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October 12, 2016

Tennessee Valley Authority
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Initial Structural Stability Assessment

Ash Pond E

EPA Final CCR Rule

TVA Gallatin Fossil Plant

Gallatin, Tennessee

1.0 PURPOSE

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

2.0 INITIAL STRUCTURAL STABILITY ASSESSMENT

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

3.0 SUMMARY OF FINDINGS

The attached report presents the initial structural stability assessment of the Ash Pond E. 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, Gabriel W. Lang, being a Professional Engineer in good standing in the State of Tennessee, 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 Gallatin Fossil Plant's Ash Pond E meets the requirements specified in 40 CFR 257.73(d)(1)-(2).

SIGNATURE _____



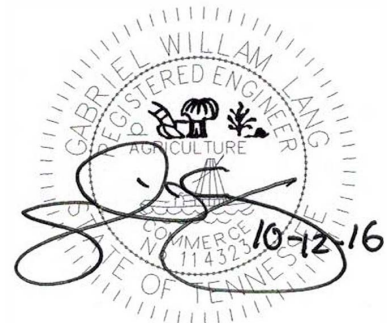
DATE 10/12/16

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ATTACHMENTS: Initial Structural Stability Assessment 40 CFR 257.73(d)(1); Existing CCR Surface Impoundments; TVA Gallatin Fossil Plant; Ash Pond E



COAL COMBUSTION PRODUCT DISPOSAL PROGRAM

Tennessee Valley Authority – Ash Pond E
Sumner County, Tennessee

Initial Structural Stability Assessment 40 CFR 257.73(d)(1) Existing CCR Surface Impoundment TVA Gallatin Fossil Plant

Prepared for



Tennessee Valley Authority
1101 Market Street
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October 12, 2016 – Rev0

Prepared by

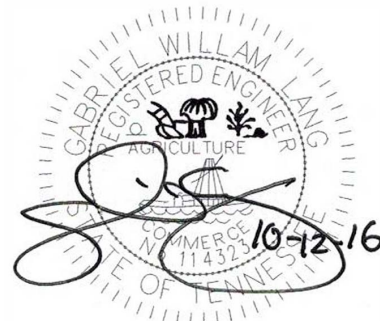




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Appendix B Sudden Drawdown Analysis

1.0 Project Background

On April 17, 2015 the “Disposal of Coal Combustion Residuals (CCR) from Electric Utilities” (CCR Rule) was published in the Federal Register. AECOM has been contracted by TVA to analyze the Structural Stability of the Gallatin Fossil Plant’s CCR surface impoundments 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.

Ash Pond E meets both criteria. A plan view showing the location of Ash Pond E is shown in Figure 1.

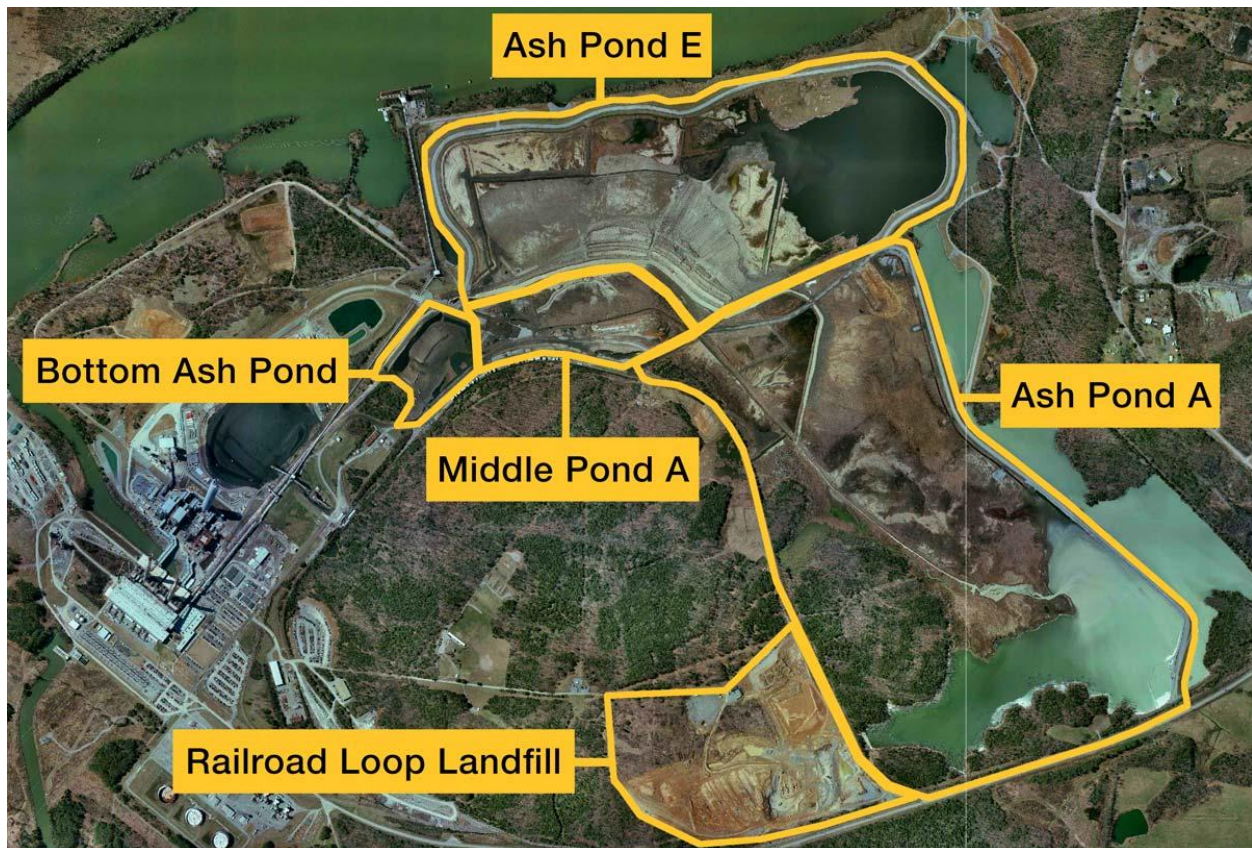


Figure 1: Ash Pond Complex

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 Gallatin Fossil Plant (GAF) is located in the northern portion of central Tennessee along the north bank of the Cumberland River. The geologic map for the area shows soil deposits consisting of alluvial clay, silt and very fine sand across the site. The map indicates a variable thickness of the alluvium that may be as much as 70 feet. The alluvial deposits are mapped primarily at lower site elevations along the power plant area and extend into the southern end of the ash pond complex. The remaining areas are underlain by residual clays resulting from in-place weathering of the parent Ordovician age limestone formations. Therefore, the majority of the ash pond complex is underlain by either alluvial or residual clays.

Specifically, the foundation of the perimeter dikes that surround Ash Pond E typically consists of residual clay and the foundation of the divider dikes consists of sluiced ash. The residual clay consisted of moist, yellow to red-brown, medium stiff to stiff, lean clay (CL) and fat clay (CH). The sluiced ash classified as wet, gray and black, loose to medium dense, silty sand (SM), clayey sand (SC), and sandy silt (ML).

A dam assessment and inspection of the Ash Pond Complex which includes Ash Pond A, Middle Pond A, Bottom Ash Pond, and Ash Pond E at GAF was completed in 2013 and 2016, respectively (see references [1] and [2]). Based on the reports, no evidence of structural weakness of the inspected units was observed. No significant signs of tension cracking, settlement, depressions, erosion, and/or deformations at the crest, slope and toe of the dikes were observed. The stability of the slopes has been confirmed through TVA's Instrumentation Program (see reference [3]). No boils or major uncontrollable seepage areas was observed along slopes or toes of the dikes.

