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October 11, 2021

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

Subject: Engineer’s Certification of 2021 Periodic Structural Stability Assessment
Middle Pond A
Tennessee Valley Authority Gallatin Fossil Plant
Gallatin, Tennessee

1.0 PURPOSE

The purpose of this document is to certify that the Tennessee Valley Authority (TVA) Gallatin Fossil Plant (GAF) Middle Pond A is in compliance with the structural stability requirements specified in 40 CFR § 257.73(d) of the United States Environmental Protection Agency (USEPA) Coal Combustion Residuals Rule (CCR Rule). According to 40 CFR § 257.73(f)(3), a periodic structural stability assessment is required five years from the posting of the initial structural stability assessment in the unit’s Operating Record. The initial structural stability assessment was placed in the facility’s operating record on October 14, 2016.

2.0 SUMMARY OF FINDINGS

Annual inspections of CCR facilities at the plant including Middle Pond A have been completed since 2016 with the most recent on July 29, 2021. It was noted that a good stand of grass is generally maintained on the slopes of the perimeter dikes, adequate impoundment freeboard was observed, routine maintenance activities are ongoing including maintaining vegetation, no evidence of sinkholes or depressions were observed, no global slope instability was observed, and outlet structures and drainage pipes were generally in good condition.

Middle Pond A has ceased to receive CCR and non-CCR waste streams. CCR and non-CCR waste streams from the plant are now routed to the Bottom Ash Dewatering Facility and treated prior to discharge through a permitted outfall. The discharge piping between the treatment system and NPDES Outfall 010 passes through a portion of Middle Pond A. The discharge piping between the treatment system and the NPDES Outfall 010 passes through a portion of Middle Pond A. The structural stability assessment has been updated to include evaluation of the discharge piping.

The structural stability of the pipe was evaluated as presented in the attachment, and the results show that the pipes meet the structural stability requirements. There have not been any other changes that would create any additional loading conditions on the perimeter dikes, outlet structures, or drainage pipes evaluated in the initial structural stability assessment. Therefore, Middle Pond A remains in compliance with the structural stability requirements in the CCR Rule.

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3.0 CERTIFICATION

I, David Skeggs, being a Professional Engineer in good standing in the State of Tennessee, do hereby certify, to the best of my knowledge, information, and belief that the information contained in this certification has been prepared in accordance with the accepted practice of engineering; that the information contained herein is accurate as of the date of my signature below; and that Middle Pond A meets the requirements of 40 CFR § 257.73(d).

SIGNATURE: ______________________ **DATE:** October 11, 2021
David Skeggs, PE

ADDRESS: 5438 Wade Park Boulevard, Suite 200
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TELEPHONE: 919-461-1100

ATTACHMENTS: AECOM (2021), *Tennessee Valley Authority - Middle Pond A 2021 Periodic Structural Stability Assessment 40 CFR 257.73(d)(1), Existing CCR Surface Impoundments, TVA Gallatin Fossil Plant (Rev0)*, October 11, 2021.

DAVID ERIC SKEGGS
No. 124294

10/11/2021

COAL COMBUSTION PRODUCT DISPOSAL PROGRAM

Tennessee Valley Authority
Sumner County, Tennessee

2021 Periodic Structural Stability Assessment
40 CFR 257.73(d)(1)
Existing CCR Surface Impoundment – Middle Pond A
TVA Gallatin Fossil Plant

Prepared for

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

October 11, 2021 – Rev 0

Prepared by

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10/11/2021

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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, a periodic structural integrity evaluation is required every five years from the initial structural stability assessment which was placed in the facility’s Operating Record on October 14, 2016. This report addresses the 2021 periodic structural stability assessment for Middle Pond A. The unit meets the conditions of paragraph §257.73(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.

Annual inspections of CCR facilities at the plant including Middle Pond A have been completed since 2016 with the most recent on July 29, 2021 (see reference [2]). It was noted that a good stand of grass is generally maintained on the slopes of the perimeter dikes, adequate impoundment freeboard was observed, routine maintenance activities are ongoing including maintaining vegetation, no evidence of sinkholes or depressions were observed, no global slope instability was observed, and outlet structures and drainage pipes were generally in good condition.

CCR and non-CCR waste streams from the plant are now routed to the Bottom Ash Dewatering Facility and treated prior to discharge through a permitted outfall. The discharge piping between the treatment system and NPDES Outfall 010 passes through a portion of Middle Pond A. Therefore, this structural stability assessment has been updated to include evaluation of the discharge piping.

A plan view showing the location of Middle Pond A is shown in **Figure 1**. Middle Pond A is part of the larger Ash Pond Complex, which consists of Ash Pond A, Ash Pond E, Middle Pond A, and the Bottom Ash Pond.

