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October 12, 2016

Tennessee Valley Authority
1101 Market Street
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Initial Structural Stability Assessment
Peabody Ash Pond
EPA Final CCR Rule
TVA Paradise Fossil Plant
Drakesboro, Kentucky

1.0 PURPOSE

This letter documents AECOM's certification of the initial structural stability assessment for the TVA Paradise Fossil Plant's Peabody Ash Pond. Based on this assessment, the Peabody Ash Pond is in compliance with the structural stability requirements in the Final CCR Rule at 40 CFR 257.73(d).

2.0 INITIAL STRUCTURAL STABILITY ASSESSMENT

As described in 40 CFR 257.73(d), documentation is required on how the Peabody Ash Pond has been designed, constructed, operated, and maintained according to the structural stability requirements listed in the section. The combined capacity of all spillways must also be designed, constructed, operated, and maintained to adequately manage flow from the 1000-year storm event based upon a hazard potential classification of "significant."

3.0 SUMMARY OF FINDINGS

The attached report presents the initial structural stability assessment of the Peabody Ash Pond. The results show that the impoundment meets the structural stability requirements set forth in 40 CFR 257.73(d)(1)-(2).

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4.0 QUALIFIED PROFESSIONAL ENGINEER CERTIFICATION

I, Nicholas S Golden PE, being a Professional Engineer in good standing in the State of Kentucky, do hereby certify, to the best of my knowledge, information, and belief:

1. that the information contained in this certification is prepared in accordance with the accepted practice of engineering;
2. that the information contained herein is accurate as of the date of my signature below; and
3. that the initial structural stability assessment for the TVA Paradise Fossil Plant's Peabody Ash Pond meets the requirements specified in 40 CFR 257.73(d)(1)-(2).

SIGNATURE ______________________ DATE 10/12/16

ADDRESS: AECOM
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ATTACHMENTS: Initial Structural Stability Assessment (40 CFR §257.73(d)(1)) for Coal Combustion Residuals (CCR)

STATE OF KENTUCKY
NICHOLAS
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LICENSED
PROFESSIONAL ENGINEER

COAL COMBUSTION PRODUCT DISPOSAL PROGRAM

TENNESSEE VALLEY AUTHORITY - PEABODY ASH POND
TVA PARADISE FOSSIL PLANT
DRAKESBORO, KENTUCKY

INITIAL STRUCTURAL STABILITY ASSESSMENT (40 CFR §257.73(d)(1)) FOR COAL COMBUSTION RESIDUALS (CCR)

EXISTING SURFACE IMPOUNDMENT

Prepared for

TVA

Tennessee Valley Authority
1101 Market Street
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10/12/16

October 12, 2016

Prepared by

AECOM

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1.0 PROJECT BACKGROUND

On April 17, 2015 the "Disposal of Coal Combustion Residuals (CCR) from Electric Utilities" (EPA Final CCR Rule) was published in the Federal Register. AECOM has been contracted by the Tennessee Valley Authority (TVA) to analyze the Structural Stability of the Paradise Fossil Plant's CCR surface impoundments (SI) and evaluate compliance with §257.73 of the EPA Final CCR Rule.

As required by §257.73 of the EPA Final CCR Rule, an initial structural integrity evaluation is required by October 17, 2016 and must include an initial structural stability assessment for each existing CCR surface impoundment that meets the conditions of paragraph (b) as follows:

1. Has a height of five feet or more and a storage volume of 20 acre-feet or more; or
2. Has a height of 20 feet or more.

Peabody Ash Pond meets the first criterion. The location of Peabody Ash Pond is shown in **Figure 1**.

Figure 1: Site Location Plan

2.0 STRUCTURAL STABILITY ASSESSMENT - §257.73(d)(1)

40 CFR 257.73(d)(1). *Periodic structural stability assessments. (1) The owner or operator of the CCR unit must conduct initial and periodic structural stability assessments and document whether the design, construction, operation, and maintenance of the CCR unit is consistent with recognized and generally accepted good engineering practices for the maximum volume of CCR and CCR wastewater which can be impounded therein. The assessment must, at a minimum, document whether the CCR unit has been designed, constructed, operated, and maintained with:*

(i) Stable foundations and abutments;

(ii) Adequate slope protection to protect against surface erosion, wave action, and adverse effects of sudden drawdown;

(iii) Dikes mechanically compacted to a density sufficient to withstand the range of loading conditions in the CCR unit;

(iv) Vegetated slopes of dikes and surrounding areas, except for slopes which have an alternate form or forms of slope protection;

(v) 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.

(vi) 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 deterioration, deformation, distortion, bedding deficiencies, sedimentation, and debris which may negatively affect the operation of the hydraulic structure; and

(vii) For CCR units with downstream slopes 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.

2.1 FOUNDATIONS AND ABUTMENTS - §257.73(d)(1)(i)

The geology of Peabody Ash Pond originally included alluvial deposits underlain by Pennsylvanian age bedrock formations. Geologic mapping indicates that Peabody Ash Pond is primarily underlain by the Carbondale Formation. The Carbondale Formation generally consists of cyclic sequences of coal, sandstone, and shale. Surface mining in the area was conducted from 1960 to 1981. Subsequent surface mining operations in this area were also conducted within the area of Peabody Ash Pond. The surface mine was operated from 1974 to 1991. As a result of these activities, the overburden at the project site has been excavated and re-placed as part of the surface mining process.

The foundation of the perimeter dike consists of mine spoils. These materials were previously excavated overburden removed during the strip mining process that was replaced following the removal of coal deposits. Accordingly, mine spoils tend to be heterogeneous. The majority of the mine spoils sampled consists of moist to wet, medium stiff to very stiff, lean clay (CL) with varying quantities of coal and rock fragments.

In 2015, an Initial Annual (Intermediate) Inspection was performed for the Peabody Ash Pond. Based on the inspection report, no evidence of actual or potential structural weakness of the inspected units was observed. No changes that could affect the operational stability of the impounding structure have been identified since the last inspection of the unit.

An assessment of the static slope stability and seepage conditions was performed by AECOM. Cone penetration testing (CPT) was used in conjunction with historically available hollow stem auger (HSA) boring, laboratory, and piezometer data to complete the geotechnical analyses. A review was made of available historical information and a site visit was performed to obtain additional site specific information. Seepage modeling indicated a phreatic surface which remains beneath the surface of the downstream slope during normal pool conditions, which is consistent with recent observations.

The phreatic surface used in the stability analysis was modeled using Seep/W software and known normal and flood pool elevations as boundary conditions. Initially, saturated permeability and anisotropy parameters from existing laboratory data and estimates from published correlations were modeled. Generally, hydraulic conductivities were originally determined from laboratory testing, while anisotropic ratios were based on published information from the United States Bureau of Reclamation. The values were then adjusted in order to calibrate the seepage model to the historical piezometer data and field CPT dissipation data. The calibration process was completed at four cross sections until seepage parameters were determined; yielding a reasonable correlation to field readings. The cross section locations were selected to be representative of the most critical cross sections, such as the maximum embankment height, the steepest embankment slopes, and the least resisting force at and beyond the downstream toe.

The number and location of cross sections reflects engineering judgment to obtain appropriate geo-spatial coverage of the dike. The final calibrated model was conservatively constructed in order to present a phreatic surface slightly higher than the measured data to account for uncertainties. The modeled cross sections are shown in **Appendix B**.

The final seepage parameters are summarized in **Table 1** below.

Table 1: Summary of Seepage Parameters

Material	K_v (cm/s)	*K_h/K_v
Raised Embankment	1.25×10^{-7}	4
Original Embankment	2.5×10^{-7}	4
Bottom Ash	1.5×10^{-5}	4
Sluiced Ash	1.5×10^{-5}	100
Clayey Mine Spoils	2.9×10^{-6}	4
Sandy Mine Spoils	9.1×10^{-6}	4
Class II Channel Lining	1	1

* Anisotropy estimated from United States Bureau of Reclamation (2014).

Generally, the field calibrated vertical permeability values were within an order of magnitude of the laboratory test data. However, the calibrated vertical permeability of the bottom ash was decreased as compared to the available laboratory tests (10-3 cm/s to 10-5 cm/s). It is suspected that the fines from fly ash may be clogging the coarser bottom ash, which would reduce the permeability and may not have been represented in the limited testing that had been previously performed.

As a part of the seepage analysis at each cross section, horizontal and vertical gradients could be determined at individual finite elements. A determination of high or critical, (vertical) exit gradients can also be performed following well established sources (including Terzaghi and Peck, USACE EM 1110-2-1901, and USBR Design Standard No. 13 Embankment Dams, Chapter 8, January, 2014). Determination of critical gradients for a soil is determined when evaluating effective stress conditions. In essence, the critical gradient occurs when the effective stress is zero. Under this condition, a "quick" condition exists in cohesionless soils, and the foundation materials may "boil" or "heave." The critical gradient (Ic) is most commonly expressed as the ratio of the buoyant unit weight of the soil (γb) to the unit weight of water (γw). The factor of safety (FS) with respect to piping is generally defined as the ratio of the critical gradient (Ic) to the predicted or measured exit gradient (Ie) determined using the SEEP/W model. However, as described in the USBR Design Standard No. 13 Embankment Dams, Chapter 8, January, 2014):

> *"It is important to recognize that the critical exit (vertical) gradient and the occurrence of boils and heaving of grains only occur in cohesionless soils. In most cohesive soils (plastic clays), with the exception of dispersive soils, inter-particle attractions create bonds between particles that make it less likely for these soils to lose strength due to seepage or for individual particles to be easily moved. Laboratory tests have shown that while sands can typically move or become quick under an upward gradient of around 1.0, clay particles may not move until threshold gradients reach values in the tens or even hundreds.* ***Thus, any type of critical gradient in cohesive soils would be difficult to measure, would vary widely among such soils (due to such variables as percentage of clay fines, type of clay minerals, water content, and density), and should definitely not be calculated by the above equation.****"*

The embankment and foundation materials of Peabody Ash Pond Dike are constructed with cohesive materials.