Also, an assessment of seepage conditions for Ash Pond E including an evaluation of piping potential of the foundation material was performed; and the results of the assessment were provided in a geotechnical evaluation report dated May 27, 2010, see reference [4]. Seepage analyses were performed at four cross sections across the perimeter and divider dikes using Geoslope, Inc.'s SEEP/W software. As part of that analysis, horizontal and vertical gradients were determined near the toe of the downstream 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 seepage analysis were compared with the critical gradient to calculate a safety factor against piping. For the analyzed cross sections of the Ash Pond E perimeter and divider dikes, the minimum computed safety factor against piping was recorded at 3.0. All seepage modeling performed indicated a factor of safety against piping of greater than or equal to 3, which meets the requirement of 3.0 stated in USACE EM 1110-2-1901.

In November 2014, unfiltered seepage was observed exiting the downstream exterior of the western Ash Pond E dike. The seepage was related to a partially abandoned reinforced concrete pipe penetrating the embankment. Therefore, plans were developed to construct a graded filter along the downstream slope and abandon the pipe. The construction activities were performed in December, 2015, and January, 2016. The design and construction documents associated with the seepage repair can be seen in references [5] and [6].

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

2.2 Slope Protection - §257.73(d)(ii)

The slopes along the perimeter dikes and divider dikes are generally protected with either dense grass or riprap; no trees or large, bushy vegetation are present on the slopes.

No additional slope protection is required based on anticipated flow velocities.

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

Construction documents (see references [7] and [8]) indicate that both the original dikes and the raised dikes were mechanically compacted. For 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. For the raised dikes, the embankments were constructed of heavy bottom ash fill placed in 9" maximum loose lifts and thoroughly compacted with loaded rubber tired earth hauling equipment making a minimum of 6 passes over each layer. The shells of the dikes were constructed of bottom ash fill well compacted in layers with heavy rubber tired equipment.

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

The slopes of the perimeter dikes and divider dikes that form Ash Pond E have been maintained with either dense grass or riprap; no trees or large, bushy vegetation are present on the slopes.

2.5 Spillway Capacity - §257.73(d)(1)(v)

Per §257.73(d)(1)(v),

(A) *All spillways must be either:*

- (1) *Of non-erodible construction and designed to carry sustained flows; or*
- (2) *Earth- or grass-lined and designed to carry short-term, infrequent flows at non-erosive velocities where sustained flows are not expected.*

(B) *The combined capacity of all spillways must adequately manage flow during and following the peak discharge from a:*

- (1) *Probable maximum flood (PMF) for a high hazard potential CCR surface impoundment; or*
- (2) *1000-year flood for a significant hazard potential CCR surface impoundment; or*
- (3) *100-year flood for a low hazard potential CCR surface impoundment.*

2.5.1 Spillway Capacity at Sustained Flows - §257.73(d)(1)(v)(A)

Ash Pond E has two 48-inch reinforced concrete pipe (RCP) spillway risers which discharge through two 30-inch steel pipes (epoxy coated) into Stilling Pond C. During normal operating conditions, the spillway risers carry sustained flows from Ash Pond E to Stilling Pond C. The spillway risers are constructed of reinforced concrete and the pipes are constructed of steel, both are non-erodible materials that are resistant to the flow of water. According to the hydrologic and hydraulic (H&H) study completed in 2016 (see reference [9]), the spillway is adequate to carry the sustained flows.

2.5.2 Spillway Capacity at Peak Discharge - §257.73(d)(1)(v)(B)

Based on the hazard assessment in 2015 (see reference [10]), Ash Pond E has been classified as a significant hazard potential. Therefore, the combined capacity of all spillways in Ash Pond E must adequately manage flow during and following the peak discharge from a 1000-year flood. According to the H&H study completed in 2016 (see reference [9]), the combined

capacity of the spillways is adequate to manage the flow during and following the peak discharge from a 1000-year flood.

2.6 Hydraulic Structures - §257.73(d)(1)(vi)

Ash Pond E receives process water from the plant and coal contact water generated from the Coal Stockpile, Coal Conveyor, and Coal Crusher during wet weather. Ash Pond E is an essential part of the process flow and coal contact water treatment. The flow is routed into two 48-inch RCP risers which discharge through two 30-inch steel pipes into Stilling Pond C. **Pictures 7 and 8** from the 2006 annual inspection, provided as an attachment to this report, show the risers.

2.6.1 Pipes

The existing pipes through the divider dike between Ash Pond E and Stilling Pond C have not been inspected since they are currently submerged. A couple of pictures show the condition of pipe inlets during the pond lowering project (see attached photos and reference [11]). From the photos, no blockage of flow or deterioration was detected on the interior of the pipes at the inlet. Visual inspections of the dikes where pipes pass through do not show any signs of deformation.

The pipes have been evaluated for buckling stability for two different limit states: usual loading conditions associated with regularly occurring pool levels and unusual loading conditions associated with the design flood event. All associated calculations, including the structure's geometry and material properties are included in **Appendix A**. The pipes satisfy the stability checks for the limit states considered.

2.6.2 Spillway - Vertical Risers

The spillway, consisting of two 48" RCP vertical risers, is located at the northeast end of Ash Pond E. The original construction drawings (see references [7] and [8]) were used to determine the dimension of the vertical risers. The risers were all constructed similarly. Each riser is founded on a reinforced concrete pad (6.5ft wide x 6.5ft long x 1.5ft deep). A reinforced concrete junction box (6ft wide x 6ft long x 4ft tall) with 12" wall thickness is attached to the foundation. Two feet lengths of 48" diameter Class III RCP were stacked on the junction box to raise the risers to their current elevations. Construction documents state that pipe joints were to be grouted to form stable and water-tight connections. Based on Ash Pond E spillway modifications completed as a part of the pond lowering project (see reference [11]), each spillway structure was covered with a grated concrete manhole cover. Water flows southwest to northeast via two 30" diameter steel pipes that extend from the junction box of the spillways through the divider dike and discharges in Stilling Pond C. The outlet pipes are approximately 183-feet long.

No cracking, deterioration or spalling was detected on the interior or exterior of the concrete risers. There were no apparent deteriorations or obstructions to water flow from the top of the risers. **Pictures 7-8** taken in 2006 from GAF Pond E Spillway Inspection show the typical condition of the risers.

The riser structures were evaluated for two different limit states. The first limit state is associated with regularly occurring pool levels (usual loading conditions). The critical condition for floatation of the riser structures occurs when the pool level is near the top of the riser structures but does not flow over. It was assumed that the riser structures were not filled with water. The buoyant force was applied at the bottom of the foundation. The critical condition for bearing capacity occurs 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 (unusual loading conditions). Evaluation for this flood event is required for a structure defined to have a significant hazard potential per CCR Rule. It has been determined that the 1000-year flood event will not overtop the pond dikes, so there will be no stream flow against the side of the riser structures that will cause instability issues. For informational purposes, analyses were completed using the maximum water flow velocity for which the structural stability of the riser structures was satisfied.

All associated calculations, including structure geometry and material properties are included in **Appendix A**. The risers satisfy all strength and stability checks for the limit states considered. Thus, no rehabilitation is required at this time.

2.6.3 Recommendation

It was noted earlier that pipe inspections for the pipes that run through the divider dikes between Ash Pond E and Stilling Pond C have not been inspected due to the submerged nature of the pipes. Based upon visual observations of the facility and hydraulic modeling, the pipes appear to be performing adequately. As part of future periodic assessments, the condition of the pipes should be confirmed via camera inspections.

