FOUR CORNERS POWER PLANT
LINED ASH IMPOUNDMENT -
Periodic Structural Integrity
Assessment

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

October 2021
AECOM Project 60664563
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Attachment

1. Introduction

This periodic update to the Structural Integrity Assessment for the Lined Ash Impoundment (LAI) 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 LAI 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:

   b. Perform a documented review of each major component of the contributing technical information from:
      i. AECOM, 2016. Final Summary Report, Structural Integrity Assessment: Lined Ash Impoundment, 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

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

The number of entries to the annual list of “Observed Conditions,” over the last 5 years of reports, has remained roughly consistent. The most consistently observed, or significant, conditions involve: (1) bulges of the exposed liner between the crest and the solids deposition level on the upstream slope of the LAI embankment; (2) settlement of the West Embankment; and (3) overfilling of the northern portion of the impoundment during the final year of operation.

The localized bulges of the exposed liner on the upstream face of the LAI embankment are caused by leaks in the upstream face HDPE liner that allow water to get between the liner and clay blanket of the embankment, thereby “floating” the HDPE liner and allowing a constrained “bulge”. The Plant has a procedure to identify bulges, identify the initiation leak, cut the liner to relieve the water pressure, and then patch the initiation and relief cuts. The design of the embankment to resist seepage relies on the combination of the 15-foot wide compacted clay blanket to minimize seepage loss and the large downstream wedge of fly and bottom ash to perform as a drain to relieve, drawdown, and prevent excessive seepage hydraulic gradients. With this embankment section, temporary ponding of water between the liner and clay low-permeability zone is not considered to have an adverse impact on the stability or structural integrity of the LAI embankments.

Settlement is measured by monuments SM-7 and SM-9 at the toe of the West Embankment of the LAI. There are no settlement monuments on the crest of the South, West, or North embankments. SM-7 and SM-9 indicate settlement of 10 and 8 inches, respectively, between 2015 and 2021. A 2021 topographic survey of the LAI embankments and reservoir indicates settlement of the central portion of the crest of the West Embankment of between 6 and 9 inches compared to the original design grade. Considering that the LAI is founded on old unlined Ash Ponds 3, 4, and 5, continued loading of the LAI will cause consolidation of the ash pond subgrade that will be expressed as broad settlement. No external bulges or other movements have been identified by the weekly or annual inspections to suggest a mechanism other than broad settlement. This form of settlement is not considered to have an adverse impact on the available storage capacity, stability, or structural integrity of the LAI embankments.

During the final years of operation of the LAI, before cessation of deposition in 2021, the properties of the flue gas desulfurization (FGD) slurry being discharged caused the material to drop out faster and at a steeper beach slope than the earlier mix of fly ash and FGD. As a result,
the northern portions of the pond (closer to the discharge locations) filled faster than the southern portions. Although the capacity of the impoundment to store the inflow design flood (IDF) was not diminished, the Plant took special measures to ensure that liquid flows were directed towards the center of the reservoir and away from portions of the embankment with diminished freeboard. Discharge ceased in April 2021. The topography of the final solids surface within the impoundment will direct runoff towards the flood pool in the southern end of the impoundment.

The 2017-2021 Annual Inspection Reports also provide information on minimum and maximum values for various types of geotechnical instrumentation installed within the embankments and foundations. Periodically, deviations or technical issues may be identified that limit or alter readings and these instances are reported in the Annual Inspection Reports. For the LAI, the instruments consist of vibrating wire and standpipe piezometers, inclinometers, buried settlement monuments, and surface settlement monuments. The records, including the SM-7 and SM-9 settlement records, were reviewed and no significant, adverse trends were identified that would cause structural instability or change in safety factor.

4. 2016 Certification – Review by Section

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

4.1 “1.4 Facility Description”

The LAI is no longer an operating CCR surface impoundment. APS provided notification, dated April 10, 2021, of its intent to close the LAI and APS ceased discharge of CCR to the LAI on or before April 10, 2021. In order to maintain adequate freeboard to contain the IDF, APS periodically pumps precipitation runoff and drain down water from the CCR solids from the free water pool at the southwest corner of the impoundment to the drop inlet tower, which discharges by gravity to the Lined Decant Water Pond (LDWP).

APS intends to close the LAI and its contents in place, similar to the closure approach used for old Pond 6. APS is currently undertaking a phased geotechnical investigation to identify safe and effective procedures to construct a soil cap over the soft contents of the impoundment.

APS is evaluating whether it wishes to restore the crest elevation of the West Embankment to its as-designed, pre-settlement elevation; this restoration would provide more flexibility for managing the normal operating pool and maintaining sufficient storage capacity for the IDF. The outcome of this evaluation will likely be reported in the 2021 (published in 2022) annual inspection report.

4.2 “2 Hazard Potential Classification”

The details presented in this section of the 2016 Report adequately represent current conditions and satisfy the requirements of the Rule.
Based on a review of the information presented in the 2016 Report, the LAI impoundment currently satisfies the criteria for Significant 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.

The only construction actions that have occurred at the facility since the 2016 Report relate to maintenance activities, measures to control the deposition of solids around the discharge locations, and geotechnical investigations. In 2020 and 2021, APS advanced several pilot roads onto the solids surface to allow geotechnical testing and to assess the stability of different portions of the surface for eventual closure construction.

4.4 “4 Structural Stability Assessment”

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

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

4.5 “5 Safety Factor Assessment”

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

AECOM is not aware of any new information that would warrant reevaluation of any material properties, cross-section configurations, or piezometric conditions of the perimeter embankment.

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

4.6 “6 Conclusions”

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

5. Recommended Additional Technical Investigations or Evaluations

None identified and none recommended.
6. Conclusion

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

7. Limitations

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

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

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

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

Certification Statement for:

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

I, Alexander W. Gourlay, being a Registered Professional Engineer in good standing in the State of 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:
Final Summary Report
Structural Integrity Assessment
Lined Ash Impoundment
Four Corners Power Plant
Fruitland, New Mexico

Prepared for:
Arizona Public Service

AECOM Job No. 60445844
August 2016
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<td>APS</td>
<td>Arizona Public Service</td>
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<tr>
<td>ASTM</td>
<td>American Society for Testing and Materials</td>
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<tr>
<td>CCR</td>
<td>Coal Combustion Residual</td>
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<tr>
<td>CFR</td>
<td>Code of Federal Regulations</td>
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<tr>
<td>CWTP</td>
<td>Combined Waste Treatment Pond</td>
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<td>EAP</td>
<td>Emergency Action Plan</td>
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<td>EL</td>
<td>Elevation</td>
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<tr>
<td>EPA</td>
<td>Environmental Protection Agency</td>
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<td>FCPP</td>
<td>Four Corners Power Plant</td>
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<tr>
<td>FGD</td>
<td>Flue Gas Desulfurization</td>
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<tr>
<td>ft</td>
<td>feet</td>
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<tr>
<td>HDPE</td>
<td>High Density Polyethylene</td>
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<tr>
<td>HPC</td>
<td>Hazard Potential Classification</td>
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<td>LAI</td>
<td>Lined Ash Impoundment</td>
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<td>LDWP</td>
<td>Lined Decant Water Pond</td>
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<td>NMOS</td>
<td>New Mexico Office of the State Engineer</td>
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<tr>
<td>pcf</td>
<td>pounds per cubic foot</td>
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<td>PMP</td>
<td>Propable Maximum Precipitation</td>
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<td>PMF</td>
<td>Propable Maximum Flood</td>
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<td>psf</td>
<td>pounds per square foot</td>
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<tr>
<td>RCRA</td>
<td>Resource Conservation and Recovery Act</td>
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<tr>
<td>USCS</td>
<td>Unified Soil Classification System</td>
</tr>
<tr>
<td>USGS</td>
<td>United States Geological Survey</td>
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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; Lined Ash Impoundment

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 Lined Ash Impoundment (LAI), NMOSE Dam No. D-634. 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 LAI located at the FCPP. The LAI is an existing CCR surface impoundment owned and operated by APS. The CCR impoundment is 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, updated, and revised to ensure the information within is accurate.

In addition, the following elements must be addressed for CCR units, such as the LAI, 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 LAI, 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:

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<td>Section 3 – Emergency Action Plan</td>
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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 (FGD) 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 LAI.

The LAI consists of a reservoir basin formed by a perimeter embankment. It receives fly ash and FGD sludge from the power plant. The LAI embankment was constructed in phases between 2003 and 2014. The impoundment was originally constructed on Ash Ponds 3 and 4 to EL 5228 ft and progressively raised in four lifts using the downstream method of construction to EL 5248, 5258, 5270 and 5280 ft. As the embankment was raised it expanded over Ash Pond 5. The LAI currently has a total surface area of about 129 acres and a total storage capacity of about 5,346 acre-ft when at the maximum storage pool water level of EL 5275.2 ft. The embankment ties into native weathered shale on the northeast and east sides of the impoundment. The LAI is licensed by NMDOE as a dam, NMDOE License No. D-634. Under NMDOE Regulations, the LAI perimeter embankment has been classified as an intermediate sized, significant hazard potential dam.

The LAI perimeter embankment is an earthen, zoned embankment dam consisting of compacted bottom ash and fly ash with a 15-foot wide compacted clay blanket along the upstream slope. The South Embankment was constructed using the existing Ash Pond 4 embankment as a starter dike and expanded downstream onto the adjacent native weathered shale. The West Embankment is founded on the Ash Pond 3 and 4 divider dike and on old hydraulically deposited fly ash within Ash Pond 3.
The Northwest Embankment is founded on the Ash Pond 5 and Ash Pond 6 embankments and hydraulically deposited fly ash within Ash Pond 6. The East Embankment lies on top of the native weathered shale. The embankment is approximately 6,600 ft in length with a maximum total height of about 107 ft near the southwest corner. The crest elevation varies between about 20 and 30 ft over the length of the embankment. The upstream and downstream slope are inclination varies across the length of the perimeter embankment as specified in Section 3.2 with upstream slopes ranging between two horizontal to one vertical (2H:1V) and 4H:1V and downstream slopes ranging between 2H:1V and 3H:1V. The toe of the embankment (EL) is 5,280 ft, creating 4.8 ft of freeboard above the maximum storage pool water level and 2.8 ft of freeboard above the maximum surcharge pool water level (URS, 2012). The upstream slope of the perimeter embankment and the entire pond are lined with a geomembrane liner system that prevents erosion of the slopes; the downstream slope is composed of compacted granular material with high frictional strength.

The foundation of the LAI embankment varies along the perimeter length. On the western and northwestern sides of the LAI, the embankment is founded on about 45 to 50 ft of hydraulically placed existing fly ash and pre-existing embankment associated with the old ash ponds. On the northeastern and eastern sides, the embankment is tied into the weathered shale bedrock. On the southern side, the embankment lies on the pre-existing ash pond embankments and weathered shale bedrock. To limit seepage into the embankments and underlying fly ash deposits, the LAI was installed with a single High Density Polyethylene (HDPE) liner that covers the impoundment basin to the embankment crest. In addition, a compacted clay blanket on the upstream slope beneath the geomembrane provides additional seepage resistance. The clay blanket is supplemented by an approximately 1280-foot-long, 32-foot high cement-bentonite slurry wall along the Northwest Embankment of the LAI. The slurry wall was installed to seal possible gaps in the clay core that were discovered during field investigations of the existing Ash Pond 5 embankment. An internal toe drain allows dewatering of impounded ash in the vicinity of the West Embankment of the LAI.

The LAI was constructed without an overflow channel. Instead, the primary outlet spillway consists of a drop inlet tower located adjacent to the West Embankment. This drop inlet tower is a vertical, eight foot diameter, HDPE pipe with multiple drilled holes, to allow decant lateral inflow of water. The drop inlet is surrounded by a placed bottom ash filter zone that filters solids from the water seeping laterally into the drop inlet tower. The drop inlet tower drains to the west through an 8-inch or adjacent 16-inch HDPE pipe, through the West Embankment of the LAI, to the LDWP. The rim of the drop inlet is set at EL 5,277.84 ft; however, due to the lateral inflow to the inlet the maximum storage pool water level (operating water level) is EL 5,275.2 ft (URS, 2012). Analysis conducted as part of the 2012 Engineering Design for the 5280 Lift (URS, 2012) shows the outlet has capacity to adequately manage flow during and following the design level storm event, defined as the 72-hour probable maximum precipitation (PMP). In addition, the LAI pond was constructed with sufficient depth to fully contain the storm run-on on top of the operational maximum storage pool water level in the event the spillway was inoperable and could not pass flow to the LDWP. This water level, defined as the maximum surcharge pool water level (flood water level), is estimated at EL 5,277.2 ft based on an expected water level rise of 2.0 ft during the probable maximum flood (PMF) (URS, 2014). The surcharge pool water level leaves 2.8 ft of freeboard below the embankment crest.

