Stantec Consulting Services Inc.
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October 5, 2018
File: rpt_001_let_175567307
Revision 0

Tennessee Valley Authority (TVA)
1101 Market Street
Chattanooga, Tennessee 37402

RE: Seismic Impact Zones
Stilling Pond (including Retention Pond)
EPA Final Coal Combustion Residuals (CCR) Rule
TVA Cumberland Fossil Plant
Cumberland City, Tennessee

1.0 PURPOSE

As described in 40 CFR § 257.63(a), an owner or operator of an existing CCR surface impoundment is required to demonstrate that the unit is not located in seismic impact zones unless the unit meets certain requirements. This letter documents Stantec's certification that the Stilling Pond (including Retention Pond) at the TVA Cumberland Fossil Plant (CUF) complies with the location restrictions for seismic impact zones in the EPA Final CCR Rule at 40 CFR § 257.63(a).

2.0 SUMMARY OF FINDINGS

The attached demonstration documents that the Stilling Pond (including Retention Pond) meets the requirements set forth in 40 CFR § 257.63(a).

3.0 QUALIFIED PROFESSIONAL ENGINEER CERTIFICATION

I, Stephen H. Bickel, 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 TVA Cumberland Stilling Pond (including Retention Pond) meets the requirements specified in 40 CFR § 257.63(a).

Stantec Consulting Services Inc.
3052 Beaumont Circle, Lexington KY 40513

SIGNATURE ______________________ DATE 10/05/2018

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ATTACHMENTS:	Seismic Impact Zones Demonstration

Seismic Impact Zone Demonstration

Stilling Pond (including Retention Pond)
Cumberland Fossil Plant
Stewart County, Tennessee

Prepared for:
Tennessee Valley Authority
Chattanooga Tennessee

Prepared by:
Stantec Consulting Services Inc.
Lexington, Kentucky

October 5, 2018
Revision 0

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1.0 INTRODUCTION

On April 17, 2015, EPA published the "Disposal of Coal Combustion Residuals (CCR) from Electric Utilities" final rule in the Federal Register. The Tennessee Valley Authority (TVA) contracted Stantec Consulting Services Inc. (Stantec) to evaluate the existing coal combustion residuals (CCR) surface impoundment, Stilling Pond (including Retention Pond), at the Cumberland Fossil Plant (CUF) regarding the requirements for the Seismic Impact Zone Location Restriction as required by the EPA Final CCR Rule, 40 C.F.R. §257.63.

1.1 OBJECTIVE

As required by §257.63 of the EPA Final CCR Rule, an owner or operator of an existing CCR surface impoundment is required to demonstrate that the unit is not located in a seismic impact zone unless the unit meets certain requirements. The objective of this report is to determine if the CUF Stilling Pond (including Retention Pond) meets the requirements for seismic impact zones.

1.2 UNIT DESCRIPTION

CUF is located on the south bank of the Cumberland River near river mile 103. The plant is approximately a half mile west of Cumberland City in Stewart County, Tennessee.

Figure 1. Site Vicinity Map

Referring to Figure 1, the Stilling Pond (including Retention Pond) is located to the northwest of the powerhouse and contains both the Retention and Stilling Ponds. It is formed by the Perimeter Dike along the east, north, and west and a Divider Dike to the south that separates it from the Dry Fly Ash Stack. The Stilling Pond encompasses approximately 55 acres. TVA has determined that the Stilling Pond is a CCR Surface Impoundment and therefore subject to the CCR rule (Stantec Consulting Services, Inc., 2016e).

The Stilling Pond is used for detention of stormwater runoff from the Gypsum Storage Area and the Dry Ash Stack, as well as process water from the Bottom Ash Pond and effluent from various plant operations. The sluice water containing CCR materials is discharged into the Bottom Ash Pond where bottom ash settles out and the effluent is conveyed by an open channel conveyance channel to the south side of the Retention Pond (Stantec Consulting Services, Inc., 2016f).

Figure 2. Stilling Pond Unit Configuration

Originally, the Stilling Pond was a portion of a larger ash disposal area enclosed by an earthfill perimeter dike constructed in 1969 to an elevation of 380 feet. In 1977, the divider dike for the Stilling Pond and Retention Pond was constructed of ash. The perimeter dikes for the ash disposal areas were raised in 1979 with clay up to elevation 395 feet. In 1986, approximately 300 linear feet of the west portion of the divider dike between the Retention Pond and the Dry Ash Stack was constructed. The remainder of the divider dike between the Retention Pond and Dry Fly Ash Stack was constructed between 1995 and 1996 (Stantec Consulting Services, Inc., 2016c).

The spillway system consists of four riser and outlet pipe structures located in the northeast corner of the Stilling Pond. The risers are 48-inch diameter, reinforced concrete pipe (RCP) sections stacked vertically. A cured-in-place pipe (CIPP) liner was installed to increase structural stability and cover the riser pipe joints. Each riser has been further stabilized against lateral movement by the placement of a circular (in plan view) crushed stone berm that extends to just below the riser crest. The outlet pipes are 36-inch-diameter, CIPP-lined RCP and approximately 217 feet in length. They extend from the risers through the Perimeter Dike to a headwall at the outlet where they discharge to the Cumberland River via the Condensing Cooling Water Discharge Channel (Stantec 2016b).

2.0 CRITERIA

The EPA Final CCR Rule § 257.53 defines a seismic impact zone as follows:

> *Seismic impact zone means an area having a 2% or greater probability that the maximum expected horizontal acceleration, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10 g in 50 years.*

The EPA Final CCR Rule § 257.63 requirements for seismic impact zones are:

> ***40 CFR § 257.63(a).*** *New CCR landfills, existing and new CCR surface impoundments, and all lateral expansions of CCR units must not be located in seismic impact zones unless the owner or operator demonstrates by the dates specified in paragraph (c) of this section that all structural components including liners, leachate collection and removal systems, and surface water control systems, are designed to resist the maximum horizontal acceleration in lithified earth material for the site.*

The word "resist" in the above language is subject to interpretation. The preamble to the CCR Rule (80 Fed. Reg. 21302, 21366 (April 17, 2015)) provides this guidance:

> *For units located in seismic impact zones, as part of any demonstration, owners and operators should include: (1) A determination of the expected peak ground acceleration from a maximum strength earthquake that could occur in the area; (2) a determination of the site-specific seismic hazards such as soil settlement; and (3) a facility design that is capable of withstanding the peak ground acceleration. Seismic designs broadly should include a response analysis to quantify the demands of earthquake motion on facility structures (i.e., landfills, surface impoundments, liners, covers, leachate collection systems, surface water handling systems), liquefaction analyses of both waste and foundation soils to evaluate stability under seismic loading, and a slope stability and deformation analyses. Design modifications to accommodate seismic risks should include use of conservative design factors, use of ductile materials, built-in redundancy for critical system components, and other measures capable of mitigating the potential for seismic upset.*

The facility should be capable of "withstanding the peak ground acceleration." The preamble (80 Fed. Reg. 21366) provides further guidance that the unit design should be able to withstand an expected earthquake with limited damage and remain capable of preventing a harmful release of CCR, leachate, and contaminants both during and after the seismic event.

3.0 DEMONSTRATION

The Stilling Pond (including Retention Pond) at CUF was evaluated with respect to the requirements outlined in Section 2.0. First, the unit's location was determined to be within a seismic impact zone. Therefore, the structural components including liners, leachate collection and removal systems, and surface water control systems, must resist the maximum horizontal acceleration in lithified earth material for the site. Since this CCR unit does not have a liner or leachate collection and removal system, the components that require consideration in this demonstration are limited to the surface water control systems (i.e., the existing spillway structures).

The failure mode of concern is an inboard slope failure that could damage the primary spillway and potentially cause an uncontrolled loss of water and CCR from the unit. A summary of the relevant engineering analyses and results are provided in this section. Outboard slope failures were not considered in this demonstration since TVA has already addressed this failure mode through the seismic safety factor demonstration (Geocomp 2016), which is posted on the Cumberland Coal Combustion Residuals website.

3.1 DESIGN EARTHQUAKE

Site-specific seismic hazard analyses were performed to determine appropriate earthquake motions for the demonstration. The slope stability analysis considered peak accelerations associated with an earthquake having a 2% probability of exceedance in 50 years (earthquake return period of about 2,500 years). At the site, this corresponds to a 6.99 M_w (moment magnitude) event with a peak horizontal acceleration of 0.187g in rock (Geocomp 2016). The peak horizontal acceleration in rock exceeds 0.10g; thus, the unit is within a seismic impact zone and further demonstration is required.

For the stability analyses, seven acceleration time histories were developed to represent expected bedrock motions under the unit during a design earthquake. Ground response analyses were used to predict the resulting seismic loads in the soil column and unit. Maximum induced cyclic stresses were computed for use in the liquefaction triggering analyses. Acceleration time histories along potential failure surfaces were also estimated, as needed for the seismic deformation analyses. Refer to Appendix A for details of the ground response analyses.

3.2 SUBSURFACE PROFILE

A general overview of the subsurface conditions of the perimeter dike of the Stilling Pond (including Retention Pond) is summarized in the table below. A more in-depth review is found in Stantec (2010). Specific details of subsurface conditions at the spillway structures are included with the geotechnical analysis in Appendix A.

Table 1. Generalized Subsurface Conditions – Stilling Pond (including Retention Pond) Perimeter Dike (Stantec 2016a)

Materials	Approximate Elevation	General Consistency/Density
Dike 1 – original perimeter dike, lean clay (CL) with areas of sand or gravel, limited areas classified as fat clay (CH), just above natural ground in most borings surrounding the Retention and Stilling Ponds	Top of dike 380 feet	Very soft to very stiff
Dike 2 (lean clay) – raised dike uphill of original perimeter dike, along outside perimeter of Retention and Stilling Ponds, lean clay to lean clay with gravel (CL)	Top of dike 395 feet	Soft to very stiff
Dike 2 (fat clay) – raised dike uphill of original perimeter dike, along outside perimeter of Retention and Stilling Ponds, fat clay to fat clay with gravel (CH), near the top of Dike 2 or may compose complete Dike 2 zone	Top of dike 395 feet	firm to very stiff
Fly ash (sluiced) – silt (ML), silty sand with gravel (SP), silty sand (SM), and sandy lean clay (CL)	Various	Very soft to medium stiff
Alluvial (clay) – lean clay (CL), silty to sandy with rock fragments	Various	Soft to very stiff
Alluvial (granular) – silty sand with gravel (SM), gravel with clay to silt and sand (GP-GC or GM)	Various	Very loose to very dense
Bedrock – interbedded limestone and shale	El. 280-371 feet	Moderately hard to hard
Bottom ash (fill)[1] – sand, silty sand (SW-SM, SP-SM), placed on inboard slope of Dike 2 for access to spillway risers	Top of berm 387 feet	Loose to dense
Rock fill[1] – crushed stone, berm to stabilize spillway risers, placed on inboard slope of bottom ash	Top of berm 372 feet	Loose to medium dense

[1] Material is unique to spillway cross-sections.

3.3 LIQUEFACTION TRIGGERING ANALYSES

The potential for triggering soil liquefaction (sand-like soils) and/or cyclic softening (clay-like soils) was evaluated for the deposits beneath the perimeter dike system at the spillway structures. Published, empirical methods were used with data from site explorations (see Appendix A).

The results showed that no liquefaction is expected in the dike fill (including the bottom ash and rock fill) or the foundation soils at the spillway structures during the design earthquake. As such, the post-earthquake slope stability analysis did not consider liquefied soil strengths. Refer to Appendix A for details of the liquefaction assessment.

3.4 SEISMIC SLOPE STABILITY AND DISPLACEMENT ANALYSES

The seismic stability of the perimeter dike system was evaluated by developing a critical cross section (at the spillways) for analysis. Conventional, two-dimensional engineering analyses were used to evaluate post-earthquake and pseudostatic stability. Seismic displacement analyses were also completed. The slope stability analysis conservatively neglected potential resistance due to the spillway structures.

The results indicate stable slopes for the post-earthquake conditions, and relatively small permanent displacements (i.e. deformations) are expected for the pseudostatic condition. The displacement analyses confirmed that predicted displacements that would impact the spillway inlet structures and/or spillway pipes are negligible (less than 1 inch). Refer to Appendix A for details of the slope stability and displacement analyses.

3.5 STRUCTURAL ANALYSES

Given the seismic loads and the expected displacement of the perimeter dike system during the design earthquake, the structural performance of the spillway inlets and pipes was considered, with respect to the potential for damage and an uncontrolled loss of water and CCR from the ash pond.

A lumped mass model of the riser structure was used for the analysis. The structure was assessed for stability at its base and at the various riser pipe joints. Since the riser pipe segments are connected via keyed joints, sliding was assumed to be a non-viable failure mode at the pipe joints. Therefore, the structure was assessed for sliding and bearing capacity failure only at its base, and resultant location (overturning) assessments were performed at the base of the structure and at the four riser pipe joints. Because overturning stability criteria was met at each riser joint, block rocking analysis at each joint did not need to be evaluated. The analysis results and supporting calculations are provided in Appendix B. The results indicate the riser structure meets stability criteria for the seismic event.

3.6 ANALYSES DISCUSSION

The geotechnical analyses indicate that deformations of the perimeter dike at the outlet structure are negligible (less than 1 inch). The structural analyses indicate that the outlet structure is unlikely to rotate, slide, or have a bearing failure during the seismic event. The seismic event is unlikely to cause damage to the spillway structure so a harmful release of CCR is not expected during and after the seismic event.

4.0 CONCLUSION

Based on this assessment, the Stilling Pond (including Retention Pond) at CUF meets the requirements of §257.63 of the EPA Final CCR Rule for seismic impact zones.

5.0 REFERENCES

Geocomp (2016). "Initial Seismic Safety Factor Assessment EPA Final CCR Rule Cumberland Fossil Plant – Stilling Pond (Including Retention Pond) Cumberland City, Tennessee," Prepared for Tennessee Valley Authority, October.

Stantec (2010). "Report of Geotechnical Exploration and Slope Stability Evaluation, Ash Pond, Cumberland Fossil Plant, Stewart County, Tennessee." Prepared for Tennessee Valley Authority. March.

Stantec (2016a). "Initial Safety Factor Assessment, Stilling Pond (including Retention Pond), EPA Final CCR Rule, TVA Cumberland Fossil Plant, Stewart County, Tennessee. Prepared for Tennessee Valley Authority. October.

Stantec (2016b). "Initial Structural Stability Assessment, Stilling Pond (including Retention Pond), EPA Final Coal Combustion Residuals (CCR) Rule, TVA Cumberland Fossil Plant, Cumberland City, Tennessee. Prepared for Tennessee Valley Authority. October.

APPENDIX A
GEOTECHNICAL ANALYSES

Tennessee Valley Authority Cumberland Fossil Plant Stewart County, Kentucky	**Seismic Impact Zones** **Slope Stability Analysis**

Calculation Package N/A

Exhibit
Seismic Impact Zones

Purpose:

- Evaluate the slope stability of the dike at the spillway location for the Stilling Pond (including Retention Pond), considering pseudostatic and post-earthquake load conditions.

Methods:

- Liquefaction screening
- Spencer's method of slices
- Newmark analysis (simplified seismic displacement analysis)

Results:

- No liquefaction is anticipated for the design earthquake.
- The factor of safety for post-earthquake load conditions is adequate.
- The estimated displacement for the pseudostatic load condition is less than about 1 inch.

Calculation Performed by: Stantec Consulting Services Inc.; May 22, 2018	
Prepared by: Enrique Farfan, PhD, PE	*Reviewed by:* Jeffrey S. Dingrando, PE, PG
Revisions: N/A	

TVA CUMBERLAND (CUF) STILLING POND (INCLUDING RETENTION POND)

SEISMIC IMPACT ZONES (EPA FINAL CCR RULE, 40 CFR §257.63)

SLOPE STABILITY ANALYSIS

1. OVERVIEW

As part of CCR Rule Seismic Impact Zone Location Restriction Demonstration (§257.63), the stability of the embankment, at the Stilling Pond (including Retention Pond) spillway structure, needs to be evaluated considering pseudostatic and post-earthquake load conditions. The failure mode of concern is an inboard slope failure that could damage the spillway and potentially cause an uncontrolled loss of water and CCR from the ash pond. Outboard failures were not considered, as TVA has already addressed this failure mode through the seismic safety factor demonstration (Geocomp 2016).

In 2017, Stantec performed a preliminary assessment evaluation of embankment stability using existing geotechnical data. The preliminary results have provided information to direct additional field investigation and laboratory testing program.

The following sections present the data and calculations performed for a refined analysis for the embankment at the Stilling Pond (including Retention Pond) spillway structure.

2. SUMMARY OF DRILLING AND LAB TESTING RESULTS

From October 17-20, 2017, Stantec performed the borings using mud rotary drilling methods. Disturbed samples were collected while performing standard penetration tests (SPT). The borings were logged in the field by a Stantec engineer. Shelby tube sampling was attempted in select depth intervals, but no material was recovered. No rock coring was performed. As-drilled boring locations were surveyed by TVA in December 2017. Locations of these borings are shown on the exploration plan view presented in Attachment A. A summary of the boring locations is included in Table 1 below.

Boring logs, boring layout, and laboratory testing results were provided in a summary memorandum to TVA to document the field and laboratory effort (Stantec 2018).

Table 1. 2017 As-Drilled Boring Locations (Stantec 2018)

Boring	Total Depth (ft)	Coordinates		Surface Elevation (NGVD29) (ft)
		Northing (Plant Local NAD27) (ft)	Easting (Plant Local NAD27) (ft)	
STN-SW-1	47.0	733,015.1	1,511,355.9	386.9
STN-SW-2	36.5	732,967.7	1,511,364.8	386.7
STN-SW-3	33.0	732,917.5	1,511,373.5	386.6

Selected soil samples from borings were sent to the laboratory for index property testing. Table 2 summarized the laboratory test program.

Table 2. Summary of 2017 Laboratory Tests (Stantec 2018)

Type of test	Number of samples
ASTM D 2216 – Moisture Content of Soil	41
ASTM D 4318 Method A – Atterberg Limits	28
ASTM D 422 – Particle Size Analysis (sieve + hydrometer)	28

A summary of laboratory test results is presented in Plates 2 and 3, where soil index properties are plotted versus boring depth along with an interpretation of the soil profile and SPT data.

The new borings were combined with existing borings (Stantec 2010, 2012) that were considered in the preliminary slope stability analysis (see Plate 1). A list of the existing borings is presented in Table 3.

Table 3. As-Drilled 2010 Borings Locations (Stantec 2010, 2012)

Boring	Total Depth (ft)	Coordinates		Ground Surface Elevation (ft)
		Northing (ft)	Easting (ft)	
SB-1	47.2	733,043.82	1,511,354.49	387.4
SB-2	50.5	732,895.08	1,511,375.22	386.8
SB-3	21.0	732,830.36	1,511,382.57	387.2
STN-57	56.5	733,365.74	1,511,360.12	381.5
STN-58	62.9	733,305.89	1,511,314.36	395.0
STN-59	35.0	732,780.76	1,511,517.22	383.0
STN-60	43.6	732,776.76	1,511,517.22	383.0

3. CROSS SECTION GEOMETRY

A cross section was generated perpendicular to the embankment at the spillway structure location (Section A-A') as shown in Plate 1. Soil regions were characterized based on available data. The boundaries between each region were derived from the following sources and corroborated with the new borings performed for this analysis.

- Basis of Design Report Cumberland Fossil Plant Ash Stilling Pond Spillway Improvement Project Work Plan 7 (Stantec 2012) Profile – Baseline
- Report of Geotechnical Exploration and Slope Stability Evaluation (Stantec 2010) Profile – Baseline (Drawings No. 10W544-06, 10W544-07)

The geometry of the cross section developed for this analysis is depicted in Plate 4 along with SPT data.

4. MATERIAL PARAMETERS

The soil parameters for this analysis were developed based on the site-specific seismic assessment performed by Geocomp (2016), the slope stability evaluation performed by Stantec (2010), and the preliminary slope stability analysis performed by Stantec for the spillway cross-section. Soil parameters were adjusted based on field and laboratory data performed for this project to reflect specific cross section conditions. In addition, the soils were screened for liquefaction potential as summarized in Section 5. A summary of soil parameters selected for the refined analysis is presented in the Tables 3 and 4.

Under pseudostatic conditions in unliquefied soils, reduction of 20% on the static undrained shear strength was considered in our analysis for those regions considered saturated. This reduction is based on recommendations by Makdisi and Seed (1977; 1978) and Hynes-Griffin and Franklin (1984) to account for the potential loss of shear resistance in unliquefied soils due to increase in pore pressures during dynamic loadings.

Undrained strength parameters reductions were calculated as follows:

- $c_{EQ} = 0.8 * c$
- $\tan(\phi_{EQ}) = 0.8 * \tan(\phi)$

The static and seismic strength parameters are summarized in Tables 4 and 5, respectively. The Rock Fill is considered free draining; therefore, a strength reduction was not applied the seismic load cases.

Plate 2 presents the soil data from the six borings performed at the spillway location, which included the three new borings. Note that the SPT data agrees in five of the six borings, boring SB-2 presents higher SPT blowcounts compared with the rest of the borings. This difference in SB-2 could be the result of varying compaction of the fill during construction of the perimeter dike.

Plate 3 presents soil data from four borings adjacent to the spillway. The SPT blowcounts in the adjacent borings are in general higher than the SPT blowcounts from borings located at the spillway, except for Boring STN-59. The difference in STN-59 is likely that the boring is located on the outboard side of the perimeter dike, and thus the subsurface conditions are quite different. Plate 4 depicts the embankment profile and the SPT data for easy reference.

Based on the 2017 SPT data, the embankment cross section at the spillway was updated to include a soft zone within the Dike 2 fill (see Plate 4). A static, undrained shear strength of 200 pounds per square foot (psf) was assigned to this soft zone, based on the low SPT blowcounts. The 2017 SPT borings also demonstrated that the dike fill material between spillway pipes was clay, not “Ash Fill” as indicated on historical TVA drawings (although it is possible that ash fill was used in localized zones around each pipe). For stability modeling purposes, this previous zone of Ash Fill was updated to be Dike 1 fill.

For soils with both drained and undrained strength parameters shown in Table 5, the seismic stability analyses utilize a bilinear strength envelope, where the lesser of the two strengths is applied depending on the normal stress at each slice of the failure mass.

Table 4. Summary of static shear strength parameters

Soil Layers	Unit Weight (pcf)	Drained Strength Parameters		Undrained Strength Parameters	
		c' (psf)	ϕ' (degrees)	c (psf)	ϕ (degrees)
Alluvial (Clay)	124	200	33	450	20
Bottom Ash	105	0	35	0	35
Bottom Ash (Unsaturated)	100	0	35	0	35
Dike 1 – Lean Clay	123	200	22	800	20
Dike 2 – Lean Clay	123	200	32	500	21
Dike 2 – Lean Clay – soft	123	200	32	200	0
Rock Fill	130	0	35	0	35

Table 5. Summary of seismic shear strength parameters

Soil Layers	Unit Weight (pcf)	Drained Strength Parameters		Undrained Strength Parameters	
		c' (psf)	ϕ' (degrees)	c_{EQ} (psf)	ϕ_{EQ} (degrees)
Alluvial (Clay)	124	200	33	360	16
Bottom Ash	105	0	35	0	29
Bottom Ash (Unsaturated)	100	0	35	0	35
Dike 1 – Lean Clay	123	200	22	640	16
Dike 2 – Lean Clay	123	200	32	400	17
Dike 2 – Lean Clay – soft	123	200	32	160	0
Rock Fill	130	0	35	0	35

5. LIQUEFACTION SCREENING

“Sand-like” soils are subject to liquefaction and can be evaluated using a simplified stress-based approach, while “clay-like” soils should be evaluated further for cyclic softening. Various guidance criteria have been proposed for separating soil behavior with respect to cyclic loading, liquefaction, and stress-strain response. Three sets of guidance criteria (Seed et al, 2003; Idriss and Boulanger, 2008; and MSHA, 2010) have been applied herein. Each utilizes soil index properties which are developed from laboratory testing. These three sets of guidance criteria may provide conflicting indications of behavior under dynamic load. These criteria are used together, with engineering judgment, to determine if a soil stratum is subject to liquefaction or cyclic softening (see Attachment B for calculation summary).