2.7 Sudden Drawdown - §257.73(d)(1)(vii)

A sudden drawdown analysis was completed for the perimeter dike between Ash Pond E and the Cumberland River. The analysis was completed using Geoslope Inc.'s SLOPE/W software, which implements a multi-stage approach as presented by Duncan, Wright, and Wong referenced in USACE EM 1110-2-1902. A composite shear strength envelope using both drained and undrained shear strength parameters was used for the dike materials.

A cross-section on the perimeter dike was selected to be representative of the most critical cross section. The material properties of the dike and the subsurface conditions had been formulated in previous geotechnical evaluations, see reference [12].

The low water level in the Cumberland River corresponds to the winter pool which is at an approximate elevation of 442.0 feet MSL and the flood pool rises to an approximate elevation of 452.0 feet MSL. The normal pool level in Ash Pond E is at an approximate elevation of 457.0 feet MSL and the flood pool rises to an approximate elevation of 461.6 feet MSL.



The slope stability was determined for a drawdown of the water level from the flood pool to the normal pool. The sudden drawdown analysis indicated a minimum safety factor against slope stability of 1.8 which exceeds the minimum acceptable safety factor of 1.1 as provided in USACE EM 1110-2-1902. See **Appendix B** for the graphic output from SLOPE/W.

3.0 Conclusion

Based on the results of the initial structural stability assessment, Ash Pond E meets the requirements of the CCR Rule as discussed in **Section 2.0**.

4.0 References

- [1] Dewberry Consultants LLC, "Coal Combustion Residue Impoundment, Round 11 - Dam Assessment Report," April 2013.
- [2] TVA, "2016 Intermediate Inspection of CCR facilities at Tennessee Valley Authority's (TVA's) Gallatin Fossil Plant (GAF)," August 1, 2016.
- [3] Stantec and AECOM, "Annual Instrumentation and Monitoring Program Final Report (Rev. 2); Fiscal Year 2015; Tennessee Valley Authority Instrumentation Monitoring Program; Coal Combustion Product (CCP) Storage Facilities; Various Plants Alabama, Kentucky, and Tennessee.," February 5, 2016.
- [4] Stantec Consulting Services Inc., "Report of Geotechnical Exploration and Slope Stability Evaluation, Ash Pond/Stilling Pond Complex, Gallatin Fossil Plant," May 27, 2010.
- [5] AECOM, "TVA GAF Pond E Seepage Repair, Filter Analysis," January 23, 2016.
- [6] URS, "Ash Pond E Seep Repair Construction Drawings," December 10, 2014.
- [7] Tennessee Valley Authority, "Construction Drawing 10W271-307 R0," October 23, 2007.
- [8] Tennessee Valley Authority, "Construction Drawing 10W271-315 R0," October 23, 2007.
- [9] AECOM, "Inflow Design Flood Control Plan; Ash Pond E; Gallatin Fossil Plant; Sumner County, Tennessee.," October 2016.
- [10] Stantec Consulting Services Inc., "Hazard Potential Classification Assessment; Ash Pond E; Gallatin Fossil Plant; Sumner County, Tennessee.," October 5, 2015.
- [11] AECOM, "Ash Pond Lowering and Flow Diversion Drawings," January 8, 2016.
- [12] URS, "Ash Pond A and E Dikes, Geotechnical Site Evaluation Report (Rev. 0)," February 21, 2014.
- [13] Stantec Consulting Services Inc., "Report of Breach Analysis for GAF Ash Pond Complex," 2013.
- [14] USACE, "USACE EM 1110-2-1902 Slope Stability," October 31, 2003.
- [15] USBR, "Design Standard No. 13: Embankment Dams," January 2014.
- [16] K. Terzaghi and R. B. Peck, Soil mechanics in engineering practice, New York: John Wiley & Sons, Inc., 1996.

PHOTOS

**GALLATIN FOSSIL PLANT
ANNUAL ASH POND DIKE STABILITY INSPECTION
2006**



Picture 7: New drainage structures from Pond E to Pond D



Picture 8: New drainage structures



Picture 1: Typical condition of pipe inlets at spillway junction box during construction of Ash Pond Lowering and Flow Diversion Project (2016)



Picture 2: Typical condition of pipe inlets at spillway junction box during construction of Ash Pond Lowering and Flow Diversion Project (2016)

APPENDIX A
HYDRAULIC STRUCTURES ASSESSMENT
CALCULATION PACKAGE

Structural Stability Assessment for Riser Structures in Ash Pond E at TVA Gallatin Fossil Plant

Prepared for

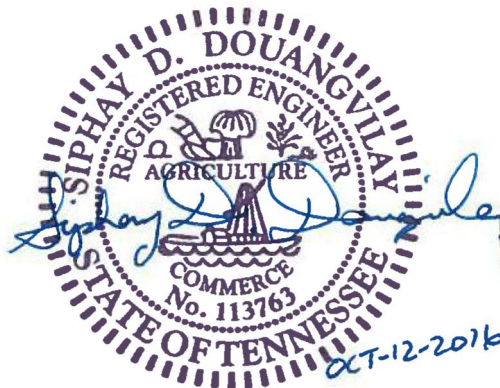


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Discussion

The following calculations detail the structural stability assessment for the existing riser structures in Ash Pond E at Tennessee Valley Authority (TVA) Gallatin Fossil Plant (GAF). The calculations were completed in accordance with United States Environmental Protection Agency's (EPA) requirements under the Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals (CCR) from Electric Utilities [RIN-250-AE81; FRL-9149-4] (EPA Final CCR Rule) section 257.73(d).

References

- 1.) TVA-CCR Rule Template 257.73 (d).
- 2.) Existing drawings listed under Riser and skimmer geometry.
- 3.) URS Ash Pond A and E Dikes Geotechnical Site Evaluation Report (Rev. O), February 21, 2014.
- 4.) USACE EM 1110-2-2100, Stability Analysis of Concrete Structures, December 1, 2005.
- 5.) USACE EM 1110-1-1905, Bearing Capacity of Soils, October 30, 1992.

Material Properties and Geometry

The material properties and geometry defined below are determined using TVA CCR rule template 257.73(d), existing project drawings, geotechnical data report, historical data, and engineering judgement.

Soil properties

Unit weight of water	$\gamma_w := 62.4 \text{pcf}$
Unit weight of foundation soil	$\gamma_s := 105 \text{pcf}$
Friction angle of foundation soil	$\phi_s := 34^\circ$
Cohesion of foundation soil	$c_s := 0 \text{psf}$

Concrete material properties

Reference TVA CCR Rule 257.73(d), 2.1.1

Unit weight of concrete	$\gamma_c := 150 \text{pcf}$
Poisson's ratio	$\nu_c := 0.2$
Unconfined compressive strength	$f_c := 3000 \text{psi}$ Class A concrete
Shear strength	$\tau_c := 0.10 \times f_c = 300 \text{psi}$
Static tensile strength	$f_t := 1.7 \text{psi} \times \frac{f_c}{\text{psi}} = 51 \text{psi}$
Dynamic tensile strength	$f_{td} := 1.5 \times f_t = 76.5 \text{psi}$
Instantaneous elastic modulus	$E_c := 57000 \times \sqrt{f_c \text{psi}} = 3.122 \times 10^6 \text{psi}$
Sustained elastic modulus	$E_{cs} := 0.70 \times E_c = 2.185 \times 10^6 \text{psi}$

Reinforcing steel material properties

Reference TVA CCR Rule 257.73(d), 2.1.1

Yield strength	$f_y := 60 \text{ksi}$
Elastic modulus	$E_s := 29000 \text{ksi}$

Riser and skimmer geometry

Reference:

- 1.) TVA Pond E Precast Outlet Boxes Structural Details Drawing No. 10W271-315 dated OCT-23-2007.
- 2.) AECOM, Spillway Modification Details Drawing No. 10W510-9 dated JAN-8-2016.
- 3.) Tennessee Department of Transportation (TDOT) Standard No. 3 Manhole Castings and Steps Drawing D-MH-4, August 1972.