Standpipe piezometers, vibrating wire piezometers, inclinometers, and survey settlement/displacement monument devices are installed at the LAI to monitor the performance of the embankment and the North Toe Buttress. Measurements from the monitoring instruments are reviewed and documented annually as part of the annual inspection. Starting on October 19, 2015, the piezometers and survey monuments are read at intervals not exceeding 30 days per the requirements of 40 CFR § 257.83(a)(1)(iii). The locations of the instruments are shown on Figure 1-2.

Inspections of the LAI are performed by a qualified person at intervals not exceeding seven days. The inspections examine the LAI for actual or potential conditions that could disrupt the operation or safety of the impoundment and documents the results of the inspection in the facility’s operating record. In addition, a more detailed annual inspection is performed by a qualified professional engineer. The annual inspection includes a review of available information on the dam including the past year of monitoring data, a field inspection of the dam, abutment, and downstream toe, and documentation of findings and recommendations in a dam safety inspection report. The most recent annual inspection of the LAI was performed on October 14, 2015 (AECOM & APS, 2016).
2 Hazard Potential Classification

This section summarizes the initial Hazard Potential Classification (HPC) for the LAI. 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 on 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.

The Rule 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 LAI 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 Classification, the structure is assessed to determine whether it meets the criteria for High Hazard Potential. The potential for loss of life differentiates between high and significant hazard potential in the Final CCR Rule; therefore, if the Dam does not meet the criteria for high hazard potential, it would be classified as Significant Hazard Potential structure.

The potential for downstream loss of life is assessed by reviewing land use in areas downstream (to the west) from the Dam, where inundation is likely in the event of a release. A dam break analysis and inundation mapping has been documented for the LAI (URS, 2009). The inundation was reportedly mapped downstream in the Chaco River to the San Juan River. No habitable structures were reported in the inundation area and the flood outflow passes beneath the Highway N36 Bridge (URS, 2009). United States Geological Survey (USGS) 7.5-Minute Quadrangle topographic map of The Hogback North, NM and associated digital orthoimage data (USGS, 2013) were also 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 LAI Dam and its immediate surroundings relative to property lines, surface water bodies, and structures that could potentially be impacted by a release, as presented in the dam break analysis report (URS, 2009), indicated that the LAI Dam does not meet the criteria for a Low Hazard Potential classification based on the proximity to an off-site surface water body (Chaco River).

The Chaco River is approximately 4,000 ft downstream from the LAI Dam. The area between the LAI and the Chaco River is unoccupied and undeveloped. No permanent or temporary dwellings, worksites, roads, or other development that would indicate the routine presence of people downstream from the LAI (off-site) were identified. Therefore, the LAI Dam does not meet the criteria for a High Hazard Potential classification based on the absence of probable loss of life resulting from failure or mis-operation. Because the LAI Dam does not meet the criteria for classification as either Low Hazard Potential or High Hazard Potential, it is classified as a Significant Hazard Potential CCR surface impoundment.
3 History of Construction

This section summarizes the history of construction for the LAI. 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 LAI. 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 LAI Construction Summary

The history of construction dating back to the original construction that began in 2003 is summarized in Table 3-1 below.
### Table 3-1. History of Construction for the LAI

<table>
<thead>
<tr>
<th>Item</th>
<th>As-Constructed/ Current</th>
<th>Comments</th>
<th>Reference Document</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Name and Address of Owner</strong></td>
<td>Arizona Public Service Company (APS): 400 North 5th Street, Phoenix, Arizona 85004</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td><strong>State ID No.</strong></td>
<td>D-634</td>
<td>---</td>
<td>Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM &amp; APS, 2016)</td>
</tr>
<tr>
<td><strong>Size Classification</strong></td>
<td>Intermediate</td>
<td>---</td>
<td>Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM &amp; APS, 2016)</td>
</tr>
<tr>
<td><strong>Hazard Classification</strong></td>
<td>Significant</td>
<td>---</td>
<td>See Section 2.2</td>
</tr>
</tbody>
</table>
| **Construction Date**       | 2004 – 5228 (original construction)  
2007 – 5248 Lift  
2010 – 5258 Lift  
2012 – 5270 Lift  
• LAI 5248 Lift Construction Completion Report (APS, 2008)  
• LAI 5258 Lift Construction Completion Report (APS, 2011)  
• LAI 5270 Lift Construction Completion Report (URS, 2012)  
• LAI 5280 Lift Construction Completion Report (URS, 2014) |
<p>| <strong>Location on USGS Quadrangle Map</strong> | The Hogback North Quadrangle: Sections 34 and 35, Township 29 North, Range 16 West | See Figure 3-1 | The Hogback North Quadrangle (USGS, 2013) |
| <strong>Statement of Purpose</strong>    | CCR Disposal            | ---      | NMSOE Certificate of Construction (2015)                                            |
| <strong>Name of Watershed</strong>       | Chaco                   | ---      | NMSOE Certificate of Construction (2015)                                            |
| <strong>Size of Watershed (ac)</strong>  | 135.9                  | ---      | LAI Engineering Design Report (URS, 2012))                                          |
| <strong>Area Capacity Curve</strong>     | Figure 3-2             | ---      | LAI Engineering Design Report (URS, 2012)                                           |
| <strong>Embankment Type</strong>         | Zoned earth and ash fill dam |           | LAI 5280 Lift Construction Completion Report (URS, 2014)                             |
| <strong>Embankment Maximum Height (ft)</strong> | 107                    | ---      | NMSOE Certificate of Construction (2015)                                            |</p>
<table>
<thead>
<tr>
<th>Item</th>
<th>As-Constructed/ Current</th>
<th>Comments</th>
<th>Reference Document</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design Total Freeboard (ft)</td>
<td>4.8</td>
<td>Residual freeboard following PMF event is 2.8 ft</td>
<td></td>
</tr>
<tr>
<td>Embankment Length (ft)</td>
<td>6,600</td>
<td></td>
<td>NMSOE Certificate of Construction (2015)</td>
</tr>
<tr>
<td>Embankment Crest Elevation (ft)</td>
<td>5,280</td>
<td>Crest elevation is along the West Embankment; crest is at higher elevation along other sections of the impoundment.</td>
<td>NMSOE Certificate of Construction (2015)</td>
</tr>
<tr>
<td>Embankment Crest Width (ft)</td>
<td>30 (West Embankment), 20 (South Embankment)</td>
<td></td>
<td>5280 Lift As-built Drawings (URS, 2014)</td>
</tr>
<tr>
<td>Embankment Slopes</td>
<td>West Embankment: 3H:1V (US and DS); East Embankment: 3H:1V (DS); 2H:1V (US); Northwest Embankment: 3H:1V (US); 4H:1V (DS); South Embankment 2H:1V (US and DS)</td>
<td>The Northwest Embankment was flattened to 4H:1V as part of the Ash Pond 6 closure project in November 2014.</td>
<td>5280 Lift As-Built Drawings (URS, 2014) Ash Pond 6 Closure Construction Completion Report (URS, 2016)</td>
</tr>
<tr>
<td>Slope Protection</td>
<td>HDPE liner and clay on upstream slope</td>
<td></td>
<td>5280 Lift As-Built Drawings (URS, 2014)</td>
</tr>
<tr>
<td>Maximum Storage Level (ft)</td>
<td>5,275.2</td>
<td></td>
<td>LAI Engineering Design Report (URS, 2012)</td>
</tr>
<tr>
<td>Surface Area (ac)</td>
<td>129.16</td>
<td>At EL 5,280 ft</td>
<td>LAI Engineering Design Report (URS, 2012)</td>
</tr>
<tr>
<td>Material Properties</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Clay Blanket**

**Physical Properties**: The clay blanket consists of compacted lean clay obtained from on-site borrow sources.

**Engineering Properties**:  
- Unit Weight = 125 pounds per cubic foot (pcf)  
- Cohesion = 3,000 psf pounds per square foot (psf)  
- Friction Angle = 20°

