December 9, 2016

Tennessee Valley Authority
1101 Market Street
Chattanooga, Tennessee 37402

Initial Structural Stability Assessment
Ash Disposal Area 4
EPA Final CCR Rule
TVA Colbert Fossil Plant
Drakesboro, Kentucky

1.0 PURPOSE
This letter documents AECOM’s certification of the initial structural stability assessment for the TVA Colbert Fossil Plant’s Ash Disposal Area 4. Based on this assessment, Ash Disposal Area 4 complies 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 whether Ash Disposal Area 4 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 Ash Disposal Area 4. 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, Michael Stepic PE, being a Professional Engineer in good standing in the State of Alabama, 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 Colbert Fossil Plant's Ash Disposal Area 4 meets the requirements specified in 40 CFR 257.73(d)(1)-(2).

SIGNATURE: [Signature]
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)

DATE 12/9/16

[Seal] No. 305711
PROFESSIONAL ENGINEER
MICHAEL J. STEPIC
COAL COMBUSTION PRODUCT DISPOSAL PROGRAM
TENNESSEE VALLEY AUTHORITY – ASH DISPOSAL AREA 4
TVA COLBERT FOSSIL PLANT
TUSCUMBIA, ALABAMA

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
Chattanooga, TN 37402-2801
December 9, 2016

Prepared by
AECOM

[Stamp]
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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 Colbert 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 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.

Ash Disposal Area 4 meets the first criterion. The location of Ash Disposal Area 4 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 Colbert Fossil Plant is underlain by the Tuscumbia Limestone formation. The Tuscumbia Limestone is of Mississippian age and consists of light to medium gray, fine- to medium-grained fossiliferous (primarily crinoid stems), cherty limestone. Chert occurs as light gray to dark bluish gray, sub-rounded nodules in layers throughout the unit. Cherty layers are laterally discontinuous, and no marker beds exist in the formation. Fractures occur commonly within the Tuscumbia Limestone. Horizontal fractures along bedding planes are the most common orientation. No faults were detected on or in the vicinity of the site.

The soils primarily consist of residual clays including silty clays and moderate to high plasticity clays. Chert fragments generally increase near the bedrock interface. Additionally, alluvial deposits can be encountered near Cane Creek.
An inspection of Ash Disposal Area 4 was completed in 2016. Based on the report, no evidence of structural weakness of the unit was observed. No significant signs of tension cracking, settlement, depressions, erosion, and/or deformations at the crest, slope and toe of dikes were observed. The stability of the slopes has been confirmed through TVA’s Instrumentation Program and the Initial Safety Factor Assessment. No boils or major uncontrollable seepage areas was observed along slopes or toes of the dikes.

Also, an assessment of seepage conditions for Ash Disposal Area 4 including an evaluation of piping potential of the foundation material was performed. Seepage analyses were performed at a critical cross section (Cross-Section D-D’) on the eastern embankment using Geoslope, Inc.’s SEEP/W software. The modeled cross section is shown in Appendix B. Vertical gradients were determined near the toe of the outboard slope. A determination of critical, vertical exit gradients was performed following established sources (including Terzaghi and Peck, USACE EM 1110-2-1901, and USBR Design Standard No. 13 Embankment Dams). Seepage exit gradients determined from the analysis were compared with the critical gradient to calculate a safety factor against piping. For the analyzed cross section, the minimum computed safety factor against piping was calculated to be 3.9 which exceed the recommendations of between 1.5 and 3.0 stated in USACE EM 1110-2-1901. Based on existing analytical data and results, the existing embankments and foundation materials are performing acceptably in regard to piping potential in comparison to current criteria.

2.2 SLOPE PROTECTION - §257.73(d)(1)(ii)

The inboard slope along the dike is armored with riprap, serving as erosion protection along the water line. The crest and the toe of the dike serve as access roads, both having a stone surface. The downstream slopes are primarily covered with well-maintained grassy vegetation, except where covered by stone.

An Intermediate Inspection of CCR Facilities at Colbert Fossil Plant which included the Ash Disposal Area 4 was performed in May 2016. Based on this inspection, the slopes were reported as being generally covered with either maintained 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.

Storm water travel along the dike slopes will not cause erosive effects based on the current slope protection and condition. Pond water will not overtop the dike crest 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 2016 Intermediate Inspection of CCR Facilities and the attached Photos.

