AECOM
564 White Pond Drive
Akron, OH 44320-1100
www.aecom.com

330-836-9111 tel
330-836-9115 fax

October 12, 2016

Tennessee Valley Authority
1101 Market Street
Chattanooga, Tennessee 37402

**Initial Structural Stability Assessment**
**Slag Ponds 2A and 2B, Slag Stilling Pond 2C**
**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 Slag Ponds 2A and 2B, and Slag Stilling Pond 2C. Based on this assessment, Slag Ponds 2A and 2B, and Slag Stilling Pond 2C are 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 Slag Ponds 2A and 2B, and Slag Stilling Pond 2C have 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 Slag Ponds 2A and 2B, and Slag Stilling Pond 2C. The results show that the impoundment meets the structural stability requirements set forth in 40 CFR 257.73(d)(1)-(2).

Remainder of page intentionally left blank.

## 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 Slag Ponds 2A and 2B, and Slag Stilling Pond 2C meets the requirements specified in 40 CFR 257.73(d)(1)-(2).

SIGNATURE ______________________ DATE 10/12/16

ADDRESS: AECOM
564 White Pond Drive
Akron, OH 44320
TELEPHONE: (330) 936-9111

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

STATE OF KENTUCKY
NICHOLAS
GOLDEN
30816
LICENSED
PROFESSIONAL ENGINEER

COAL COMBUSTION PRODUCT DISPOSAL PROGRAM

TENNESSEE VALLEY AUTHORITY – SLAG PONDS 2A AND 2B,
AND SLAG STILLING POND 2C
TVA PARADISE FOSSIL PLANT
DRAKESBORO, KENTUCKY

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

## EXISTING CCR SURFACE IMPOUNDMENTS

Prepared for

TVA

Tennessee Valley Authority
1101 Market Street
Chattanooga, TN 37402-2801

STATE OF KENTUCKY
NICHOLAS
GOLDEN
30816
LICENSED
PROFESSIONAL ENGINEER

10/12/16

October 12, 2016

Prepared by

AECOM

## Table of Contents

**1.0 Project Background** ........................................ **1**

**2.0 Structural Stability Assessment - §257.73(d)(1)** ........................................ **1**

2.1 Foundations and Abutments - §257.73(d)(1)(i) ........................................ 2
2.2 Slope Protection - §257.73(d)(1)(ii) ........................................ 5
2.3 Embankment Dike Compaction - §257.73(d)(1)(iii) ........................................ 6
2.4 Vegetated Slopes - §257.73(d)(1)(iv) ........................................ 6
2.5 Spillway Condition and Capacity - §257.73(d)(1)(v) ........................................ 7
2.6 Spillway Structural Integrity - §257.73(d)(1)(vi) ........................................ 8
2.6.1 Site Inspection and Findings ........................................ 8
2.6.2 Structural Assessment ........................................ 9
2.7 Sudden Drawdown Assessment - §257.73(d)(1)(vii) ........................................ 9

**3.0 Conclusion** ........................................ **9**

**4.0 References** ........................................ **10**

## TABLES

Table 1: Summary of Seepage Parameters

## FIGURES

Figure 1 – Site Location Map

## PHOTOS

Photo 1 – Slag Pond 2A and 2B splitter dike condition, west side
Photo 2 – Slag Pond 2A and 2B splitter dike condition, top
Photo 3 – Slag Pond 2A and 2B splitter dike condition, east side
Photo 4 – Slag Pond 2A 48” spillway inlet
Photo 5 – Slag Pond 2B dike east dike condition
Photo 6 – Slag Pond 2B flume spillway inlet
Photo 7 – Slag Pond 2B and Slag Stilling Pond 2C flume spillway
Photo 8 – Slag Stilling Pond 2C flume spillway outlet
Photo 9 – Slag Stilling Pond 2C Emergency Spillway
Photo 10 – Slag Stilling Pond 2C dike conditions
Photo 11 - Slag Stilling Pond 2C dike conditions
Photo 12 - Slag Stilling Pond 2C spillway skimmer, north
Photo 13 - Slag Stilling Pond 2C spillway skimmer, middle
Photo 14 - Slag Stilling Pond 2C spillway skimmer, south
Photo 15 - Slag Stilling Pond 2C spillway outlets
Photo 16 – Flow inside 36” concrete pipe (Typ.)

## APPENDICES

Appendix A: Hydraulic Structures Assessment Calculation Package

Appendix B: Geotechnical Exploration and Analysis Cross Sections and Boring Location

## 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 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.

Slag Ponds 2A and 2B, and Slag Stilling Pond 2C collectively meet both criteria. The location of Slag Ponds 2A and 2B, and Slag Stilling Pond 2C is shown in **Figure 1**.

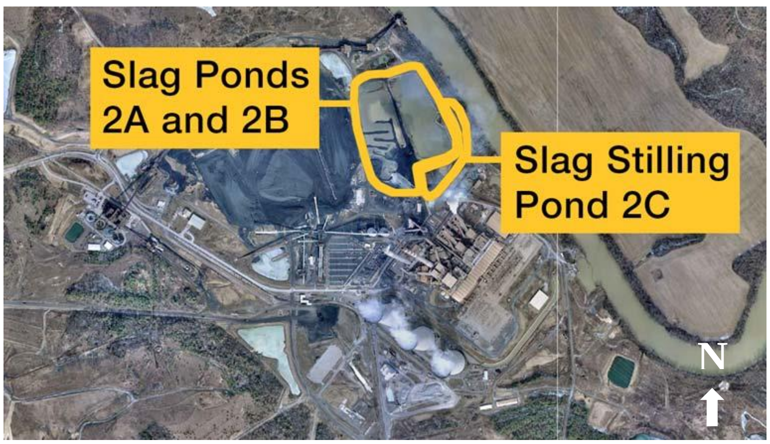



**Figure 1: Site Location Map**

## 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 the area encompassing Slag Ponds 2A and 2B and Slag Stilling Pond 2C includes alluvial deposits underlain by Pennsylvanian age bedrock formations. Geologic mapping indicates the site is primarily underlain by two geologic formations of Pennsylvanian age, the Carbondale and Shelburn Formations. Both formations generally consist of sandstone, which weathers to a dense sand. The Shelburn Formation underlies the Carbondale Formation. Underlying the Shelburn sandstone is a shale unit that is typically light-gray to black, and carbonaceous. Coal underlies the shale unit.

The foundation of the perimeter dike consists of mine spoils. These materials are previously excavated overburden removed during the strip mining process and then replaced following removal of coal deposits. Accordingly, mine spoils tend to be heterogeneous. The majority of the mine spoils sampled consists of very stiff lean clay (CL), clayey sand, (SC), or clayey gravel (GC) with varying quantities of gravel sized rock fragments. It should be noted that the USCS classification is almost entirely CL when the gradations are corrected for gravel content.

In 2015, an Initial Annual (Intermediate) Inspection was performed for Slag Ponds 2A and 2B, and Slag Stilling Pond 2C was completed. Based on the inspection report, no evidence of actual or potential structural weakness of the inspected units was observed. No changes that may have affected the operational stability of the impounding structure were 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 initially determined from laboratory testing, while anisotropic ratios were determined 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 site specific vibrating wire piezometer data. The calibration process was performed until parameters were determined which yielded a reasonable correlation to field readings. Six stability cross sections were constructed based on the data developed.

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) | *$K_h/K_v$ |
|---|---|---|
| Minespoil Fill | $3 \times 10^{-6}$ | 4 |
| Clayey Alluvium | $6 \times 10^{-6}$ | 4 |
| Sandy Alluvium | $3 \times 10^{-4}$ | 2 |
| Residuum | $7 \times 10^{-7}$ | 4 |

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

Generally, the final adjusted values were within an order of magnitude of the laboratory test data.

As part of the seepage analysis at each cross section, horizontal and vertical gradients can 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 results from an evaluation of 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 ($\gamma_b$) to the unit weight of water ($\gamma$w).The factor of safety (FS) with respect to piping is generally defined as the ratio of the critical gradient ($I_c$) to the predicted or measured exit gradient ($I_e$) determined as described above 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 modeled phreatic surface at the cross sections constructed indicated flow through embankments with exit gradients at the toe being 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."

The embankment of the dike is constructed of predominantly clayey minespoil materials, and the foundation materials of the dike where seepage exit flow is expected consists of clayey alluvium.

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.

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.

As discussed, the embankment of the dike is constructed of predominantly clayey minespoil materials, and the foundation materials of the dike where seepage is present consists of clayey alluvium. A sandy alluvial layer approximately 5 feet in thickness is present at a depth of 20 feet below the clayey alluvium layer at the downstream toe of the embankment.