For soils identified as having clay-like behavior, additional criteria should be considered to determine if significant strength loss is likely due to cyclic loading. Note that the evaluation for cyclic softening in clay-like soils is often completed for the whole layer or deposit, and not for individual data points of penetration resistance as done for sandy soils. Three sets of guidance criteria (Seed et al, 2003; Bray and Sancio, 2006, and MSHA, 2010) have been applied herein. Again, each utilizes soil index properties which are developed from laboratory testing.

An initial screening for soil liquefaction susceptibility was performed on the new borings (STN-SW-1, STN-SW-2 and STN-SW-3). The clay-like soils are not susceptible to cyclic softening. Based on SPT blowcount corrections/normalizations and liquefaction triggering criteria from Boulanger and Idriss (2014), the sand-like soils encountered are generally too dense to liquefy, regardless of the size of the design earthquake. Detailed calculations are provided in Attachment C.

In summary, the soils in the vicinity of the spillways are not susceptible to liquefaction. Therefore, for post-earthquake stability, the seismic strengths from Table 5 are used.

6. SLOPE STABILITY ANALYSIS

The evaluation of the stability of the inboard slope for the embankment section at the spillway location was performed using the method of slices as described by Spencer (1967), where two equations, one with respect to moment equilibrium and another with respect to horizontal force equilibrium are satisfied using a constant relationship between the interslice shear and normal forces. The computer program Geostudio (2018) was used for the analysis.

Following assumptions were considered for our pseudostatic and post-earthquake analysis:

- Based on the Geocomp (2016) report, peak ground acceleration (PGA) on rock for the design earthquake at Cumberland Fossil Plant is 0.187g (PGA_{ROCK}). A seismic coefficient (k_h) of ½ of PGA_{ROCK} (k_h = 0.094g) was applied in the pseudostatic analysis, based on guidance in Hynes-Griffin and Franklin (1984). Using this value of k_h, a pseudostatic factor of safety of 1.0 or greater is associated with 1 meter or less displacement, which is typically tolerable for an embankment dam.

- For the post-earthquake analysis, the seismic coefficient is set to zero (k_h=0).

- The headwater is equal to the normal operating pool in the Stilling Pond (including Retention Pond), elevation 374.2 ft (per the 2017 pond lowering project).

- The phreatic surface was assumed to vary linearly within the embankment, between the headwater (374.2 ft) and an assumed tailwater elevation of 363 ft. For an inboard slope failure, the tailwater assumption has little impact on the results.

- An appropriate tension crack was specified based on the Dike 2 soil properties.

- Any potential resistance from the spillway structure itself is conservatively neglected in the slope stability analysis.

Table 6 summarizes the load cases considered in this analysis and the soil properties applied for each case (also see Attachment D).

Table 6. Summary of seismic slope stability results

Load Case	Soil Strengths	Calculated FS
Pseudostatic	Seismic	1.3
Post-Earthquake	Seismic	1.9

The pseudostatic analysis resulted in a factor of safety (FS) greater than 1, which implies that permanent embankment displacements should be relatively small at the spillway location. However, a simplified seismic displacement (i.e., Newmark) analysis was performed to better estimate the magnitude of the displacement.

7. SIMPLIFIED SEISMIC DISPLACEMENT ANALYSIS

To support the Newmark analysis, a ground response analysis is performed to propagate the design earthquake ground motions from the top of rock, through the soil column, to the base of the sliding mass. A soil column that represents the average conditions within and below the sliding mass of soil was modeled using the computer software Strata. Seven acceleration time histories developed for the Cumberland Fossil Plant site by Geocomp (2016) were considered in the analysis. Shear modulus reduction and damping ratio curves for each soil zone were based on empirical equations developed by Ishibashi and Zhang (1993).

The yield acceleration (i.e., k_h value at which FS=1.0) was determined to be 0.172g. The accelerations imposed by the design earthquake are then compared to the yield acceleration, Considering the seven time-histories, the largest permanent displacement was less than 1 inch (see Attachment E).

8. CONCLUSION

The liquefaction potential of the soils at the spillway structure were investigated, and we concluded based on this analysis that soils will not liquefy due to the design earthquake. Considering reduced (but not liquefied) seismic shear strengths, a post-earthquake FS of 1.9 was calculated for the inboard slope stability of the Stilling Pond (including Retention Pond) perimeter dike at the spillway structure location. Also considering seismic shear strengths, the pseudostatic factor of safety was greater than one; and a Newmark analysis was performed to estimate permanent displacement due to the design earthquake. The Newmark analysis indicates a permanent displacement of 1 inch or less.

9. REFERENCES

Boulanger, R.W. and Idriss, I.M. (2014). “CPT and SPT based liquefaction triggering procedures.” Report No. UCD/CGM-14/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California, Davis, CA.

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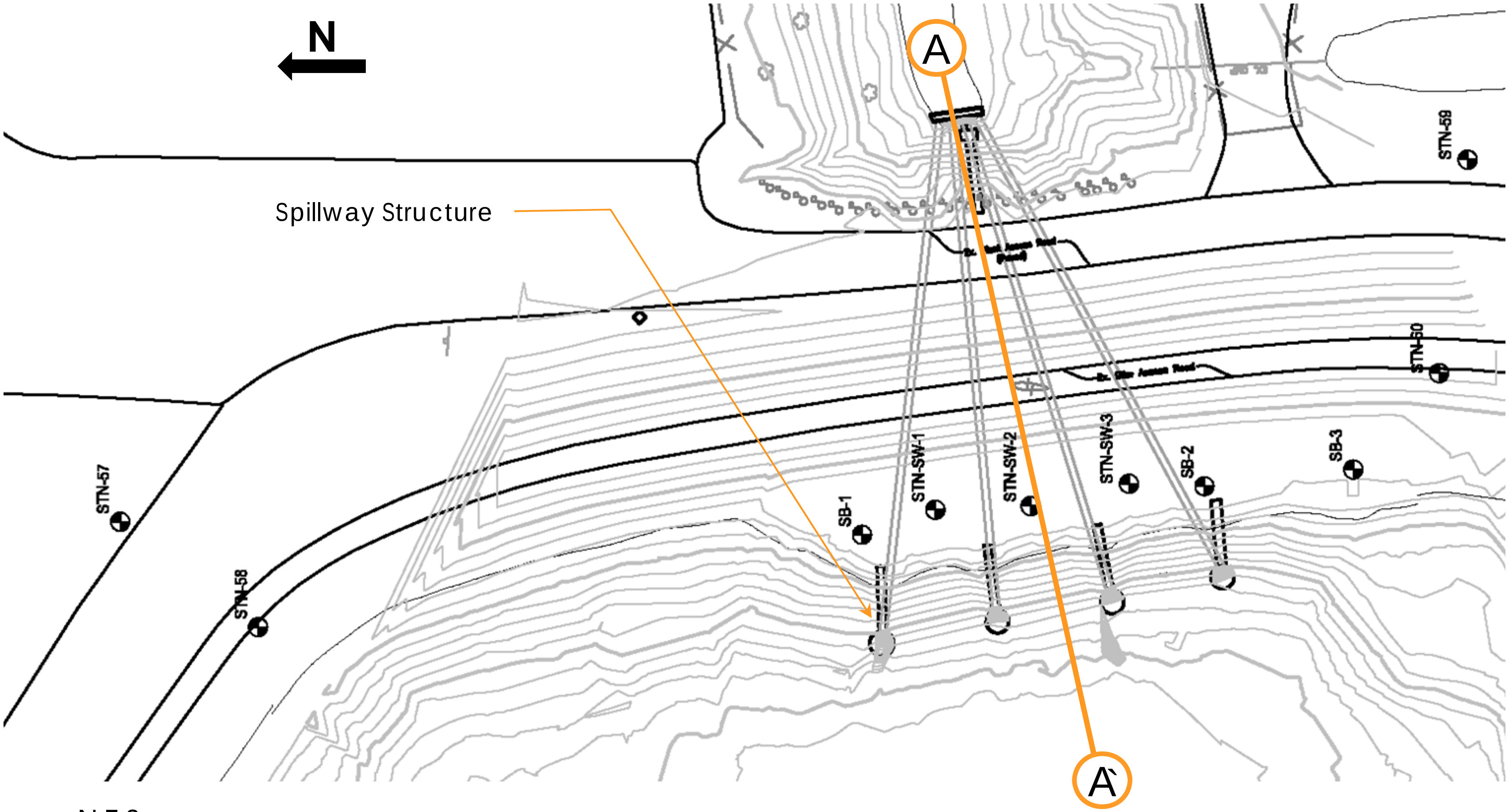
ATTACHMENT A



N.T.S

New Borings
STN-SW-1
STN-SW-2
STN-SW-3

Plate 1
Boring Location Plan

TVA CCR Rule: Location Restrictions – Phase 2
Cumberland Fossil Plant
Stewart County, Tennessee

Stantec

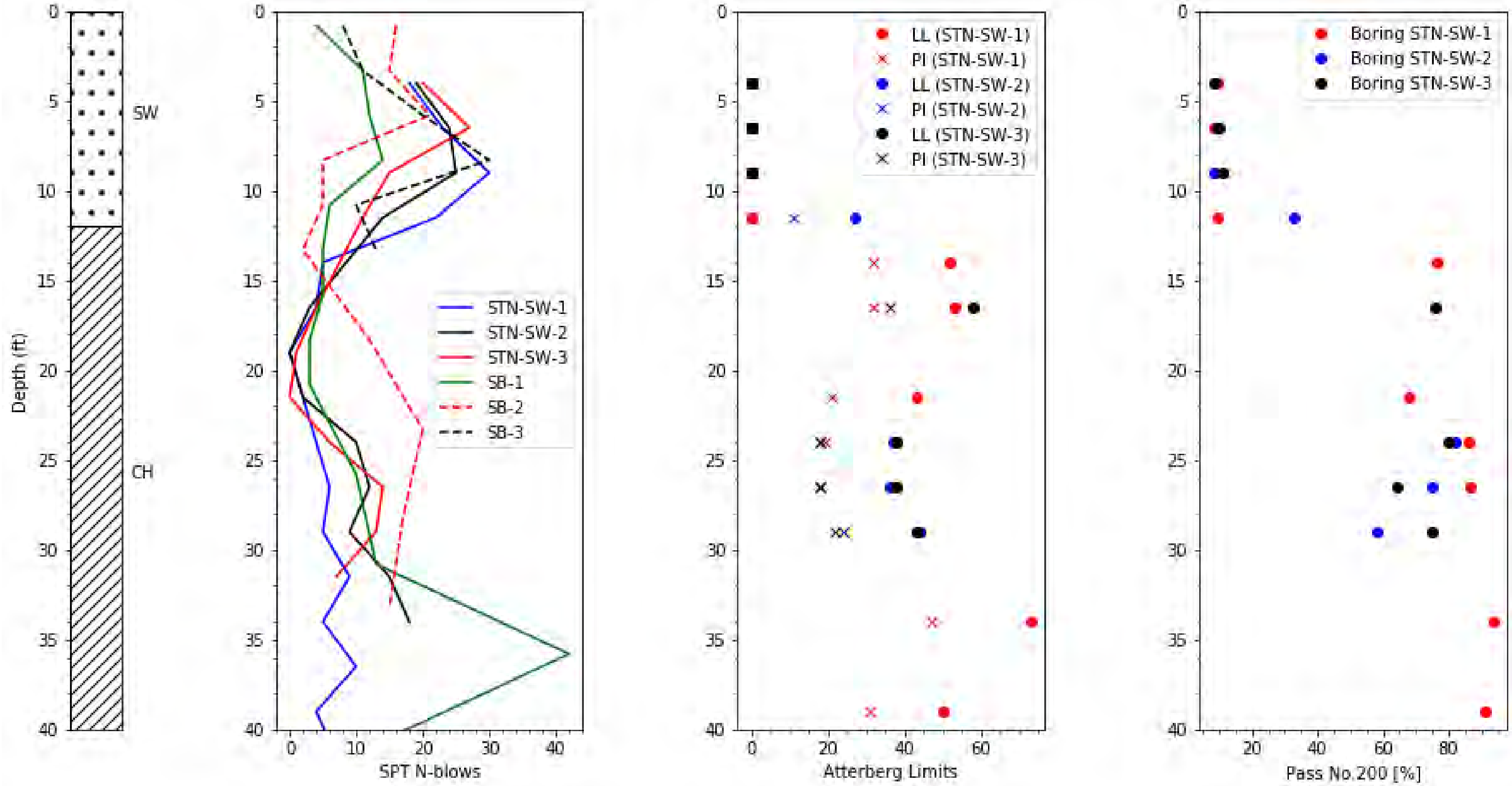


Plate 2
Soil Data Profiles

TVA CCR Rule: Location Restrictions – Phase 2
Cumberland Fossil Plant
Stewart County, Tennessee

Stantec

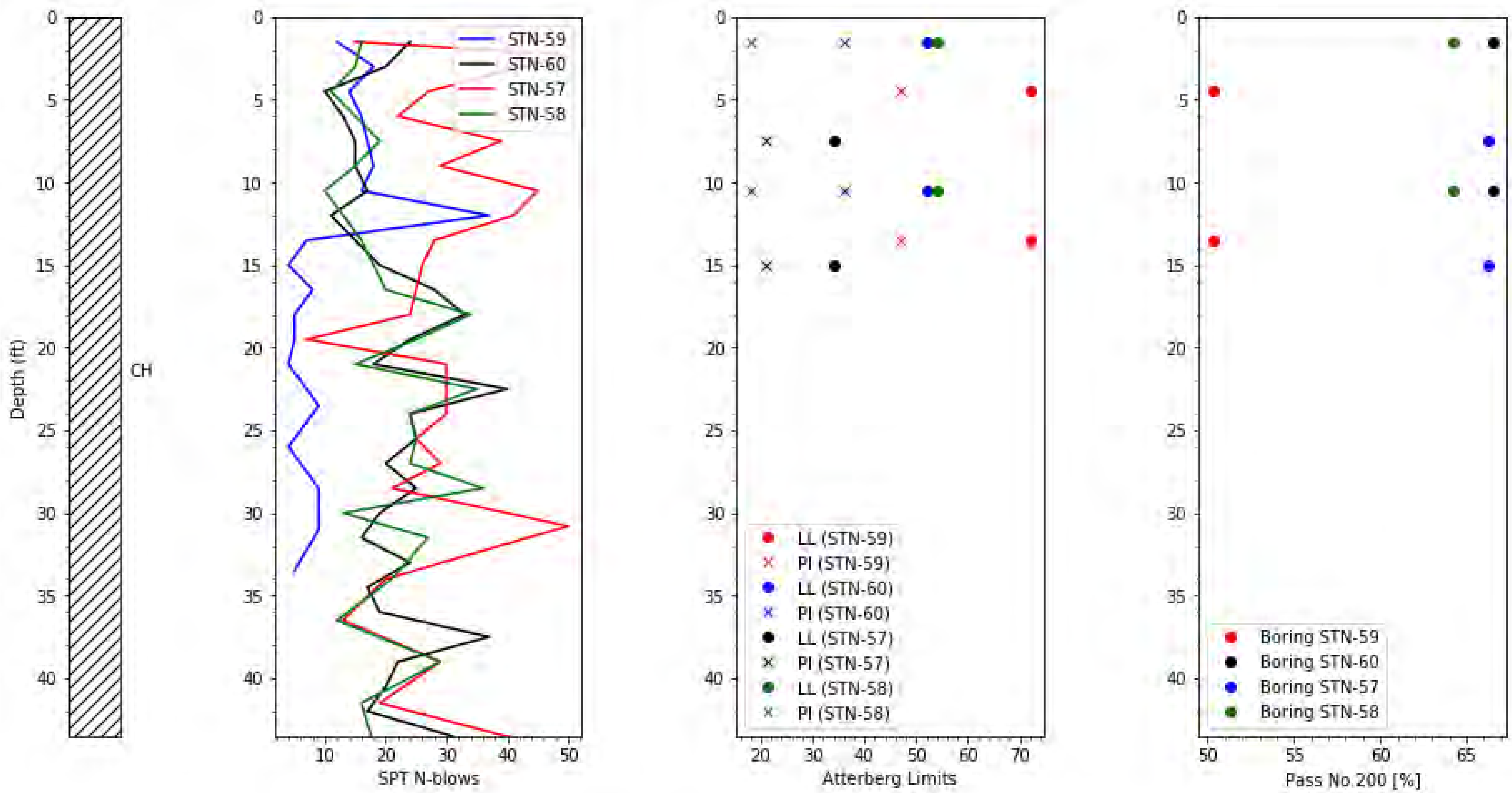
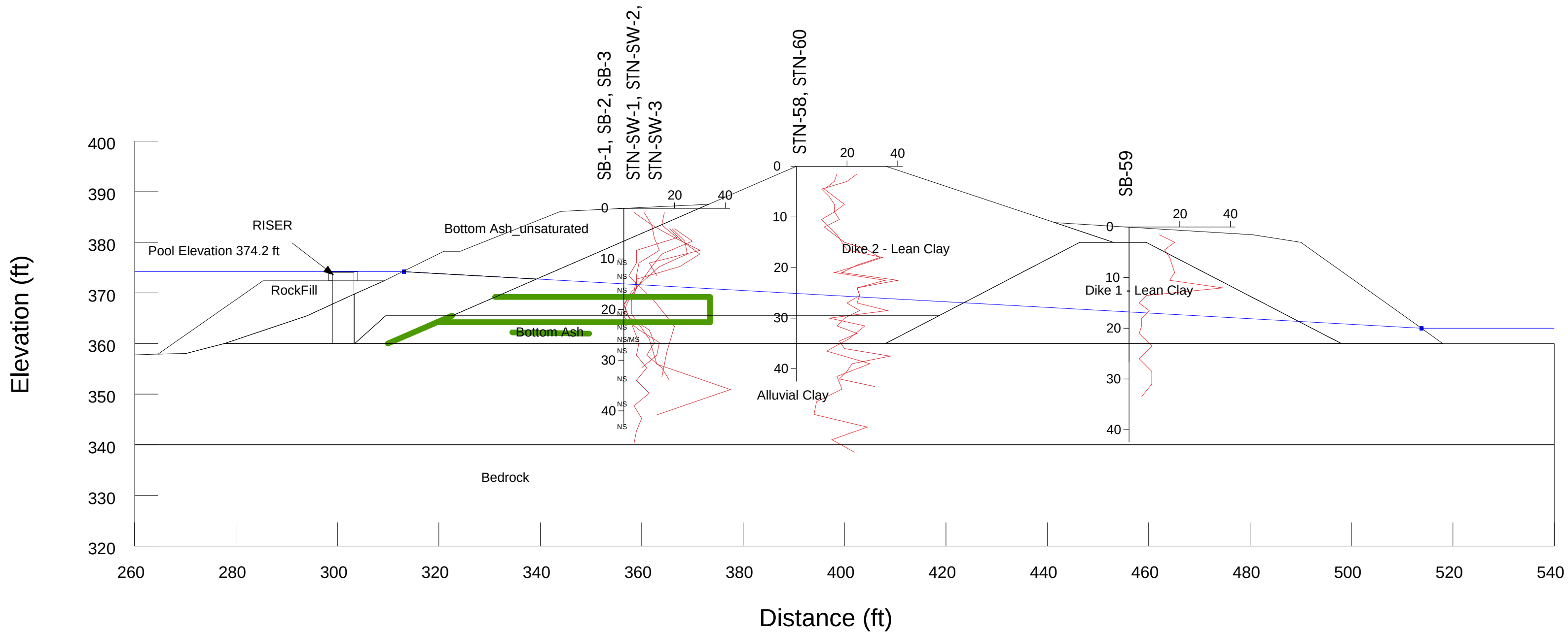


Plate 3
Soil Data Profiles

TVA CCR Rule: Location Restrictions – Phase 2
Cumberland Fossil Plant
Stewart County, Tennessee

Stantec



Legend

NS No soil liquefaction

Water table

Updated regions

SPT data

Plate 4
SPT data and Embankment Profile

TVA CCR Rule: Location Restrictions – Phase 2
Cumberland Fossil Plant
Stewart County, Tennessee

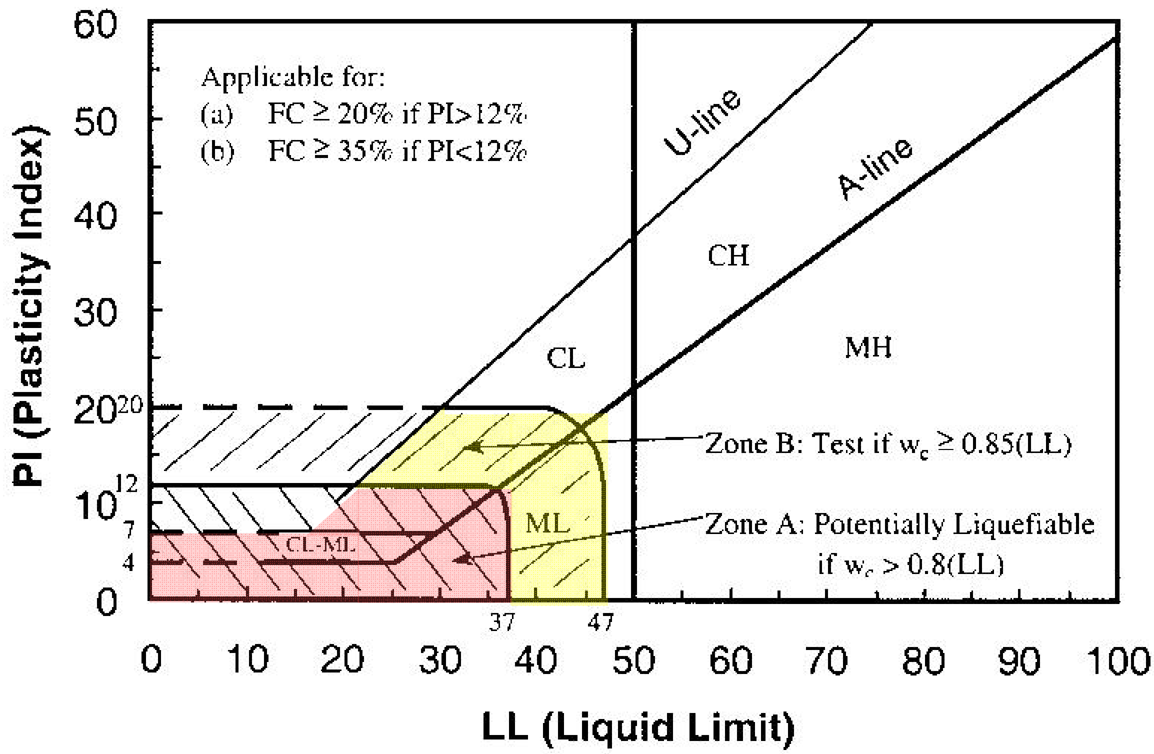
Stantec

ATTACHMENT B

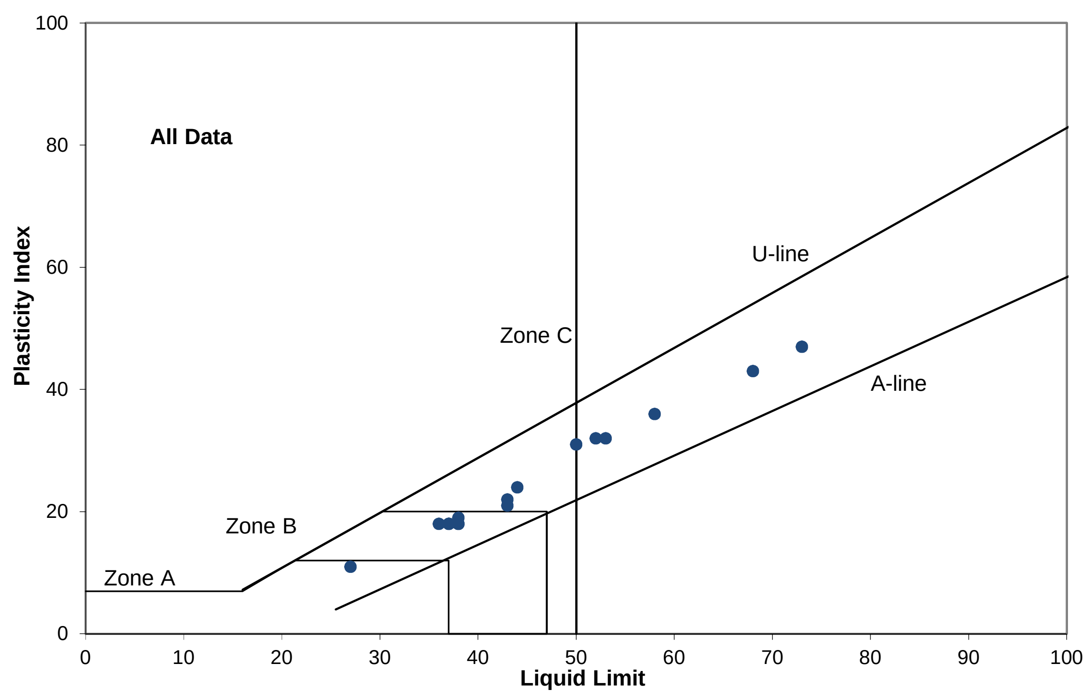
CLAM - **C**lay-like **L**aboratory-Based **A**ssessment of **M**aterials

