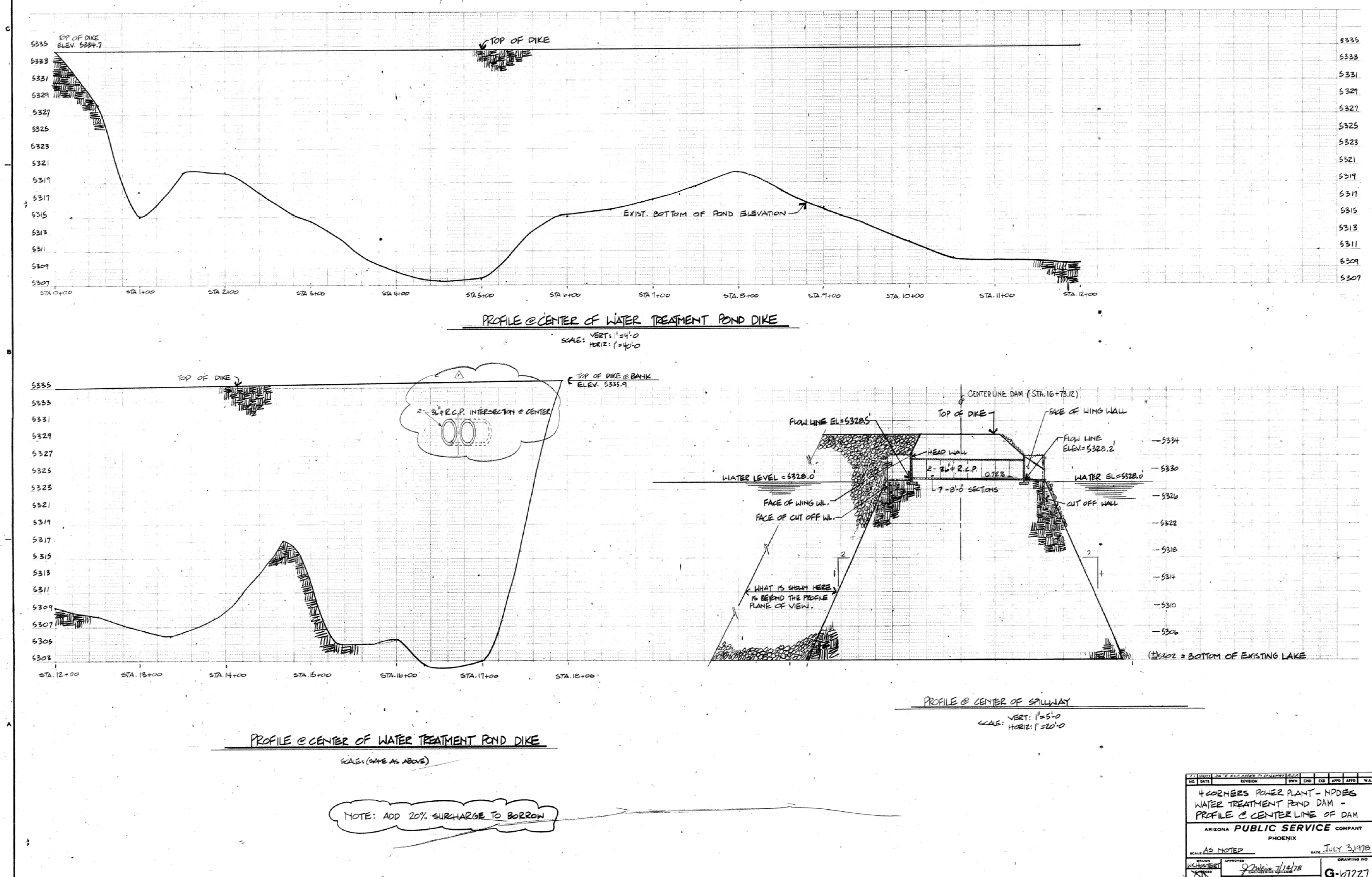
ARIZONA **PUBLIC SERVICE** COMPANY
PHOENIX

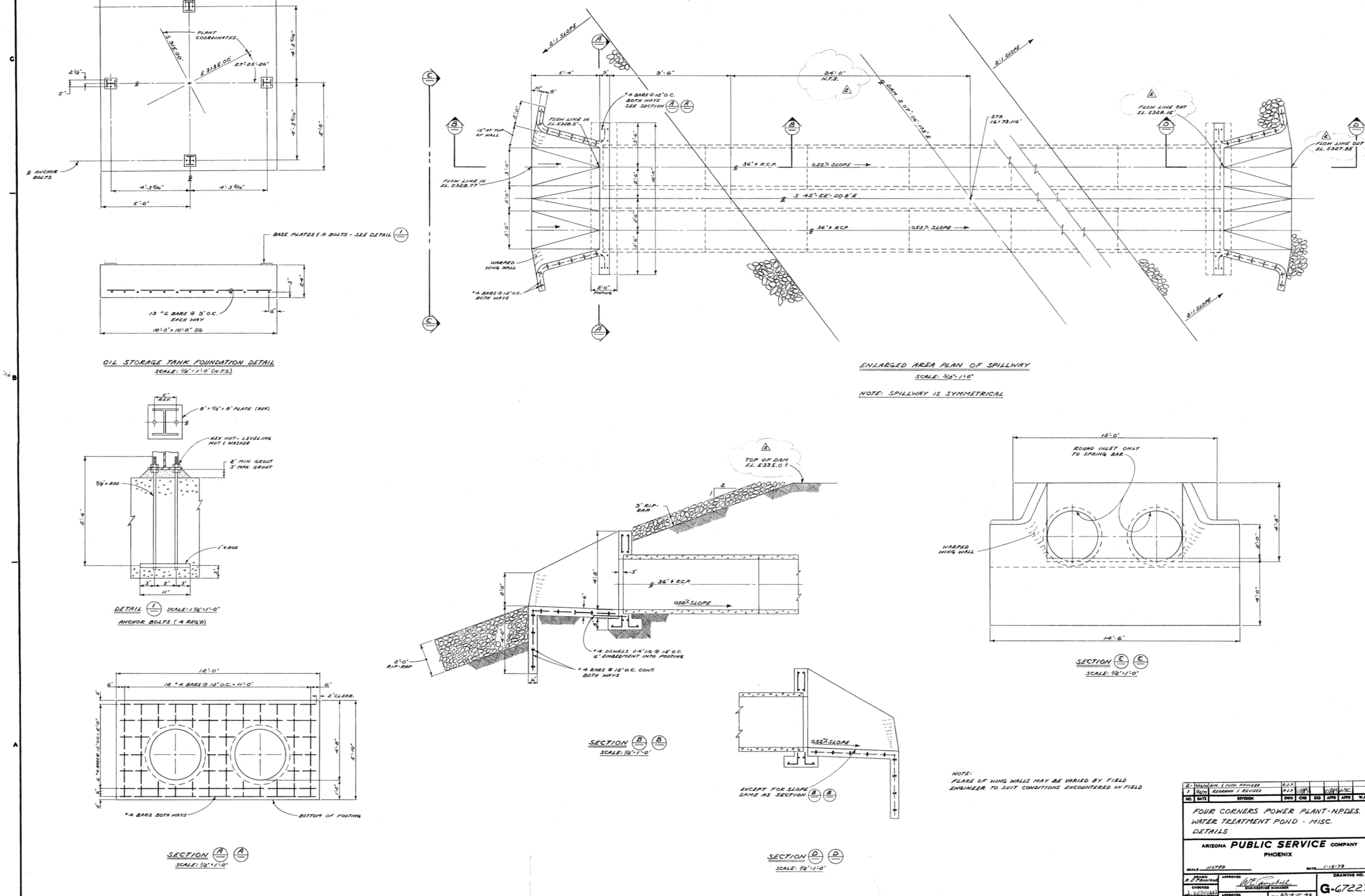
SCALE 1/4" = 1' 0" DATE 6-8-78

DRAWN RSB
CHECKED
EXAMINED OTIB
APPROVED J. Weir 7/14/78 ENGINEERING MANAGER
APPROVED
W.A. 99-4-105-92
DRAWING NO. G- 67227
SHEET 1 OF 5

"INDEX"


WATER TREATMENT POND
FILL
DREDGE
ELEV. 5305
FILL
ELEV. 5331 ON HIGH SIDE
HIGH SIDE
GRADE TO DRAIN
OIL STORAGE TANK
SPILLWAY
PLANT NORTH
NORTH
39°23'-00"
TOE OF FILL
FOR CONTINUATION SEE APS. DWG. #G-67226 SHEET #3
CENTER OF STG. TANK FOUNDATION. S. 9+15.000 E. 31+36.000
SEE APS. DWG. #G-67227 SHT. 4 FOR DETAILS OF: SPILLWAY, STORAGE TANK FOUNDATION.
JOIN EXIST.
4-CORNERS POWER PLANT - NPDES WATER TREATMENT POND - PLAN
ARIZONA PUBLIC SERVICE COMPANY
PHOENIX
G-67227
2 - 5


TOP OF DIKE ELEV. 5334.7
TOP OF DIKE
EXIST. BOTTOM OF POND ELEVATION
STA 0+00
STA 1+00
STA 2+00
STA 3+00
STA 4+00
STA 5+00
STA 6+00
STA 7+00
STA 8+00
STA 9+00
STA. 10+00
STA. 11+00
STA. 12+00
PROFILE @ CENTER OF WATER TREATMENT POND DIKE
SCALE: VERT: 1"=4'-0 HORIZ: 1"=40'-0
TOP OF DIKE
TOP OF DIKE @ BANK ELEV. 5335.9
2-36"Ø R.C.P. INTERSECTION @ CENTER
STA. 12+00
STA. 13+00
STA. 14+00
STA. 15+00
STA. 16+00
STA. 17+00
STA. 18+00
PROFILE @ CENTER OF WATER TREATMENT POND DIKE
SCALE: (SAME AS ABOVE)
CENTER LINE DAM (STA. 16+73.12)
TOP OF DIKE
FACE OF WING WALL
FLOW LINE EL=5328.5'
FLOW LINE ELEV=5328.2'
HEAD WALL
2-36"Ø R.C.P.
0.72%
WATER LEVEL = 5328.0'
WATER EL=5328.0'
7-8'-0 SECTIONS
FACE OF WING WL.
FACE OF CUT OFF WL.
CUT OFF WALL
WHAT IS SHOWN HERE IS BEYOND THE PROFILE PLANE OF VIEW.
(±)5302 = BOTTOM OF EXISTING LAKE
PROFILE @ CENTER OF SPILLWAY
SCALE: VERT: 1"=5'-0 HORIZ: 1"=20'-0
NOTE: ADD 20% SURCHARGE TO BORROW
4 CORNERS POWER PLANT - NPDES WATER TREATMENT POND DAM - PROFILE @ CENTER LINE OF DAM
ARIZONA PUBLIC SERVICE COMPANY
PHOENIX
SCALE AS NOTED
DATE JULY 3, 1978
DRAWING NO. G-67227

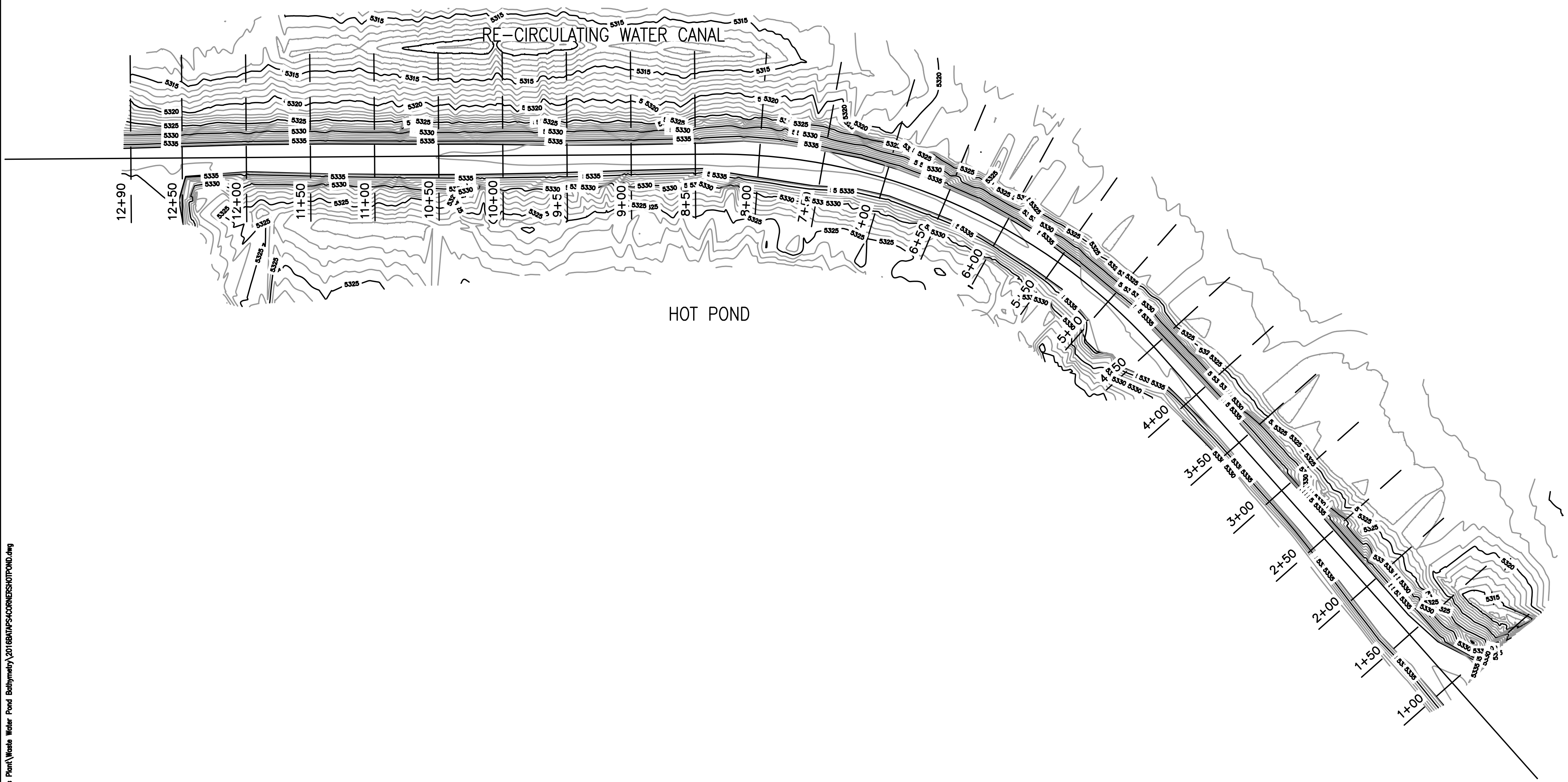

OIL STORAGE TANK FOUNDATION DETAIL
SCALE: 1/2" = 1'-0" (N.T.S.)
ENLARGED AREA PLAN OF SPILLWAY
SCALE: 3/8" = 1'-0"
NOTE: SPILLWAY IS SYMMETRICAL
DETAIL 1 SCALE: 1 1/2" = 1'-0"
ANCHOR BOLTS (4 REQ'D)
SECTION C C
SCALE: 1/2" = 1'-0"
SECTION B B
SCALE: 1/2" = 1'-0"
SECTION A A
SCALE: 1/2" = 1'-0"
SECTION D D
SCALE: 1/2" = 1'-0"
NOTE:
FLARE OF WING WALLS MAY BE VARIED BY FIELD ENGINEER TO SUIT CONDITIONS ENCOUNTERED IN FIELD
FOUR CORNERS POWER PLANT - N.P.D.E.S.
WATER TREATMENT POND - MISC.
DETAILS
ARIZONA PUBLIC SERVICE COMPANY
PHOENIX
G-67227