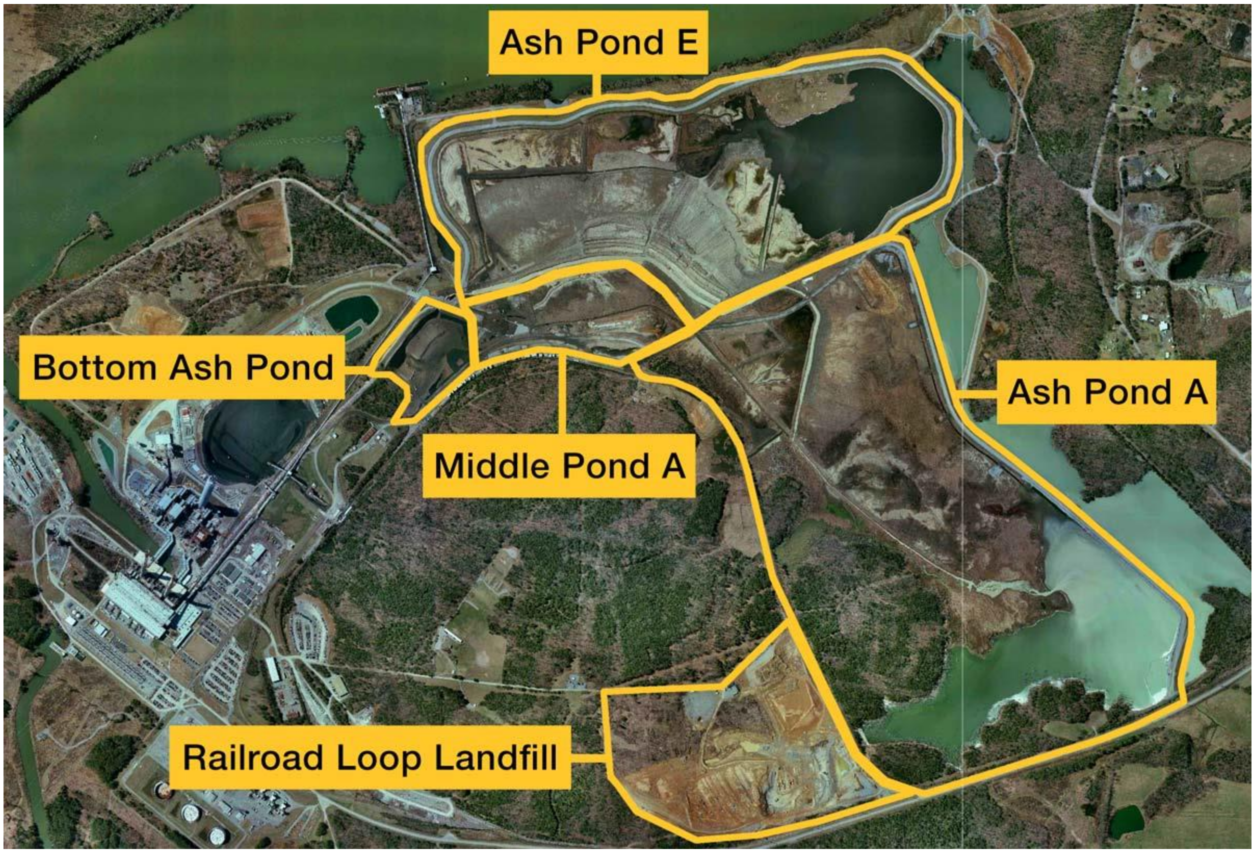

Figure 1: Ash Pond Complex

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 A, Middle Pond A, and Bottom Ash Pond typically consist 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 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 (see reference [1]). Weekly inspections of the facility are performed, and annual inspections from 2016 to 2021 have also been completed, with the most recent on July 29, 2021 (see reference [2]). Based on the assessment and inspection 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.

An assessment of seepage conditions for Middle Pond A 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 , see reference [3]. Seepage analyses were performed at two cross sections across the divider dike 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. The minimum computed factor of safety against piping was recorded at 3.1.

Seepage conditions have been analyzed in accordance with acceptable methodologies. All seepage modeling performed indicated a factor of safety against piping of greater than 3, which exceeds the requirement of 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 and heave potential in comparison to current criteria. Further, no physical or visual evidence of piping, heave, or uplift has been observed during the annual inspections performed from 2016 through 2021.

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

The slopes along the 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 [4] and [5]) indicate that both the original divider dike and the raised dike for Middle Pond A 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 dikes and divider dikes that form Middle Pond A 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)

Middle Pond A does not have any spillway features. Water collected in the unit is discharged through storm water culvert pipes.

According to the periodic inflow design flood control plan review for Middle Pond A (see reference [6]), the RCP culverts vertical spillways are adequate to carry the sustained flows.

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

Based on the periodic hazard assessment review completed in 2021 (see reference [7]), Middle Pond A has been classified as a low hazard potential. Therefore, the combined capacity of the culverts discharging from Middle Pond A must adequately manage flow during and following the peak discharge from a 100-year flood.

According to the periodic inflow design flood control plan review for Middle Pond A (see reference [7]), the capacity of the culverts are adequate to manage the flow during and following the peak discharge from a 100-year flood.

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

CCR and non-CCR waste streams to Middle Pond A have been ceased. Stormwater entering the Bottom Ash Pond travels from the western portion of Bottom Ash Pond through three 48-inch HDPE triple barrel pipes and a single 48” HDPE culvert underneath the existing haul road into the southeast corner of Middle Pond A. Flow in the southeast corner of Middle Pond A is directed through two 48-inch CMPs and a 48-inch HDPE pipe from Middle Pond A through a divider dike into Ash Pond A.

2.6.1 Pipes

Currently the culvert pipes are dry except for during rain events and therefore are able to be visually inspected on a regular basis. They appear to be in good condition. Visual inspections of the dikes where pipes pass through do not show any signs of deformation.

CCR and non-CCR waste streams from the plant are now routed to a Bottom Ash Dewatering Facility (BADW) and treated prior to discharge through a permitted outfall. The treated water is discharged into a 54-inch HDPE pipe which was installed above ground along the railroad tracks adjacent to the northern side of the Bottom Ash Pond. The 54-inch HDPE goes underneath a paved haul road running through in Middle Pond A where it discharges into a junction box and continues through a 63-inch HDPE pipe. The 63-inch HDPE pipe discharges into a concrete lined ditch that directs the treated flow into the Cumberland River via the permitted NPDES Outfall 010. Because the piping runs through Middle Pond A, these pipes have been included in this periodic 2021 assessment.

All 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. Calculations associated with the new discharge pipe, including the structure's geometry and material properties are included in **Appendix A.1**. Calculations completed on existing structures in 2016 have not changed and are provided for reference in **Appendix A.2**. The pipes satisfy the stability checks for the limit states considered. Additionally, visual inspections of the Middle Pond A dike where the pipe passes through does not show any signs of deformation.

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

Middle Pond A will not experience the sudden drawdown condition because the 100-yr. floodplain of the Cumberland River does not reach the toe of the downstream dike.

3.0 Conclusion

Based on the results of this 2021 periodic structural stability assessment, Middle Pond A meets the requirements of the CCR Rule as discussed in **Section 2.0**. No structural deficiencies have been noted during annual inspections, and conditions of the units have been improved with the removal of CCR and non-CCR waste streams from the unit. Discharge piping installed to convey treated wastewater to Outfall 010 has been incorporated into the assessment and satisfy the stability checks for the limit states considered.

4.0 References

[1] Dewberry Consultants LLC, "Coal Combustion Residue Impoundment, Round 11 - Dam Assessment Report," April 2013.

[2] TVA, "2020 Annual Inspection of CCR facilites at Tennessee Valley Authority's (TVA's) Gallatin Fossil Plant (GAF)," December 18, 2020.

[3] AECOM, "Geotechnical Exploration and Analysis, CCR Rule Compliance (Rev. 0), Bottom Ash Pond and Middle Pond A," October 2016.

[4] Tennessee Valley Authority, "Construction Drawing 10N 273-02 R1," November 7, 1988.

[5] Tennessee Valley Authority, "Construction Drawing 10N274 R2," June 23, 1986.

[6] AECOM, "2021 Periodic Inflow Design Flood Control Plan; Middle Pond A; Gallatin Fossil Plant; Sumner County, Tennessee," October 11, 2021.

[7] AECOM, "2021 Periodic Hazard Potential Classification Assessment; Middle Pond A; Gallatin Fossil Plant; Sumner County, Tennessee," October 11, 2021.

[8] USACE, "USACE EM 1110-2-1902 Slope Stability," October 31, 2003.

[9] USBR, "Design Standard No. 13: Embankment Dams," January 2014.

[10] K. Terzaghi, R. B. Peck and G. Mesri, Soil Mechanics in Engineering Practice, 3rd Edition, New York: John Wiley & Sons, Inc., 1996.