2.3 EMBANKMENT DIKE COMPACTION - §257.73(d)(1)(iii)

The dike which forms Ash Disposal Area 4 is approximately 6700 feet in length and consists of an original and raised dike. The Dike ranges in height from approximately 20 to 40 feet in height, with the original perimeter dike being constructed to 440 feet MSL and the raised dike to
460 feet MSL. The raised portion was completed to the upstream of the original crest and is partially founded on ash.

Construction documents from the initial Ash Disposal Area 4 construction in 1972 include sections and notes for both the original and raised dikes. Such construction documents note that the fill was to be compacted to at least 95% of standard compaction maximum density, and have moisture content with no more than 2% above optimum, as determined by laboratory testing.

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 downstream slopes of Ash Disposal Area 4 are vegetated with well-established and maintained grass except were riprap or stone is present. The impoundment is free from brushy and tree like vegetation, with trees in several locations having been removed in 2009.

An Intermediate Inspection of CCR Facilities at Colbert Fossil Plant which included the Ash Disposal Area 4 was performed in May 2016. This report states that a good stand of grass is generally maintained on the slopes of the perimeter dikes. 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 2016 Intermediate Inspection of CCR Facilities and the attached Photos.

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

Under existing conditions, the drainage area for Ash Disposal Area 4 is approximately 52 acres. The drainage area consists of only the impoundment area as there are no run-on flows from outside of the pond’s perimeter dikes. The COF ceased coal burning operations on March 23, 2016. As a result, bottom ash is no longer sluiced to Ash Disposal Area 4. Ash Disposal Area 4 is considered to be an inactive surface impoundment and closure activities are scheduled to begin during December 2016.

During operations, water flowed through an open channel along the bottom ash stack and into the main impoundment area. From the main impoundment area, water flowed through a section of the internal divider dike and into the stilling area. Water then discharged through the spillway and into a concrete channel that leads to Cane Creek, located north of the impoundment. In May 2016, all stop logs were removed from the spillway structure to lower the pond operating level. Currently a water elevation (ELE 449) lower than the spillway inverts is maintained through the usage of siphons and pumps.

More information on the existing Ash Disposal Area 4 operations can be found in the History of Construction Report prepared for CCR Certification by AECOM and the attached Photos.
An H&H computer model was developed using HEC-HMS to examine the hydraulic behavior of the Ash Disposal Area 4 complex during the Inflow Design Flood (IDF). The required IDF used in the model calculation is based on the pond’s hazard classification. Since the Ash Disposal Area 4 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 COF Plant. Existing topographic and survey information for the Ash Disposal Area 4 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 Ash Disposal Area 4 would not overtop the dike crest during a 1,000-year design storm. The freeboard for the Ash Disposal Area 4 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)**

The Ash Disposal Area 4 spillway device consists of four chambers with stop log weirs in the pond and the associated outlet headwall located near the toe of the pond embankment. The spillway structure was constructed by cast-in-place concrete with each of the four chambers being approximately 9’-4” wide, 5’-4” deep, and 7’-0” in height. The front wall incorporates a stop log weir of adjustable height. A davit arm hoist is mounted on the top of the structure to add/remove stop logs as necessary to adjust the weir elevation.

Also affixed to the front of each chamber is a skimmer structure consisting of a half circle corrugated metal pipe at 96” in diameter. 27” HDPE pipes run from the rear wall of the spillway chambers, through an underground filter diaphragm, and out the headwall structure at the toe of the embankment. The headwall is 4’-0” tall and 31’-0” in length with 13’-0¾” long wingwalls on both ends, turned in at 60°. The headwall sits on a large concrete slab, which has 12” high sills for energy dissipation. The pond water flowing from the headwall flows into a culvert beneath an access road into a concrete lined discharge channel to Cane Creek.

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 February 18, 2016, AECOM conducted a site inspection to evaluate the condition of the Ash Disposal Area 4 spillway structure. The inspection was performed from the shore of pond 4 and from the pedestrian grates mounted on top of the spillway chambers. The water elevation of Pond 4 was not lowered for additional inspection access. Likewise, the headwall was inspected from the soil behind the headwall, as well as from a nearby timber access platform. The inspection was visual and non-destructive. Conditions for all four chambers were in similar condition.
2.6.1.1 Spillway Chamber Concrete

No evidence of concrete delamination or spalling was present on the exposed spillway chamber concrete. Minor hairline cracking, likely associated with initial shrinkage, is present, but poses no structural concern. There is staining inside the chambers, likely occurring from storm events, but does not appear to be deteriorating the concrete (Photo 7). The chamber appears to be functioning adequately.