EMBANKMENT SURVEY

(J. Marbles Land Surveying, LLC, 2016)

BATHYMETRIC SURVEY

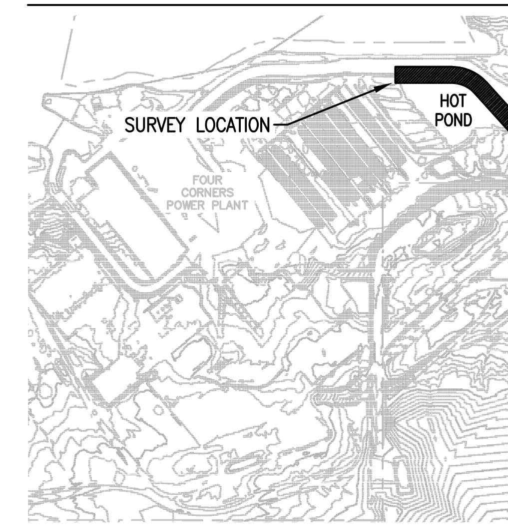
RE-CIRCULATING CANAL AND HOT POND BERM
APS - 4 CORNERS PLANT
03/22/16



NOTES

1. BASIS OF BEARING IS NEW MEXICO STATE PLANE COORDINATE SYSTEM, NORTH AMERICAN DATUM OF 1983 (NAD83) AND THE NORTHERN AMERICAN VERTICAL DATUM OF 1988 (NAVD88) WEST ZONE, US SURVEY FEET.
2. THE 1.5" ALUMINUM CAP IDENTIFIED AS EMMA WAS USED FOR CONTROL. THE COORDINATES OF EMMA ARE:
   N:2,066,529.495
   E:2,528,708.477
   ELEV: 5382.251

SITE MAP



100 50 0 100
SCALE: 1"=100'
IF PRINTED 11x17

Path: C:\JMLS\WORK\APS Criado\4 Corners Plant\Waste Water Pond Bathymetry\2016BATAPS4CORNERSHOTPOND.dwg

J. MARBLES LAND SURVEYING, LLC
"ridiculam aspiciens superuisorem"
P. O. Box 2039 · St. Johns, Arizona 85936 · Cell: (928) 245.7040 · jmarbles2013@gmail.com
Soil Investigation for Standard Septic Systems · FEMA Flood Plain Certifications · Boundary & Land Division Surveys · Topographical & ALTA Surveys · Aerial Control & Site Control Surveys · Bathymetric Surveys

BATHYMETRIC SURVEY
OF THE BERM BETWEEN THE RECIRCULATING WATER CANAL AND THE HOT POND
APS - 4 CORNERS POWER PLANT
03/22/16

RECORD CREATED AT THE REQUEST OF:	AECOM - FRANCES ACKERMAN		
DATE	032216	DESCRIPTION	
REVISION DATE		DESCRIPTION	
REVISION DATE		DESCRIPTION	
FILE:	2016BATAPS4CORNERSHOTPOND.dwg	SHEET NO.	
DRAWN BY: SAW	REVIEWED BY: SAW	1 OF 14	

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	FCPP CWTP SIA	**Client Name**	APS
Project Location	Four Corners Power Plant	**PM Name**	Corey Jablonski
Project Number / Office Code	60445844	**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:	Jed Stoken, P.E.	Comments Required by:	3/8/16
Work Product Originator:	Lee Wright, 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: 3/10/16

Verification

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

Select:

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

or

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

and

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

Checker / Reviewer Signature — Date: 3/10/16

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 8/12/16 Date
DISTRIBUTION	Project Central File – Quality File Folder Other – Specify:

TABLE OF CONTENTS

Figures

Tables

1 INTRODUCTION

The purpose of this calculation is to calculate safety factors for the existing Coal Combustion Residual (CCR) surface impoundments at Arizona Public Service (APS)'s Four Corners Power Plant near Farmington, New Mexico. Specifically, the CCR surface impoundment embankment that will be evaluated is the Combined Water Treatment Pond (CWTP).

2 ANALYSIS CRITERIA

The analyses were performed to meet the regulations set forth in the United States Environmental Protection Agency (EPA) 40 CFR Parts 257.73(e) Structural Integrity Criteria for Existing CCR Impoundments (EPA 2015). The Rule requires safety factor assessments for units containing coal combustion residuals. The safety factors for various embankment loading and tailwater conditions must meet the values outlined in the Rule. For the CWTP, 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
- 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 based on the topographic and bathymetric data collected on March 22, 2016 (J. Marbles Surveying, 2016). Accumulation of sediment in the CWTP was disregarded in the safety factor calculation because the pond is periodically dredged.
- The subsurface stratigraphy was based on borings AECOM drilled along the crest of the CWTP in February 2016 and APS drawing G-67227 Sheet 3 (APS 1978).

- Material properties used in the safety factor assessment were based on laboratory tests conducted on samples of embankment fill recovered during a geotechnical investigation performed for this project and historical laboratory testing and geotechnical analyses conducted for the nearby Lined Ash Impoundment. Material properties were developed in a separate calculation (AECOM, 2016).

- The pore pressure distribution within the embankment was developed from interpretation of the water levels in the pond and the downstream discharge cooling canal.

- The maximum storage pool was based on the measured operating level of the pond, understanding that the pond level is stable over time. The maximum surcharge pool water level was conservatively taken to be the embankment crest elevation.

- The seismic loading 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

Assumptions used in this calculation package include:

- The CWTP and Morgan Lake maintain relatively constant water levels.

- The riprap armoring shown on the as-built drawings was assumed to be no longer present based on field observations made from the crest during the AECOM 2016 site exploration performed in February 2016.

- The samples tested as part of the 2016 AECOM laboratory program are representative of the overall material present at the CWTP embankment.

5 STABILITY ANALYSIS

Safety factor calculations 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

Safety factors were calculated for two cross sections of the CWTP Embankment (shown on Figure 1, attached) selected to represent different embankment geometries, heights, and stratigraphic conditions to provide confidence that the critical cross section was analyzed. 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, side slope configuration, pond and canal depths, and stratigraphic conditions. Subsurface soil profiles were developed using information from the borings presented in Section 2 of this report. The cross sections analyzed are:

CWTP Section 1: This cross section represents the maximum section in the western portion of the CWTP embankment where a layer of low blow-count bottom ash was encountered. The embankment is approximately 22 feet high, with a 1.4H:1V downstream slope and a 1.8H:1V upstream slope. The embankment fill consists of Sandy Lean Clay over approximately 14 feet of bottom ash with bedrock at 36 feet bgs. MCS penetration testing in the upper zone of the bottom ash at the CWTP-4 location resulted in an uncorrected N-value equal to 2 (indicating very loose soil). This condition was represented in the model using a 5-foot thick layer of "Upper Bottom Ash" having a strength lower than the underlying bottom ash (see discussion of material properties in Section 3.3).

CWTP Section 2: This cross section represents the maximum section in the eastern portion of the CWTP embankment having the steepest downstream slope. The embankment is approximately 20 feet high, with a 1.5H:1V downstream slope and a 1.9H:1V upstream slope. The embankment fill consists of Sandy Lean Clay over approximately 3 feet of bottom ash with bedrock at 24 feet bgs. No SPT blow counts were recorded for the limited thickness of the bottom ash in CWTP-2. Therefore, the bottom ash material in CWTP Section 2 was represented in the model as "Upper Bottom Ash" and assigned a lower strength than the underlying Bottom Ash (see discussion of material properties in Section 3.3).

The continuity of the facing slabs observed below the water surface and on the design drawings of the embankment slopes in March 2016 is unknown. Therefore, this layer was, conservatively, not included in the stability models.

5.2 Material Properties

A material properties calculation package was prepared to present the methods and information supporting the parameter selection for the embankment fill, bottom ash, and bedrock foundation at the CWTP. The material properties identified in the calculation and used in the safety factor assessment are presented in Table 1 below.

Table 1 – Material Properties Used for the Safety Factor Assessment

Material	Saturated Unit Weight, γsat (pcf)	Moist Unit Weight, γm (pcf)	Drained Strengths: Effective Friction Angle, ϕ' (degrees)	Drained Strengths: Effective Cohesion, c' (psf)	Undrained Strengths: Total Friction Angle, ϕ (degrees)	Undrained Strengths: Total Cohesion, c (psf)
Embankment Fill	130	115	36	175	19	250
Upper Bottom Ash	91	80	31	0	-	-
Bottom Ash	91	80	39	0	--	--
Bedrock	125	120	--	--	20	600

5.3 Embankment Pore Pressure Distribution

Per 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.

Water levels were measured in the CWTP and the discharge cooling canal in December, 2015 (URS, 2016). These measurements were considered to be the most reliable indicators of pore pressure distribution within the CWTP embankment. The pore pressure distribution in the embankment was estimated using water level measurements and assuming a straight line phreatic surface between the water level in the pond in the pond and in the canal. The water level in the CWTP is controlled through the positioning of gates on the discharge pipes and was measured at 5,332.6 feet in December 1, 2015; the water level is reportedly relatively constant over time. The canal is hydraulically connected to Morgan Lake via open-channel flow; the downstream water level was assumed to be relatively constant at 5329.5 feet, measured on December 1, 2015 (AECOM, 2016).

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 loading.

The three loading conditions are described below.

Long-Term, Maximum Storage Pool

The maximum storage pool loading is the maximum water level that can be maintained that will result in the full development of a steady-state seepage condition. This loading condition is evaluated to document whether the CCR surface impoundment can withstand a 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 lowest elevation of the invert of the spillway (i.e., the lowest overflow point of the perimeter of the embankment) using shear strengths expressed as effective stress and with pore water pressures that correspond to the long-term condition.

The long-term, maximum storage pool in the CWTP is 5332.6 feet based on data AECOM recorded in December 2015 during field activities. Drained shear strengths (effective stress) parameters summarized in Table 1 were used for the long-term, maximum storage pool loading condition based on Corps of Engineers recommendations (USACE, 2003).

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 inflow design flood state. This loading condition is evaluated to document whether the CCR surface impoundment 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 which persists for a length of time sufficient for steady-state seepage or hydrostatic conditions to fully develop within the embankment.

The maximum surcharge pool in the CWTP was conservatively assumed to be at the crest elevation (5336.5 feet) to coincide with a complete loss of freeboard. Drained shear strengths (effective stress) parameters summarized in Table 1 were used for the maximum surcharge pool loading condition based on Corps of Engineers recommendations (USACE, 2003).

Seismic Loading

Seismic loading was evaluated to document whether the CCR surface impoundment is 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 pseudostatic analysis was used to represent the seismic loading.

The peak horizontal bedrock acceleration for a site classification of B “Rock”, based on the United States Geological Survey (USGS) National Seismic Hazard Map – 2008 mapping with a 2% probability of exceedance in 50 years, is 0.05895g as presented in Attachment A (USGS, 2008). A site classification of C was assigned to the site according to Figure 2 and based on the presence of up to 21 feet of fill over the bedrock surface in the vicinity of the CWTP embankment.

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	$2{,}500 < \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
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 $w \geq 40\%$, 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 1615.1.1 Site Class Definitions (IBC, 2003)

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

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

$$PGA_M = 1.2(0.0807g)$$

$$PGA_M = 0.0707g$$

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 G-2)

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 (NEHRP, 2009)

The PGA at the ground surface for Site Class C was used to estimate the peak transverse acceleration at the crest of the embankment, PGA_{crest} = 0.243g, as shown on Figure G-3 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).

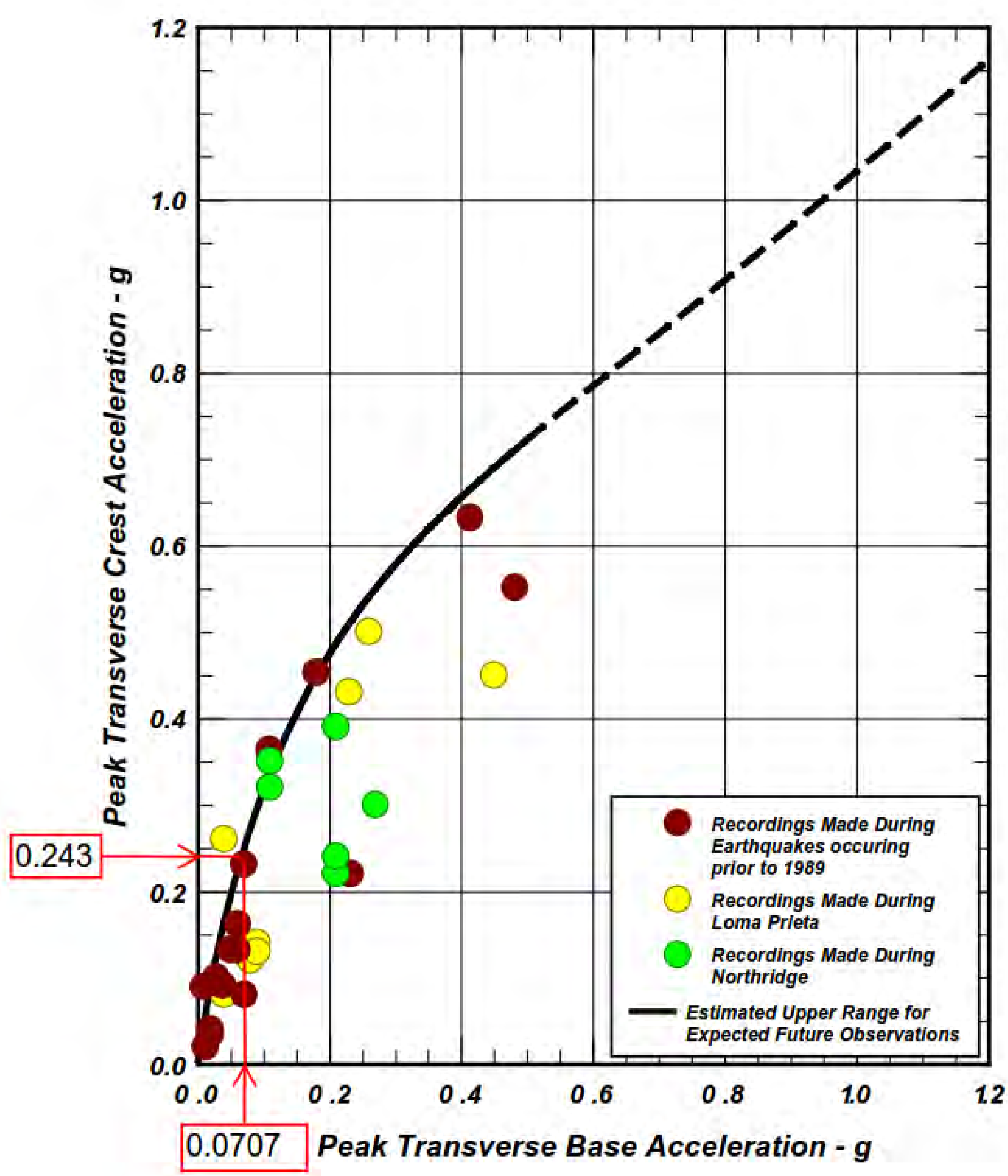


Figure 4 - Variations of Peak Transverse Crest Acceleration v. Peak Transverse Base Acceleration Based on Holzer (1998)

Finally, the PGA_{crest} was adjusted because the “maximum acceleration ratio” varies with the depth of the sliding mass relative to the embankment height (Makdisi and Seek, 1977). Figure G-4 (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:

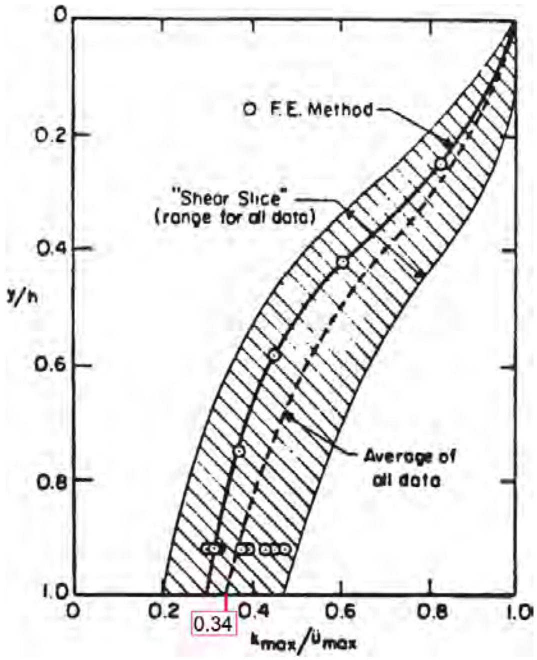


FIG. 9 VARIATION OF "MAXIMUM ACCELERATION RATIO" WITH DEPTH OF SLIDING MASS

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

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

$$k_{max} = 0.34(0.243g)$$

$$k_{max} = 0.10438g \approx 0.083g$$

The pseudostatic analyses incorporated a horizontal seismic coefficient of 0.083g.