The upper 2 feet of the downstream slope extending to the water line consists of KYTC Type II Rip Rap underlain by geotextile fabric. Accordingly, the exit gradients at the toe are predominantly horizontal, as is typical for seepage flow through an embankment or foundation. Again as stated by the in the USBR Design Standard No. 13 Embankment Dams, Chapter 8, January, 2014:

> ***"Although formulae exist for computing factors of safety for conditions of critical exit (vertical) gradients, there is much more uncertainty when it comes to determining internal (horizontal) gradients that are capable of initiating internal erosion."***

Therefore, while it is possible to measure vertical exit gradients and calculate critical gradients based on the available project information, performing a factor of safety calculation against piping is not appropriate for this project.

Concerning uplift or heaving, at two cross sections, B-B' and C-C', a sandy mine-spoil layer approximately 5 feet in thickness is overlain by a confining clay layer at the downstream toe. At the downstream toe of an embankment, if the seepage pressures in a pervious layer are higher than the overburden pressure of the confining layer, uplift of the confining layer may occur. In simplest terms, the factor of safety against uplift can be calculated in total stresses (or forces) as the total downward pressure exerted by the weight of the confining layer divided by the upward water pressure at the base of the layer.

Downstream piezometers within the sand layer have not been installed, so piezometric data within this layer is not available. Seepage analyses assumed steady state conditions have developed between Peabody Ash Pond and Jacob's Creek at normal pool conditions. However, if Jacob's Creek is excluded from the seepage analysis, the phreatic surface is shown to exit at the downstream toe and maintain at the ground surface. Therefore, it is concluded that pore water pressures within the sand layer will not exceed the pore water pressure at the ground surface, and that the elevation of the downstream water body, Jacob's Creek, doesn't represent the pore water pressure within the sand layer, but acts as a buttress against potential heave. On this basis, the calculated factor of safety against heave was found to be greater than 2 at each cross section using the total force analysis (described in USBR Design Standard No. 13 Embankment Dams, Chapter 8, January, 2014, page 8-8 through 8-10). This factor of safety exceeds the recommended value of 1.5 for existing structures.

Furthermore, additional considerations make it less likely that heave would occur at these locations. First, the sand layer encountered during drilling at these cross sections was limited in thickness to 5 feet, whereas it is understood that the classical model for heaving involves layers of greater thicknesses. In addition, the sand layer and clay layer consist of mine spoils re-placed following strip mining, indicating irregular placement, composition, thickness, and continuity in comparison to many natural deposits and engineered fills. Second, sand layer classified as SC, or clayey sand, laboratory testing indicates a fines content of 40 to 45 percent and a field calibrated vertical permeability (9 $\times 10^{-6}$ cm/s) within an order of magnitude of the overlying clay layer (3 x 10^{-6} cm/s). Therefore, it is questionable how pervious the sand layer is in comparison to the overlying clay layer.

Seepage conditions have been analyzed as part of the study for Peabody Ash Pond in accordance with acceptable methodologies. The existing embankments and foundation materials are performing acceptably in regard to piping and heave potential in comparison to current criteria and existing analytical data. Further, no physical or visual evidence of piping, heave, or uplift has been observed in the field during multiple visits to the site between 2015 and 2016.

More information on the assessment of foundations and abutments can be found in the Initial Safety Factor Assessment prepared for CCR Certification by AECOM.

2.2 Slope Protection - §257.73(d)(1)(ii)

The toe of the upstream slope of the dike is armored with riprap from about 2 feet above the water-line to below the water line. Well-maintained grassy vegetation is present between the riprap and crest. The crest of the dike serves as an access road with a gravel surface. The upper portion of the downstream slope consists of well maintained, grassy vegetation, while the lower portions of the downstream slope have been recently modified (2014). Specifically, trees and large vegetation were removed, grades adjusted, and riprap placed along the toe.

In 2015, an Initial Annual (Intermediate) Inspection was performed for the Peabody Ash Pond. Based on the report, the slopes are generally covered with either dense grass or riprap; no trees or large, bushy vegetation were present on the slopes. No evidence of burrowing animals was observed. No evidence of actual or potential structural weakness of the inspected units was observed.

Water travel over the dike slopes will not cause erosive effects based on the current slope protection and condition. Water will not overtop the dikes of the Stilling Ponds during a 1,000-year storm event. No additional slope protection is required based on anticipated erosive flows.

More information on the assessment of slope protection can be found in the 2015 Annual (Intermediate) Site Inspection and the attached **Photos**.

2.3 Embankment Dike Compaction - §257.73(d)(1)(iii)

The dike which forms Peabody Ash Pond is approximately 5,500 feet in length and varies in height from 12 to 20 feet. The dike was originally constructed with surface coal mining spoils to a crest elevation of approximately 400 feet MSL and was later raised to approximately elevation 408 feet MSL. The raised portion was completed to the upstream of the original crest and is partially founded on ash.

Construction documents indicate that when connecting the new raised dike to the original dike, extreme care was used to provide an impervious and stable connection. The raised dike construction documents note that before placing new fill on existing dikes, the existing surface was stripped of all vegetation, stumps, crushed stone, and loose material. Surfaces were scarified, and new fill compacted to bond with existing fill. Soil was placed and compacted in 18 inch successive lifts. Each lift was uniformly compacted with a smooth wheel vibratory roller to at least 95% of maximum dry density as determined by ASTM D-698 procedures (Standard Proctor). All placed soil was to be within ±3% of its optimum moisture content. No rocks larger than 10 inches in diameter were placed within the dike fill. In-place density tests using a sand cone (ASTM D2167) or nuclear (ASTM D2922) test methods were performed at a minimum rate of one test per each 5,000 cubic yards of fill placed or a minimum of one test per day that earth fill is placed.

Construction documents noting the methods of embankment dike compaction can be found in the History of Construction Report prepared for CCR Certification by AECOM**.**

2.4 Vegetated Slopes - §257.73(d)(1)(iv)

The toe of the upstream slope of the dike is armored with riprap from about 2 feet above the water-line to below the water line. Well-maintained grassy vegetation is present between the riprap and crest. The upper portion of the downstream slope consists of well maintained, grassy vegetation, while the lower portions of the downstream slope have been recently modified in 2014. Specifically, trees and large vegetation were removed, grades adjusted, and Class III Channel Lining with riprap placed along the toe.

In 2015, an Initial Annual (Intermediate) Inspection was performed for the Peabody Ash Pond. Based on the report, the slopes are generally covered with either dense grass or riprap; no trees or large, bushy vegetation are present on the slopes. No evidence of burrowing animals was observed. No evidence of actual or potential structural weakness of the inspected units was observed.

More information on the assessment of vegetated slopes can be found in the 2015 Initial Annual (Intermediate) Inspection and the attached **Photos**.

2.5 SPILLWAY CONDITION AND CAPACITY - §257.73(d)(1)(v)

Under existing conditions, the drainage area for Peabody Ash Pond is approximately 1028 acres. Peabody Ash Pond receives sluiced fly ash flows via a long ditch that enters the pond near the southwest corner of the impoundment. Decant water flows through an open channel in the internal divider dike to Peabody Stilling Pond. From the Stilling Pond, water flows west to east via three 36” Class IV concrete pipes and discharges into Jacob’s Creek. The lengths of the spillway pipes, measured north to south, are 404 feet, 392 feet and 416 feet, respectively. The skimmer device is composed of a 5ft section of 120” dia. galvanized corrugated metal pipe with interior steel angle bracing. An emergency overflow spillway located on the eastern dike is made by a depressed section of rip-rap. This spillway is at elevation 407.4 feet MSL, which is 0.6 feet MSL below the eastern dike. The primary and emergency spillways are constructed with non-erosive material to adequately protect against water erosion.

More information on the existing spillway structures condition can be found in the History of Construction Report prepared for CCR Certification by AECOM and the attached **Photos**.

AECOM performed a hydrologic and hydraulic (H&H) study of the existing Peabody Ponds. This analysis was performed in the Initial Inflow Design Flood Control Plan prepared for CCR Certification by AECOM.

An H&H computer model was developed using HEC-HMS to examine the hydraulic behavior of Peabody Ash and Stilling Pond complex during the Inflow Design Flood (IDF). The required IDF used in the model calculation is based on the pond’s hazard classification. Since Peabody Ash Pond was classified as a significant hazard, the required IDF is a 1,000-year flood.