APPENDIX A.1

HYDRAULIC STRUCTURES ASSESSMENT CALCULATIONS (BADW DISCHARGE PIPE)

2021 Periodic Structural Stability Assessment for Pipe Structures in Middle Pond A at TVA Gallatin Fossil Plant

Prepared for

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Prepared by

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SIPHAY D. DOUANGVILAY
No. 113763
Oct 15 2021

Discussion

The following calculations detail the structural stability assessment for the existing Interim Flow Management pipe structures in Middle Pond A at Tennessee Valley Authority (TVA) Gallatin Fossil Plant (GAF). CCR and non-CCR waste streams are routed to the Bottom Ash Dewatering Facility (BADW) and flows through the settling tank/polishing tank to be treated and then flows through a 54-inch HDPE pipe through Middle Pond A and a portion of Ash Pond E to a permitted outfall. The 54-inch HDPE was installed at the surface and was covered with a stone mound. The 54-inch connects to a 63-inch HDPE pipe via a manhole structure that is outside the limits of the CCR units. The following calculations detail the structural pipe stability of the 54-inch HDPE pipe.

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.) AECOM Gallatin Interim Flow Management Project Drawing No. 10W262-012 dated December 27, 2018.
2.) AWWA M55 - PE Pipe Design and Installation, January 1, 2006.

Material Properties and Geometry

The material properties and geometry defined below are determined using 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	$\phi_s := 34°$
Cohesion of foundation soil	$c_s := 0\text{psf}$

54-inch HDPE through Middle Pond A

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 loads. The buckling analysis was performed for the existing 54-inch outer diameter HDPE pipe. The calculations below references AWWA M55 - PE Pipe - Design and Installation for the HDPE pipes.

Apparent modulus of elasticity	$E_{pipe} := 28250\text{psi}$ (AWWA M55, Table 5-6, long term)
HDPE outside diameter	$OD := 54\text{in}$
HDPE wall thickness	$t_{pipe} := 2.077\text{in}$
HDPE inside diameter	$ID := OD - 2 \cdot t_{pipe} = 49.846 \cdot \text{in}$
Dimension ratio	$DR := \frac{OD}{t_{pipe}} = 26$
Dike crest elevation	$EL_{crest} := 489\text{ft}$
Normal pool elevation	$EL_{np} := 468.3\text{ft}$
Flood pool elevation	$EL_{fp} := 473\text{ft}$
Pipe invert elevation	$EL_{in} := 480\text{ft}$
Height of maximum soil cover	$H_{cover} := EL_{crest} - \left(EL_{in} + OD - t_{pipe}\right) = 4.673\,\text{ft}$
Height of soil above normal pool	$H_{soil\_np} := \min\left(H_{cover}, EL_{crest} - EL_{np}\right) = 4.673\,\text{ft}$
Height of soil submerged, normal pool	$H_{submerged\_np} := H_{cover} - H_{soil\_np} = 0\,\text{ft}$
Height of soil above flood pool	$H_{soil\_fp} := \min\left(H_{cover}, EL_{crest} - EL_{fp}\right) = 4.673\,\text{ft}$
Height of soil submerged, flood pool	$H_{submerged\_fp} := H_{cover} - H_{soil\_fp} = 0\,\text{ft}$
Modulus of soil reaction	$E' := 1300\text{psi}$ (Fine-grained soils relative compaction 90%, AWWA Table 6-1)
Safety factor for design	$FS_{PE} := 2$

Allowable Buckling - normal pool

Buoyancy factor $$R_{b\_np} := 1 - 0.33 \cdot \frac{H_{submerged\_np}}{H_{cover}} = 1$$

Soil elastic support factor $$B' := \frac{1}{1 + 4 \cdot e^{\frac{-0.065}{ft} \cdot H_{cover}}} = 0.253$$

Allowable external pressure for constrained pipe - buckling

$$P_{CA\_np} := \frac{5.65}{FS_{PE}} \cdot \sqrt{R_{b\_np} \cdot B' \cdot E' \cdot \frac{E_{pipe}}{12 \cdot (DR - 1)^3}} = 19.889\,psi$$

Allowable Buckling - flood pool

Buoyancy factor $$R_{b\_fp} := 1 - 0.33 \cdot \frac{H_{submerged\_fp}}{H_{cover}} = 1$$

Allowable external pressure for constrained pipe - buckling

$$P_{CA\_fp} := \frac{5.65}{FS_{PE}} \cdot \sqrt{R_{b\_fp} \cdot B' \cdot E' \cdot \frac{E_{pipe}}{12 \cdot (DR - 1)^3}} = 19.889\,psi$$

Calculate Applied Loads

Dead load - usual condition

$$DL_u := \left[\gamma_s \cdot H_{soil\_np} + \left(\gamma_s - \gamma_w\right) \cdot H_{submerged\_np}\right] \cdot R_{b\_np} + \gamma_w \cdot H_{submerged\_np} = 3.407\,psi$$

Dead load - unusual condition

$$DL_{un} := \left[\gamma_s \cdot H_{soil\_fp} + \left(\gamma_s - \gamma_w\right) \cdot H_{submerged\_fp}\right] \cdot R_{b\_fp} + \gamma_w \cdot H_{submerged\_fp} = 3.407\,psi$$

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

$$\text{cover} := \begin{pmatrix} 1.5 \\ 2.0 \\ 2.5 \\ 3.0 \\ 3.5 \\ 4.0 \\ 6.0 \\ 8.0 \\ 10.0 \end{pmatrix} \text{ft} \qquad \text{live}_{\text{load}} := \begin{pmatrix} 13.9 \\ 9.5 \\ 7.0 \\ 5.4 \\ 4.3 \\ 3.6 \\ 2.0 \\ 1.3 \\ 0.8 \end{pmatrix} \text{psi}$$