LAI 5280 Lift Construction Completion Report (URS, 2014)  
LAI Engineering Design Report (URS, 2012)
<table>
<thead>
<tr>
<th>Item</th>
<th>As-Constructed/ Current</th>
<th>Comments</th>
<th>Reference Document</th>
</tr>
</thead>
<tbody>
<tr>
<td>Physical Properties</td>
<td>Compacted bottom ash and fly ash.</td>
<td>---</td>
<td>LAI 5280 Lift Construction Completion Report (URS, 2014)</td>
</tr>
<tr>
<td><strong>Engineering Properties</strong></td>
<td><strong>Bottom Ash</strong></td>
<td></td>
<td>LAI Engineering Design Report (URS, 2012)</td>
</tr>
<tr>
<td></td>
<td>Unit Weight = 75.1 pcf</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cohesion = 0 psf</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>Friction Angle = 42°</td>
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<td></td>
<td><strong>Fly Ash</strong></td>
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<td></td>
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<tr>
<td></td>
<td>Unit Weight = 90 psf</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Cohesion = 0 psf</td>
<td></td>
<td></td>
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<tr>
<td></td>
<td>Friction Angle = 30° (dry), 28° (saturated)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation</td>
<td><strong>Physical Properties</strong></td>
<td></td>
<td>LAI Engineering Design Report (URS, 2012)</td>
</tr>
<tr>
<td></td>
<td>The LAI was constructed on top of Ash Ponds 3, 4, and 5.</td>
<td></td>
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<tr>
<td></td>
<td>The perimeter embankment began as an extension of the Ash</td>
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<tr>
<td></td>
<td>Pond 3 and 4 embankments. A series of downstream raises</td>
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<tr>
<td></td>
<td>resulted in the LAI South and East Embankments being</td>
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<tr>
<td></td>
<td>founded on native weathered shale, the LAI Northwest</td>
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<tr>
<td></td>
<td>Embankment being founded on the Ash Pond 3 and 5</td>
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<tr>
<td></td>
<td>embankments, and the LAI West Embankment being founded on</td>
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<tr>
<td></td>
<td>existing fly ash impounded within Ash Pond 3. The</td>
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<td></td>
<td>initial construction included addition of geogrid</td>
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<td></td>
<td>reinforcement below the West Embankment to limit</td>
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<tr>
<td></td>
<td>potential settlement.</td>
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<td></td>
<td><strong>Engineering Properties</strong></td>
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<tr>
<td></td>
<td><strong>Native Weathered Shale:</strong></td>
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<tr>
<td></td>
<td>Unit Weight = 130 pcf</td>
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<td></td>
<td>Effective Cohesion = 500 psf</td>
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<tr>
<td></td>
<td>Effective Friction Angle = 30°</td>
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<td></td>
<td><strong>Fly Ash</strong></td>
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<td></td>
<td>Unit Weight = 90 psf</td>
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<td></td>
<td>Cohesion = 0 psf</td>
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<tr>
<td></td>
<td>Friction Angle = 28° (saturated)</td>
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<tr>
<td>Abutments</td>
<td><strong>Physical Properties</strong></td>
<td></td>
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<tr>
<td></td>
<td><strong>Native Weathered Shale:</strong></td>
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<td></td>
<td>Unit Weight = 130 pcf</td>
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<td>Effective Cohesion = 500 psf</td>
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<td></td>
<td>Effective Friction Angle = 30°</td>
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<td></td>
<td><strong>Fly Ash</strong></td>
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<td></td>
<td>Unit Weight = 90 psf</td>
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<td>Cohesion = 0 psf</td>
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<td></td>
<td>Friction Angle = 28° (saturated)</td>
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<td>Item</td>
<td>As-Constructed/ Current</td>
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<td>Reference Document</td>
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<tr>
<td>Engineering Properties</td>
<td>Native Weathered Shale:</td>
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<tr>
<td></td>
<td>• Unit Weight = 130 pcf</td>
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<td></td>
<td>• Effective Cohesion = 500 psf</td>
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</tr>
<tr>
<td></td>
<td>• Effective Friction Angle = 30°</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spillway</td>
<td>A drop inlet structure consisting of an 8-foot diameter perforated HDPE pipe allows flows to pass into the adjacent LDWP.</td>
<td>The LAI maximum storage level was established to allow full containment of the PMF and maintain a residual freeboard to account for wave run-up.</td>
<td>LAI Engineering Design Report (URS, 2012)</td>
</tr>
<tr>
<td>Construction Specifications</td>
<td>Bottom Ash:</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>• Fill lift thickness = 8 inches (5280 Lift)</td>
<td></td>
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<tr>
<td></td>
<td>• Fill lift thickness = 8 inches (5258 and 5270 Lifts)</td>
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<tr>
<td></td>
<td>• Minimum degree of compaction = 95% of the Standard Proctor (5280 Lift)</td>
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<tr>
<td></td>
<td>• Minimum degree of compaction = method specification designed to meet 95% of the Standard Proctor (5258 and 5270 Lifts)</td>
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</tr>
<tr>
<td></td>
<td>• Field test frequency</td>
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<tr>
<td></td>
<td>o Method specification (5258, 5270, and 5280 Lifts)</td>
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<td></td>
<td>o Random testing (5258 and 5270 Lifts)</td>
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<tr>
<td></td>
<td>o One nuclear density test per 2,000 yds$^3$ placed (5280 Lift)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>o One Sand Cone Field Density test per four nuclear density gauge tests (5280 Lift)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Specifications apply to the field testing portions of the 5258, 5270, and 5280 Lifts unless otherwise noted.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Bottom ash was exclusively used as embankment fill until the 5270 Lift.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• Zoned layers of either bottom ash or fly ash material were used as embankment fill for the 5270 Lift.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• The 5280 Lift utilized both bottom ash and fly ash, but did not blend the materials together.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• LAI Method Specification Compacted Bottom Ash Test Fill Section (Western Technologies, 2003)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• LAI 5258 Lift Specifications (APS, 2008)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• LAI 5270 Lift (GEOMAT, 2010)</td>
<td></td>
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</tr>
<tr>
<td></td>
<td>• LAI 5270 Lift Specifications (URS, 2010)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>• LAI 5280 Lift Specifications (URS, 2011)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Item</td>
<td>As-Constructed/ Current</td>
<td>Comments</td>
<td>Reference Document</td>
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<tr>
<td>------</td>
<td>-------------------------</td>
<td>----------</td>
<td>--------------------</td>
</tr>
</tbody>
</table>
| Fly Ash (5280 Lift): | • Fill lift thickness = 8 inches  
• Initial minimum degree of compaction = 95% (Standard Proctor)  
• Field test frequency  
  ◦ Method specification  
  ◦ One nuclear density test per 2,000 yds³ placed  
  ◦ One Sand Cone Field Density test per four nuclear density gauge tests | • Specifications apply to the field testing portions of the 5258, 5270, and 5280 Lifts unless otherwise noted.  
• Bottom ash was exclusively used as embankment fill until the 5270 Lift.  
• A bottom ash-fly ash blended material was used as embankment fill for the 5270 Lift. The 5280 Lift utilized both bottom ash and fly ash, but did not blend the materials together. | • LAI Method Specification Compacted Bottom Ash Test Fill Section (Western Technologies, 2003)  
• LAI 5258 Lift Specifications (APS, 2008)  
• LAI 5270 Lift (GEOMAT, 2010)  
• LAI 5270 Lift Specifications (URS, 2010)  
LAI 5280 Lift Specifications (URS, 2011) |
| Bottom Ash and Fly Ash Ballast (combined material – 5270 Lift): | • Fill lift thickness = 8 inches  
• Initial minimum degree of compaction = 95% (Standard Proctor)  
• Field test frequency  
  ◦ Method specification  
  ◦ One nuclear density test per 250 yds³ placed | | |
| Compacted Clay: | • Fill was moisture-conditioned to be within -1 to +3 percent of the optimum moisture content  
• Initial minimum degree of compaction = 95% (Standard Proctor)  
• Field test frequency  
  ◦ Method specification  
  ◦ One nuclear density test per 250 yds³ placed  
  ◦ One Standard Proctor per 5,000 yds³ placed  
One Sand Cone Field Density test per four nuclear density gauge tests | | |
<p>| Detailed Drawings | Appendix A | --- | --- |</p>
<table>
<thead>
<tr>
<th>Item</th>
<th>As-Constructed/ Current</th>
<th>Comments</th>
<th>Reference Document</th>
</tr>
</thead>
</table>
| Type and Purpose of Instrumentation | **Standpipe piezometers installed to monitor the phreatic levels in and below the embankment.**  
**Vibrating wire piezometers installed to monitor the phreatic levels in and below the embankment.**  
**Inclinometers to monitor horizontal deflections within the West Embankment and North Toe Buttress profiles.**  
**Settlement rods to monitor vertical movement of the embankment and North Toe Buttress.**  
**Settlement monuments to monitor horizontal and vertical deflections within the West Embankment.** | ---                           | LAI and LDWP Comprehensive Instrumentation Plan (URS, 2014)                      |
| Location of Instrumentation       | The West Embankment and North Toe Buttress.                                               | See Figure 1-2              | LAI and LDWP Comprehensive Instrumentation Plan (URS, 2014)                      |
| Provisions for Surveillance, Maintenance and Repair | **Visual inspections of the dam by a qualified person on a frequency not exceeding seven days.**  
**Visual inspections of the dam conducted annually by a professional engineer.**  
**Phreatic level behavior from piezometric measurements collected on a frequency not exceeding 30 days.**  
**Embankment settlement using movement monuments survey data collected on a frequency not exceeding 30 days.** | ---                           | Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016) |
| Record of Structural Instability   | Reports of seepage along downstream toe of South Embankment during prior construction stages. Flows now believed to have been drainage from placed bottom ash embankment fill. | Drainage is now captured by French drain. | Annual CCR Impoundment and Landfill Inspection Report 2015 (AECOM & APS, 2016) |
4 Structural Stability Assessment

This section summarizes the structural stability assessment for the LAI. 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 LAI was constructed in phases starting in 2003. The impoundment was originally constructed on Ash Ponds 3 and 4 to EL 5228 ft and progressively raised in four lifts using the downstream method of construction to EL 5248, 5258, 5270 and 5280 ft. As the embankment was raised it expanded onto Ash Ponds 5 and 6. The foundation of the perimeter embankment varies around the LAI. The South Embankment was constructed using the existing Ash Pond 4 embankment as a starter dike and expanded downstream onto the adjacent native weathered shale. The West Embankment is founded on the Ash Pond 3 and 4 divider dike and on old hydraulically deposited fly ash within Ash Pond 3. The Northwest Embankment is founded on the Ash Pond 5 and Ash Pond 6 embankments and hydraulically deposited fly ash within Ash Pond 6. The East Embankment lies on top of the native weathered shale. The fly ash beneath the West Embankment is approximately 45 ft thick. Prior to construction of the embankment along the West Embankment, the foundation surface was reinforced with geogrid and compacted bottom ash. The underlying weathered shale appears competent within the embankment footprint based on exploratory borings drilled to bedrock during the 2016 AECOM Geotechnical Investigation.

Along the northern portion of the West Embankment in an area bounded to the south by the LDWP embankment and to the north by the Ash Pond 6 embankment, an approximately 10 to 15 foot thick layer of wet fly ash was identified downstream at the toe of the LAI embankment prior to construction of the EL 5280 ft lift. This area, referred to as the North Toe area, is in the vicinity of a decant pond when Ash Pond 3 was active. In the decant pond, finer fly ash materials were deposited with relatively low densities. The area was also poorly drained, due to the underlying weathered shale that prevents downward flow. To stabilize the embankment slope, a North Toe Buttress was constructed in conjunction with the EL 5280 ft lift. The buttress was constructed by first installing wick drains on a nine foot triangular pattern to depths of up to 50 ft in order to allow pore pressure relief ("drainage") of the material as the buttress was installed. Next a wedge-shaped buttress of compacted bottom ash was installed from a 20-foot height at the toe of the LAI embankment, westward at a 20H:1V slope for a distance of 400 ft. The primary effect of the North Toe Buttress is to provide stability for the downstream toe of the West Embankment, but a secondary effect was also to drain, densify, and strengthen the wet fly ash as a result of the wick drainage and the surcharge load from the buttress fill. Water level readings from piezometers installed through the North Toe Buttress after construction indicate that the fly ash is draining.

To limit seepage through the LAI embankment and foundation, the bottom and the upstream slope of the impoundment is lined with an HDPE geomembrane liner to the crest. In addition, a 15-foot thick layer of compacted clay is installed on the upstream slope beneath the geomembrane effectively providing a composite liner system. The clay layer is supplemented by an approximately 1290-foot long by 32-foot high, cement-bentonite slurry wall along the Northwest Embankment of the LAI. The slurry wall was installed to seal possible gaps in the clay core that were discovered during field investigations of the existing Ash Pond 5 embankment. Piezometer readings from the most recent dam inspection report (AECOM & APS, 2016) show water levels below the foundation of the Western and Northwestern Embankments, an indication that the liner system is limiting seepage.

Review of the measured displacements of the survey rods and inclinometers at the LAI, as presented in the 2015 annual dam inspection report (AECOM & APS, 2016), indicates no significant settlements or horizontal displacement along the crest of the dam within the year. The relatively small settlement and horizontal movements measured at the LAI are an indication of stability in the dam foundation.
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.”

The upstream slopes of the LAI are lined with an HDPE liner, which protects the slopes from erosion, wave action, and adverse effects of sudden drawdown. The downstream slopes consist of compacted bottom and fly ash and are not vegetated; however, the granular nature of bottom ash allows infiltration in preference to runoff and erosion. Additionally, APS has a program to regularly inspect, identify, and repair any erosion rills. The 2015 annual dam inspection report (AECOM & APS, 2016) reported that the downstream slopes of the embankments show evidence of minor erosion rilling, presumably caused by rainfall runoff. APS maintains the affected areas by regrading and recompacting eroded areas.

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 LAI embankment is composed primarily of compacted bottom and fly ash and compacted clay, which has been demonstrated to compact readily with various ranges of compaction and hauling equipment. The embankment was constructed by placement of soils in mechanically compacted thin lifts of eight inches or less. Construction control of the compaction process was maintained using a method procedure where the soil preparation, placement, watering, discing (if necessary), and compaction are specified based on the results of testing during earthwork. Quality control testing was performed to check the bottom ash was reaching the minimum project requirements for compaction, defined as 95 percent of the Standard Proctor dry density (American Society for Testing and Materials [ASTM] D698).

Construction records of the embankment indicate quality control testing of the bottom ash and compacted clay fills were performed for each raise of the LAI embankment. Quality control testing consisted of Standard Proctor moisture-density relationship determination (ASTM D698), sieve analysis testing (ASTM C136, C117), Atterberg Limit testing (ASTM D4318), in-place moisture content and density measurement using the nuclear method (ASTM D6938), and in-place density measurement using the sand cone method (ASTM D1556). Construction completion reports (GEOMAT, 2014) indicate that all locations tested met the project minimum density requirements.

Based on review of the construction records/completion report for the LAI raises, the embankments appear to be constructed with well compacted materials.

4.4 Slope Vegetation

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

As noted in Section 4.2, the downstream slope, which is comprised of compacted bottom and fly ash, are not vegetated. APS has a program to regularly inspection and repair erosion rills. The upstream slope consists of a dual HDPE liner and therefore 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 section. 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 LAI consists of a drop inlet tower, located adjacent to the West Embankment. This drop inlet is a vertical, eight foot diameter, polyethylene pipe with multiple drilled holes to allow inflow of water. This drop inlet is surrounded by gravel
and sorted bottom ash that filters solids from the water flowing into the drop inlet. The drop inlet tower drains to the west through an 8-inch or adjacent 16-inch HDPE pipe, through the West Embankment of the LAI, to the LDWP. Analysis conducted as part of the 2012 Engineering Design for the EL 5280 ft Lift (URS, 2012), shows the spillway has capacity to adequately manage flow during and following the peak discharge from the full PMF, which exceeds the requirement for the significant hazard rating for this CCR Unit. In addition, the LAI pond was constructed with sufficient depth to fully contain the storm run-on on top of the operational maximum storage pool water level in the event the spillway was inoperable and could not pass flow to the LDWP. Recent inspections of the spillway (AECOM & APS, 2016), found it to be in good working order with no visible damage.

Based on the 2012 Engineering Design for the EL 5280 Lift and the most recent inspection report, the LAI has been designed, constructed, and maintained to adequately manage flow during and following the peak discharge of the 72-hour PMP event, as required for significant hazard potential impoundments. Furthermore, 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.”

Three hydraulic structures penetrate the LAI embankment. The first structure consists of the two outlet pipes connected to the drop inlet tower as discussed in Section 4.5. The outlets penetrate the Western Embankment at about Sta 54+25 connecting the drop outlet to the LDWP. Water drains via gravity to the west through an 8-inch HDPE pipe. A 16-inch diameter HDPE pipe, located above the 8-inch drain, provides additional outflow capacity if the 8-inch pipe is blocked or in the case of large (e.g., storm) flows. A clean-out is located at the west end of the 8-inch pipe. The pipes were encased in flowable fill through the embankment. The most recent inspection report of the LAI (AECOM & APS, 2016) speculated the 8-inch pipe was partially blocked with sediment due to low flow volume observed at the outlet. The 16-inch pipe appeared to be flowing normally. The interconnection of the 8-inch pipe with the LDWP is not required for flood management purposes since the minimum freeboard of the LAI is sufficient to store the directly-incident PMP; however, continued monitoring of the flow from the 8-inch pipe and, if warranted, removal of the lodged material is recommended.