2.6.1.2 Skimmers, Bracing, and Connections

The skimmer devices exhibit surface rust on the corrugated metal pipes, associated steel bracings, and skimmer mount connection angles (Photos 8 and 10). Surface rust has not caused any weakness or loss of material at this time. There was a noticeable kink in one of the skimmer bracing rods in the southernmost chamber, most likely from construction when two rod pieces may have been spliced together (Photo 9). Neoprene dampening shims were installed to mount the skimmer devices between the bolted angle plate and the headwall. The neoprene shims caused the angle not to be seated squarely on the concrete face (Photo 12). This connection still appears to be functioning adequately.

2.6.1.3 Stop Logs, Guides, and Connections

The stop logs are used to establish the weir elevation for the pond water have mineral deposits and algae on their surface, but does not appear to be causing any issues at this time (Photo 14).

The stop log guides consist of a light gauge angle and channel welded together. Sliding surfaces with neoprene gaskets are bonded to the inside of the channels. The bonding agent has begun to break down, and the sliding surfaces have pulled away from channel flanges (Photo 15). This may cause some difficulty in sliding additional stop logs down from the top. Neoprene dampening shims were installed between the angle legs and the concrete faces with holes for the anchors to pass through. These shims were placed correctly and are remaining in place and in good condition.

2.6.1.4 Pedestrian Grating, Railings and Crane

The pedestrian railing and grating is in excellent condition with only a few minor spots of surface rust (Photo 16). The two davit arm hoist baseplates used to raise and lower stop log sections into the spillway chambers were not tested for operation, but all components appear to be in good condition.

2.6.1.5 Outlet headwall

The outlet structure is generally in very good condition. There are no signs of forward rotation or sliding. There is no significant soil erosion occurring behind the headwall or wing walls (Photos 3 and 4). There is a small amount of concrete deterioration at the end of the northern wing wall, but does not appear to be the type of spalling that is associated with rebar corrosion (Photo 17).

There is minor vertical cracking with efflorescence above the 27” dia. HDPE pipe openings (Photo 18) and mineral buildup below the two 4” HDPE pipes exiting the headwall (Photo 19),
but neither affect function, nor pose any structural concern. The energy dissipation sills are in good condition with no signs of erosion.

See the attached Photos for typical conditions of the spillway device.

### 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. At this state, the structural stability was once again checked with regard to floatation, sliding, moment equilibrium, and bearing capacity. Various parts of the structure were again checked to ensure adequate structural capacity.

The headwall was evaluated for only one limit state. The phreatic surface is below the footing slab under normal conditions, and will not rise appreciably for the 1000 year flood. Therefore, groundwater will not affect the design, and the same analysis will apply for the normal operating conditions and the flood event. The concrete lined discharge channel tail water could rise and submerge the headwall, but any buoyant force generated would be counteracted by water above the footing slab. The headwall structure stability was checked with regard to sliding, moment equilibrium, and bearing capacity, and various parts of the structure where checked for capacity.

It should be noted that no live load surcharge was applied to the rear wall of the spillway structure for the analyses. Haul trucks travel the perimeter road behind the spillway, but the road is far enough from the spillway to effectively reduce any live load surcharge.

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)

The Sudden Drawdown Assessment does not apply to the Ash Disposal Area 4. Currently water level is maintained at an elevation lower than the inverts of the spillway outlet via a siphon structure. The stop log weirs of the outlet structure have also been removed.
3.0 CONCLUSION

Based on the initial structural stability assessment, the requirements of Rule §257.73(d)(1) for the Ash Disposal Area 4 have been met.

4.0 REFERENCES


2. AECOM, Ash Disposal Area 4, Initial Inflow Design Flood Control System Plan (40 CFR 257.82) prepared for Coal Combustion Residuals (CCR) Existing Surface Impoundments, 2016.