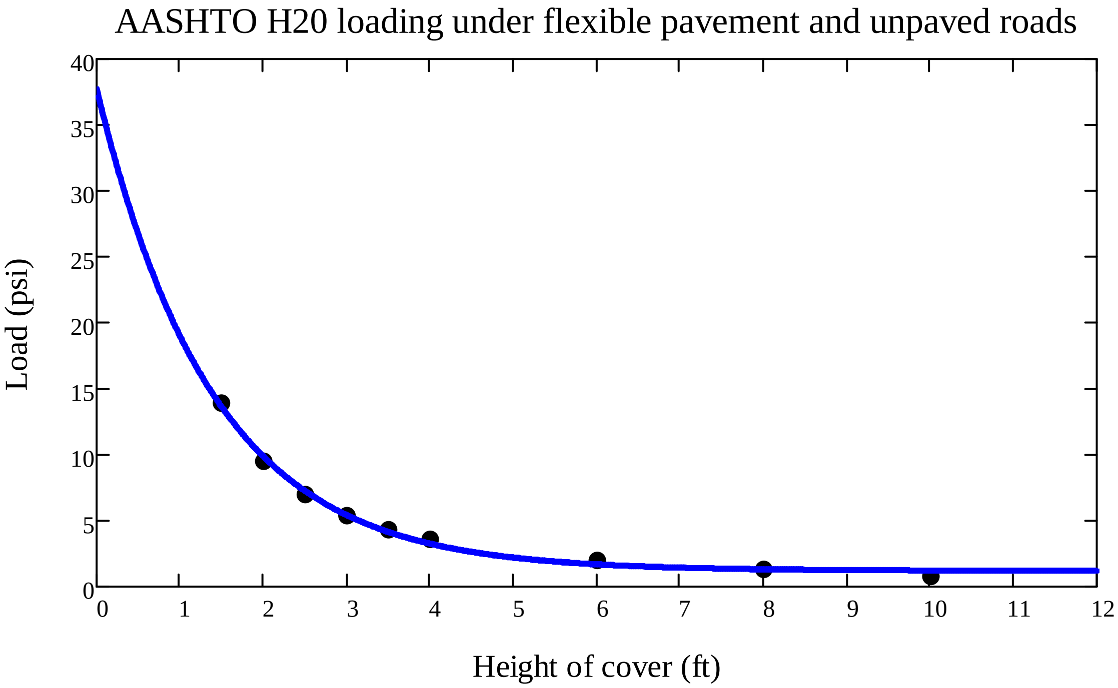
Exponential curve fit

$$\text{efit} := \text{expfit}\left(\frac{\text{cover}}{\text{ft}}, \frac{\text{live}_{\text{load}}}{\text{psi}}\right) = \begin{pmatrix} 36.548 \\ -0.72 \\ 1.203 \end{pmatrix}$$

$$\text{aa} := \text{efit}_0 = 36.548 \qquad \text{bb} := \text{efit}_1 = -0.720 \qquad \text{cc} := \text{efit}_2 = 1.203$$

$$f(x) := \text{aa}\cdot e^{\text{bb}\cdot x} + \text{cc} \rightarrow 36.548373145638216\cdot e^{-0.71974758459671384\cdot x} + 1.2033362039448283$$

Plot height of cover versus load for unpaved roads



Live load on pipe $LL := f\left(\frac{H_{cover}}{ft}\right)\cdot psi = 2.468\,psi$

Total load - usual condition $TL_u := DL_u + LL = 5.876\,psi$

Total load - unusual condition $TL_{un} := DL_{un} + LL = 5.876\,psi$

Check critical buckling pressure - usual loading condition

$$check_{Pcr\_u} := \left| \begin{array}{l} \text{"Satisfactory"} \ \text{ if } \ P_{CA\_np} \geq TL_u \\ \text{"No good"} \ \text{ otherwise} \end{array} \right. = \text{"Satisfactory"}$$

Check critical buckling pressure - unusual loading condition

$$check_{Pcr\_un} := \left| \begin{array}{l} \text{"Satisfactory"} \ \text{ if } \ P_{CA\_fp} \geq TL_{un} \\ \text{"No good"} \ \text{ otherwise} \end{array} \right. = \text{"Satisfactory"}$$

The following calculations detail pipe structural stability assuming construction equipment travels over the pipe.

Outside pipe diameter $D_o := 54in$ (54" DR 26 PE 4710 HDPE)

Minimum pipe thickness $t_{pipe} := 2.077in$

Average Inner pipe diameter $D_i := 49.597in$

Standard dimension ratio of pipe $SDR := 26$

In-place density of cover material $\gamma_p := 125pcf$ (Compacted unclassified structural fill)

Maximum soil cover $H_f := 5ft$ (max. depth of embedment)

Minimum soil cover underneath equipment $H_c := 2.25ft$

Operating weight of dozer $W_{dozer} := 87600lbf$

Approximate ground contact area $A_{gc} := 5554in^2$

Live load pressure for Equipment Loading $P_q := \frac{W_{dozer}}{A_{gc}} = 15.772\,psi$

Time lag factor $T_L := 1.5$

Pressure applied from equipment $P_L := T_L \cdot \gamma_p \cdot H_c + P_q = 18.702\,psi$

Maximum surcharge load $P_t := T_L \cdot \gamma_p \cdot H_f = 6.51 \cdot psi$

Maximum pressure from embankment loading and/or equipment loading $P_{max} := \max(P_t, P_L) = 18.702\,psi$

WALL CRUSHING

Pipe wall compressive stress $\sigma_c := \frac{P_{max} \cdot (SDR - 1)}{2} = 233.776 \cdot psi$ (AWWA Eq 5-15)

Allowable compressive yield strength of pipe $\sigma_y := 1150psi$ (PPI Table C.1, PE 4710 for 73°F)

Check allowable compressive strength of pipe $check_{wc} := \left| \begin{array}{l} \text{"Satisfactory"} \text{ if } \sigma_c \le \sigma_y \\ \text{"No good"} \text{ otherwise} \end{array} \right. = \text{"Satisfactory"}$

WALL BUCKLING

Modulus of soil reaction for pipe bedding $E' := 1300psi$ (conservatively assumed based on stone or crushed aggregate bedding with moderate compaction)

Long-term elastic modulus of pipe $E_{pipe} := 28200psi$ (ISCO, 50 yr for 73°F)

Groundwater height above pipe $H_w := 0ft$ (assumed top of ground)

Buoyancy reduction factor $R_b := 1 - \left(0.33 \cdot \frac{H_w}{H_f}\right) = 1$ (AWWA Eq 5-11)

Elastic support factor $B' := \frac{1}{1 + 4 \cdot e^{\frac{-0.065}{ft} \cdot H_f}} = 0.257$ (AWWA Eq 5-12)

Critical constrained wall buckling pressure $P_{wb} := 5.65 \cdot \sqrt{R_b \cdot B' \cdot E' \cdot \frac{E_{pipe}}{12 \cdot (SDR - 1)^3}} = 40.056\,psi$ (AWWA Eq 5-10)

Safety factor against wall buckling $FS_{wb} := \frac{P_{wb}}{P_{max}} = 2.142$

Check safety factor against wall buckling $check_{wb} := \left| \begin{array}{l} \text{"Satisfactory"} \text{ if } FS_{wb} \ge 2.0 \\ \text{"No good"} \text{ otherwise} \end{array} \right. = \text{"Satisfactory"}$

BENDING STRAIN DUE TO RING DEFLECTION

Bedding constant $K_b := 0.1$

"Short term" apparent elastic modulus of pipe $E_{pipe\_s} := 110000\text{psi}$ (ISCO for 73°F)

Percent ring deflection

$$\Delta Y\% := \frac{P_{max} \cdot K_b}{\dfrac{2 \cdot E_{pipe\_s}}{3 \cdot (SDR - 1)^3} + 0.061 \cdot E'} = 2.227 \cdot \%$$

(AWWA Eq 5-8)

Check ring deflection

$$check_{\Delta Y\%} := \left| \begin{array}{l} \text{"Satisfactory" if } \Delta Y\% \le 7.5\% \\ \text{"No good" otherwise} \end{array} \right. = \text{"Satisfactory"}$$

For non-pressurized pipes, a 7.5% deflection limit provides a large safety factor against instability and is considered a safe design deflection. The percent ring deflection is less than this deflection limit.