The second hydraulic structure is the solid wall outlet of a buried slotted drain pipe, known as the “dead pool drain”, installed above the geomembrane liner at the inside toe of the West Embankment of the LAI. The pipe runs from a point approximately 200 ft south of the drop inlet spillway to the southwest corner of the LAI, where it transitions to a solid wall section, penetrates the embankment, and outfalls to a sump. This pipe is embedded in flowable fill within the starter embankment associated with the EL 5228 ft Lift. Accumulated water is pumped from the sump through a four inch pipe to the southeast corner of the LDWP. Operation of the dead pool drainpipe is not required for safe operation of the LAI but is encouraged because it assists with dewatering, consolidation, and strengthening of the impounded CCRs. During the most recent impoundment inspection (AECOM & APS, 2016), the drain system was pressurized but not operational; however, recent discussions with APS Personnel indicate the pump system is now operational.

The third hydraulic structure is a seepage intercept drain located within the downstream toe of the South Embankment. The drain was constructed beyond the embankment toe during the EL 5270 ft lift construction, was covered further by the downstream slope of the EL 5280 ft lift, and will be buried fully by Cell 3 of the Dry Fly Ash Disposal Area landfill that will eventually buttress the South Embankment of the LAI. The drain was originally intended to provide improved control of drainage from the embankment fill and prevent water from ponding between the South Embankment and the Dry Fly Ash Disposal Area landfill to the south. The toe drain consists of a buried channel with a 13-foot wide bottom and 1.5H:1V side slopes. The channel is about 3.5 ft deep and extends westward from the east end of the South Embankment, to an existing channel located along the outside toe of the south embankment of the LDWP. Two 10-inch inside diameter perforated drain pipes are installed in the channel and surrounded by a two-stage filter/drain. The pipes outfall into a channel adjacent to the LDWP where a headwall is located which facilitates monitoring flow rates from the two pipes. The flows from the drain are conveyed through the channel to a sump and then pumped to the LDWP. The most recent inspection report (AECOM & APS, 2016), observed no water flow from the toe drain pipes. The lack of flow is attributed to a lack of drainage from fresh embankment fill, a lack of seepage flow from the LAI itself, and an absence of shallow saturation in the foundation.
Other than the low flow volume observed in the 8-inch pipe connected to the drop inlet spillway, the hydraulic structures
penetrating the LAI embankment appear to be working effectively with no evidence of subsidence or other indication of
potential deterioration of the surrounding embankment.

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 LDWP is located near the downstream toe of the Western Embankment of the LAI and contains water decanted from the
LAI for reuse at the power plant. The LDWP is contained by an embankment with a double liner and leak detection system;
therefore, making it unlikely under normal operating conditions to inundate the toe of the LAI. If the embankment or
containment system were to breach in some manner, the water would follow the graded terrain that slopes away from the LAI
and not inundate the LAI slopes.

Since it is unlikely that water from the LDWP would inundate the downstream slope of the LAI and no other bodies of water are
present that could also reasonably inundate the slopes, no structural stability deficiencies are presently associated with
inundation of the downstream slope of the LAI by an adjacent body of water.

4.8 Other Issues

No deficiencies were identified for the LAI that could affect the structural stability of the impoundment. The most recent dam
inspection (AECOM & APS, 2016) reported observations of minor erosion rills on the downstream slopes. APS reportedly has
been maintaining affected areas by regrading and recompacting eroded areas. It is recommended that the program be
continued and that rills be repaired if the depth exceeds one foot.

4.9 Structural Stability Assessment Results

AECOM did not identify any structural stability deficiencies that would affect the structural condition of the LAI CCR surface
impoundment based on the documents provided and reviewed as part of this assessment. AECOM assesses that the design,
construction, operation and maintenance of the LAI 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 LAI. This assessment is intended to satisfy the requirement of Rule 40 CFR § 257.73(e).

5.1 Methodology and Design Criteria

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

5.2 Critical Cross Section

Safety factors were calculated for three cross sections of the LAI 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, stratigraphic conditions, and presence of sensitive soils. Subsurface soil profiles were developed using information from the EL 5280 ft Lift (URS, 2012). The locations of the cross sections along the LAI embankment are shown in Figure 5-1. The three cross sections analyzed are:

Section A (West Embankment): This cross-section is located at Section A (Sta 43+76.99) as shown in Figure 5-1 and the as-built section in Appendix A. The section represents the maximum height along the West Embankment. The embankment is approximately 76 ft high from crest to downstream toe at this location with a crest width of 30 ft. The upstream slope is inclined at 2H:1V, while the downstream slope is inclined at 3H:1V. The embankment at this section consists of compacted bottom and fly ash with a 15-foot wide compacted clay blanket on the upstream slope. The embankment bears partially on the pre-existing Ash Pond 3 and 4 Divider Dike and partially on 46 ft of old hydraulically placed fly ash associated with Ash Pond 3.

Section M (South Embankment): This cross-section is located at Section M (Sta 36+47.02) as shown in Figure 5-1 and the as-built section in Appendix A. The section represents the maximum height along the South Embankment. The embankment is approximately 92 ft high from crest to downstream toe at this location with a crest width of 30 ft. Both the upstream and downstream slopes are inclined at 2H:1V. A pre-existing Ash Pond 4 Embankment was used as a starter dam for building the South Embankment using a downstream construction method. The pre-existing embankment consisted of compacted bottom and fly ash with a 40-foot wide layer of compacted clay on the upstream face. The subsequent raises of the LAI are constructed with compacted bottom and fly ash and a 15-foot wide blanket of compacted clay on the upstream slope. The clay blanket is keyed into the clay of the pre-existing (original) embankment. A toe drain that runs parallel with the embankment is located near the downstream toe of the embankment. The South Embankment is founded on native weathered shale.

Section X (North Toe Buttress): This cross-section is located at Section X (Sta 59+62.02) as shown in Figure 5-1 and the as-built section in Appendix A. The section crosses the North Toe Buttress at the northern end on the West Embankment. The North Toe Buttress was constructed against the base of the West Embankment to a height of 20 ft and is inclined downward toward the west at approximately 20H:1V to an approximate length of 400 ft. The buttress consists of a 1.5-foot thick bottom ash drain layer overlain by a combination of compacted bottom and fly ash to the approximate elevation of 5,227 ft at its highest point. The buttress overlies hydraulically placed wet fly ash (sensitive fines) associated with Ash Pond 3, improved by the installation of full-depth vertical wick drains at nine foot spacing.

The West Embankment at the Section X location is approximately 70 ft high from crest to downstream toe with a crest width of 30 ft. The upstream slope is inclined at 2H:1V and the downstream slope is inclined at 3H:1V. The embankment consists of
compacted bottom and fly ash with a 15-foot wide compacted clay blanket on the upstream slope. The embankment bears partially on the pre-existing Ash Pond 3 and 4 Divider Dike and partially on 25 ft of old hydraulically placed fly ash associated with Ash Pond 3.

5.3 Subsurface Stratigraphy

Idealized models of subsurface stratigraphic conditions for each cross section were developed based on as-built drawings (Appendix A). The stratigraphic units described as follows were used to develop SLOPE/W models for each cross section.

**Compacted Bottom and Fly Ash:** The LAI Embankment primarily consists of compacted bottom and fly ash. The bottom and fly ash provides stability to the embankment, but because of its relatively high hydraulic conductivity is not relied upon to control seepage from the pond which is managed by a HDPE geomembrane liner and compacted clay liner on the upstream slope. The compacted bottom classifies as a Silty Sand (SM) and the fly ash as a Silt (ML) based on the Unified Soil Classification System (USCS).

**Compacted Clay:** The LAI Embankment includes a less pervious layer of compacted clay along the upstream slope. The layer is about 15 ft wide and runs from the toe to the crest. The clay material was obtained from local weathered shale, broken down and mechanically compacted in lifts. The compacted clay consists predominately of Lean Clay (CL) based on the USCS.

**Existing Fly Ash:** Fly ash and FGD slurry waste product from the power generating process is associated with the decommissioned Ash Pond 3 and Ash Pond 4. The fly ash was pumped from the plant to the ash ponds and allowed to settle hydraulically. The LAI was constructed partially on top of the existing fly ash deposits. The existing fly ash classifies as a silt (ML) based on the USCS.

**New Fly Ash:** New fly ash and FGD slurry waste product from the power generating process is stored within the LAI impoundment.

**Buttress Material:** The North Toe Buttress consists of compacted bottom ash. The buttress material provides stability to the toe of the Western Embankment between the LDWP embankment to the south and the Ash Pond 6 embankment to the north. The buttress also has a secondary effect of increase the density and confinement pressure of the underlying wet existing fly ash.

**Sensitive Fines:** Sensitive soils consisting of wet fly ash material associated with hydraulic deposition of fly ash within Ash Pond 3. Sensitive fines were observed in the North Toe area and underlie portions of the North Toe buttress. The sensitive fines are potentially susceptible to liquefaction.

**Weathered Shale:** Bedrock beneath the embankment consists of weathered shale of the Cretaceous-age Lewis Shale Formation.

5.4 Material Properties

Material properties for soil, rock and embankment construction materials were developed based on an analysis and interpretation of historical geologic and geotechnical data presented in the URS Corporation, “Engineering Design Report Lined Ash Impoundment 5280 Lift” (URS, 2012). The material properties developed by the embankment designers and subsequent investigators were assessed for reliability and applicability to this safety factor assessment. The EL 5280 ft Lift Engineering Design Report (URS, 2012) presents soil strength parameters obtained from laboratory testing.

The material properties selected for use in the slope stability analyses of the LAI Embankment are presented in Table 5-1 and are the same as those used for the URS slope stability evaluation (2012). Additional properties for the residual strength of the sensitive fines material (liquefaction) were developed for this assessment. Details of the residual strength properties development are presented in the Appendix B calculation.
Table 5-1. Selected Material Parameters – LAI Safety Factor Assessment

<table>
<thead>
<tr>
<th>Material</th>
<th>Moist Unit Weight, $\gamma_m$ (pcf)</th>
<th>Saturated Unit Weight, $\gamma_{sat}$ (pcf)</th>
<th>Drained Strength</th>
<th>Undrained Strength</th>
<th>Residual Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Cohesion, $c'$ (psf)</td>
<td>Friction Angle, $\phi'$ (degrees)</td>
<td>Cohesion, $c$ (psf)</td>
</tr>
<tr>
<td>Compacted Bottom and Fly Ash</td>
<td>90</td>
<td>90</td>
<td>0</td>
<td>35</td>
<td>-</td>
</tr>
<tr>
<td>Existing Fly Ash (Top)</td>
<td>90</td>
<td>90</td>
<td>0</td>
<td>30</td>
<td>-</td>
</tr>
<tr>
<td>Existing Fly Ash (Bottom)</td>
<td>90</td>
<td>90</td>
<td>0</td>
<td>28</td>
<td>-</td>
</tr>
<tr>
<td>New Fly Ash (Impounded)</td>
<td>90</td>
<td>90</td>
<td>-</td>
<td>-</td>
<td>304</td>
</tr>
<tr>
<td>Compacted Bottom Ash</td>
<td>75.1</td>
<td>75.1</td>
<td>0</td>
<td>42</td>
<td>-</td>
</tr>
<tr>
<td>Compacted Clay</td>
<td>125</td>
<td>130</td>
<td>300</td>
<td>20</td>
<td>-</td>
</tr>
<tr>
<td>Weathered Shale (Native Ground)</td>
<td>120</td>
<td>125</td>
<td>-</td>
<td>-</td>
<td>-</td>
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<tr>
<td>Sensitive Fines (Drained)</td>
<td>80</td>
<td>-</td>
<td>0</td>
<td>18.5</td>
<td>-</td>
</tr>
<tr>
<td>Sensitive Fines (Liquefaction)</td>
<td>80</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Buttress Fill</td>
<td>75</td>
<td>-</td>
<td>0</td>
<td>20</td>
<td>-</td>
</tr>
<tr>
<td>Drain Sand</td>
<td>110</td>
<td>-</td>
<td>0</td>
<td>30</td>
<td>-</td>
</tr>
</tbody>
</table>

5.5 Embankment Pore Pressure Distribution

Water levels within the embankment are anticipated to be low because of the geosynthetic liner that lines the pond basin and the compacted clay layer that extends along the upstream slope of the embankment. Standpipe and vibrating wire piezometers are installed along the West Embankment of the LAI, monitored on an interval not exceeding 30 days, and reported annually in an inspection report. These instruments were considered to be the most reliable indicators of pore pressure distribution within the LAI embankment. They indicate the water levels are beneath the embankment and within the underlying existing fly ash deposits that make up a majority of the foundation below the West Embankment (AECOM & APS, 2016). Consequently, the phreatic levels in the stability cross sections were modeled below the embankments within the existing fly ash foundation or weathered shale and the steady-state seepage condition within the embankment was modeled as a dry condition. The locations of the piezometers are shown on Figure 1-2.