The water level in the CWTP for the seismic loading analysis was set to EL 5332.6 feet to match the long-term, maximum storage pool. Drained shear strengths (effective stress) parameters summarized in Table 1 were used for free-draining soils (bottom ash) undrained shear strengths (total stress) parameters summarized in Table 1 were used for low-permeability soils (embankment fill, weathered shale) for the seismic loading condition based on Corps of Engineers recommendations (USACE, 2003).

Liquefaction Loading

Liquefaction loading was evaluated and documented in a separate calculation package.

6 ANALYSIS RESULTS AND CONCLUSIONS

The results of the safety factor assessment are presented in Attachment B. Table 2 summarizes the results of the safety factor assessment.

Table 2 – Safety Factor Results

Loading Condition	Required Factor of Safety	Calculated Factor of Safety	
		Section 1	Section 2
Long-term, maximum storage pool	1.50	1.74	1.64
Maximum surcharge pool	1.40	1.65	1.54
Seismic – Mohr-Coulomb Parameters	1.00	1.20	1.20
Liquefaction Triggering	1.20	> 2.0	

7 REFERENCES

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8 ATTACHMENTS

ATTACHMENT A USGS 2008 Seismic PSH Deaggregation

ATTACHMENT B SLOPE/W Output Figures

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Filename: P:\PROJECTS\ARIZONA_PUBLIC_SERVICE\60445844_FCSIAINS\5_TECHNICAL\5_6_CADD\FIGURES\A21264.DWG

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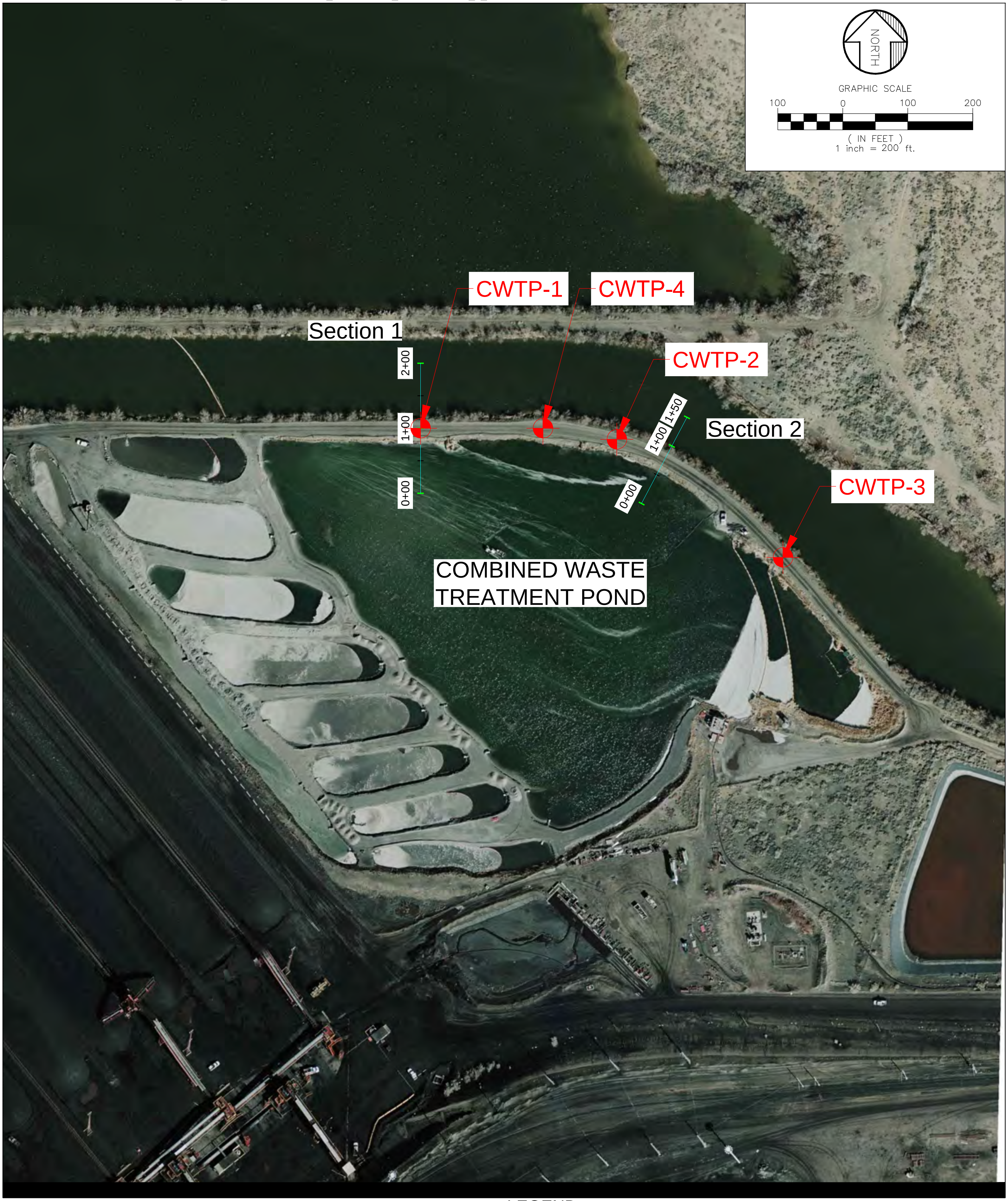


FOUR CORNERS POWER PLANT
SAFETY FACTOR ASSESSMENT REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445844

LEGEND
2016 AECOM BORING LOCATION (CWTP-#)

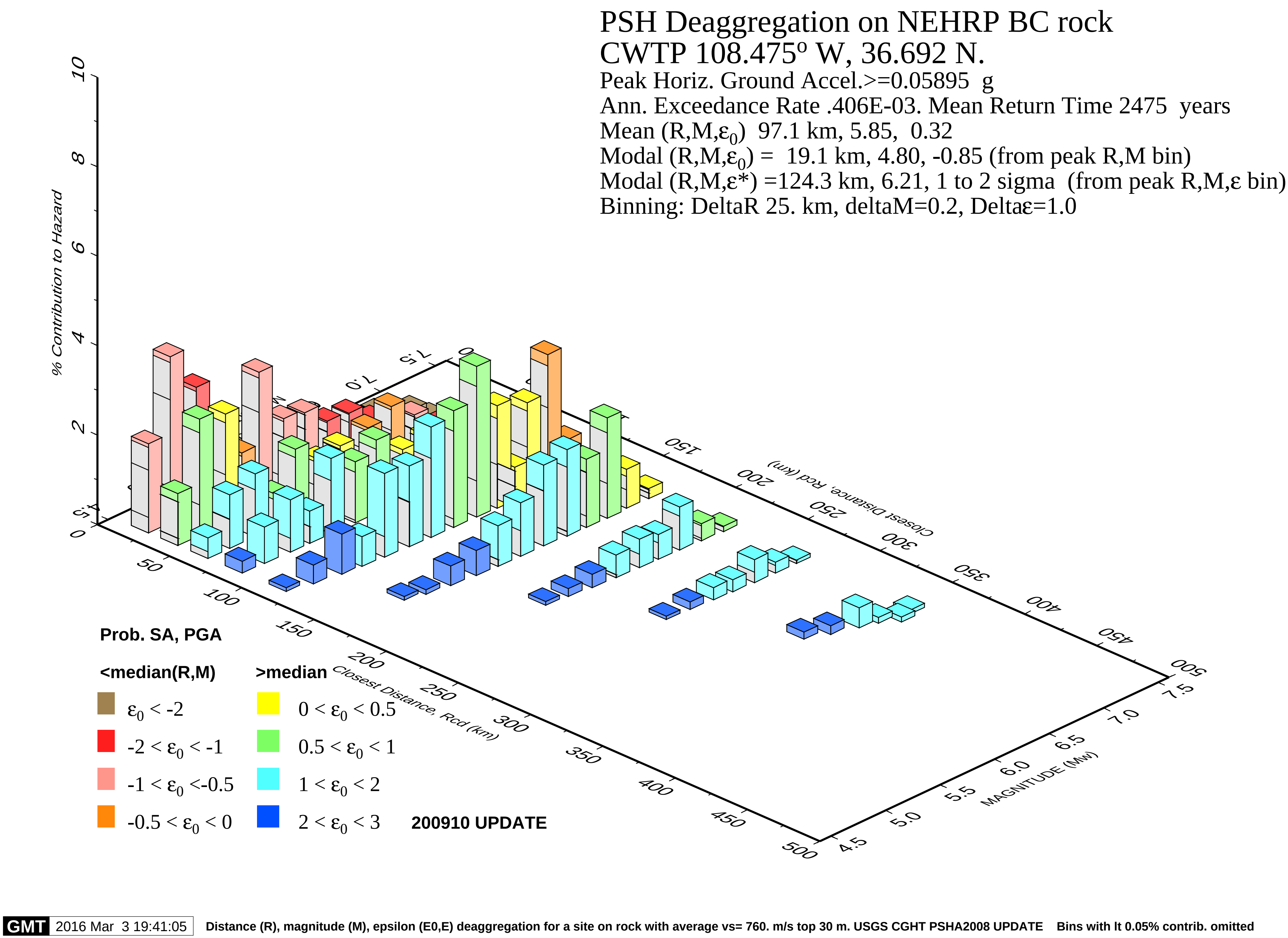
AECOM

COMBINED WASTE TREATMENT POND (CWTP) WITH BORING AND SECTION LOCATIONS

FIGURE 1

ATTACHMENT A

USGS 2008 Seismic PSH Deaggregation


PSH Deaggregation on NEHRP BC rock
CWTP 108.475° W, 36.692 N.
Peak Horiz. Ground Accel.>=0.05895 g
Ann. Exceedance Rate .406E-03. Mean Return Time 2475 years
Mean (R,M,ε0) 97.1 km, 5.85, 0.32
Modal (R,M,ε0) = 19.1 km, 4.80, -0.85 (from peak R,M bin)
Modal (R,M,ε*) =124.3 km, 6.21, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 25. km, deltaM=0.2, Deltaε=1.0
% Contribution to Hazard
Closest Distance, Rcd (km)
MAGNITUDE (Mw)
Prob. SA, PGA
<median(R,M)
>median
ε0 < -2
-2 < ε0 < -1
-1 < ε0 <-0.5
-0.5 < ε0 < 0
0 < ε0 < 0.5
0.5 < ε0 < 1
1 < ε0 < 2
2 < ε0 < 3
200910 UPDATE
GMT 2016 Mar 3 19:41:05
Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted

ATTACHMENT B

SLOPE/W Output Figures

Slope Stability Analysis
Section 1
Combined Waste Treatment Pond

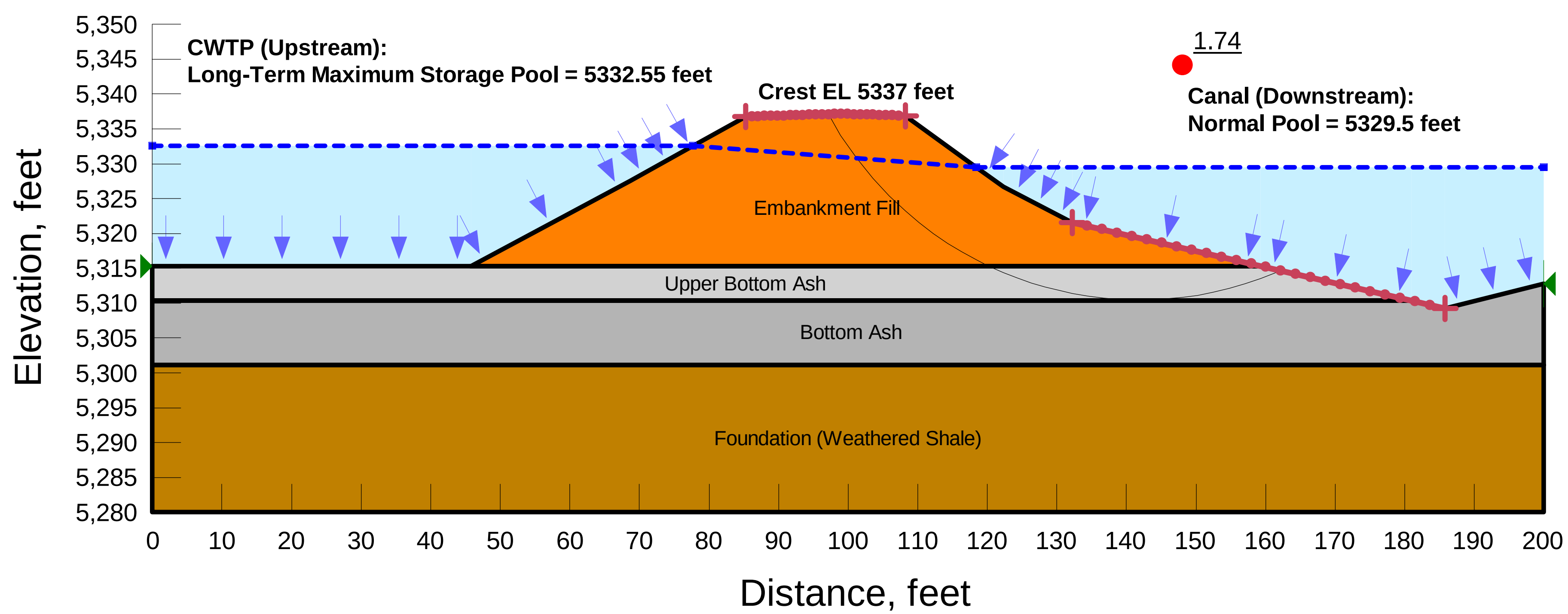
Four Corners Power Plant
Arizona Public Service

Note:
The results of the 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.