APPENDIX A.2

HYDRAULIC STRUCTURES ASSESSMENT CALCULATIONS (2016 PIPE EVALUATION)

Structural Stability Assessment for Pipe Structures in Middle Pond A and Bottom Ash Pond at TVA Gallatin Fossil Plant

Prepared for

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OCT-12-2016

Discussion

The following calculations detail the structural stability assessment for the existing pipe structures in Bottom Ash Pond and Middle Pond A at Tennessee Valley Authority (TVA) Gallatin Fossil Plant (GAF). Bottom ash is sluiced into the southern portion of Bottom Ash Pond. The bottom ash settles in the Bottom Ash Pond. Process flows travel from the western portion of Bottom Ash Pond through three 48-inch RCPs at the northwest corner into Middle Pond A and from the eastern portion through a 36-inch RCP and a 48-inch corrugated metal pipe (CMP) into the southeast corner of Middle Pond A. Flow in the southeast corner of Middle Pond A is directed through a 48” RCP through the existing haul road and, thereafter, through two 48-inch CMPs and a 48-inch HDPE pipe from Middle Pond A through a divider dike into Ash Pond A. Then, the process flow is routed through a siphon system consisting of six, 18-inch diameter HDPE pipes from Ash Pond A to Stilling Pond B. Water from Ash Pond A is drawn through the submerged inlet of each siphon, lifted over the divider dike by siphon action, and is discharged downstream into Stilling Pond B. The siphon system operates as the primary spillway during normal operating conditions. During small to moderate storm events, the siphon operator may adjust the flow rate through the siphon system to pass the storm event using only the siphons. During large storm events, the three RCP spillway risers, which discharge through three 30-inch RCP into Stilling Pond B, will begin to pass flow along with the siphon system.

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).

The CMPs were checked for adequate cover above the pipes. Manufacturer literature requires minimum 12 inches of cover and provides maximum cover of 48-inch CMP to be about 62 feet. The CMPs in both Bottom Ash Pond and Middle Pond A have adequate cover.

References

1.) TVA-CCR Rule Template 257.73 (d).
2.) URS Ash Haul Road A Drawing No. 10W263-07 dated September 26, 2014.
3.) AWWA M55 - PE Pipe Design and Installation, January 1, 2006.

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	$\phi_s := 34°$
Cohesion of foundation soil	$c_s := 0\text{psf}$

Concrete properties

Compressive strength	$f_c := 3000\text{psi}$
Static tensile strength	$f_t := 1.7\text{psi}\cdot\left(\frac{f_c}{\text{psi}}\right)^{\frac{2}{3}} = 353.614\cdot\text{psi}$

48-inch RCP Northwest corner of Bottom Ash Pond to Middle Pond A

Check Shear and Maximum Moment in Pipes - Usual Condition

Inner diameter of pipe, $ID_p := 48in$ Wall thickness of pipe, $t_{wp} := 5.75in$

Outer diameter of pipe, $OD_p := ID_p + 2 \cdot t_{wp} = 59.5 \cdot in$ Area of pipe, $A_p := \frac{\pi}{4} \cdot \left(OD_p^{\ 2} - ID_p^{\ 2}\right) = 6.743\,ft^2$

Unit weight of dike embankment, $\gamma_{emb} := 120pcf$ Height of dike embankment, $H_{emb} := 25ft$

Depth from top of dike to water level, $dw := 12ft$ (top of dike at EL 490.5', normal pool at EL 478.5')

Vertical arching factor, $VAF := 1.45$ Effective length of pipe, $L_e := 1ft$

Load due to earth pressure, $W_e := VAF \cdot \gamma_{emb} \cdot OD_p \cdot H_{emb} = 21.569 \cdot \frac{kip}{ft}$

Force from earth pressure, $F_{W\_e} := W_e \cdot L_e = 21.569 \cdot kip$

Load due to water, $W_D := \gamma_w \cdot OD_p \cdot \left(H_{emb} - dw\right) = 4.022 \cdot \frac{kip}{ft}$

Force from water, $F_{D\_p} := W_D \cdot L_e = 4.022 \cdot kip$

Moment load due to earth pressure, $M_{o\_Fwe} := \frac{W_e \cdot L_e^{\ 2}}{8} = 2.696 \cdot ft \cdot kip$

Momend load due to water, $M_{o\_Fdp} := \frac{W_D \cdot L_e^{\ 2}}{8} = 0.503 \cdot ft \cdot kip$

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 $\phi_b := 0.55$

Maximum factored shear load $V_{u\_p} := \max\left(1.4 \cdot F_{D\_p}, 1.2 \cdot F_{D\_p} + 1.6 \cdot F_{W\_e}\right) = 39.337 \cdot kip$

Shear capacity of pipe $V_{c\_p} := \phi_c \cdot 0.10 \cdot f_c \cdot A_p = 160.206 \cdot kip$

check shear capacity

$$checkV_{c\_p} := \begin{array}{|l} \text{"Satisfactory"} \ \text{ if } \ V_{c\_p} \geq V_{u\_p} \\ \text{"No good"} \ \text{ otherwise} \end{array} = \text{"Satisfactory"}$$

Maximum factored moment load $M_{u\_p} := \max\left(1.4 \cdot M_{o\_Fdp}, 1.2 \cdot M_{o\_Fdp} + 1.6 \cdot M_{o\_Fwe}\right) = 4.917 \cdot ft \cdot kip$

Section modulus of riser

$$S_{x\_p} := \frac{\frac{\pi}{64} \cdot \left(OD_p^{\ 4} - ID_p^{\ 4}\right)}{\frac{OD_p}{2}} = 6.899 \cdot ft^3$$

Moment capacity in compression $M_{cc\_p} := \phi_b \cdot f_c \cdot S_{x\_p} = 1639.158 \cdot ft \cdot kip$

Moment capacity in tension $M_{ct\_p} := \phi_b \cdot f_t \cdot S_{x\_p} = 193.21 \cdot ft \cdot kip$

check moment capacity

$$checkM_{c\_p} := \begin{array}{|l} \text{"Satisfactory"} \ \text{ if } \ \min\left(M_{cc\_p}, M_{ct\_p}\right) \geq M_{u\_p} \\ \text{"No good"} \ \text{ otherwise} \end{array} = \text{"Satisfactory"}$$

Check Shear and Maximum Moment in Pipes - Unusual Condition

Depth from top of dike to water level, $dw\_un := 10.2ft$ (top of dike at EL 490.5', flood pool at EL 480.3')

Load due to water, $W_{D\_un} := \gamma_w \times OD_p \times (H_{emb} - dw\_un) = 4.579 \times \frac{kip}{ft}$

Force from water, $F_{D\_p\_un} := W_{D\_un} \times L_e = 4.579 \times kip$

Moment load due to earth pressure, $M_{o\_Fwe} = 2.696 \times ft \times kip$

Momend load due to water, $M_{o\_Fdp\_un} := \frac{W_{D\_un} \times L_e^{2}}{8} = 0.572 \times ft \times kip$

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 $\phi_b = 0.55$

Maximum factored shear load $V_{u\_p\_un} := \max(1.4 \times F_{D\_p\_un}, 1.2 \times F_{D\_p\_un} + 1.6 \times F_{W\_e}) = 40.005 \times kip$