All structure dimensions and invert elevations are modeled using the best available information under current operating conditions of the PAF Plant. Existing topographic and survey information for the Peabody Ash Pond Complex was provided by TVA. Drainage areas, volumes, and other site geometry were determined using the AutoCAD Civil 3D software package in conjunction with survey data provided by TVA

The modeling results indicate the Peabody Ash Pond would not overtop during a 1,000-year design storm, and the freeboard for Peabody Ash Pond during this storm event is adequate.

More information on the assessment of spillway capacity can be found in the Initial Inflow Design Flood Control Plan prepared for CCR Certification by AECOM.

2.6 SPILLWAY STRUCTURAL INTEGRITY - §257.73(d)(1)(vi)

Peabody Stilling Pond has three spillways that are used as the primary pond discharge devices. Each spillway consists of a foundation pad, a concrete junction box, a vertical 48” concrete riser topped with a metal skimmer device, and a horizontal 36” concrete culvert pipe. All water entering the spillway travels east and discharges into Jacobs Creek.

The spillway foundation consists of a reinforced concrete pad measuring 6.5 feet x 6.5 feet x 1.5 feet. Topping each pad is a hollow reinforced concrete box having 1 foot thick walls measuring 6 feet x 6 feet x 4 feet. The vertical 48” risers are class IV concrete pipes with 4” wall thickness. The skimmer devices consist of 5 foot sections of galvanized corrugated metal pipe being 10 feet in diameter and are braced with interior steel angles. The horizontal 36” Class IV concrete pipes have lengths from north to south of 404 feet, 392 feet and 416 feet, respectively.

Refer to the Construction Documents in the History of Construction Report prepared for CCR Certification by AECOM for additional information on the existing spillway structures.

2.6.1 SITE INSPECTION AND FINDINGS

On October 8th 2015, AECOM conducted a site inspection to evaluate the condition of the three risers at Peabody Stilling Pond and the outlet of the concrete pipes at Jacob’s Creek. Conditions for all three spillways were in similar condition. The skimmer devices are open at the top, and they exhibit minor surface rust on the corrugated metal pipe and associated steel bracings. The visible portions of the outer concrete risers are in good condition. Cracking, deterioration or spalling was not detected. There are no apparent deteriorations or obstructions to water flow from the top of the riser. The water level was measured in at roughly 3” above the top of the riser #2 (nearest to walkway). In-depth inspection was restricted due to the present water level and the risers’ distance to the edge of the pond. Using a probing rod, the corrugated metal pipe skimmer was checked for deterioration at the base. None was detected. See **Photos 11 & 12** for typical condition of the risers.

The spillway outlet concrete pipes are in good condition with minor spalls at the outer circumference. The access panels to the 36” pipes reveal exposed rebar from the cut-out, however the surrounding concrete is still in good condition. The water level is low inside the pipes and flow is steady without any obstructions. See **Photos 13, 15, and 16** for the typical condition of the 36” concrete pipes.

Evaluation of historical video inspection recordings performed in 2014 showed a clear pathway for the water flow without any obstruction or debris present inside the spillways. The interior of the pipes are in acceptable condition. **Photo 16** was from the available videos and represents the conditions described above.

2.6.2 STRUCTURAL ASSESSMENT

The riser structures were evaluated for two different limit states. The first is associated with regularly occurring reservoir levels. The critical condition for floatation of the riser structures occurs when the reservoir level is near the top of the riser structures, but water is not flowing over. It was assumed that the riser structures were not filled with water. The buoyant force is acting on the outside, but the riser structures are not filled with water. The critical condition for bearing capacity at the base of the riser structures is when the risers are filled with water. Sliding and overturning moment were not checked for this limit state because the structure is subjected to equalized hydrostatic pressure.

The second limit state is associated with loading under the 1,000-year storm event. Evaluation for this flood event is required for a significant hazard potential unit per rule §257.73(d). It has been determined that the 1,000-year storm event will not likely overtop the pond, so there will not be a flow velocity on the side of the riser structures. It was decided to determine the flow velocity for which the bearing capacity check, sliding check and moment equilibrium are satisfied. An additional check was included to confirm that the risers do not tip off the base under the specified flow velocities. This is a direct comparison of overturning moment applied by flow velocity to resisting moment of self-weight, with no safety factor.

The existing structures satisfy the factor of safety requirements for both limit states under each condition evaluated.

See **Appendix A** for the results for each limit state including the associated calculations, structure geometry and material properties.

2.7 SUDDEN DRAWDOWN - §257.73(d)(1)(vii)

Two lagoons encompassing a total of about 80 acres and consisting predominantly of open water front the downstream toe of Peabody Ash Pond dike for a distance of approximately 2,000 linear feet on the southern end of the Ash Pond. It is expected that the lagoons are man-made remnants of surface mining activities.

Jacob's Creek flows along the downstream toe from the lagoons north and east toward its discharge point. Along the majority of the length of Jacob's Creek, the ground surface on the eastern side of the stream rises steeply upward, resulting in a relatively narrow stream channel. However, a topographically flat area in the channel is present approximately halfway between the lagoons and discharge point, which allows the stream to widen and the wetted perimeter to increase.

Based on the width of the stream channel along the majority of Jacob's Creek, AECOM judges that the potential exists for flood-water elevations to drop quickly in comparison with the rate at which water elevations in Peabody Ash Pond would drop. Therefore, it is possible that steady state seepage conditions at flood pool could prevail within the embankment after the Jacob's Creek water elevation has fallen to normal pool elevations. The analysis was performed based on peak drained (effective stress) conditions.

2.7.1 MATERIAL PROPERTIES

The raised embankment of Peabody Ash Pond dike consists of moist, stiff, lean clay (CL) with some rock fragments. The embankment materials consist of mine spoils, and the embankment was constructed in a controlled manner with compactive effort. The embankment fill extends from the crest elevation (approximately 408 feet) to elevations of approximately 400 feet to 395 feet, resulting in an embankment varying from 8 to 13 feet in thickness.

The original embankment fill materials generally consist of moist, stiff, silty, lean clay (CL) with irregular quantities of silt, sand, coal and rock fragments indicating that the original embankment was also constructed using the local mine spoil materials. Construction records associated with the original embankment were not found. Overall, the original embankment fill extends from the crest elevation of approximately 401 feet MSL to elevations of approximately 394 feet to 384 feet MSL. Therefore, the thickness of the original embankment varies from 6 to 15 feet.

2.7.2 Critical Cross Section Selection

Upon determination of the subsurface conditions, analysis cross-sections were generated. The cross section locations were selected to be representative of the most critical cross sections, such as the maximum embankment height, the steepest embankment slopes, and the least resisting force at and beyond the downstream toe. The number and location of cross sections also reflects engineering judgment to obtain appropriate geo-spatial coverage of the dike. A total of three cross sections were constructed for these analyses as shown in **Appendix B.** Sudden drawdown analysis was not performed at cross section A-A', as the downstream water body consists of a 40 acre lagoon, the flood pool elevation of which would not be expected to decrease fast enough to induce sudden drawdown conditions.

2.7.3 Water Levels

The lagoons east of the perimeter dike appear to be connected by a weir and feature normal pool elevations of 396 to 399 feet MSL with flow from the west to east into Jacob's Creek. The ordinary high water elevation of Jacob's Creek varies from 396 feet MSL to approximately 387 feet MSL near its discharge point. The 100-year water surface for Jacob's Creek is approximately 402.0.

Normal operating level of Peabody Ash Pond and Peabody Stilling Pond is 405 feet MSL. Flood pool elevation of Peabody Ash Pond and Peabody Stilling Pond during a 100 year storm is approximately 407 feet MSL.

2.7.4 Analysis

Sudden drawdown analysis was completed to evaluate the existing conditions at Peabody Ash Pond Dike at three cross sections. This was done with Slope/W software, which implements a multi-stage analysis approach known as the "Improved Method for Rapid Drawdown" by Duncan, Wright, and Wong (1990) referenced in the United States Army Corps of Engineers Manual EM 1110-2-1902. In this approach, a three phase analysis is adopted, and a composite shear strength envelope using both drained and undrained shear strength parameters is used as described in EM 1110-2-1902. At each analyzed location, drawdown is assumed to consist of flood pool to normal pool.

The minimum factor of safety varies under each loading condition. The result from the Slope/W module output plots are summarized below in **Table 2**.

Table 2: Summary of Minimum Static Slope Stability Factors of Safety

Loading Condition	Acceptable Factor of Safety (Minimum)	Cross Section Location			
		A-A'	B-B'	C-C'	D-D'
Steady State Normal Pool	1.50	1.64	1.86	1.53	1.65
Steady State Flood Pool	1.40	1.61	1.75	1.45	1.56
Sudden Drawdown	1.30	N/A*	1.70	1.39	1.37

*Sudden drawdown deemed inappropriate due to size of downstream water body

The analysis indicated acceptable static slope stability factors of safety for sudden drawdown condition at each cross section analyzed.

3.0 Conclusion

Based on the initial structural stability assessment, the requirements of **Rule §257.73(d)(1)** for the Peabody Ash Pond have been met.