Seepage analyses assumed steady state conditions at normal pool conditions. However, if the downstream waterbody 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 a sand layer would not exceed the pore water pressure at the ground surface, and that the elevation of the downstream water body does not 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 the cross sections was limited in thickness to 5 feet, whereas it is understood that the classical model for heaving involves a pervious later of much greater thickness. In addition, as stated in the USBR Design Standard No. 13 Embankment Dams, Chapter 8, January, 2014, page 8-9 through 8-10:

> *"An additional factor not explicitly accounted for in uplift computations is the actual shear strength or cohesion of the confining layer, particularly if the layer is clayey. For high seepage uplift pressures to cause a rupture in (or even to lift) a clay layer likely takes more than simply exceeding the weight of the layer; those pressures must also overcome the cohesive strength of the clay layer. Lacking any meaningful or efficient way of accounting for this factor, most evaluations simply discount the benefit of this factor in reducing the potential for an uplift failure. Given these two considerations, uplift computations may under predict the actual the actual factor of safety against uplift of a cohesive confining layer."*

Consolidated undrained triaxial testing performed indicates significant cohesion of the clayey alluvial layer. Therefore, in excluding cohesive strength, the calculated factors of safety against uplift likely under-predict the actual factor of safety for the Slag Pond dike. Seepage conditions have been analyzed 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 based on 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)

Slag Ponds 2A and 2B, and Slag Stilling Pond 2C share the same perimeter dike and are separated by two interior dikes. The dikes share similar characteristics in that they have similar designed dimensions and were constructed with compacted mine spoils. The dike crests range in elevation were the dikes of 2A is generally higher and the dikes of 2C tend to be lower. The dike slopes are generally no steeper than 3H:1V. The top of the dikes serve as access roads, covered with either gravel or bottom ash. The dike sides are generally covered with grassy vegetation and riprap along shorelines and outlet structures.

In 2015, an Initial Annual (Intermediate) Inspection was performed for Slag Ponds 2A and 2B, and Slag Stilling Pond 2C. Based on the inspection 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.

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 Initial Annual (Intermediate) Inspection and the attached **Photos**.

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

Slag Ponds 2A and 2B, and Slag Stilling Pond 2C all share the same perimeter dike from the original construction of Ash Disposal Area 2, which was built in 1963 when the plant opened. Construction documents indicate that the original 1963 perimeter dike was mechanically compacted. During construction of the original dike, all trees were cleared in the area and the embankments were constructed of unclassified material placed in layers 12 ± inches thick and compacted by hauling equipment.

According to the construction documents, in the 1970 nearly all of the perimeter dikes were relocated and the internal dikes were raised, increasing the capacity of the impoundments. During the 1970 construction of the raised and relocated dikes, all existing weak surface soils were to be removed and proof-rolled with loaded hauling equipment to ensure adequate bearing capability before placement of new material. Dike cores were to be constructed with 6-inch layers of impervious fill and compacted with sheepsfoot rollers. Dike shells were to consist of strip mine spoils being compacted in layers. Test fills were to be performed prior to construction to determine the best possible layered thicknesses.

A new perimeter dike was constructed diagonally through the middle of the Settling Pond (Slag Stilling Pond 2C), sometime before 1984. This dike closed off the northern section of the pond. The dikes were built with a 16 foot flat surface on top and side slopes of 3:1. The top of the dikes serve as access roads made of gravel or bottom ash. The dike slopes are primarily grassy vegetation with riprap at outlets and shorelines.

More information on the construction of the Slag Ponds 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 perimeter and divider dikes consist of mostly well-maintained grassy vegetation and riprap. In Slag Pond 2A the outer dike consists primarily of grassy vegetation. Slag Pond 2B's perimeter dike consists mostly of riprap. Slag Stilling Pond 2C consists mostly of grassy vegetation with some riprap at the immediate vicinity of the concrete flume spillway.

In 2015, an Initial Annual (Intermediate) Inspection was performed for Slag Ponds 2A and 2B, and Slag Stilling Pond 2C at TVA's Paradise Fossil Plant. Based on the inspection 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 Slag Ponds 2A and 2B, and Slag Stilling Pond 2C is approximately 60 acres. The drainage area is west and south of the ponds.

Boiler slag is sluiced into the south of Slag Pond 2A. Flow from Slag Pond 2A travels to Slag Pond 2B through two 48-inch culverts and one 60-inch culvert, all three of which penetrate through the internal divider dike. Flow is then directed through a series of baffles and then exits into Slag Stilling Pond 2C to the east through a 38-foot wide concrete flume. At the south end of Slag Stilling Pond 2C, sluice water is decanted into three spillway devices which discharge through a permitted NDES outfall into the Green River. The spillways consist of 36” slip-lined culvert pipes with 48” concrete vertical risers topped by skimmer devices made from 5 foot sections of 120-inch diameter galvanized corrugated metal pipe with interior bracing. According to construction documents, the respective lengths of the 36” spillway concrete pipes, north to south, are 201 feet, 182 feet and 192 feet. An emergency spillway is located on the east side of Slag Stilling Pond 2C, consisting of grouted rip rap. The primary and emergency spillways are constructed with non-erosive material to adequately protect against water erosion.

More information on existing spillway structure conditions 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 Slag Ponds 2A and 2B, and Slag Stilling Pond 2C. This analysis was performed for 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 Slag Ponds 2A and 2B, and Slag Stilling Pond 2C during the Inflow Design Flood (IDF). The required IDF used in the model calculations is based on each pond’s hazard classification. Since Slag Ponds 2A and 2B, and Slag Stilling Pond 2C were classified as significant hazards, 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 Slag Ponds 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 model results indicate the ponds would not overtop during a 1000-year design storm. The freeboard for Slag Ponds 2A and 2B, and Slag Stilling Pond 2C during this storm event is acceptable.

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)

Water discharges into the Green River through the east perimeter dike of the Slag Pond Complex with three spillways located in Slag Stilling Pond 2C. The spillways contain 48" concrete vertical risers, each topped with a skimmer device. The skimmer device is composed of a 5 ft. section of 120" dia. galvanized corrugated metal pipe with interior steel angles as bracing. Water flows west to east via three 36" Class IV concrete pipes that extend from a concrete junction box and discharge into the Green River. The respective lengths of the spillway concrete pipes, north to south are 201 feet, 182 feet and 192 feet. 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 junction box having 1 foot thick walls measuring 6 feet x 6 feet x 4 feet.

Refer to 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 Slag Stilling Pond 2C. The north riser was accessed via the adjacent boardwalk. The others were viewed from the pond shore. The evaluation also included viewing of available historical inspection videos depicting the condition inside the concrete pipes and risers.

The three 48" concrete risers are similar in this area, and a skimmer device is attached to the top of each riser. The skimmer devices are open at the top. At the inlet zone, in-depth inspection was restricted due to the current water level and the risers' distance from the walkway/shore. At the outlet area, the fence encompassing all of the outlet area prevented an in-depth inspection. The level of bottom ash sediment in the pond was determined by means of probing a rod around the skimmer devices. The accumulation measured starts at approximately 42 inches from the water surface. There was no deterioration discovered at the base of the corrugated metal pipe skimmers. The observable areas of the outer concrete risers are in good condition. Cracking, deterioration or spalling was not detected. Surface rusts is present on the 120" dia. corrugated metal pipes and associated steel bracings. There are no apparent deteriorations or obstructions to water flow from the top of the risers. The water level was measured in the field at roughly 7" above the top of the north riser #1 (nearest to walkway). **Photos 12-15** show the typical condition of the risers.

Due to the fenced area, the outer surface of the outlet pipes couldn't be evaluated. Videos of the pipes' interior were available and evaluated in the next section. **Photo 16** depicts the typical inside condition at the outlet.

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 17** was taken from the available videos and represents the conditions described above. The inside of the spillway pipes have been lined since the time of the recordings.

### 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 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 1000 year flood event. Evaluation for this flood event is required for a significant hazard potential unit per TVA-CCR Rule Template 257.73(d). It has been determined that the 1000 year flood event will not likely overtop the pond, so there will not be a flow velocity on the side of the riser structures. It was thus decided to determine the flow velocity for which the bearing capacity check, sliding check and moment equilibrium are satisfied, for informational purposes. An additional check was included to confirm that the risers do not tip off the base under the specified flow velocity. 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 Assessment - §257.73(d)(1)(vii)

Sudden drawdown effects between the slag ponds themselves was determined negligible. In the case that interior dike failure occurred due to sudden drawdown, there would be no release of material since they share the same perimeter dike. Additionally, the 100-year floodplain of the Green River does not reach the toe of the downstream dike of Slag Ponds 2A and 2B, and Slag Stilling Pond 2C.

# 3.0 Conclusion

Based on the initial structural stability assessment, the requirements of **Rule §257.73(d)(1)** for the Slag Ponds 2A and 2B, and Slag Stilling Pond 2C have been met.