Shear capacity of pipe $V_{c\_p} = 160.206 \times kip$

check shear capacity

$$checkV_{c\_p\_un} := \left| \begin{array}{l} \text{"Satisfactory" if } V_{c\_p} \text{ ³ } V_{u\_p\_un} \\ \text{"No good" otherwise} \end{array} \right. = \text{"Satisfactory"}$$

Maximum factored moment load

$$M_{u\_p\_un} := \max(1.4 \times M_{o\_Fdp\_un}, 1.2 \times M_{o\_Fdp\_un} + 1.6 \times M_{o\_Fwe}) = 5.001 \times ft \times kip$$

Moment capacity in compression $M_{cc\_p} = 1639.158 \times ft \times kip$

Moment capacity in tension $M_{ct\_p} = 193.21 \times ft \times kip$

check moment capacity

$$checkM_{c\_p\_un} := \left| \begin{array}{l} \text{"Satisfactory" if } \min(M_{cc\_p}, M_{ct\_p}) \text{ ³ } M_{u\_p\_un} \\ \text{"No good" otherwise} \end{array} \right. = \text{"Satisfactory"}$$

36-inch RCP Northeast corner of Bottom Ash Pond to Middle Pond A

Check Shear and Maximum Moment in Pipes - Usual Condition

Inner diameter of pipe, $ID_p := 36in$ Wall thickness of pipe, $t_{wp} := 4.75in$

Outer diameter of pipe, $OD_p := ID_p + 2\cdot t_{wp} = 45.5\cdot in$ Area of pipe, $A_p := \frac{\pi}{4}\cdot\left(OD_p^2 - ID_p^2\right) = 4.223\,ft^2$

Unit weight of dike embankment, $\gamma_{emb} := 120pcf$ Height of dike embankment, $H_{emb} := 25ft$

Depth from top of dike to water level, $dw := 4.5ft$ (top of dike at EL 482.5', normal pool at EL 478')

Vertical arching factor, $VAF := 1.45$ Effective length of pipe, $L_e := 1ft$

Load due to earth pressure, $W_e := VAF\cdot\gamma_{emb}\cdot OD_p\cdot H_{emb} = 16.494\cdot\frac{kip}{ft}$

Force from earth pressure, $F_{W\_e} := W_e\cdot L_e = 16.494\cdot kip$

Load due to water, $W_D := \gamma_w\cdot OD_p\cdot\left(H_{emb} - dw\right) = 4.85\cdot\frac{kip}{ft}$

Force from water, $F_{D\_p} := W_D\cdot L_e = 4.85\cdot kip$

Moment load due to earth pressure, $M_{o\_Fwe} := \frac{W_e\cdot L_e^2}{8} = 2.062\cdot ft\cdot kip$

Momend load due to water, $M_{o\_Fdp} := \frac{W_D\cdot L_e^2}{8} = 0.606\cdot ft\cdot kip$

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 $\phi_b := 0.55$

Maximum factored shear load $V_{u\_p} := \max\left(1.4\cdot F_{D\_p}, 1.2\cdot F_{D\_p} + 1.6\cdot F_{W\_e}\right) = 32.21\cdot kip$

Shear capacity of pipe $V_{c\_p} := \phi_c\cdot 0.10\cdot f_c\cdot A_p = 100.336\cdot kip$

check shear capacity $checkV_{c\_p} := \left|\begin{array}{l}\text{"Satisfactory"} \text{ if } V_{c\_p} \ge V_{u\_p} \\ \text{"No good"} \text{ otherwise}\end{array}\right. = \text{"Satisfactory"}$

Maximum factored moment load $M_{u\_p} := \max\left(1.4\cdot M_{o\_Fdp}, 1.2\cdot M_{o\_Fdp} + 1.6\cdot M_{o\_Fwe}\right) = 4.026\cdot ft\cdot kip$

Section modulus of riser $S_{x\_p} := \frac{\frac{\pi}{64}\cdot\left(OD_p^4 - ID_p^4\right)}{\frac{OD_p}{2}} = 3.254\cdot ft^3$

Moment capacity in compression $M_{cc\_p} := \phi_b\cdot f_c\cdot S_{x\_p} = 773.248\cdot ft\cdot kip$

Moment capacity in tension $M_{ct\_p} := \phi_b\cdot f_t\cdot S_{x\_p} = 91.144\cdot ft\cdot kip$

check moment capacity

$$checkM_{c\_p} := \left|\begin{array}{l}\text{"Satisfactory"} \text{ if } \min\left(M_{cc\_p}, M_{ct\_p}\right) \ge M_{u\_p} \\ \text{"No good"} \text{ otherwise}\end{array}\right. = \text{"Satisfactory"}$$

Check Shear and Maximum Moment in Pipes - Unusual Condition

Depth from top of dike to water level, $dw\_un := 2.2\,ft$ (top of dike at EL 482.5', flood pool at EL 480.3')

Load due to water, $W_{D\_un} := \gamma_w \cdot OD_p \cdot \left(H_{emb} - dw\_un\right) = 5.394 \cdot \frac{kip}{ft}$

Force from water, $F_{D\_p\_un} := W_{D\_un} \cdot L_e = 5.394 \cdot kip$

Moment load due to earth pressure, $M_{o\_Fwe} = 2.062 \cdot ft \cdot kip$

Momend load due to water, $M_{o\_Fdp\_un} := \dfrac{W_{D\_un} \cdot L_e^{\,2}}{8} = 0.674 \cdot ft \cdot kip$

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 $\phi_b = 0.55$

Maximum factored shear load $V_{u\_p\_un} := \max\left(1.4 \cdot F_{D\_p\_un}, 1.2 \cdot F_{D\_p\_un} + 1.6 \cdot F_{W\_e}\right) = 32.863 \cdot kip$

Shear capacity of pipe $V_{c\_p} = 100.336 \cdot kip$

check shear capacity

$$checkV_{c\_p\_un} := \left| \begin{array}{l} \text{"Satisfactory"} \ \text{ if } \ V_{c\_p} \ge V_{u\_p\_un} \\ \text{"No good"} \ \text{ otherwise} \end{array} \right. = \text{"Satisfactory"}$$

Maximum factored moment load

$$M_{u\_p\_un} := \max\left(1.4 \cdot M_{o\_Fdp\_un}, 1.2 \cdot M_{o\_Fdp\_un} + 1.6 \cdot M_{o\_Fwe}\right) = 4.108 \cdot ft \cdot kip$$

Moment capacity in compression $M_{cc\_p} = 773.248 \cdot ft \cdot kip$

Moment capacity in tension $M_{ct\_p} = 91.144 \cdot ft \cdot kip$

check moment capacity

$$checkM_{c\_p\_un} := \left| \begin{array}{l} \text{"Satisfactory"} \ \text{ if } \ \min\left(M_{cc\_p}, M_{ct\_p}\right) \ge M_{u\_p\_un} \\ \text{"No good"} \ \text{ otherwise} \end{array} \right. = \text{"Satisfactory"}$$

48-inch RCP Southeast corner of Middle Pond A to Middle Pond A

Check Shear and Maximum Moment in Pipes - Usual Condition

Inner diameter of pipe, $ID_p := 48in$ Wall thickness of pipe, $t_{wp} := 5.75in$