Long-Term Maximum Storage Pool - Downstream Slope
File Name: APS CWTP Section 1 final.gsz
Date: 4/7/2016
Method: Spencer

Factor of Safety: 1.74

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Embankment Fill	130 pcf	108 pcf	100 psf	36 °
Bottom Ash	97.5 pcf	60 pcf	0 psf	39 °
Foundation (Weathered Shale)	125 pcf	120 pcf	600 psf	20 °
Upper Bottom Ash	97.5 pcf	60 pcf	0 psf	31 °



Slope Stability Analysis
Section 1
Combined Waste Treatment Pond

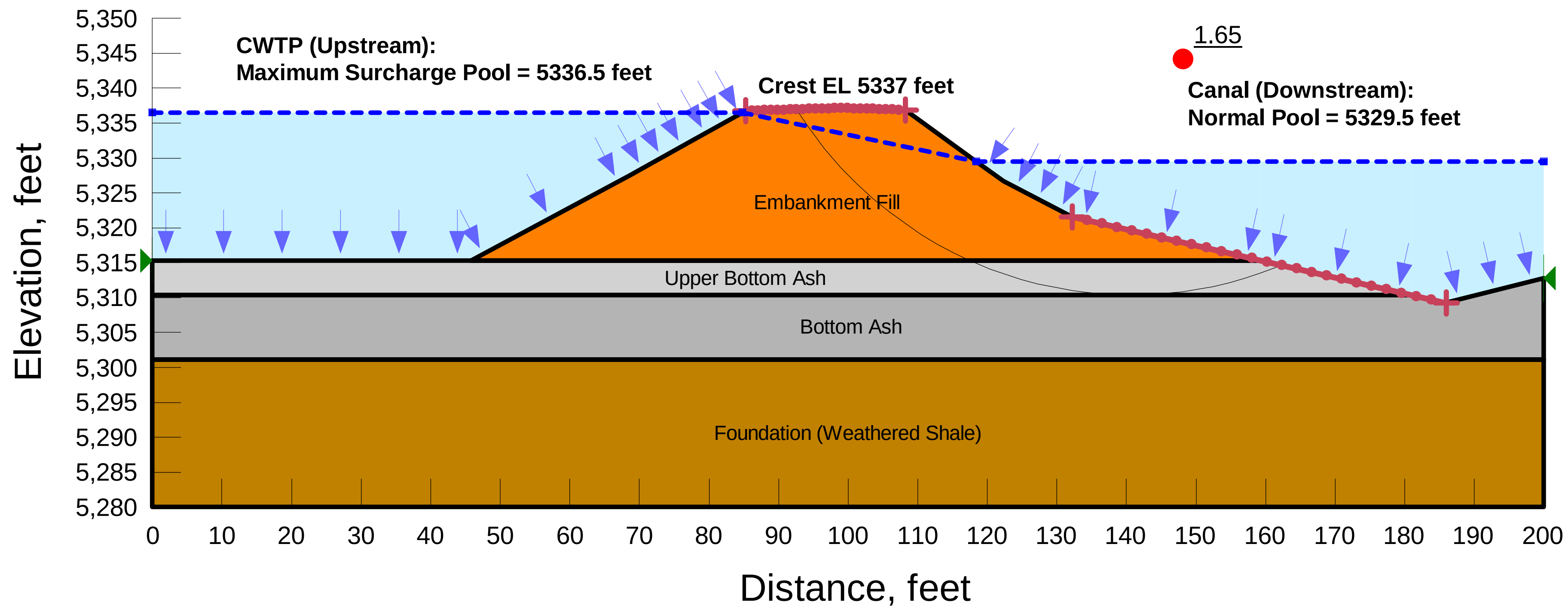
Four Corners Power Plant
Arizona Public Service

Note:
The results of the 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.

Maxium Surcharge Pool - Downstream Slope
File Name: APS CWTP Section 1 final.gsz
Date: 4/7/2016
Method: Spencer

Factor of Safety: 1.65

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Embankment Fill	130 pcf	108 pcf	100 psf	36 °
Bottom Ash	97.5 pcf	60 pcf	0 psf	39 °
Foundation (Weathered Shale)	125 pcf	120 pcf	600 psf	20 °
Upper Bottom Ash	97.5 pcf	60 pcf	0 psf	31 °



Slope Stability Analysis
Section 1
Combined Waste Treatment Pond

Four Corners Power Plant
Arizona Public Service

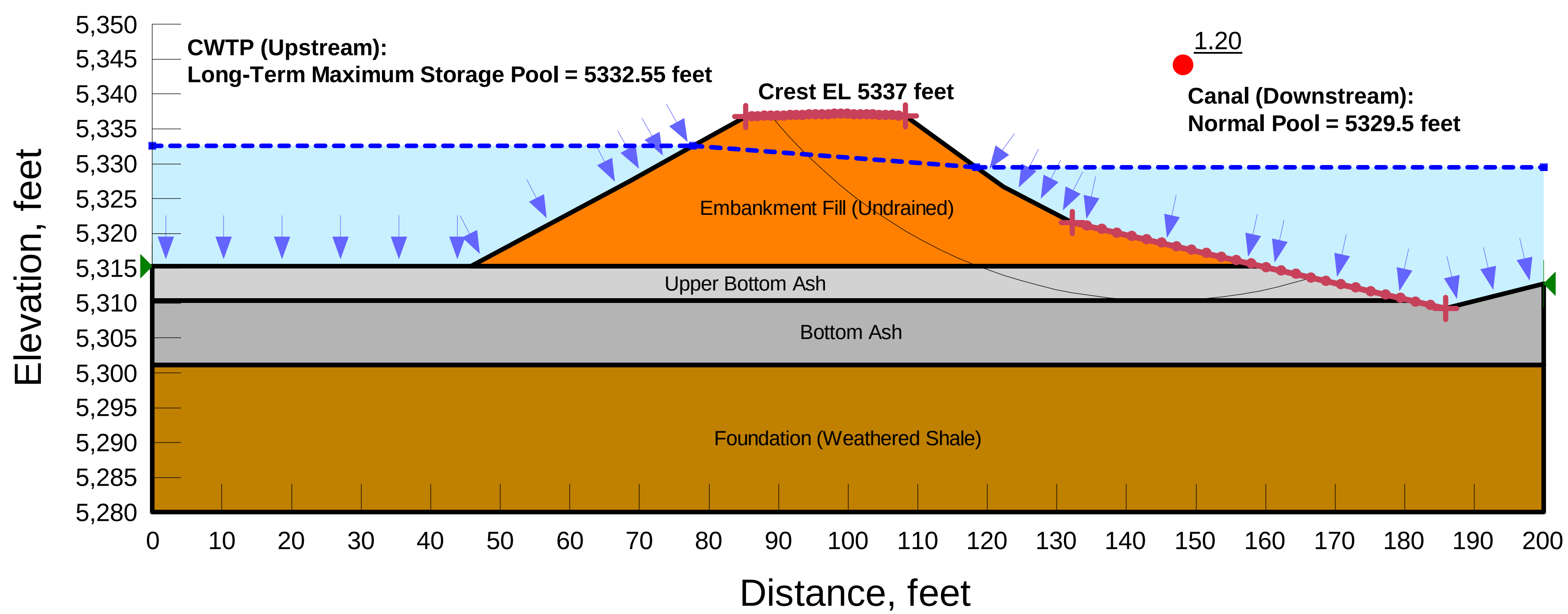
Note:
The results of the analysis shown here are based on available subsurface information, laboratory test results, and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Pseudostatic Loading - Downstream Slope - Mohr Coulomb
File Name: APS CWTP Section 1 final.gsz
Date: 4/7/2016
Method: Spencer

Factor of Safety: 1.20

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Bottom Ash	97.5 pcf	60 pcf	0 psf	39 °
Foundation (Weathered Shale)	125 pcf	120 pcf	600 psf	20 °
Embankment Fill (Undrained)	130 pcf	108 pcf	250 psf	19 °
Upper Bottom Ash	97.5 pcf	60 pcf	0 psf	31 °

Horz Seismic Coef.: 0.083



Slope Stability Analysis
Section 2
Combined Waste Treatment Pond

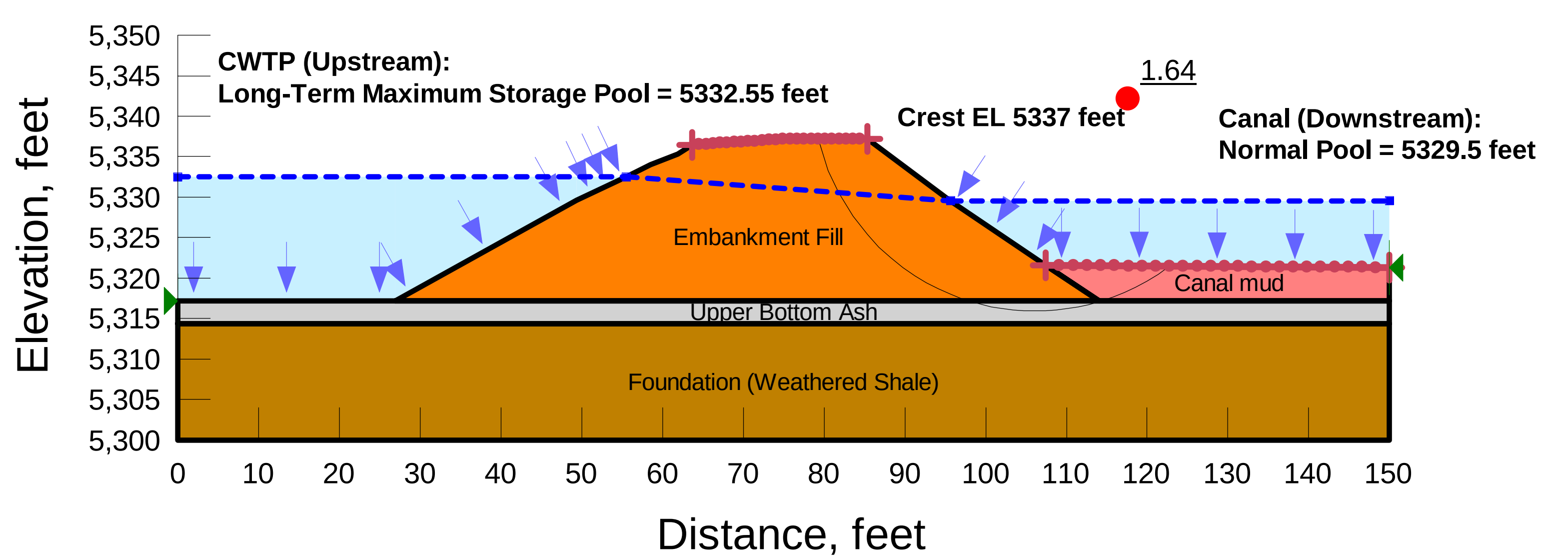
Four Corners Power Plant
Arizona Public Service

Note:
The results of the 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.

Long-Term Maximum Storage Pool - Downstream Slope
File Name: APS CWTP Section 2 final.gsz
Date: 4/7/2016
Method: Spencer

Factor of Safety: 1.64

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Embankment Fill	130 pcf		100 psf	36 °
Foundation (Weathered Shale)	125 pcf	108 pcf	600 psf	20 °
Upper Bottom Ash	97.5 pcf	120 pcf	0 psf	31 °
Canal mud	100 pcf	60 pcf	0 psf	0 °



Slope Stability Analysis
Section 2
Combined Waste Treatment Pond

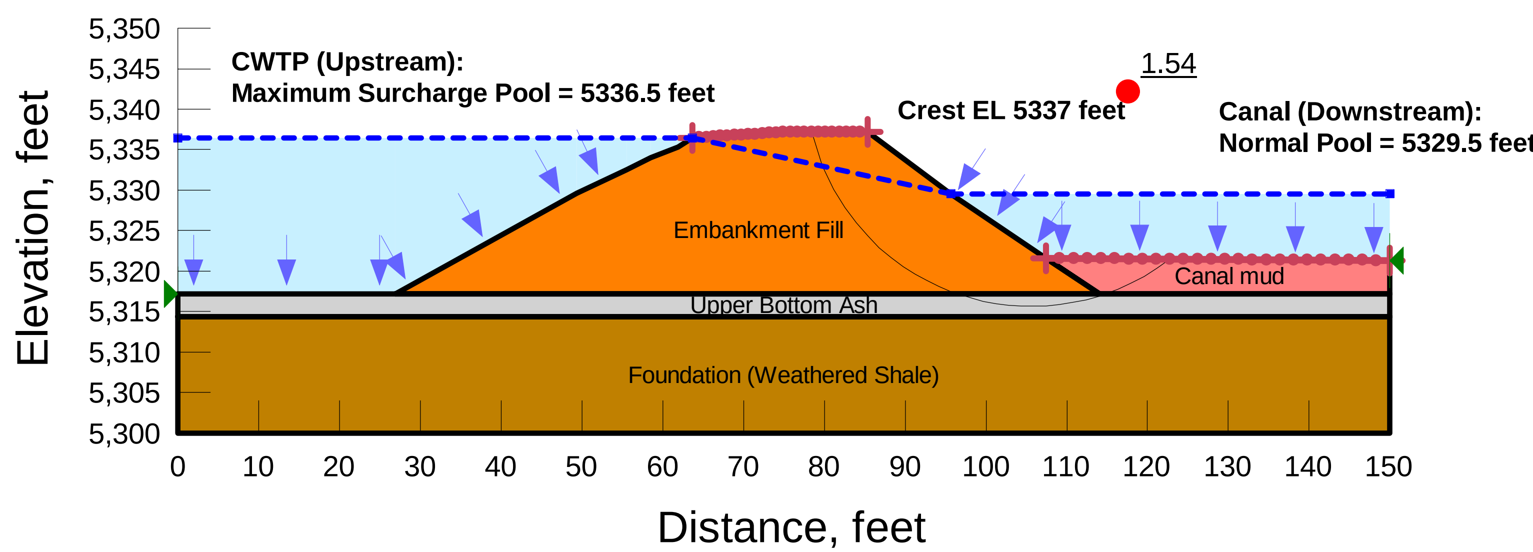
Four Corners Power Plant
Arizona Public Service

Note:
The results of the 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.

Maximum Surcharge Pool - Downstream Slope
File Name: APS CWTP Section 2 final.gsz
Date: 4/7/2016
Method: Spencer

Factor of Safety: 1.54

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Embankment Fill	130 pcf		100 psf	36 °
Foundation (Weathered Shale)	125 pcf	108 pcf	600 psf	20 °
Upper Bottom Ash	97.5 pcf	120 pcf	0 psf	31 °
Canal mud	100 pcf	60 pcf	0 psf	0 °



Slope Stability Analysis
Section 2
Combined Waste Treatment Pond

Four Corners Power Plant
Arizona Public Service

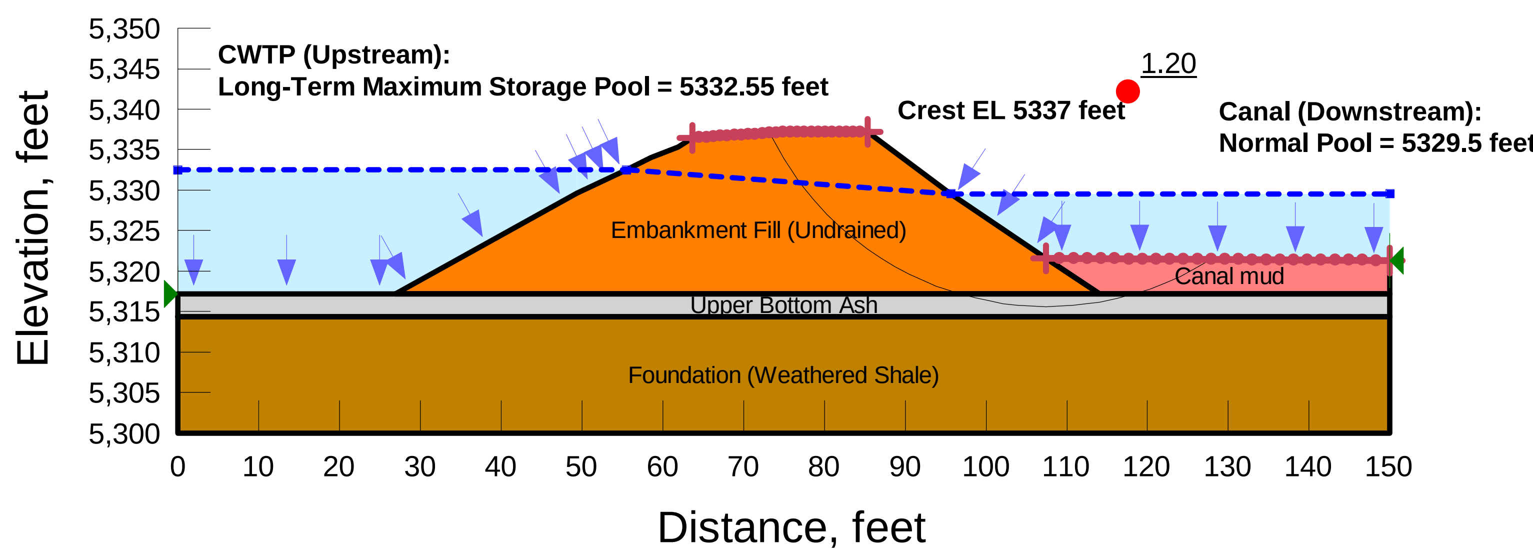
Note:
The results of the analysis shown here are based on available subsurface information, laboratory test results, and approximate soil properties. No warranties can be made regarding the continuity of subsurface conditions between the borings.