# Photos


AECOM

**TVA PARADISE FOSSIL PLANT**

PARADISE, KENTUCKY

SITE LOCATION MAP
SLAG PONDS 2A & 2B;
SLAG STILLING POND 2C

**Photo 1 – Slag Pond 2A & 2B splitter dike condition, west side**

**Photo 2 –Slag Pond 2A & 2B splitter dike condition, top**

**Photo 3 –Slag Pond 2A & 2B splitter dike condition, east side**

**Photo 4 – Slag Pond 2A 48" spillway inlets**

**Photo 5 –Slag Pond 2B dike east dike condition**

**Photo 6 – Slag Pond 2B flume spillway inlet**

**Photo 7 – Slag Pond 2B and Slag Stilling Pond 2C flume spillway**

**Photo 8 – Slag Stilling Pond 2C flume spillway outlet**

**Photo 9 – Slag Stilling Pond 2C Emergency Spillway**

**Photo 10 –Slag Stilling Pond 2C dike conditions**

**Photo 11 –Slag Stilling Pond 2C dike conditions**

**Photo 12 – Slag Stilling Pond 2C spillway skimmer, north**

**Photo 13 – Slag Stilling Pond 2C spillway skimmer, middle**

**Photo 14 – Slag Stilling Pond 2C spillway skimmer, south**

**Photo 15 – Slag Stilling Pond 2C spillway outlets**

**Photo 16 – Flow inside 36” concrete pope (Typ.)**

# APPENDIX A

## HYDRAULIC STRUCTURES ASSESSMENT CALCULATION PACKAGE

AECOM

# 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 (1970 to 1978), AECOM geotechnical data report for Slag Ponds (2016), historical data and engineering judgment.

**i. Foundation Soil**

| | Symbol | Value | Units | Reference /Equation |
|---|---|---|---|---|
| Friction Angle | Φ | 32 | degrees | Per AECOM Geotechnical Data Report (2016) – Residual Clays |
| | | 0.56 | radians | |
| Unit Weight | | 123 | pcf | Per AECOM Geotechnical Data Report (2016) – Residual Clays |
| Apparent Cohesion of Foundation | c | 0.00 | pcf | Neglected Per TVA CCR 257.73d - 2.1.5 |
| Ultimate Bearing Capacity | $q_{ult}$ | 5500 | psf | Per AECOM Geotechnical Data Report (2016) – Residual Clays |

**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 A |
| 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 | $\Upsilon_c$ | 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 | $\rho_w$ | 1.94 | slugs/ft$^3$ | |
| Unit Weight | $\gamma_{water}$ | 62.4 | pcf | |

**v. Sluiced Ash**

| | Symbol | Value | Units | Equation |
|---|---|---|---|---|
| Saturated Density | $\rho_{ash}$ | 3.106 | slugs/ft$^3$ | |
| Saturated Unit Weight | $\gamma_{ash}$ | 100 | pcf | Per AECOM Geotechnical Data Report (2016) |

**b. Geometry**

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

| Date | Drawing No. | Description |
|---|---|---|
| 13-Jul-70 | 10N3203 R6 | General Plan and Sections |
| 4-Nov-77 | 10W3267-2 R1 | Ash Disposal Area 2A & 2B Section and Details |
| 21-Feb-78 | 10W3214 R6 | SpillwaysType A & B |
| 30-Jul-93 | 10W3257-4 | Weir & Skimmer Details |

| **References** | |
|---|---|
| 1. | Per Existing Drawing Notes, Concrete is Class "A". Typical compressive strength of concrete range 3000 - 6000psi, use 3,000 psi |
| 2. | Per Existing Drawing, Riser Invert Elevation is 398.00 ft. |
| 3. | Bottom of Base Elevation is 396.5ft |
| 4. | Water Surface Elevation per HydroCAD Model is 406.5 ft |

AECOM

# Appendix B - Hydraulic Structures Assessment Calculation Package

## Geometric Properties of Riser

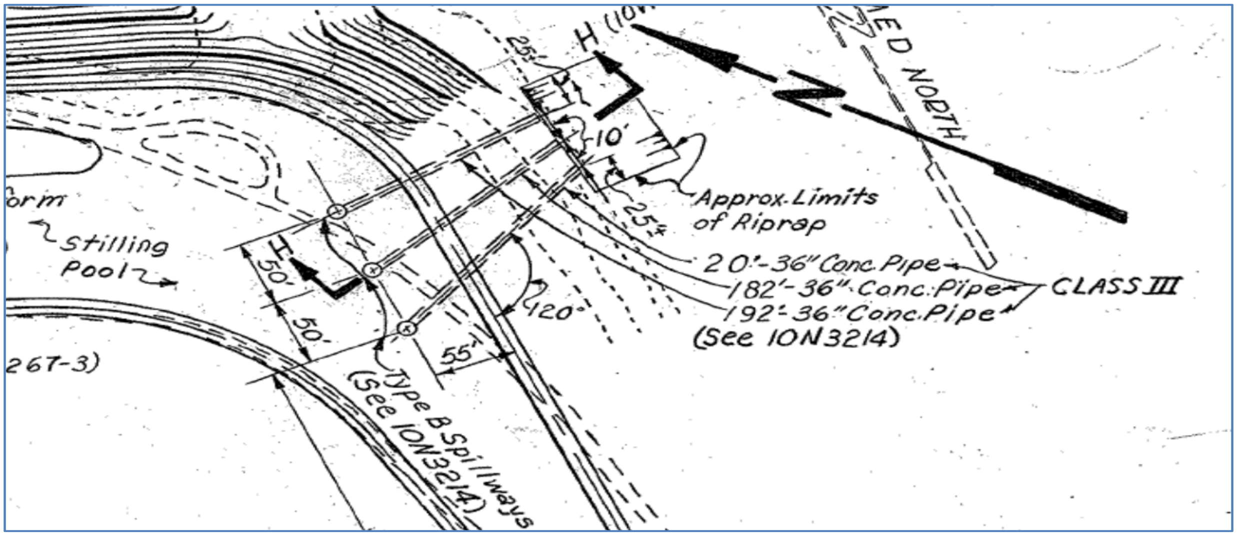
| | Symbol | Value | Units | Reference |
|---|---|---|---|---|
| Top Elevation of Structure | $EL_{TOP}$ | 406.5 | ft | Exist. Water Surface Elev. |
| Bottom Elevation of Structure (Elevation to Bottom of Base) | $EL_{BOT}$ | 396.5 | ft | Dwg # 10W3267-2 R1 |
| Total Structure Height | H | 9.50 | ft | Water level is 6" above riser |
| Height of Water inside (from bottom of riser) | $H_i$ | 0.00 | ft | Critical for Floatation |
| Height of Ash outside (from bottom of base) | $H_0$ | 6.50 | ft | |
| Height of Water outside (from Bottom Ash) | $H_0$ | 3.50 | ft | Field Measurements |
| **Cross section Shape: Round or Rectangular** | Round (Top) Square Box ( Bottom) | | | |

| Cross Sectional Properties of the Bottom Concrete Box Riser (Square) | | | | |
|---|---|---|---|---|
| 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 | |

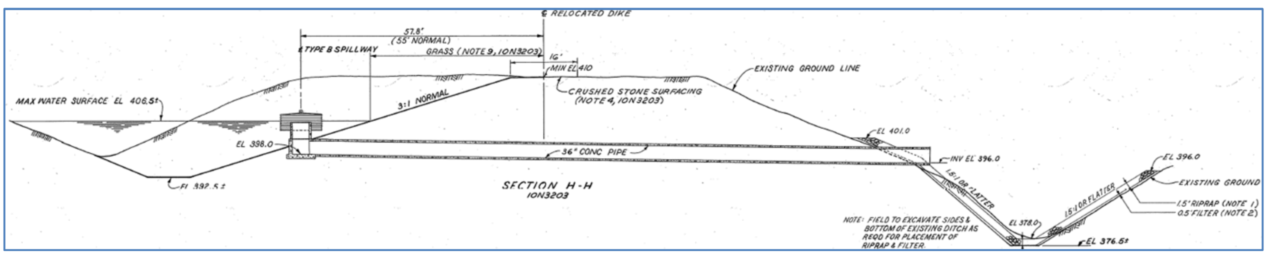
| Cross Sectional Properties of the Top of Riser (Circular) | | | | |
|---|---|---|---|---|
| External Radius of Riser | $r_0$ | 2.33 | ft | Dwg # 10W3214 R6 |
| Internal Radius of Riser | $r_i$ | 2 | ft | |
| Weir & Skimmer Weight at Top | $W_{TOP}$ | 2 | kips | See calcs |
| | | | | |
| **Foundation Properties** | | | | |
| Width of Footing | B | 6.50 | ft | Dwg # 10W3214 R6 |
| Length of Footing | L | 6.50 | ft | |
| Depth of Footing | $D_{footing}$ | 1.5 | ft | |