Stantec Project Number:	175567307
Project Name:	CUF Ash Pond Spillway Sesmic Stability Evaluation: Refined Analysis
Material	

										Sand-like versus Clay-like Behavior (-1 indicates result does not meet criteria, green shading indicates result does meet criteria, no results shown for non-plastic material)																**Susceptibility of Clay-like Soils to Cyclic Softening (-1 indicates result does not meet criteria, green shading indicates result does meet criteria, no results shown for Sand-like materials)**								
										Using Criteria published by Seed et al (2003)										Using Criteria published by Idriss and Boulanger (2008)		Using criteria published by MSHA (2010)				Using Criteria published by Seed et al (2003)		Using Criteria published by Bray and Sancio (2006)						
									Note: NP = Non-Plastic	Meets criteria for sand-like behavior		In Zone B		in B with w >= .85LL		Meets criteria for clay-like behavior				Meets criteria for sand-like behavior	Meets criteria for clay-like behavior	Meets criteria for sand-like behavior	Meets criteria for clay-like behavior	Borderline soils (treat as sand-like)		Meets all criteria for B (clay-like and potentially liquefiable, -2 indicates zone A but susceptible, -3 indicates not applicable due to fines content		Clay-like soil is susceptible (must meet both)		Clay-like soil is not susceptible (must meet one or both)		Clay-like soil is moderately susceptible		
Elevation at Midpoint (ft)	Lab ID	Boring	Depth(s) (ft)	Soil Classification	NMC (w_c) (%)	% Passing #200	% Passing #40	LL	PI	LL in Zone A (see plot)	PI in Zone A (see plot)	LL	PI	LL	PI	LL in Zone B (see plot)	PI in Zone B (see plot)	LL in Zone C (see plot)	PI in Zone C (see plot)	PI < 7	PI >= 7	PI <= 7	P40>=35%, P200>=20%, and PI>=10	7 < PI < 10, or does not meet P40 or P200	Overall Judgement based on 3 methods (sand-like or clay-like)	LL	PI	w_c/LL >= 0.85	PI <= 12	w_c/LL < 0.80	PI > 18	Intermediate w_c/LL (see plot)	Intermediate PI (see plot)	Overall Judgement based on 2 methods (susceptibility)
		STN-SW-1	14	CH	25.5	76.6	80.9	52	32	-1	-1	-1	-1	-1	-1	-1	-1	52	32	-1	32	-1	32	-1	**Clay-like**	-1	-1	-1.00	-1	0.49	32	-1.00	-1	**Not Susceptible**
		STN-SW-1	16.5	CH	30.9	76.1	82.1	53	32	-1	-1	-1	-1	-1	-1	-1	-1	53	32	-1	32	-1	32	-1	**Clay-like**	-1	-1	-1.00	-1	0.58	32	-1.00	-1	**Not Susceptible**
		STN-SW-1	21.5	CL	28.2	67.6	74	43	21	-1	-1	-1	-1	-1	-1	-1	-1	43	21	-1	21	-1	21	-1	**Clay-like**	-1	-1	-1.00	-1	0.66	21	-1.00	-1	**Not Susceptible**
		STN-SW-1	24	CL	23.2	85.9	95.9	38	19	-1	-1	38	19	-1	-1	38	19	-1	-1	-1	19	-1	19	-1	**Clay-like**	-1	-1	-1.00	-1	0.61	19	-1.00	-1	**Not Susceptible**
		STN-SW-1	26.5	CL	25	86.7	96.1	37	18	-1	-1	37	18	-1	-1	37	18	-1	-1	-1	18	-1	18	-1	**Clay-like**	-1	-1	-1.00	-1	0.68	18	-1.00	-1	**Not Susceptible**
		STN-SW-1	34	CH	36.9	93.5	96.1	73	47	-1	-1	-1	-1	-1	-1	-1	-1	73	47	-1	47	-1	47	-1	**Clay-like**	-1	-1	-1.00	-1	0.51	47	-1.00	-1	**Not Susceptible**
		STN-SW-1	39	CH	30.4	91.2	95.8	50	31	-1	-1	-1	-1	-1	-1	-1	-1	50	31	-1	31	-1	31	-1	**Clay-like**	-1	-1	-1.00	-1	0.61	31	-1.00	-1	**Not Susceptible**
		STN-SW-1	44	CH	34	86.5	89.4	68	43	-1	-1	-1	-1	-1	-1	-1	-1	68	43	-1	43	-1	43	-1	**Clay-like**	-1	-1	-1.00	-1	0.50	43	-1.00	-1	**Not Susceptible**
		STN-SW-2	11.5	SC	18.9	33	50.5	27	11	27	11	-1	-1	-1	-1	-1	-1	-1	-1	-1	11	-1	11	-1	**Clay-like**	-1	-1	-1.00	-1	0.70	11	-1.00	-1	**Not Susceptible**
		STN-SW-2	24	CL	24.6	82	94.8	37	18	-1	-1	37	18	-1	-1	37	18	-1	-1	-1	18	-1	18	-1	**Clay-like**	-1	-1	-1.00	-1	0.66	18	-1.00	-1	**Not Susceptible**
		STN-SW-2	26.5	CL	22.6	75.2	91.2	36	18	-1	-1	36	18	-1	-1	36	18	-1	-1	-1	18	-1	18	-1	**Clay-like**	-1	-1	-1.00	-1	0.63	18	-1.00	-1	**Not Susceptible**
		STN-SW-2	29	CL	23.7	58	66.6	44	24	-1	-1	-1	-1	-1	-1	-1	-1	44	24	-1	24	-1	24	-1	**Clay-like**	-1	-1	-1.00	-1	0.54	24	-1.00	-1	**Not Susceptible**
		STN-SW-3	16.5	CH	33.5	76.2	81.3	58	36	-1	-1	-1	-1	-1	-1	-1	-1	58	36	-1	36	-1	36	-1	**Clay-like**	-1	-1	-1.00	-1	0.58	36	-1.00	-1	**Not Susceptible**
		STN-SW-3	24	CL	26.8	79.9	88.4	38	18	-1	-1	38	18	-1	-1	38	18	-1	-1	-1	18	-1	18	-1	**Clay-like**	-1	-1	-1.00	-1	0.71	18	-1.00	-1	**Not Susceptible**
		STN-SW-3	26.5	CL	31.3	64.5	71.7	38	18	-1	-1	38	18	-1	-1	38	18	-1	-1	-1	18	-1	18	-1	**Clay-like**	-1	-1	-1.00	-1	-1.00	-1	0.82	18	**Moderately Susceptible**
		STN-SW-3	29	CL	30.5	74.9	81.4	43	22	-1	-1	-1	-1	-1	-1	-1	-1	43	22	-1	22	-1	22	-1	**Clay-like**	-1	-1	-1.00	-1	0.71	22	-1.00	-1	**Not Susceptible**

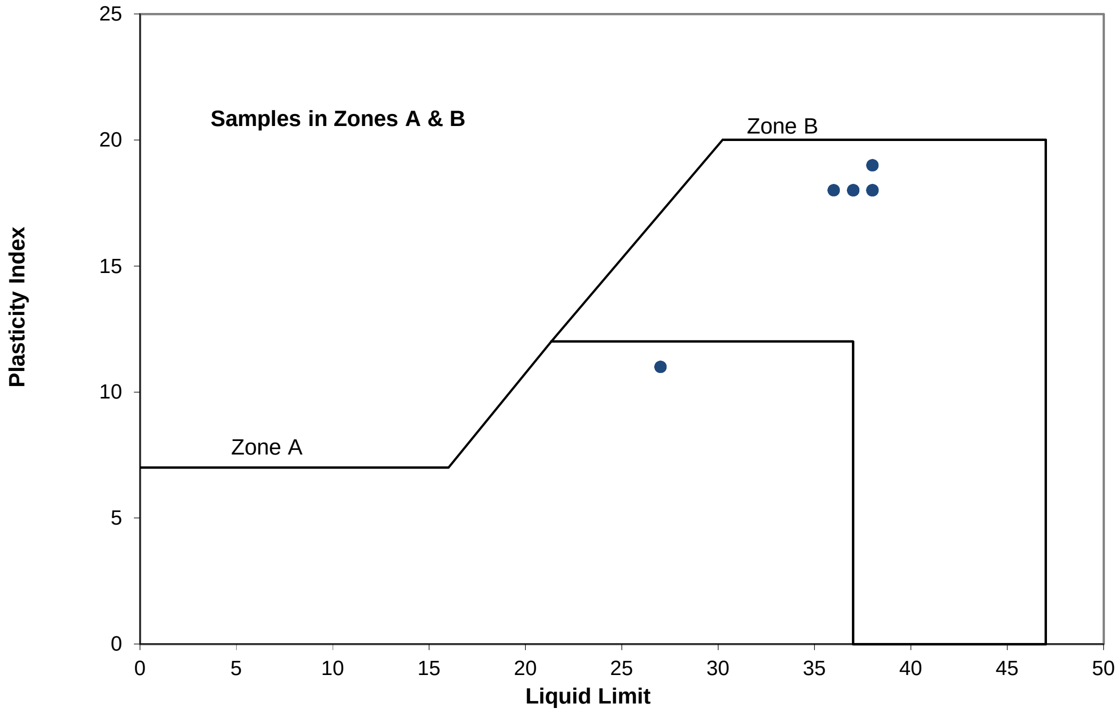


(a)



(b)

Screening Criteria for Liquefiable Fine-Grained Soils (Seed et al. 2003)



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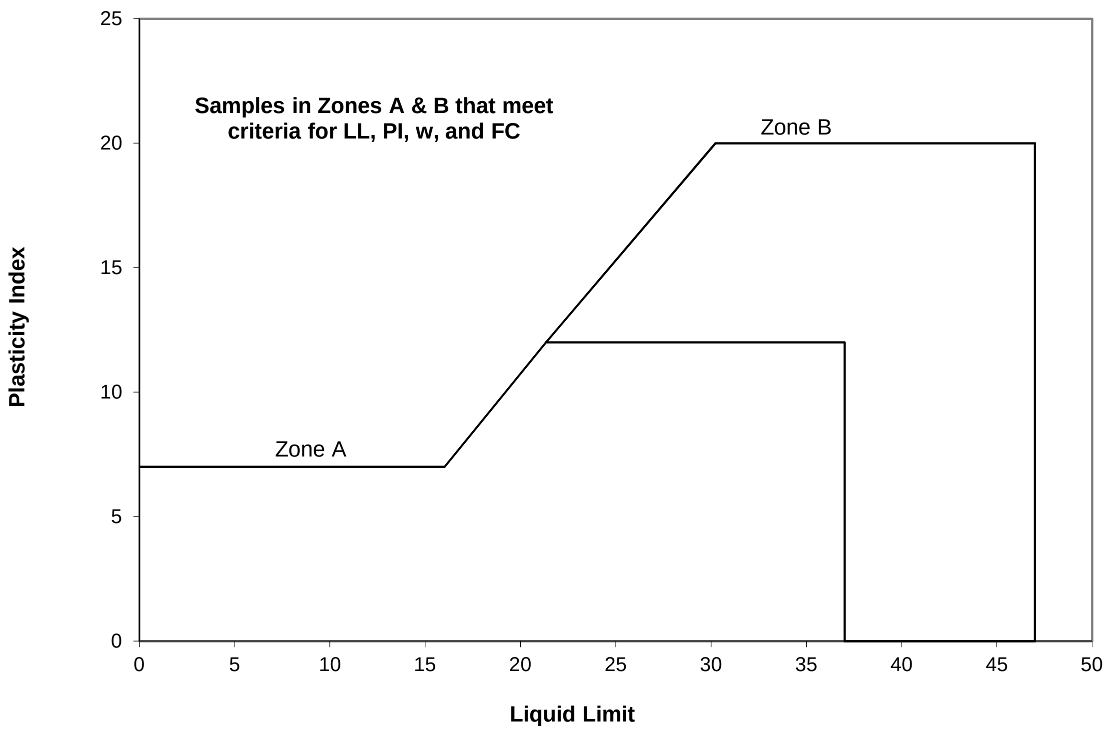
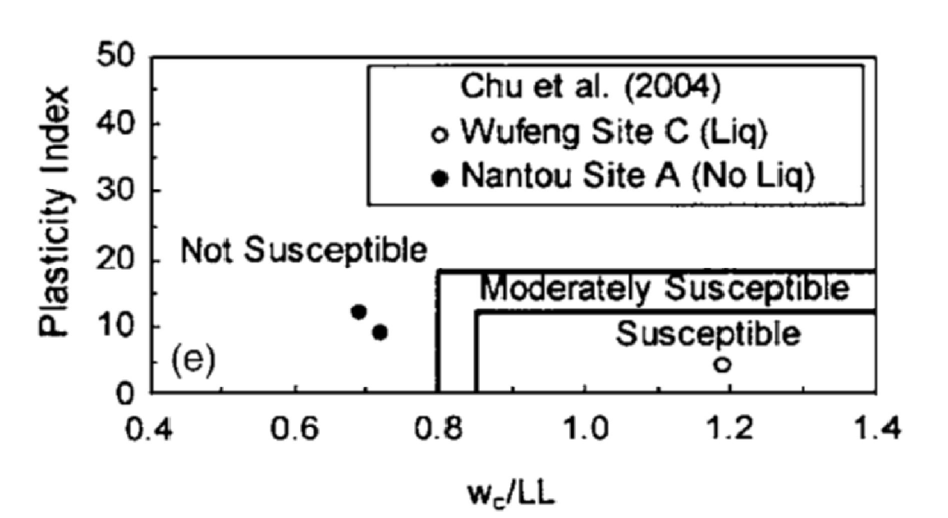
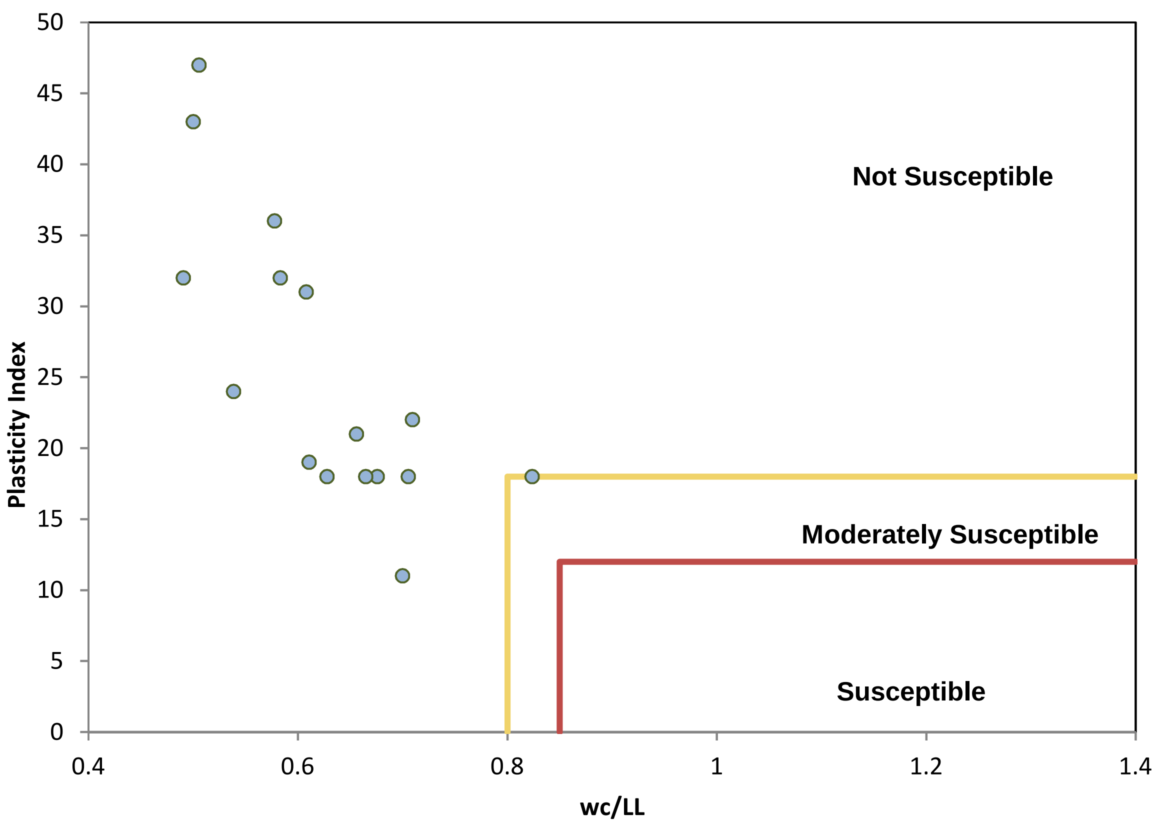


(d)

Screening Criteria for Liquefiable Fine-Grained Soils (Seed et al. 2003)

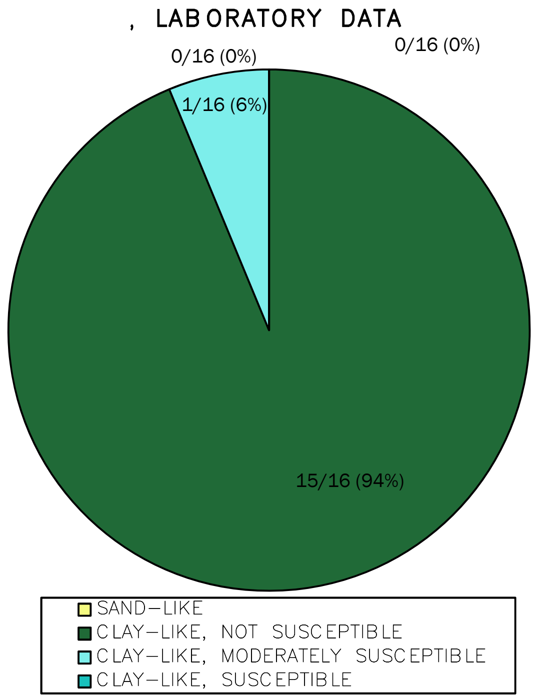




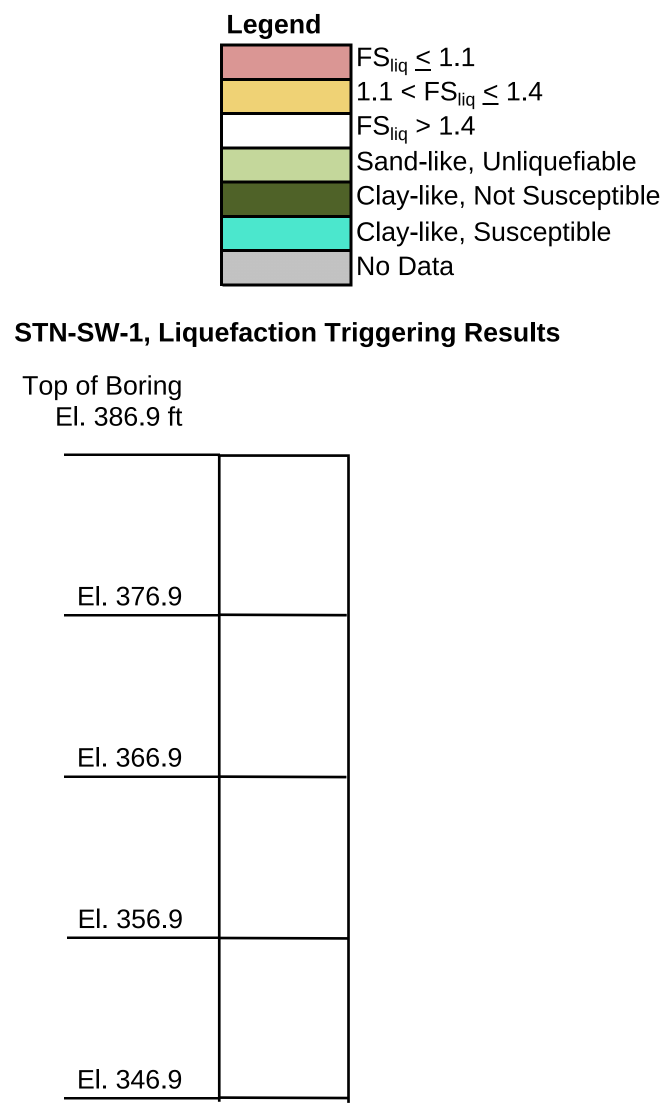
Screening Criteria for Assessing Liquefaction in Fine Grained Soils (Bray and Sancio 2006)

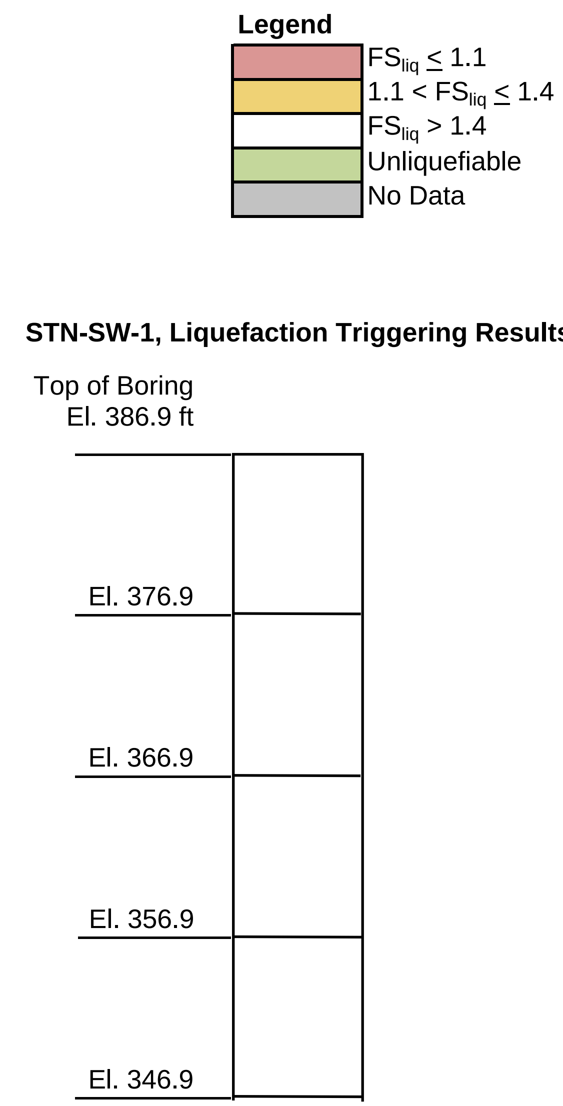
Fine-grained Screening and Cyclic Softening Susceptibility Summary