Pseudostatic Loading - Downstream Slope - Mohr Coulomb
File Name: APS CWTP Section 2 final.gsz
Date: 4/7/2016
Method: Spencer

Factor of Safety: 1.20

Material Type:	Unit Weight Saturated:	Unit Weight Above Water:	Cohesion:	Friction Angle:
Foundation (Weathered Shale)	125 pcf		600 psf	20 °
Embankment Fill (Undrained)	130 pcf	120 pcf	250 psf	19 °
Upper Bottom Ash	97.5 pcf	108 pcf	0 psf	31 °
Canal mud	100 pcf	60 pcf	0 psf	0 °

Horz Seismic Coef.: 0.083



Appendix C. Liquefaction Triggering Calculation

URS | Quality - It's Good Business

QMS Form 3-4 (MM)

Date: September 2014

IE QMS

Check and Review Record

Project Name	FCPP SIA	**Client Name**	APS
Project Location	Four Corners Power Plant	**PM Name**	Corey Jablonski
Project Number / Office Code	60445844	**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:	Lee Wright, P.E.	Comments Required by:	4/12/16
Work Product Originator:	Jed Stoken, P.E.	Title of Work Product:	CWTP Liquefaction Potential 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:		

8/12/16

Project Manager (or Designee) Signature **Date**

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:

Lee Wright 4/14/16

Checker / Reviewer Signature **Date**

Verification

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

Select:

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

or

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

and

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

Lee Wright 4/14/16

Checker / Reviewer Signature **Date**

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** 8/12/16 **Date**
DISTRIBUTION	Project Central File – Quality File Folder Other – Specify:

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 1 of 10

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Figures

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 2 of 10

1 INTRODUCTION

The objective of this calculation is to present the results of a liquefaction potential analysis performed based on standard penetration tests (SPT)s obtained from field investigations at the Combined Waste Treatment Pond (CWTP) at the Four Corners Power Plant (FCPP) owned and operated by Arizona Public Service (APS). The factor of safety against liquefaction is calculated using a Microsoft Excel spreadsheet that follows the procedures outlined in the Soil Liquefaction During Earthquakes Monograph (Idriss and Boulanger 2008).

2 ANALYSIS CRITERIA

The analysis was 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 Impoundments (EPA 2015). The regulations require embankments constructed of soils susceptible to liquefaction, identified through liquefaction potential analyses, to be analyzed for post-earthquake stability. No specific requirement for assessing the liquefaction potential of the soils is presented in the EPA rules; however, it is typical practice to assess liquefaction potential based on a minimum factor of safety against liquefaction of 1.0 to 1.1. For this calculation soils determined to have factors of safety greater than or equal to 1.10 are not susceptible to liquefaction, while those less than 1.10 are susceptible to liquefaction.

3 ANALYSIS INPUTS

The following inputs were used in the analysis:

- The subsurface stratigraphy and SPT blowcounts were based on borings drilled during the AECOM 2016 Geotechnical Exploration program (see Attachment A).

- Soil properties, including plasticity index and fines content, were based on laboratory tests conducted on samples recovered during the AECOM 2016 Geotechnical Exploration program (see Attachment B).

- Seismic loading criteria, including peak ground acceleration (PGA) and design earthquake magnitude (Mw) were 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) (see Attachment C).

4 LIQUEFACTION ANALYSIS

SPT-based liquefaction triggering analyses were performed according to the simplified procedure for estimating earthquake induced stresses and as described in Idriss and Boulanger

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 3 of 10

(2008). This procedure uses SPT blowcounts measured at the site along with laboratory tests of plasticity, fines content, and overburden stress condition to estimate the cyclic resistance ratio (CRR) of the soil deposit and compares the result to the cyclic stress ratio (CSR) estimated from the PGA, earthquake magnitude, and overburden stress.

4.1 Analysis Profiles

Four profiles of liquefaction potential with depth were analyzed at the CWTP based on the four borings, CWTP-1 to CWTP-4, drilled at the site as presented in Figure 1. For each profile, a result figure was produced with the corrected SPT blowcounts, CRR, CSR, factor of safety against liquefaction (FS_{liq}), maximum shear strain (γ_{max}), and vertical reconsolidation strain (ε_v).

Figure 1. AECOM 2016 Geotechnical Exploration Boring Locations

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 4 of 10

A general subsurface profile of the four borings consisted of the following:

- Embankment material consisting of Sandy Lean Clay (CL);
- Foundation material consisting of bottom ash (bottom ash material was not observed in borings CWTP-2 and CWTP-3); and
- Bedrock consisting of a sandstone or hard shale.

SPT blowcounts were measured at approximate 5-foot depth intervals with Shelby tube samples substituted at selected depths. The SPTs were conducted with a non-standard size sampler (3-inch O.D./2.4-inch I.D. Ring Sampler); therefore, blowcounts were adjusted by a factor of 0.44 to correct to a standard size split spoon sampler.

4.2 Analysis Method

In accordance with the Idriss and Boulanger (2008) procedure, the factor of safety against liquefaction is defined as the cyclic resistance ratio divided by the cyclic stress ratio:

$$FS_{liq} = \frac{CRR}{CSR}$$

where:

FS_{liq} = the factor of safety against liquefaction
CRR = the cyclic resistance ratio
CSR = the cyclic stress ratio

The earthquake-induced CSR is defined as 65% of the maximum CSR, and may be estimated using the following equation from the Seed-Idriss Simplified Liquefaction Procedure:

$$CSR = 0.65\frac{\tau_{max}}{\sigma'_v} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{max}}{g}R_d$$

where:

τ_{max} = maximum earthquake induced shear stress
σ'_v = vertical effective stress
σ_v = vertical total stress
a_{max}/g = maximum horizontal acceleration (as a fraction of gravity) at the ground surface
R_d = shear stress reduction factor to account for dynamic response of soil profile

The CRR is a function of soil properties and behavior, duration of shaking, and effective overburden. It may be estimated for a standard earthquake (M=7.5, σ'_v=1 atm) based on measured tip resistance using the following equation:

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 5 of 10

$$CRR_{M=7.5,\sigma'_{vc}=1} = exp\left(\frac{(N_1)_{60CS}}{14.1} + \left(\frac{(N_1)_{60CS}}{126}\right)^2 - \left(\frac{(N_1)_{60CS}}{23.6}\right)^3 + \left(\frac{(N_1)_{60CS}}{25.4}\right)^4 - 2.8\right)$$

where:

$(N_1)_{60CS}$ = normalized clean-sand-equivalent of the SPT resistance corrected for field conditions

The CRR for the reference earthquake is corrected to the earthquake-specific CRR using the following equation to correct for magnitude and overburden:

$$CRR_{M,\sigma'_v} = CRR_{M=7.5,\sigma'_{vc}=1} \cdot MSF \cdot K_\sigma$$

where:

MSF = magnitude scaling factor

K_σ = overburden correction factor

4.3 Peak Ground Acceleration

The maximum horizontal acceleration, a_{max}, was estimated using the 2008 NSHRP PSHA Interactive Deaggregation website available through the U.S. Geological Survey. Based on the 2008 source and attenuation models, the 2475-year (2-percent probability of exceedance in 50 years) peak horizontal acceleration for Site Class "B" rock was determined to be 0.05895g (United States Geological Survey [USGS] 2015) (see Attachment C). A Site Classification of C "Very Dense Soil and Soft Rock" was assigned to the site based on the average properties in the top 100 feet, which includes weathered bedrock, as illustrated in Table 1615.1.1 from the IBC (2003) and shown in Figure 2.

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	$2{,}500 < \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	$\bar{s}_u \geq 2{,}000$
D	Stiff soil profile	$600 \leq \bar{v}_s \leq 1{,}200$	$15 \leq \bar{N} \leq 50$	$1{,}000 \leq \bar{s}_u \leq 2{,}000$
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 $w \geq 40\%$, 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 m², 1 pound per square foot = 0.0479 kPa. N/A = Not applicable

Figure 2. Site Classification Definitions from Table 1615.1.1 of the IBC (2003)

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

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

$$PGA_M = 1.2(0.05895g)$$

$$PGA_M = 0.0707g$$

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)

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. Site Coefficient F_{PGA} from Table 11.8-1 of NEHRP (2009)

The PGA at the ground surface for Site Class C (PGA_M) was then used to estimate the peak transverse acceleration at the crest of the embankment, PGA_{crest} = 0.243g, 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 Holzer (USGS, 1998).

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 7 of 10

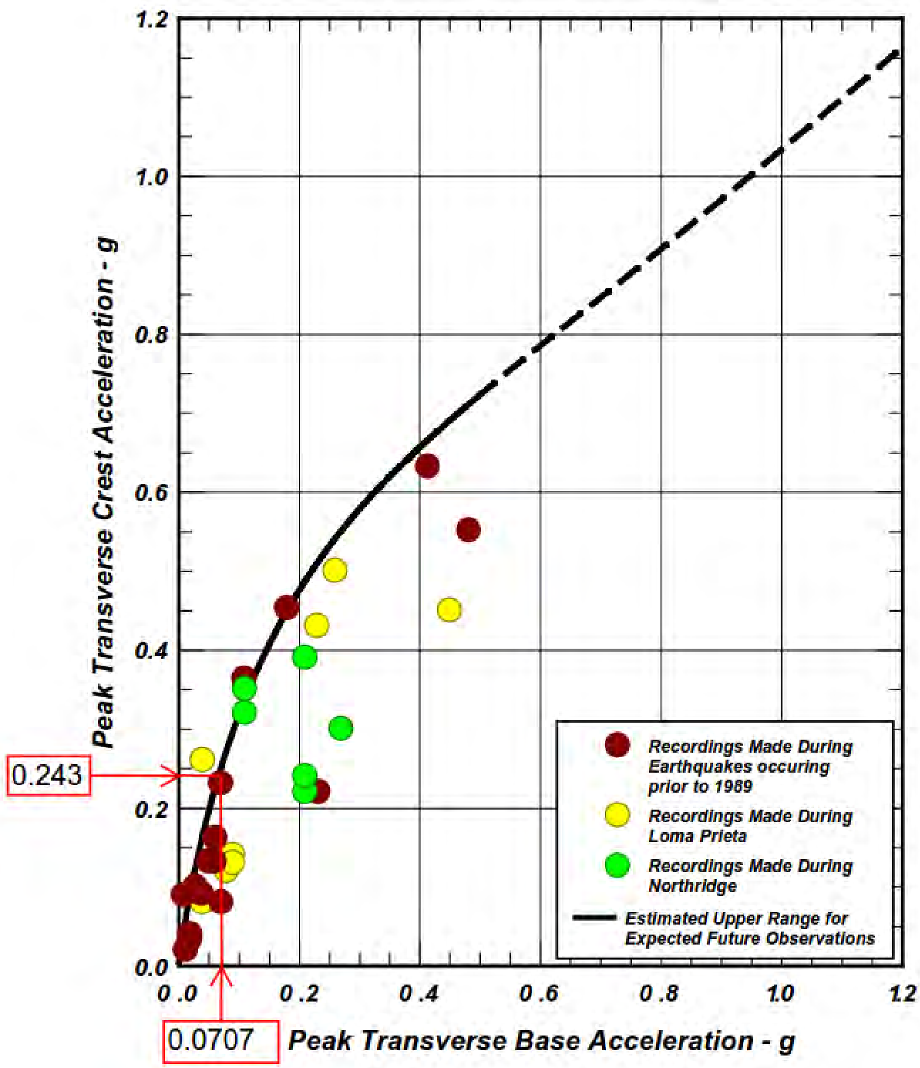

Figure 4. Variations of Peak Transverse Crest Acceleration v. Peak Transverse Base Acceleration Based on Holzer (1998)

4.4 Shear Stress Reduction Factor

Based on the previously performed amplification of the PGA at the crest, the peak crest transverse acceleration, a_{max}, does not correspond with the case histories included in the liquefaction case history database upon which the Idriss and Boulanger (2008) procedure is based. The shear stress reduction factor, R_d, is used to correct for dynamic site effects below the ground surface; however, based on engineering judgment and experience, the R_d relationship provided in Idriss and Boulanger (2008) does not adequately account for dynamic soil response within the embankment. Ideally, R_d would be determined from a site-response

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 8 of 10

analysis; however, a site-response analysis was not performed for the project site. Consequently, the R_d was estimated to consider the dynamic response of the embankment.

First, based on past experience with 2-dimensional site-response analyses, the acceleration is generally less than or equal to the free-field (site-corrected) acceleration at the base of the embankment. Second, the amplification of the acceleration increases more rapidly as the height within the embankment increases to the maximum at the crest. Therefore, the acceleration for a given depth within the embankment was interpolated from these two boundary conditions. Below the embankment, R_d was assumed to behave similarly to that of the free-field conditions, which is probably conservative as a result of the overburden pressure and additional confinement beneath the embankment. The diagram in Figure 5 shows how R_d was modified for the analysis.

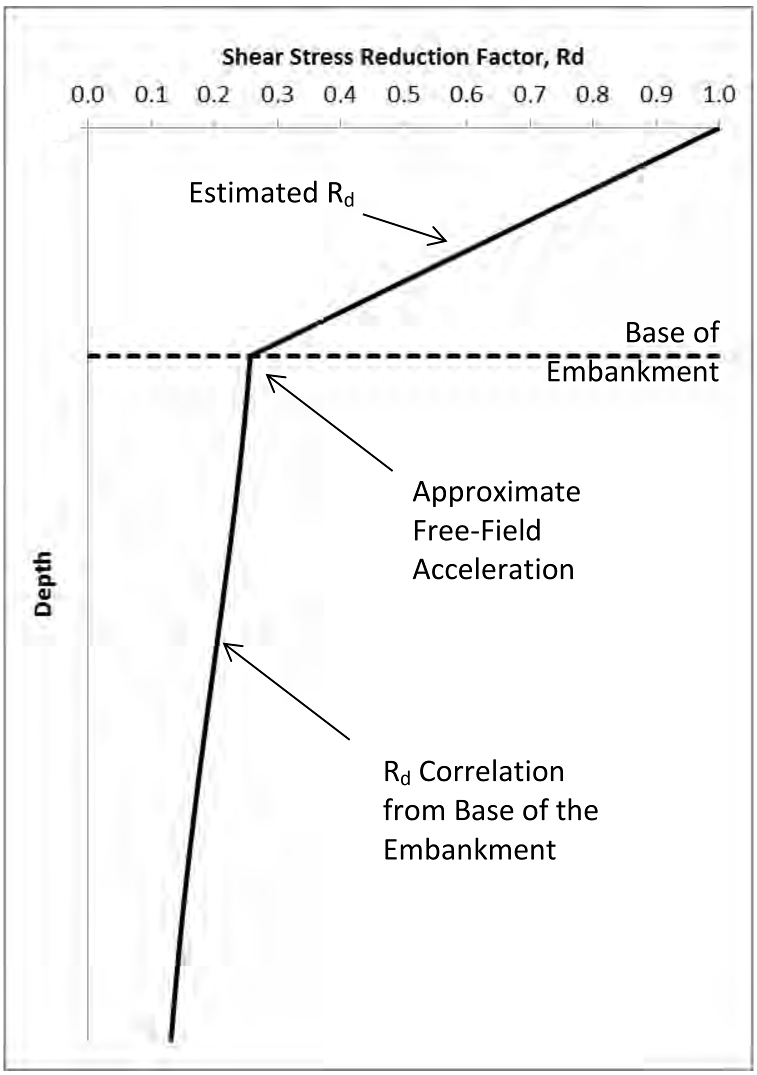

Figure 5. Estimated R_d Factor

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 9 of 10

4.5 Groundwater Conditions

The phreatic level for the analysis of each boring was based on water table depth noted during drilling and assumed phreatic levels within the embankments for the long-term maximum storage pool levels. Based on these sources, the phreatic level in the analysis was estimated at 5 feet below the ground surface or EL 5,331 ft.