4.0 References

1. Environmental Protection Agency, “Final Rule: Disposal of Coal Combustion Residuals from Electric Utilities”, Federal Register, April 17, 2015.
2. AECOM, Peabody Ash Pond, Initial Inflow Design Flood Control System Plan (40 CFR 257.82) prepared for Coal Combustion Residuals (CCR) Existing Surface Impoundments, 2016.
3. AECOM, Peabody Ash Pond, Initial Safety Factor Assessment (40 CFR 257.73(e)(1)) for Coal Combustion Residuals (CCR) Existing Surface Impoundments, 2016.
4. TVA Paradise Fossil Plant Wastewater Flow Schematic Rev 9-11.
5. TriAD, Paradise Fossil Plant, Initial Annual (Intermediate) Inspection, 2016.
6. USACE, "USACE EM 1110-2-1902 Slope Stability," October 31, 2003.
7. USBR Design Standard No. 13 Embankment Dams, Chapter 8, January, 2014.
8. K. Terzaghi and R. B. Peck, Soil mechanics in engineering practice, New York: John Wiley & Sons, Inc., 1996.

Photos

AECOM

TVA PARADISE FOSSIL PLANT

PARADISE, KENTUCKY

PHOTO LOCATION MAP
PEABODY ASH POND

Photo 1 – Southeast dike condition

Photo 2 – Southeast dike condition

Photo 3 – East dike condition

Photo 4 – Stilling Pond dike condition

Photo 5 – East dike condition

Photo 6 – Northeast dike condition

Photo 7 – North dike condition

Photo 8 – North dike condition

Photo 9 – North dike condition

Photo 10 – General view of risers

Photo 11 – Spillway skimmer, middle and south

Photo 12 – Spillway skimmer, middle (Typ.)

Photo 13 – 36" Pipe spillway outlets (east)

Photo 14 – Outlet flow into Jacob's Creek

Photo 15 – Access location near outlet end of 36" concrete pipes (Typ.)

Photo 16 – Flow inside 36" concrete pipe (Typ.)

Appendix A

Hydraulic Structures Assessment Calculation Package

Appendix B – Hydraulic Structures Assessment Calculation Package

a. Material Properties

The properties defined below are determined using the TVA-CCR rule template 257.73 (d), existing plans (1996), AECOM geotechnical data report for Peabody Ash Pond and Stilling Pond (2015), historical data and engineering judgment.

i. Foundation Soil

	Symbol	Value	Units	Reference /Equation
Friction Angle	Φ	26	degrees	Per AECOM Geotechnical Data Report (2015) – Sandy Mine Spoils
		0.45	radians	
Unit Weight		129	pcf	Per AECOM Geotechnical Data Report (2015) – Sandy Mine Spoils
Apparent Cohesion of Foundation	c	0.00	pcf	Neglected Per TVA CCR 257.73d - 2.1.5
Ultimate Bearing Capacity	q_{ult}	3000	psf	Per AECOM Geotechnical Data Report (2015) – Sandy Mine Spoils

ii. Concrete (No adjustment made to concrete base on good condition)

	Symbol	Value	Units	Equation
Unconfined Compressive Strength	f_c	3000	psi	Assumed for Class X
Shear Strength		300	psi	$0.1*f_c$ - TVA CCR Rule 257.3 (d) - 2.1.1
Static Tensile Strength	f_t	353.61	psi	$1.7*(f_c)^{2/3}$ - TVA CCR Rule 257.3 (d) - 2.1.1
Dynamic Tensile Strength		530.42	psi	$1.5*f_t$ - TVA CCR Rule 257.3 (d) - 2.1.1
Instantaneous Modulus of Elasticity	E_c	3122019	psi	$57000*\sqrt{f'_c}$ - TVA CCR Rule 257.3 (d) - 2.1.1
Sustained Modulus of Elasticity	E_c	2185413	psi	$0.7*E_c$ - TVA CCR Rule 257.3 (d) - 2.1.1
Unit Weight	Υ_c	150	pcf	TVA CCR Rule 257.3 (d) - 2.1.1
Poisson's Ratio	ν	0.2		TVA CCR Rule 257.3d - 2.1.1

iii. Reinforcing Steel

	Symbol	Value	Units	Equation
Yield Strength of Steel	f_y	60	ksi	Per TVA Exist. Dwg # 10W3257-3
Modulus of Elasticity	E_s	29000	ksi	TVA CCR Rule 257.3 (d) - 2.1.1

iv. Water

	Symbol	Value	Units	Equation
Density	ρ_w	1.94	$slugs/ft^3$	
Unit Weight	γ_{water}	62.4	pcf	

v. Sluiced Ash

	Symbol	Value	Units	Equation
Saturated Density	ρ_{ash}	3.106	$slugs/ft^3$	
Saturated Unit Weight	γ_{ash}	100	pcf	Per AECOM Geotechnical Data Report (2015)

b. Geometry

Plans used are from Existing Drawings of Paradise Fossil Plant Tennessee Valley Authority.

Date	Drawing No.	Description
Jan-4-1996	10W3274-4	Plan Discharge Spillways
Jan-4-1996	10W3274-4	Typical Profile at Inlet
Jul-30-1993	10W3257-3	Type B - Spillways
Jul-30-1993	10W3257-4	Weir & Skimmer Details

References	
1.	Per Existing Drawing Notes, Concrete is Class "X". Typical compressive strength of concrete range 3000 - 6000psi, conservatively use 3,000 psi
2.	Per Existing Drawing, Riser Invert Elevation is 397.00 ft.
3.	Per Existing Drawings, Bottom of Foundation Elevation is 395.5ft
4.	Water Surface Elevation per HydroCAD Model is 404.9 ft, use 405 ft

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Appendix B - Hydraulic Structures Assessment Calculation Package

Geometric Properties of Riser

	Symbol	Value	Units	Reference
Top Elevation of Structure	EL_{TOP}	405	ft	Exist. Water Surface Elev.
Bottom Elevation of Structure (Elevation to top of Foundation)	EL_{BOT}	395.5	ft	Dwg # 10W3274-4
Total Structure Height	H	9.50	ft	
Height of Water inside (from bottom of riser)	H_i	0.00	ft	Critical for Floatation
Height of Water outside (from bottom of riser)	H_0	9.50	ft	

Cross section Shape: Round or Rectangular	Round (Top)

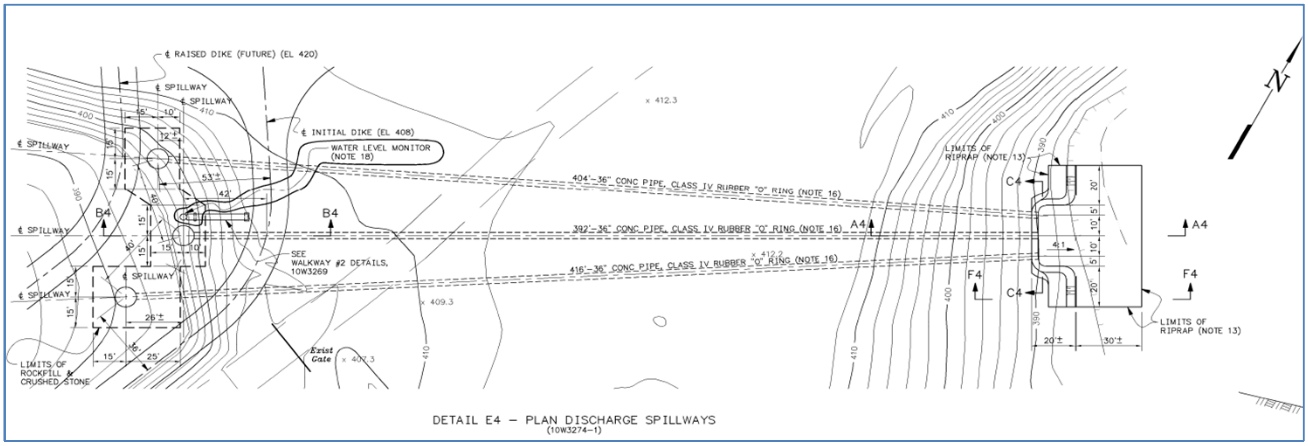
Cross Sectional Properties of the Concrete Box Riser (Rectangular)				
External Width in direction of excitation (longitudinal)	$2a_0$	6	ft	Concrete Base Box
External Depth in direction of excitation (longitudinal)	$2b_0$	6	ft	
Internal Width in direction of excitation (longitudinal)	$2a_i$	4	ft	
Internal Depth in direction of excitation (longitudinal)	$2b_i$	4	ft	