PHOTOS
Photo 3 – Headwall

Photo 4 – Culvert Beneath Access Road
Photo 5 – Culvert Discharging into Concrete Lined Channel

Photo 6 – Concrete Lined Channel
Photo 7 – Typical Concrete Wall Staining

Photo 8 – Typical Skimmer Rust
Photo 9 – Kink at Splice in Bracing Rod

Photo 10 – Typical Rust on Bracing Rods
Photo 11 – Typical Surface Rust on Connection Angles

Photo 12 – Typical Improper Angle Seating
Photo 13 – Neoprene Shim Fallen Out of Connection

Photo 14 – Typical Algae and Mineral Deposits on Stop Logs
Photo 15 – Typical Bonding Agent Failure

Photo 16 – Pedestrian Grating and Railing
Photo 17 – Minor Concrete Deterioration at End of Northern Wingwall

Photo 18 – Typical Cracking with Efflorescence above 27” HDPE Openings
Photo 19 – Typical Mineral Buildup beneath 4” HDPE Openings
APPENDIX A
HYDRAULIC STRUCTURES ASSESSMENT CALCULATION PACKAGE
Structure Geometry
See URS Ask Pond 4 - High Hazard Removal & Spillway Replacement
Record Drawings dated 8/10/11

Material Properties (Per Record Drawings)
- Concrete Strength = 4 ksi
- Rein. Yield Stress = 60 ksi
- Rolled Steel Plates, Shapes, Bars & Rod = A36
- Corrugated Metal Pipe = ASTM A760 Type I
- Bolts = 5/8” ASTM A307
- Stop Logs = FRP designed for differential head of 6'

Soil properties per original design calculations performed by URS
dated 3/19/10 and Stantec report dated 12/7/09

At Spillway
- Soil bearing capacity = 6000 psf
- Soil-to-slab friction coefficient = 0.35
- Soil-to-wall friction angle $\phi = 11^\circ$
- Soil-to-soil friction angle $\phi = 28^\circ$
- Retained soil = equivalent fluid weighing 40 pcf
  (This includes the effect of the sloping backfill and accounts
   for a portion of the soil being buoyant under the water table.)
  $\gamma = 130$ pcf

At Headwall
- Soil bearing capacity = 6000 psf
- Soil-to-slab friction coefficient = 0.50
- Soil-to-wall friction angle $\phi = 11^\circ$
- Retained soil = equivalent fluid weighing 65 pcf
  (This includes the effect of the sloping backfill.)
  $\gamma = 130$ pcf
- Water table is below headwall floor slab
Evaluate spillway & headwall under regularly occurring water elevation and 1000 year flood elevation.

Structure is considered a significant hazard potential unit (1000 yr flood per TVA-CCR Rule Template 257.73 (d))

Spillway - Regularly Occurring Water Elevation

Per record drawings, normal operation stop log elevation 452.58
Critical scenario - water surface drops just below stop logs and chamber is empty

Evaluate 1 chamber of width
Consider flotation, sliding, moment equilibrium, and bearing capacity
Job TVA COF
Description Colbert Pond 4 Spillway
Computed by MRW
Checked by RN

\[ W_w = 3'(0.667')(9.333')(0.0624 \text{ kip}) = 1.165 \text{ kip} \]
\[ W_{tw} = 1'(0.667')(9.333')(0.150 \text{ kip}) = 0.934 \text{ kip} \text{ increase 30% for stop logs} \]
\[ W_F = 7.333'(0.833')(9.333')(0.150 \text{ kip}) = 8.551 \text{ kip} \]
\[ W_{dw} = 7'(1.333')(0.150 \text{ kip}) = 5.599 \text{ kip} \]
\[ W_{rw} = 7'(0.667')(9.333')(0.150 \text{ kip}) = 6.536 \text{ kip} \]

\[ \text{subtract opening } \frac{\pi (2')^2}{4} (0.667')(0.150 \text{ kip}) = 0.398 \text{ kip} \]
\[ \text{Total } W = 6.138 \text{ kip} \]

\[ W_s = 7'(1')(9.333')(0.130 \text{ kip}) = 8.493 \text{ kip} \]
\[ W_k = 1.667'(1')(9.333')(0.150 \text{ kip}) = 2.334 \text{ kip} \]
\[ W_e = 1'(4')(8')(0.120 \text{ kip}) = 3.840 \text{ kip} \]
\[ F_{H1} = F_{H2} = 0.5(3.333')(1.239 \text{ ksf})(9.333') = 4.275 \text{ kip} \]
\[ F_e = 0.5(7.333')(0.313 \text{ ksf})(9.333') = 11.441 \text{ kip} \]
\[ F_{FE} = 11.441 \text{ kip} + \tan 1^\circ = 2.224 \text{ kip} \]
\[ F_B = 3.333'(7.333')(9.333')(0.0624 \text{ kip}) = 16.369 \text{ kip} \]