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 Loading

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 the anticipated 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 design operating level stated in the EL 5280 ft Lift Engineering Design Report (URS, 2012). The loading condition uses shear strengths expressed as effective stress and with pore water pressures that correspond to the long-term condition.

For the LAI embankment, the safety factor was calculated for the long-term, maximum storage pool at 5,275.2 ft (URS, 2012).

**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 LAI embankment, the safety factor was calculated for the maximum surcharge pool at 5,277.2 ft (URS, 2012).

**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, $\text{PGA}_{\text{M}}$;
3. Calculate the maximum transverse acceleration at the crest of the embankment, $\text{PGA}_{\text{crest}}$, using the $\text{PGA}_{\text{M}}$ from step two; and
4. Adjust the $\text{PGA}_{\text{crest}}$ 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 for Section M which is founded on weathered shale and 0.104g for Sections A and X which are founded on existing fly ash.

The water level in the LAI for the seismic loading analysis was set to 5,275.2 ft 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 (existing fly ash 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 and low phreatic surfaces within the embankment.

**Liquefaction Loading:** 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.
Saturated, low-density sensitive fines were encountered in the North Toe Area adjacent to the downstream slope of the West Embankment. This material was assessed during the design of the EL 5280 ft lift to be potentially susceptible to liquefaction (URS, 2012); therefore, a liquefaction loading analysis was performed for Section X.

The water level in the LAI for the liquefaction loading analysis was set to 5,275.2 ft to match the long-term, maximum storage pool. For the liquefaction loading condition, effective shear strength parameters were used for free-draining soils (bottom ash, fly ash, and compacted clay), total shear strength parameters were used for low-permeability soils (existing fly ash and weathered shale), and residual strength parameters were used for the potentially liquefiable soils (saturated sensitive fines). These material strength parameters are summarized in Table 5-1.

### 5.7 Safety Factor Assessment Results

Table 5-2 summarizes the results of the safety factor analysis for the LAI, 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

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Required Safety Factor[1]</th>
<th>Calculated Safety Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Section A (West Embankment)</td>
</tr>
<tr>
<td>Long-term, maximum storage pool</td>
<td>1.50</td>
<td>2.17</td>
</tr>
<tr>
<td>Maximum surcharge pool</td>
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<td>Seismic</td>
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<td>Liquefaction</td>
<td>1.20</td>
<td>-</td>
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Note: (1) From 40 CFR § 257.73(e)(1)(i) through (iv) (EPA, 2015)

The calculated factors of safety for the three critical cross sections along the LAI Perimeter Embankment exceeded the required minimum values for the long-term, maximum storage pool; the maximum surcharge pool; the seismic (pseudo-static); and liquefaction loading conditions.
6 Conclusions

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

- The LAI is classified as a Significant Hazard Potential CCR surface impoundment.
- The LAI embankment is founded on stable foundations and abutments. Seepage is managed by a single HDPE liner across the impoundment, which is underlain by a clay blanket on the upstream slopes extending to the crest of the embankment.
- The embankment has a single HDPE liner with clay on the upstream slope to prevent erosion. The downstream slopes are constructed with bottom ash and are not vegetated. The granular nature of bottom ash generally allows infiltration in preference to runoff and erosion. APS has a regular program of inspection and repair of erosion rills.
- Based on the available information and quality control test results, the LAI embankment was mechanically compacted to a density sufficient to withstand the range of loading conditions anticipated at the site.
- The LAI is capable of adequately managing the flow during and following the peak discharge from the PMF event. Flows resulting from the PMF event are discharged via a riser that discharges into the adjacent LDWP, which has been designed, constructed, operated, and maintained with sufficient storage volume above the maximum storage pool water level to store the PMF, and maintain at least two ft of freeboard.
- Factors of safety greater than the minimum values required by the CCR Rule were calculated for three cross sections along the LAI embankment for loading conditions associated with the maximum storage pool water level, maximum surcharge pool water level, design level seismic event, and liquefaction of the saturated sensitive fines in the North Toe area.
- Based on review of available records concerning the LAI and the results of the stability analyses, no deficiencies were noted that would affect the structural condition of the dam.
7 Limitations

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

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


APS, 2008, “Specifications for the 5258 Lift of the Lined Ash Impoundment at the Four Corners Power Plant Units 1-3 – San Juan County, New Mexico.”


APS, 2010, Lined Ash Impoundment Pond Drawings No. 156868 (5258 Lift As-Built), November 8.


NMOSE, San Juan Basin Regional Water Plan. http://www.ose.state.nm.us/Planning/RWP/Regions/02_SanJuan/2003/10-04-03Section01ExecutiveSummary.pdf.


URS, 2011, “Specifications for the 5280 Lift of Lined Ash Impoundment at the Four Corners Power Plant Units 1-3 – San Juan County, New Mexico,” October.


United States Environmental Protection Agency (EPA), 2015, 40 CFR Parts 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), 2013, 7.5-Minute Series The Hogback North, New Mexico Quadrangle Map.

Western Technologies, 2003, "Lined Pond Project: Method specification Compacted Bottom Ash Test Fill Section, Four Corners Power Plant."
Figures
EXISTING ASH POND #6

NORTH TOE BUTTRESS

LINED ASH IMPOUNDMENT (LAI)

LINED DECANT WATER POND (LDWP)
SITE TOPOGRAPHIC MAP

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

FIGURE 3-1
### LAI (Basin I) - Water Storage

<table>
<thead>
<tr>
<th>Reservoir Elevation (ft)</th>
<th>Surface Area (sf)</th>
<th>Total Surface Area (acre)</th>
<th>Average Surface Area (acre)</th>
<th>Elevation Difference (ft)</th>
<th>Reservoir Storage (acre-ft)</th>
<th>Cumulative Storage (acre-ft)</th>
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<td>86.90</td>
<td>401.33</td>
</tr>
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*From LAI Engineering Design Report (URS 2012)*
**Reservoir Elevation** | **Surface Area** | **Total Surface Area** | **Elevation Difference** | **Cumulative Storage**
--- | --- | --- | --- | ---
5208 | 87,120 | 2.00 | 0.00 | 2.00
5228 | 3,092,760 | 71.00 | 20.00 | 969.00
5248 | 3,571,920 | 82.00 | 20.00 | 2,530.00
5258 | 3,746,160 | 86.00 | 10.00 | 3,456.00
5270 | 5,418,090 | 124.38 | 12.00 | 4,718.29
5280 | 5,626,367 | 129.16 | 10.00 | 5,986.02

---

*From LAI Engineering Design Report (URS 2012)*
FIGURE 5-1

CROSS SECTION LOCATIONS
SAFETY FACTOR ASSESSMENT

FOUR CORNERS POWER PLANT
STRUCTURAL INTEGRITY REPORT
ARIZONA PUBLIC SERVICE
Project No. 60445844
Appendix A.
Historic Drawings
5248 LIFT AS-BUILT DRAWINGS

(APS, 2008)
Notes:
1) Red lines represent approximate location of panel limits.
5258 LIFT AS-BUILT DRAWINGS

(APS, 2010)
5270 LIFT AS-BUILT DRAWINGS

(APS, 2012)
NOTES:
1. EMBREMENT SHOULD BE PLACED AROUND THE MANHOLE RISER FOR THE FULL
   HEIGHT OF THE MANHOLE.
2. EMBREMENT SHALL EXTEND A MINIMUM OF THREE AND A HALF (3.5) FEET
   FROM THE RISER OR TO THE TRENCH WALLS, WHICHEVER IS THE GREATER
   DISTANCE.
3. EMBREMENT AROUND MANHOLE RISER IS REQUIRED TO BE CLASS I OR II
   MATERIAL PER ASTM D2231, COMPACTED TO A MINIMUM OF 90% STANDARD
   PROCTOR DENSITY.
4. MANHOLES SHALL BE INSTALLED IN A DRY TRENCH WITH A STABLE
   FOUNDATION. THE FOUNDATION SHOULD CONSIST OF A MINIMUM OF 8" OF
   CLASS I MATERIAL COMPACTED TO A MINIMUM OF 90% STANDARD
   PROCTOR DENSITY.
5. WHEN VEHICLE LOADS ARE PRESENT, A CONCRETE CAP OR OTHER SUCH
   STRUCTURE DESIGNED TO WITHSTAND THESE LOADS SHOULD BE PLACED
   OVER THE MANHOLE SO THAT THE LOADS ARE TRANSMITTED INTO THE
   SURROUNDING SOIL AND NOT DIRECTLY INTO THE RISER.
6. THE FOLLOWING PARAMETERS WERE ASSUMED:
   a) MAXIMUM SOIL DENSITY OF 120 LB/FT³
   b) GROUND WATER NOT TO EXCEED TOP OF THE MANHOLE. FLOATATION
      OF MANHOLE MAY NEED TO BE ADDRESSED. WHEN A POLYETHYLENE ANCHOR
      CONNECTION RING IS INCLUDED, IT MUST BE USED IN CONJUNCTION WITH A
      CONCRETE ANCHOR BY OTHERS. THE PE ANCHOR CONNECTION RING IS NOT
      DESIGNED TO RESTRAIN THE STRUCTURE BY ITSELF.
   c) AMBIENT (73°F) OPERATING TEMPERATURE.
   d) STRUCTURAL LOADS APPLIED TO HOPE MANHOLE NOT TO EXCEED 1000 LB.
   LOAD TO BE EQUALLY DISTRIBUTED ABOUT CIRCUMFERENCE OF MANHOLE.
7. PLACE LIFTING LUGS PER TOBOKA, FOR PRODUCTION USE ONLY.
8. CONTRACTOR TO VERIFY ALL DIMENSIONS AND MANHOLE DESIGN.
9. STANDARD BELL WITH THE EXCEPTION OF A 2.5" BOTTOM THICKNESS.
10. ED) 1/8" PERFORATIONS WILL BE DRILLED INTO THE RISER.
    PERFORATIONS WILL BE ANGLED 30 DEG DOWN TO THE SPOGOT END. THE
    HOLES WILL BE ON 11 INCH CENTERS (11" X 11") WITH 23 HOLES PER ROW.
    THE ROWS WILL BE 6 INCHES APART AND EVERY OTHER ROW WILL BE
    OFFSET BY 5-1/2 INCHES. THERE WILL BE A TOTAL OF 23 ROWS.
11. TWO SETS OF GRIP SUPPORTS WILL BE INCLUDED TO HOLD THE PIPE AS ROUND
    AS POSSIBLE FOR SHIPPING.

SAFETY NOTE TO PURCHASER
MANHOLE AND TANKS PRESENT CONFINED SPACE HAZARDS AND FALL HAZARDS.
IT IS THE RESPONSIBILITY OF THE OWNER/PURCHASER TO FOLLOW ALL APPLICABLE
OSHA CONFINED SPACE ENTRY PROCEDURES AND TO REQUIRE USE OF A FALL
PROTECTION DEVICE FOR ENTRY INTO ALL MANHOLES OR TANKS.
5280 LIFT AS-BUILT DRAWINGS

(URS, 2014)
**Bill of Materials**

<table>
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<th>UNIT MEAS.</th>
<th>PART #</th>
<th>DWG. #</th>
<th>SIZE</th>
<th>SD/CLASS</th>
<th>DESCRIPTION</th>
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<tr>
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<td>19</td>
<td>FT.</td>
<td>-</td>
<td>-</td>
<td>98&quot; I.D.</td>
<td>CLASS 315</td>
<td>HOPE - SW RISER STOCK (8 X 8)</td>
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<td>2</td>
<td>4</td>
<td>EA.</td>
<td>-</td>
<td>-</td>
<td>4&quot; THK</td>
<td></td>
<td>LIFTING LUGS (SEE LIFTING LUG DETAIL)</td>
</tr>
</tbody>
</table>

**Notes:**

1. Embedment should be placed around the manhole riser for the full height of the manhole.
2. Embedment shall extend a minimum of three and a half (3.5) feet from the riser or to the trench wall, whichever is the greater distance.
3. Embedment around manhole riser is required to be Class I or II material per ASTM D2331, compacted to a minimum of 95% standard proctor density.
4. Manholes shall be installed in a dry trench with a stable foundation. The foundation should consist of a minimum of 8" of Class I material compacted to a minimum of 65% standard proctor density.
5. When vehicle loads are present, a concrete cap or other such structure designed to withstand these loads should be placed over the manhole so that the loads are transmitted into the surrounding soil and not directly into the riser.
6. The following parameters were assumed:
   a. Maximum soil density of 120 lbs/ft³.
   b. Ground water not to exceed top of the manhole. Floatation of manhole may need to be addressed when a polyethylene anchor connection ring is included. It must be used in conjunction with a concrete anchor or others. The PE anchor connection ring is not designed to resist the structure by itself.
   c. Ambient (75°F) operating temperature.
   d. Structural loads applied to HIPE manhole not to exceed 1000 lbs.
7. Load to be equally distributed about circumference of manhole.
8. Place lifting lugs per 708864A. (For production use only)
9. Contractor to verify all dimensions and manhole design.
10. Standard bell with the exception of a 2.5" to thickness.
11. SWP 1/2" perforations will be drilled into the riser. The holes will be on 11-inch centers with 27 holes per row. The rows will be 6" apart and every other row will be offset by 5-1/2".
12. Two sets of wood supports will be included to hold the pipe as round as possible for shipping.