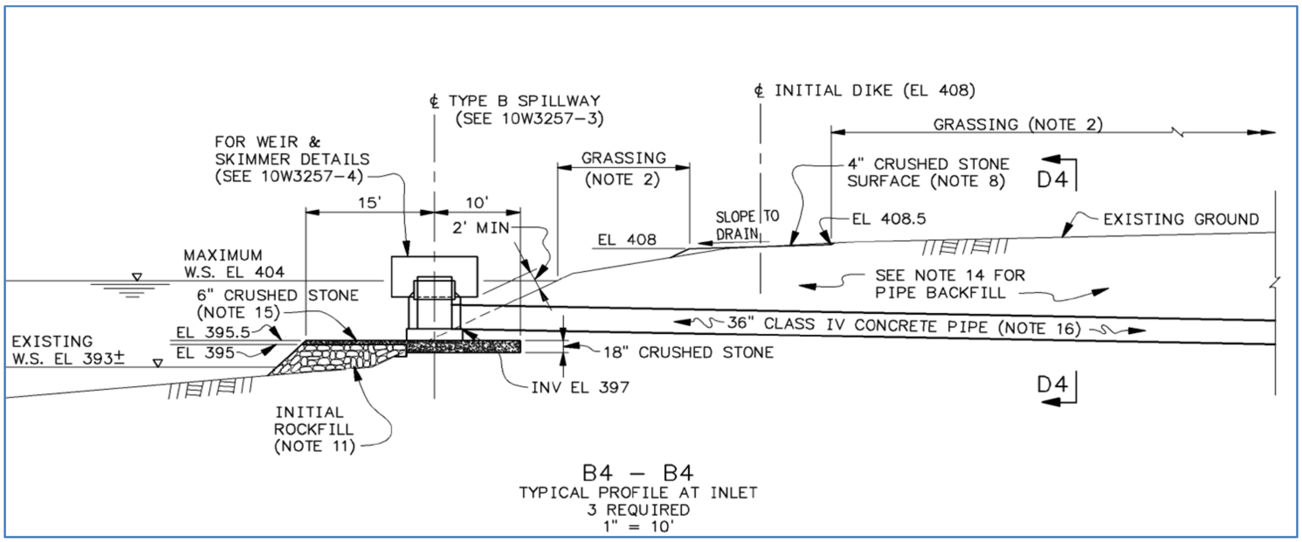
Cross Sectional Properties of the Riser (Circular)				
External Radius of Riser	r_0	2.33	ft	Dwg # 10W3257-3
Internal Radius of Riser	r_i	2	ft	Dwg # 10W3257-3
Weir & Skimmer Weight at Top	W_{TOP}	2	kips	See calcs
Foundation Properties				
Width of Footing	B	6.50	ft	Dwg # 10W3257-3
Length of Footing	L	6.50	ft	Dwg # 10W3257-3
Depth of Footing	$D_{footing}$	1.5	ft	Dwg # 10W3257-3

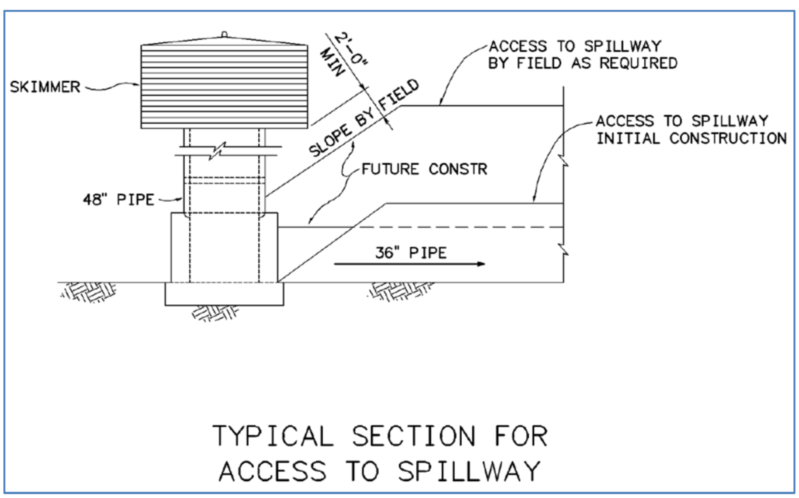
Notes: Transition at the bottom of the riser was ignored for analyses