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

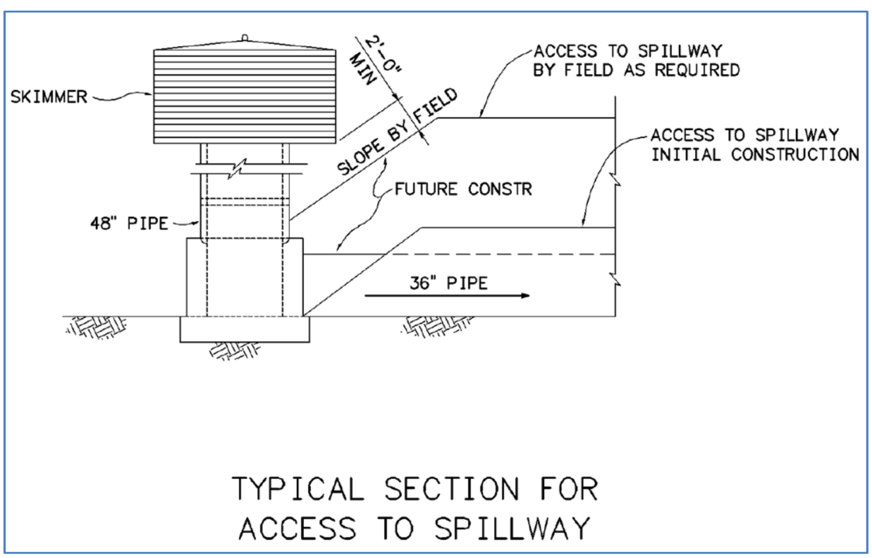
## Existing Drawings




**Plan and Elevation**




**Section**



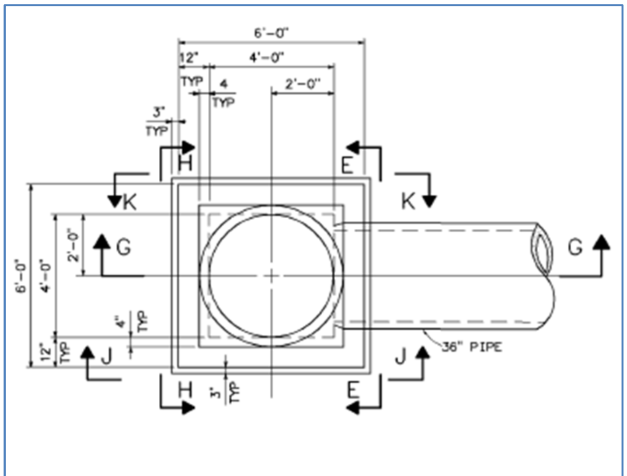

Skimmer

AECOM

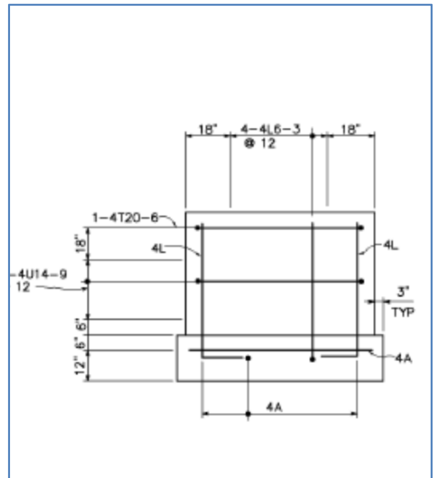
# Appendix B - Hydraulic Structures Assessment Calculation Package