Outer diameter of pipe, $OD_p := ID_p + 2 \cdot t_{wp} = 59.5 \cdot in$ Area of pipe, $A_p := \frac{\pi}{4} \cdot \left(OD_p{}^2 - ID_p{}^2\right) = 6.743\,ft^2$

Unit weight of dike embankment, $\gamma_{emb} := 120pcf$ Height of dike embankment, $H_{emb} := 25ft$

Depth from top of dike to water level, $dw := 21.7ft$ (top of dike at EL 490', normal pool at EL 468.3')

Vertical arching factor, $VAF := 1.45$ Effective length of pipe, $L_e := 1ft$

Load due to earth pressure, $W_e := VAF \cdot \gamma_{emb} \cdot OD_p \cdot H_{emb} = 21.569 \cdot \frac{kip}{ft}$

Force from earth pressure, $F_{W\_e} := W_e \cdot L_e = 21.569 \cdot kip$

Load due to water, $W_D := \gamma_w \cdot OD_p \cdot \left(H_{emb} - dw\right) = 1.021 \cdot \frac{kip}{ft}$

Force from water, $F_{D\_p} := W_D \cdot L_e = 1.021 \cdot kip$

Moment load due to earth pressure, $M_{o\_Fwe} := \frac{W_e \cdot L_e{}^2}{8} = 2.696 \cdot ft \cdot kip$

Momend load due to water, $M_{o\_Fdp} := \frac{W_D \cdot L_e{}^2}{8} = 0.128 \cdot ft \cdot kip$

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 $\phi_b := 0.55$

Maximum factored shear load $V_{u\_p} := \max\left(1.4 \cdot F_{D\_p}, 1.2 \cdot F_{D\_p} + 1.6 \cdot F_{W\_e}\right) = 35.735 \cdot kip$

Shear capacity of pipe $V_{c\_p} := \phi_c \cdot 0.10 \cdot f_c \cdot A_p = 160.206 \cdot kip$

check shear capacity $checkV_{c\_p} := \left| \begin{array}{l} \text{"Satisfactory" if } V_{c\_p} \ge V_{u\_p} \\ \text{"No good" otherwise} \end{array} \right. = \text{"Satisfactory"}$

Maximum factored moment load $M_{u\_p} := \max\left(1.4 \cdot M_{o\_Fdp}, 1.2 \cdot M_{o\_Fdp} + 1.6 \cdot M_{o\_Fwe}\right) = 4.467 \cdot ft \cdot kip$

Section modulus of riser $S_{x\_p} := \frac{\frac{\pi}{64} \cdot \left(OD_p{}^4 - ID_p{}^4\right)}{\frac{OD_p}{2}} = 6.899 \cdot ft^3$

Moment capacity in compression $M_{cc\_p} := \phi_b \cdot f_c \cdot S_{x\_p} = 1639.158 \cdot ft \cdot kip$

Moment capacity in tension $M_{ct\_p} := \phi_b \cdot f_t \cdot S_{x\_p} = 193.21 \cdot ft \cdot kip$

check moment capacity

$$checkM_{c\_p} := \left| \begin{array}{l} \text{"Satisfactory" if } \min\left(M_{cc\_p}, M_{ct\_p}\right) \ge M_{u\_p} \\ \text{"No good" otherwise} \end{array} \right. = \text{"Satisfactory"}$$

Check Shear and Maximum Moment in Pipes - Unusual Condition

Depth from top of dike to water level, $dw\_un := 10.4ft$ (top of dike at EL 490', flood pool at EL 479.6')

Load due to water, $W_{D\_un} := \gamma_w \cdot OD_p \cdot (H_{emb} - dw\_un) = 4.517 \cdot \frac{kip}{ft}$

Force from water, $F_{D\_p\_un} := W_{D\_un} \cdot L_e = 4.517 \cdot kip$

Moment load due to earth pressure, $M_{o\_Fwe} = 2.696 \cdot ft \cdot kip$

Momend load due to water, $M_{o\_Fdp\_un} := \frac{W_{D\_un} \cdot L_e^{\;2}}{8} = 0.565 \cdot ft \cdot kip$

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 $\phi_b = 0.55$

Maximum factored shear load $V_{u\_p\_un} := \max(1.4 \cdot F_{D\_p\_un}, 1.2 \cdot F_{D\_p\_un} + 1.6 \cdot F_{W\_e}) = 39.931 \cdot kip$

Shear capacity of pipe $V_{c\_p} = 160.206 \cdot kip$

check shear capacity $checkV_{c\_p\_un} := \begin{cases} \text{"Satisfactory"} & \text{if } V_{c\_p} \ge V_{u\_p\_un} \\ \text{"No good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$

Maximum factored moment load

$$M_{u\_p\_un} := \max(1.4 \cdot M_{o\_Fdp\_un}, 1.2 \cdot M_{o\_Fdp\_un} + 1.6 \cdot M_{o\_Fwe}) = 4.991 \cdot ft \cdot kip$$

Moment capacity in compression $M_{cc\_p} = 1639.158 \cdot ft \cdot kip$

Moment capacity in tension $M_{ct\_p} = 193.21 \cdot ft \cdot kip$

check moment capacity

$$checkM_{c\_p\_un} := \begin{cases} \text{"Satisfactory"} & \text{if } \min(M_{cc\_p}, M_{ct\_p}) \ge M_{u\_p\_un} \\ \text{"No good"} & \text{otherwise} \end{cases} = \text{"Satisfactory"}$$

48-inch HDPE from Middle Pond A to Ash Pond A

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 loads. The buckling analysis was performed for the existing 48-inch outer diameter HDPE pipe. Drawing 10W263-02 specifies the pipe to be in accordance with TDOT specifications. TDOT references AWWA M55 - PE Pipe - Design and Installation for the HDPE pipes.

Apparent modulus of elasticity	$E_{pipe} := 28250psi$ (AWWA M55, Table 5-6, long term)
HDPE outside diameter	$OD := 48in$
HDPE wall thickness	$t_{pipe} := 3.556in$
HDPE inside diameter	$ID := OD - 2 \times t_{pipe} = 40.888 \times in$
Dimension ratio	$DR := \frac{OD}{t_{pipe}} = 13.498$
Dike crest elevation	$EL_{crest} := 479.5ft$
Normal pool elevation	$EL_{np} := 468.3ft$
Flood pool elevation	$EL_{fp} := 473ft$
Pipe invert elevation	$EL_{in} := 471.4ft$
Height of maximum soil cover	$H_{cover} := EL_{crest} - \left(EL_{in} + OD - t_{pipe}\right) = 4.396\,ft$
Height of soil above normal pool	$H_{soil\_np} := \min\left(H_{cover}, EL_{crest} - EL_{np}\right) = 4.396\,ft$
Height of soil submerged, normal pool	$H_{submerged\_np} := H_{cover} - H_{soil\_np} = 0\,ft$
Height of soil above flood pool	$H_{soil\_fp} := \min\left(H_{cover}, EL_{crest} - EL_{fp}\right) = 4.396\,ft$
Height of soil submerged, flood pool	$H_{submerged\_fp} := H_{cover} - H_{soil\_fp} = 0\,ft$
Modulus of soil reaction	$E' := 1300psi$ (Fine-grained soils relative compaction 90%, AWWA Table 6-1)
Safety factor for design	$FS_{PE} := 2$