Figure 1: Modified Pond E Spillway Risers from 10W510-9

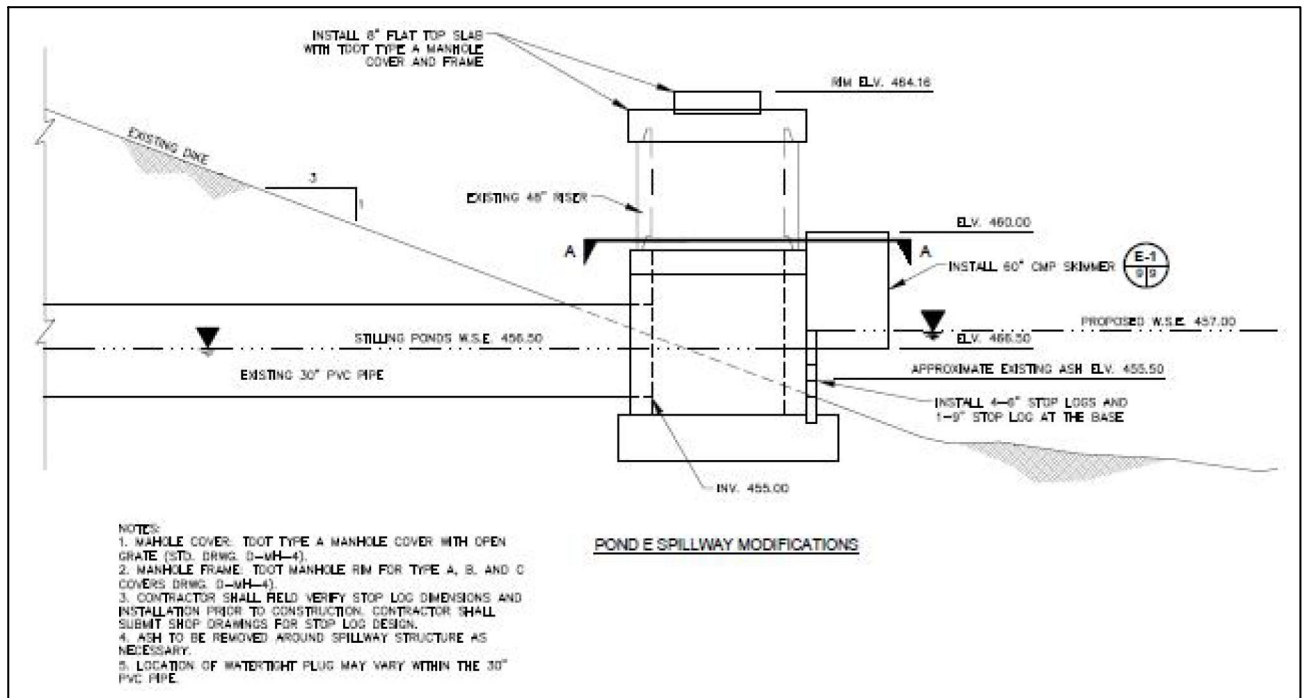
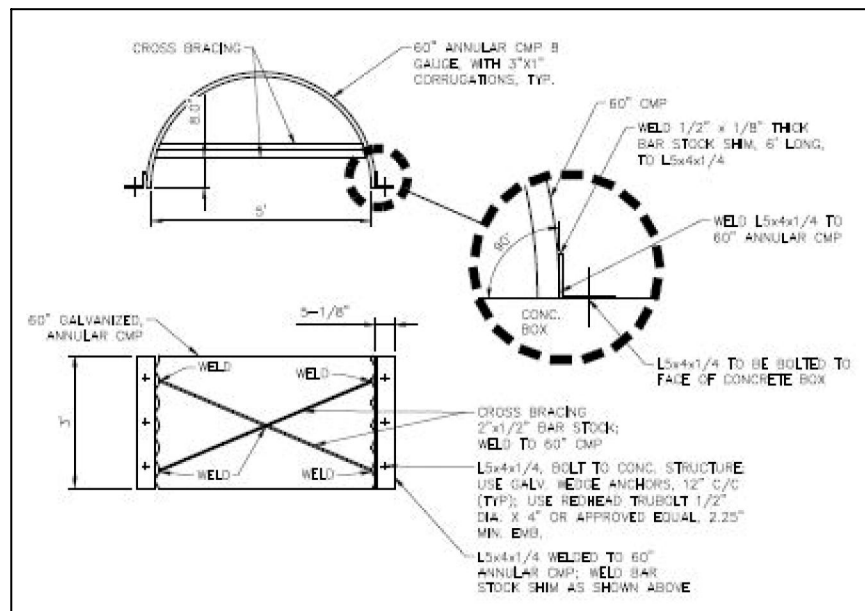


Figure 2: 60" CMP Skimmer Attachment from 10W510-9



Riser geometry

Top elevation of riser structure	$EL_{top} := 462.35\text{ft}$ (not including skimmer)
Bottom elevation (at top of foundation)	$EL_{bot} := 454.85\text{ft}$
Height of structure	$H_{structure} := EL_{top} - EL_{bot} = 7.5\text{ft}$
Height of water outside (from top of foundation)	$H_{wo} := H_{structure} = 7.5\text{ft}$
Foundation - width of footing	$B_{ftg} := 6\text{ft}$
Foundation - length of footing	$L_{ftg} := 6\text{ft} + 8\text{in} = 6.667\text{ft}$
Foundation - depth of footing	$D_{ftg} := 1\text{ft} + 2.75\text{in} = 1.229\text{ft}$
Foundation - volume	$V_{ftg} := B_{ftg} \times L_{ftg} \times D_{ftg} = 49.167 \times \text{ft}^3$
Foundation - weight	$W_{ftg} := V_{ftg} \times \gamma_c = 7.375 \times \text{kip}$
Junction box - external width	$B_{box} := 5\text{ft} + 4\text{in} = 5.333\text{ft}$
Junction box - wall thickness	$t_{box} := 8\text{in} = 0.667\text{ft}$
Junction box - height	$h_{box} := 5.0\text{ft}$
Junction box - volume	$V_{box} := \frac{\pi}{4} \times (B_{box}^2 - (B_{box} - 2 \times t_{box})^2) \times h_{box} = 62.222 \times \text{ft}^3$
Junction box - weight	$W_{box} := V_{box} \times \gamma_c = 9.333 \times \text{kip}$
Riser pipe - inner diameter	$ID_{rsr} := 4.0\text{ft}$
Riser pipe - thickness	$t_{rsr} := 4.0\text{in}$
Riser pipe - outer diameter	$OD_{rsr} := ID_{rsr} + 2 \times t_{rsr} = 4.667\text{ft}$
Riser pipe - total height	$h_{rsr} := H_{structure} - h_{box} = 2.5\text{ft}$
Riser pipe - cross sectional area	$A_{rsr} := \frac{\pi}{4} \times (OD_{rsr}^2 - ID_{rsr}^2) = 4.538 \times \text{ft}^2$
Riser pipe - volume	$V_{rsr} := A_{rsr} \times h_{rsr} = 11.345 \times \text{ft}^3$
Riser pipe - weight	$W_{rsr} := V_{rsr} \times \gamma_c = 1.702 \times \text{kip}$

Skimmer geometry

Skimmer - 60" annular CMP 8 gauge

$$\text{unit weight } \gamma_{\text{cmp}} := 161 \text{ plf; length } L_{\text{cmp}} := 3 \text{ ft; weight } W_{\text{cmp}} := \frac{1}{2} \gamma_{\text{cmp}} \times L_{\text{cmp}} = 241.5 \text{ lbf}$$