\[ \text{Net downward to create friction on bottom of floor slab } \]
\[ W_w + W_{tw} + W_{dw} + W_{fs} + W_{rw} + W_s + F_{FE} - F_B = 1.165 \text{ kip} + 1.401 \text{ kip} + 5.599 \text{ kip} + 3.840 \text{ kip} + 8.551 \text{ kip} + 6.138 \text{ kip} + 8.493 \text{ kip} + 2.224 \text{ kip} - 16.369 \text{ kip} = 21.042 \text{ kip} \]
\[ F_t = 21.042 \text{ kip}(0.35) = 7.365 \text{ kip} \]
Passive Coefficient
\[ \frac{h \sin \theta}{1 - \sin \theta} = \frac{1 + \sin \theta}{1 - \sin \theta} = 2.77 \]

\[ F_p = 0.5 \times 3.50 \times 1.260 \times 9.333 = 20.579 \text{ kips} \]

Regularly occurring water elevation is a "Usual Load Combination" (USLC)
Per TVA-CCR Rule Template 257.73(d) 2.1.5
min allowable flotation stability factor of safety for USLC = 1.3

\[ F_{s_f} = \frac{W_e + W_s + F_p}{V} = \frac{37.411 \text{ kips}}{16.369 \text{ kips}} = 2.285 > 1.3 \checkmark \]

Per TVA-CCR Rule Template 257.73(d) 2.1.5
min allowable sliding stability factor of safety for USLC = 2.0

\[ F_s = \frac{F_e + F_p}{F_e} = \frac{7.365 \text{ kips} + 20.579 \text{ kips}}{11.441 \text{ kips}} = 2.442 > 2.0 \checkmark \]

Per TVA-CCR Rule Template 257.73(d) 2.1.5
moment equilibrium stability requirement for USLC is 100% of base in compression

Sum moments about right corner, ignoring shear key
\[ 1.165^k (1) + 1.401^k (6) + 5.599^k (5.667^k) + 3.840^k (3.667^k) + 8.551^k (8.667^k) + 6.138^k (1.383^k) + 2.224^k (1) + 8.493^k (0.5^k) + 11.441^k (2.611^k) - 16.369^k (3.667^k) = 67.030 \text{ kip-ft} \]

Total Vertical = \[ 1.165^k + 1.401^k + 5.599^k + 3.840^k + 8.551^k + 6.138^k + 2.224^k + 8.493^k - 16.369^k = 21.042^k \]

Resultant located \[ 67.030 \text{ kip-ft}/21.042^k = 3.19^k \text{ from right corner} \]
Job: TVA COF  
Description: Colbert Pond y Spillway Evaluation

Project No. 60432289  
Computed by: MBW  
Checked by: FJW  
Date: 3/14/16  
Date: 3/25/16

Per TVA-CCE Rule Template 257.73(d) 2.1.5

\[ q = \frac{P(1 + \frac{e}{L})}{BL} = \frac{21.042(1 + 6(0.477'))}{7.333'} = 0.427 \text{ ksf} / 0.187 \text{ ksf} \]

\[ FS = \frac{6 \text{ ksf}}{0.427 \text{ ksf}} = 14.1 > 3.0 \checkmark \]

Check longitudinal floor slab rebar

1' width at center

Apply 0.7 load factor

\[ W_0 = 0.12 \text{ klf} \]
\[ W_{ps} = 0.125 \text{ klf} \]
\[ F_E = 0.257 \text{ klf} \]
\[ F_{soil} = 0.307 \text{ klf} \]

If simply supported \( \frac{wL^2}{8} \) top, \( \leq \frac{wL^2}{10} \)

\[ M_u = 0.107(0.558 \text{ klf})(9.333')^2 = 5.201 \text{ kip-ft} = 62,408 \text{ kip-ft} \]

*5 bars @ 12''

\[ \bar{M} = 0.9 \left( \frac{A_s f_y}{2(0.85)} \right)E_b \]

\[ 1.2 M_r = 1.2(0.472 \text{ ksf})(1000 \text{ in}^2) = 113,760 \text{ kip-in} \]

\[ 1.33 M_u = 1.33(62,408 \text{ kip-in}) = 83,003 \text{ kip-in} < 97,679 \text{ kip-in} \checkmark \]
check transverse floor slab rebar

1' width

\[ M_s = 0.9(1.125k)(5.333') - 0.9(0.150k)(1.333') - 0.9(0.48k)(2') - 0.9(0.708k)(2.833') + 
1.2(0.239k)(5.666')(2.833') + 1.6(0.320k)(5.666')(2.833') = 8,634 \text{ kip-ft} = 103,610 \text{kip-ft} \]