## Existing Drawings

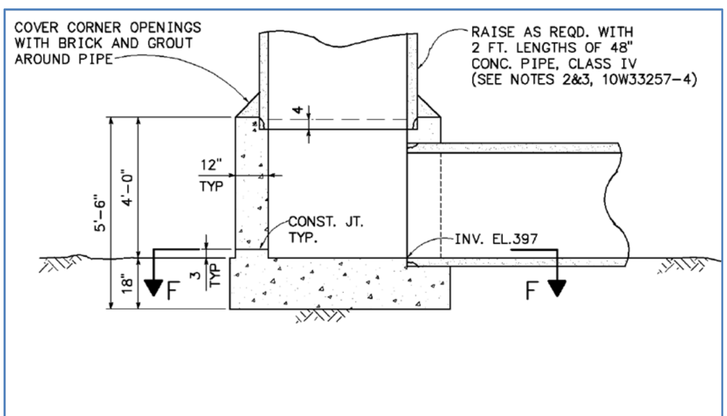
Spillway Typical Sections



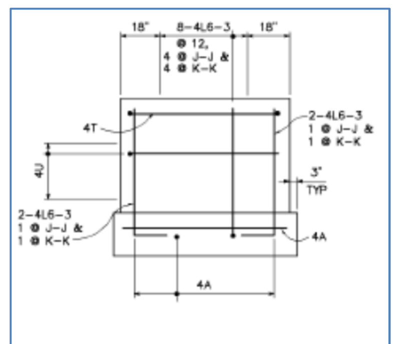

PLAN–SPILLWAY TYPE B




H–H

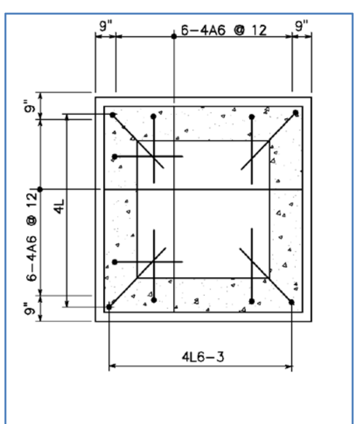



G–G




J–J & K–K

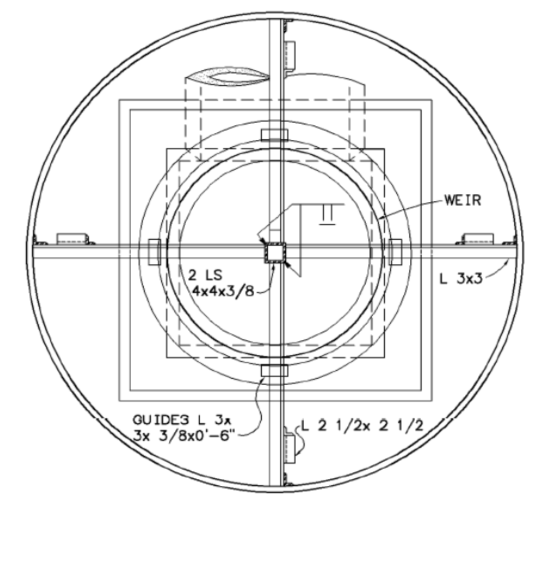
K–K OPPOSITE HAND



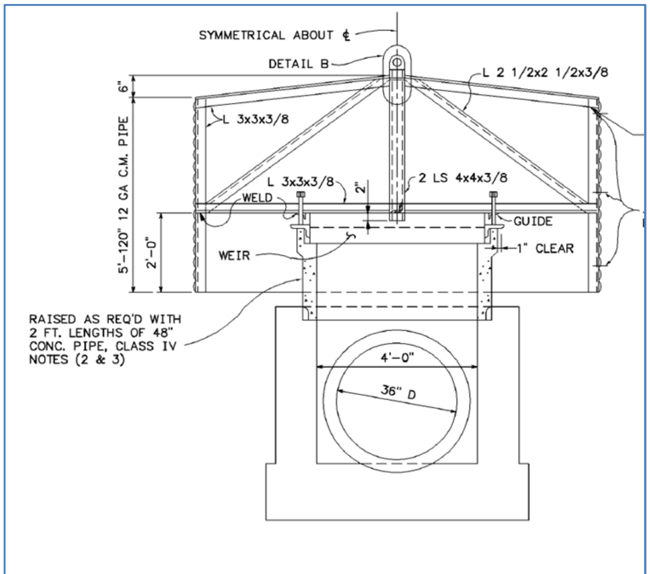

F–F

## Existing Drawings

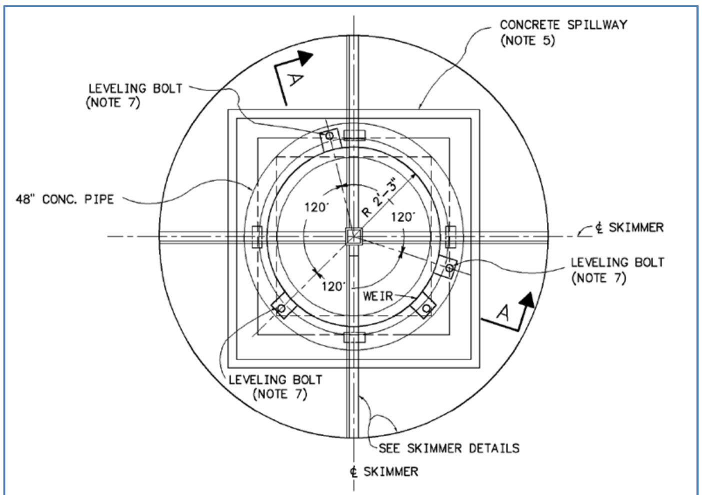
Skimmer Details



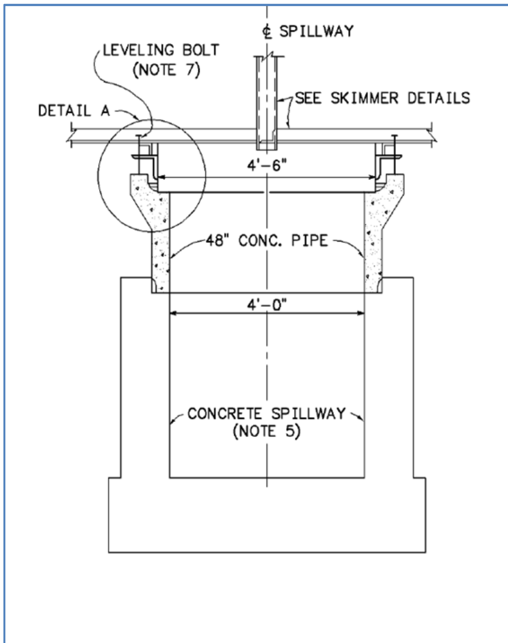

SECTIONAL PLAN




SECTIONAL ELEVATION

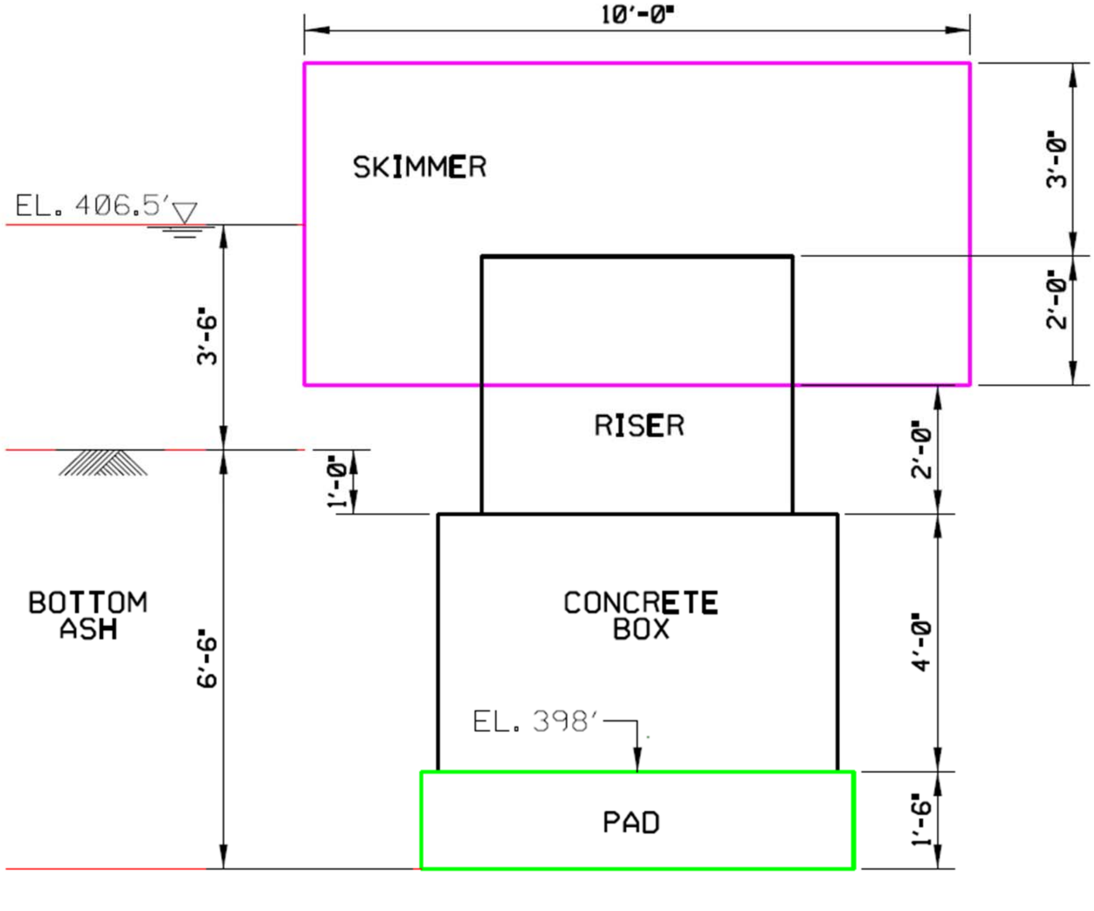



PLAN




SECTION A–A

## Field Observation




RISER
(FIELD OBSERVATION)

## c. Limit States

### i. Regularly Occuring Rerservoir 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. 10N3203 R6, 10W3214 R6 & 10W3267-2 R1

Geotechnical Data Report prepared by URS date January 2. 2013

| | | | |
|---|---|---|---|
| Structure Type | Normal | | |
| Bottom Ash from Base | 6.50 | ft | |
| Height of water above Bottom Ash | 3.50 | ft | Field Measurement |
| Height of Bottom Ash from Conc. Box | 1.00 | ft | |

**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) | See Weir and Skimmer Calcs |
| Weir and Skimmer | | | 2.00 | |
| 48" Riser Pipe | 4.54 | 4.00 | 2.72 | added Sections of 2ft |
| 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 Displaced | | | |
|---|---|---|---|
| | Area ($ft^2$) | water 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 | |
| Weight of Ash on Conc box and Base | 0 | conservative |

Floatation Check:

@ bottom of foundation 1.52 OK

**Bearing Capacity**

| | | | |
|---|---|---|---|
| Structure Type | Normal | | |
| Load Condition Category | Usual | | |
| Bearing Pressure: $Q_{all}$ = | 5500 | $lbs/ft^2$ | Soft Clay & Silt |
| 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 | |
| Bottom Ash on Conc. Box | 18.90 | 1.00 | 2.07 | 1ft of Bottom Ash on Conc. Box |
| Bottom Ash on Base (3 sides) | 4.63 | 5.00 | 2.53 | 5ft of Bottom Ash on Base |
| Weight of Structure : $W_S$ = | | | 30.83 | |

| 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 = 898.37 $lbs/ft^2$

**Bearing Capacity Check:**

@ bottom of foundation 6.12 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

| Material | Quantity | Unit weight | Thickness (in) | Height (in) | Length (ft) | Weight (lbs) |
|---|---|---|---|---|---|---|
| **Weir** | | | | | | |
| 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. 10N3203 R6, 10W3214 R6 & 10W3267-2 R1

Geotechnical Data Report prepared by URS date January 2. 2013

| | |
|---|---|
| Structure Type | Normal |
| Load Condition Category | Unusual |

**Floatation Stability**

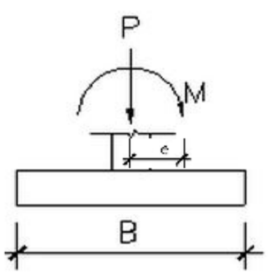
The riser will be completely submerged during this flood event. Floatation will not be an issue for this event.

| Sliding Friction | | | | | |
|---|---|---|---|---|---|
| Angle of internal friction | Φ | 32 | 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 | 37.96 | 17.21 | 12.966 | * 4.06 | No Sliding | 3.19 | 1.5 |
| | | | | | | | | Pass |

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

| Loads | | | |
|---|---|---|---|
| Dead Load at Base | DL | 37.96 | kips |
| Bouyancy Force at Base | Fb | 17.21 | kips |
| Axial Load at Base | P = DL-Fb | 20.75 | 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 in Compresion**

| | | |
|---|---|---|
| Resultant | e=M/P | 1.22 |
| | B/6 | 1.08 |
| | L/6 | 1.08 |
| | e>(B/6) | Yes |
| Resultant | | 2.03 |
| % of Base in Compression | | 93.52% |

| | | | | |
|---|---|---|---|---|
| Maximum Bearing Pressure when e > L/6 | $Q_{max}$ | 2P/[3(B/2-e)L] | 1.050368 | ksf |
| Maximum Bearing Pressure when e<L/6 | $Q_{max}$ | P/A+M/s | 1.045955 | ksf |

| | | | | | |
|---|---|---|---|---|---|
| $q_{ult}$ | 5.5 | > | $Q_{max}$ | 1.05 | ksf |

| | | |
|---|---|---|
| FOS | 5.24 | |
| UNLC FOS | 2.6 | Passes |

**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 Strength | $f_c$ | 3000 | psi |
| Concrete Tensile Strength | $f_t$ | 353.61 | psi |
| Concrete Shear Strength | .1*fc | 300 | psi |
| Φ = | | 0.55 | Plain Concrete |