Existing Drawings

Plan and Elevation

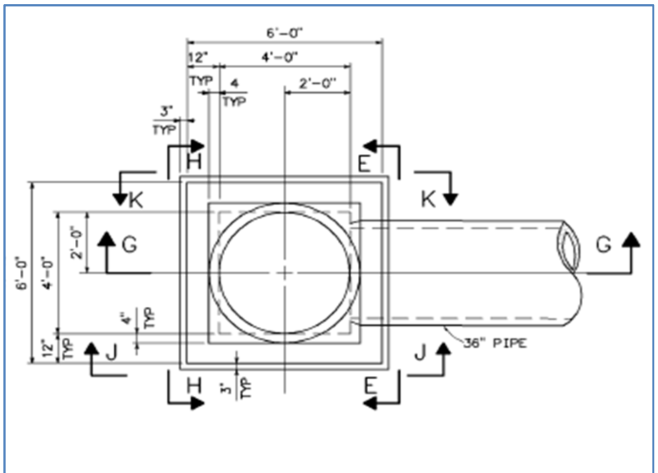




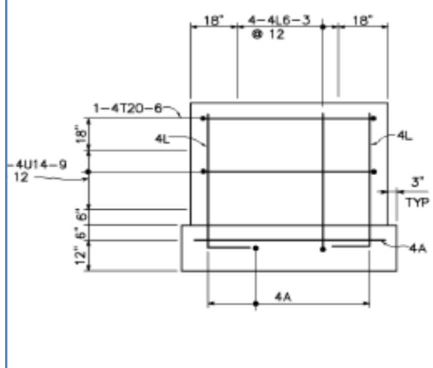


Existing Drawings

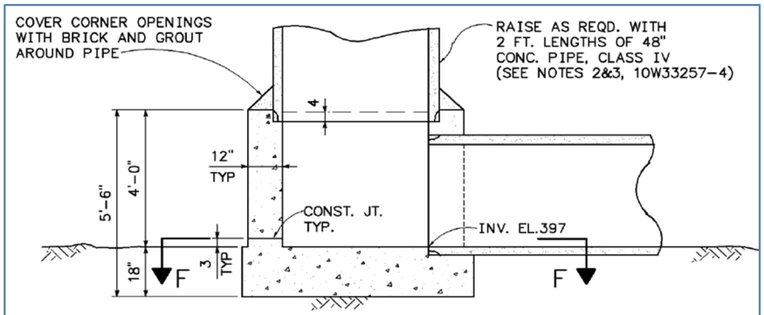
Spillway Typical Sections



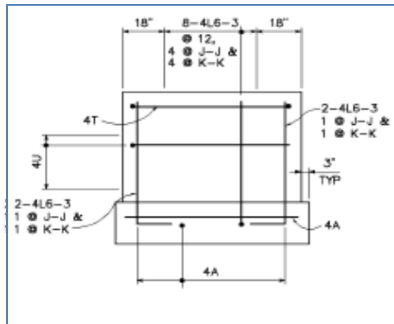
PLAN–SPILLWAY TYPE B



H–H

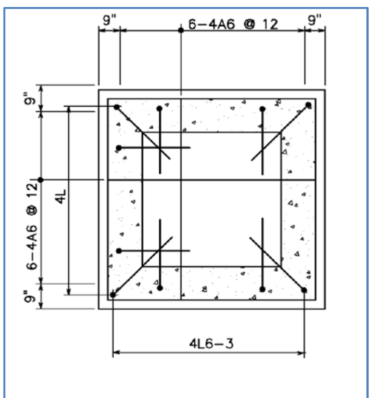


G–G



J–J & K–K

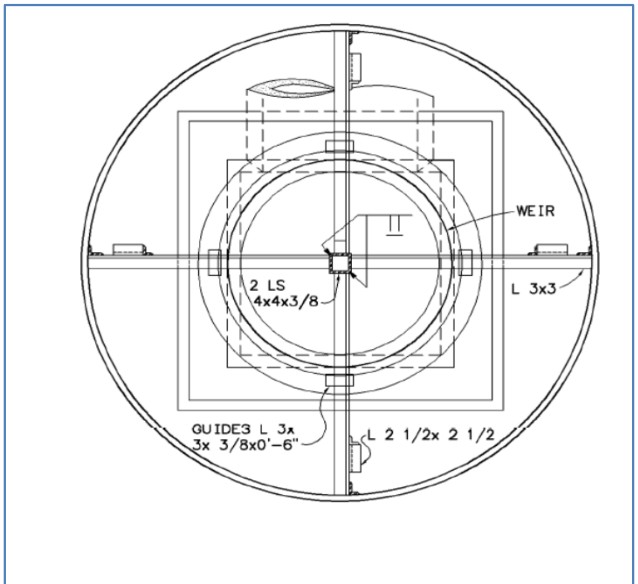
K–K OPPOSITE HAND



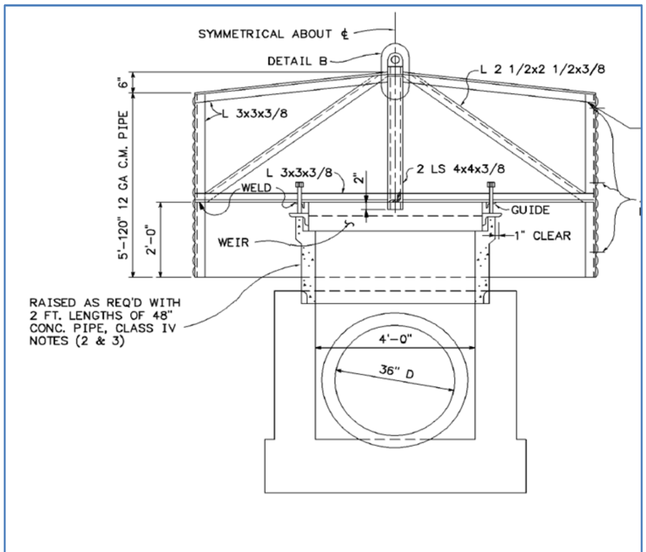
F–F

Existing Drawings

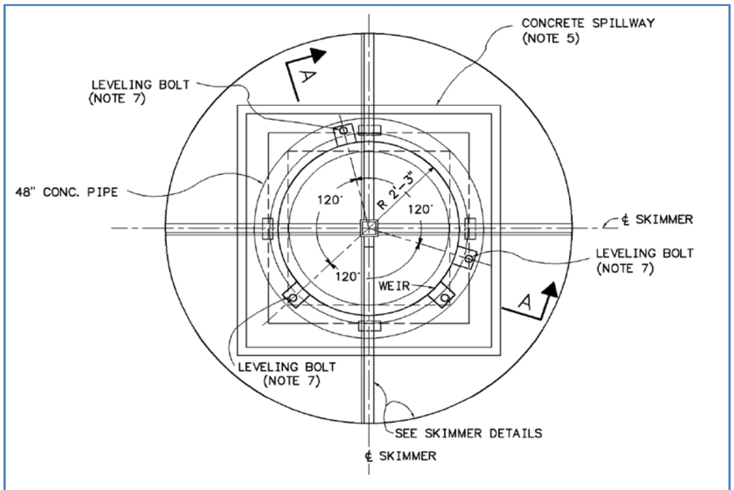
Weir and Skimmer Details



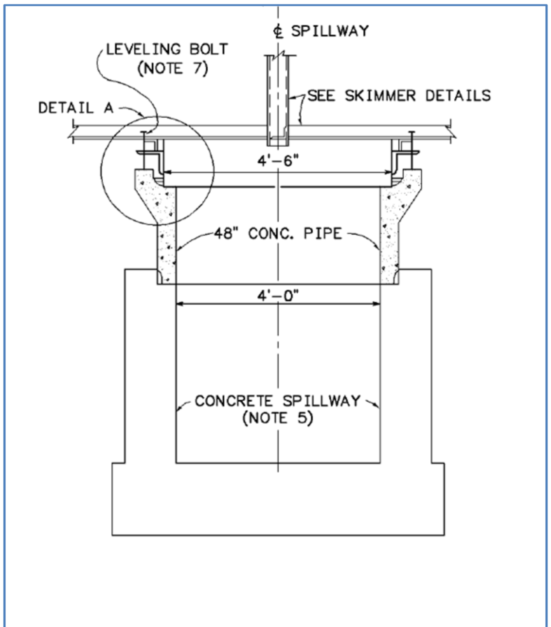
SECTIONAL PLAN



SECTIONAL ELEVATION



PLAN



SECTION A—A

c. Limit States

i. Regularly Occuring Reservoir Levels - Usual

References

TVA-CCR Rule Core Template (257.73(d))

USACE EM 1110-2-2100

USACE EM 1110-2-2400

USGS - Uniform Hazard Response Spectra (UHRS)

Existing TVA plans - Drawing No. 10W3274 - 1,2, 4 & 10W3257-3,4

Geotechnical Data Report prepared by AECOM dated December 18, 2015

Structure Type: Normal

Floatation Stability

Load Condition Category: Usual

Minimum Allowable Factor of Safety = 1.3 Per USACE EM 110-2-2100

Assume No water in Pipe - Worst case Scenario

Floatation Factor of Safety = $FS_f = (W_S+W_C+S) /(U-W_{G})$

Weight of structure			
	Area (ft^2)	Height (ft)	Weight (kips)
Weir and Skimmer			2.00
48" Riser Pipe	4.54	4.00	2.72
Concrete Box	20.00	4.00	12.00
Foundation	42.25	1.50	9.51
Weight of Structure : W_S =			26.23

See Weir and Skimmer Calcs

Weight of Water Displaced			
	Area (ft^2)	Height (ft)	Weight (kips)
48" Riser Pipe	17.10	4.00	4.27
Concrete Box	36.00	4.00	8.99
Foundation	42.25	1.50	3.95
Uplift Force : U =			17.21

Surchage: S = 0

Weight of water above Surface: W_G = 0

Floatation Check:

@ bottom of Base 1.52 OK

Bearing Capacity

Structure Type	Normal	
Load Condition Category	Usual	
Bearing Pressure: Q_{all} =	3000.00	lbs/ft^2 Sandy Mine Spoils
Allowable Factor of Safety =	3	Per TVA-CCR 257.73(d) - Table 5

Assume No water in Pipe

Bearing Capacity Factor of Safety = $FS_f = (Q_{all}) / (W_s + W_w)$

Weight of Structure				
	Area (ft^2)	Height (ft)	(kips)	
Weir and Skimmer			2.00	See Weir and Skimmer Calcs
48" Riser Pipe	4.54	4.00	2.72	
Concrete Box	20.00	4.00	12.00	
Foundation	42.25	1.50	9.51	
Weight of Structure : W_S =			26.23	

Weight of water inside riser			
	Area (ft^2)	Height (ft)	Weight (kips)
Inside 48" Riser Pipe	12.57	4.00	3.14
Inside Concrete Box	16.00	4.00	3.99
Weight of water inside riser: W_w =			7.13

Base Pressure = 789.57 lbs/ft^2

Bearing Capacity Check:

@ bottom of foundation 3.80 OK

Check is conservative. Includes water inside riser, but does not include buoyant force lifting up on structure.

Weir and Skimmer Weights

From Bill of Material - Dwg # 10W3257-4

Weir						
Material	Quantity	Unit weight	Thickness (in)	Height (in)	Length (ft)	Weight (lbs)
Angle L6x6x3/4 x 6"	3	28.80 lb/ft			0.50	43.2
1" Dia. Heavy Duty Bolt with Nut - 10"	3	Used Portland Bolt Weight Calculator				9.3
1/4" Stainless Steel Plate (Type 304)	1	501.12 lb/cf	0.25	12	14.50	151.38
Skimmer						
120" x 12" Corrugated Metal Pipe	1	187.00 lb/ft			5.00	935
1/2" Dia. Galvanized Bolts - 1.5"	12	Used Portland Bolt Weight Calculator				1.2
Angle L2 1/2 x 2 1/2x 3/8		5.90 lb/ft			23.00	135.7
Angle L3 x 3x 3/8		7.17 lb/ft			64.00	458.88
Angle L4 x 4x 3/8		9.72 lb/ft			8.00	77.76
					Sub-total	1812.42
				Added 5% for Misc.		1903.041
					Use	2000
					Total (Kips)	**2**

ii. 1000 Year Flood (Significant Hazard Potential Unit) - Unusual

References

TVA-CCR Rule Core Template (257.73(d))

USACE EM 1110-2-2100

USACE EM 1110-2-2400

USGS - Uniform Hazard Response Spectra (UHRS)

Existing TVA plans - Drawing No. 10W3274 - 1,2, 4 & 10W3257-3,4

Geotechnical Data Report prepared by AECOM dated December 18, 2015

Structure Type	Normal
Load Condition Category	Unusual

Floatation Stability

The riser will be completely submerged during this flood event. Floatation will not be an issue for this event.

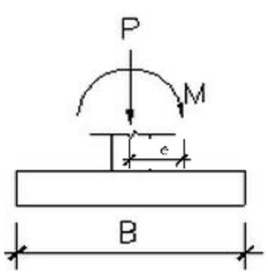
Appendix B - Hydraulic Structures Assessment Calculation Package

Sliding Friction					
Angle of internal friction	**Φ**	**26**	**degrees**	**Per TVA-CCR 257.73d, Foundation Cohesion, c =**	**0**

Base Sliding (1000 Yr Flood)								
Nodes	**Height, h_i**	**Weight @ Nodes**	**Bouyant Force**	**Sliding Resistance (F_N)**	**Sliding Force (F_D = Shear Force Parallel to the base)**	**Sliding/No Sliding**	**Sliding Factor of Safety**	**UNLC Sliding Factor of Safety TVA-CCR- (Table 3)**
	(ft)	kips	kips	kips	kips			
Base	0.00	33.36	17.21	7.877	* 4.06	No Sliding	1.94	1.5
								Pass

* Sliding force for Flood Water Max Velocity of 6 ft/s

Loads			
Dead Load at Base	DL	33.36	kips
Bouyancy Force at Base	Fb	17.21	kips
Axial Load at Base	P = DL-Fb	16.15	kips
Moment at center Base	M	25.39	k-ft



Foundation Dimensions			
Base Width	B	6.50	ft
Base Length	L	6.50	ft
Area	A	42.25	ft^2
Section Modulus	S	45.77083	ft^3

Per USACE EM 110-2-2100 Table 3-5 - 75% of Base incCompresion

Resultant	e=M/P	1.57
	B/6	1.08
	L/6	1.08
	e>(B/6)	Yes
Resultant		1.68
% of Base in Compression		77.43%

Maximum Bearing Pressure when e > L/6	Q_{max}	2P/[3(B/2-e)L]	0.987373	ksf
Maximum Bearing Pressure when e<L/6	Q_{max}	P/A+M/s	0.937079	ksf

q_{ult}	3	>	Q_{max}	0.99	ksf

FOS	3.04	
UNLC FOS	2.6	Passes

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Appendix B - Hydraulic Structures Assessment Calculation Package

Strength Calculation - (1000 yr Flood - Significant Hazard Potential Unit)

Shear at Critical Section

Assumptions:

1) A reduction of area occurs at the stacked pipes joints.
2) No reinforcing is accounted for in the critical section.