4.5 bars @ 8''

\[ \phi M_s = 0.9(0.465 \text{ in}^2)(60 \text{ ksi})(6.666'' - 0.465 \text{ in}^2(60 \text{ ksi}) = 159,350 \text{ kip-in} \]

\[ 1.2 M_c = 113,760 \text{ kip-in} \checkmark \]

Check rear wall

spans horizontally between chamber walls

1' width 1/2 up from base

Hydrostatic soil

\[ 0.042 \text{ ksf, 0.187 ksi} \]

\[ 1.2(0.042 \text{ ksf}) + 1.6(0.187 \text{ ksf}) = 0.380 \text{ ksf} \]

\[ M_s = 0.107 \text{w}l^2 = 0.107(0.350 \text{ ksf})(2.833')^2 = 326 \text{ kip-ft} = 39,145 \text{ kip-in} \]

4.5 bars @ 12'' horizontal

\[ \phi M_s = 0.9(0.31 \text{ in}^2)(60 \text{ ksi})(4'' - 0.31 \text{ in}^2(60 \text{ ksi}) = 63,144 \text{ kip-in} \]

\[ 1.2 M_c = 1.2(0.974 \text{ ksi})(512 \text{ in}^-) = 72,806 \text{ kip-in} \]

\[ 1.33 M_o = 1.33(39,145 \text{ kip-in}) = 52,063 \text{ kip-in} < \phi M_s \checkmark \]
Check side wall
side wall cantilevers vertically because no support at front face of chamber

\[ M_0 = 1.2(0.281 k)(1') + 1.6(0.98 E_k)(2.33') = 3.995 \text{ kip} \cdot \text{ft} = 47.944 \text{ kip} \cdot \text{in} \]

4" bars @ 12" vertically

\[ \phi M_x = 0.9(0.20 \times 260 \text{ ksi}) \left( \frac{1'}{2} - 0.20 \times \frac{260 \text{ ksi}}{21850 \text{ ksi}} \right) = 41.612 \text{ kip} \cdot \text{in} \]

\[ \frac{41.612 \text{ kip} \cdot \text{in}}{47.944 \text{ kip} \cdot \text{in}} = 0.868 \]

Side wall loaded like shown.
Assume when soil reaches full height, it is transferred directly into rear wall. Check soil

\[ \text{up 3.5'} \]

\[ M_0 = 1.2(0.281 k)(1') + 1.6(0.245 k)(1.15') = 0.795 \text{ kip} \cdot \text{ft} = 9.536 \text{ kip} \cdot \text{in} \]

\[ 1.33 M_0 = 12.683 \text{ kip} \cdot \text{in} < \phi M_x = 41.612 \text{ kip} \cdot \text{in} \checkmark \]

Interior walls will always receive equal hydrostatic pressure on both sides, so okay by inspection.
Check shear key
1' wide

\[ M_U = 1.6 \times 1.60 \times 1.111' = 2.844 \text{ kip-ft} \]
\[ f_t = 34.130 \text{ kip-in} \]

\[ CM_x = 0.9 \times 0.465 \times \frac{60 \text{ ksf}}{26.854 \times 12} (8.688' - 0.965' \times 0.465 \times 12) = 209.570 \text{ kip-in} \]

\[ 1.2 M_c = 1.2 \times 0.474 \times \frac{1728 \text{ in}^3}{6} = 163.814 \text{ kip-in} \]
Spillway- 1000 yr Flood
1000 yr flood el. = 454.85
For 1000 yr flood, water will overtop step logs & chambers will be full.

Evaluate 1 chamber of width
Consider floatation, sliding, moment equilibrium, and bearing capacity

\[ W_w = 5.27 \left(5.667 \times 8.000 \times 0.0624 \text{kip/ft}^3 \right) = 14.709 \text{ kip (chamber is full)} \]

\[ F_H = \text{Equal & opposite hydrostatic pressure on front & rear wall} \]

\[ F_B = 6.100 \left(7.333 \times 9.333 \times 0.0624 \text{kip/ft}^3 \right) = 26.051 \text{ kip} \]