Hydrostatic force is equal all around the Riser

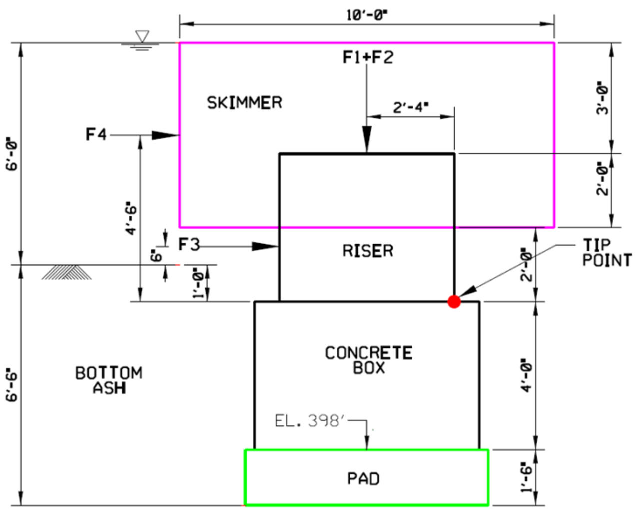
| Hydrodynamic Force | | | | |
|---|---|---|---|---|
| Drag Coefficient | $C_d$ | 1.25 | USACE - Drag Coefficient shall not be less than 1.25 | |
| Velocity of Flood water | V | 6.00 | ft/s | Assume Max For 1000 yr Flood - conservative |
| Mass Density of water | $\rho_w$ | 1.94 | $slugs/ft^3$ | |
| Flood Elevation | H | 12.50 | ft | Assume skimmer submerged (distance above Bottom Ash) |
| 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 | |

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 -Strength Capacity, in accordance with ACI 350 Code | | Use for plain concrete |
|---|---|---|
| ACI 350 -Capacity Reduction Factor | | |
| Shear | 0.75 | 0.55 |
| Bending | 0.9 | 0.55 |

| Factored Load | |
|---|---|
| Loads due to Fluids | 1.4 |

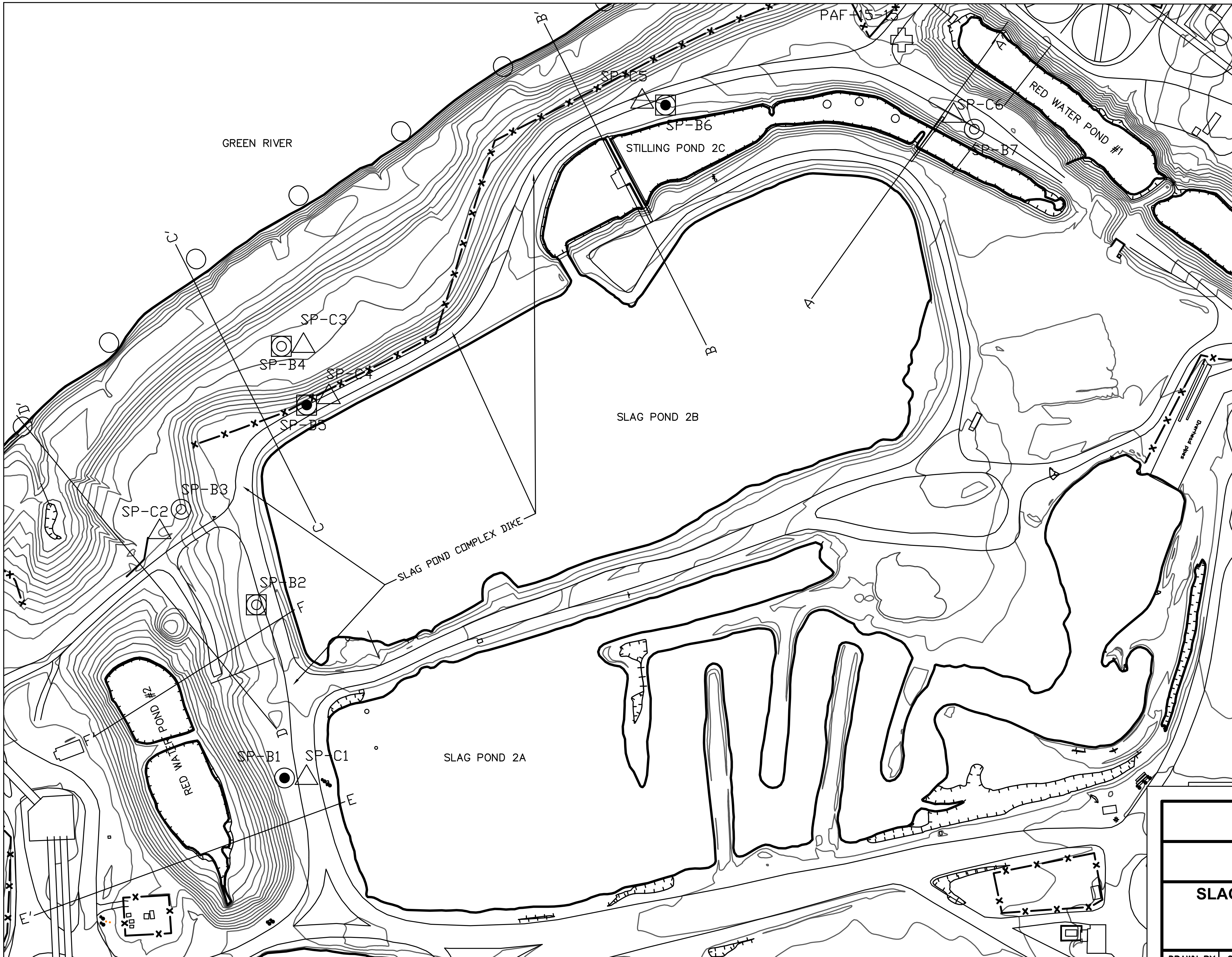
**Strength Calculation - (1000 yr Flood - Significant Hazard Potential Unit)**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 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




N

0 75 150
SCALE IN FEET

## LEGEND

| | | |
|---|---|---|
| SP-BX | O | URS Soil Boring |
| SP-BX | ◉ | URS Soil Boring with Rock Core |
| SP-BX | ▣ | URS Soil Boring with Piezometer |
| SP-CX | △ | URS Cone Penetration Test Sounding |
| PAF-15-15 | ✚ | URS Environmental Monitoring Well |

S:\TVA CCP\PAF\slag pond\Analysis\PAF Slag Pond Section Cuts.dwg User:DouangvilayS Oct 11, 2016 - 3:17pm