Section Properties			
Exterior Radius	r_0	2.33	ft
Interior Radius	r_i	2.00	ft
Wall thickness	t	4	in
Pipe Cross Sectional Area	A_{pipe}	4.538	ft^2
Percent Area Reduction	% $A_{reduced}$	25%	
Reduced Area	$A_{reduced}$	3.403	ft^2
Section Modulus	S_x	4.592	ft^3

Material Properties			
Concrete Compressive Streng	f_c	3000	psi
Concrete Tensile Strength	f_t	353.61	psi
Concrete Shear Strength	.1*fc	300	psi
Plain Concrete, Φ =	0.55	ACI 318 - for both flexure and shear	

Hydrostatic force is equal all around the Riser

Hydrodynamic Force				
Drag Coefficient	C_d	1.25	USACE - Drag Coefficient shall not be less than 1.25	
Velocity of Flood water	V	6.00	ft/s	Assume Max For 1000 yr Flood - conservative
Mass Density of water	ρ_w	1.94	$slugs/ft^3$	
Flood Elevation	H	12.50	ft	Assume skimmer submerged
Base Width	W	6.50	ft	
Submerged Area of face of the Riser	A	93.08	ft^2	
Hydrodynamic Force { $(C_d \rho V^2/2)*A$ }	F_d	4.06	kips	Acts @ H/2
Lateral Force Location	H/2	6.25	ft	

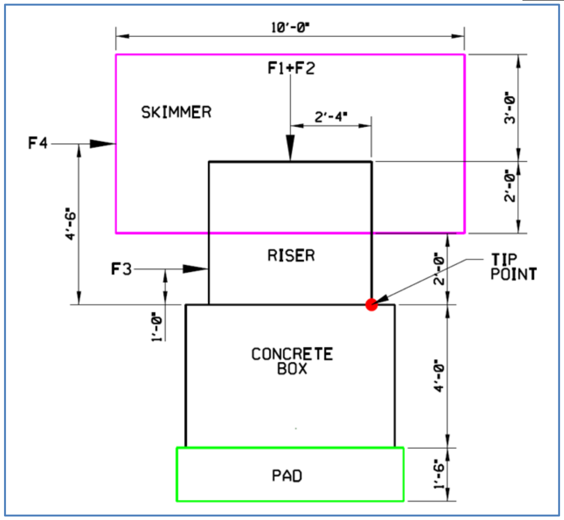
Appendix B - Hydraulic Structures Assessment Calculation Package

Overturning Check during 1000 yr Flood

	Weight (kips)	Arm (ft)	Moment $_{Resist.}$ (k-ft)
$F_{1,Resist}$ (Riser)	2.72	2.33	6.35
$F_{2,Resist}$ Skimmer	2.00	2.33	4.67
Total Moment $_{Resist.}$			11.02

	Force (kips)	Arm (ft)	Moment $_{Overturn}$ (k-ft)
$F_{3,Overturn}$ (Riser)	0.41	1.00	0.41
$F_{4,Overturn}$ (Skimmer)	2.18	4.50	9.82
Total Moment $_{Overturn}$			10.23

Riser will not tip over



Per TVA - CCR Rule 257.73 (d) -Strength Capacity, in accordance with ACI 350 Code

ACI 350 -Capacity Reduction Factor		use for plain concrete
Shear	0.75	0.55
Bending	0.9	0.55

Factored Load	
Loads due to Fluids	1.4

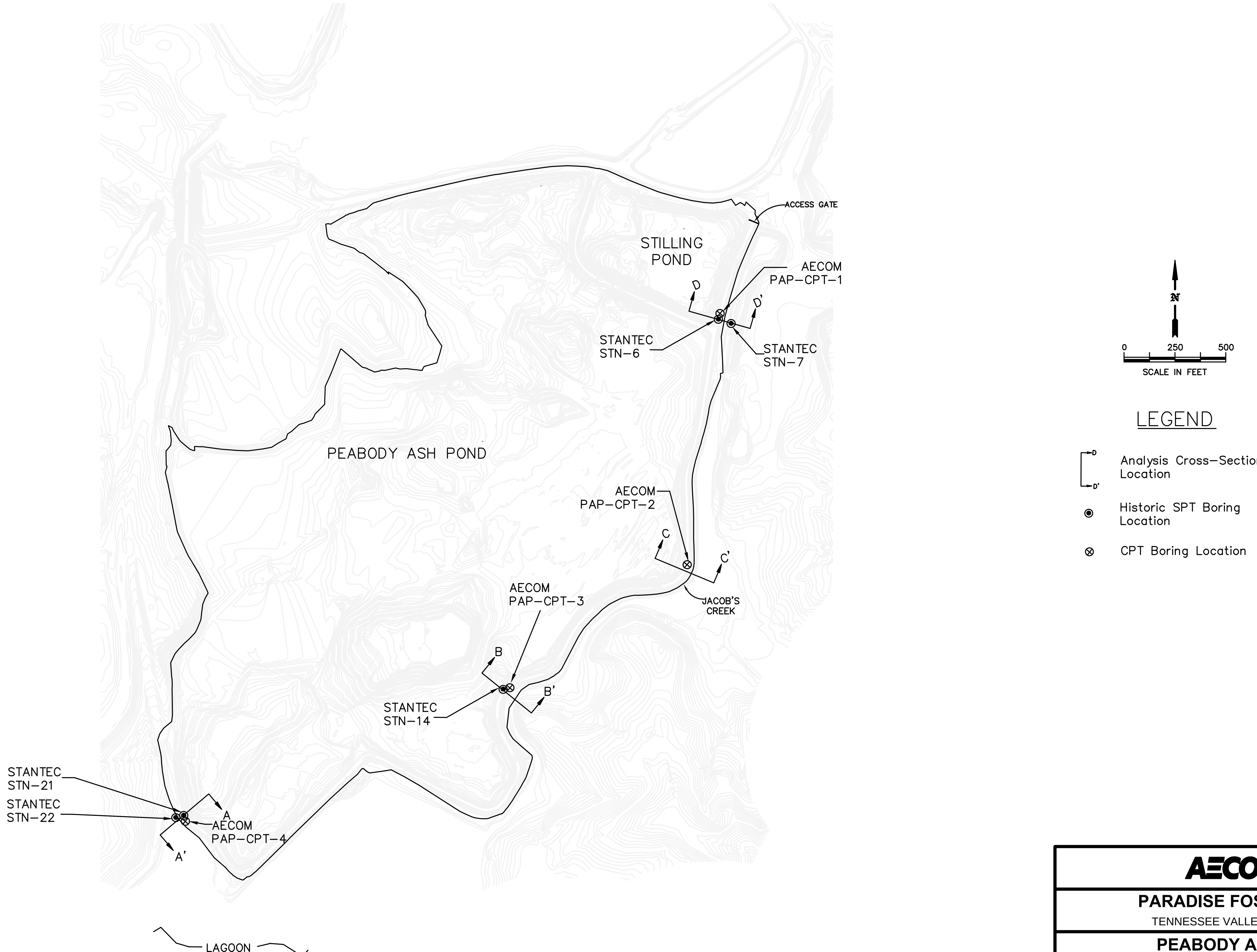
Strength Calculation - (1000 yr Flood - Significant Hazard Potential Unit)

Shear Capacity at Critical Section	$V = \Phi*.10*f_c*A_{reduced}$		80.86459	kips	>	V_{max}	3.63	kips	**PASSES**
Shear Capacity of Gross Section	$V = \Phi*.10*f_c*A_{pipe}$		107.8195	kips	>	V_{max}	3.63	kips	**PASSES**

Maximum Moment in Compression	$Mc = \Phi*f_c*S_x$		1091.03	K-ft	>	M_{max}	14.32	K-ft	**PASSES**
Maximum Moment in Tension	$Mt = \Phi*f_t*S_x$		128.6013	K-ft	>	M_{max}	14.32	K-ft	**PASSES**

APPENDIX B

GEOTECHNICAL EXPLORATION AND ANALYSIS
CROSS SECTIONS AND BORING LOCATION



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PARADISE FOSSIL PLANT

TENNESSEE VALLEY AUTHORITY

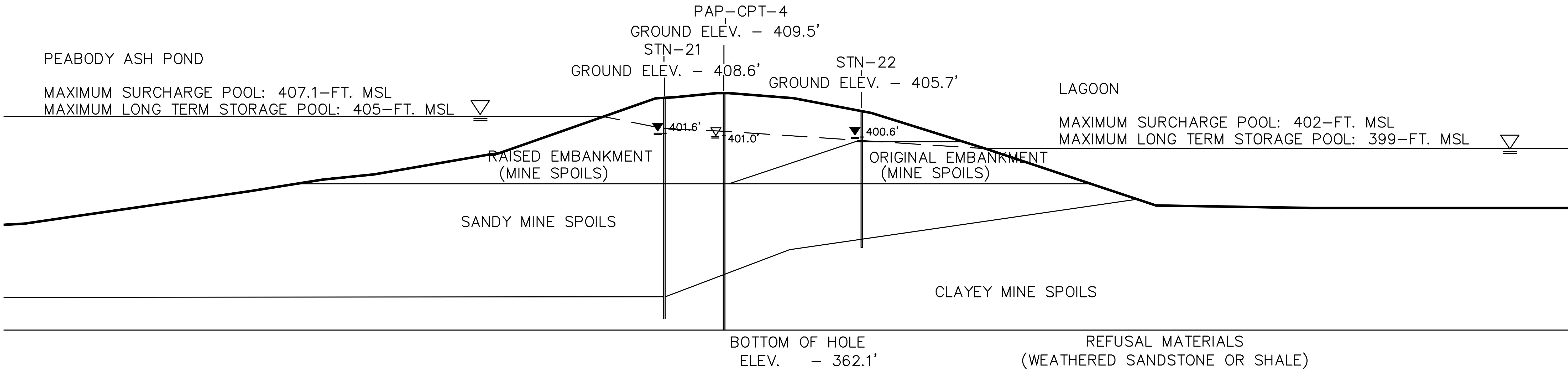
PEABODY ASH POND
BORING PLAN VIEW

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NC	MW	60439833	09/16/16	1

\\10.73.7.1\shared\TVA CCP\PAF\Peabody Ash Pond\Drawings\Peabody CPT-Historic Plan Viewupdated.dwg User:DouangvilayS Oct 12, 2016 - 9:37am