**Safety Note to Owner/Purchaser:**

Manholes and tanks present confined space hazards and fall hazards. It is the responsibility of the owner/purchaser of the PLESCO manhole or tank to train, equip, and require all entrants to follow applicable OSHA confined space entry procedures and to require use of a fall protection device for entry into all manholes or tanks.

**Notes:**

1. All dimensions are in inches.
2. Fractional tolerance ± 2".
3. Angular tolerance ± 2°.

---

**Reference Drawings**

- No reference drawings provided.

**Industrial Pipe Fittings**

- No specific fittings mentioned.

---

**URS Corporation**

- Address: 3000 River Place, Suite 1000, Denver, CO 80209
- Phone: (303) 832-4000

**URS Corporation**

- Address: 3000 River Place, Suite 1000, Denver, CO 80209
- Phone: (303) 832-4000

---

**WCWRK Safely Today**

- Logo: APS Energy
- Additional notes about safety and compliance with applicable regulations.
### Bill of Materials

<table>
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<th>PART</th>
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<th>D/W.</th>
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<td>-</td>
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<td>-</td>
<td>-</td>
<td>2&quot;</td>
<td>THK</td>
<td>-</td>
<td>-</td>
<td>LIFTING LUGS (SEE LIFTING LUG DETAIL)</td>
</tr>
</tbody>
</table>

**Notes:**

1. Chairman should be placed around the manhole riser for the full height of the manhole.
2. Embankment shall extend a minimum of three and a half (3.5) feet from the riser or to the trench wall, whichever is the greater distance.
3. Embankment around manhole riser required to be Class I or II material per ASTM D2337, compacted to a minimum of 95% standard proctor density.
4. Manholes shall be installed in a dry trench with a stable foundation. The foundation should consist of a minimum of 6" of Class I material, compacted to a minimum of 95% standard proctor density.
5. When vehicle loads are present, a concrete cap or other such structure designed to withstand these loads should be placed over the manhole so that the loads are transmitted into the surrounding soil and not directly into the riser.
6. The following parameters were assigned:
   - Maximum soil density of 120 LBS/FT.
   - Ground water not to exceed top of the manhole. Floation of manhole may need to be addressed. When a polyethylene anchor connection ring is included, it must be used in conjunction with a concrete anchor by others. The PE anchor connection ring is not designed to restrain the structure by itself.
   - Ambient (75°F) operating temperature.
   - Structural loads applied to manhole not to exceed 1000 LBS.
   - Load to be equally distributed about circumference of manhole.
   - Place lifting lug per 709664. (FOR PRODUCTION USE ONLY)
   - Contractor to verify all dimensions and manhole design.
   - Standard bell, with the exception of a 2 1/2" wall thickness.
   - (40S) 1/2" wall perforations will be drilled into the riser. The holes will be on 11"-inch centers (of 12) with 27 holes per row. The rows will be 6" apart and every other row will be offset by 5-1/2".
   - Two sets of wood supports will be included to hold the pipe as round as possible for shipping.

**Safety Note To Owner/Purchaser:**

Manholes and tanks present confined space hazards and fall hazards.

It is the responsibility of the owner/purchaser of the filled manhole or tank to train, equip, and require all contractors to follow applicable.

OSHA confined space entry procedures and to require use of a fall protection device for entry into all manholes or tanks.

1. All dimensions are in inches.
2. Fractional tolerance ± 2
3. Angular tolerance ± 2°

---

**Industrial Pipe Fittings**

~ Reference Datasheet ~

**MH 916110**

**Design & Manufacturing**

Arizona Public Service Co.

**Work Safely Today**

This drawing or document is the property of the undersigned and shall not be used or reproduced in any form or manner without written consent of the owner/holder. Copying, reproduction, or infringement is a violation of federal law and will result in prosecution.
NOTES:
1. Embedment should be placed around the manhole riser for the full height of the manhole.
2. Embedment shall extend a minimum of three and a half (3.5) feet from the riser on to the trench wall; whichever is the greater distance.
3. Embedment around manhole riser is required to be Class I or II material, per ASTM D2321, compacted to a minimum of 95% standard proctor density.
4. Manholes shall be installed in a dry trench with a stable foundation. The foundation should consist of a minimum of 8" of Class I material, compacted to a minimum of 95% standard proctor density.
5. When vehicle loads are present, a concrete cap or other such structure designed to withstand these loads should be placed over the manhole so that the loads are transmitted into the surrounding soil and not directly into the riser.
6. The following parameters were assumed:
   a) Maximum soil density of 120 lbs/ft³.
   b) Ground water not to exceed top of the manhole. Flooding of manholes may need to be addressed when a polyethylene anchor connection ring is included. It must be used in conjunction with a concrete anchor by others. The PE anchor connection ring is not designed to restrain the structure by itself.
   c) Ambient (73°F) operating temperature.
   d) Structural loads applied to PE manholes not to exceed 1000 lbs.
   e) Load to be equally distributed about circumference of manhole.
7. Place lifting lugs per 708466. (For production use only)
8. Contractor to verify all dimensions and manhole design.
9. Standard bell with the exception of a 2.5" to thickness.
10. (0.1") 1/2" perforations will be drilled into the riser. Perforations will be angled 20° down to the spigot end. The holes will be on 11 inch centers (of 10) with 27 holes per row. The rows will be 6 inches apart and every other row will be offset by 5-1/2 inches. There will be a total of 23 rows.
11. Two sets of wood supports will be included to hold the pipe as round as possible for shipping.

SAFETY NOTE TO OWNER/PURCHASER

Manholes and tanks present confined space hazards and fall hazards. It is the responsibility of the owner/purchaser of the flooded manhole or tank to train, equip, and require all entrants to follow applicable OSHA confined space entry procedures and to require use of a fall protection device for entry into all manholes or tanks.
Appendix B.
Safety Factor Calculation
# IE QMS Check and Review Record

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<tr>
<td>PM Name</td>
<td>Frances Ackerman, R.G., P.E.</td>
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<tr>
<td>PIC Name</td>
<td>Alexander Gourlay, P.E.</td>
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<td>Constructability Review</td>
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<th>Jed Stoken, P.E.</th>
<th>Comments Required by:</th>
<th>Lee Wright, P.E.</th>
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<td>Lee Wright, P.E.</td>
<td>Title of Work Product:</td>
<td>Lined Ash Impoundment Safety Factor Assessment</td>
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## Review Scope

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

### Project Manager (or Designee) Signature

8/12/16

### Comments

- □ ☑ Checker / Reviewer has no comments.

#### 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**

7/14/16

---

(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**

7/14/16
## IE QMS Check and Review Record

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

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<th>Approval</th>
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**DISTRIBUTION**
- Project Central File – Quality File Folder
- Other – Specify:

---

**QMS Form 3-4 (MM)**
**Date:** September 2014
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3 Analysis Inputs ............................................................................................................................... 2
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Tables

Table 1 Material Properties Used for the Safety Factor Assessment
Table 2 Safety Factor Results
1 INTRODUCTION

The purpose of this calculation is to perform limit equilibrium slope stability analyses to assess the stability of the existing Coal Combustion Residual (CCR) surface impoundment embankments at Arizona Public Service (APS)’s Four Corner Power Plant in Fruitland, New Mexico. Specifically, the CCR surface impoundment embankments that will be evaluated are associated with the Lined Ash Impoundment (LAI).

2 ANALYSIS CRITERIA

The analyses were performed to meet the regulations set forth in the United States Environmental Protection Agency’s (EPA) 40 CFR Part 257.73(e) Structural Integrity Criteria for existing CCR surface impoundments (the Rule) (EPA 2015). The Rule requires safety factor assessments be performed for units containing coal combustion residuals and the resulting safety factors for various embankment loading and tailwater conditions must meet the values outlined in the Rule. For the LAI, 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:

- The geometry for the cross-sections was based on the as-built drawing set of the LAI 5280 lift (current embankment configuration; APS Drawing Number 161907), for Sections A (West Embankment), M (South Embankment), and X (North Toe Buttress) presented in the 5280 Lift Design Report (URS, 2012).
- The subsurface stratigraphy was replicated from stability model cross-sections developed as part of the design calculations in the 5280 Lift Design Report (URS, 2012).
The safety factor calculations were performed using the software program SLOPE/W, commercially available through GEO-SLOPE International, Ltd. (GEO-SLOPE International, 2012).

Material properties used in the safety factor assessment were based on previously reported material properties developed for the 5280 Lift Design Report (URS, 2012).

Pore pressure distribution within the embankment was developed from an interpretation of water level readings using piezometers installed on and near the LAI embankment. Water level measurements are presented in the 2015 Annual CCR Impoundment and Landfill Inspection Report (APS and AECOM, 2016).

The maximum operational water level at the southwest corner of the LAI is 5,275.2 feet, as presented in the 5280 Lift Design Report (URS, 2012).

The maximum surcharge water level accounts for containment of the PMF on top of the maximum operational water level in the LAI. The maximum surcharge water level is 5,277.2 feet as presented in the 5280 Lift Design Report (URS, 2012).

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).

4 ASSUMPTIONS

Assumptions used in this calculation package include:

- The embankment geometry and subsurface conditions have not substantially changed from the 5280 lift design.
- Phreatic levels measured in and around the embankment are typical of the conditions present in the analysis cases considered.
- The residual strength ratio ($S_r/\sigma'_{vo}$) is applicable to the saturated sensitive fines beneath the North Toe Buttress in the Liquefaction Loading analysis.
5 SAFETY FACTOR CALCULATIONS

The safety factor assessments were performed for three cross-sections along the LAI embankment. The 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

Factors of safety were calculated for critical cross-sections of the LAI West and South Embankments. Each of these cross-sections was developed and analyzed as part of the 5280 Lift Design. No revisions have been made to the previously defined stratigraphic conditions. The critical cross-section is the cross-section that is anticipated to be most susceptible to structural failure for a given loading condition. The critical cross-section thus represents a “most-severe” case. Section locations were selected based on variation in the embankment height and stratigraphic conditions to represent the most severe case.

Section A – West Embankment: The maximum section of the LAI West Embankment was modeled from Section A of the 5280 Lift Construction Drawings (URS, 2012). The West Embankment has an approximately 3 horizontal to 1 vertical (3H:1V) downstream slope and a crest width of 30 feet. The lower third of the 5280 Lift consists of a combination of compacted bottom and fly ash, the upper two-thirds consists of compacted bottom ash, and there is a 15-foot thick compacted clay layer on the upstream side of the embankment. The embankment is founded partially on the pre-existing Ash Pond 3 and 4 Divider Dike and partially on old hydraulically placed fly ash associated with Ash Pond 3.

Section M – South Embankment: The South Embankment of the LAI was modeled from the Section M of the Construction Drawings (URS, 2012). The embankment is founded on native weathered shale. The pre-existing Ash Pond 4 Embankment was used as a starter dam for building the South Embankment using a downstream construction method. The pre-existing Ash Pond 4 Embankment consists of compacted bottom ash with a 40-foot wide layer of compacted clay on the upstream face. The subsequent raises of the LAI impoundment consist of compacted bottom ash with a 15-foot wide blanket of compacted clay on the upstream
The clay blanket on the upstream face of the LAI embankment is keyed into the clay of the existing (original) embankment.

The South Embankment has an approximately 2H:1V downstream slope and a crest width of 30 feet. A buried toe drain runs parallel with the embankment near the downstream toe. The drain consists of two, 10-inch inside diameter, perforated pipes surrounded by a two-stage sand and gravel filter within a drainage channel. The channel is 12-foot deep with a 13-foot wide bottom and 1.5H:1V to 1H:1V side slopes.