Soil Classification	Clay-like vs. Sand-like	Clay-like Susceptibility	Sample Count	Sample Percent
CH			**6**	**38%**
	Clay-like		6	100%
		Not Susceptible	6	100%
CL			**9**	**56%**
	Clay-like		9	100%
		Moderately Susceptible	1	11%
		Not Susceptible	8	89%
SC			**1**	**6%**
	Clay-like		1	100%
		Not Susceptible	1	100%
Grand Total			**16**	**100%**



ATTACHMENT C


Legend
$FS_{liq} \leq 1.1$
$1.1 < FS_{liq} \leq 1.4$
$FS_{liq} > 1.4$
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-SW-1, Liquefaction Triggering Results
Top of Boring
El. 386.9 ft
El. 376.9
El. 366.9
El. 356.9
El. 346.9


Legend
$FS_{liq} \leq 1.1$
$1.1 < FS_{liq} \leq 1.4$
$FS_{liq} > 1.4$
Unliquefiable
No Data
STN-SW-1, Liquefaction Triggering Results
Top of Boring
El. 386.9 ft
El. 376.9
El. 366.9
El. 356.9
El. 346.9

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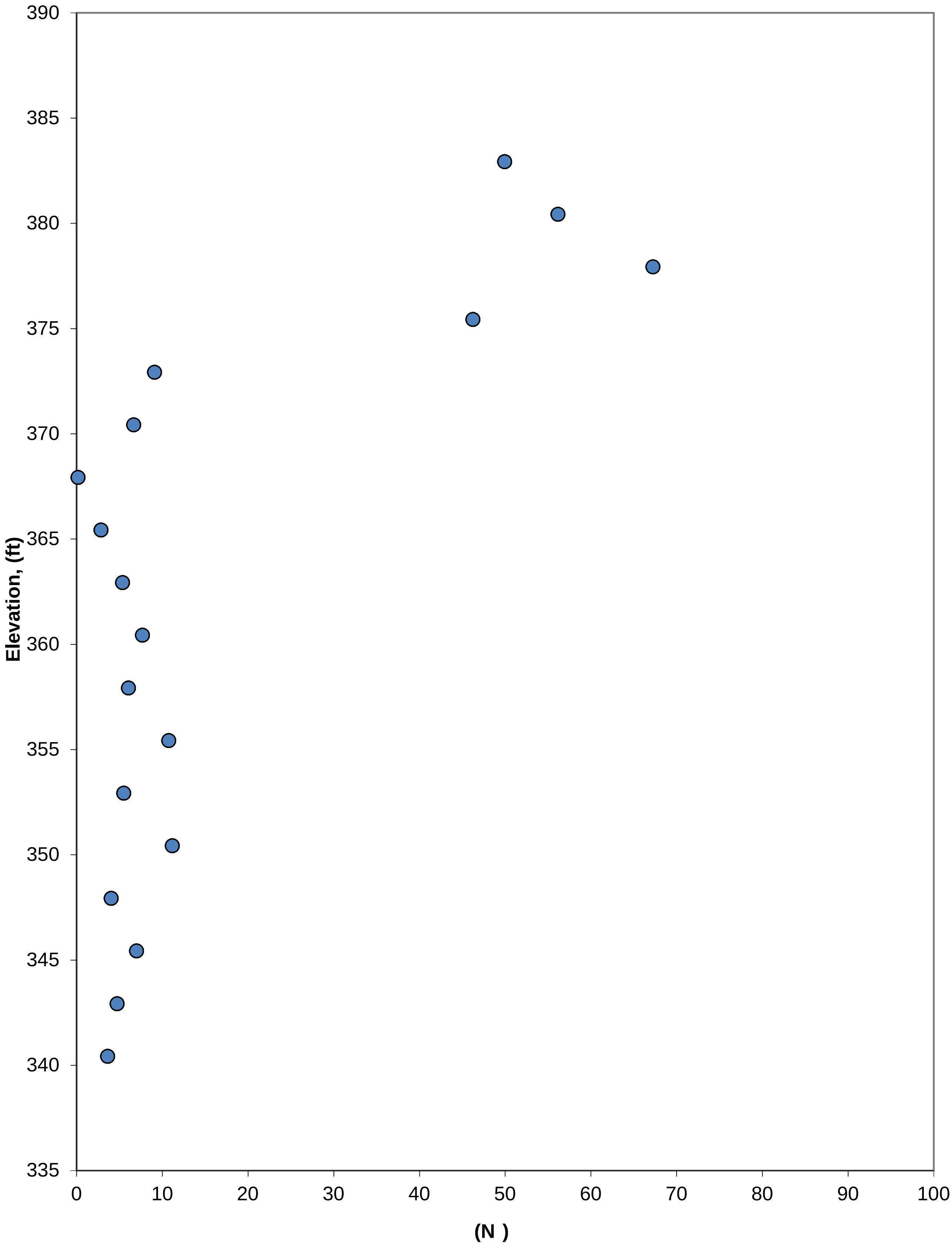
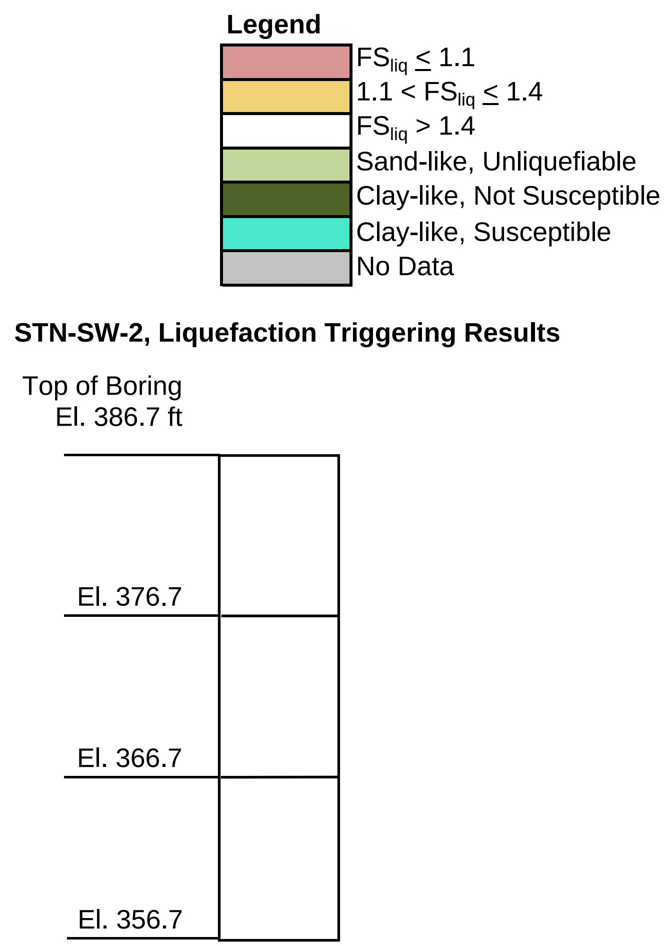
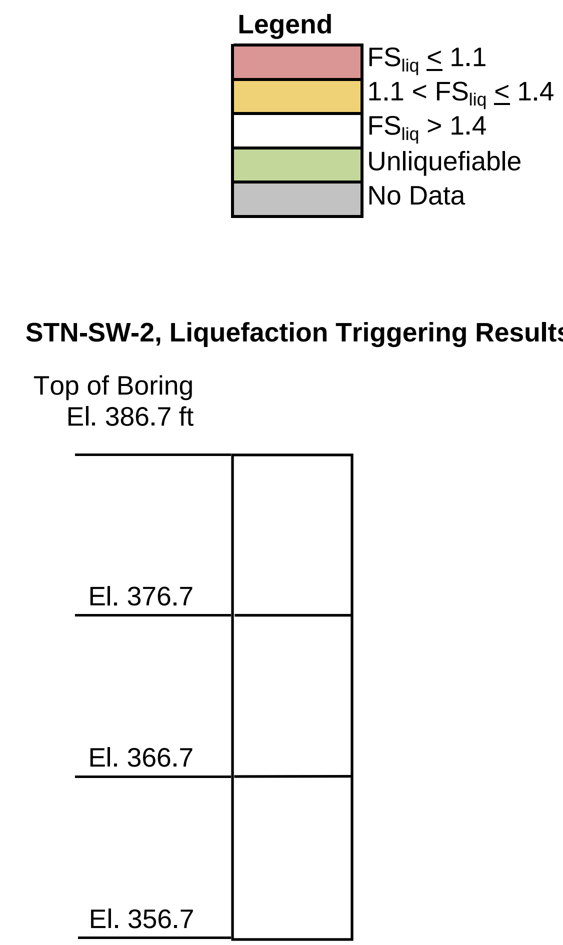
Simplified Stress-Based Approach



5/23/2018, 1 of 1


Legend
$FS_{liq} \leq 1.1$
$1.1 < FS_{liq} \leq 1.4$
$FS_{liq} > 1.4$
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-SW-2, Liquefaction Triggering Results
Top of Boring
El. 386.7 ft
El. 376.7
El. 366.7
El. 356.7


Legend
$FS_{liq} \leq 1.1$
$1.1 < FS_{liq} \leq 1.4$
$FS_{liq} > 1.4$
Unliquefiable
No Data
STN-SW-2, Liquefaction Triggering Results
Top of Boring
El. 386.7 ft
El. 376.7
El. 366.7
El. 356.7

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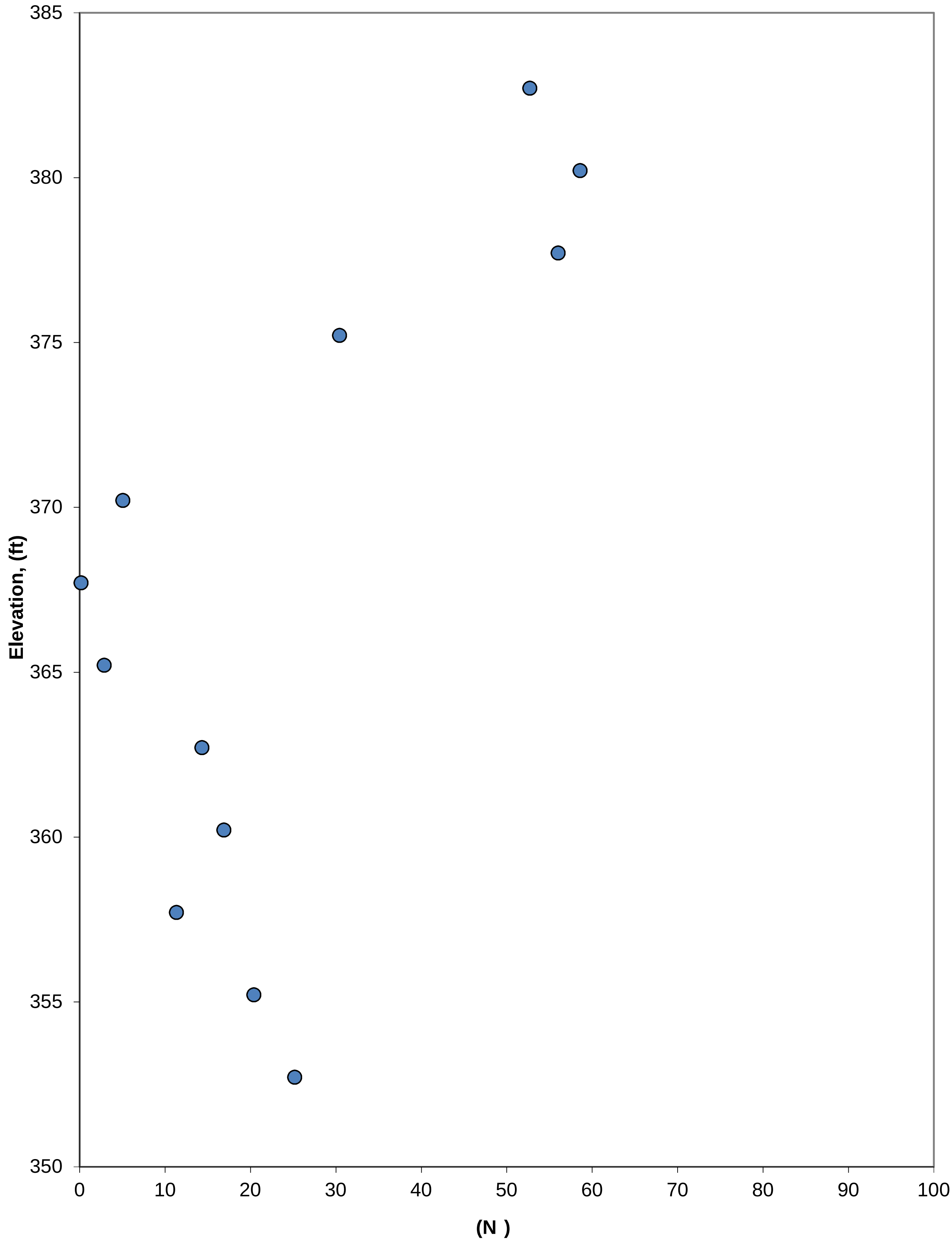
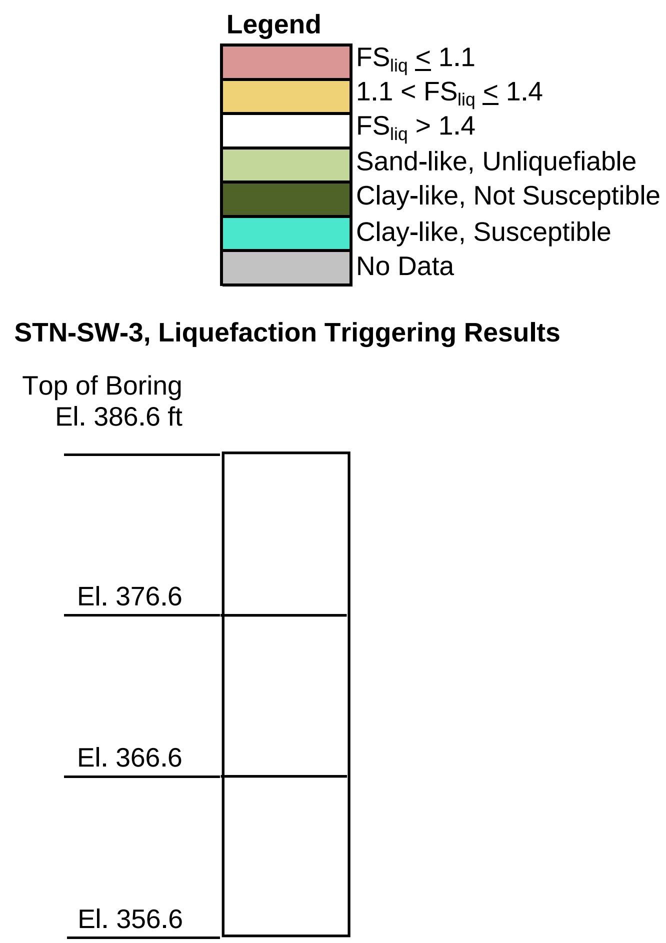
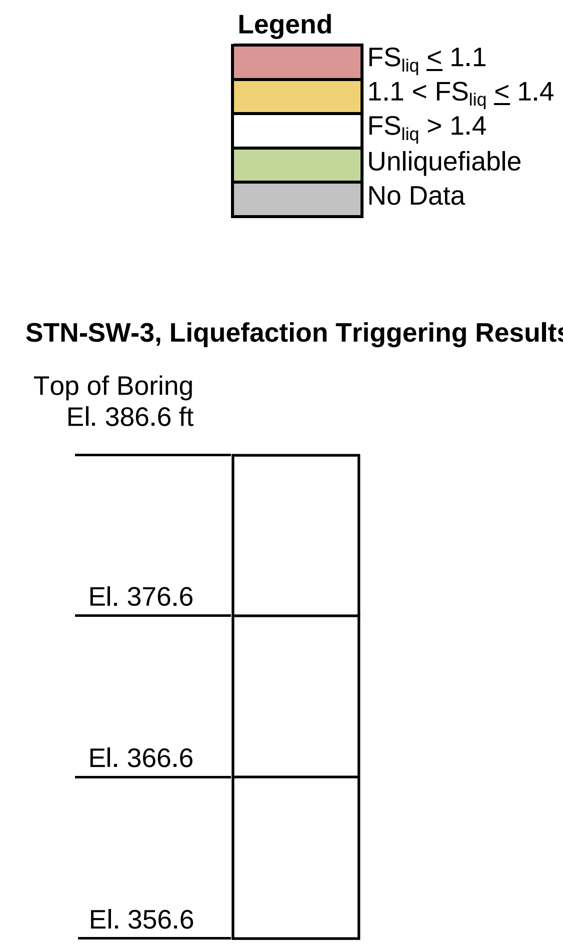
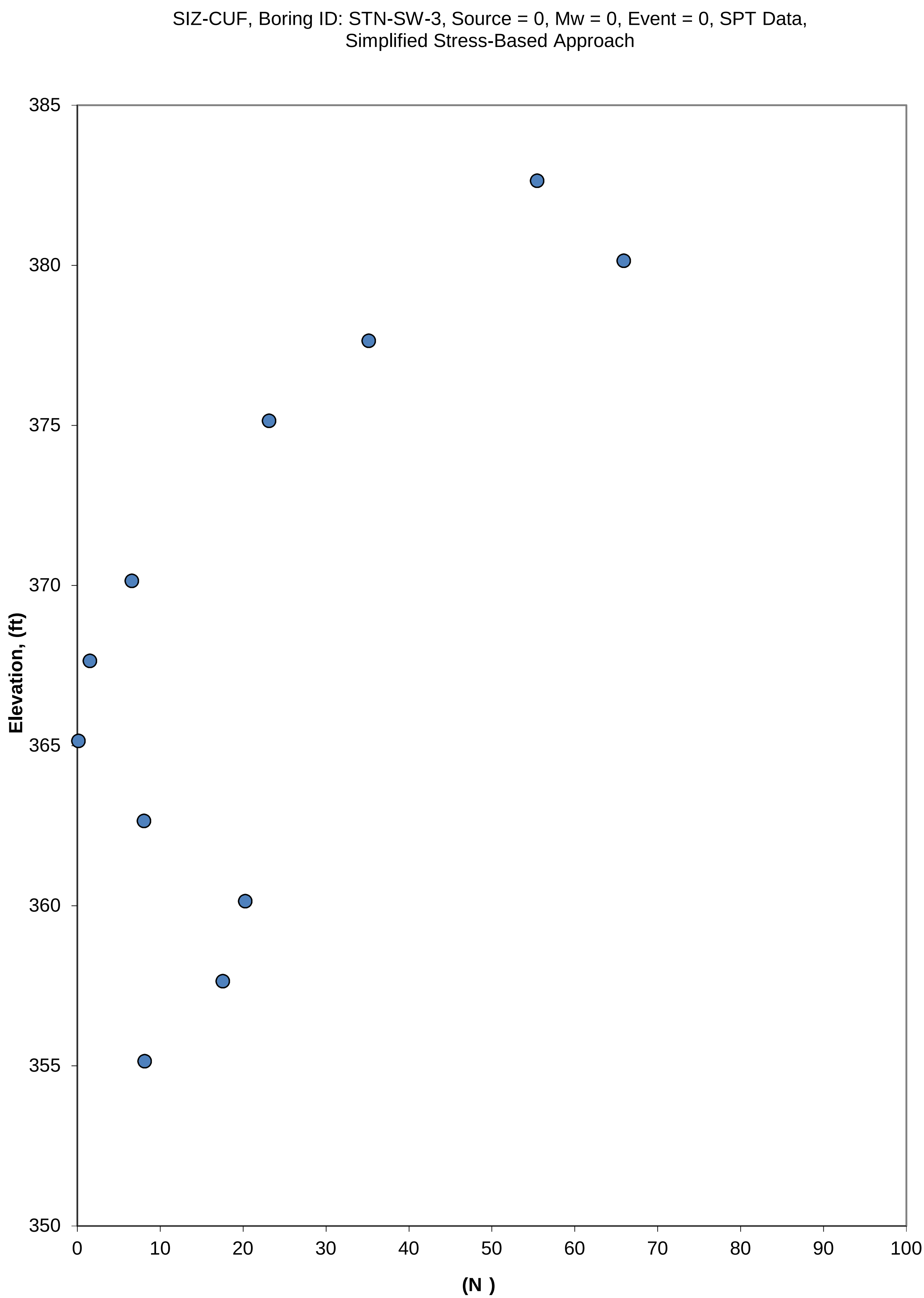
Simplified Stress-Based Approach




Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Sand-like, Unliquefiable
Clay-like, Not Susceptible
Clay-like, Susceptible
No Data
STN-SW-3, Liquefaction Triggering Results
Top of Boring
El. 386.6 ft
El. 376.6
El. 366.6
El. 356.6


Legend
FSliq ≤ 1.1
1.1 < FSliq ≤ 1.4
FSliq > 1.4
Unliquefiable
No Data
STN-SW-3, Liquefaction Triggering Results
Top of Boring
El. 386.6 ft
El. 376.6
El. 366.6
El. 356.6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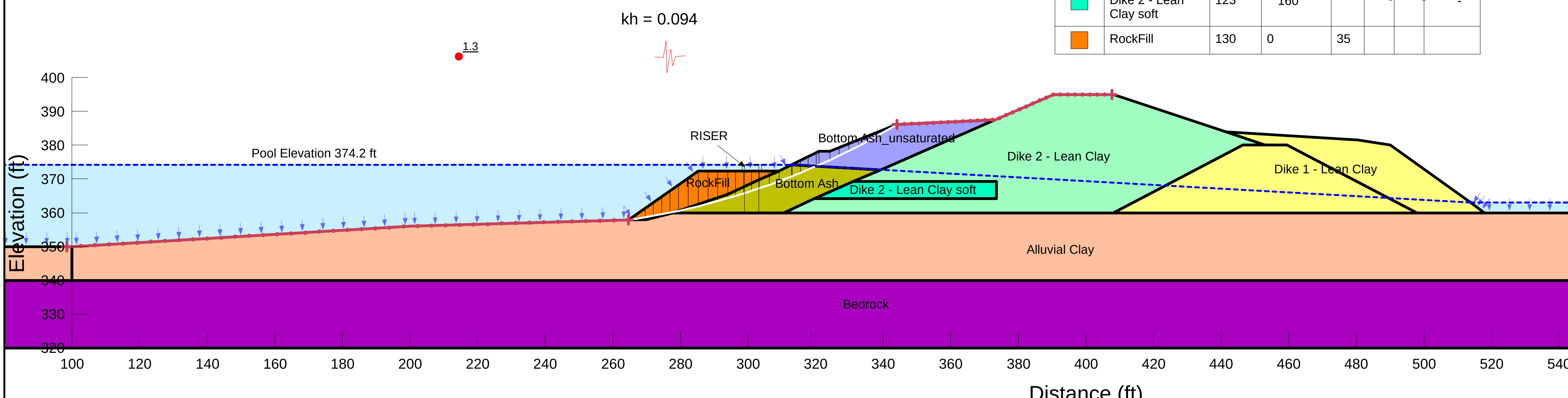
Simplified Stress-Based Approach
Elevation, (ft)
385
380
375
370
365
360
355
350
0
10
20
30
40
50
60
70
80
90
100
(N)

ATTACHMENT D

TVA	Cumberland Fossil Plant Facility	**Pseudostatic Slope Stability Analysis**
	Stewart County, Tennessee	
	Cumberland Spillway Section, Profile - Section A-A'	

Note: The results of the analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties.The drawing depicts approximate subsurface conditions based on historical drawings or specific borings at the time of drilling. No warranties can be made regarding the continuity of subsurface conditions.