Notes:

1. Lagoon water surface elevations obtained from Stantec report (2010).



KEY

— — — Modeled Phreatic Surface Calibrated with Piezometer Data

▼ Average Subsurface Water Elevation from Stantec Monitoring (9/2009–1/2010)

▽ Average Subsurface Water Elevation Estimated from CPT Dissipation Test (10/20/2015)

0 20' 40'

GRAPHIC SCALE

AECOM

PARADISE FOSSIL PLANT

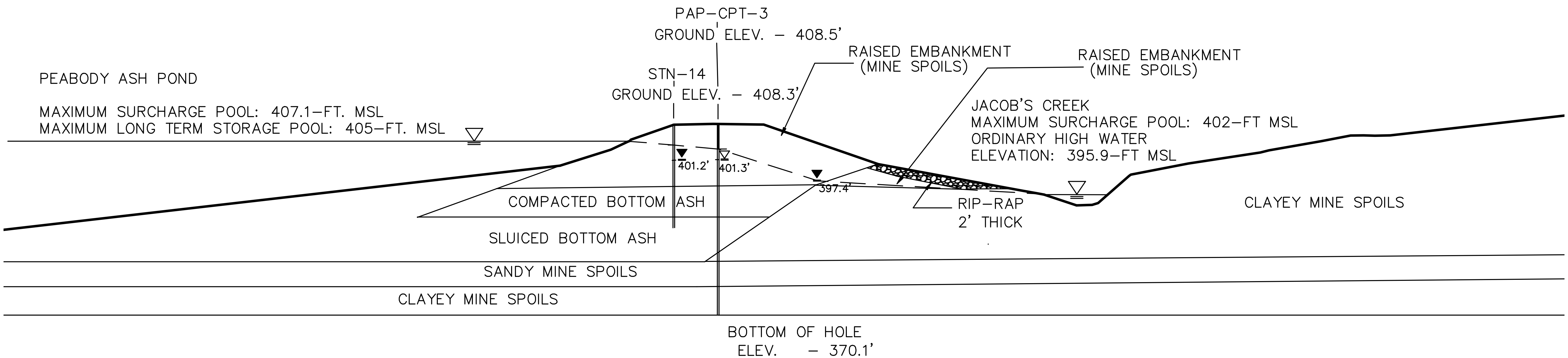
TENNESSEE VALLEY AUTHORITY

PEABODY ASH POND
CROSS SECTION A-A'

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NC	MW	60442564	09/16/16	2

Notes:

1. Jacob's Creek Elevations are ordinary high water elevations provided in Peabody Ash Pond Spillway and Scope Improvements drawings completed by TVA (5/16/2014).



KEY

— — — Modeled Phreatic Surface Calibrated with Piezometer Data

▼ Average Subsurface Water Elevation from Stantec Monitoring (9/2009–1/2010)

▽ Average Subsurface Water Elevation Estimated from CPT Dissipation Test (10/20/2015)

0 20' 40'

GRAPHIC SCALE

AECOM

PARADISE FOSSIL PLANT

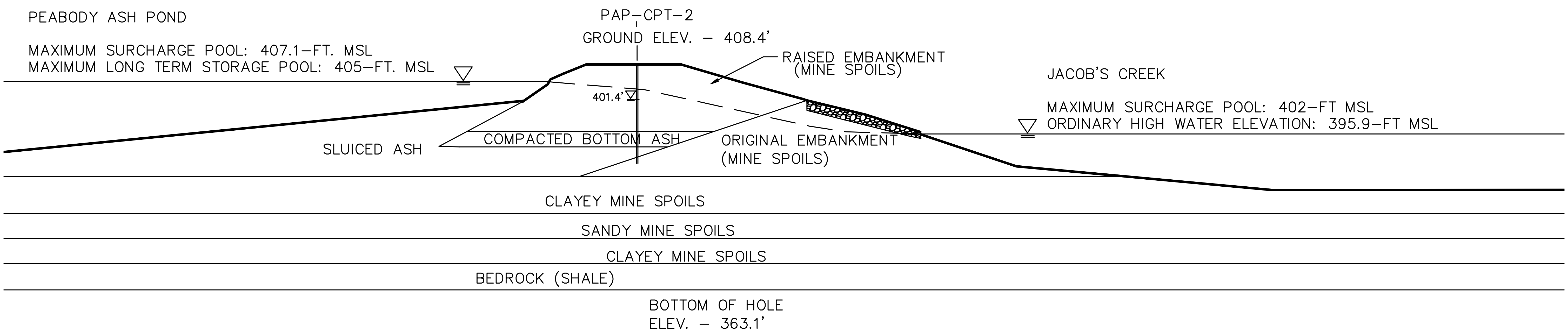
TENNESSEE VALLEY AUTHORITY

PEABODY ASH POND
CROSS SECTION B-B'

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NC	MW	60442564	09/16/16	3

Notes:

1. Jacob's Creek Elevations are ordinary high water elevations provided in Peabody Ash Pond Spillway and Scope Improvements drawings completed by TVA (5/16/2014).



KEY

— — — Modeled Phreatic Surface Calibrated with Piezometer Data

▽ Average Subsurface Water Elevation Estimated from CPT Dissipation Test (10/20/2015)

0 20' 40'

GRAPHIC SCALE

AECOM

PARADISE FOSSIL PLANT

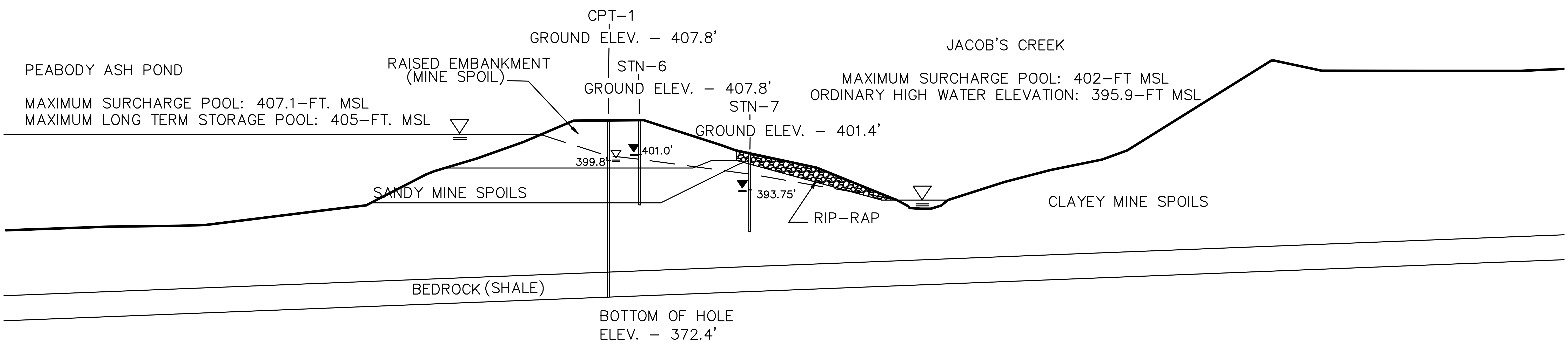
TENNESSEE VALLEY AUTHORITY

PEABODY ASH POND
CROSS SECTION C-C'

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NC	MW	60442564	09/16/16	4

Notes:

1. Jacob's Creek Elevations are ordinary high water elevations provided in Peabody Ash Pond Spillway and Scope Improvements drawings completed by TVA (5/16/2014).



KEY

— — — Modeled Phreatic Surface Calibrated with Piezometer Data

▼ Average Subsurface Water Elevation from Stantec Monitoring (9/2009–1/2010)

▽ Average Subsurface Water Elevation Estimated from CPT Dissipation Test (10/20/2015)

0 20' 40'

GRAPHIC SCALE

AECOM

PARADISE FOSSIL PLANT

TENNESSEE VALLEY AUTHORITY

PEABODY ASH POND
CROSS SECTION D-D'

DRAWN BY:	CHECKED BY:	PROJECT No:	DATE:	EXHIBIT
NC	MW	60442564	09/16/16	5

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