Skimmer - Angle L5x3.5x1/4

$$\text{quantity } q_a := 2; \text{ unit weight } \gamma_a := 7 \text{ plf; length } L_a := 3 \text{ ft; weight } W_a := q_a \times \gamma_a \times L_a = 42 \text{ lbf}$$

Skimmer - 1/2"x1/8" thick bar stock shim, 6' long

$$\text{unit weight } \gamma_{b1} := 0.213 \text{ plf; length } L_{b1} := 6 \text{ ft; weight } W_{b1} := \gamma_{b1} \times L_{b1} = 1.278 \text{ lbf}$$

Skimmer - 2"x1/2" bar stock bracing

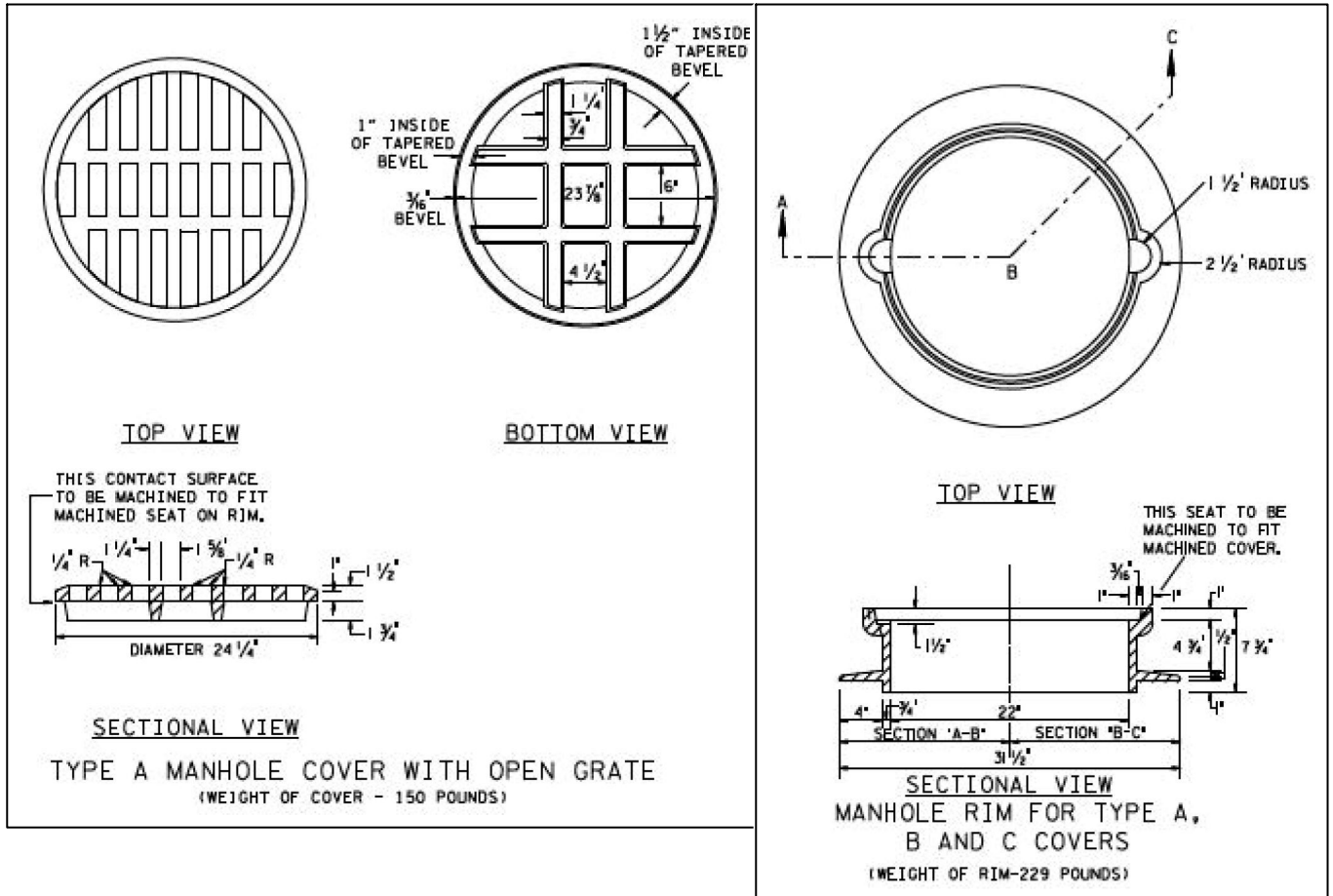
$$\text{quantity } q_{b2} := 2; \text{ unit weight } \gamma_{b2} := 3.4 \text{ plf; length } L_{b2} := \sqrt{(5 \text{ ft})^2 + (3 \text{ ft})^2} = 5.831 \text{ ft};$$
$$\text{weight } W_{b2} := q_{b2} \times \gamma_{b2} \times L_{b2} = 39.65 \text{ lbf}$$

Weight of skimmer plus 1% for miscellaneous items

$$W_{\text{skmr}} := (1 + 1\%) \times (W_{\text{cmp}} + W_a + W_{b1} + W_{b2}) = 0.328 \text{ kip}$$

Manhole cover and frame geometry

Figure 3: Manhole cover and frame details from TDOT standard drawing D-MH-4



Weight of manhole cover

$$W_{mh_cover} := 150\text{lbf}$$

Weight of manhole rim

$$W_{mh_rim} := 229\text{lbf}$$

Total weight of manhole

$$W_{mh} := W_{mh_cover} + W_{mh_rim} = 379\text{lbf}$$

Floatation Stability - Usual Load Condition

Floatation stability evaluation is based on current criteria published in U.S. Army Corps of Engineers (USACE) EM 1110-2-2100, Stability Analysis of Concrete Structures. For floatation stability, the following minimum allowable safety factors for the usual and unusual loading conditions are required.

Table 1: Floatation Stability Minimum Allowable Factor of Safety

<u>Load Condition</u>	<u>Minimum allowable factor of safety</u>
Usual Load Condition	$FS_{FL_u} := 1.3$
Unusual Load Condition	$FS_{FL_un} := 1.2$

Weight of structure $W_s := W_{ftg} + W_{box} + W_{rsr} + W_{skmr} + W_{mh} = 19.117 \text{ kip}$
 Weight of water contained within structure $W_c := 0 \text{ kip}$ (Assume worse case with no water in structure)
 Surcharge loads $S := 0 \text{ kip}$
 Weight of water above top surface of structure $W_G := 0 \text{ kip}$

Calculate uplift force
 Weight of water displaced by foundation $W_{wd_ftg} := \gamma_w \times V_{ftg} = 3.068 \text{ kip}$
 Weight of water displaced by junction box $W_{wd_box} := \gamma_w \times B_{box}^2 \times h_{box} = 8.875 \text{ kip}$
 Weight of water displaced by riser pipe $W_{wd_rsr} := \gamma_w \times \frac{\pi}{4} \times OD_{rsr}^2 \times h_{rsr} = 2.668 \text{ kip}$
 Uplift force $U := W_{wd_ftg} + W_{wd_box} + W_{wd_rsr} = 14.611 \text{ kip}$

Floatation stability safety factor $FS_{FL} := \frac{W_s + W_c + S}{U - W_G} = 1.31$ must be $\geq FS_{FL_u} = 1.3$

Check floatation safety factor for Usual Load Condition

$$\text{checkFS}_{FL} := \begin{cases} \text{"Satisfactory"} & \text{if } FS_{FL} \geq FS_{FL_u} \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$$



Bearing Capacity - Usual Load Condition

The bearing capacity evaluation is based on current criteria published in U.S. Army Corps of Engineers (USACE) EM 1110-2-2100, Stability Analysis of Concrete Structures. For bearing capacity, the following minimum allowable safety factors for the usual and unusual loading conditions are required.