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (psf)
	Alluvial Clay	124	200		33	16	441.2
	Bedrock						
	Bottom Ash	105	0	29			
	Bottom Ash_unsaturated	100	0	35			
	Dike 1 - Lean Clay	123	200		22	16	3,751.7
	Dike 2 - Lean Clay	123	200		32	17	626.7
	Dike 2 - Lean Clay soft	123	160		-	-	-
	RockFill	130	0	35			

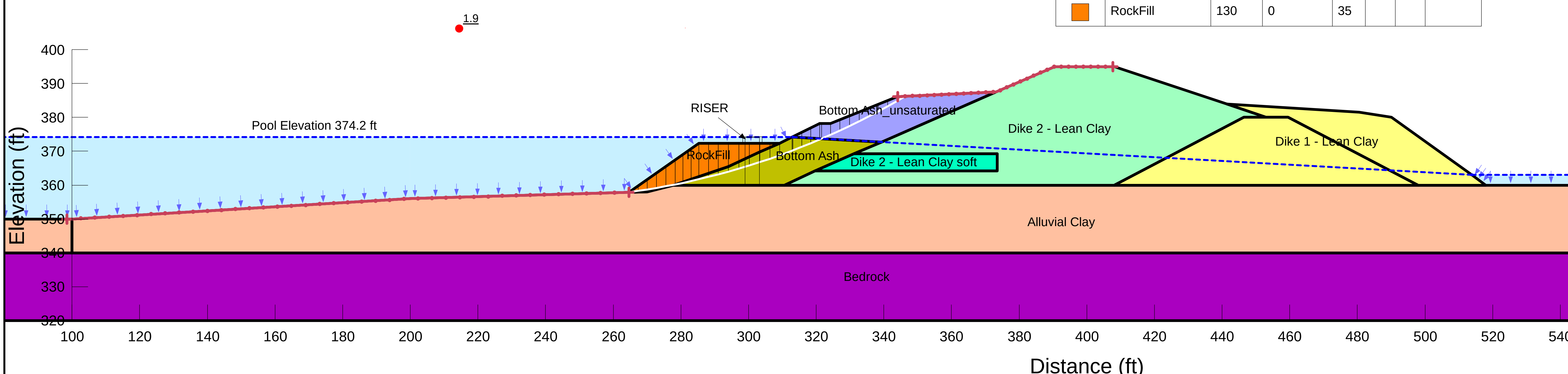


Project No. 175567307

06/04/2018

Cumberland Fossil Plant Facility	**Post-Earthquake Slope Stability Analysis**	Note: The results of the analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties.The drawing depicts approximate subsurface conditions based on historical drawings or specific borings at the time of drilling. No warranties can be made regarding the continuity of subsurface conditions.
Stewart County, Tennessee		
Cumberland Spillway Section, Profile - Section A-A'		

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (psf)
	Alluvial Clay	124	200		33	16	441.2
	Bedrock						
	Bottom Ash	105	0	29			
	Bottom Ash_unsaturated	100	0	35			
	Dike 1 - Lean Clay	123	200		22	16	3,751.7
	Dike 2 - Lean Clay	123	200		32	17	626.7
	Dike 2 - Lean Clay soft	123	160		-	-	-
	RockFill	130	0	35			



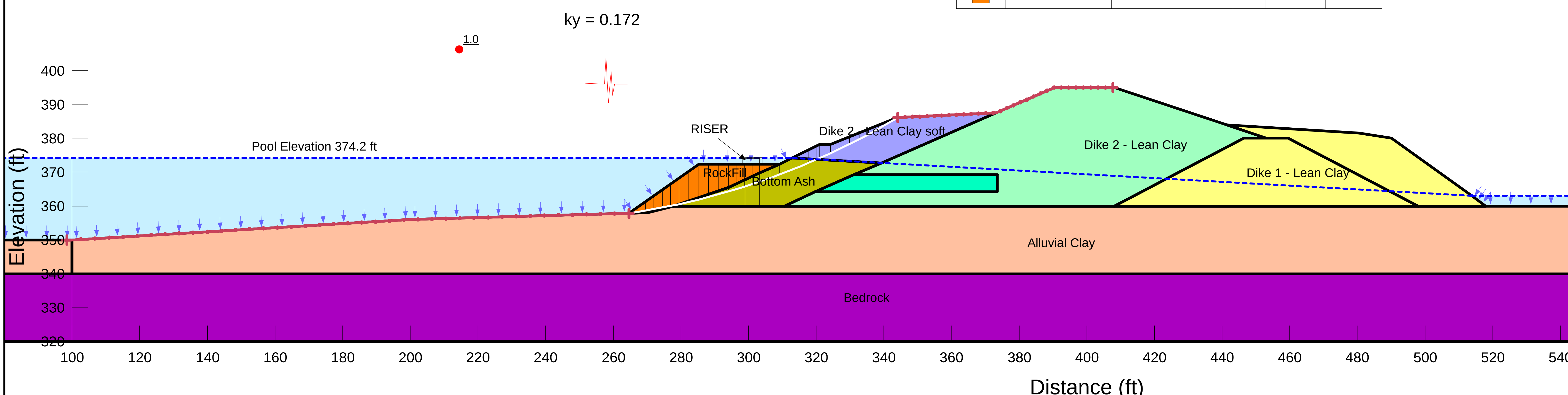
Project No. 175567307

06/04/2018

ATTACHMENT E

Cumberland Fossil Plant Facility	**Pseudostatic Slope Stability Analysis**	Note: The results of the analysis shown here are based on available subsurface information, laboratory test results and approximate soil properties.The drawing depicts approximate subsurface conditions based on historical drawings or specific borings at the time of drilling. No warranties can be made regarding the continuity of subsurface conditions.
Stewart County, Tennessee	Yield Acceleration	
Cumberland Spillway Section, Profile - Section A-A'		

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi 1 (°)	Phi 2 (°)	Bilinear Normal (psf)
	Alluvial Clay	124	200		33	16	441.2
	Bedrock						
	Bottom Ash	105	0	29			
	Bottom Ash_unsaturated	100	0	35			
	Dike 1 - Lean Clay	123	200		22	16	3,751.7
	Dike 2 - Lean Clay	123	200		32	17	626.7
	Dike 2 - Lean Clay soft	123	160		-	-	-
	RockFill	130	0	35			



Project No. 175567307

06/04/2018

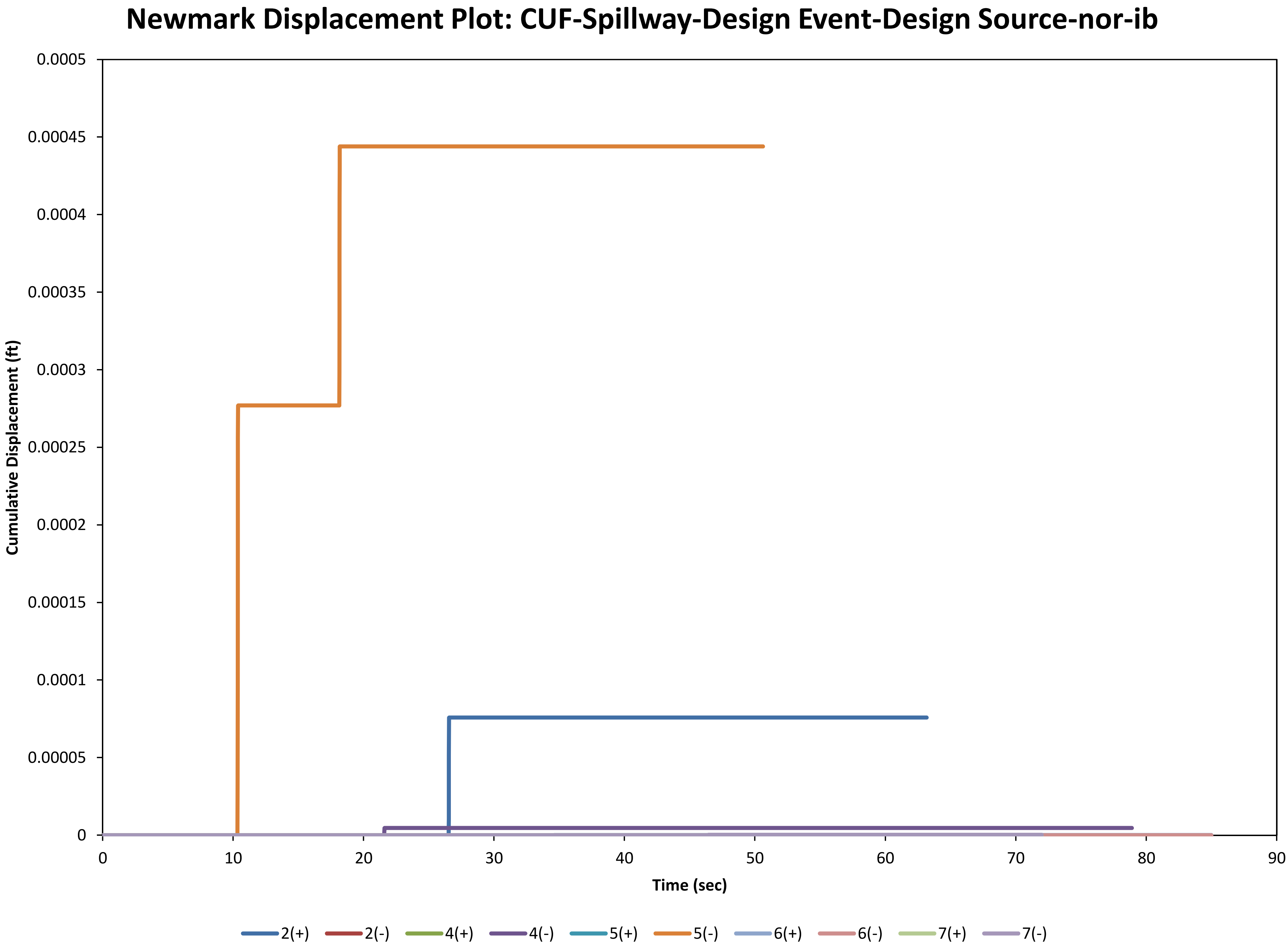
Facility	Cumberland
Section/Group	Spillway
Profile Name	seism_normal
Top of Profile Elevation	383.8 ft
Depth to water table during EQ	11 ft
Surcharge Pressure	0 psf

Strata Profile

Non-input Information								**Strata "Soil Types" Input**				**Strata "Soil Profile" Input**		
Top Depth (ft)	**Bottom Depth (ft)**	**Layer Name**	**Layer Type**	**G_{max} (psf)**	**K_o**	**Mean Effective Stress at midpoint (psf)**	**Plasticity Index**	**Name (for Strata)**	**Unit Weight (pcf)**	**Modulus Reduction (G/Gmax) Model**	**Damping Model**	**Thickness (ft)**	**Soil Type (for Strata)**	**Shear Wave Velocity (fps)**
0	1.69	Bottom Ash	Soil	1.50E+06	0.5	59.2	0	Bottom Ash_1	105	I&Z, p'=0-400, PI=0	I&Z, p'=0-400, PI=0	1.69	Bottom Ash_1	679
1.69	3.38	Bottom Ash	Soil	1.50E+06	0.5	177.5	0	Bottom Ash_2	105	I&Z, p'=0-400, PI=0	I&Z, p'=0-400, PI=0	1.69	Bottom Ash_2	679
3.38	5.07	Bottom Ash	Soil	1.50E+06	0.5	295.8	0	Bottom Ash_3	105	I&Z, p'=0-400, PI=0	I&Z, p'=0-400, PI=0	1.69	Bottom Ash_3	679
5.07	6.76	Bottom Ash	Soil	1.50E+06	0.5	414.1	0	Bottom Ash_4	105	I&Z, p'=400-800, PI=0	I&Z, p'=400-800, PI=0	1.69	Bottom Ash_4	679
6.76	8.45	Bottom Ash	Soil	1.50E+06	0.5	532.4	0	Bottom Ash_5	105	I&Z, p'=400-800, PI=0	I&Z, p'=400-800, PI=0	1.69	Bottom Ash_5	679
8.45	10.14	Bottom Ash	Soil	1.50E+06	0.5	650.7	0	Bottom Ash_6	105	I&Z, p'=400-800, PI=0	I&Z, p'=400-800, PI=0	1.69	Bottom Ash_6	679
10.14	11	Bottom Ash	Soil	1.50E+06	0.5	739.9	0	Bottom Ash_7	105	I&Z, p'=400-800, PI=0	I&Z, p'=400-800, PI=0	0.86	Bottom Ash_7	679
11	11.47	Bottom Ash	Soil	1.50E+06	0.5	776.7	0	Bottom Ash_8	105	I&Z, p'=400-800, PI=0	I&Z, p'=400-800, PI=0	0.47	Bottom Ash_8	679
11.47	13.26	Lean Clay	Soil	1.97E+06	0.5	819.5	22	Lean Clay_9	123	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.79	Lean Clay_9	718
13.26	14.6	Lean Clay	Soil	1.97E+06	0.5	882.7	22	Lean Clay_10	123	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.34	Lean Clay_10	718
14.6	15.84	Soft Lean Clay	Soil	9.13E+05	0.5	932.8	22	Soft Lean Clay_11	118	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.24	Soft Lean Clay_11	499
15.84	17.08	Soft Lean Clay	Soil	9.13E+05	0.5	978.7	22	Soft Lean Clay_12	118	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.24	Soft Lean Clay_12	499
17.08	18.32	Soft Lean Clay	Soil	9.13E+05	0.5	1024.7	22	Soft Lean Clay_13	118	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.24	Soft Lean Clay_13	499
18.32	19.56	Soft Lean Clay	Soil	9.13E+05	0.5	1070.7	22	Soft Lean Clay_14	118	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.24	Soft Lean Clay_14	499
19.56	19.6	Soft Lean Clay	Soil	9.13E+05	0.5	1094.4	22	Soft Lean Clay_15	118	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	0.04	Soft Lean Clay_15	499
19.6	21.08	Lean Clay Bottom	Soil	1.31E+06	0.5	1123.5	22	Lean Clay Bottom_16	120	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.48	Lean Clay Bottom_16	593
21.08	22.56	Lean Clay Bottom	Soil	1.31E+06	0.5	1180.4	22	Lean Clay Bottom_17	120	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.48	Lean Clay Bottom_17	593
22.56	23.8	Lean Clay Bottom	Soil	1.31E+06	0.5	1232.6	22	Lean Clay Bottom_18	120	I&Z, p'=800-1600, PI=20-25	I&Z, p'=800-1600, PI=20-25	1.24	Lean Clay Bottom_18	593
23.8	Half-Space	Bedrock	Half-Space	1.16E+08	0.5	Half-Space	0	Bedrock	150	NA, Half-Space	Damping = 0.5%	Half Space	Bedrock	5000

Site: **Cumberland**

						For All Motions
Section	**EQ**	**Pool**	**Motion**	**Yield Acc. (g)**	**Max. HEA (g)**	**Max. Deformation (ft)**
Spillway	Design Event-Design Source	nor	ib	0.172	0.19	<0.1



Newmark Displacement Plot: CUF-Spillway-Design Event-Design Source-nor-ib
Cumulative Displacement (ft)
0.0005
0.00045
0.0004
0.00035
0.0003
0.00025
0.0002
0.00015
0.0001
0.00005
0
0
10
20
30
40
50
60
70
80
90
Time (sec)
2(+)
2(-)
4(+)
4(-)
5(+)
5(-)
6(+)
6(-)
7(+)
7(-)

APPENDIX B
STRUCTURAL ANALYSES

Seismic Assessment of Riser Structure at Cumberland Fossil Plant

Stability analysis was performed for the ash pond riser tower at Cumberland Fossil Plant. The loading condition analyzed was the 2500-year earthquake event. The structure was assessed according to guidelines presented in US Army Corps of Engineers EM 1110-2-2400 “Structural Design and Evaluation of Outlet Works”. Hydrodynamic added masses were calculated per the methodology outlined in “Earthquake Analysis and Response of Intake Towers” by Anil A. Chopra.

A lumped mass model of the riser structure was used for the analysis. The structure was assessed for stability at its base and at the various riser pipe joints. Since the pipe segments are connected via keyed joints, sliding was assumed to be a non-viable failure mode at the pipe joints. Therefore, the structure was assessed for sliding and bearing capacity failure only at its base,and resultant location (overturning) assessments were performed at the base of the structure and at the four riser pipe joints. The analysis results and supporting calculations are provided in the following pages. The structural capacity of the pipe cross sections were assumed to be adequade by inspection. The results indicate the riser structure meets stability criteria for the 2500-year earthquake event.

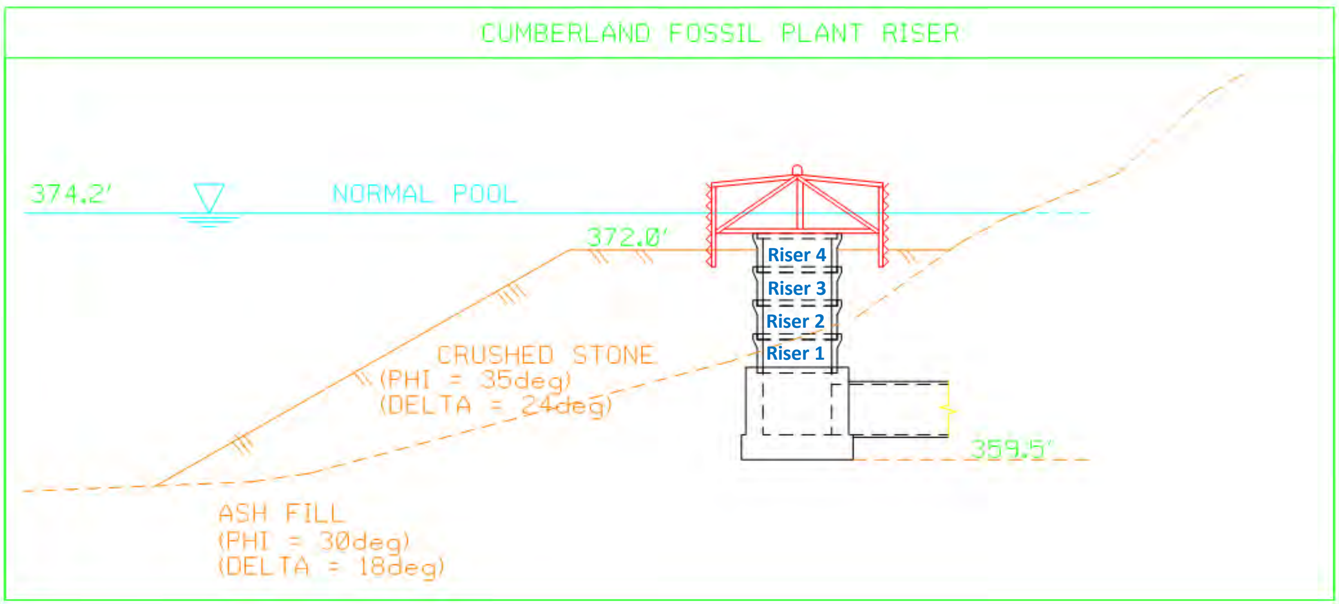
Project: Cumberland Fossil Plant - Seismic Stability Demonstration For Inlet Structure
Site: CUF
Load: 2500 Year Earthquake

By: C. Gabriel
Date: 7/9/2018

Stability Analysis Summary of Results							
Failure Plane	Elevation (ft)	Sliding Stability Safety Factor		Resultant Location*		Bearing Capacity SF	
		Calculated	Required	Calculated	Required	Calculated	Required
Base of Structure	359.50	6.44	1.10	0.12	Within Base	1.21	1.00
Bottom of Riser 1	365.00	N/A[1]		0.08		N/A[1]	
Bottom of Riser 2	367.00			0.13			
Bottom of Riser 3	369.00			0.16			
Bottom of Riser 4	371.00			0.26			

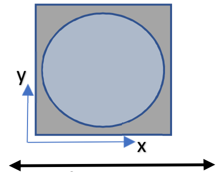
* : Normalized resultant locations within the base of the structure range from 0 to 1.0

1 : Failure mode is not applicable along this failure plane



Project: Cumberland Fossil Plant - Seismic Stability Demonstration For Inlet Structure
Site: CUF
Load: 2500 Year Earthquake
Direction: x-axis (Upstream/downstream)
Calculations: Global Stability Analysis
Failure Plane: Base of Structure

By: C. Gabriel
Date: 7/6/2018
Checked: JTP/PRS
Check Date: 7/6/2018



Direction of Ground Motion

Unit Weight of Water (pcf)	62.4	Allowable Bearing Capacity (ksf)	2.25	Horiz. Seis. Coef.: k_h	0.368	
Saturated Soil Unit Weight (pcf)	119	Backfill Elevation (ft)	372	Active LEP Coef.: k_a	0.270	Static Earth
Soil Internal Friction Angle (deg)	32.5	Reservoir Elevation (ft)	374.2	Passive LEP Coef.: k_p	3.840	
Soil-Wall Friction Angle (deg)	21	Water Depth (ft) = H_o	14.70	Active LEP Coef.: k_{ae}	0.620	Seismic Earth
		Mass Density of Water ($lbm*s^2/ft^4$)	1.939	Passive LEP Coef.: k_{pe}	4.940	

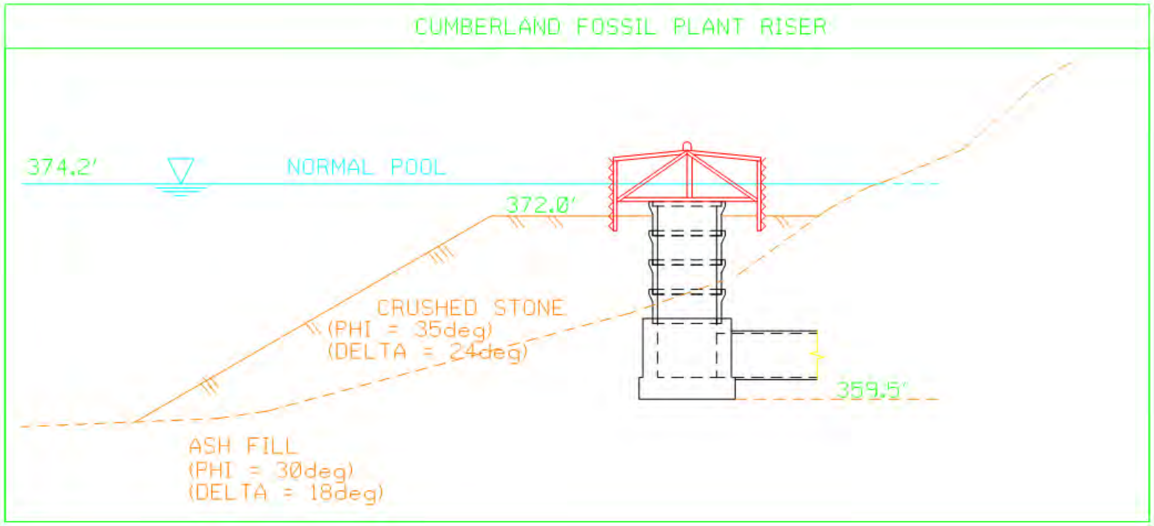
Node ID	Elevation (ft)	Arm, Z_o (ft)	Height, L (ft)	Width (ft)	Self Weight (kip)	Inertial Force (kip)	Hydrodynamic Added Mass Inside (kip)	Hydrodynamic Added Mass Outside[1] (kip)	Static Soil Pressure Active (Kip)	Static Soil Pressure Passive (Kip)	Dynamic Soil Pressure Active (Kip)	Dynamic Soil Pressure Passive (Kip)
	359.50											
1 (bottom of base)	359.50	0.00	0.00	6.50	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2 (mid base)	360.25	0.75	2.13	6.50	9.5	3.5	0.0	0.0	2.4	34.3	0.6	-1.9
3 (mid box)	363.00	3.50	2.66	6.00	10.9	4.0	2.7	0.0	2.2	31.4	2.3	-7.2
4 (mid riser 1)	365.57	6.07	2.29	5.85	1.4	0.5	1.8	0.0	1.3	19.1	3.3	-10.4
5 (mid riser 2)	367.57	8.07	2.00	4.67	1.4	0.5	1.6	0.0	0.6	9.0	3.1	-9.9
6 (mid riser 3)	369.57	10.07	2.00	4.67	1.4	0.5	1.5	0.0	0.3	4.9	3.9	-12.3
7 (mid riser 4)	371.57	12.07	3.41	4.67	1.4	0.5	4.3	0.0	0.1	1.0	3.3	-10.3
8 (centroid of skimmer)	376.38	16.88	3.53	10.00	2.0	0.7	0.0	0.0	0.0	0.0	0.0	0.0
Top Elevation	377.50				27.9	10.3	11.8	0.0	7.0	99.8	16.5	-52.0