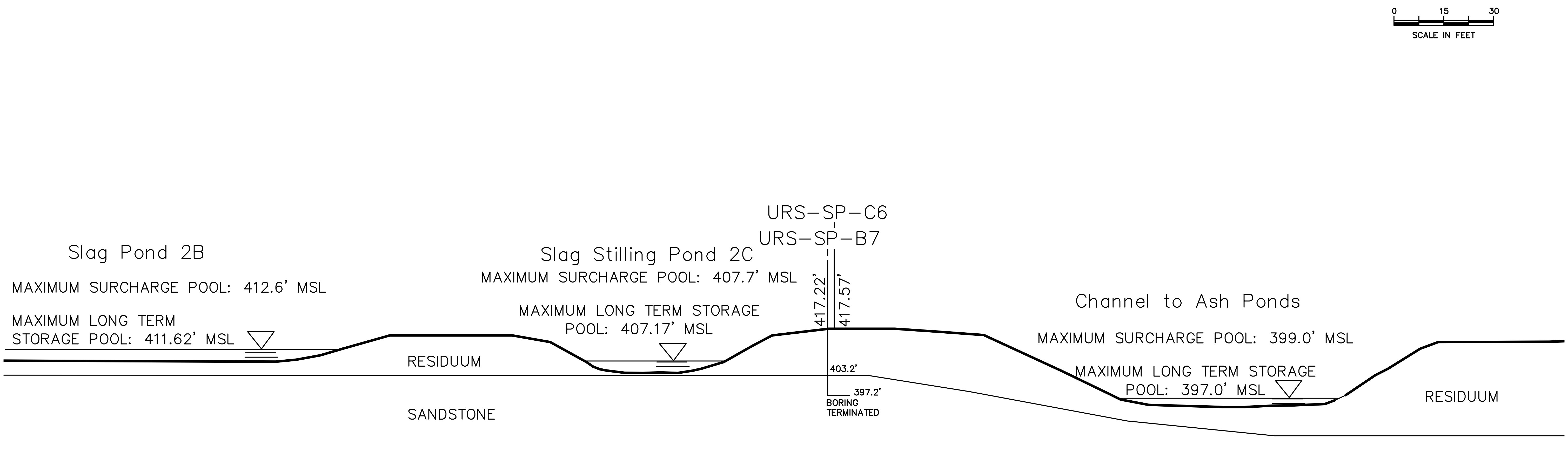
AECOM

**PARADISE FOSSIL PLANT**

TENNESSEE VALLEY AUTHORITY

**SLAG POND 2A, SLAG POND 2B, AND SLAG STILLING POND 2C BORING PLAN VIEW**

| DRAWN BY: | CHECKED BY: | PROJECT No: | DATE: | EXHIBIT |
|---|---|---|---|---|
| NC | MW | 60442564 | 09/16/16 | 1 |



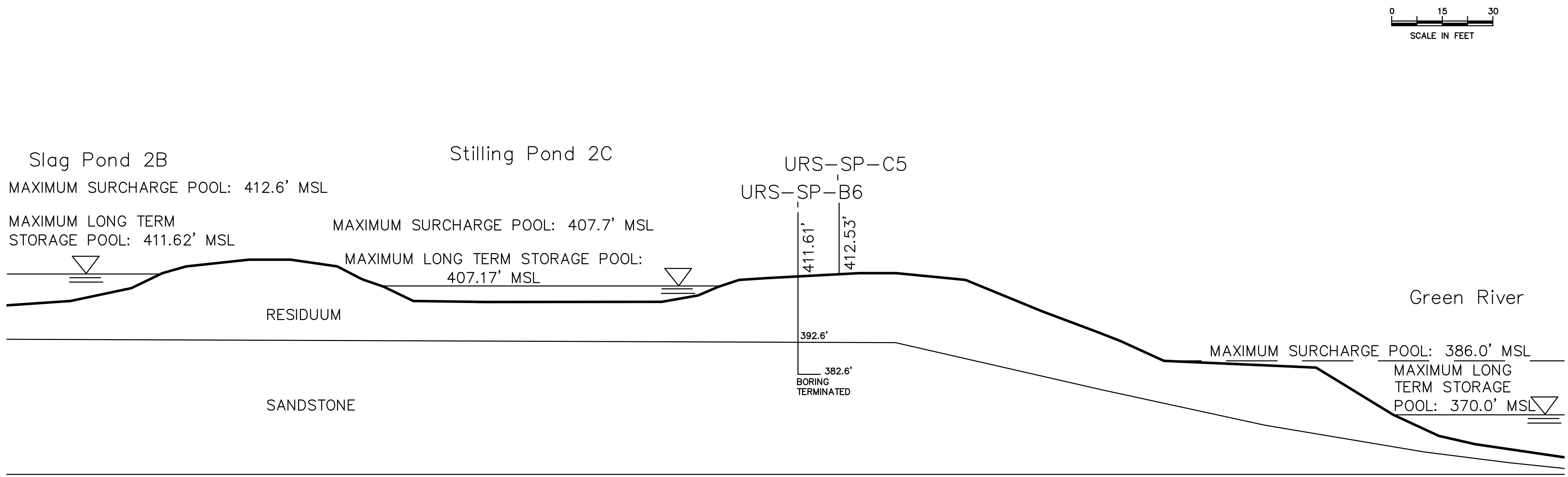

NOTE:
BATHYMETRY FOR SLAG POND 2B, STILLING POND 2C AND THE CHANNEL TO ASH PONDS IS ASSUMED.

AECOM

**PARADISE FOSSIL PLANT**
TENNESSEE VALLEY AUTHORITY

**SLAG POND 2A, SLAG POND 2B, AND SLAG STILLING POND 2C CROSS SECTION A-A'**

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




NOTE:
BATHYMETRY FOR SLAG POND 2B, STILLING POND 2C AND THE GREEN RIVER IS ASSUMED.

AECOM

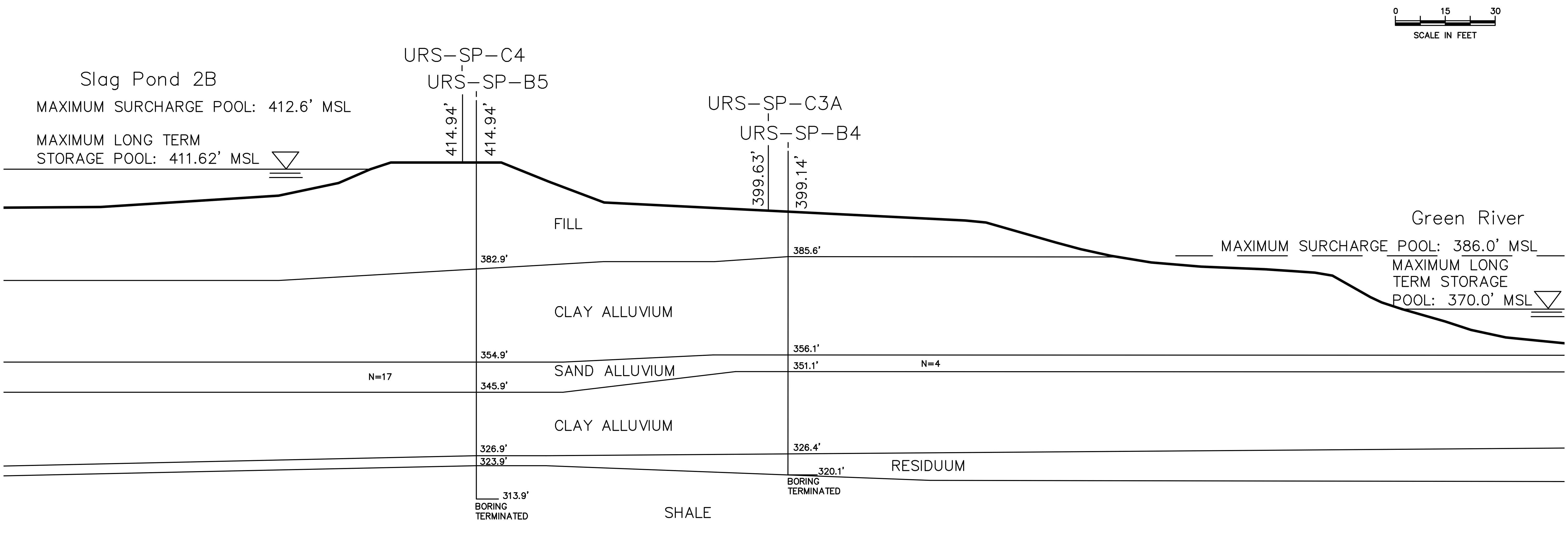
**PARADISE FOSSIL PLANT**

TENNESSEE VALLEY AUTHORITY

**SLAG POND 2A, SLAG POND 2B, AND SLAG STILLING POND 2C CROSS SECTION B-B'**

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

S:\TVA CCP\PAF\slag pond\Analysis\PAF Slag Pond Section Cuts.dwg User:DouangvilayS Oct 11, 2016 - 3:17pm




NOTE:
BATHYMETRY FOR SLAG POND 2B AND THE GREEN RIVER IS ASSUMED.

AECOM

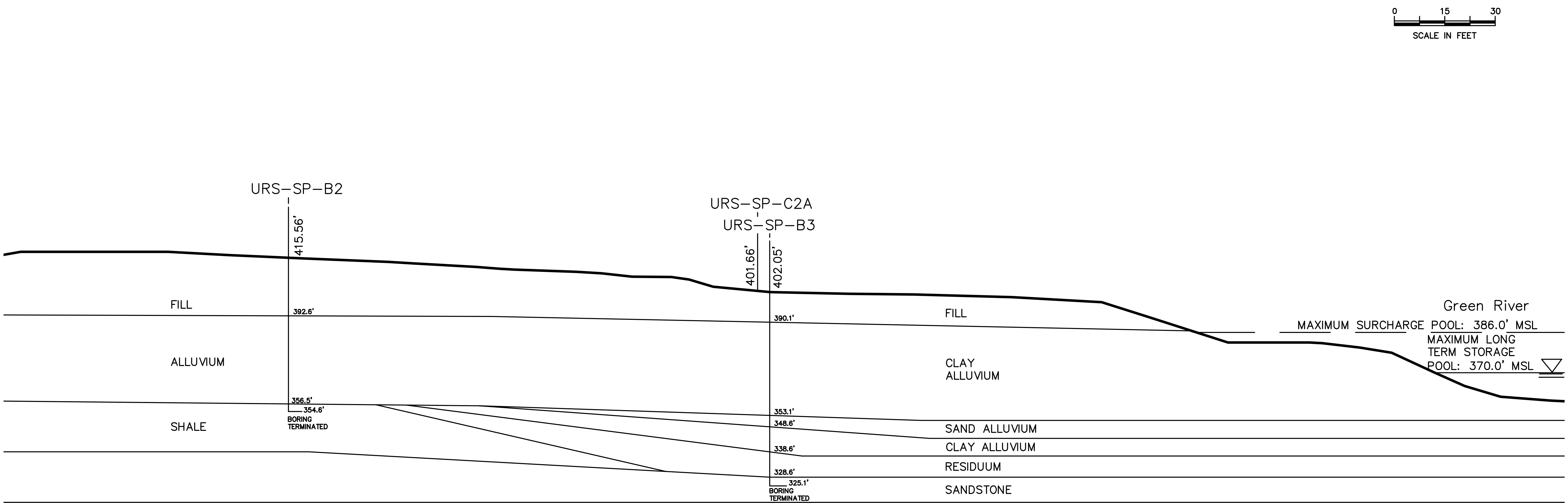
**PARADISE FOSSIL PLANT**

TENNESSEE VALLEY AUTHORITY

**SLAG POND 2A, SLAG POND 2B, AND SLAG STILLING POND 2C CROSS SECTION C-C'**

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

S:\TVA CCP\PAF\slag pond\Analysis\PAF Slag Pond Section Cuts.dwg User:DouangvilayS Oct 11, 2016 - 3:22pm