Table 2: Bearing Capacity Minimum Allowable Factor of Safety

Load Condition	Minimum allowable factor of safety
Usual Load Condition	$FS_{BC_u} := 3.0$
Unusual Load Condition	$FS_{BC_un} := 2.6$

Recall foundation soil parameters

Unit weight of foundation soil	$\gamma_s = 105 \text{ pcf}$
Friction angle of foundation soil	$\phi_s = 34^\circ$
Cohesion of foundation soil	$c_s = 0 \text{ psf}$

Recall foundation dimensions

Foundation - width of footing	$B_{ftg} = 6 \text{ ft}$
Foundation - length of footing	$L_{ftg} = 6.667 \text{ ft}$
Foundation - depth of footing	$D_{ftg} = 1.229 \text{ ft}$
Depth to base of foundation	$D := 1.879167 \text{ ft}$
Depth from ground surface to water	$D_w := 0 \text{ ft}$
Inclined load angle	$\theta := 0^\circ$

Bearing Capacity and Correction Factors based on USACE EM 1110-1-1905, Table 4-3

$$N_\phi := \tan^2 \left(45^\circ + \frac{\phi_s}{2} \right) = 3.537$$

$$N_q := N_\phi \times e^{\pi \tan(\phi_s)} = 29.44$$

$$N_c := (N_q - 1) \times \cot(\phi_s) = 42.164$$

$$N_\gamma := (N_q - 1) \times \tan(1.4 \times \phi_s) = 31.146$$

Shape factors

$$\text{cohesion } \zeta_{cs} := 1 + 0.2 \times N_\phi \times \frac{B_{ftg}}{L_{ftg}} = 1.637 ; \text{ wedge } \zeta_{\gamma s} := 1 + 0.1 \times N_\phi \times \frac{B_{ftg}}{L_{ftg}} = 1.318 ;$$

$$\text{surcharge } \zeta_{qs} := 1 + 0.1 \times N_\phi \times \frac{B_{ftg}}{L_{ftg}} = 1.318$$

Bearing Capacity - Usual Load Condition (continued)

Inclined loading factors

$$\text{cohesion } \zeta_{ci} := \frac{\alpha}{\dot{e}} - \frac{\theta}{90^\circ} \frac{\ddot{O}^2}{\phi} = 1 \quad ; \quad \text{wedge } \zeta_{\gamma i} := \frac{\alpha}{\dot{e}} - \frac{\theta}{\phi_s} \frac{\ddot{O}^2}{\phi} = 1 \quad ; \quad \text{surcharge } \zeta_{qi} := \frac{\alpha}{\dot{e}} - \frac{\theta}{\phi_s} \frac{\ddot{O}^2}{\phi} = 1$$

Foundation depth factors

$$\text{cohesion } \zeta_{cd} := 1 + 0.2 \times N_\phi \times \frac{1}{B_{ftg}} \times \frac{D}{2} = 1.118 \quad ; \quad \text{wedge } \zeta_{\gamma d} := 1 + 0.1 \times N_\phi \times \frac{1}{B_{ftg}} \times \frac{D}{2} = 1.059 \quad ;$$

$$\text{surcharge } \zeta_{qd} := 1 + 0.1 \times N_\phi \times \frac{1}{B_{ftg}} \times \frac{D}{2} = 1.059$$

Correction factors

$$\text{cohesion } \zeta_c := \zeta_{cs} \times \zeta_{ci} \times \zeta_{cd} = 1.829 \quad ; \quad \text{wedge } \zeta_\gamma := \zeta_{\gamma s} \times \zeta_{\gamma i} \times \zeta_{\gamma d} = 1.396 \quad ; \quad \text{surcharge } \zeta_q := \zeta_{qs} \times \zeta_{qi} \times \zeta_{qd} = 1.396$$

$$\text{Ultimate bearing capacity} \quad q_u := c_s \times N_c \times \zeta_c + \frac{1}{2} \times B_{ftg} \times (\gamma_s - \gamma_w) \times N_\gamma \times \zeta_\gamma + (\gamma_s - \gamma_w) \times D \times N_q \times \zeta_q = 8.847 \text{ksf}$$

$$\text{Net bearing capacity} \quad q_{net} := q_u - (\gamma_s - \gamma_w) \times D = 8.767 \text{ksf}$$

$$\text{Weight of structure} \quad W_s = 19.117 \text{kip}$$

Weight of water contained within structure (Assume worst case with water filled in structure)

$$\text{Weight of water inside junction box} \quad W_{wc_box} := \gamma_w \times (B_{box} - 2 \times t_{box})^2 \times h_{box} = 4.992 \text{kip}$$

$$\text{Weight of water inside riser} \quad W_{wc_rsr} := \gamma_w \times \frac{\pi}{4} \times D_{rsr}^2 \times h_{rsr} = 1.96 \text{kip}$$

$$\text{Total weight of water contained in structure} \quad W_C := W_{wc_box} + W_{wc_rsr} = 6.952 \text{kip}$$

$$\text{Total applied pressure (ignoring uplift pressure)} \quad q_{applied} := \frac{W_s + W_C}{B_{ftg} \times L_{ftg}} = 0.652 \text{ksf}$$

$$\text{Bearing capacity safety factor} \quad FS_{BC} := \frac{q_{net}}{q_{applied}} = 13.45 \quad \text{must be } \geq FS_{BC_u} = 3$$

Check bearing capacity for Usual Load Condition

$$\text{checkFS}_{BC} := \begin{cases} \text{"Satisfactory"} & \text{if } FS_{BC} \geq FS_{BC_u} \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$$

Sliding Stability- Unusual Load Condition

The sliding stability evaluation is based on current criteria published in U.S. Army Corps of Engineers (USACE) EM 1110-2-2100, Stability Analysis of Concrete Structures. For sliding stability, the following minimum allowable safety factors for the usual and unusual loading conditions are required.

Table 3: Sliding Stability Minimum Allowable Factor of Safety

Load Condition	Minimum allowable factor of safety
Usual Load Condition	$FS_{SL_u} := 2.0$
Unusual Load Condition	$FS_{SL_un} := 1.5$

Calculate lateral drag force on side of riser structure. For submerged condition, the riser is assumed to be subjected to flood water flow.

Assumed maximum flood water velocity $V_{max} := 7 \frac{ft}{s}$

Drag coefficient $C_D := 1.25$ (USACE - drag coefficient not less than 1.25)

Density of water $\rho := 1.937 \frac{slug}{ft^3}$

Height of riser structure with manhole $H_{eff} := H_{structure} + D_{ftg} + 6.75in = 9.292 ft$
 (manhole top is 6.75" above riser top)

Frontal area (face of riser)

$$A_{fa} := B_{ftg} \times D_{ftg} + B_{box} \times h_{box} + OD_{rsr} \times h_{rsr} + 31.5in \times 6.75in = 47.185 ft^2$$

Lateral drag force on riser $F_D := C_D \frac{1}{2} \rho V_{max}^2 \times A_{fa} = 2.799 kip$

Normal force $F_N := W_s + W_C + S + W_G - U = 11.458 kip$

Sliding stability safety factor $FS_{SL} := \frac{c_s \times B_{ftg} \times L_{ftg} + F_N \times \tan(\phi_s)}{F_D} = 2.76$ must be $\geq FS_{SL_un} = 1.5$

Check sliding stability for Unusual Load Condition

$$checkFS_{SL} := \begin{cases} \text{"Satisfactory"} & \text{if } FS_{SL} \geq FS_{SL_un} \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$$



Overturning Stability- Unusual Load Condition

The overturning stability evaluation is based on current criteria published in U.S. Army Corps of Engineers (USACE) EM 1110-2-2100, Stability Analysis of Concrete Structures. For overturning stability, the following criteria for the usual and unusual loading conditions are required.