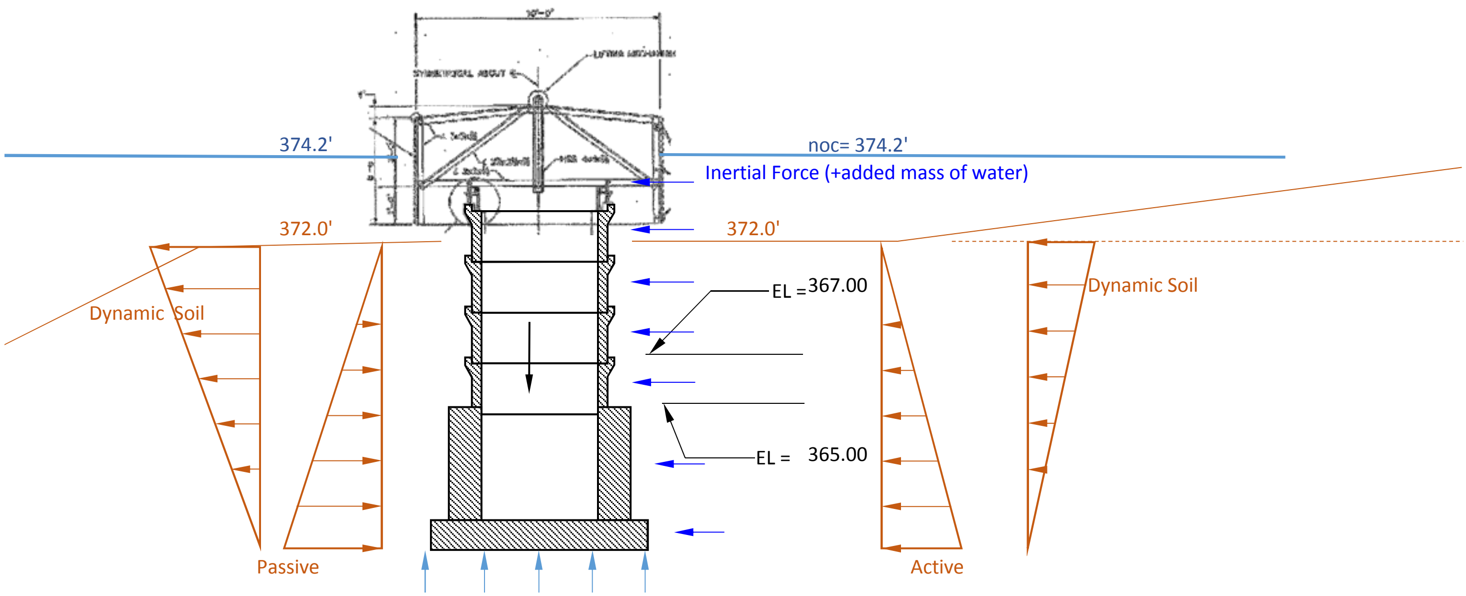
Load Summary	Load (kip)	Moment (kip-ft)	
Uplift	38.76	125.95	Drive
Soil Weight on Base	11.38	36.99	Resist
Water Weight on Base	16.00	52.00	Resist
Water Weight Inside	11.21	36.43	Resist
Self-Weight	27.88	90.61	Resist
Self-Weight Inertia	10.26	47.30	Drive
Static Active Soil (horiz)	7.02	29.23	Drive
Static Passive Soil (horiz)	99.78	199.10	Resist
Static Active Soil (vert)	2.51	16.34	Resist
Static Passive Soil (vert)	0.00	0.00	Drive
Seismic Active Soil (horiz)	16.54	138.56	Drive
Seismic Passive Soil (horiz)	-52.00	0.00	Resist
Seismic Active Soil (vert)	5.93	38.54	Resist
Seismic Passive Soil (vert)	0.00	0.00	Drive
Hydrodynamic Added Mass	11.81	99.88	Drive

Sliding Stability Assessment			
Total Vertical Load (kip)	36.16		
Total Horizontal Load (kip)	-2.15		
Total Interface Resisting Load (kip)	13.88		
Req'd Sliding Safety Factor	1.10		
Sliding FS W/ Partial Passive Soil Resistance		6.44	
Percent of Passive Resistance Needed		100%	
Overturning Stability and Bearing Capacity Assessments			
Total Resisting Moment (kip-ft)	470.01		
Total Driving Moment (kip-ft)	440.92		
Net Moment (kip-ft)	29.09		
Resultant Location from Toe (Normalized)		0.12	Within Base
Eccentricity from Center of Base (ft)	2.45		
Allowable Bearing Capacity (ksf)	3.38		
Bearing Capacity Assessment			
Bearing Pressure at Toe (ksf)		2.79	OK
Bearing Pressure at Heel (ksf)		-1.08	OK

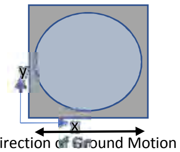
(1): Surrounding water above ground surface is negligible.

Failure Plane: Base of Structure





Project:	**Cumberland Fossil Plant - Seismic Stability Demonstration For Inlet Structure**
Site:	**CUF**
Load:	**2500 Year Earthquake**
Direction:	**x-axis (Upstream/downstream)**
Calculations:	**Global Stability Analysis**
Failure Plane:	**Bottom of Riser 1**



Direction of Ground Motion

By:	C. Gabriel
Date:	7/6/2018
Checked:	JTP/PRS
Check Date:	7/6/2018

Unit Weight of Water (pcf)	62.4	Allowable Bearing Capacity (ksf)	2.25	Horiz. Seis. Coef.: k_h	0.368	
Saturated Soil Unit Weight (pcf)	119	Backfill Elevation (ft)	372.00	Active LEP Coef.: k_a	0.270	Static
Soil Internal Friction Angle (deg)	32.5	Reservoir Elevation (ft)	374.20	Passive LEP Coef.: k_p	3.840	Earth
Soil-Wall Friction Angle (deg)	21	Water Depth (ft) = H_o	9.20	Active LEP Coef.: k_{ae}	0.620	Seismic
		Mass Density of Water ($lbm*s^2/ft^4$)	1.939	Passive LEP Coef.: k_{pe}	4.940	Earth

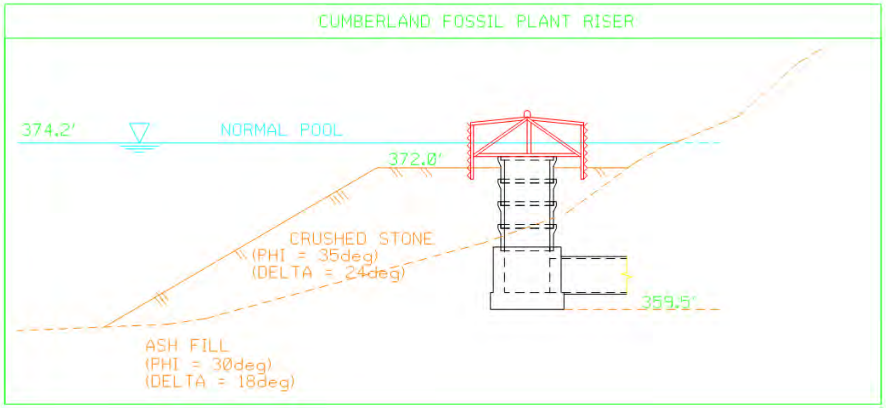
Node ID	Elevation	Arm, Z_o	Height, L	Width	Self Weight	Inertial Force		Hydrodynamic Added Mass				Static Soil Pressure				Dynamic Soil Pressure			
								Inside		Outside[1]		Active		Passive		Active		Passive	
	(ft)	(ft)	(ft)	(ft)	(kip)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)
	365.00																		
(bottom of riser 1)	365.00	0.00	0.00	5.85	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4 (mid riser 1)	365.57	0.57	1.29	4.67	1.4	0.5	0.3	1.8	1.0	0.0	0.0	0.6	0.3	8.3	4.7	0.2	0.1	-0.5	-0.3
5 (mid riser 2)	367.57	2.57	2.00	4.67	1.4	0.5	1.3	1.6	4.0	0.0	0.0	0.7	1.7	9.6	24.6	0.9	2.3	-2.8	-7.2
6 (mid riser 3)	369.57	4.57	2.00	4.67	1.4	0.5	2.3	1.5	6.8	0.0	0.0	0.4	1.8	5.5	25.2	1.7	7.6	-5.2	-23.9
7 (mid riser 4)	371.57	6.57	3.41	4.67	1.4	0.5	3.3	4.3	28.4	0.0	0.0	0.1	0.7	1.5	9.8	2.0	13.5	-6.4	-42.3
8 (centroid of skimmer)	376.38	11.38	3.53	10.00	2.0	0.7	8.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Top Elevation	377.50				**7.4**	**2.7**	**15.5**	**9.2**	**40.2**	**0.0**	**0.0**	**1.7**	-	**24.9**	-	**4.8**	-	**-15.0**	-

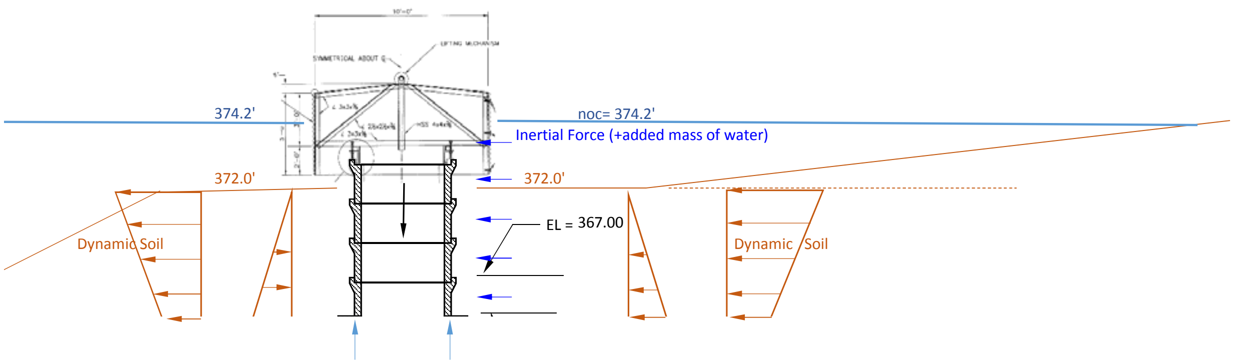
Load Summary	Load (kip)	Moment (kip-ft)	
Uplift	8.2	24.1	Drive
Soil Weight on Base	7.5	21.9	Resist
Water Weight on Base	10.9	31.8	Resist
Water Weight Inside	0.0	0.0	Resist
Self-Weight	7.4	21.7	Resist
Self-Weight Inertia	2.7	12.6	Drive
Static Active Soil (horiz)	1.7	4.1	Drive
Static Passive Soil (horiz)	24.9	23.1	Resist
Static Active Soil (vert)	0.6	3.7	Resist
Static Passive Soil (vert)	0.0	0.0	Drive
Seismic Active Soil (horiz)	4.8	22.3	Drive
Seismic Passive Soil (horiz)	-15.0	0.0	Resist
Seismic Active Soil (vert)	1.7	10.0	Resist
Seismic Passive Soil (vert)	0.0	0.0	Drive
Hydrodynamic Added Mass	9.2	40.2	Drive

Sliding Stability Assessment			
Total Vertical Load (kip)	**19.9**		
Total Horizontal Load (kip)	**8.5**		
Total Interface Resisting Load (kip)	**7.6**		
Req'd Sliding Safety Factor	1.10		
Sliding FS		N/A	
Overturning Stability and Bearing Capacity Assessments			
Total Resisting Moment (kip-ft)	**112.2**		
Total Driving Moment (kip-ft)	**103.3**		
Net Moment (kip-ft)	**8.8**		
Resultant Location from Toe (Normalized)		**0.08**	Within Base
Eccentricity from Center of Base (ft)	**2.48**		
Bearing Capacity Assessment			
Bearing Pressure at Toe (ksf)		N/A	
Bearing Pressure at Heel (ksf)		N/A	

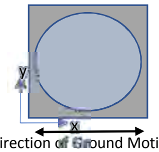
(1): Surrounding water above ground surface is negligible.

Failure Plane: **Bottom of Riser 1**





Project:	**Cumberland Fossil Plant - Seismic Stability Demonstration For Inlet Structure**		By:	C. Gabriel
Site:	**CUF**		Date:	7/6/2018
Load:	**2500 Year Earthquake**		Checked:	JTP/PRS
Direction:	**x-axis (Upstream/downstream)**		Check Date:	7/6/2018
Calculations:	**Global Stability Analysis**			
Failure Plane:	**Bottom of Riser 2**			



Direction of Ground Motion

Unit Weight of Water (pcf)	62.4	Allowable Bearing Capacity (ksf)	2.25	Horiz. Seis. Coef.: k_h	0.368	
Saturated Soil Unit Weight (pcf)	119	Backfill Elevation (ft)	372.00	Active LEP Coef.: k_a	0.270	Static
Soil Internal Friction Angle (deg)	32.5	Reservoir Elevation (ft)	374.20	Passive LEP Coef.: k_p	3.840	Earth
Soil-Wall Friction Angle (deg)	21	Water Depth (ft) = H_o	7.20	Active LEP Coef.: k_{ae}	0.620	Seismic
		Mass Density of Water ($lbm*s^2/ft^4$)	1.939	Passive LEP Coef.: k_{pe}	4.940	Earth

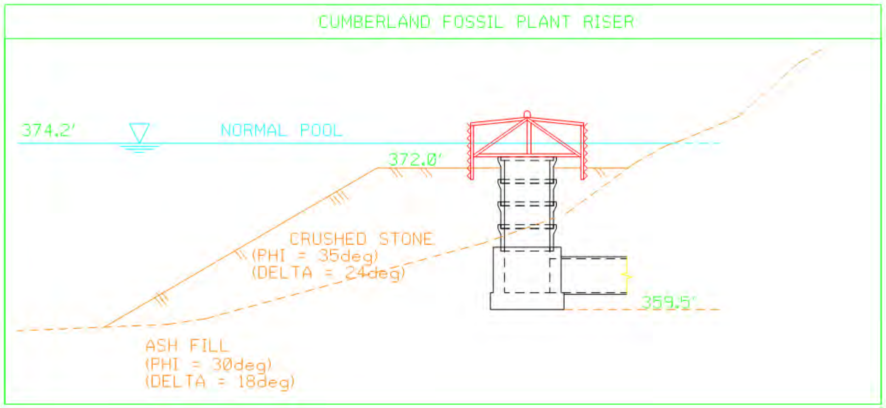
Node ID	Elevation	Arm, Z_o	Height, L	Width	Self Weight	Inertial Force		Hydrodynamic Added Mass				Static Soil Pressure				Dynamic Soil Pressure			
								Inside		Outside[1]		Active		Passive		Active		Passive	
	(ft)	(ft)	(ft)	(ft)	(kip)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)
	367.00	0.00	0.00	4.67	0.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-
(bottom of riser 2)	367.00	0.00	0.00	4.67	0.0	-	-	-	-	-	-	-	-	-	-	-	-	-	-
5 (mid riser 2)	367.57	0.57	1.29	4.67	1.4	0.5	0.3	1.6	0.9	0.0	0.0	0.4	0.2	5.7	3.2	0.2	0.1	-0.5	-0.3
6 (mid riser 3)	369.57	2.57	2.00	4.67	1.4	0.5	1.3	1.5	3.8	0.0	0.0	0.4	1.0	5.5	14.2	0.9	2.3	-2.8	-7.2
7 (mid riser 4)	371.57	4.57	3.41	4.67	1.4	0.5	2.3	4.3	19.7	0.0	0.0	0.1	0.5	1.5	6.8	1.4	6.3	-4.3	-19.8
8 (centroid of skimmer)	376.38	9.38	3.53	10.00	2.0	0.7	6.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Top Elevation	377.50				**6.07**	**2.23**	**10.73**	**7.37**	**24.46**	**0.00**	**0.00**	**0.89**	**-**	**12.69**	**-**	**2.43**	**-**	**-7.64**	**-**

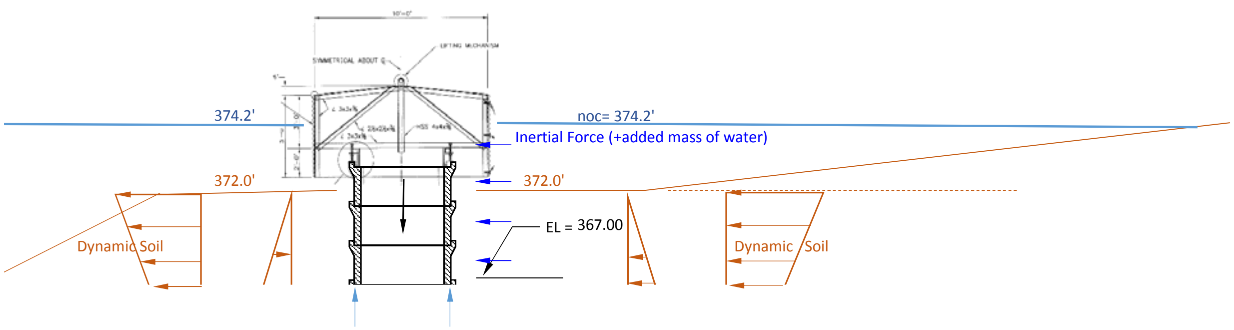
Load Summary			
	Load (kip)	Moment (kip-ft)	
Uplift	2.0	4.8	Drive
Soil Weight on Base	5.4	12.5	Resist
Water Weight on Base	8.5	19.8	Resist
Water Weight Inside	0.0	0.0	Resist
Self-Weight	6.1	14.2	Resist
Self-Weight Inertia	2.2	10.3	Drive
Static Active Soil (horiz)	0.9	1.5	Drive
Static Passive Soil (horiz)	12.7	8.4	Resist
Static Active Soil (vert)	0.3	1.5	Resist
Static Passive Soil (vert)	0.0	0.0	Drive
Seismic Active Soil (horiz)	2.4	8.1	Drive
Seismic Passive Soil (horiz)	-7.6	0.0	Resist
Seismic Active Soil (vert)	0.9	4.1	Resist
Seismic Passive Soil (vert)	0.0	0.0	Drive
Hydrodynamic Added Mass	7.4	24.5	Drive

Sliding Stability Assessment			
Total Vertical Load (kip)	**19.1**		
Total Horizontal Load (kip)	**7.9**		
Total Interface Resisting Load (kip)	**7.3**		
Req'd Sliding Safety Factor	1.10		
Sliding FS		N/A	
Overturning Stability and Bearing Capacity Assessments			
Total Resisting Moment (kip-ft)	**60.5**		
Total Driving Moment (kip-ft)	**49.2**		
Net Moment (kip-ft)	**11.3**		
Resultant Location from Toe (Normalized)		**0.13**	Within Base
Eccentricity from Center of Base (ft)	**1.74**		
Bearing Capacity Assessment			
Bearing Pressure at Toe (ksf)		N/A	
Bearing Pressure at Heel (ksf)		N/A	

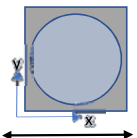
Failure Plane: **Bottom of Riser 2**

(1): Surrounding water above ground surface is negligible.


CUMBERLAND FOSSIL PLANT RISER
374.2'
NORMAL POOL
372.0'
CRUSHED STONE
(PHI = 35deg)
(DELTA = 24deg)
359.5'
ASH FILL
(PHI = 30deg)
(DELTA = 18deg)


374.2'
noc= 374.2'
Inertial Force (+added mass of water)
372.0'
372.0'
EL = 367.00
Dynamic Soil
Dynamic Soil

Project:	**Cumberland Fossil Plant - Seismic Stability Demonstration For Inlet Structure**
Site:	**CUF**
Load:	**2500 Year Earthquake**
Direction:	**y-axis (Upstream/downstream)**
Calculations:	**Global Stability Analysis**
Failure Plane:	**Bottom of Riser 3**



Direction of Ground Motion

By:	C. Gabriel
Date:	7/6/2018
Checked:	JTP/PRS
Check Date:	7/6/2018

Unit Weight of Water (pcf)	62.4	Allowable Bearing Capacity (ksf)	2.25	Horiz. Seis. Coef.: k_h	0.368	
Saturated Soil Unit Weight (pcf)	119	Backfill Elevation (ft)	372	Active LEP Coef.: k_a	0.270	Static
Soil Internal Friction Angle (deg)	32.5	Reservoir Elevation (ft)	374.2	Passive LEP Coef.: k_p	3.840	Earth
Soil-Wall Friction Angle (deg)	21	Water Depth (ft) = H_o	5.20	Active LEP Coef.: k_{ae}	0.620	Seismic
		Mass Density of Water ($lbm*s^2/ft^4$)	1.939	Passive LEP Coef.: k_{pe}	4.940	Earth

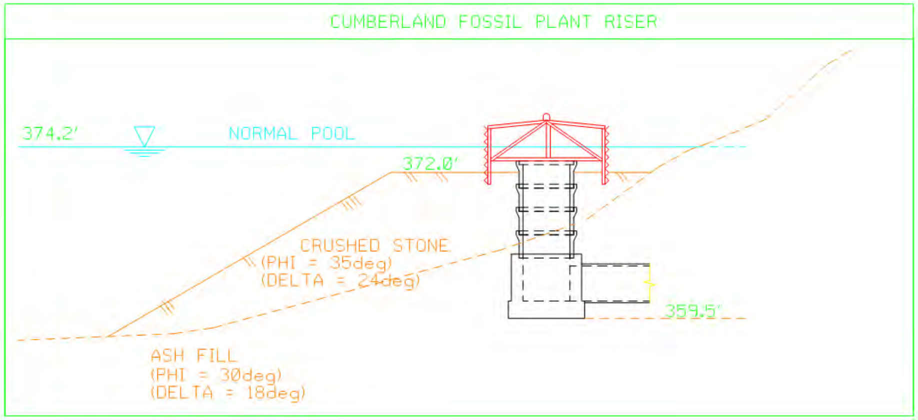
Node ID	Elevation	Arm, Z_o	Height, L	Width	Self Weight	Inertial Force		Hydrodynamic Added Mass Inside		Hydrodynamic Added Mass Outside[1]		Static Soil Pressure Active		Static Soil Pressure Passive		Dynamic Soil Pressure Active		Dynamic Soil Pressure Passive	
	(ft)	(ft)	(ft)	(ft)	(kip)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)
	369.00																		
(bottom of riser 3)	369.00	0.00	0.00	4.67	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00
6 (mid riser 3)	369.57	0.57	1.29	4.67	1.36	0.50	0.29	1.49	0.85	0.00	0.00	0.22	0.12	3.07	1.75	0.16	0.09	-0.5	-0.29
7 (mid riser 4)	371.57	2.57	3.41	4.67	1.36	0.50	1.29	4.32	11.10	0.00	0.00	0.10	0.27	1.49	3.84	0.71	1.84	-2.2	-5.77
8 (centroid of skimmer)	376.38	7.38	3.53	10.00	1.99	0.73	5.40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00
Top Elevation	377.50				**4.71**	**1.73**	**6.98**	**5.81**	**11.95**	**0.00**	**0.00**	**0.32**	**0.39**	**4.57**	**5.59**	**0.88**	**1.93**	**-2.75**	**-6.06**

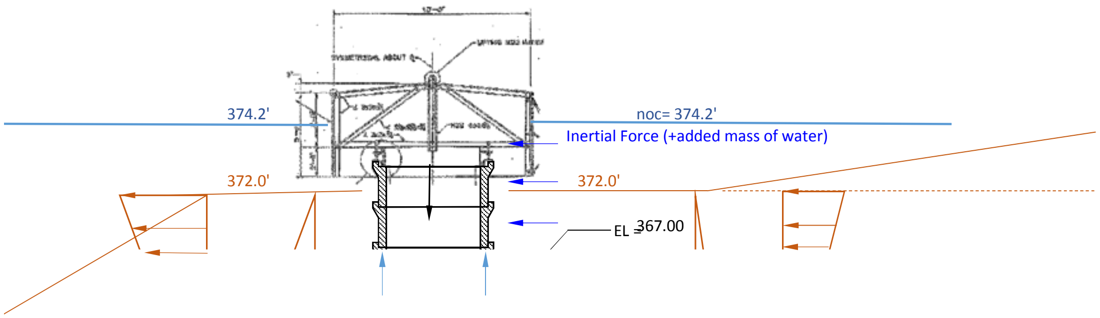
Load Summary	Load (kip)	Moment (kip-ft)	
Uplift	1.48	3.46	Drive
Soil Weight on Base	3.20	7.47	Resist
Water Weight on Base	6.13	14.31	Resist
Water Weight Inside	0.00	0.00	Resist
Self-Weight	4.71	11.00	Resist
Self-Weight Inertia	1.73	7.99	Drive
Static Active Soil (horiz)	0.32	0.32	Drive
Static Passive Soil (horiz)	4.57	1.82	Resist
Static Active Soil (vert)	0.00	0.00	Resist
Static Passive Soil (vert)	0.00	0.00	Drive
Seismic Active Soil (horiz)	0.88	1.76	Drive
Seismic Passive Soil (horiz)	-2.75	0.00	Resist
Seismic Active Soil (vert)	0.00	0.00	Resist
Seismic Passive Soil (vert)	0.00	0.00	Drive
Hydrodynamic Added Mass	5.81	11.95	Drive

Sliding Stability Assessment			
Total Vertical Load (kip)	**12.56**		
Total Horizontal Load (kip)	**6.92**		
Total Interface Resisting Load (kip)	**4.82**		
Req'd Sliding Safety Factor	1.10		
Sliding FS		N/A	
Percent of Passive Resistance Needed		100%	
Overturning Stability and Bearing Capacity Assessments			
Total Resisting Moment (kip-ft)	**34.60**		
Total Driving Moment (kip-ft)	**25.48**		
Net Moment (kip-ft)	**9.12**		
Resultant Location from Toe (Normalized)		**0.16**	Within Base
Eccentricity from Center of Base (ft)	**1.61**		
Bearing Capacity Assessment			
Bearing Pressure at Toe (ksf)		N/A	
Bearing Pressure at Heel (ksf)		N/A	

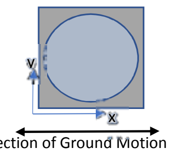
Failure Plane: **Bottom of Riser 3**

(1): Surrounding water above ground surface is negligible.