Net downward to create friction on bottom of floor slab

\[ W_w + W_{fw} + W_{sw} + W_{g} + W_{fs} + W_{fw} + W_{s} + F_{fe} = F_B \]

\[ 14.909 \text{ kip} + 1.401 \text{ kip} + 5.599 \text{ kip} + 3.840 \text{ kip} + 8.551 \text{ kip} + 6.138 \text{ kip} + 8.493 \text{ kip} + 2.224 \text{ kip} - 26.051 \text{ kip} = 25.104 \text{ kip} \]

\[ F_f = 25.104 \text{ kip} (0.35) = 8.786 \text{ kip} \]

1000 yr flood is an "Unusual Load Combination" (UNLC)
min allowable floatation stability factor of safety for UNLC=1.2

\[ F_{fe} = 51.155 \text{ kip} \]

\[ \frac{1.964}{26.051 \text{ kip}} \]

min allowable sliding stability factor of safety for UNLC=1.5

\[ F_S = \frac{F_f + F_p}{F_{fe}} = \frac{8.786 \text{ kip} + 20.579 \text{ kip}}{11.471 \text{ kip}} = 2.567 > 1.5 \]
moment equilibrium stability requirement for UNLC is 75% of base in compression

Sum moments about right corner, ignoring shear key

\[ 14.909 \text{kip} (4.500') + 1.401 \text{kip} (6') + 5.599 \text{kip} (3.667') + 3.840 \text{kip} (3.667') + 8.551 \text{kip} (3.667') + 6.133 \text{kip} (1.333') + 2.224 \text{kip} (1') + 8.495 \text{kip} (0.5') + 11.441 \text{kip} (2.611') - 26.051 \text{kip} (3.667') = 901.442 \text{kip \cdot ft} \]

Resultant located 901.442 kip \cdot ft / 25.104 kip = 3.603' from right corner
e = 0.064' right of center

\[ \therefore \text{Base is 100% in compression} \checkmark \]

Min. allowable bearing capacity factor of safety for UNLC = 2.6

\[ \frac{25.104 \text{kip}}{9.333' (7.333')} = 1 \quad \frac{0.386 \text{ ksf}}{0.348 \text{ ksf}} \]

\[ F_S = \frac{6 \text{ ksf}}{0.386 \text{ ksf}} = 15.5 > 2.6 \checkmark \]

Check longitudinal floor slab rebar

Apply 0.9 load factor

\[ w_L = 0.329 \text{ ksf} \]

\[ w_K = 0.12 \text{ ksf} \]

\[ w_F = 0.12 \text{ ksf} \]

Apply 1.2 load factor

Apply 1.6 load factor

Factored upward load = 0.558 ksf

Same as previous check \: \: \text{Okay} \checkmark
check transverse floor slab rebar
5' width

\[ M_u = -1.2(1.841\times2.833) - 0.9(0.15\times4.333) - 0.9(0.18\times1.2) - 0.9(0.73\times2.833) \]
\[ + 1.2(0.381\times5.467\times2.833) + 1.6(0.386\times5.467\times2.833) = 7,664 \text{ kip-ft} = 91.974 \text{ kip-in} \]

less than previous check :: okay ✓

Check rear wall
hydrostatic is equal & opposite on rear wall, therefore only soil load
less than previous check :: okay ✓

Check side wall
hydrostatic is equal & opposite on side wall, therefore only soil load
less than previous check :: okay ✓

Interior walls will always receive equal hydrostatic pressure on both sides, so okay by inspection.

Shear key check same as before
Headwall - Regularly Occurring Water Elevation and 1000 yr Flood
Per STANTEC report & VKS, phreatic surface is below footing for headwall & will not rise appreciably for 1000 yr flood. Therefore, groundwater will not affect design and same checks apply to regularly occurring Pond 4 water elevation & 1000 yr flood.

Channel tailwater could rise & submerge headwall, but any buoyant force will be counteracted by water above the footing.

Also, new headwall is doweled into existing structure below, but conservatively not accounted for in design.

Energy Dissipation
Still - Not Continuous

Original design cases showed hydrodynamic force developed on the sill to be negligible. Ignore for analysis as F_H = 0

\[
\begin{align*}
F_g & = 0 \\
W_{SW} & = W_{BW} \\
W_{FS} & = W_{F_F}
\end{align*}
\]
Job: TVA COF  
Description: Colbert Pond 4 Spillway Evaluation

\[ W_{fw} = 1'(1') \times 0.150 \text{ kip per ft} = 0.150 \text{ klf} \]

\[ W_{fs} = 1'(12.75') \times 0.150 \text{ kip per ft} = 1.913 \text{ klf} \]

\[ W_{rw} = 4'(1') \times 0.150 \text{ kip per ft}^2 = 0.600 \text{ klf} \]

\[ W_s = 4'(1') \times (0.130 \text{ kip per ft}^2) = 0.520 \text{ klf} \]

\[ F_e = \frac{1}{2} \times (5') \times (0.325 \text{ ksf}) = 0.813 \text{ klf} \] at 1.667' up from base

\[ F_{pc} = 0.813 \text{ klf} \times \tan 11' = 0.158 \text{ klf} \]