Table 4: Overturning Stability Acceptance criteria

Load Condition	Acceptance criteria
Usual Load Condition	100% base in compression
Unusual Load Condition	75% base in compression, resultant within base

Overturning moment due to lateral drag force $M_D := F_D \times \frac{H_{eff}}{2} = 13.004 \text{ ft} \cdot \text{kip}$

Eccentricity (resultant from center of foundation) $ecc := \frac{M_D}{F_N} = 1.135 \text{ ft}$

check if eccentricity is within middle 1/3 of foundation $\frac{B_{ftg}}{6} = 1 \text{ ft}$

check_{ecc} := $\begin{cases} \text{"100% base in compression"} & \text{if } ecc \leq \frac{B_{ftg}}{6} \\ \text{"Less than 100% base in compression"} & \text{otherwise} \end{cases} = \text{"Less than 100% base in compression"}$

Sum moments about toe of riser

$$M_O := -M_D + (F_N - W_{skmr}) \times \frac{B_{ftg}}{2} + W_{skmr} \times 8in = 20.606 \text{ ft} \cdot \text{kip}$$

Resultant distance from toe of riser $x_R := \frac{M_O}{F_N} = 1.798 \text{ ft}$

Resultant ratio $x_{R_ratio} := \frac{x_R}{B_{ftg}} = 0.3$

Base area in compression (USACE EM 1110-2-2502, Figure 4-4) $base_c := \min(3 \times x_{R_ratio}, 1) = 90\%$

Check overturning percent base in compression acceptance criteria for Unusual Load Condition

check_{base_c} := $\begin{cases} \text{"Satisfactory"} & \text{if } base_c \geq 75\% \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$

Check overturning resultant acceptance criteria for Unusual Load Condition

check_{rslt} := $\begin{cases} \text{"Satisfactory, resultant within base"} & \text{if } 0ft \leq x_R \leq B_{ftg} \\ \text{"No good, resultant outside base"} & \text{otherwise} \end{cases} = \text{"Satisfactory, resultant within base"}$

Bearing Capacity - Unusual Load Condition

Maximum bearing pressure for $e < \frac{B_{ftg}}{6}$ $Q_{max} := \frac{2 \times F_N}{3 \times L_{ftg} \times \left(\frac{B_{ftg}}{e} - \frac{e}{2} \right)} = 0.614 \text{ ksf}$

Bearing capacity safety factor $FS_{bc} := \frac{q_{net}}{Q_{max}} = 14.27$ must be $\geq FS_{BC_un} = 2.6$

Check bearing capacity for Unusual Load Condition

$$check_{FS_{bc}} := \begin{cases} \text{"Satisfactory"} & \text{if } FS_{bc} \geq FS_{BC_un} \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$$

Check Riser Does Not Tip Over - Unusual Load Condition

Overtuning moment due to lateral drag force $M_{o_LD} := F_D \times \left(\frac{H_{eff}}{e} - D_{ftg} - h_{box} \right) = -4.432 \text{ ft}\cdot\text{kip}$

Resisting moment from weight of riser and manhole $M_r := (W_{mh} + W_{rsr}) \times \frac{OD_{rsr}}{2} = 4.855 \text{ ft}\cdot\text{kip}$

Check riser does not tip over for Unusual Load Condition

$$check_{riser} := \begin{cases} \text{"Satisfactory, riser does not tip over"} & \text{if } M_r \geq |M_{o_LD}| \\ \text{"No Good"} & \text{otherwise} \end{cases} = \text{"Satisfactory, riser does not tip over"}$$

Check Shear and Maximum Moment in Risers - Unusual Load Condition

Define load and resistance factors based on ACI 350 (conservatively use plain concrete values)

shear reduction factor $\phi_c := 0.55$; bending moment reduction factor $\phi_b := 0.55$

load factor for fluids $FL := 1.4$

Maximum factored shear load $V_u := FL \times F_D = 3.919 \text{ kip}$

Shear capacity of riser $V_c := \phi_c \times 0.10 \times f_c \times A_{rsr} = 107.819 \text{ kip}$

check shear capacity $\text{check} V_c := \begin{cases} \text{"Satisfactory"} & \text{if } V_c \geq V_u \\ \text{"No good"} & \text{otherwise} \end{cases}$

Maximum factored moment load $M_u := FL \times M_{o_LD} = -6.205 \text{ ft} \cdot \text{kip}$

Section modulus of riser $S_{x_rsr} := \frac{\frac{\pi}{64} \times (OD_{rsr}^4 - ID_{rsr}^4)}{\frac{OD_{rsr}}{2}} = 4.592 \text{ ft}^3$

Moment capacity in compression $M_{cc} := \phi_b \times f_c \times S_{x_rsr} = 1091.03 \text{ ft} \cdot \text{kip}$

Moment capacity in tension $M_{ct} := \phi_b \times f_t \times S_{x_rsr} = 128.601 \text{ ft} \cdot \text{kip}$

check moment capacity $\text{check} M_c := \begin{cases} \text{"Satisfactory"} & \text{if } \min(M_{cc}, M_{ct}) \geq M_u \\ \text{"No good"} & \text{otherwise} \end{cases}$

Material Properties and Geometry

The material properties and geometry defined below are determined using TVA CCR rule template 257.73(d), existing project drawings, geotechnical data report, historical data, and engineering judgement.

Soil properties

Unit weight of water	$\gamma_{ww} := 62.4 \text{pcf}$
Unit weight of foundation soil	$\gamma_{sw} := 105 \text{pcf}$
Friction angle of foundation soil	$\phi_{sw} := 34^\circ$
Cohesion of foundation soil	$c_{sw} := 0 \text{psf}$

Pipe buckling was analyzed as part of the CCR Rule demonstration. Buckling is caused by excessive vertical loading applied to the pipe through cover and surcharge. Drawing 10W271-307 specifies the pipe to be in accordance with ASTM A252.

Reference AWWA Manual M11 - Steel Pipe - A Guide for Design and Installation, 4th ed.

Apparent modulus of elasticity	$E_{\text{pipe}} := 29000 \text{ksi}$
Steel inner diameter	$ID := 30 \text{in}$
Steel wall thickness	$t_{\text{pipe}} := \frac{3}{8} \text{in}$
Steel outer diameter	$OD := ID + 2t_{\text{pipe}} = 30.75 \text{in}$
Moment of inertia of pipe	$I_{\text{pipe}} := \frac{t_{\text{pipe}}^3}{12} = 4.395 \cdot 10^{-3} \text{in}^3$
Dike crest elevation	$EL_{\text{crest}} := 475 \text{ft}$
Normal pool elevation	$EL_{\text{np}} := 457 \text{ft}$
Flood pool elevation	$EL_{\text{fp}} := 461.6 \text{ft}$
Pipe invert elevation	$EL_{\text{in}} := 455 \text{ft}$
Height of maximum soil cover	$H_{\text{cover}} := EL_{\text{crest}} - (EL_{\text{in}} + OD) = 17.438 \text{ft}$
Height of soil above normal pool	$H_{\text{soil_np}} := \min(H_{\text{cover}}, EL_{\text{crest}} - EL_{\text{np}}) = 17.438 \text{ft}$
Height of soil submerged, normal pool	$H_{\text{submerged_np}} := H_{\text{cover}} - H_{\text{soil_np}} = 0 \text{ft}$
Height of soil above flood pool	$H_{\text{soil_fp}} := \min(H_{\text{cover}}, EL_{\text{crest}} - EL_{\text{fp}}) = 13.4 \text{ft}$
Height of soil submerged, flood pool	$H_{\text{submerged_fp}} := H_{\text{cover}} - H_{\text{soil_fp}} = 4.038 \text{ft}$
Modulus of soil reaction	$E' := 1300 \text{psi}$ (Fine-grained soils relative compaction 90%, AWWA Table 6-1)
Safety factor for design	$FS_{\text{PE}} := 2$