**Section X – North Toe Buttress:** The North Toe Buttress area of the LAI West Embankment was modeled from Section X-X of the Construction Drawings (URS, 2012). The North Toe Buttress was constructed against the base of the West Embankment to a height of 20 feet above the elevation at the toe of the West Embankment, and is inclined downward toward the west at approximately 20H:1V. The North Toe Buttress extends to the west approximately 400 feet. The buttress consists of a 1.5-foot thick bottom ash drain layer overlain by a combination of compacted bottom and fly ash to the approximate elevation of 5,227 feet at its highest point.
5.2 Material Properties

Material properties used in the safety factor assessment were based on previously reported material properties developed for the LAI 5280 Lift Design Report (URS, 2012), except as discussed in the following paragraphs. Table 1 presents these values.

Table 1 – Material Properties Used for the Safety Factor Assessment

<table>
<thead>
<tr>
<th>Material</th>
<th>Moist Unit Weight, $\gamma_m$ (pcf)</th>
<th>Saturated Unit Weight, $\gamma_{sat}$ (pcf)</th>
<th>Drained Strength Cohesion, $c'$ (psf)</th>
<th>Friction Angle, $\phi'$ (degrees)</th>
<th>Undrained Strength Cohesion, $c$ (psf)</th>
<th>Friction Angle, $\phi$ (degrees)</th>
<th>Residual Strength Shear Strength Ratio ($S_r/\sigma_v'$)</th>
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</thead>
<tbody>
<tr>
<td>Compacted Bottom and Fly Ash</td>
<td>90</td>
<td>90</td>
<td>0</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Existing Fly Ash (Top)</td>
<td>90</td>
<td>90</td>
<td>0</td>
<td>30</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Existing Fly Ash (Bottom)</td>
<td>90</td>
<td>90</td>
<td>0</td>
<td>28</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>New Fly Ash (Impounded)</td>
<td>90</td>
<td>90</td>
<td>-</td>
<td>-</td>
<td>304</td>
<td>0</td>
<td>-</td>
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<tr>
<td>Compacted Bottom Ash</td>
<td>75.1</td>
<td>75.1</td>
<td>0</td>
<td>42</td>
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<td>-</td>
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<tr>
<td>Compacted Clay</td>
<td>125</td>
<td>130</td>
<td>300</td>
<td>20</td>
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<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Weathered Shale (Native Ground)</td>
<td>120</td>
<td>125</td>
<td>-</td>
<td>-</td>
<td>500</td>
<td>30</td>
<td>-</td>
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<tr>
<td>Sensitive Fines (Drained)</td>
<td>80</td>
<td>-</td>
<td>0</td>
<td>18.5</td>
<td>-</td>
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<td>-</td>
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<tr>
<td>Sensitive Fines (Liquefaction)</td>
<td>80</td>
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<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>0.05</td>
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<tr>
<td>Buttress</td>
<td>75</td>
<td>-</td>
<td>0</td>
<td>20</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Drain Sand</td>
<td>110</td>
<td>-</td>
<td>0</td>
<td>30</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Based on the results of the liquefaction assessment performed as part of the 5280 Lift design, the saturated fly ash (sensitive fines) layer was found to be potentially liquefiable (URS, 2012). For this calculation, both post-cyclic shear strength ratios for the sensitive fines encountered beneath the North Toe Buttress were estimated using empirical correlations to CPT data. The post-cyclic, residual, shear strength was estimated using the Idriss and Boulanger (2008) method based on either SPT or CPT data. The current stability analysis includes developing residual shear strength ratios for the sensitive fines from CPT data to estimate the liquefaction loading safety factor for Section X.

The residual undrained shear strength ratio presented in Table 1 was calculated using the Idriss and Boulanger (2008) method based on the CPT results. The residual strength was estimated as follows:

**CPT Locations**

Eight CPT soundings were used to estimate the residual strength of the sensitive fines beneath the NTB. A general subsurface profile of the soundings consisted of fly ash over a shale bedrock. The eight CPT soundings, as presented in Figure 1, were CPT-8, CPT-9, CPT-10, CPT-11, CPT-12, CPT-13, CPT-16, and CPT-17.

![Figure 1 – URS 2011 Geotechnical Exploration CPT Sounding Locations](image-url)
The groundwater level below the NTB was assumed to be at the top of the liquefaction-susceptible sensitive fines layer. This is a conservative assumption that will produce lower bound residual strength estimates.

Idriss and Boulanger (2008) Method

In accordance with the Idriss and Boulanger (2008) method, the residual strength of the sensitive fines without void redistribution effects (expected for the site) is defined by the following equation, which is presented graphically in Figure 2:

\[
\frac{S_r}{\sigma'_vo} = \exp \left( \frac{q_{c1Ncs-Sr}}{24.5} - \left( \frac{q_{c1Ncs-Sr}}{61.7} \right)^2 + \left( \frac{q_{c1Ncs-Sr}}{106} \right)^3 - 4.42 \right) 
\times \left( 1 + \exp \left( \frac{q_{c1Ncs-Sr}}{11.1} - 9.82 \right) \right) \leq \tan \phi'
\]

where:
- \( S_r \) = Residual strength of the liquefied material
- \( \sigma'_vo \) = Initial effective overburden stress
- \( q_{c1Ncs-Sr} \) = Equivalent clean sand CPT normalized corrected tip resistance
- \( \phi' \) = Effective friction angle

Figure 2 – Correlation between the normalized residual shear strength ratio for liquefied soils and overburden-corrected CPT penetration resistance from Figure 90 of Idriss and Boulanger (2008)
The equivalent clean sand CPT normalized corrected tip resistance, \( q_{C1Ncs-Sr} \), is defined as the normalized corrected tip resistance corrected for fines content and may be estimated using the following equation from Idriss and Boulanger (2008):

\[
q_{C1Ncs-Sr} = q_{C1N} + \Delta q_{C1N-Sr}
\]

where:
- \( q_{C1Ncs-Sr} \) = Equivalent clean sand CPT normalized corrected tip resistance
- \( q_{C1N} \) = CPT normalized corrected tip resistance
- \( \Delta q_{C1N-Sr} \) = Equivalent clean sand adjustment values

The equivalent clean sand adjustment value can be determined using the table shown in Figure 3, which correlates the adjustment value to the fines content.

![Figure 3](image)

**Figure 3 – Approximate values of \( \Delta q_{C1N-Sr} \) for CPT correlation with residual strengths from Table 5 of Idriss and Boulanger (2008)**

An effective friction angle, \( \phi' \), equal to 18.5 degrees was used in the analysis based on the estimated value developed for the fly ash material as part of the 5280 Lift Design (URS, 2012). This limits the residual strength ratio, \( S_r/\sigma_{vo}' \), to a maximum value of the following:

\[
\left( \frac{S_r}{\sigma_{vo}'} \right)_{\text{max}} = \tan \phi' = \tan(18.5^\circ) = 0.335
\]

Where:
- \( (S_r/\sigma_{vo}')_{\text{max}} \) = Maximum residual strength ratio = 0.335
- \( \phi' \) = Effective friction angle = 18.5°
Residual Strength Analysis Results

The result of the CPT-based residual strength analysis is presented in Figure 4 below. Figure 4 presents the residual strength ratio calculated using the measured CPT tip resistance in the liquefaction-susceptible sensitive fines material based on the elevation where the measurement was recorded. The geometric mean value of the calculated residual strength ratio is $S_r/\sigma'_vo = 0.058$, which is rounded to 0.05 for the analysis as shown in the Figure below. This value is presented in Table 1 to characterize the strength of the sensitive fines beneath the NTB in stability analyses after liquefaction of the material.

Figure 4 – Residual Strength Ratio Result for the Liquefaction-Susceptible Sensitive Fines Beneath the NTB
DESIGN CALCULATION

<table>
<thead>
<tr>
<th>Calculation Title:</th>
<th>CCR Unit:</th>
<th>Project No:</th>
<th>Date:</th>
<th>Page No:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Factor of Safety Assessment</td>
<td>Lined Ash Impoundment Safety Factor Assessment</td>
<td>60445844</td>
<td>7/13/16</td>
<td>Page 11 of 20</td>
</tr>
</tbody>
</table>

5.3 Embankment Pore Pressure Distribution

There are a total of 53 piezometers installed in and below the LAI and NTB. Water levels in these piezometers are monitored on a monthly basis and the maximum and minimum water levels in each are reported annually (APS and AECOM, 2016). These data were considered to be the most reliable indicators of pore pressure distribution in and below the embankment. The pore pressure distribution in each section was evaluated using water level measurements obtained from the piezometers.

Piezometers P-7, P-8, P-10, and P-11 were used to evaluate the porewater pressure conditions in the embankment in the vicinity of Sections A and M. No positive porewater pressures have been recorded in P-7, P-8 or P-10 between 2008 and 2016, although positive porewater pressures have been recorded in the existing fly ash in P-11 over the same time period. Although this condition has only been recorded in one of the three nested piezometers at the P-11 location, the recorded positive porewater pressure was conservatively included in the stability models for Sections A and M.

Piezometers P-100 through P-111 where used to evaluate the porewater pressure conditions in the vicinity of the North Toe Buttress (Section X). Positive porewater pressures were recorded within the saturated fly ash below the North Toe Buttress and the recorded positive porewater pressures were included in the stability models for Section X.

The regional groundwater level in the vicinity of the LAI embankment was based on an AECOM 2016 Hydrogeologic assessment of the entire Four Corners Power Plant. Regional water levels below the embankment ranged from approximate 5,145 feet beneath the South Embankment section to 5,160 feet beneath the West Embankment section.

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 four loading conditions are described in the following subsections.

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 long-term, maximum storage pool water elevation used in the calculation was based on the maximum operating level developed as part of the LAI 5280 Lift design analysis (URS, 2012). Factors of safety were calculated using shear strengths expressed as effective stress, except the New Fly Ash was modeled using a total (undrained) shear strength based on the results of CPT testing and engineering judgement (URS, 2012).

For this analysis, the long-term maximum storage pool elevation at the southwest corner of the LAI was 5,275.2 feet (URS, 2012).

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 water level used in the calculation was based on the estimated water level associated with the PMF on top of the maximum operating level developed as part of the LAI 5280 Lift analysis (URS, 2012).

For this analysis, the maximum surcharge pool at the southwest corner of the LAI was 5,277.2 feet (URS, 2012).
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 for Sections A and M. A post-cyclic analysis was used to represent the seismic loading condition for Section X. The post-cyclic analysis was performed due to the presence of potentially liquefiable sensitive fines in the foundation of the North Toe Buttress.

The peak horizontal bedrock acceleration for a Site Class “B” rock, based on the United States Geological Survey (USGS) National Seismic Hazard Map, with a 2% probability of exceedance in 50 years, is 0.05895g, as presented in Attachment A (USGS, 2008). A Site Classification of D “Stiff Soil” was assigned to the West Embankment and North Toe Buttress sections (Section A and Section X) based on the average properties in the top 100 feet, which consists of approximately 40 feet of fly ash above native weathered shale. A Site Classification of C “Very Dense Soil and Soft Rock” was assigned to the South Embankment section (Section M), which is founded on natural ground primarily consisting of weathered shale. Site Class definitions are summarized in Table 20.3-1 from ASCE 7-10 (ASCE 2013) and shown in Figure 5.

![Figure 5 – Table 20.3-1 Site Classification from ASCE 7-10 (2013)](image-url)
The peak ground acceleration at the ground surface for site class C and D at the crest is calculated using the following procedure:

\[ PGA_{Ground\ Surface, M} = F_{PGA}(PGA) \]

\[ PGA_{ground\ Surface, C} = 1.2(0.05895g) \quad PGA_{Ground\ Surface, D} = 1.6(0.05895g) \]

\[ PGA_{Ground\ Surface, C} = 0.071g \quad PGA_{Ground\ Surface, D} = 0.094g \]

Where:
- \( PGA_{ground\ Surface, M} \) = 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 International Code Council’s 2015 International Building Code (IBC, 2015) as shown in Figure 6.

![Figure 6 – Table 1613.3.3(1) from the IBC (2015)](image)

The PGA at the ground surface for Site Classes C and D (PGA\_{Ground\ Surface}) were then used to estimate the peak transverse acceleration at the crest of the embankment, \( PGA_{C, crest} = 0.243g \) and \( PGA_{D, crest} = 0.307g \) as shown on Figure 7 and based on variations in recorded peak crest accelerations versus those recorded at the base of earth and rock fill dams recorded values for Loma Prieta and other earthquakes by Holzer (USGS, 1998).
Figure 7 – Variations of Peak Transverse Crest Acceleration v. Peak Transverse Base Acceleration Based on Holzer (1998)

Makdisi and Seed (1977) note that the “maximum acceleration ratio” varies with the depth of the sliding mass relative to the embankment height. Figure 8 presents the relationship between maximum acceleration ratio \( \frac{k_{\text{max}}}{u_{\text{max}}} \) and depth of sliding mass \( \frac{y}{h} \). For deep-seated failure surfaces that involve the entire vertical profile of the embankment 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:
Therefore:

\[
\frac{k_{\text{max}}}{u_{\text{max}}} = 0.34
\]

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

\[
k_{\text{max}} = 0.34(u_{\text{max}})
\]

\[
k_{\text{max,C}} = 0.34(0.243g) \quad k_{\text{max,D}} = 0.34(0.307g)
\]

\[
k_{\text{max,C}} = 0.083g \quad k_{\text{max,D}} = 0.104g
\]

The pseudostatic analyses incorporated a horizontal seismic coefficient of 0.083g for Section M  \\
and a horizontal seismic coefficient of 0.104 for Sections A and X.