NOTE:
BATHYMETRY FOR THE GREEN RIVER IS ASSUMED.

AECOM

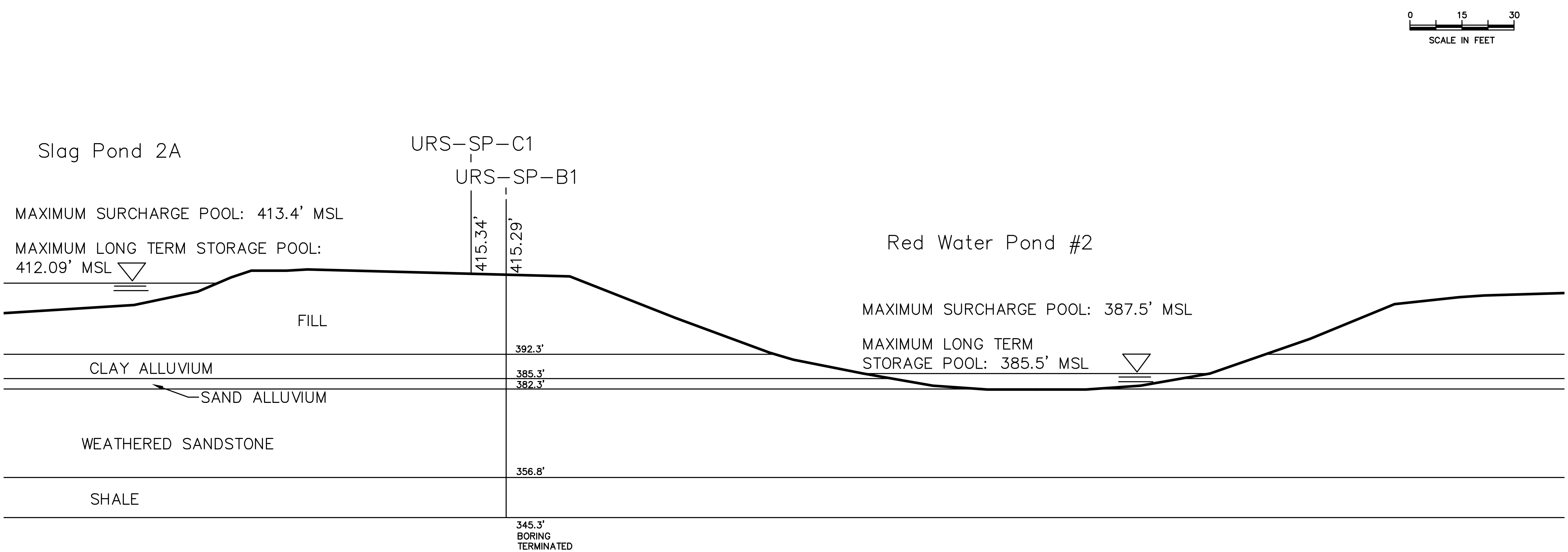
**PARADISE FOSSIL PLANT**

TENNESSEE VALLEY AUTHORITY

**SLAG POND 2A, SLAG POND 2B, AND SLAG STILLING POND 2C CROSS SECTION D-D'**

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

S:\TVA CCP\PAF\slag pond\Analysis\PAF Slag Pond Section Cuts.dwg User:DouangvilayS Oct 11, 2016 – 3:22pm




NOTE:
BATHYMETRY FOR SLAG POND 2A AND RED WATER POND #2 IS ASSUMED.

AECOM

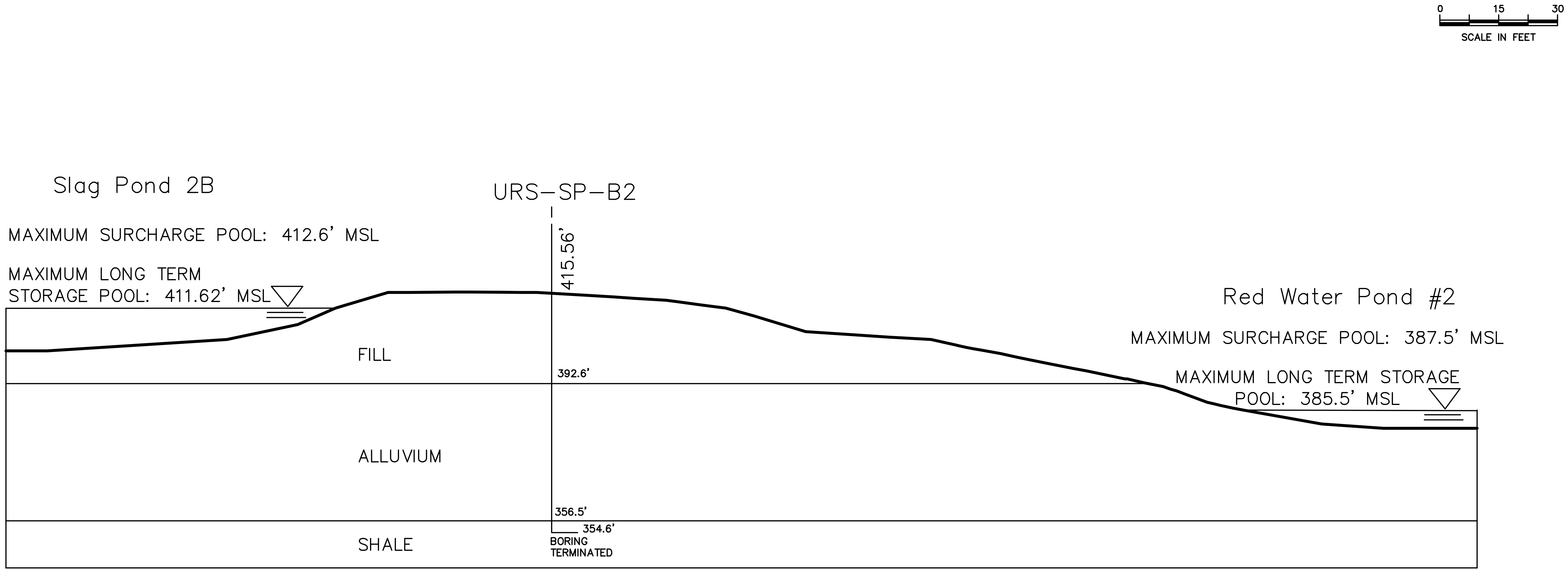
**PARADISE FOSSIL PLANT**

TENNESSEE VALLEY AUTHORITY

**SLAG POND 2A, SLAG POND 2B, AND SLAG STILLING POND 2C CROSS SECTION E-E'**

| DRAWN BY: | CHECKED BY: | PROJECT No: | DATE: | EXHIBIT |
|---|---|---|---|---|
| NC | MW | 60442564 | 09/16/16 | 6 |

S:\TVA CCP\PAF\slag pond\Analysis\PAF Slag Pond Section Cuts.dwg User:DouangviloyS Oct 11, 2016 - 3:18pm




NOTE:
BATHYMETRY FOR SLAG POND 2B AND RED WATER POND #2 IS ASSUMED.

AECOM

**PARADISE FOSSIL PLANT**

TENNESSEE VALLEY AUTHORITY

**SLAG POND 2A, SLAG POND 2B, AND SLAG STILLING POND 2C CROSS SECTION F-F'**

| DRAWN BY: | CHECKED BY: | PROJECT No: | DATE: | EXHIBIT |
|---|---|---|---|---|
| NC | MW | 60442564 | 09/16/16 | 7 |