CUMBERLAND FOSSIL PLANT RISER
374.2'
NORMAL POOL
372.0'
CRUSHED STONE
(PHI = 35deg)
(DELTA = 24deg)
359.5'
ASH FILL
(PHI = 30deg)
(DELTA = 18deg)


374.2'
noc= 374.2'
Inertial Force (+added mass of water)
372.0'
372.0'
EL = 367.00

Project:	**Cumberland Fossil Plant - Seismic Stability Demonstration For Inlet Structure**	By:	C. Gabriel
Site:	**CUF**	Date:	7/6/2018
Load:	**2500 Year Earthquake**	Checked:	JTP/PRS
Direction:	**y-axis (Upstream/downstream)**	Check Date:	7/6/2018
Calculations:	**Global Stability Analysis**		
Failure Plane:	**Bottom of Riser 4**		



Direction of Ground Motion

Unit Weight of Water (pcf)	62.4	Allowable Bearing Capacity (ksf)	2.25	Horiz. Seis. Coef.: k_h	0.368	
Saturated Soil Unit Weight (pcf)	119	Backfill Elevation (ft)	372	Active LEP Coef.: k_a	0.270	Static
Soil Internal Friction Angle (deg)	32.5	Reservoir Elevation (ft)	374.2	Passive LEP Coef.: k_p	3.840	Earth
Soil-Wall Friction Angle (deg)	21	Water Depth (ft) = H_o	3.20	Active LEP Coef.: k_{ae}	0.620	Seismic
		Mass Density of Water ($lbm*s^2/ft^4$)	1.939	Passive LEP Coef.: k_{pe}	4.940	Earth

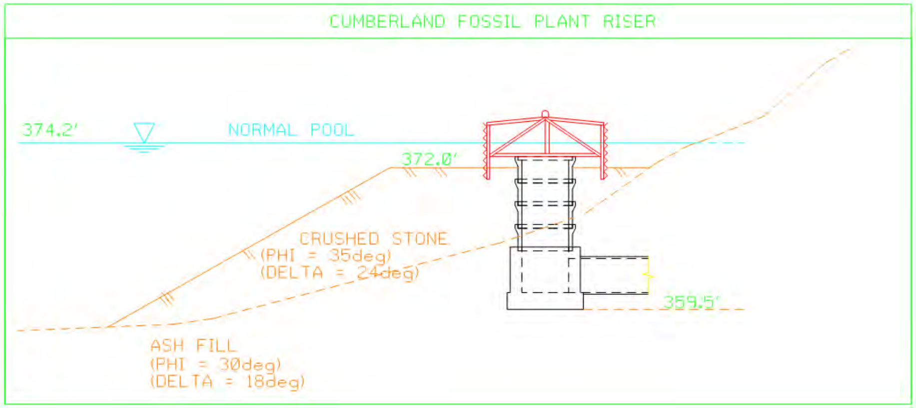
Node ID	Elevation	Arm, Z_o	Height, L	Width	Self Weight	Inertial Force		Hydrodynamic Added Mass				Static Soil Pressure				Dynamic Soil Pressure			
								Inside		Outside[1]		Active		Passive		Active		Passive	
	(ft)	(ft)	(ft)	(ft)	(kip)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)	(kip)	(kip-ft)
	371.00																		
(bottom of riser 4)	371.00	0.00	0.00	4.67	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00
7 (mid riser 4)	371.57	0.57	2.69	4.67	1.36	0.50	0.29	4.32	2.46	0.00	0.00	0.04	0.02	0.51	0.29	0.10	0.06	-0.3	-0.17
8 (centroid of skimmer)	376.38	5.38	3.53	10.00	1.99	0.73	3.94	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00
Top Elevation	377.50				**3.35**	**1.23**	**4.23**	**4.32**	**2.46**	**0.00**	**0.00**	**0.04**	**0.02**	**0.51**	**0.29**	**0.10**	**0.06**	**-0.31**	**-0.17**

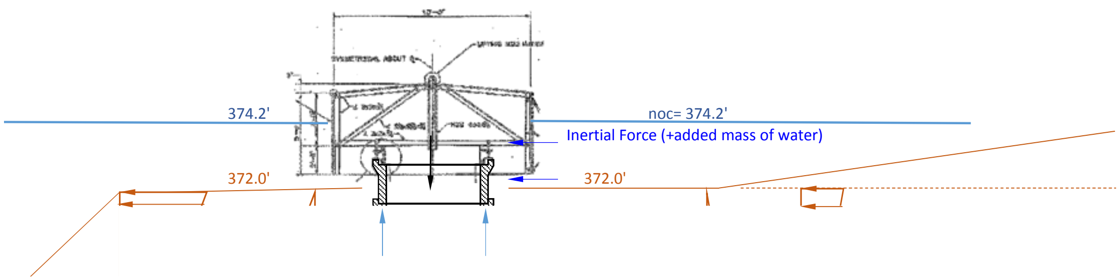
Load Summary	Load (kip)	Moment (kip-ft)	
Uplift	0.91	2.13	Drive
Soil Weight on Base	1.10	2.57	Resist
Water Weight on Base	3.77	8.80	Resist
Water Weight Inside	0.00	0.00	Resist
Self-Weight	3.35	7.82	Resist
Self-Weight Inertia	1.23	5.68	Drive
Static Active Soil (horiz)	0.04	0.01	Drive
Static Passive Soil (horiz)	0.51	0.07	Resist
Static Active Soil (vert)	0.00	0.00	Resist
Static Passive Soil (vert)	0.00	0.00	Drive
Seismic Active Soil (horiz)	0.10	0.07	Drive
Seismic Passive Soil (horiz)	-0.31	0.00	Resist
Seismic Active Soil (vert)	0.00	0.00	Resist
Seismic Passive Soil (vert)	0.00	0.00	Drive
Hydrodynamic Added Mass	4.32	2.46	Drive

Sliding Stability Assessment			
Total Vertical Load (kip)	**7.31**		
Total Horizontal Load (kip)	**5.48**		
Total Interface Resisting Load (kip)	**2.81**		
Req'd Sliding Safety Factor	1.10		
Sliding FS W/ Partial Passive Soil Resistance		N/A	
Percent of Passive Resistance Needed		100%	
Overturning Stability and Bearing Capacity Assessments			
Total Resisting Moment (kip-ft)	**19.26**		
Total Driving Moment (kip-ft)	**10.35**		
Net Moment (kip-ft)	**8.91**		
Resultant Location from Toe (Normalized)		**0.26**	Within Base
Eccentricity from Center of Base (ft)	**1.12**		
Bearing Capacity Assessment			
Bearing Pressure at Toe (ksf)		N/A	
Bearing Pressure at Heel (ksf)		N/A	

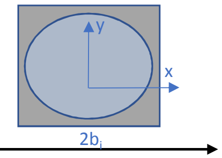
Failure Plane: **Bottom of Riser 4**

(1): Surrounding water above ground surface is negligible.


CUMBERLAND FOSSIL PLANT RISER
374.2'
NORMAL POOL
372.0'
CRUSHED STONE
(PHI = 35deg)
(DELTA = 24deg)
359.5'
ASH FILL
(PHI = 30deg)
(DELTA = 18deg)


374.2'
noc= 374.2'
Inertial Force (+added mass of water)
372.0'
372.0'

Project:	**Cumberland Fossil Plant - Seismic Stability Demonstration of Inlet Structure**	By:	C. Gabriel
Site:	**CUF**	Date:	2/7/2018
Load:	**Hydrodynamic Added Mass**		
Location:	**Inside Water**		
Direction:	**x-axis**		



Direction of Ground Motion

Base Thickness (ft)	1.5	Reservoir Elevation (ft)	374.2
		Water Depth (ft) = H_i	13.20
		Mass Density of Water ($lbm*s^2/ft^4$)	1.939

Node	Elevation (ft)	Z_i (ft)	l (ft)	$2a_i$ (ft)	$2b_i$ (ft)	$A_i(z)$ (ft^2)	$a_i/b_i = \tilde{a}_i/\tilde{b}_i$ (Eq. 8.10b)	$\tilde{a}_i/H_i$ (Eq. 8.10a)	$\tilde{r}_i/H_i$ (Figure 8.18)	Z_i/H_i	$m^i_a(z)/m^i_\infty$ (Table 8.5)	m^i_∞ ($lbm\text{-}s^2/ft^2$)	$m^i_a(z)$ ($lbm*s^2/ft$)	$w^i_a(z)$ (kip)
	359.500													
1 (bottom of base)	359.500	0.00	0.00	0.0	0.0	0.00	-	-	-	0.0000	1.0000	0.00	0.00	0.00
2 (mid base)	360.250	0.75	0.63	0.0	0.0	0.00	-	-	-	0.0568	1.0000	0.00	0.00	0.00
3 (mid box)	363.000	3.50	2.66	4.0	4.0	16.00	1.000	0.1710	0.1710	0.2652	0.9996	31.03	82.51	2.65
4 (mid riser 1)	365.570	6.07	2.29	4.0	4.0	12.57	1.000	0.1515	0.1515	0.4598	0.9984	24.37	55.60	1.79
5 (mid riser 2)	367.570	8.07	2.00	4.0	4.0	12.57	1.000	0.1515	0.1515	0.6114	0.9924	24.37	48.37	1.56
6 (mid riser 3)	369.570	10.07	2.00	4.0	4.0	12.57	1.000	0.1515	0.1515	0.7629	0.9528	24.37	46.44	1.49
7 (mid riser 4)	371.570	12.07	3.41	7.1	7.1	39.06	1.000	0.2671	0.2671	0.9144	0.5207	75.75	134.29	4.32
8 (mid skim)	376.380	16.88	3.53	10.0	10.0	78.54	1.000	0.3788	0.3788	-	0.0000	152.32	0.00	0.00
Top Elevation	377.500													

Calculations were conducted using methodology outline in Earthquake Analysis and Response of Intake-Outlet Towers, Section 8.2.2

Z_o, Z_i	: Outside/inside height of section from exterior/interior base of the structure
l	: Tributary length of section in vertical direction
$A_o(z)$, $A_i(z)$	: Area enclosed by cross section of outside/inside surface at elevation z
$2a_o$, $2a_i$	: Cross sectional dimension of the outside/inside surface of a non-circular tower in the perpendicular direction to ground motion
$2a\sim_o$, $2a\sim_i$	: Cross sectional dimension of the outside/inside surface of an equivalent elliptical tower for a non-circular tower in the perpendicular direction to ground motion
$2b_o$, $2b_i$	: Cross sectional dimension of the outside/inside surface of a non-circular tower in the direction parallel to ground motion
$2b\sim_o$,$2b\sim_i$	: Cross sectional dimension of the outside/inside surface of an equivalent elliptical tower for a non-circular tower in the direction to ground motion
H_o , H_i	: Outside/inside water depth
$r\sim_o$, $r\sim_i$	: Radius of an equivalent cylindrical tower for added mass computations for outside/inside water
$m^o_a(z)$, $m^i_a(z)$	: Added hydrodynamic mass for outside/inside water
$m^o_\infty(z)$, $m^i_\infty(z)$	: Added hydrodynamic mass for a infinitely long tower for outside/inside water

Project: Cumberland Fossil Plant - Seismic Stability Demonstration of Inlet Structure

Site: CUF

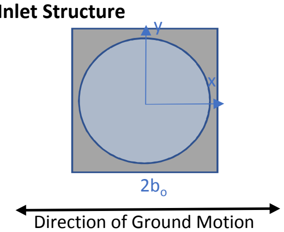
Load: Hydrodynamic Added Mass

Location: Surrounding Water

Direction: x-axis

By: C. Gabriel

Date: 2/7/2018



Reservoir Elevation (ft)	374.2
Water Depth (ft) = H_o	14.70
Mass Density of Water ($lbm*s^2/ft^4$)	1.939

Node	Elevation (ft)	Z_o (ft)	l (ft)	$2a_o$ (ft)	$2b_o$ (ft)	$A_o(z)$ (ft^2)	$a_o/b_o = \tilde{a}_o/\tilde{b}_o$ (Eq. 8.5b)	$\tilde{a}_o/H_o$ (Eq. 8.5a)	$\tilde{r}_o/H_o$ (Table 8.3)	Z_o/H_o	$m^o_a(z)/m^o_\infty$ (Table 8.4)	$m^o_\infty/p_w A_o$ (Table 8.1)	m^o_∞ ($lbm\text{-}s^2/ft^2$)	$m^o_a(z)$ ($lbm\text{-}s^2/ft$)	$w^o_a(z)$ (kip)
	359.500														
1 (bottom of base)	359.500	0.00	0.00	6.50	6.50	42.25	1.000	0.2495	0.2495	0.0000	0.8983	1.19	97.18	0.00	0.00
2 (mid base)	360.250	0.75	2.13	6.50	6.50	42.25	1.000	0.2495	0.2495	0.0510	0.8981	1.19	97.18	185.46	5.97
3 (mid box)	363.000	3.50	2.66	6.00	6.00	36.00	1.000	0.2303	0.2303	0.2381	0.8865	1.19	82.81	195.28	6.28
4 (mid riser 1)	365.570	6.07	2.29	5.39	5.39	22.78	1.000	0.1832	0.1832	0.4129	0.8565	1.00	44.18	86.46	2.78
5 (mid riser 2)	367.570	8.07	2.00	4.67	4.67	17.13	1.000	0.1588	0.1588	0.5490	0.8146	1.00	33.22	54.12	1.74
6 (mid riser 3)	369.570	10.07	2.00	4.67	4.67	17.13	1.000	0.1588	0.1588	0.6850	0.7408	1.00	33.22	49.22	1.58
7 (mid riser 4)	371.570	12.07	3.41	7.38	7.38	42.79	1.000	0.2511	0.2511	0.8211	0.6030	1.00	82.99	170.39	5.48
8 (mid skim)	376.380	16.88	3.53	10.00	10.00	78.54	1.000	0.3401	0.3401	-	0.0000	1.00	152.32	0.00	0.00
Top Elevation	377.500														

Calculations were conducted using methodology outline in Earthquake Analysis and Response of Intake-Outlet Towers, Section 8.2.2

Z_o, Z_i	: Outside/inside height of section from exterior/interior base of the structure
l	: Tributary length of section in vertical direction
$A_o(z)$, $A_i(z)$	: Area enclosed by cross section of outside/inside surface at elevation z
a_o, a_i	: Cross sectional dimension of the outside/inside surface of a non-circular tower in the perpendicular direction to ground motion
$a\sim_o$, $a\sim_i$	: Cross sectional dimension of the outside/inside surface of an equivalent elliptical tower for a non-circular tower in the perpendicular direction to ground motion
b_o , b_i	: Cross sectional dimension of the outside/inside surface of a non-circular tower in the direction to ground motion
$b\sim_o$,$b\sim_i$	: Cross sectional dimension of the outside/inside surface of an equivalent elliptical tower for a non-circular tower in the direction parallel to ground motion
H_o , H_i	: Outside/inside water depth
$r\sim_o$, $r\sim_i$	: Radius of an equivalent cylindrical tower for added mass computations for outside/inside water
$m^o_a(z)$, $m^i_a(z)$	: Added hydrodynamic mass for outside/inside water
$m^o_\infty(z)$, $m^i_\infty(z)$	: Added hydrodynamic mass for a infinitely long tower for outside/inside water

Project: **Cumberland Fossil Plant - Seismic Stability Demonstration of Inlet Structure**
Site: **CUF**
Load: **Hydrodynamic Added Mass**
Location: **Inside Water**

Data From Figure 8.18					
	a/H		r/H		slope
a/b	x1	x2	y1	y2	m
0.200	0	0.205	0	1	4.88
0.250	0	0.25	0	1	4.00
0.333	0	0.325	0	1	3.08
0.500	0	0.498	0	1	2.01
0.667	0	0.67	0	1	1.49
1.000	0	0.7	0	0.7	1.00
1.500	0	0.7	0	0.47	0.67
2.000	0	0.7	0	0.35	0.50
3.000	0	0.7	0	0.225	0.32
4.000	0	0.7	0	0.175	0.25
5.000	0	0.7	0	0.145	0.21

Table 8.5											
z/H_i	r_i/H_i										
	0.05	0.1	0.15	0.2	0.25	0.3	0.4	0.5	0.6	0.8	1
0	1	1	1	1	0.999	0.996	0.983	0.958	0.922	0.834	0.74
0.04	1	1	1	1	0.999	0.996	0.983	0.957	0.922	0.833	0.74
0.08	1	1	1	1	0.999	0.996	0.982	0.956	0.92	0.831	0.737
0.12	1	1	1	1	0.998	0.995	0.981	0.954	0.917	0.827	0.734
0.16	1	1	1	1	0.998	0.994	0.978	0.95	0.913	0.822	0.729
0.2	1	1	1	0.999	0.998	0.993	0.976	0.946	0.907	0.816	0.722
0.24	1	1	1	0.999	0.997	0.992	0.972	0.94	0.9	0.808	0.714
0.28	1	1	1	0.999	0.996	0.99	0.967	0.933	0.892	0.798	0.704
0.32	1	1	1	0.998	0.994	0.987	0.961	0.925	0.882	0.786	0.693
0.36	1	1	1	0.998	0.992	0.983	0.954	0.915	0.87	0.773	0.679
0.4	1	1	0.999	0.997	0.99	0.979	0.946	0.903	0.856	0.757	0.664
0.44	1	1	0.999	0.995	0.986	0.973	0.935	0.889	0.84	0.74	0.647
0.48	1	1	0.998	0.993	0.982	0.965	0.923	0.873	0.821	0.72	0.628
0.52	1	1	0.998	0.99	0.976	0.956	0.907	0.854	0.8	0.697	0.607
0.56	1	1	0.996	0.985	0.967	0.944	0.889	0.831	0.775	0.672	0.583
0.6	1	0.999	0.994	0.979	0.956	0.928	0.866	0.805	0.747	0.644	0.557
0.62	1	0.999	0.992	0.975	0.949	0.919	0.853	0.79	0.731	0.628	0.542
0.64	1	0.999	0.99	0.969	0.941	0.908	0.839	0.774	0.715	0.612	0.527
0.66	1	0.998	0.987	0.963	0.931	0.896	0.824	0.757	0.697	0.595	0.511
0.68	1	0.998	0.983	0.956	0.921	0.882	0.807	0.738	0.678	0.576	0.495
0.7	1	0.997	0.979	0.947	0.908	0.867	0.788	0.718	0.657	0.557	0.477
0.72	1	0.995	0.973	0.936	0.893	0.849	0.767	0.696	0.635	0.536	0.458
0.74	1	0.993	0.965	0.923	0.876	0.83	0.744	0.673	0.612	0.514	0.439
0.76	1	0.99	0.956	0.908	0.857	0.807	0.719	0.647	0.587	0.491	0.418
0.78	1	0.985	0.944	0.889	0.834	0.782	0.692	0.619	0.56	0.467	0.396
0.8	0.999	0.979	0.928	0.867	0.807	0.753	0.661	0.589	0.53	0.44	0.373
0.82	0.999	0.969	0.908	0.84	0.776	0.72	0.627	0.556	0.499	0.412	0.349
0.84	0.997	0.956	0.882	0.807	0.74	0.682	0.589	0.52	0.465	0.382	0.322
0.86	0.995	0.936	0.849	0.767	0.698	0.639	0.548	0.48	0.428	0.35	0.295
0.88	0.99	0.908	0.807	0.719	0.648	0.59	0.501	0.436	0.387	0.316	0.265
0.9	0.979	0.867	0.752	0.661	0.589	0.533	0.448	0.388	0.343	0.278	0.233
0.92	0.956	0.807	0.682	0.589	0.52	0.466	0.389	0.334	0.294	0.237	0.198
0.94	0.907	0.719	0.589	0.501	0.437	0.389	0.32	0.274	0.24	0.192	0.16
0.96	0.806	0.589	0.466	0.388	0.335	0.295	0.24	0.204	0.177	0.141	0.117
0.98	0.588	0.388	0.295	0.24	0.204	0.178	0.143	0.12	0.104	0.082	0.067
1	0	0	0	0	0	0	0	0	0	0	0

Project: **Cumberland Fossil Plant - Seismic Stability Demonstration of Inlet Structure**
Site: **CUF**
Load: **Hydrodynamic Added Mass**
Location: **Surrounding Water**