Allowable Buckling - normal pool

Buoyancy factor

$$R_{b\_np} := 1 - 0.33 \cdot \frac{H_{submerged\_np}}{H_{cover}} = 1$$

Soil elastic support factor

$$B' := \frac{1}{1 + 4 \cdot e^{\frac{-0.065}{ft} \cdot H_{cover}}} = 0.25$$

Allowable external pressure for constrained pipe - buckling

$$P_{CA\_np} := \frac{5.65}{FS_{PE}} \cdot \sqrt{R_{b\_np} \cdot B' \cdot E' \cdot \frac{E_{pipe}}{12 \cdot (DR - 1)^3}} = 55.884\,psi$$

Allowable Buckling - flood pool

Buoyancy factor

$$R_{b\_fp} := 1 - 0.33 \cdot \frac{H_{submerged\_fp}}{H_{cover}} = 1$$

Allowable external pressure for constrained pipe - buckling

$$P_{CA\_fp} := \frac{5.65}{FS_{PE}} \cdot \sqrt{R_{b\_fp} \cdot B' \cdot E' \cdot \frac{E_{pipe}}{12 \cdot (DR - 1)^3}} = 55.884\,psi$$

Calculate Applied Loads

Dead load - usual condition

$$DL_u := \left[\gamma_s \cdot H_{soil\_np} + (\gamma_s - \gamma_w) \cdot H_{submerged\_np}\right] \cdot R_{b\_np} + \gamma_w \cdot H_{submerged\_np} = 3.206\,psi$$

Dead load - unusual condition

$$DL_{un} := \left[\gamma_s \cdot H_{soil\_fp} + (\gamma_s - \gamma_w) \cdot H_{submerged\_fp}\right] \cdot R_{b\_fp} + \gamma_w \cdot H_{submerged\_fp} = 3.206\,psi$$

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

$$\mathrm{cover} := \begin{pmatrix} 1.5 \\ 2.0 \\ 2.5 \\ 3.0 \\ 3.5 \\ 4.0 \\ 6.0 \\ 8.0 \\ 10.0 \end{pmatrix} \mathrm{ft} \qquad \mathrm{live}_{\mathrm{load}} := \begin{pmatrix} 13.9 \\ 9.5 \\ 7.0 \\ 5.4 \\ 4.3 \\ 3.6 \\ 2.0 \\ 1.3 \\ 0.8 \end{pmatrix} \mathrm{psi}$$

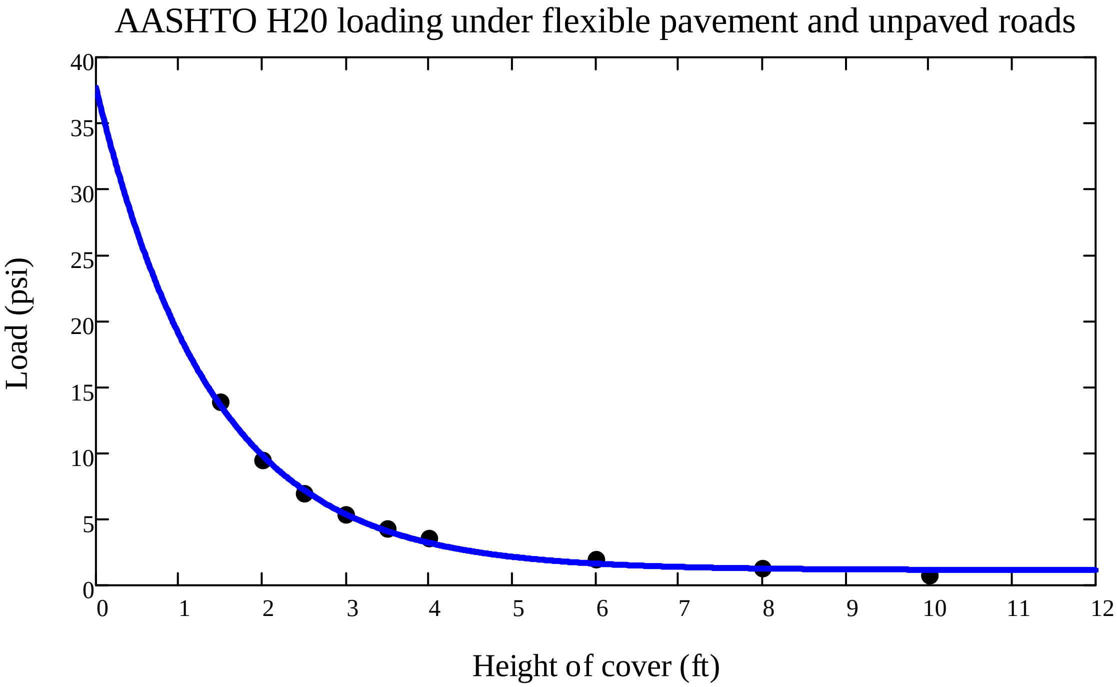
Exponential curve fit

$$\mathrm{efit} := \mathrm{expfit}\left(\frac{\mathrm{cover}}{\mathrm{ft}}, \frac{\mathrm{live}_{\mathrm{load}}}{\mathrm{psi}}\right) = \begin{pmatrix} 36.548 \\ -0.72 \\ 1.203 \end{pmatrix}$$

$$\mathrm{aa} := \mathrm{efit}_0 = 36.548 \qquad \mathrm{bb} := \mathrm{efit}_1 = -0.720 \qquad \mathrm{cc} := \mathrm{efit}_2 = 1.203$$

$$f(x) := \mathrm{aa}\cdot e^{\mathrm{bb}\cdot x} + \mathrm{cc} \rightarrow 36.548373145638216\cdot e^{-0.71974758459671384\cdot x} + 1.2033362039448283$$

Plot height of cover versus load for unpaved roads



Live load on pipe $LL := f\left(\frac{H_{cover}}{ft}\right)\cdot psi = 2.747\,psi$

Total load - usual condition $TL_u := DL_u + LL = 5.953\,psi$

Total load - unusual condition $TL_{un} := DL_{un} + LL = 5.953\,psi$

Check critical buckling pressure - usual loading condition

$$check_{Pcr\_u} := \left| \begin{array}{l} \text{"Satisfactory"} \ \text{ if } \ P_{CA\_np} \geq TL_u \\ \text{"No good"} \ \text{ otherwise} \end{array} \right. = \text{"Satisfactory"}$$

Check critical buckling pressure - unusual loading condition

$$check_{Pcr\_un} := \left| \begin{array}{l} \text{"Satisfactory"} \ \text{ if } \ P_{CA\_fp} \geq TL_{un} \\ \text{"No good"} \ \text{ otherwise} \end{array} \right. = \text{"Satisfactory"}$$