The water level in the LAI for the seismic loading analysis was set to EL 5,275.2 feet to match the long-term, maximum storage pool. Drained shear strengths expressed as effective cohesion and friction angles, as summarized in Table 1, were used to define the strengths for free-
draining soils (bottom ash) and the sensitive fines beneath the Section X toe buttress. The sensitive fines are potentially susceptible to liquefaction during the seismic event; the stability of the embankment under the reduced strength conditions caused by liquefaction of the sensitive fines is evaluated in the liquefaction loading case. Undrained shear strengths expressed as total cohesion and friction angles, as 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**

A liquefaction triggering analysis was performed for the North Toe Buttress (Section X), as part of the 5280 Lift Design (URS, 2012). The CPT-based empirical liquefaction analysis indicated that lenses of the saturated fly ash with the potential to liquefy during the design earthquake are present in CPT soundings 8 through 13, 15, 16, and 17. The liquefiable materials were generally only thin lenses of materials identified by the Soil Behavior Type as sensitive fines (“clay-like” behavior classification).

The liquefaction triggering analysis results were based on the use of a relatively conservative $K_\alpha$ correction factor of 0.2 versus the recommended value of 0.9. The empirical analyses indicate significantly fewer and thinner lenses of liquefiable materials when the recommended $K_\alpha$ correction factor of 0.9 is used (URS, 2012). However, these empirical methods are based on natural geo-materials and not man-made materials like fly ash. A fact that is further supported by the discrepancy of the soil behavior classification of “clay-like” assigned by the CPT soundings versus the index property testing and general knowledge of the fly ash material that indicates that fly ash is a non-cohesive fine grained material. Therefore, a laboratory liquefaction triggering evaluation was performed using cyclic Direct Simple Shear (DSS) strength testing (URS, 2012).

The results of two cyclic DSS tests on pluviated samples indicated that under the maximum cyclic stress ratio (CSR) calculated in the empirical liquefaction analyses, the saturated fly ash in the North Toe Buttress area did not undergo liquefaction. However, two of the four specimens subjected to cyclic DSS tests exhibited contractive behavior, indicating they would be susceptible to liquefaction though they did not actually liquefy (URS, 2012).
Based on the results of the empirical analyses and the two cyclic DSS tests, the potential for liquefaction at the design earthquake magnitude is low but cannot unequivocally be stated that the saturated fly ash will not liquefy during the design earthquake for the project (URS, 2012). Consequently, the liquefaction loading condition was evaluated for Section X at the North Toe Buttress.

The water level in the LAI for the liquefaction loading analysis was set to EL 5,275.2 feet to match the long-term, maximum storage pool. The potentially liquefiable materials in the section were designated as the sensitive fines below the phreatic surface. Drained shear strengths expressed as effective cohesion and friction angles, as summarized in Table 1, were used to in the analysis, except for the potentially liquefiable sensitive fines which used a residual shear strength ratio of 0.05.

6 ANALYSIS RESULTS AND CONCLUSIONS

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

<table>
<thead>
<tr>
<th>Loading Condition</th>
<th>Required Factor of Safety</th>
<th>Calculated Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Section A West Embankment</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Section M South Embankment</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Section X North Toe Buttress</td>
</tr>
<tr>
<td>Long-term, maximum storage pool</td>
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</table>
7 REFERENCES


### 8 ATTACHMENTS

**ATTACHMENT A**  USGS Seismic Acceleration  
**ATTACHMENT B**  SLOPE/W Output Figures
ATTACHMENT A

USGS Seismic Acceleration
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

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 A (West Embankment)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
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.

Figure 1) Long-Term, Maximum Storage Pool
File Name: Section A.gsz
Date: 7/13/2016
Method: Spencer

Factor of Safety: 2.17

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight</th>
<th>Cohesion (psf)</th>
<th>Friction Angle (°)</th>
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<tr>
<td>Compacted Bottom Ash</td>
<td>75.1 pcf</td>
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<td>42</td>
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<tr>
<td>Existing Fly Ash (Top)</td>
<td>90 pcf</td>
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<td>20</td>
</tr>
<tr>
<td>Weathered Shale (Native Ground)</td>
<td>120 pcf</td>
<td>500</td>
<td>30</td>
</tr>
<tr>
<td>Existing Fly Ash (Bottom)</td>
<td>90 pcf</td>
<td>0</td>
<td>28</td>
</tr>
<tr>
<td>Compacted Bottom and Fly Ash</td>
<td>90 pcf</td>
<td>0</td>
<td>35</td>
</tr>
</tbody>
</table>

Maximum Storage Pool
Water Surface EL = 5275.2 feet

Distance (Feet)
Elevation (Feet)
Slope Stability Analysis
Section A (West Embankment)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
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.

Factor of Safety: 2.08

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight</th>
<th>Cohesion (psf)</th>
<th>Friction Angle (°)</th>
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<tbody>
<tr>
<td>Compacted Bottom Ash</td>
<td>75.1 pcf</td>
<td>0</td>
<td>42°</td>
</tr>
<tr>
<td>Existing Fly Ash (Top)</td>
<td>90 pcf</td>
<td>0</td>
<td>30°</td>
</tr>
<tr>
<td>New Fly Ash</td>
<td>90 pcf</td>
<td>304</td>
<td>0°</td>
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<tr>
<td>Compacted Clay</td>
<td>125 pcf</td>
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<td>20°</td>
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<td>28°</td>
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<tr>
<td>Compacted Bottom and Fly Ash</td>
<td>90 pcf</td>
<td>0</td>
<td>35°</td>
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</table>

Figure 2) Maximum Surcharge Pool
File Name: Section A.gsz
Date: 7/13/2016
Method: Spencer

Maximum Surcharge Pool
Water Surface EL = 5277.2 feet
Slope Stability Analysis
Section A (West Embankment)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
Arizona Public Service

Material | Unit Weight | Cohesion | Friction Angle
---|---|---|---
Compacted Bottom Ash | 75.1 pcf | 0 psf | 42 °
Existing Fly Ash (Top) | 90 pcf | 0 psf | 30 °
New Fly Ash | 90 pcf | 304 psf | 0 °
Compacted Clay | 125 pcf | 300 psf | 20 °
Weathered Shale (Native Ground) | 120 pcf | 500 psf | 30 °
Existing Fly Ash (Bottom) | 90 pcf | 0 psf | 28 °
Compacted Bottom and Fly Ash | 90 pcf | 0 psf | 35 °

Factor of Safety: 1.35

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.

Figure 3) Seismic Loading
File Name: Section A.gsz
Date: 7/13/2016
Method: Spencer

Horz Seismic Coef.: 0.104

Maximum Storage Pool
Water Surface EL = 5275.2 feet

Distance (Feet)  Elevation (Feet)
Slope Stability Analysis
Section M (South Embankment)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
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.

Figure 4) Long-Term, Maximum Storage Pool
File Name: Section M.gsz
Date: 7/13/2016
Method: Spencer

Factor of Safety: 1.55

<table>
<thead>
<tr>
<th>Material Type</th>
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<th>Cohesion:</th>
<th>Friction Angle:</th>
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<tbody>
<tr>
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<td>75.1 pcf</td>
<td>0 psf</td>
<td>42 °</td>
</tr>
<tr>
<td>Existing Fly Ash (Top)</td>
<td>90 pcf</td>
<td>0 psf</td>
<td>30 °</td>
</tr>
<tr>
<td>New Fly Ash</td>
<td>90 pcf</td>
<td>304 psf</td>
<td>0 °</td>
</tr>
<tr>
<td>Compacted Clay</td>
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<td>28 °</td>
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<tr>
<td>Compacted Bottom and Fly Ash</td>
<td>90 pcf</td>
<td>0 psf</td>
<td>35 °</td>
</tr>
<tr>
<td>Drain Sand</td>
<td>110 pcf</td>
<td>0 psf</td>
<td>30 °</td>
</tr>
</tbody>
</table>

Unit Weight:
- Compacted Bottom Ash: 75.1 pcf
- Existing Fly Ash (Top): 90 pcf
- New Fly Ash: 90 pcf
- Compacted Clay: 125 pcf
- Weathered Shale (Native Ground): 120 pcf
- Existing Fly Ash (Bottom): 90 pcf
- Compacted Bottom and Fly Ash: 90 pcf
- Drain Sand: 110 pcf

Cohesion:
- Compacted Bottom Ash: 0 psf
- Existing Fly Ash (Top): 0 psf
- New Fly Ash: 304 psf
- Compacted Clay: 300 psf
- Weathered Shale (Native Ground): 500 psf
- Existing Fly Ash (Bottom): 0 psf
- Compacted Bottom and Fly Ash: 0 psf
- Drain Sand: 0 psf

Friction Angle:
- Compacted Bottom Ash: 42 °
- Existing Fly Ash (Top): 30 °
- New Fly Ash: 0 °
- Compacted Clay: 20 °
- Weathered Shale (Native Ground): 30 °
- Existing Fly Ash (Bottom): 28 °
- Compacted Bottom and Fly Ash: 35 °
- Drain Sand: 30 °

Distance (Feet)
0 50 100 150 200 250 300 350 400 450 500 550 600 650 700 750 800 850 900 950 1,000

Elevation (Feet)
5,100 5,125 5,150 5,175 5,200 5,225 5,250 5,275 5,300

5,280 Feet Crest El.

Water Surface EL = 5275.2 feet

Maximum Storage Pool
New Fly Ash
Existing Fly Ash (Top)
Existing Fly Ash (Bottom)
Weathered Shale (Native Ground)
Drain Sand
Slope Stability Analysis
Section M (South Embankment)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
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.

Figure 5) Maximum Surcharge Pool
File Name: Section M.gsz
Date: 7/13/2016
Method: Spencer

Factor of Safety: 1.55

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit Weight</th>
<th>Cohesion</th>
<th>Friction Angle</th>
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<tbody>
<tr>
<td>Compacted Bottom Ash</td>
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<tr>
<td>Drain Sand</td>
<td>110 pcf</td>
<td>0 psf</td>
<td>30 °</td>
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</table>

Maximum Surcharge Pool
Water Surface EL = 5277.2 feet
Slope Stability Analysis
Section M (South Embankment)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
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.

Figure 6) Seismic Loading
File Name: Section M.gsz
Date: 7/13/2016
Method: Spencer

Factor of Safety: 1.27

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<tr>
<td>Drain Sand</td>
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</table>

Horz Seismic Coef.: 0.083

Maximum Storage Pool
Water Surface EL = 5275.2 feet
Slope Stability Analysis
Section X (North Toe Buttress)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
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.

Factor of Safety: 2.47

<table>
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<td>0 psf</td>
<td>35 °</td>
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<tr>
<td>Sensitive Fines (Drained)</td>
<td>80 pcf</td>
<td>0 psf</td>
<td>18.5 °</td>
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<tr>
<td>Buttress</td>
<td>75 pcf</td>
<td>0 psf</td>
<td>20 °</td>
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</tbody>
</table>
Slope Stability Analysis
Section X (North Toe Buttress)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
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.

Figure 8) Maximum Surcharge Pool
File Name: Section X - Static.gzs
Date: 7/12/2016
Method: Spencer

Factor of Safety: 2.41

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<tr>
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<td>18.5 °</td>
</tr>
<tr>
<td>Buttress</td>
<td>75 pcf</td>
<td>0 psf</td>
<td>20 °</td>
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</tbody>
</table>
Slope Stability Analysis
Section X (North Toe Buttress)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
Arizona Public Service

Note:
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Factor of Safety: 1.71

<table>
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<tr>
<td>Buttress</td>
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Horz Seismic Coef.: 0.104

Maximum Storage Pool
Water EL = 5275.2 feet

5280 Feet Crest El.
20 ft High Buttress/Surcharge
Slope Stability Analysis
Section X (North Toe Buttress)
Lined Ash Impoundment

Four Corners Power Plant
Fruitland, New Mexico
Arizona Public Service

Figure 10) Liquefaction Loading
File Name: Section X - Post-Seismic.gsz
Date: 7/12/2016
Method: Spencer

Factor of Safety: 1.90

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<tr>
<td>Compacted Bottom and Fly Ash</td>
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<tr>
<td>Sensitive Fines (Drained)</td>
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Note:
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