5 ANALYSIS RESULTS AND CONCLUSIONS

Output figures showing results of the SPT-based liquefaction triggering analysis are included in Attachment D. The results of this analysis indicate that foundation and embankment soils are not likely to be triggered by the design 2,475-year earthquake. This is largely due to the low seismic hazard present in the Four Corners area. Based on the results of this analysis, post-liquefaction stability are not required for the CWTP site.

6 REFERENCES

The following references were used in performing this calculation:

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.

Idriss, I.M. and Boulanger, R.W., 2008, “Soil Liquefaction During Earthquakes,” Earthquake Engineering Research Institute, Monograph No MNO-12.

International Code Council (ICC), Inc., 2003, “2003 International Building Code [IBC],” May.

National Earthquake Hazards Reduction Program (NEHRP), 2009, “Recommended Seismic Provisions for New and Other Structures,” FEMA P-750, 2009 Edition.

United States Environmental Protection Agency (EPA), 2015, 40 CFR § 257 and 261 – Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities; Final Rule, Federal Register Vol. 80, No. 74, April 17.

United States Geological Survey (USGS), 2008. *2008 Interactive Deaggregations*. http://geohazards.usgs.gov/deaggint/2008/. Accessed March 3, 2016.

Analysis CALCULATION				
Calculation Title: CWTP Liquefaction Potential Analysis	**Project Title:** APS FCPP Structural Integrity Assessment	**Project No:** 60445844	**Date:** 3/21/16	**Page No:** Page 10 of 10

7 ATTACHMENTS

ATTACHMENT A AECOM 2016 Exploration Boring Logs

ATTACHMENT B AECOM 2016 Exploration Laboratory Test Results

ATTACHMENT C USGS 2008 Seismic PSH Deaggregation

ATTACHMENT D Liquefaction Potential Graphic Results

ATTACHMENT A

AECOM 2016 Exploration Boring Logs

URS

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BORING LOG: CWTP-1

SHEET 1 of 3
URS PROJECT #: 60445844

PROJECT: FCPP SIA
CLIENT: APS

CONTRACTOR: Cascade Drilling
DRILLER: Mike Lester
FIELD ENGINEER: Lee Wright

COORDINATES N: 36.69
E: -108.48
LOCATION: Four Corners Power Plant
REF. ALIGNMENT:
STATION:
OFFSET:

RIG TYPE: CME 85
DRILLING METHOD: HSA
HAMMER TYPE: Manual

COMMENTS:

SURFACE ELEV.: 5336
TOTAL DEPTH: 36 FT

START DATE: 02/03/2016 **TIME:** 10:14 AM
FINISH DATE: 02/03/2016 **TIME:** 11:17 AM

Type/Symbol	Augers	Ring Sampler	Split Spoon	D&M Lined	Shelby Tube	Cuttings
		R	S	DM	U	CU
I.D.	4.25"	2.5"	1.375"	2.5"	2.8"	--
O.D.	9"	3"	2"	3"	3"	--
Length	60"	18"	30"	18"	30"	--
Hammer WT.	140 lbs.		Drill Rod Size			
Hammer Fall	30 in.		I.D. (O.D.)			

GROUNDWATER DATA

Date	Time	Water Depth (ft)	Casing Depth (ft)	Hole Depth (ft)	Symbol
02/03/16	10:20 AM	11.0	10.0	10.0	▽
02/03/16	10:23 AM	14.3	15.0	15.0	▼

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	SAMPLE TYPE	NUMBER	SYMBOL	DEPTH (FT) FROM	DEPTH (FT) TO	MOISTURE, %	DRY DENSITY (PCF)	SOIL (Blows/6 in.) 0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
															EMBANKMENT FILL: SANDY LEAN CLAY (CL); dark brown; stiff; dry; mostly low plasticity clay; little fine to coarse sand; little silt; little fine to coarse gravel.
			BU	B-1		0	5							5 gal.	
5			R	S1B S1A		4	5.5			5	6	6	12	8	
10															
	CL		U	ST1		9.5	11.5	18	113	P	U	S	H	12	

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SHEET 2 of 3
URS PROJECT #: 60445844

PROJECT: FCPP SIA
CLIENT: APS

CONTRACTOR: Cascade Drilling
DRILLER: Mike Lester
FIELD ENGINEER: Lee Wright

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE SYMBOL	SAMPLE DEPTH (FT) FROM	SAMPLE DEPTH (FT) TO	MOISTURE, %	DRY DENSITY (PCF)	SOIL (Blows/6 in.) 0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
															EMBANKMENT FILL: SANDY LEAN CLAY (CL); dark brown; stiff; dry; mostly low plasticity clay; little fine to coarse sand; little silt; little fine to coarse gravel.
			R	S2B S2A		13.5	15			6	8	12	20	10	▼
15															
			R	S3B S3A		18.5	20			10	12	15	27	10	
20															
															FILL: BOTTOM ASH (SM); medium dense; gray, brown, black; wet; mostly fine to coarse sand; little trace silt.
			R	S4A		23.5	25			7	8	13	21	5	
25															
	SM		R	S5B S5A		28.5	30	65	57	3	4	7	11	9	
30															

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SHEET 3 of 3

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PROJECT: FCPP SIA
CLIENT: APS

CONTRACTOR: Cascade Drilling
DRILLER: Mike Lester
FIELD ENGINEER: Lee Wright

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE SYMBOL	SAMPLE DEPTH (FT) FROM	SAMPLE DEPTH (FT) TO	MOISTURE, %	DRY DENSITY (PCF)	SOIL (Blows/6 in.) 0/6	SOIL (Blows/6 in.) 6/12	SOIL (Blows/6 in.) 12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
															FILL: BOTTOM ASH (SM); medium dense; gray, brown, black; wet; mostly fine to coarse sand; little silt.
35															
			R	S6A		35.58	36			50/5"	-	-	NA	5	
															Auger refusal at 36 feet below ground surface. See groundwater table for groundwater data.

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BORING LOG: CWTP-2

SHEET 1 of 2
URS PROJECT #: 60445844

PROJECT: FCPP SIA
CLIENT: APS

CONTRACTOR: Cascade Drilling
DRILLER: Mike Lester
FIELD ENGINEER: Lee Wright

COORDINATES N: 36.69
E: -108.47
LOCATION: Four Corners Power Plant
REF. ALIGNMENT:
STATION:
OFFSET:

RIG TYPE: CME 85
DRILLING METHOD: HSA
HAMMER TYPE: Manual

COMMENTS:

SURFACE ELEV.: 5336
TOTAL DEPTH: 23.75 FT
START DATE: 02/03/2016 **TIME:** 02:25 PM
FINISH DATE: 02/03/2016 **TIME:** 03:18 PM

Type/Symbol	Augers	Ring Sampler R	Split Spoon S	D&M Lined DM	Shelby Tube U	Cuttings CU
I.D.	4.25"	2.5"	1.375"	2.5"	2.8"	--
O.D.	9"	3"	2"	3"	3"	--
Length	60"	18"	30"	18"	30"	--
Hammer WT.	140 lbs.		Drill Rod Size			
Hammer Fall	30 in.		I.D. (O.D.)			

GROUNDWATER DATA

Date	Time	Water Depth (ft)	Casing Depth (ft)	Hole Depth (ft)	Symbol
02/03/16	02:32 PM	14.8	15.0	15.0	∇
02/03/16	03:18 PM	18.1	23.8	23.8	▼

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	TYPE	NUMBER	SYMBOL	DEPTH FROM (FT)	DEPTH TO (FT)	MOISTURE, %	DRY DENSITY (PCF)	0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
	CL														**EMBANKMENT FILL:** SANDY LEAN CLAY (CL); stiff; brown; dry; mostly low plasticity clay; some fine to coarse sand; trace fine to coarse gravel.
			BU	B-1		0	5							5 gal.	
5			R	S1B S1A		4	5.5			8	12	13	25	12	
10			U	ST1		9.5	11.5	24	101	P	U	S	H	19.8	

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BORING LOG: CWTP-2

SHEET 2 **of** 2

URS PROJECT #: 60445844

PROJECT: FCPP SIA

CLIENT: APS

CONTRACTOR: Cascade Drilling

DRILLER: Mike Lester

FIELD ENGINEER: Lee Wright

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	SAMPLE TYPE	NUMBER	SYMBOL	DEPTH (FT) FROM	DEPTH (FT) TO	MOISTURE, %	DRY DENSITY (PCF)	SOIL (Blows/6 in.) 0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
															EMBANKMENT FILL: SANDY LEAN CLAY (CL); stiff; brown; dry; mostly low plasticity clay; some fine to coarse sand; trace fine to coarse gravel.
			R	S2B S2A		13.5	15			10	10	13	23	10	**EMBANKMENT FILL:** SANDY CLAY WITH GRAVEL AND SILT (CL); brown and gray; very stiff; wet; mostly low plasticity clay; some fine to medium sand; little fine to coarse gravel; little silt; trace cobbles.
15															
	CL														
			U	-		18.5	19			P	U	S	H	0	Possible cobble encoutered at 18.5 feet.
20															
	SM														**FILL:** BOTTOM ASH (SM); medium dense; gray, brown, black; wet; mostly fine to coarse sand; little silt.
	SM		R	S3C S3B S3A		22.25	23.75			26	38	40	78	18	**WEATHERED SANDSTONE:** SILTY SAND (SM); light brown; wet; mostly fine sand; some nonplastic silt.
	CL														**WEATHERED SHALE:** LEAN CLAY (CL); dark brown; hard; dry; mostly low plasticity clay.
															Auger refusal at 23.75 feet below ground surface. See groundwater table for groundwater data.

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BORING LOG: CWTP-3

SHEET 1 of 2

URS PROJECT #: 60445844

PROJECT: FCPP SIA
CLIENT: APS

CONTRACTOR: Cascade Drilling
DRILLER: Mike Lester
FIELD ENGINEER: Lee Wright

COORDINATES N: 36.69
E: -108.47
LOCATION: Four Corners Power Plant

REF. ALIGNMENT:
STATION:
OFFSET:

RIG TYPE: CME 85
DRILLING METHOD: HSA
HAMMER TYPE: Manual

COMMENTS:

SURFACE ELEV.: 5336
TOTAL DEPTH: 23.5 FT

START DATE: 02/04/2016 **TIME:** 09:50 AM
FINISH DATE: 02/04/2016 **TIME:** 11:06 AM

Type/Symbol	Augers	Ring Sampler R	Split Spoon S	D&M Lined DM	Shelby Tube U	Cuttings CU
I.D.	4.25"	2.5"	1.375"	2.5"	2.8"	--
O.D.	9"	3"	2"	3"	3"	--
Length	60"	18"	30"	18"	30"	--
Hammer WT.	140 lbs.		Drill Rod Size			
Hammer Fall	30 in.		I.D. (O.D.)			

GROUNDWATER DATA

Date	Time	Water Depth (ft)	Casing Depth (ft)	Hole Depth (ft)	Symbol
02/04/16	09:57 AM	4.0	10.0	10.0	▽
02/04/16	11:15 AM	13.2	0.0	NA	▼

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	TYPE	NUMBER	SYMBOL	DEPTH FROM (FT)	DEPTH TO (FT)	MOISTURE, %	DRY DENSITY (PCF)	0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
	CL														**EMBANKMENT FILL:** SANDY CLAY (CL); very stiff; brown; dry; mostly low plasticity clay; some to little fine to coarse sand.
			BU	B-1		0	5							5 gal.	
5			R	S1C S1B S1A		4	5.5			5	11	12	23	18	
10			U	ST1		9	11	19 19 21	110 111 109	P	U	S	H	22	Coarser material near the top of the Shelby tube.

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SHEET 2 of 2

URS PROJECT #: 60445844

PROJECT: FCPP SIA

CLIENT: APS

CONTRACTOR: Cascade Drilling

DRILLER: Mike Lester

FIELD ENGINEER: Lee Wright

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	SAMPLE TYPE	NUMBER	SYMBOL	DEPTH (FT) FROM	DEPTH (FT) TO	MOISTURE, %	DRY DENSITY (PCF)	SOIL (Blows/6 in.) 0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
	SM														**WEATHERED SANDSTONE:** SILTY SAND (SM); brown; very dense; moist to wet; mostly fine to coarse sand; little silt; trace clay (weathered sandstone).
			R	S2C S2B S2A		13.5	15			14	17	50	67	18	
15			U	-		15	15.5			P	U	S	H	0	
			R	S3B S3A		17	17.75	20	108	20	50/3"	-	>50	9	Driller reports hard drilling at 17 feet.
			R	S4A		19	19.46			50/5.5"	-	-	NA	5.5	
20															
			R	-		23.5				50/0"	-	-	NA	0	Auger refusal at 23.5 feet below ground surface. See groundwater table for groundwater data.

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BORING LOG: CWTP-4

SHEET 1 **of** 3
URS PROJECT #: 60445844

PROJECT: FCPP SIA
CLIENT: APS

CONTRACTOR: Cascade
DRILLER: Mike Lester
FIELD ENGINEER: Lee Wright

COORDINATES N: 36.69
E: -108.47
LOCATION: Four Corners Power Plant
REF. ALIGNMENT:
STATION:
OFFSET:

RIG TYPE: CME 85
DRILLING METHOD: HSA
HAMMER TYPE: Manual

COMMENTS:

SURFACE ELEV.: 5336
TOTAL DEPTH: 37 FT

START DATE: 02/04/2016 **TIME:** 12:50 PM
FINISH DATE: 02/04/2016 **TIME:** 01:30 PM

Type/Symbol	Augers	Ring Sampler R	Split Spoon S	D&M Lined DM	Shelby Tube U	Cuttings CU
I.D.	4.25"	2.5"	1.375"	2.5"	2.8"	--
O.D.	9"	3"	2"	3"	3"	--
Length	60"	18"	30"	18"	30"	--
Hammer WT.	140 lbs.		Drill Rod Size			
Hammer Fall	30 in.		I.D. (O.D.)			