Net downward to create friction on bottom of floor slab

\[ W_{fw} + W_{fs} + W_{rw} + W_s + F_e \]

\[ 0.150 \text{ klf} + 1.913 \text{ klf} + 0.600 \text{ klf} + 0.520 \text{ klf} + 0.158 \text{ klf} = 3.341 \text{ klf} \]

\[ F_r = 3.341 \text{ klf} \times (0.5) = 1.671 \text{ klf} \]

min allowable sliding stability factor of safety for USLC = 2.0

\[ F_S = \frac{1.671 \text{ klf}}{0.813 \text{ klf}} = 2.055 > 2.0 \checkmark \]

moment equilibrium stability requirement for USLC is 100% of base in compression.

Sum moments about right corner:

\[ 0.150 \text{ klf} \times (10.5') + 1.913 \text{ klf} \times (0.375') + 0.600 \text{ klf} \times (1.5') + 0.520 \text{ klf} \times (0.5') + 0.158 \text{ klf} \times (1') \]

\[ + 0.813 \text{ klf} \times (1.667') = 16.441 \text{ kip ft} \]

Total \( V_r = 3.341 \text{ klf} \)

Resultant located \( \frac{16.441 \text{ kip ft}}{3.341 \text{ klf}} = 4.922' \) from right corner

\[ e = 1.453' \text{ right of center} \]

\[ y_c = 2.125' \]
$e \leq \frac{L}{6}$ base is 100% in compression

Min allowable bearing capacity factor of safety for USLC = 3.0

$q = \frac{P}{(1 + \frac{6}{L})} = \frac{3.341 \text{kif}}{(1 + \frac{6(1.453)}{12.75})} = \frac{0.441 \text{kif}}{0.083 \text{kif}}$

$FS = \frac{6 \text{ksf}}{0.441 \text{ksf}} = 13.6 > 3.0 \checkmark$

Check transverse floor slab rebar

$$M_u = -0.9(0.15)(8.5') - 0.9(1.43)(5.375) + 1.6(0.083)(10.75')(5.375') = 8.029 \text{kip-ft} = 96.345 \text{kip-ft}$$

*5 @ 10"

$$\Phi M_u = 0.9(0.372 i^2)(60 \text{ksi})(8.688' - \frac{0.372 i^2}{2}(60 \text{ksi}) = 169.030 \text{kip-in}$$

$$1.2 M_u = 1.2(0.474 \text{ksi})(1728 \text{in}^4) = 163.814 \text{kip-in} \checkmark$$

Check headwall rebar

$$M_u = 1.6\left(\frac{4}{3}\right)(0.26 \text{ksf})(1.333') = 1.109 \text{kip-ft} = 13.309 \text{kip-ft}$$

*5 @ 10"
Stop logs spec'd for differential head of 6'. This is the max height of the stop logs per chamber geometry.

Stop log guide frames and 96" CMP bolted to spillway w/ 6" long 3/8" dia. wedge anchors. Minimal load on connection (self-weight). Water flow does not apply additional load to anchors. Okay by inspection.
Tennessee Valley Authority
Colbert Fossil Plant
Ash Pond 4

Steady-State Seepage Analysis
Long-Term Existing Conditions
Cross-Section D-D’

Date: 11/30/2016
Contour parameter: Total Head ft
Method: Steady-State

Critical gradient: i(crit) = 0.93
Exit gradient: i(exit) = ΔH/ΔL = 0.24

FS against piping = 3.9

<table>
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<tr>
<th>Color</th>
<th>Name</th>
<th>Model</th>
<th>Sat Kx (Rises)</th>
<th>Ky/Kx’ Ratio</th>
<th>Volumetric Water Content (%)</th>
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<td>Upper Clay Dike (Saturated)</td>
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</table>

Maximum gradient occurs at (-82.00,415.00)
Total Head: 414,009 ft

At (-81.84,413.89)
Total Head: 414,263 ft

Project #: 60452541