Allowable Buckling - normal pool

Buoyancy factor $R_{b_np} := 1 - 0.33 \times \frac{H_{submerged_np}}{H_{cover}} = 1$

Soil elastic support factor $B' := \frac{1}{1 + 4 \times \frac{-0.065}{ft} \times H_{cover}} = 0.437$

Allowable external pressure for constrained pipe - buckling

$$P_{CA_np} := \frac{1}{FS_{PE}} \times \sqrt{32 \times R_{b_np} \times B' \times E' \times \frac{E_{pipe} \times I_{pipe}}{OD^3}} = 141.158 \text{ psi}$$

Allowable Buckling - flood pool

Buoyancy factor $R_{b_fp} := 1 - 0.33 \times \frac{H_{submerged_fp}}{H_{cover}} = 0.924$

Allowable external pressure for constrained pipe - buckling

$$P_{CA_fp} := \frac{1}{FS_{PE}} \times \sqrt{32 \times R_{b_fp} \times B' \times E' \times \frac{E_{pipe} \times I_{pipe}}{OD^3}} = 135.658 \text{ psi}$$

Calculate Applied Loads

Dead load - usual condition

$$DL_u := \hat{e} \times \gamma_s \times H_{soil_np} + (\gamma_s - \gamma_w) \times H_{submerged_np} \times R_{b_np} + \gamma_w \times H_{submerged_np} = 12.715 \text{ psi}$$

Dead load - unusual condition

$$DL_{un} := \hat{e} \times \gamma_s \times H_{soil_fp} + (\gamma_s - \gamma_w) \times H_{submerged_fp} \times R_{b_fp} + \gamma_w \times H_{submerged_fp} = 11.877 \text{ psi}$$

Live load for AASHTO H20 loading under unpaved roads (AWWA M55, Table 5-3)

1.5 2.0 2.5 3.0 3.5 4.0 6.0 8.0 10.0	cover :=	1.5 2.0 2.5 3.0 3.5 4.0 6.0 8.0 10.0	live _{load} :=	13.9 9.5 7.0 5.4 4.3 3.6 2.0 1.3 0.8	psi
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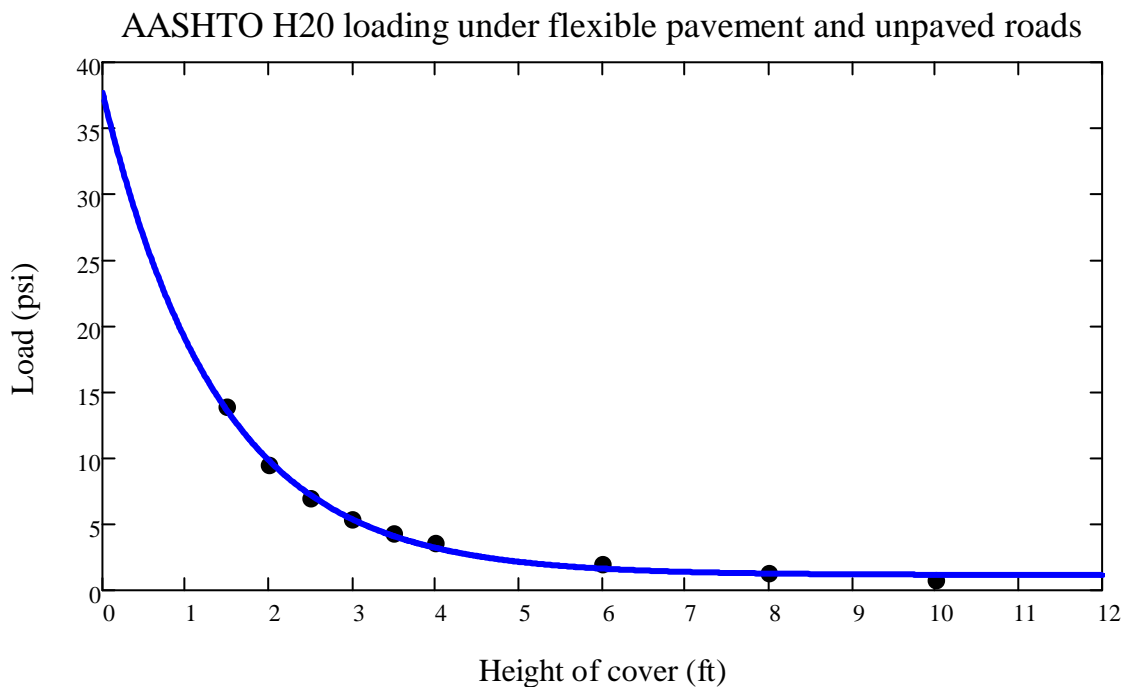
Exponential curve fit

$$efit := \expfit\left(\frac{cover}{ft}, \frac{live_{load}}{psi}\right) = \frac{aa}{e^{bbx}} + cc$$

aa := efit₀ = 36.548 bb := efit₁ = -0.720 cc := efit₂ = 1.203

$$f(x) := aa e^{bbx} + cc = 36.548373145638216 e^{-0.71974758459671384x} + 1.2033362039448283$$

Plot height of cover versus load for unpaved roads



Live load on pipe $LL := f_c \frac{a_1^{cover} \bar{\sigma}}{e \cdot ft \cdot \bar{\sigma}} \text{psi} = 1.203 \text{ psi}$

Total load - usual condition $TL_u := DL_u + LL = 13.918 \text{ psi}$

Total load - unusual condition $TL_{un} := DL_{un} + LL = 13.08 \text{ psi}$

Check critical buckling pressure - usual loading condition

$$\text{check}_{P_{cr_u}} := \begin{cases} \text{"Satisfactory"} & \text{if } P_{CA_np}^3 \cdot TL_u = \text{"Satisfactory"} \\ \text{"No good"} & \text{otherwise} \end{cases}$$

Check critical buckling pressure - unusual loading condition

$$\text{check}_{P_{cr_un}} := \begin{cases} \text{"Satisfactory"} & \text{if } P_{CA_fp}^3 \cdot TL_{un} = \text{"Satisfactory"} \\ \text{"No good"} & \text{otherwise} \end{cases}$$



APPENDIX B

SUDDEN DRAWDOWN ANALYSIS

Stability Analysis

Section C

Ash Pond E

Steady State Slope Stability - Sudden Drawdown

Factor of Safety: 1.80

Minimum Slip Surface Depth: 10 ft

Gallatin Fossil Plant

Tennessee Valley Authority

Date: 9/21/2016
 Method: Spencer
 Entry and Exit
 File Name: GAF_Section_C.gsz

Material Type	Unit Weight	Cohesion	Friction Angle	Cohesion Undrained	Friction Angle Undrained
Pond E Clay Dike	125 pcf	200 psf	28 °	1,500 psf	0 °
Residual Clay	125 pcf	200 psf	27 °	1,000 psf	0 °
Sluiced Bottom Ash	105 pcf	0 psf	30 °	0.1 psf	29.9 °
Sluiced Ash	85 pcf	0 psf	26 °	400 psf	0 °
Pond E 1969 Clay Dike	125 pcf	200 psf	28 °	1,000 psf	0 °

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