GROUNDWATER DATA

Date	Time	Water Depth (ft)	Casing Depth (ft)	Hole Depth (ft)	Symbol
02/04/16	12:59 PM	8.0	10.0	10.0	▽
02/04/16	01:50 PM	7.0	0.0	15.8	▼

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	TYPE	NUMBER	SYMBOL	DEPTH FROM (FT)	DEPTH TO (FT)	MOISTURE, %	DRY DENSITY (PCF)	0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
	CL														**EMBANKMENT FILL:** SANDY CLAY WITH GRAVEL (CL); dark brown; very stiff; most to wet; mostly low plasticity clay; some to little fine to coarse sand; some fine to coarse rounded to subrounded gravel.
5			R	S1B S1A		4	5.5			10	11	12	23	12	
			BU	B-1		5	10							5 gal.	
10															
			U	BAG1		10	12			P	U	S	H	6	

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BORING LOG: CWTP-4

SHEET 2 of 3

URS PROJECT #: 60445844

PROJECT: FCPP SIA

CLIENT: APS

CONTRACTOR: Cascade

DRILLER: Mike Lester

FIELD ENGINEER: Lee Wright

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	SAMPLE TYPE	NUMBER	SYMBOL	DEPTH (FT) FROM	DEPTH (FT) TO	MOISTURE, %	DRY DENSITY (PCF)	SOIL (Blows/6 in.) 0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
															EMBANKMENT FILL: SANDY CLAY WITH GRAVEL (CL); dark brown; very stiff; most to wet; mostly low plasticity clay; some to little fine to coarse sand; some fine to coarse rounded to subrounded gravel.
15			U	ST1		15	17			P	U	S	H	18	
															FILL: BOTTOM ASH (SM); gray; very loose; wet; mostly fine to coarse angular sand; little nonplastic silt.
			R	S2C S2B S2A		18.5	20	84	63	3	1	1	2	18	
20															
			R	S3C S3B S3A		23.5	25	51	59	4	5	7	12	18	Medium dense to dense; some nonplastic silt.
25															
	SM														
			R	S4C S4B S4A		28.5	30	52	60	9	10	11	21	18	
30															

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BORING LOG: CWTP-4

SHEET 3 of 3

URS PROJECT #: 60445844

PROJECT: FCPP SIA
CLIENT: APS

CONTRACTOR: Cascade
DRILLER: Mike Lester
FIELD ENGINEER: Lee Wright

DEPTH BELOW SURFACE (FT)	USCS	GRAPHIC	SAMPLE TYPE	NUMBER	SYMBOL	DEPTH (FT) FROM	DEPTH (FT) TO	MOISTURE, %	DRY DENSITY (PCF)	SOIL (Blows/6 in.) 0/6	6/12	12/18	N	REC. (in.)	VISUAL MATERIAL CLASSIFICATION AND REMARKS
															FILL: BOTTOM ASH (SM); gray; medium dense to dense; wet; mostly fine to coarse angular sand; some nonplastic silt.
			R	S5C S5B S5A		33.5	35			12	14	18	32	18	
35															
	CL														**WEATHERED SHALE:** LEAN CLAY (CL); dark brown; hard; dry; mostly low plasticity clay. Auger refusal at 37 feet below ground surface. See groundwater table for groundwater data.

ATTACHMENT B

AECOM 2016 Exploration Laboratory Test Results

Terracon

PROJECT: APS Four Corners Power Plant Structural Integrity Assessment
LOCATION: Kirkland, NM
MATERIAL: Undisturbed Sample
SAMPLE SOURCE: SEE BORING

JOB NO: 65151169
WORK ORDER NO: 1
LAB NO: SEE BELOW
DATE ASSIGNED: 2/9/16

DENSITY OF SOIL IN PLACE BY THE DRIVE-CYLINDER METHOD (ASTM D2937)

LAB #	BORING	MOISTURE WET WT. (g)	MOISTURE DRY WT. (g)	MOISTURE CONTENT	DIA. (cm)	HGT. (cm)	WET WEIGHT & RINGS (g)	DRY DENSITY (pcf)
3	Shelby Tube CWTP-1 @ 9.5'-11.5'	2,345.7	1,990.1	17.9%	7.21	19.32	1,682.8	113.0
7	SPT CWTP-1 @ 28.5'-30.0'	228.4	138.5	64.9%	4.87	17.66	491.3	56.5
11	Shelby Tube CWTP-2 @ 9.5'-11.5'	2,800.3	2,252.2	24.3%	7.21	34.11	2,800.3	100.9
17	Shelby Tube CWTP-3 @ 9.5'-11.5'	3,748.6	3,138.6	19.4%	7.21	43.61	3,748.6	110.0
20	SPT CWTP-3 @ 17.0'-17.75'	621.9	517.8	20.1%	4.87	29.68	1,154.3	108.4
27	SPT CWTP-4 @ 18.5'-20.0'	1,553.6	843.0	84.3%	4.87	44.76	1,553.6	63.0
28	SPT CWTP-4 @ 23.5'-25.0'	1,211.5	801.9	51.1%	4.87	45.22	1,211.5	59.4
29	SPT CWTP-4 @ 28.5'-30.0'	1,233.4	810.2	52.2%	4.87	45.34	1,233.4	59.8

REVIEWED BY Cliff Marty

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MECHANICAL SIEVE ANALYSIS
GROUP SYMBOL, USCS (ASTM D-2487)

PERCENT PASSING BY WEIGHT

				Silt or	SAND								GRAVEL									COBBLES	
				Clay	Fine			Medium			Coarse		Fine				Coarse						
Location & Depth	USCS	LL	PI	#200	#100	#50	#40	#30	#16	#10	#8	#4	1/4"	3/8"	1/2"	3/4"	1"	1 1/4"	1 1/2"	2"	3"	6"	Lab #
Shelby Tube CWTP-1 @ 9.5'-11.5'	CL	35	19	57	65	67	67	68	70	73	75	86	86	87	88	89	93	95	100	100	100	100	3
SPT CWTP-1 @ 28.5'-30.0'	SM	NV	NP	16	27	45	56	68	86	94	96	100	100	100	100	100	100	100	100	100	100	100	7
Shelby Tube CWTP-2 @ 9.5'-11.5'	CL	33	17	51	78	81	82	82	84	87	88	97	97	98	98	99	99	100	100	100	100	100	11
Shelby Tube CWTP-3 @ 9.5'-11.5'	SC	32	16	45	63	65	66	67	68	70	71	78	79	81	83	86	88	89	96	100	100	100	17
SPT CWTP-3 @ 17.0'-17.75'	SM	NV	NP	24	88	93	94	95	95	95	96	97	97	97	98	100	100	100	100	100	100	100	20
SPT CWTP-4 @ 18.5'-20.0'	SM	NV	NP	22	31	36	39	44	59	73	78	92	93	94	94	94	94	94	100	100	100	100	27

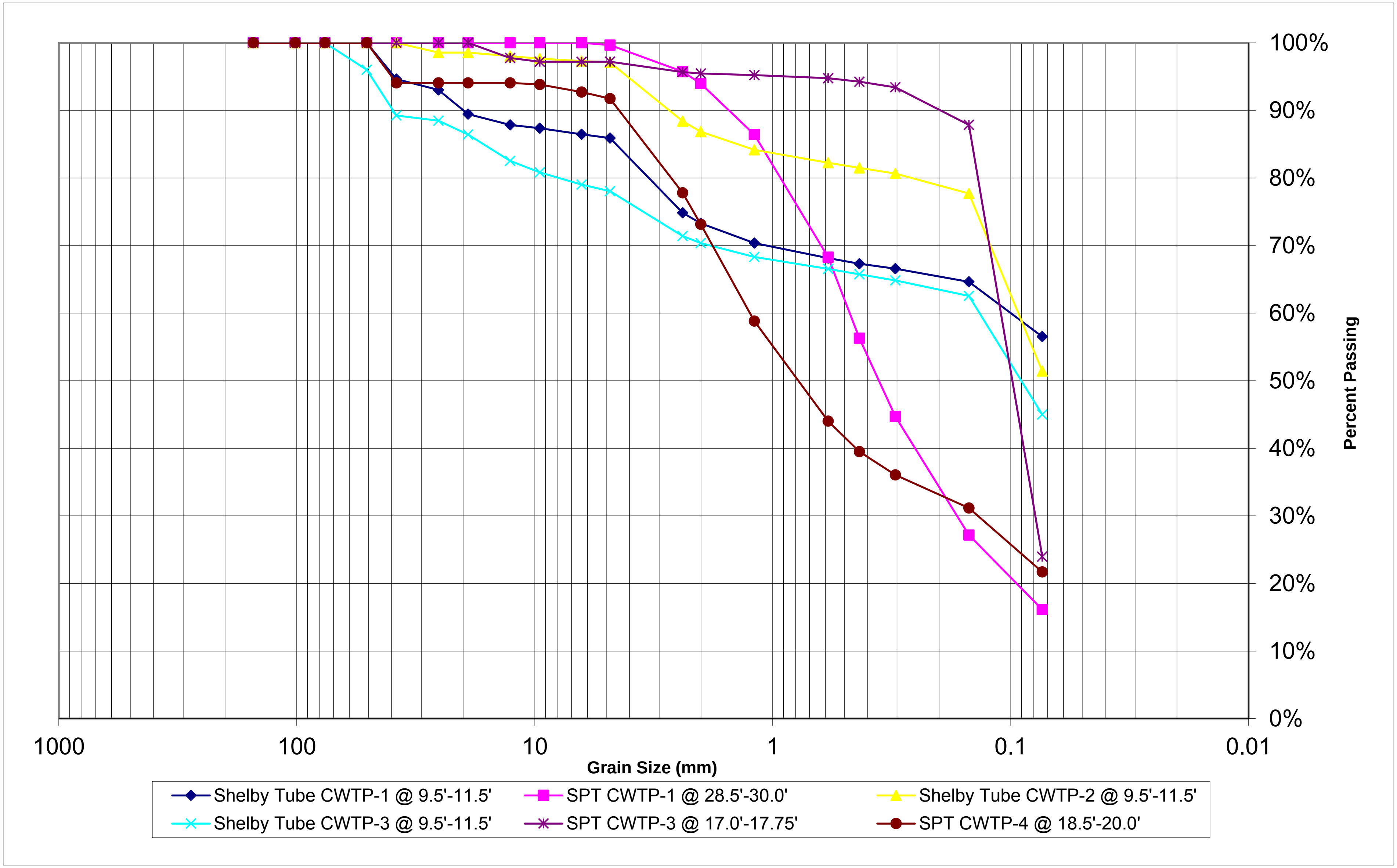
AASHTO R18

REVIEWED BY

Terracon

PROJECT:	APS Four Corners Power Plant Structural Integrity Assessment	**JOB NO:**	65151169
LOCATION:	Kirkland, NM		
SAMPLE SOURCE:	SEE BELOW	**DATE ASSIGNED:**	2/9/16