Table 8.3											
a_0/H_0	a_0/b_0										
	0.2	0.25	0.333333	0.5	0.666667	1.0000	1.5	2	3	4	5
0	0	0	0	0	0	0.0000	0	0	0	0	0
0.05	0.146	0.117	0.094	0.071	0.06	0.0500	0.043	0.04	0.037	0.036	0.035
0.1	0.279	0.228	0.185	0.141	0.12	0.1000	0.087	0.08	0.075	0.072	0.07
0.15	0.408	0.337	0.274	0.211	0.18	0.1500	0.131	0.121	0.112	0.108	0.105
0.2	0.536	0.445	0.363	0.28	0.24	0.2000	0.175	0.162	0.15	0.144	0.141
0.25	0.661	0.551	0.45	0.348	0.299	0.2500	0.219	0.203	0.188	0.181	0.177
0.3	0.785	0.656	0.536	0.416	0.358	0.3000	0.263	0.245	0.227	0.218	0.213
0.4	1.026	0.861	0.707	0.551	0.475	0.4000	0.352	0.328	0.305	0.294	0.287
0.5	--	1.062	0.875	0.685	0.591	0.5000	0.441	0.412	0.385	0.371	0.363
0.6	--	--	1.04	0.817	0.708	0.6000	0.531	0.497	0.465	0.449	0.44
0.7	--	--	--	0.949	0.823	0.7000	0.621	0.583	0.546	0.528	0.517

Table 8.4											
z/H_0	r_0/H_0										
	0.05	0.1	0.15	0.2	0.25	0.3	0.4	0.5	0.6	0.8	1
0	0.995	0.981	0.958	0.93	0.898	0.865	0.799	0.737	0.681	0.587	0.513
0.04	0.995	0.981	0.958	0.93	0.898	0.865	0.798	0.736	0.68	0.586	0.512
0.08	0.995	0.981	0.958	0.929	0.897	0.863	0.797	0.735	0.679	0.585	0.511
0.12	0.995	0.98	0.957	0.928	0.895	0.861	0.795	0.732	0.676	0.582	0.509
0.16	0.995	0.979	0.955	0.926	0.893	0.859	0.791	0.729	0.673	0.579	0.506
0.2	0.994	0.978	0.954	0.923	0.89	0.855	0.787	0.724	0.668	0.575	0.502
0.24	0.994	0.977	0.951	0.92	0.886	0.85	0.782	0.719	0.663	0.569	0.497
0.28	0.994	0.976	0.949	0.916	0.881	0.845	0.775	0.712	0.656	0.563	0.491
0.32	0.993	0.974	0.945	0.911	0.875	0.838	0.768	0.704	0.648	0.555	0.484
0.36	0.993	0.972	0.941	0.905	0.868	0.83	0.759	0.695	0.639	0.546	0.475
0.4	0.992	0.969	0.936	0.898	0.859	0.821	0.748	0.684	0.628	0.536	0.466
0.44	0.991	0.965	0.93	0.89	0.85	0.81	0.736	0.672	0.616	0.525	0.456
0.48	0.99	0.961	0.923	0.881	0.838	0.798	0.723	0.658	0.602	0.512	0.444
0.52	0.988	0.956	0.914	0.869	0.825	0.783	0.707	0.642	0.586	0.497	0.43
0.56	0.986	0.95	0.904	0.856	0.81	0.766	0.689	0.624	0.568	0.481	0.415
0.6	0.983	0.942	0.891	0.84	0.791	0.747	0.668	0.603	0.548	0.462	0.399
0.62	0.982	0.937	0.883	0.83	0.781	0.736	0.657	0.591	0.537	0.452	0.389
0.64	0.98	0.931	0.875	0.82	0.77	0.724	0.644	0.579	0.525	0.441	0.38
0.66	0.977	0.925	0.866	0.809	0.757	0.711	0.631	0.566	0.513	0.43	0.37
0.68	0.975	0.918	0.856	0.797	0.744	0.697	0.617	0.552	0.499	0.418	0.359
0.7	0.972	0.91	0.844	0.784	0.729	0.682	0.602	0.537	0.485	0.405	0.347
0.72	0.968	0.901	0.831	0.769	0.713	0.665	0.585	0.521	0.47	0.392	0.335
0.74	0.963	0.89	0.817	0.752	0.696	0.647	0.567	0.504	0.454	0.377	0.323
0.76	0.958	0.878	0.801	0.734	0.676	0.627	0.548	0.486	0.436	0.362	0.309
0.78	0.951	0.863	0.782	0.713	0.655	0.606	0.527	0.466	0.417	0.345	0.294
0.8	0.943	0.846	0.761	0.69	0.631	0.582	0.504	0.444	0.397	0.328	0.279
0.82	0.932	0.826	0.736	0.663	0.604	0.555	0.478	0.421	0.375	0.309	0.262
0.84	0.919	0.802	0.708	0.634	0.574	0.526	0.451	0.395	0.351	0.288	0.244
0.86	0.901	0.773	0.674	0.599	0.541	0.493	0.42	0.367	0.326	0.266	0.225
0.88	0.878	0.737	0.635	0.56	0.502	0.456	0.386	0.336	0.297	0.242	0.204
0.9	0.846	0.693	0.588	0.514	0.457	0.413	0.348	0.301	0.266	0.215	0.181
0.92	0.802	0.636	0.531	0.459	0.405	0.364	0.304	0.262	0.23	0.186	0.156
0.94	0.736	0.561	0.459	0.392	0.343	0.306	0.254	0.217	0.19	0.153	0.128
0.96	0.634	0.459	0.366	0.308	0.267	0.236	0.193	0.164	0.143	0.114	0.095
0.98	0.455	0.306	0.236	0.194	0.166	0.146	0.118	0.099	0.086	0.068	0.056
1	0	0	0	0	0	0	0	0	0	0	0

Miscellaneous Calculations to Support Stability Analysis

Material Parameters

- Unit Weight of Concrete: $\gamma_c := 150\text{pcf}$

- Unit Weight of Water: $\gamma_w := 62.4\text{pcf}$

- Soil moist unit weight: $\gamma_{soil} := 119\text{pcf}$

- Soil internal friction angle: $\phi_{soil} := 32.5\text{deg}$

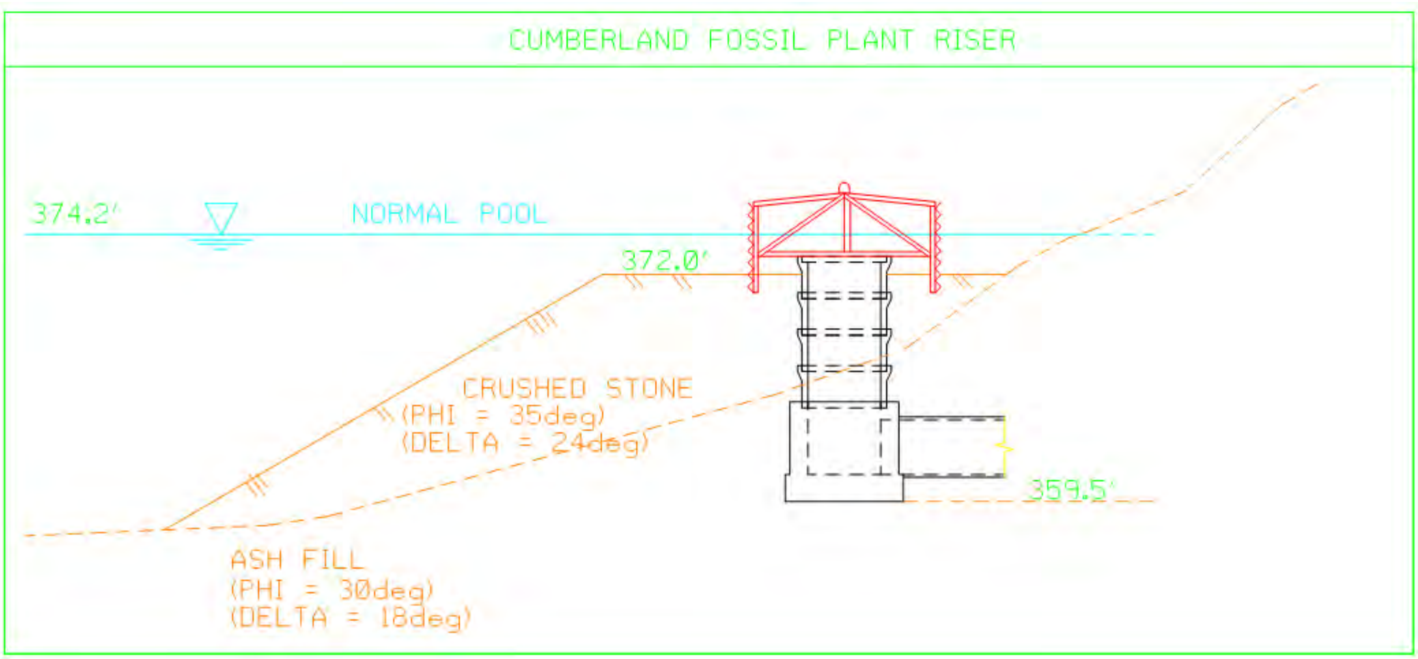
- Soil/backfill elevation: $El_{soil} := 372\text{ft}$

- Reservoir elevation: $El_{res} := 374.2\text{ft}$

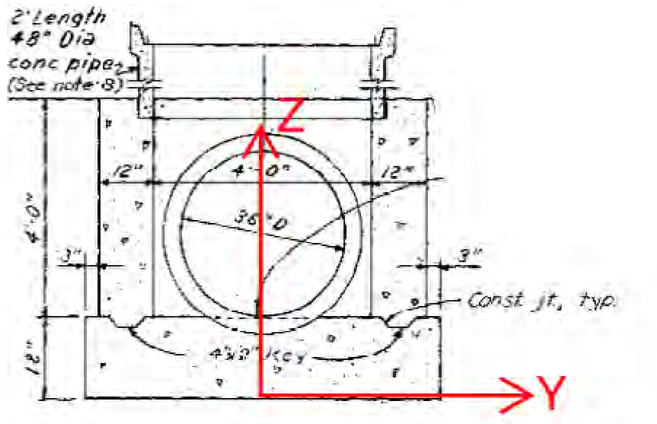
- Base elevation: $El_{base} := 359.5\text{ft}$

- Top of base box elevation: $El_{box} := 365.0\text{ft}$

- Soil/Backfill height: $h_{soil} := El_{soil} - El_{base} = 12.50\,\text{ft}$

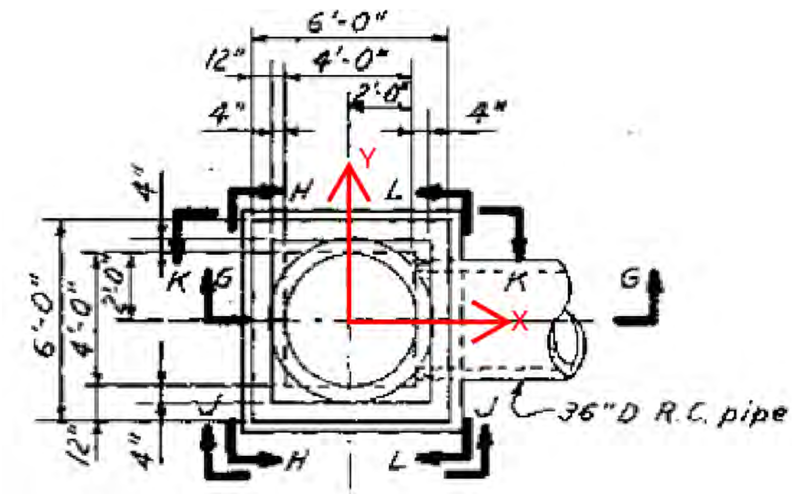


Foundation / Base



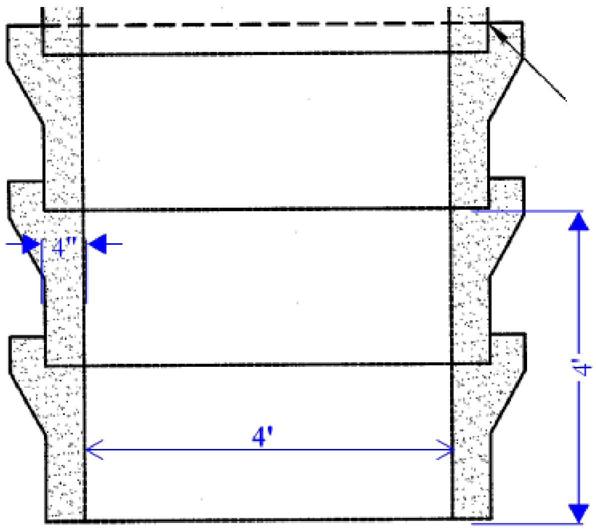
- Base Length: $l_{bx} := 6.5ft$
- Base Width: $l_{by} := 6.5ft$
- Base Thickness: $t_b := 1.5ft$
- Weight of Base: $\boxed{w_{base} := \gamma_c \cdot (l_{bx} \cdot l_{by} \cdot t_b) = 9.51 \cdot kip}$

Box / Pedestal



- Wall Thickness: $t_{wall} := 1.0ft$
- External Width: $l_e := 6.0ft$
- Internal Width: $l_i := l_e - 2t_{wall} = 4.00\,ft$
- Outlet Pipe Diameter $d_{outlet} := 3.0ft$
- Height of Box: $h_{box} := 4ft$
- Weight of Box: $\boxed{w_{box} := \gamma_c \cdot t_{wall} \cdot \left[\left[2 \cdot (l_e + l_i) h_{box} \right] - \left(\pi \cdot \frac{d_{outlet}^{\,2}}{4} \right) \right] = 10.94 \cdot kip}$

Riser Pipes



- riser pipe wall thickness: $t_p := 4\text{in}$

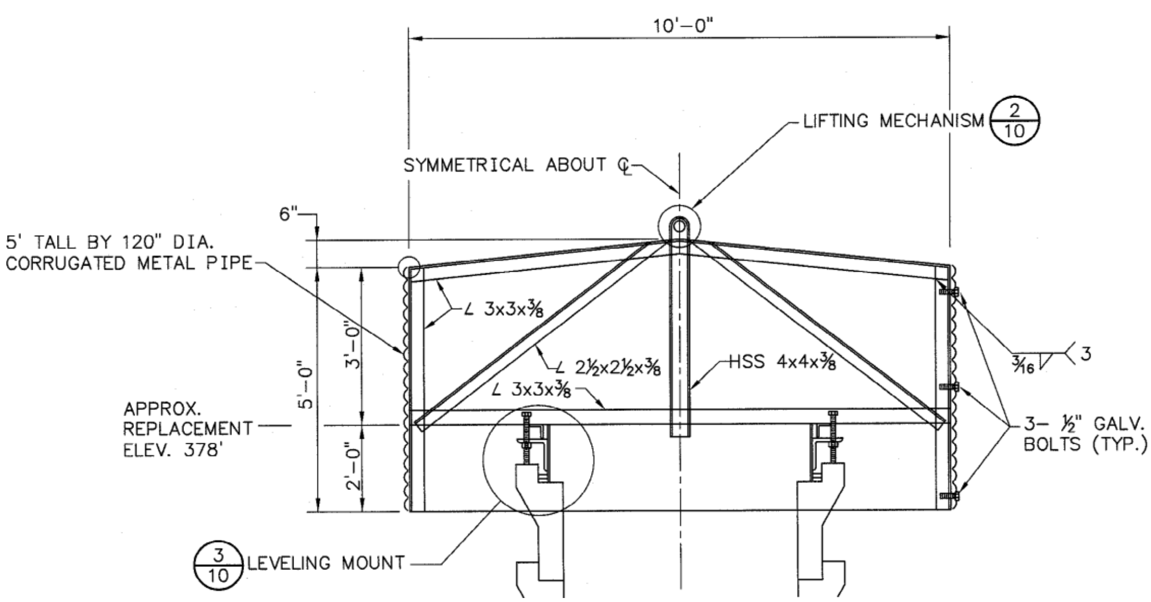
- riser inside diameter: $d_{in} := 4\text{ft}$

- height of single riser: $h_{riser} := 2\text{ft}$

- riser outside diameter: $d_{out} := d_{in} + 2t_p = 4.67\,\text{ft}$

- weight of single riser: $\boxed{w_{riser} := \left[\gamma_c \cdot h_{riser} \cdot \pi \cdot \frac{\left(d_{out}^{\ 2} - d_{in}^{\ 2}\right)}{4}\right] = 1.36 \cdot \text{kip}}$

Skimmer



- L 3x3x3/8 : $w_{3x3x3\_8} := 7.2\frac{lbf}{ft}\cdot 62ft = 0.4\cdot kip$

- L 2.5x2.5x3/8 : $w_{2.5x2.5x3\_8} := 5.3\frac{lbf}{ft}\cdot 24ft = 0.1\cdot kip$

- HSS : $w_{hss} := 17.27\frac{lbf}{ft}\cdot 6.0ft = 0.1\cdot kip$

- corrugated metal pipe, 120" inside diameter: $w_{corug\_pipe} := 183\frac{lbf}{ft}\cdot 5ft = 0.92\cdot kip$

- skimmer weight:
  +25% for miscelleanous steel
  $w_{skimmer} := \left[w_{corug\_pipe} + w_{hss} + \left(w_{2.5x2.5x3\_8} + w_{3x3x3\_8}\right)\right]\cdot 1.25 = 1.99\cdot kip$

- total dead load of structure: $W_{dead} := w_{skimmer} + 4w_{riser} + w_{box} + w_{base} = 27.88\cdot kip$

Inertial centroid of the Intake Structure:

$$y_{centroid} := \frac{\left[w_{skimmer}\cdot 16.88ft + w_{riser}\cdot(6.07ft + 8.07ft + 10.07ft + 12.07ft) + w_{box}\cdot 3.5ft + w_{base}\cdot 0.75ft\right]}{W_{dead}}$$

$y_{centroid} = 4.61\,ft$

Soil Weight on Structure

- soil on square base: $w_{s1} := \left(\gamma_{soil} - \gamma_w\right)\cdot\left(l_{bx}^{\,2} - l_e^{\,2}\right)\cdot\left(h_{soil} - t_b\right) = 3.89\cdot kip$

- soil area above box around risers: $A_1 := 4.22ft^2$ Measured in Auto CAD

- average thickness of A_1: $t_1 := \frac{A_1}{El_{soil} - El_{box}} = 0.6\,ft$

- soil on box, around risers: $w_{s2} := \left(\gamma_{soil} - \gamma_w\right)\left[\left(l_e\right)^2 - \frac{\pi}{4}\cdot d_{out}^{\,2}\right]\cdot\left(El_{soil} - El_{box}\right) = 7.5\cdot kip$

- soil weight on structure: $\boxed{w_{soil} := w_{s1} + w_{s2} = 11.38\cdot kip}$

Water Weight on Structure

- water on square base: $w_{w1} := \gamma_w\left(l_{bx}^{\,2} - l_e^{\,2}\right)\cdot\left(El_{res} - El_{base} - t_b\right) = 5.1\cdot kip$

- water area above box around risers: $A_2 := 6.49ft^2$ Measured in Auto CAD

- average thickness of A_2: $t_2 := \frac{A_2}{El_{res} - El_{box}} = 0.71\,ft$

- water on box, around risers: $w_{w2} := \gamma_w\left[\left(l_e\right)^2 - \frac{\pi}{4}\cdot d_{out}^{\,2}\right]\cdot\left(El_{res} - El_{box}\right) = 10.8\cdot kip$

- surrounding water weight on structure: $\boxed{w_{w\_out} := w_{w1} + w_{w2} = 16\cdot kip}$

- Water weight inside tower: $w_{w.in} := \gamma_w \cdot \left[l_i^2 \cdot h_{box} + \left(\pi \frac{d_{in}^2}{4} \right) \cdot \left(El_{res} - El_{base} - t_b - h_{box} \right) \right] = 11.21\,kip$

Soil Load Parameters: Mononobe-Okabe

- active wall friction angle: $\delta := 21deg$

- passive wall friction angle: $\delta_p := 21deg$

- wall batter angle from vertical: $\alpha := 0deg$

- horizontal seismic coeff.: $k_h := 0.368$

- vertical seismic coeff.: $k_v := 0$

- slope backfill active side: $\beta := 0.0deg$

- seismic inertia angle: $\theta := atan(k_h) = 20.2 \cdot deg$

- at-rest lateral soil coefficient: $K_o := 1 - \sin(\phi_{soil}) = 0.463$

- active lateral soil coefficient:

$$K_a := \frac{\cos(\phi_{soil} - \alpha)^2}{\cos(\alpha)^2 \cdot \cos(\delta + \alpha) \cdot \left[1 + \left(\frac{\sin(\delta + \phi_{soil}) \cdot \sin(\phi_{soil} - \beta)}{\cos(\delta + \alpha) \cdot \cos(\alpha - \beta)} \right)^{0.5} \right]^2} = 0.27$$

- passive lateral soil coefficient:

$$K_p := \frac{\cos(\phi_{soil} + \theta)^2}{\cos(\alpha)^2 \cdot \cos(\delta_p - \alpha) \cdot \left[1 - \left(\frac{\sin(\delta_p + \phi_{soil}) \cdot \sin(\phi_{soil} + \beta)}{\cos(\delta_p - \alpha) \cdot \cos(\beta - \alpha)} \right)^{0.5} \right]^2} = 3.84$$

- seismic active lateral soil coefficient:

$$K_{AE} := \frac{\cos(\phi_{soil} - \theta - \alpha)^2}{\cos(\theta) \cdot \cos(\alpha)^2 \cdot \cos(\delta_p + \alpha + \theta) \cdot \left(1 + \sqrt{\frac{\sin(\phi_{soil} + \delta_p) \cdot \sin(\phi_{soil} - \theta - \beta)}{\cos(\delta_p + \alpha + \theta) \cdot \cos(\beta - \alpha)}} \right)^2} = 0.62$$

- seismic passive lateral soil coefficient:

$$K_{PE} := \frac{\cos\left(\phi_{soil} + \alpha - \theta\right)^2}{\cos(\theta)\cdot\cos(\alpha)^2\cdot\cos\left(\delta_p - \alpha + \theta\right)\cdot\left[1 - \left(\frac{\sin\left(\delta + \phi_{soil}\right)\cdot\sin\left(\phi_{soil} + \beta - \theta\right)}{\cos(\beta - \alpha)\cdot\cos\left(\delta_p - \alpha + \theta\right)}\right)^{0.5}\right]^2} = 4.94$$

- Active Force: $F_A := \frac{1}{2}\cdot(119pcf - 62.4pcf)\cdot K_a\cdot h_{soil}^{\ 2}\cdot l_{bx} = 7.76\,kip$

- Active Moment: $M_A := F_A\cdot\frac{h_{soil}}{3} = 32.32\,ft\cdot kip$

- Passive Force: $F_P := \frac{1}{2}\cdot(119pcf - 62.4pcf)\cdot K_p\cdot h_{soil}^{\ 2}\cdot l_{bx} = 110.51\,kip$

- Seismic Active Force: $F_{AE} := \frac{1}{2}\cdot 119pcf\cdot\left(K_{AE} - K_a\right)\cdot h_{soil}^{\ 2}\cdot l_{bx} = 21.14\,kip$

- Seismic Active Moment: $M_{AE} := F_{AE}\cdot 0.67\cdot h_{soil} = 177.06\,ft\cdot kip$

- Seismic Passive Force: $F_{PE} := \frac{1}{2}\cdot 119pcf\cdot\left(K_{PE} - K_p\right)\cdot h_{soil}^{\ 2}\cdot l_{bx} = 66.37\,kip$

- Equivalent Passive Force: $F_{Ptotal} := F_P + F_{PE} = 176.88\,kip$

- Total Passive Moment: $M_{Ptotal} := F_{Ptotal}\cdot\frac{h_{soil}}{3} = 737.00\,ft\cdot kip$

- Total Active Vertical: $F_{vA} := \left(F_A + F_{AE}\right)\cdot\sin(\delta) = 10.36\,kip$

- Total Active Vertical Moment: $M_{vA} := F_{vA}\cdot 6.5ft = 67.32\,ft\cdot kip$