MECHANICAL SIEVE ANALYSIS

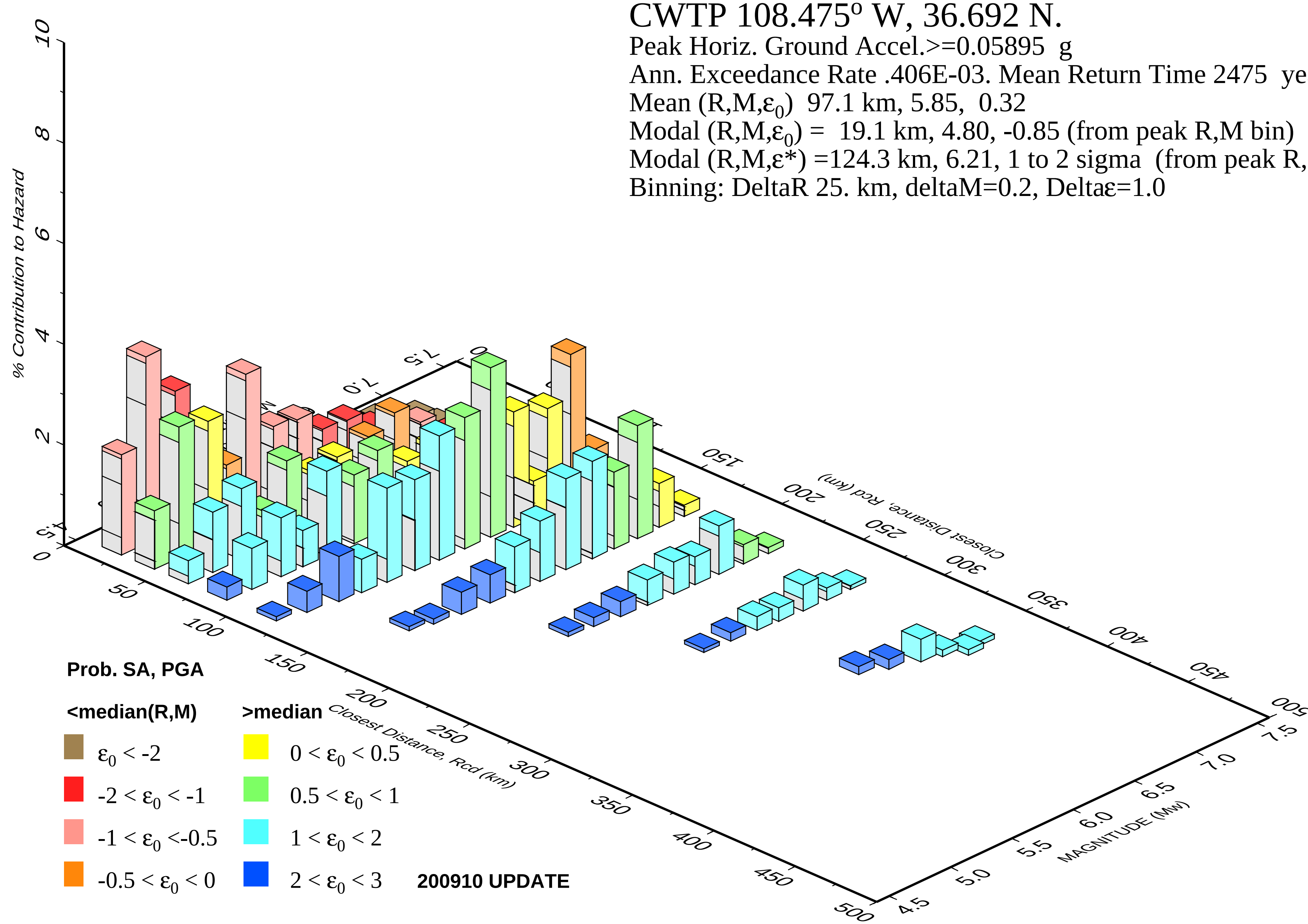


REVIEWED BY

ATTACHMENT C

USGS 2008 Seismic PSH Deaggregation

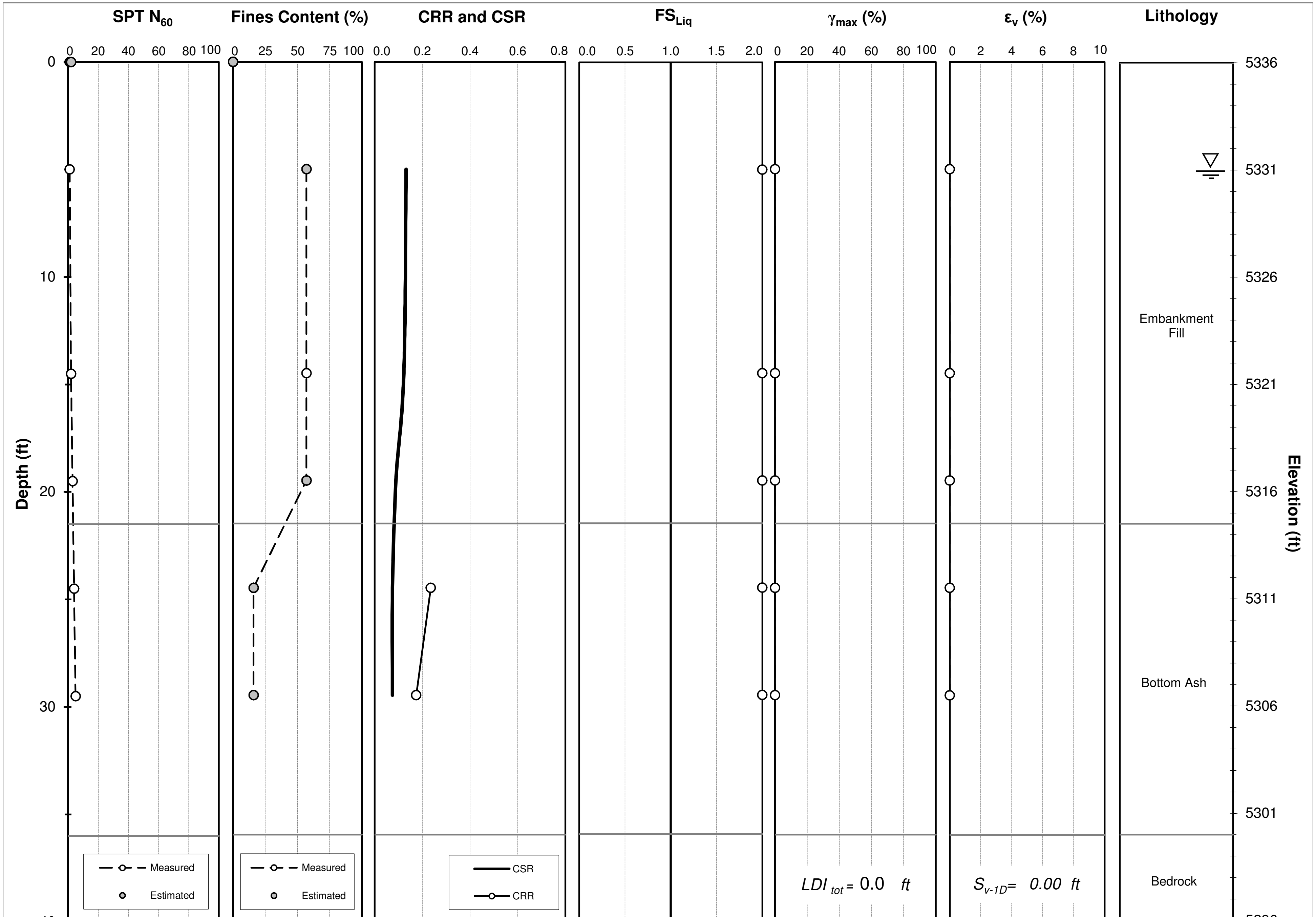
PSH Deaggregation on NEHRP BC rock
CWTP 108.475° W, 36.692 N.
Peak Horiz. Ground Accel.>=0.05895 g
Ann. Exceedance Rate .406E-03. Mean Return Time 2475 years
Mean (R,M,ε_0) 97.1 km, 5.85, 0.32
Modal (R,M,ε_0) = 19.1 km, 4.80, -0.85 (from peak R,M bin)
Modal (R,M,ε*) =124.3 km, 6.21, 1 to 2 sigma (from peak R,M,ε bin)
Binning: DeltaR 25. km, deltaM=0.2, Deltaε=1.0



GMT 2016 Mar 3 19:41:05 **Distance (R), magnitude (M), epsilon (E0,E) deaggregation for a site on rock with average vs= 760. m/s top 30 m. USGS CGHT PSHA2008 UPDATE Bins with lt 0.05% contrib. omitted**

ATTACHMENT D

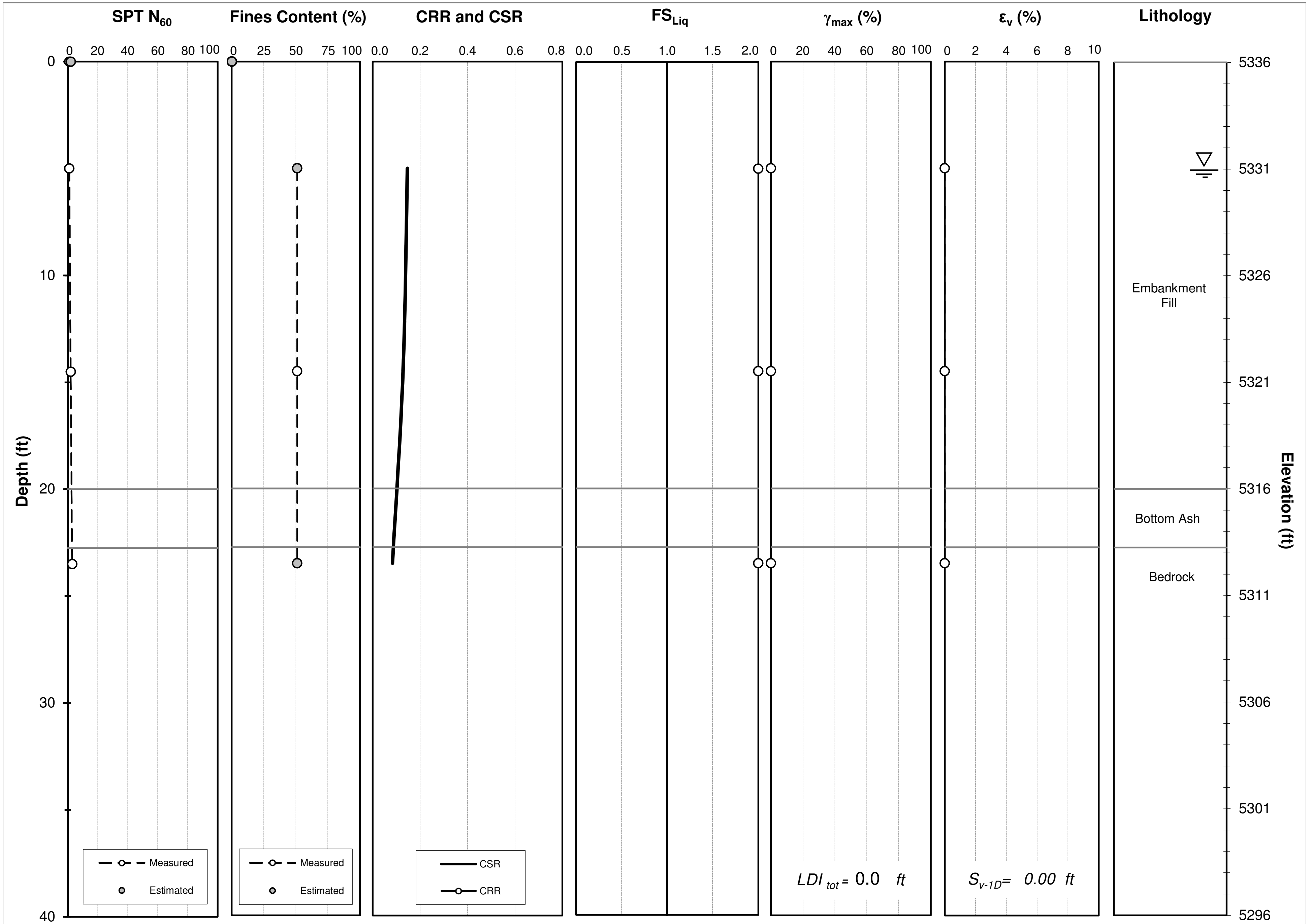
Liquefaction Potential Graphic Results



Peak ground acceleration, PGA = 0.243g
Earthquake Magnitude, M = 5.85
Ground Motion: 2,475-Year Mean

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

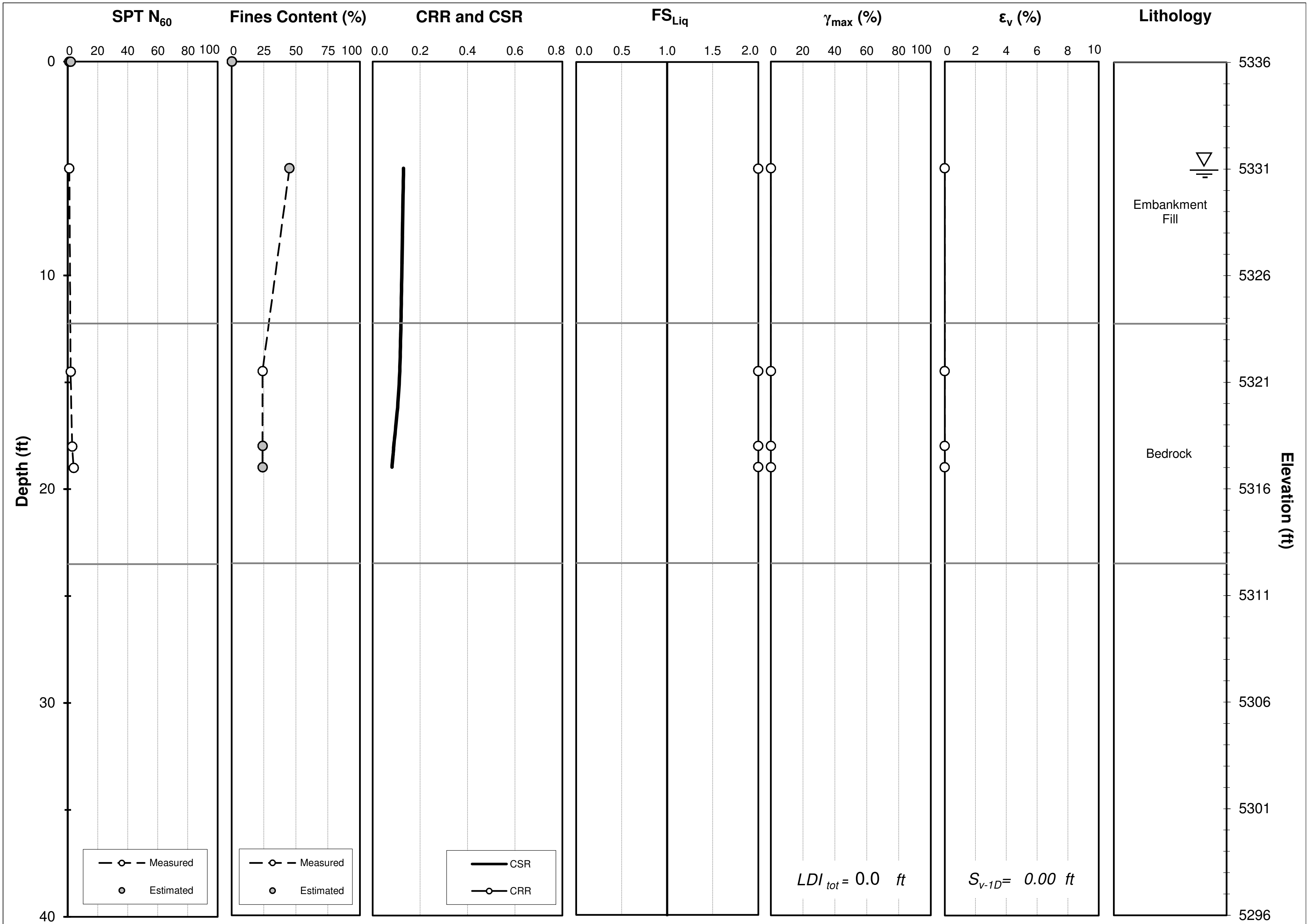
Title: CWTP Liquefaction Analysis: CWTP-1		Project No.: 60445844
Project: APS FCPP Structural Integrity	Date: April 2016	AECOM



Peak ground acceleration, PGA = 0.273g
Earthquake Magnitude, M = 5.85
Ground Motion: 2,475-Year Mean

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

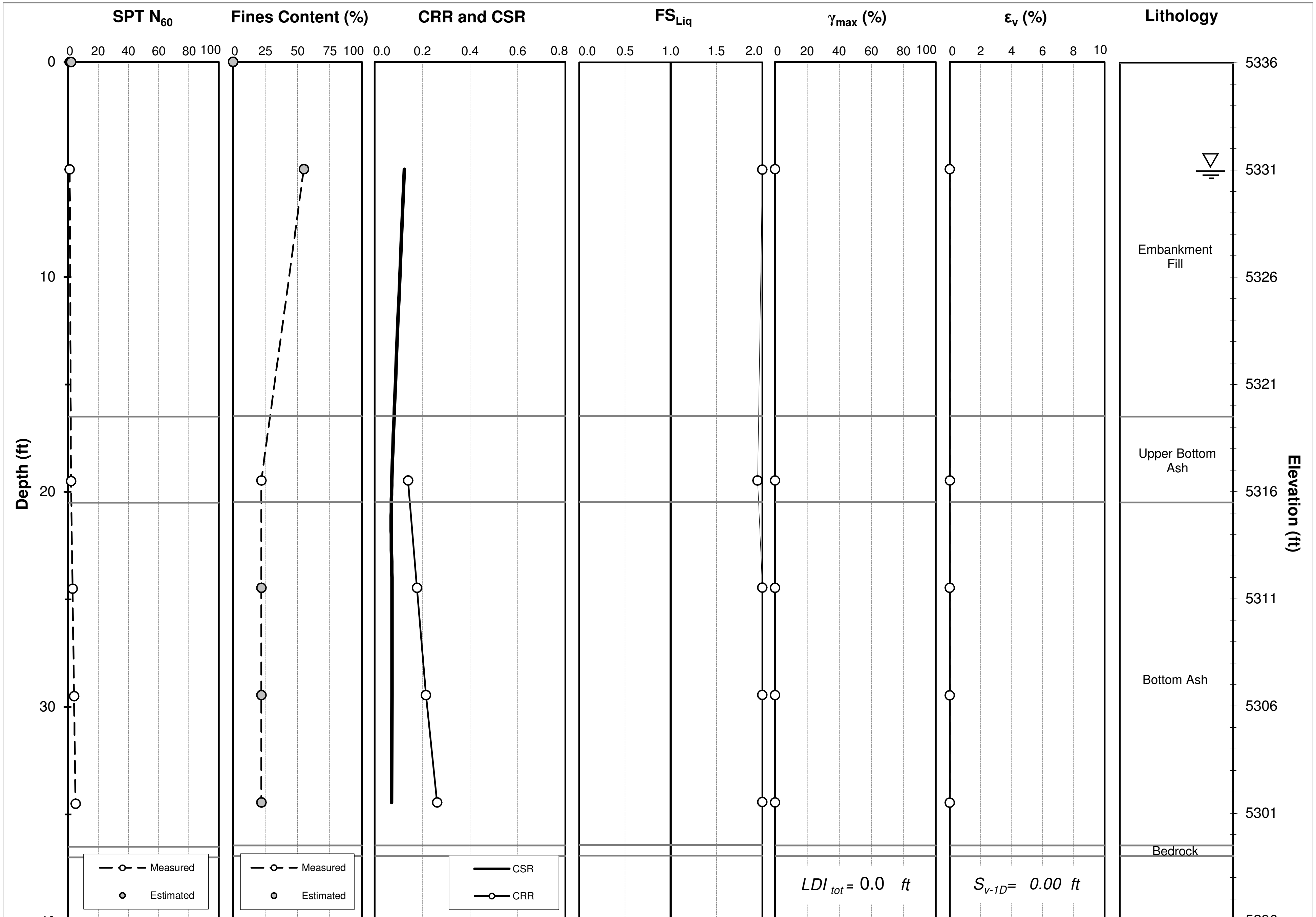
Title: CWTP Liquefaction Analysis: CWTP-2		Project No.: 60445844
Project: APS FCPP Structural Integrity	Date: April 2016	AECOM



Peak ground acceleration, PGA = 0.243g
Earthquake Magnitude, M = 5.85
Ground Motion: 2,475-Year Mean

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: CWTP Liquefaction Analysis: CWTP-3		Project No.: 60445844
Project: APS FCPP Structural Integrity	Date: April 2016	AECOM



Peak ground acceleration, PGA = 0.243g
Earthquake Magnitude, M = 5.85
Ground Motion: 2,475-Year Mean

Method: Idriss & Boulanger (2008), *Soil Liquefaction during Earthquakes*, EERI MNO-12

Title: CWTP Liquefaction Analysis: CWTP-4		Project No.: 60445844
Project: APS FCPP Structural Integrity	Date: April 2016	AECOM

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